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STATE-OF-THE-ART OF MARINE SOIL MECHANICS AND FOUNDATION ENGINEERING

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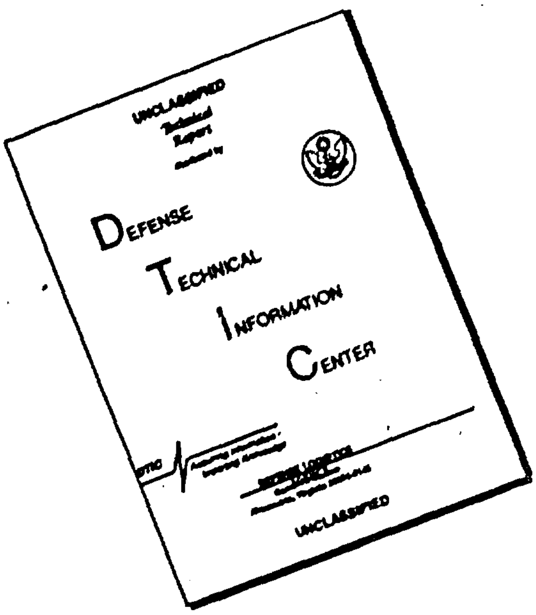

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13. ABSTRACT

This report presents the state-of-the-art of marine soil mechanics and foundation engineering. The study involved an extensive literature search and personal contacts with individuals in Government and industry involved in offshore work. The fields of subbottom exploration, laboratory and in situ testing, soil properties, and marine foundation engineering were examined to delineate existing capabilities and limitations. The viewpoint taken was that of a soils and foundation engineer attempting to plan, design, and construct an offshore foundation with the available knowledge and experience. The study was limited to water depths of 600 ft or less. Uncertainties exist in (a) questionable soil property inputs for design procedures due to sample disturbance and inadequate knowledge about sea floor characteristics, (b) performance expectations based on inadequate full-scale foundation performance data, and (c) construction technology to place the foundation as designed. Recommendations are made for research projects to reduce these uncertainties and to advance the state-of-the-art of marine soil mechanics and foundation engineering.

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FOREWORD

The study covered in this report forms a part of the Coastal Engineering Research Program of the U. S. Army Coastal Engineering Research Center; that program is a part of the Civil Works program of the U. S. Army Corps of Engineers.

This study was authorized by letter and IAO CERC-69-77, both dated 10 June 1969, to the Director, U. S. Army Engineer Waterways Experiment Station (WES), from the Director, U. S. Army Engineer Coastal Engineering Research Center (CERC), subject: "Funding Support for Marine Soil Mechanics Program," as a part of the Civil Works research program of CERC. Substantial support was also provided in Fiscal Years 1971 and 1972 under ES 547, "Sub-Aqueous Soil Mechanics," by the Office, Chief of Engineers.

The report was prepared largely by Mr. S. C. Ling under the direction of Mr. W. E. Strohm, Jr., Chief, Engineering Studies Section, with substantial revisions by Mr. J. R. Compton, Chief, Embankment and Foundation Branch, Soils and Pavements Laboratory, WES. The section entitled "Background" in Part I and the sections entitled "Dredging" and "Underwater Fills" in Part V consist largely of material extracted from a paper entitled "Control of Underwater Construction," by S. J. Johnson, J. R. Compton, and S. C. Ling, presented at the ASTM Symposium on Underwater Soil Sampling, Testing, and Construction Control, 27 June-2 July 1971. Guidance in the preparation of this report was furnished by Mr. S. J. Johnson, Special Assistant, Soils and Pavements Laboratory. Part II was reviewed by Mr. A. L. Mathews, Chief (retired), Inspection and Exploration Section, Soil and Rock Mechanics Branch, Soils and Pavements Laboratory. General direction was provided by Mr. A. A. Maxwell,

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Acting Chief (deceased), and Mr. J. P. Sale, Chief, Soils and Pavements Laboratory.

Directors of WES during the conduct of this study and preparation of this report were COL Levi A. Brown, CE, and COL Ernest D. Peixotto, CE. Technical Director was Mr. F. R. Brown. Directors of CERC were LTC Myron D. Snoke, CE, LTC Edward M. Willis, CE, and LTC Don S. McCoy, CE. Technical Directors were Messrs. J. M. Caldwell and T. Saville, Jr.

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CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report can be converted to metric or nautical units as follows:

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
miles (U. S. statute)	1.609344	kilometers
square miles (U. S. statute)	2.589988	square kilometers
square feet	0.092903	square meters
cubic feet	0.0283168	cubic meters
cubic yards	0.764555	cubic meters
pounds	0.45359237	kilograms
tons (2000 lb)	907.185	kilograms
pounds (force) per square inch	0.6894757	newtons per square centimeter
pounds (force) per square foot	47.8803	newtons per square meter
pounds per cubic foot	16.0185	kilograms per cubic meter
inches per second	2.54	centimeters per second
feet per second	0.3048	meters per second
kips (force)	4,448.222	newtons
knots	1.852	kilometers per hour

SUMMARY

This report presents the state-of-the-art of marine soil mechanics and foundation engineering. The study involved an extensive literature search and personal contacts with individuals in Government and industry involved in offshore work. The fields of subbottom exploration, laboratory and in situ testing, soil properties, and marine foundation engineering were examined to delineate existing capabilities and limitations. The viewpoint taken was that of a soils and foundation engineer attempting to plan, design, and construct an offshore foundation with the available knowledge and experience. The study was limited to water depths of 600 ft or less. Uncertainties exist in (a) questionable soil property inputs for design procedures due to sample disturbance and inadequate knowledge about sea floor characteristics, (b) performance expectations based on inadequate full-scale foundation performance data, and (c) construction technology to place the foundation as designed. Recommendations are made for research projects to reduce these uncertainties and to advance the state-of-the-art of marine soil mechanics and foundation engineering.

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STATE-OF-THE-ART OF MARINE SOIL MECHANICS AND FOUNDATION ENGINEERING

PART I: INTRODUCTION

Purpose

1. The purpose of this study was to determine the state-of-the-art of marine soil mechanics and engineering and, based on the results of this study, to present recommendations for future research. The areas investigated included (a) subbottom exploration, (b) testing of marine soils, (c) marine soil properties, (d) marine foundation engineering, (e) dredging, and (f) underwater fills. This study made possible the identification of potential problem areas and recommendations for research needed to increase knowledge of the continental shelf and to advance the field of marine soil mechanics and foundation engineering.

Scope

2. The scope of this study involved (a) an extensive literature search and (b) personal contacts by telephone, letter, and/or visit (see Appendix A) with other individuals engaged in marine soil mechanics and engineering. Consideration was limited to water depths less than 600 ft* (100 fathoms), which approximates the internationally recognized legal depth of the continental shelf and encompasses most underwater areas of economic interest. This report covers subsoil exploration, in situ and laboratory testing, structure foundations, dredging, and fills.

Background

3. Underwater construction engineering is rapidly increasing and

* A table of factors for converting British units of measurement to metric units is presented on page ix.

becoming highly diversified, as shown in table 1. As a consequence, there is increased awareness that capabilities of analyzing the special problems in underwater construction have lagged behind. Requirements for deepwater facilities have been vastly extended by the development of superships, having drafts of 80 to 100 ft, for transporting crude oil and bulk commodities. The sensational expansion of oil and gas production offshore is well known, and future years may well see increasing utilization of other offshore natural resources. Shallow-water construction in water depths generally less than 100 ft is becoming increasingly important as complete land utilization of present coastal areas becomes a reality, and land must be made available for industrial or urban use or for airfields. A generalized summary of experience in underwater construction of various types is given in table 2.

4. The increasing interest in soil mechanics aspects of marine construction is demonstrated by the recent appearance of papers on the subject (for example, reference 1) and papers presented at specialty conferences such as the Conference on Marine Geotechnique at the University of Illinois, Urbana, Illinois, 1-4 May 1966;[2] the two ASCE conferences on Civil Engineering in the Oceans in September 1967[3] and December 1969;[4] the ASTM Specialty Conference on Underwater Sampling, Testing, and Construction Control in June 1971;[5] and yearly Offshore Technology Conferences at Houston, Texas.

PART II: SUBBOTTOM EXPLORATION

5. Subbottom exploration may involve sampling and testing of the materials below the water-soil interface and it may also include geophysical techniques of measuring and recording the response of the subbottom deposits to various energy sources. In this section, the major emphasis is on sampling equipment and methods, with a brief discussion of geophysical techniques. Testing methods and procedures are discussed in Part III.

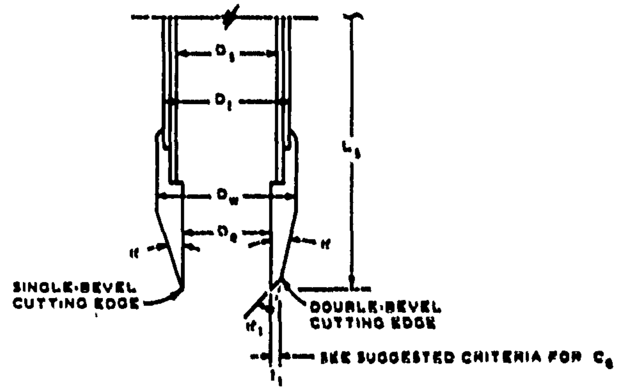
Sampling

6. Sampling is the exploration technique by which physical samples of the ocean bottom materials are obtained for examination or testing by engineers and scientists. A thorough discussion of subsurface exploration and sampling is found in Hvorslev's treatise on the subject;[6] Hvorslev divided subsurface exploration into four phases: (a) fact finding and geological survey, (b) reconnaissance or general exploration, (c) detailed exploration--small undisturbed samples, and (d) special exploration--large undisturbed samples. The quality required of the soil sample, sampling equipment, and sampling operation increases in order from phase a to phase d. Hvorslev also listed five disturbances to which soil samples are subjected: (a) change in stress conditions, (b) change in water content and void ratio, (c) disturbance of the soil structure, (d) chemical changes, and (e) mixing or segregation of soil constituents. The quality of the soil sample will depend on how effectively the design of the sampler and the sampling procedure minimize these disturbances. Realizing that disturbance (a) cannot be totally avoided during the sampling operation and preparation of laboratory test specimens, Hvorslev considers a sample "suitable for all laboratory tests and for practical purposes considered undisturbed" if none of disturbances (b) through (e) above occurred.

7. In the process of advancing a borehole, the soil at the bottom of the hole will tend to expand upward because of the removal of

overlying material. Inside wall skin friction along the sampling tube or liner will impede the movement of soil along the outer edge of the sample. The combined effect of the penetrating force and inside skin friction, which increases as the sample length increases, eventually attains a value which is greater than the bearing capacity of the soil below the sampler. After this point is reached, the soil below the sampler will be deflected downward, stretched, and reduced in thickness before it enters the sampler. Eventually, the inside skin friction will become so great that it will prevent further entry of soil into the sampler. A cone or wedge is then formed ahead of the sampler, and further penetration will not result in additional sample length. The heaving, impeding, and deflecting actions all contribute to disturbances (a), (b), (c), and (e)

8. The use of drilling mud in the borehole reduces the heaving tendency of the soil below the bottom of a borehole. Hvorslev established design requirements for long samplers (for use in other than coarse-grained dense or hard soils) to reduce the skin friction and other factors that contribute to sample disturbance. These and more recently suggested criteria, shown in fig. 1, were intended to achieve sampler characteristics that would prevent significant disturbance of the soil sample. The terms in fig. 1 were taken from references 6, 7, and 8. Compliance with these criteria does not guarantee an undisturbed soil sample but will reduce disturbance of the sample. A positive inside clearance ratio, C_i, within the limits shown, allows some reduction of inside skin friction created by lateral expansion of the soil as it enters the sampling tube and yet provides sufficient inside skin friction to prevent the soil sample from slumping inside the tube and possibly falling out of the tube during the sampler's return to the surface. The outside clearance ratio, C_o, within the limits shown, allows sufficient increase in wall thickness of the cutter head over the wall thickness of the sampling tube to reduce the outside skin friction and permit deeper penetration and yet does not permit the thickness to be so great as to impede the cutting ability of the cutting head. The limits on the area ratio, C_a, control the amount of excess soil



DEFINITIONS (FROM REFS. 6 AND 7)

L_s = SAFE SAMPLING LENGTH
D_s = INSIDE DIAMETER OF BARREL
D_t = OUTSIDE DIAMETER OF BARREL
D_w = OUTSIDE DIAMETER OF CUTTER
D_e = INSIDE DIAMETER OF CUTTER
α = TAPER OF CUTTING EDGE
α_1 = TAPER USED CLOSE TO EDGE OF A DOUBLE-BEVELED CUTTING EDGE
t_1 = MAXIMUM THICKNESS TO WHICH α_1 MAY BE USED

C_i = INSIDE CLEARANCE RATIO = $\frac{D_s - D_e}{D_e} \times 100$ (CONTROLS INSIDE FRICTION)

C_o = OUTSIDE CLEARANCE RATIO = $\frac{D_w - D_t}{D_t} \times 100$ (CONTROLS OUTSIDE FRICTION)

C_a = VOLUME, AREA, OR KERF RATIO = $\frac{D_w^2 - D_e^2}{D_e^2} \times 100 \left(\frac{\text{VOL. OF DISPLACED SOIL}}{\text{VOL. OF SAMPLE}}\right)$

C_s* = RATIO OF SAFE SAMPLING LENGTH TO INSIDE DIAMETER OF BARREL = $\frac{L_s}{D_s}$

* SYMBOL USED IN REF. 7.

CHARACTERISTIC	SUGGESTED CRITERIA	SOURCE
C_i	0 TO 0.5% FOR VERY SHORT SAMPLERS; 0.75 TO 1.5% FOR LONG SAMPLERS. SMALLER C_i MAY SUFFICE IF SAMPLER HAS SLIDING STEEL FOILS.	HVORSLEV[6]
C_o	0% FOR COHESIONLESS SOILS; ≤2 TO 3% FOR COHESIVE SOILS UNLESS α IS VERY SMALL.	HVORSLEV[6]
C_a	≤10%; MAY BE >10% WITH A PISTON SAMPLER OR SHARP CUTTING EDGE. SUGGESTED RELATIONS BETWEEN C_a AND α FOR D_s = 3" ARE AS FOLLOWS: C_a, % / α, DEG / C_a, % / α, DEG 5 / 15 / 40 / 5 10 / 12 / 80 / 4 20 / 9 α_1 MAY BE 60° WITH t_1 UP TO ABOUT 0.3 MM, OR, FOR COARSER GRAINED SOILS THAN CLAYS, WITH t_1 UP TO ABOUT THE D_{10} GRAIN SIZE.	HVORSLEV[6] IGESS[8]
α	≤10° EXCEPT AT CUTTING EDGE WHERE α_1 = 20° OR 30° TO AVOID AN EASILY DAMAGED EDGE. SEE C_a ABOVE.	HVORSLEV[6] IGESS[8]
C_s	≥5 TO 10 FOR DENSE TO LOOSE COHESIONLESS SOILS, AND ≤10 TO 20 FOR STIFF TO VERY SOFT COHESIVE SOILS FOR PROPERLY DESIGNED AND OPERATED DRIVE SAMPLER WITH D_s OF 2 TO 3 IN. SMALLER C_s FOR LARGER D_s AND VICE-VERSA. GREATER C_s FOR HIGH PENETRATION SPEED OR C_i.	HVORSLEV[6]
	SUGGESTED RELATIONS BETWEEN SOIL TYPE AND OPTIMUM C_s (S_t IS THE SENSITIVITY OF A SOIL) ARE AS FOLLOWS: TYPE OF SOIL / C_s CLAY (S_t < 5) / 20 CLAY (5 ≤ S_t ≤ 30) / 12 CLAY (S_t > 30) / 10 LOOSE COHESIONLESS SOIL / 12 MEDIUM LOOSE COHESIONLESS SOIL / 6	IGESS[8]
D_s	< 50 MM (≈ 2") NOT DESIRABLE FOR UNDISTURBED SOIL SAMPLING. MANY EXPERTS REQUIRE D_s ≥ 75 MM (≈ 3") FOR SAMPLES OBTAINED BY PUSHING.	IGESS[8]

Fig. 1. Criteria for tube samplers

entering the sampler due to the displacing action of a thick cutting head wall. The maximum allowable value of the ratio of safe length to inside diameter, C_s , controls the buildup of inside skin friction and essentially stops the sampling before a downward deflection of the soil below the sampler occurs. Ito and Tanaka[9] found in sampling a clayey silt (Kanto loam) that a cutting edge taper, α , of 7 deg resulted in the smallest amount of sample disturbance, but an α of 15 deg resulted in the least amount of force necessary to drive the sampler. They had varied α from 5 to 90 deg in their investigation. The cutting edge taper of 7 deg is well within Hvorslev's criterion that α should be less than 10 deg. An angle of taper, α , less than 5 deg has been suggested by others for sampling cohesive fine-grained soils.[7,10-12] Richards and Parker[7] prefer the single-beveled edge over the double-beveled edge for sampling cohesive fine-grained soils.

9. The flow characteristics of the check valve above the sampling tube must be sufficient to prevent overpressures from developing above the sample, or at the ear of the piston if a piston sampler is used, since overpressures would impede the movement of soil into the sampling tube. The fixed-piston sampler is the most desirable of currently used undisturbed sampling devices. Hvorslev[13] has suggested, though, that a downward movement of the piston equal to the deflection of the soil strata below the sampler would be better; this, however, would require further research to determine the amount of the movement and the means by which piston movement could be controlled.

10. Sampling equipment and procedures have been improved since 1949, but the causes of disturbances and the design criteria for tube-type samplers used to obtain high-quality undisturbed samples, as established by Hvorslev, are still valid and are also applicable to marine subbottom exploration and sampling. Richards and Parker[7,11] discuss in more detail the application of Hvorslev's work to marine-type tube samplers.

11. Both Hopkins[14] and Onarati[15] have published criteria for marine bottom samplers used from surface vessels. These criteria are

listed below to present a frame of reference in considering the various samplers available:

- a. The sampler should have a minimum number of moving parts.
- b. Parts should be corrosion resistant.
- c. The sampler should be sturdy enough to endure repeated handling on deck and impacts on bottom.
- d. The bulk and weight of the sampler should be such that the sampler is not overly dangerous to handle on deck and should be within the lifting limits of shipboard cranes and winches.
- e. The sampler should properly orient itself prior to entry into the bottom.
- f. Sufficient weight or power should be provided to obtain desired penetration into the bottom.
- g. There should be little or no disturbance of the soil during the sampler's penetration and withdrawal.
- h. No sample loss should occur during retrieval of the sampler from the bottom to the water surface.
- i. The sample should be easily removed from the sampler.

12. Sampling is accomplished from a variety of transportation modes. Barges or other nonpowered platforms can be towed to the test site and anchored in position. Self-propelled vessels with cranes, winches, and A-frames capable of lowering and raising test equipment over the side or through center wells can be used. Submersibles with manipulating arms and viewing ports have been used.[16,17] Bottom crawlers are envisioned for future sea floor work.

13. Some geophysical survey work is usually done prior to a sampling operation in the ocean. In the simplest form, this would involve a bathymetric survey utilizing a precision depth recorder (PDR) or a similar recorder which can provide continuous acoustic reflection traces of the sea bottom. The signatures recorded can be of help in selecting the type of sample to be used. For example, a continuous flat trace could imply sediments, variations in strength of the trace could imply density variations, a rugged trace could imply rock outcrops with the possibility of sediments in the valley, etc. Where the presence of rock is suspected, it would be prudent to attempt sampling first with a rock

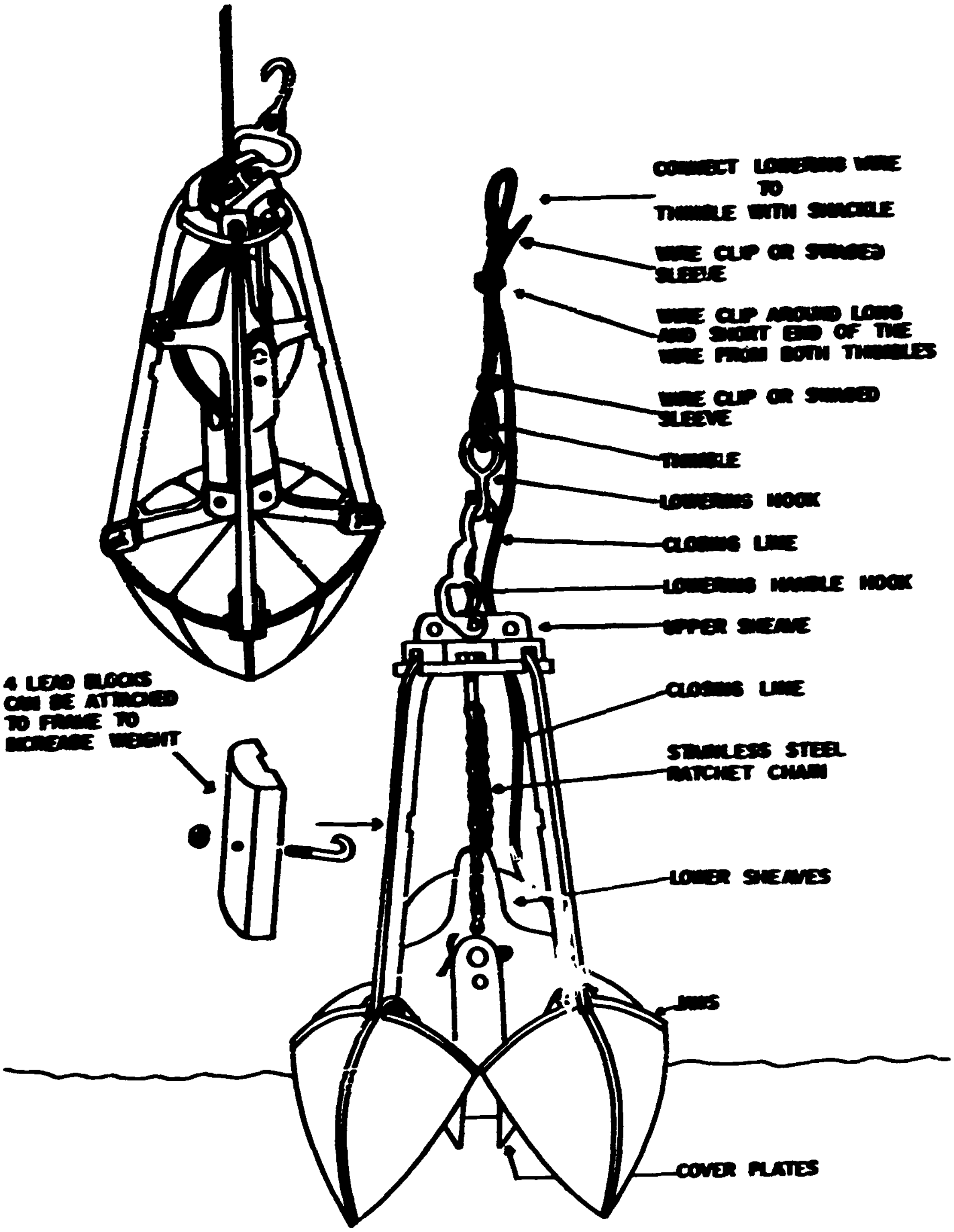
core barrel, or hard soil sampler. Where the presence of sediments is suspected, a normal soil sampler can be used. In a more sophisticated form, continuous reflection surveys may be performed, giving traces of the signatures not only of the water-bottom interface but also of those of the subbottom soils or rock. Evaluation of these signatures is often used in determining the type of subbottom sampling to be done.

14. Samplers can be grouped into three major categories: (a) surficial grab or scoop samplers, (b) single-entry drive samplers, and (c) repeated-entry drive samplers. Although some samplers could logically fall into more than one category, they are described under one category for convenience of discussion. The choice of which samplers to use depends on (a) the purpose of the investigation and the exploration phase, (b) expected bottom characteristics, (c) water depth, (d) available shipboard equipment, and (e) economics.

Surficial Grab or Scoop Samplers

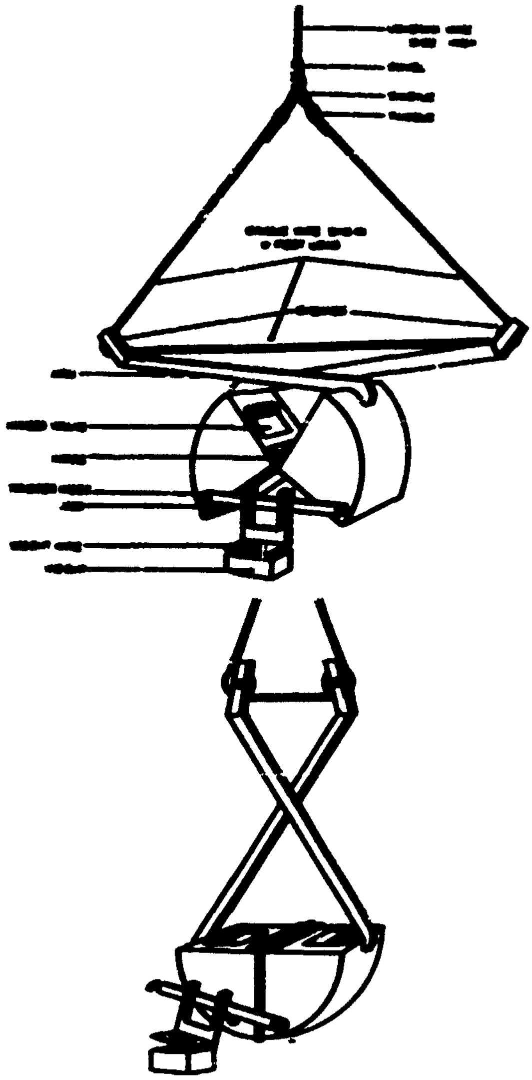
15. Oceanographers use small surficial grab samplers, which are operated from winches aboard surface vessels, to obtain disturbed soil samples down to generally less than 1 ft below the sea floor. Two types of oceanographic grab samplers are described in table 3 and shown in figs. 2 and 3. Sampling to greater depths has been accomplished by using standard construction equipment with large buckets when near the shoreline in water depths up to ≈100 ft. The grab samplers have a set of jaws that are open during the descent to and penetration in the sea floor, forced to close on the bottom and thus obtain a bite of the sea floor by the upward pull exerted on the cable during retrieval operations, and held closed by the tension on the cable during the ascent of the sampler to the sea surface. The samples obtained with these samplers are disturbed because the biting action of the jaws will distort the sample and because washing out of fines is generally unavoidable during the ascent of the sampler through the water mass. These samplers are useful in the fact-finding and geological survey phase of exploration.

16. Scoop or dredge-type samplers are also described in table 3,



(Courtesy of U. S. Naval Oceanographic Office)

Fig. 2. Orange peel bucket sampler for obtaining 0.15-cu-ft sample (from reference 21)



(Courtesy of U. S. Naval Oceanographic Office)

Fig. 3. Van Veen sampler with modified trigger (from reference 21)

and two common types are shown in fig. 4. The sampler is constructed of cylindrical or rectangular metal frames with the forward end open and the aft end either covered with a grating or having a steel mesh bag attached. These samplers are dragged along the bottom and are generally used by oceanographers for collecting rock samples. They have also been used to sample hard cohesive soils. The forward edges of these samplers are sharpened for scraping and chipping off rock fragments as they are dragged along the bottom. The grating or open mesh bag on the back end of the dredge permits the passage of water yet retains the rock sample. The scoop sampler, like the grab sampler, is useful in the fact-finding and geological survey phase of exploration.

17. Surficial grab or scoop samplers are of minimal value in soil mechanics and foundation engineering. However, in the process of examining available past records for a specific site, the engineer may find that soil classification and geologic conclusions were based on tests of soil samples obtained with these samplers. This information must be used cautiously by the engineer because the soil samples are disturbed ones and were removed from only the upper foot of the sea floor. Hopkins[14] and Holmes[19] present more thorough descriptions of surficial grab or scoop samplers.

(From Submarine Geology, 2nd Ed. by Francis P. Shepard (Harper & Row, 1963, page 251))

a. Pipe dredge b. Frame dredge

Fig. 4. Pipe dredge and frame dredge samplers (from reference 18)

Single-Entry Drive Samplers

18. Single-entry drive samplers are the tube-type subbottom samplers which either use gravity to achieve penetration into the sea floor or are propelled into the sea floor by other means. Of the common oceanographic bottom sampling devices, these are potentially the most useful to the soils engineer. For this reason, they will be discussed in more detail than the previous samplers. They may be used for the reconnaissance phase of exploration. Also, in cases in which foundation loads are small and the influence of applied loads is not too deep, they may be used in

the detailed phase of exploration. These samplers are generally attached to cables and are lowered and raised by winches aboard surface vessels. Due to the drift of the surface vessel and the continuous random motion of the water mass, it is practically impossible for these samplers to be returned to their previous locations for reentry into the same hole for continuous or incremental sampling. The gravity single-entry drive sampler will be discussed first, then the propelled single-entry drive sampler. Both work best in soils that are not too coarse-grained, not too dense or hard, and not too loose or soft. If the soil is too coarse-grained, dense, or hard, it is difficult to achieve bottom penetration; if the soil is too loose or soft, it is difficult to retain the sample.

Gravity Single-Entry Drive Samplers

19. Gravity single-entry drive samplers consist of interchangeable sampling tubes and an upper assembly which provides support both for the drive weights and the sampling tubes. By common usage, these samplers are separated into two classifications: (a) gravity corers* and (b) piston corers.* Both achieve penetration into the bottom by gravity, but the piston corer differs from the hollow-tube gravity corer in that it utilizes a piston mechanism to create a partial vacuum in the tube above the entrapped soil as the tube slips past the piston and into the bottom. The vacuum is helpful in holding soil in the tube during raising operations and in decreasing sampling disturbance. The principle involved is the same one which is utilized in onshore piston samplers.[6] Except for the piston mechanism, the two corers are, for all practical purposes, the same. In fact, the piston corer can be, and often is, used as a gravity corer by removing the piston.

20. The kinetic energy available for bottom penetration by a gravity single-entry drive sampler is a function of the velocity of the sampler, i.e., $KE = 1/2\ mv^2$, where KE is kinetic energy, m is mass,

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* Oceanographers commonly refer to drive samplers and rotary core barrel samplers as "corers," and this terminology is often used in this report.

and v is velocity. In early sampling in the ocean, the velocity attainable by a corer was restricted by the winch speed. The practice then was to lower the corer at the maximum safe speed at which the cable could be let out. Velocities up to 20 ft/sec were obtainable by letting the winch run on the brake for the last 300 ft of descent.[22] In general, though, velocities were much less than this, with the result that kinetic energy supplied only a small part of the total energy available for penetration.

21. In 1940, Hvorslev and Stetson conceived the first free-fall release mechanism to increase the velocity of a corer using a pilot weight which would trigger the corer assembly. The corer assembly was suspended above the pilot weight and permitted the corer to free-fall from a predetermined height into the bottom.[22] The pilot weight could be a dead weight or a second short penetration corer used to obtain a shallower sample of the surface soils. Fig. 5 shows a release mechanism.

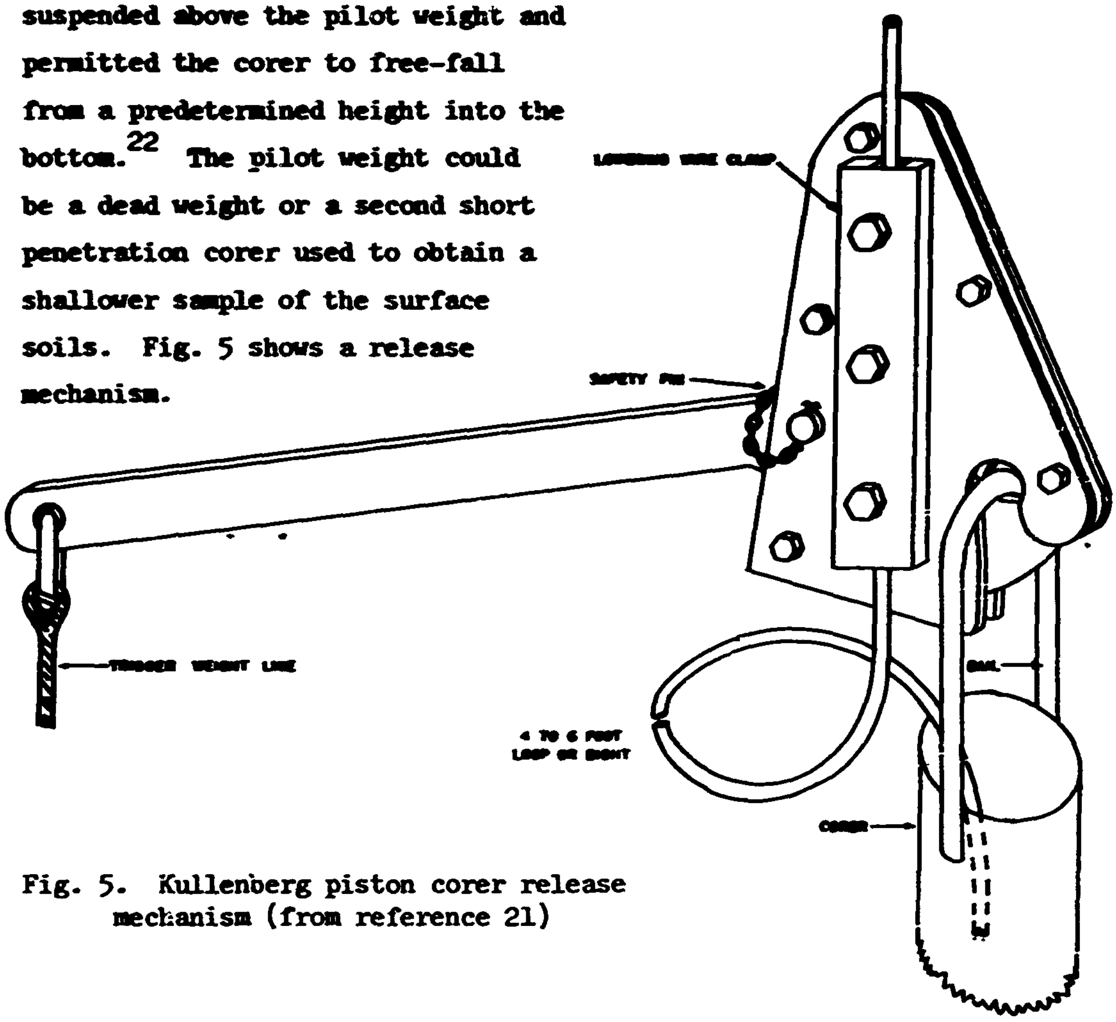
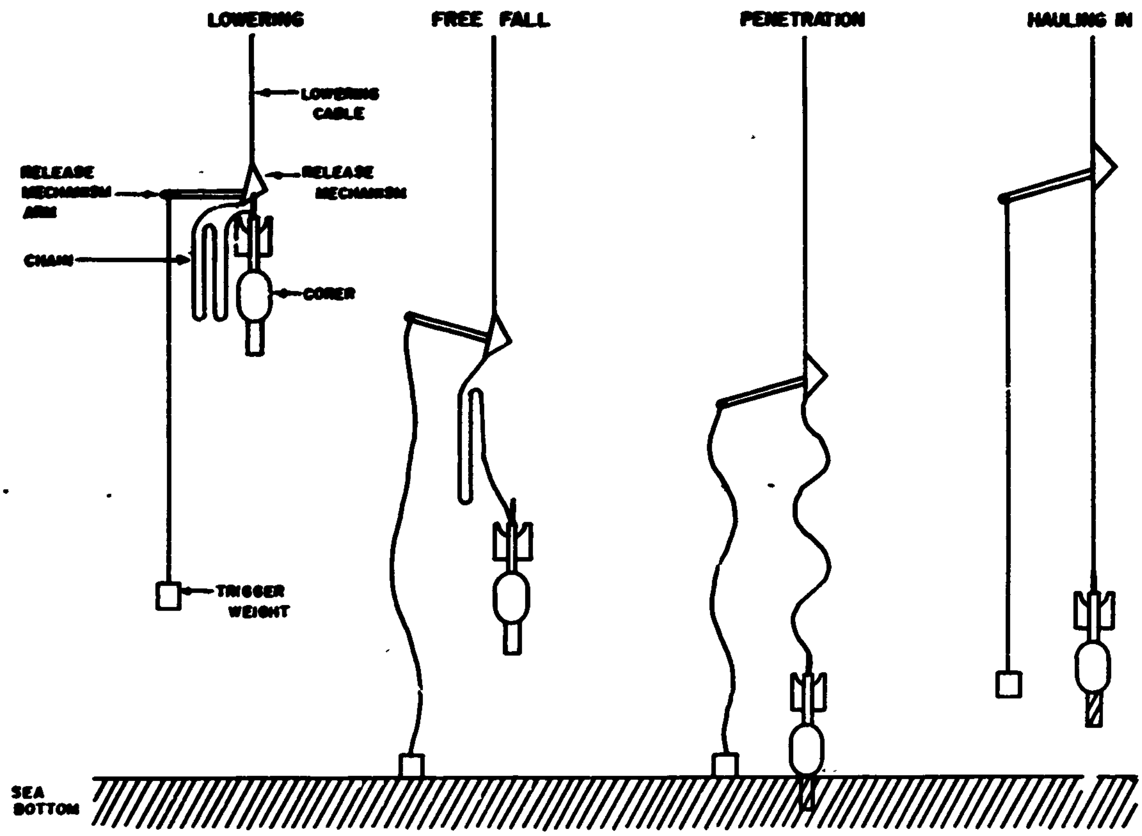


Fig. 5. Kullenberg piston corer release mechanism (from reference 21)

(Courtesy of U. S. Naval Oceanographic Office)

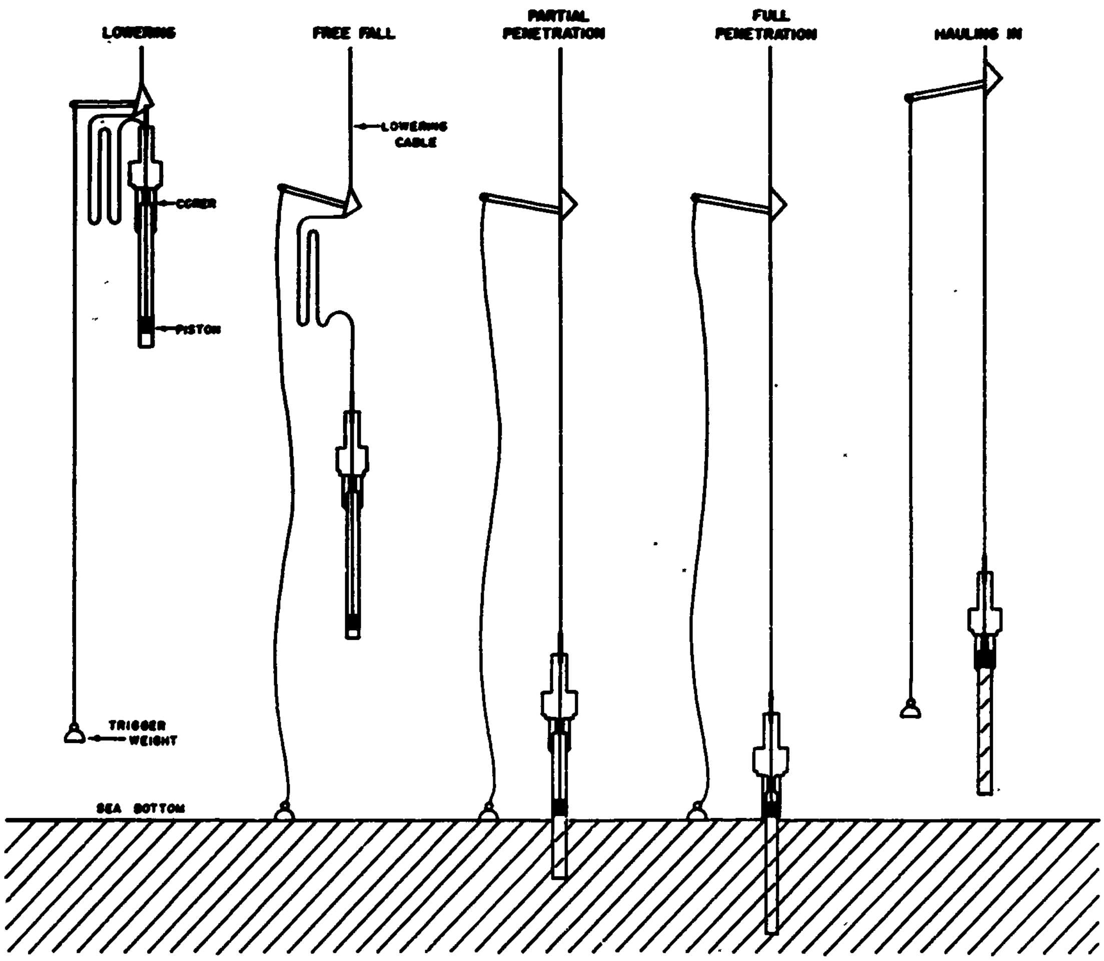
Figs. 6 and 7 depict the operation of (a) a gravity-type corer, and (b) a piston-type corer using release mechanisms.

22. Terminal and striking velocities and other pertinent characteristics of corers free-falling through water are discussed in references 22-25. The penetration of a free-fall sampler into the sea floor depends on the energy available during the sampling operation, the geometry of the sampler, and the variation in soil resistance with depth. The energy is comprised of kinetic energy, which is proportional to the mass and the square of the velocity of entry of the sampler into the soil, and of potential energy, which is proportional to the mass, the free-fall distance, and the depth of penetration. The geometry of the sampler will influence the ease with which the sampler penetrates the



(Courtesy of U. S. Naval Oceanographic Office)

Fig. 6. Principle of operation of gravity-type corer
(from reference 21)



(Courtesy of U. S. Naval Occanographic Office)

Fig. 7. Principle of operation of piston-type corer (from reference 21)

sea floor and the amount of soil resistance mobilized in resisting the sampler's entry. The depth of penetration is inversely proportional to the increase in soil resistance with depth. In the corers to be discussed, the individual designers have varied the corer mass, impact velocity, and corer geometry to improve penetration. However, penetration alone is not enough because the quality of the sample obtained is often more important. The penetration of corers into the sea floor has been analyzed by Schmid[25] and Korites.[26,27]

23. Sampling tubes in use are made of metal or plastic with

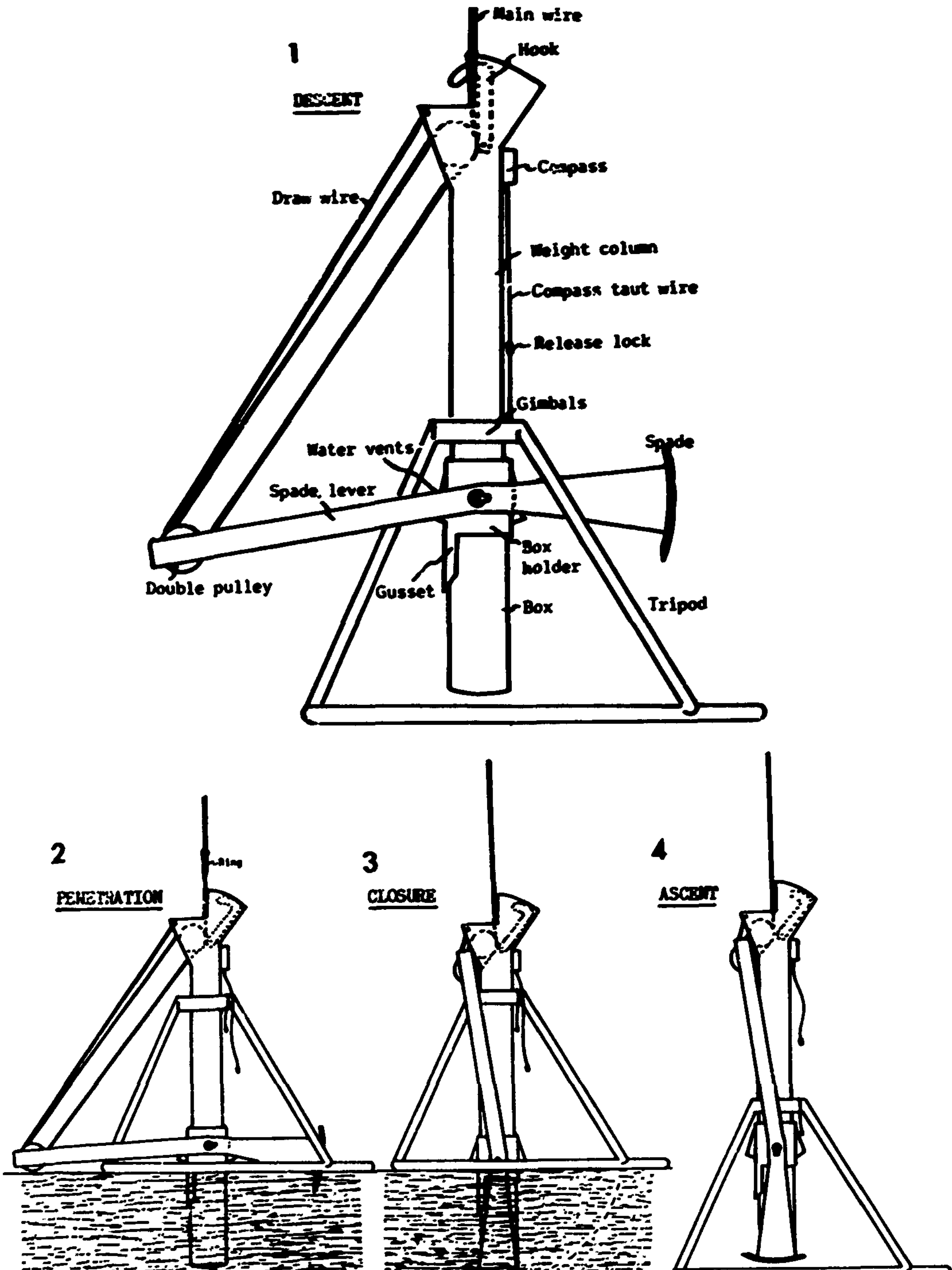
(Courtesy of American Society of Civil Engineers)

Fig. 8. Richards single, cylindrical, plastic-tube barrel corer (from reference 7)

circular, square, or rectangular cross sections. Figs. 8 and 9 show a cylindrical, plastic-tube corer and a bottom-rest, rectangular, metallic-tube corer, respectively. Plastic liners generally used with the metal barrels are commonly made of cellulose acetate butyrate (CAB). The assembly of the cylindrical metal-tube Kullenberg piston corer with a plastic liner is shown in fig. 10. Since a CAB liner has a relatively poor degree of impermeability,[28] a sample may lose water if it is stored in its plastic liner for a significant length of time prior to testing. Other plastics which apparently are less permeable are polycarbonate, polyethylene, and polyvinyl chloride (PVC). PVC pipe has been used successfully as the barrel of some corers.[21,29,30] Delrin plastic has been used successfully by Richards.[7]

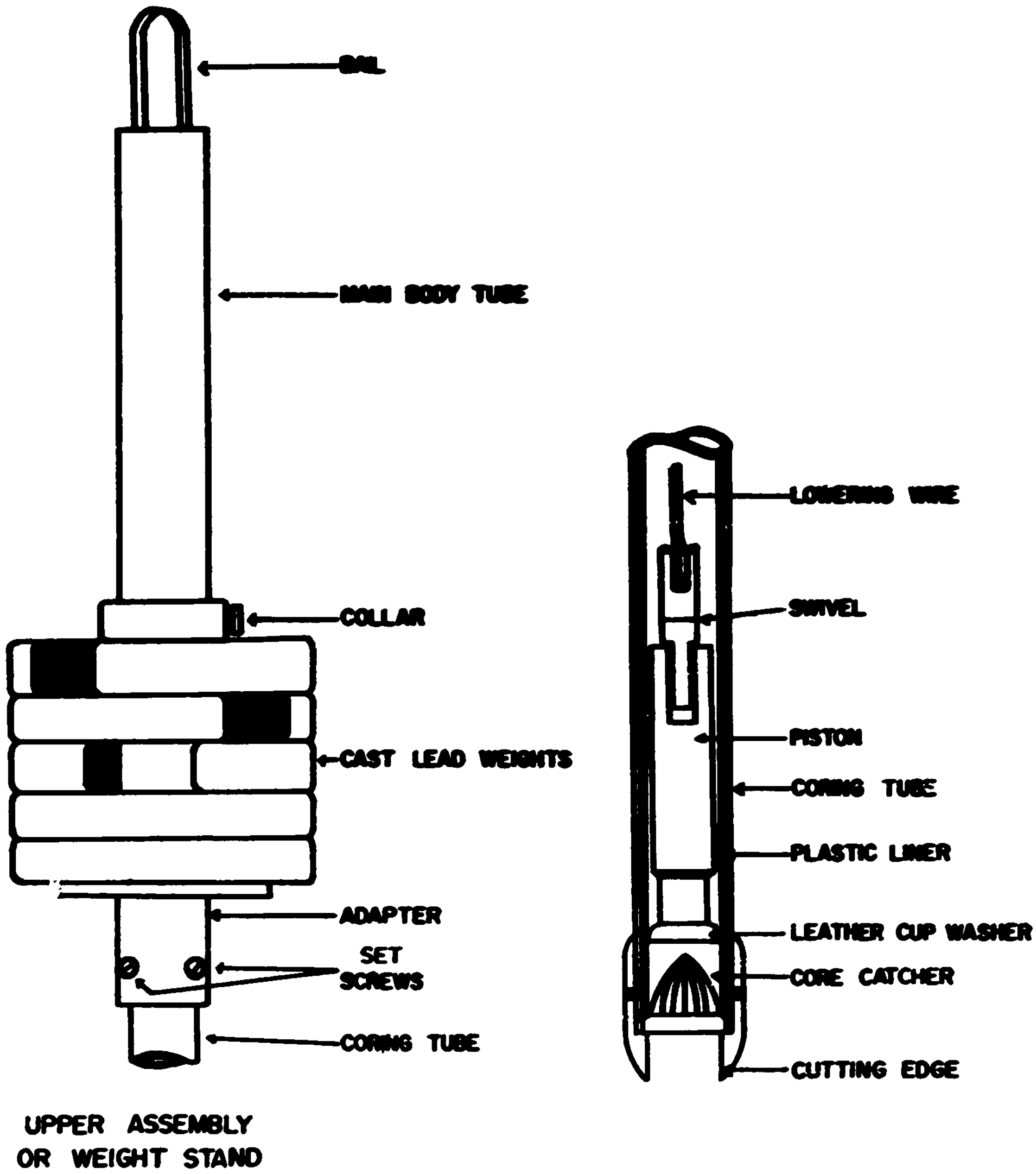
24. Prior to sampling, the depth of water should be known, and general knowledge of the bottom composition should be obtained. The water depth establishes the length of cable needed to lower the corer to the bottom, thus eliminating the possibility of excess cable being let out with subsequent kinking. Preliminary information on bottom composition can be obtained from past experience in the area, strength of the bottom reflection trace on the depth recorder, the traces on a subbottom profiler, or from surface grab samples. These observations will minimize the chances of losing or cracking a corer assembly due to attempts to sample a bottom which is too hard.

25. Richards,[11] McManus,[30] Rosfelder and Marshall,[12] and Inderbitzen[31] present information comparing current marine corers with Hvorslev's criteria[6] for sampling tubes. Table 4 lists characteristics of some common corers currently in use. This table shows that, with the exception of the USNEL spade corer (fig. 9), none of the corers listed meets all of Hvorslev's criteria.



(Courtesy of The University of Illinois Press)

Fig. 9. USNEL bottom-rest, rectangular, metallic-tube spade corer (from reference 12)



(Courtesy of U. S. Naval Oceanographic Office)

Fig. 10. Kullenberg piston corer assembly (from reference 21)

26. Table 5 gives ranges of corer diameters and barrel lengths for samplers currently in use.[7] Keeping in mind Hvorslev's safe-length criterion, the maximum safe lengths obtainable would be 20 ft for a 12-in. gravity corer and 10 ft for a 6-in. piston corer. For many

engineering purposes, exploration to depths much greater than 20 ft is required, and the single-entry drive sampler cannot, of course, meet these needs.

27. The piston corer will generally obtain a longer sample and will hold the sample in the corer better than the gravity corer. However, because of the dynamic penetration of single-entry samplers, samples obtained with a piston corer are often more disturbed than those taken with a gravity corer.[7,31,36,37] This is due primarily to the difficulty of maintaining a stationary piston during sampling operations and to the practice of raising the sampler with the winch cable directly linked to the piston. The maintenance of a stationary piston depends on the stability of the work platform and the connection between this platform and the piston. Ships and barges are moving platforms, and the roughness and the natural swell of the seas are transmitted via the winch, main cable, and piston cable to the piston and impart an oscillatory action to the piston relative to a fixed horizontal plane. This causes a pumping action behind the sample instead of a uniform increase of the expected partial vacuum. The pumping motion impedes sampling during a downward stroke and accelerates it during an upward stroke.

28. Raising the sampler with the piston cable can cause a strong pull on the piston, which usually results in the vertical upward displacement of the piston until it seats itself on the stop collar of the tube assembly. This often occurs in cases in which the piston is not seated firmly on the piston stop collar of the sampler tube assembly (due to insufficient penetration of the sampler to fill the full length of the sample tube). The force and displacement result in a distorting, sucking action on the soil already in the tube as well as a sucking in of excessively disturbed soil at the bottom of the sampler.

29. Many investigators[6,10,11,23,37,38] have examined the problems of piston coring. These investigators have studied the disturbance created by elastic relaxation waves traveling the main cable and the degree of disturbance in various portions of the core sample, compared piston core samples with gravity core samples, and considered the importance of sample disturbance relative to the sample's purpose.

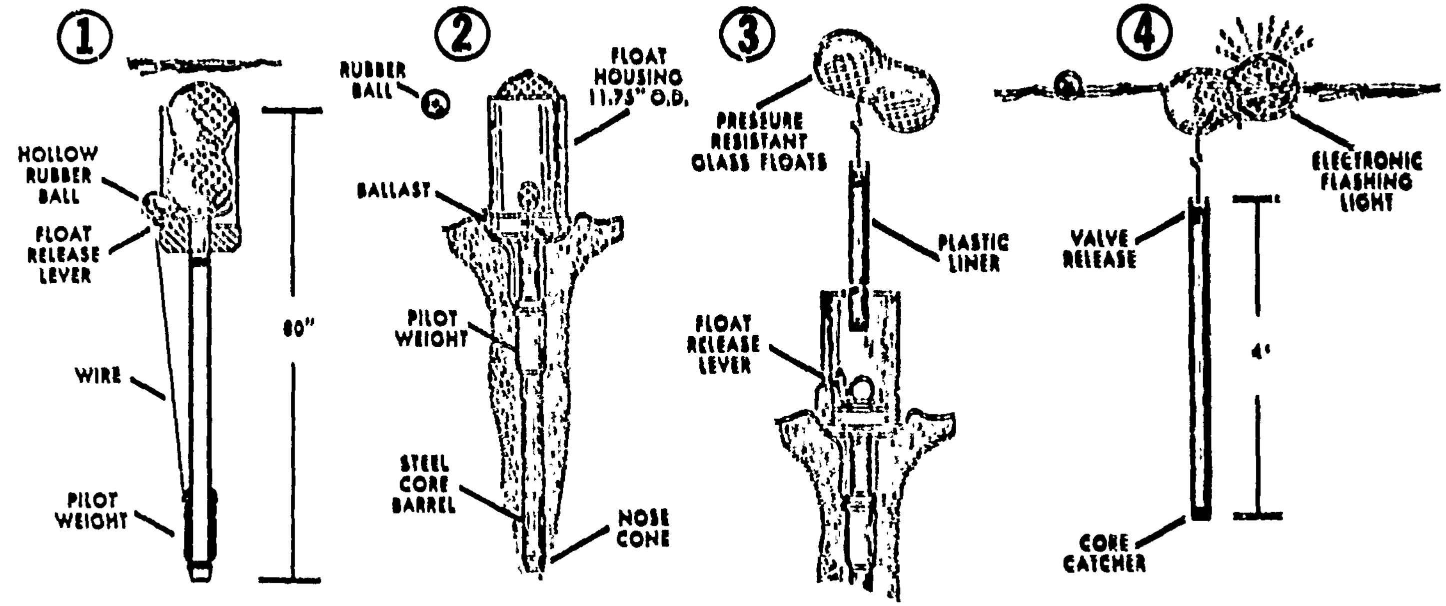
30. The advantage of piston coring over gravity coring is shown in table 5; much deeper penetrations are possible with the same diameter corer. The possibility of sample loss is also decreased because of the partial vacuum above the sample. For engineering purposes, it must be decided whether a long disturbed sample is more or less desirable than a short, less disturbed sample which gives no information on the material at greater depth.

31. Several innovations to improve gravity-type, single-entry drive samplers have been suggested and/or used. One of these is the free or unattached corer.[39-42] This is an untethered gravity corer which may be dropped into the water from a moving ship or an airplane. It includes an expendable outer barrel and weight assembly which remain on the ocean bottom after a sampling operation. The sample, inside a plastic liner, is floated to the sea surface by attached buoyant flasks which are triggered loose from the expendable portions of the corer after the corer penetrates the bottom. The corer is designed to take 4-ft cores. Fig. 11 shows the "Boomerang" corer[32] and its operation. Successful recoveries of these corers have been made 90 percent of the time, and only a few of these did not contain soil samples.[43] The characteristics of the "Boomerang" corer are listed in table 4.

32. Piston immobilizers have been devised to divorce the piston from the upward pull of the main cable during the raising of the sampler from the bottom and, at the same time, to restrain the piston from moving down due to its own weight and the weight of the soil sample.[15,44-46] Fig. 12 shows an immobilizer produced by Benthos, Inc. The upper half separates from the piston assembly and is pulled up to the upper end of the tube assembly so that the lift force is applied at the piston stop or collar assembly of the corer. The piston is immobilized from moving down by wedging brass balls against the inside of the core barrel.

33. Inside wall friction can be reduced by providing a mechanism that allows retention of the sampled soil in the sampler, but separates the sampled soil from the movement of the core barrel as the barrel moves past. The Swedish foil sampler[47] provides such an arrangement. As the sample moves down, steel foil strips unreel (at a rate equal to

HOW THE "BOOMERANG" CORER WORKS:



The Boomerang Corer is launched with its plastic liner and floatation spheres in place. The lead pilot weight holds down the glass float release lever. To prevent premature tripping, this lever is also held down by a hollow rubber ball.

At about 30' the hollow ball floats free of the corer which plummets downward reaching a terminal velocity of 450 meters per minute. As the corer plunges into the sediment, taking a sample, the pilot weight slides up the barrel, allowing the lever to release the glass floats.

The glass floats rise, pulling the core liner free of the core barrel. A very tight sealing valve at the top of the liner protects the core sample from being washed, and a core catcher keeps the sample from falling out of the bottom.

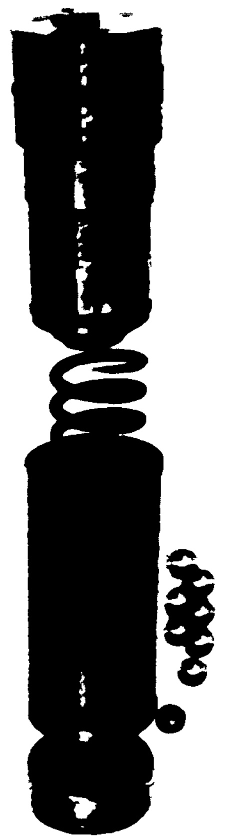
One float ball is covered with fluorescent orange paint for daytime sighting; the other float ball has a self-contained flash unit for night-time sighting which can be seen for several miles. Recovery can be made by ship or helicopter.

(Courtesy of Benthos, Inc., North Falmouth, Massachusetts)

Fig. 11. Boomerang corer (from reference 32)

the penetration rate) from storage reels near the back of the cutter head and enclose the soil entering the sampling tube. The soil and the foil encasing it remain stationary inside the tube as the tube slips past and down, thus eliminating friction between the soil sample and the inside wall of the sampling tube. This sampler is not a common marine sediment sampler, but has been used successfully on land and in shallow water.[48,49]

34. A somewhat similar arrangement (the Chmelik corer) using a flexible-type liner has been developed for marine use.[50,51] Fig. 13 shows the assembly of the cutting head, flexible liner storage, piston, and core barrel of the Chmelik corer and the operation of the liner. As the cutting head and core barrel slip past the piston, the flexible liner (synthetic casing used in the meat packing industry) unfolds and encases the soil sample as the core barrel slips by without touching the soil. Lubricants have also been used for reducing both inside and outside wall friction.[52]

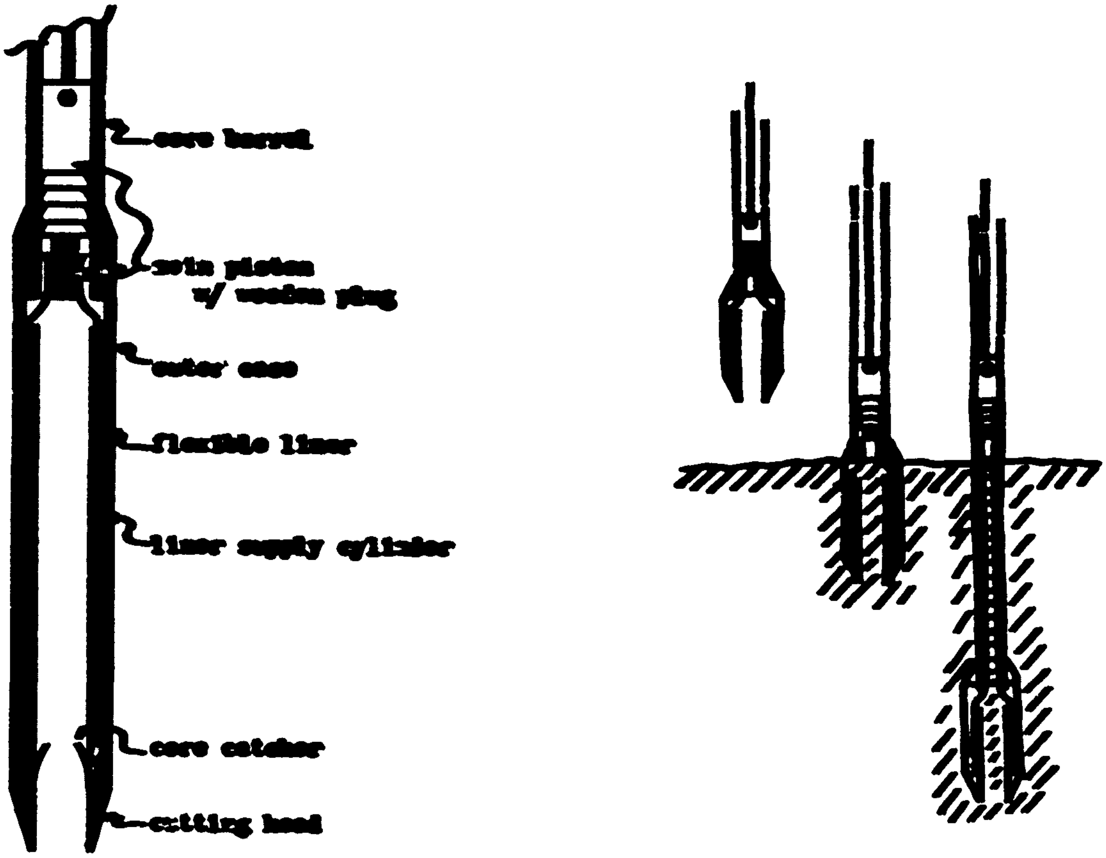


(Courtesy of Benthos, Inc., North Falmouth, Massachusetts)

Fig. 12. Piston immobilizer produced by Benthos, Inc.

35. From the previous discussion, it is apparent that gravity-type, single-entry drive samplers will take deeper samples of the bottom than surficial grab or scoop samplers. However, the gravity samplers are still generally insufficient for deep foundation engineering purposes. These corers generally tend to preserve the in situ stratigraphy, but they disturb the soil samples to various degrees.

36. The gravity corer obtains a less-disturbed sample than the piston corer because of the difficulty of maintaining a stationary piston and the practice of raising the sampler by the piston. Piston immobilizers have been developed to eliminate the effects of raising the corer by the piston. Other means suggested or used to increase the efficiency of gravity, single-entry drive samplers and/or to decrease sample disturbance



(Courtesy of Dept. of Oceanography, Texas A&M University)

Fig. 13. Chmelik flexible liner corer (from reference 51)

include (a) the use of a flexible liner system, (b) the use of lubricants on the walls of the corer, (c) the use of bottom-rest systems which are essentially free of sea surface influences during the sampling operations, and (d) the use of supplementary propulsion systems to aid in penetration.

<u>Propelled Single-Entry Drive Samplers</u>

37. Propelled single-entry drive samplers using rocket fuel, vibration, or other means for driving a core tube into the bottom have been suggested and/or developed to obtain greater penetration or to sample hard bottoms.

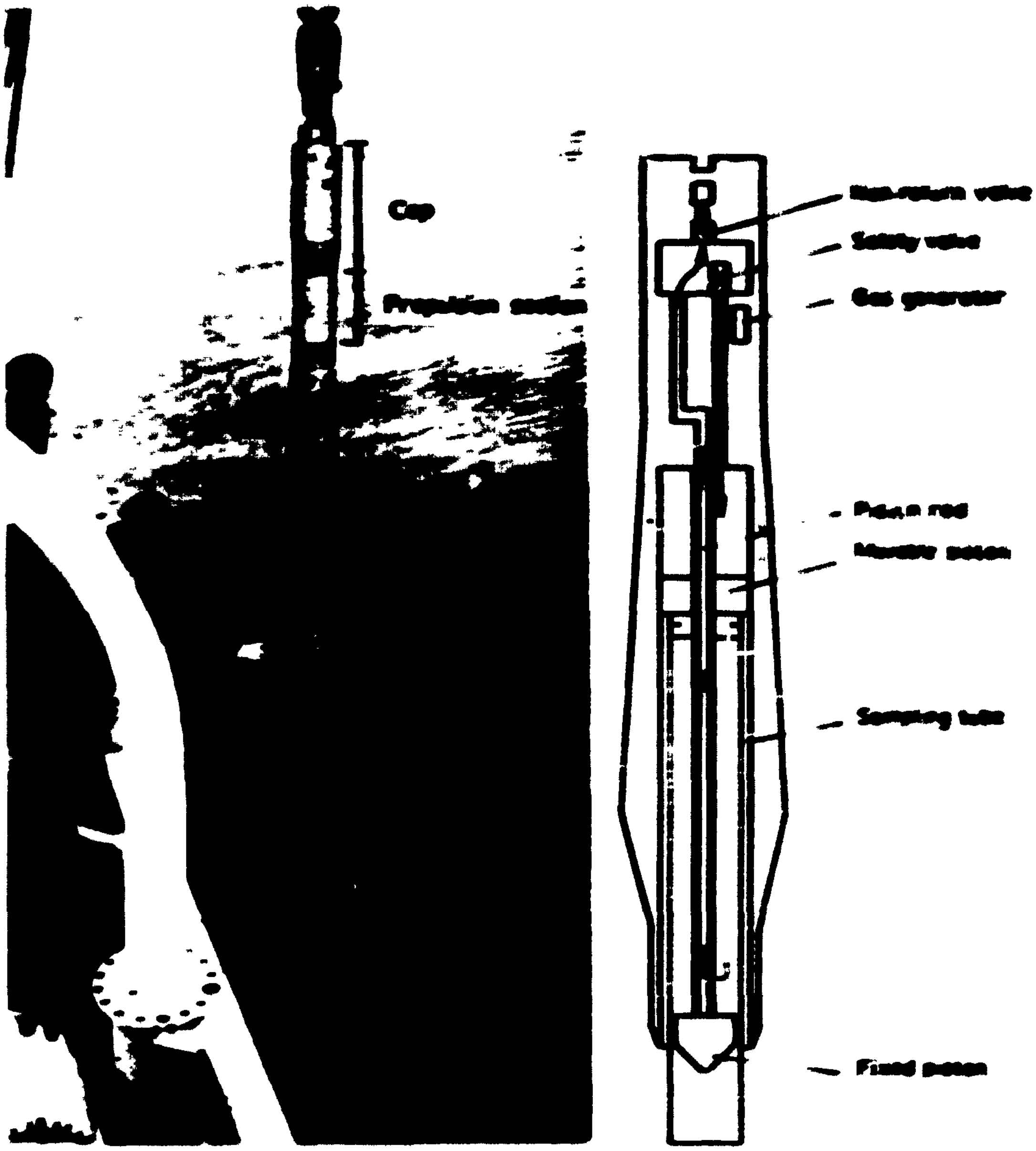
<u>Rocket-fueled samplers</u>

38. An example of the rocket-fueled sampler is the Norwegian

Geotechnical Institute (BGI) gas-operated sea floor sampler (fig. 14).[34] This sampler obtains penetration through its mass and the velocity it attains prior to entering the bottom. After the Torpedo (a nickname given the sampler because of its shape) comes to rest, a thin-walled sampling tube is driven past a fixed piston into the soil below at a rate of approximately 0.2 m/sec by forces created behind the movable piston (fig. 14b) through the controlled generation of gas from the timed ignition of a solid rocket propellant. Air between the fixed and movable pistons escapes through the vent holes into the hollow piston rod and out the nonreturn valve at the top of the Torpedo. When the moving piston passes the vent holes upon completion of its sampling stroke, the excess gas generated behind the piston is vented through the vent holes. The tube takes a sample 54 mm (2.1 in.) in diameter and 1.65 m (5.4 ft) long. This is essentially a fixed-piston operation due in part to the short period of time (less than 20 sec) needed to drive the sampler. The sampler is designed to operate in water depths as great as 350 m (1150 ft) and weighs 510 g (1120 lb) in air. The sampler is considered safer than an earlier explosive-type corer, which utilized rifle primer and gunpowders to shoot the sampling tube into the sea floor, because the Torpedo's main charge and igniter can be burned while held in the hand.[6,53] With the mass of the sampler being constant, the depth of penetration of the sampler is determined by its impact velocity, which is controlled by the winch speed or by the free-fall distance if the sampler is allowed to free-fall to the bottom. By varying this velocity on successive lowerings of the sampler, the depth at which actual sampling takes place can be predetermined. Though reentry into the same hole is not possible, incremental sampling in a small localized area is possible. Maximum depth of penetration of the sampling tube is 9.7 m (32 ft). It is predicted that if the free-fall release technique is used with this sampler, penetrations will reach a depth of 20 m (66 ft). The sampler's characteristics are listed in table 4.

Vibration samplers

39. Marine samplers using vibration to cause the sampling tube to



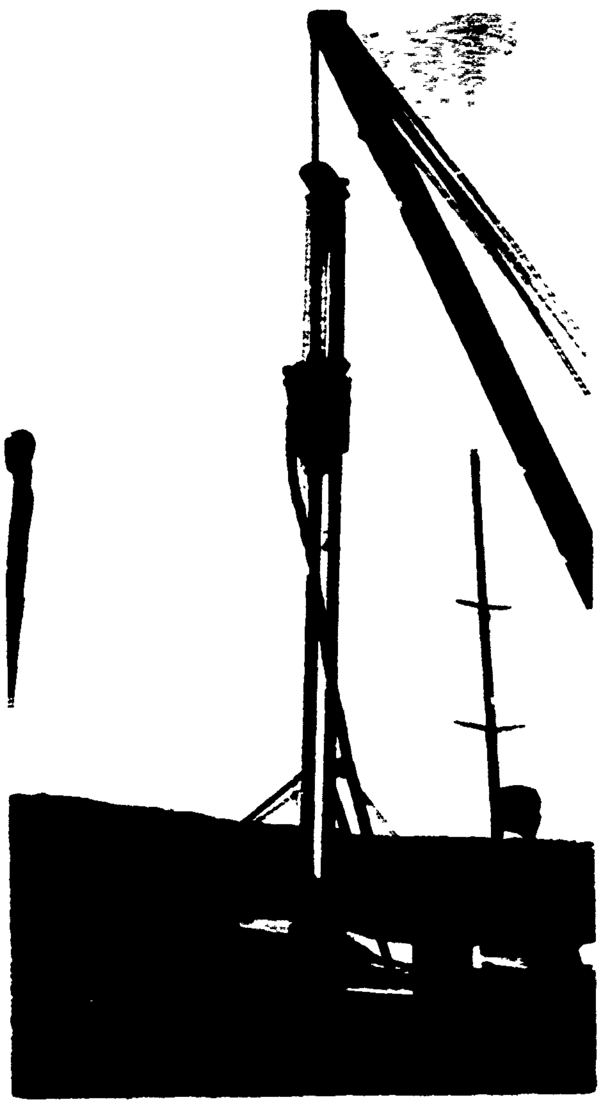
Courtesy of Norwegian Geotechnical Institute, Oslo, Norway

a. [illegible] of NGI gas-operated sampler hanging over the side of [illegible]. The sampling tube is in the extended position.

b. Diagrammatic drawing of the NGI gas-operated sampler showing principal [illegible] and the [illegible] of [illegible].

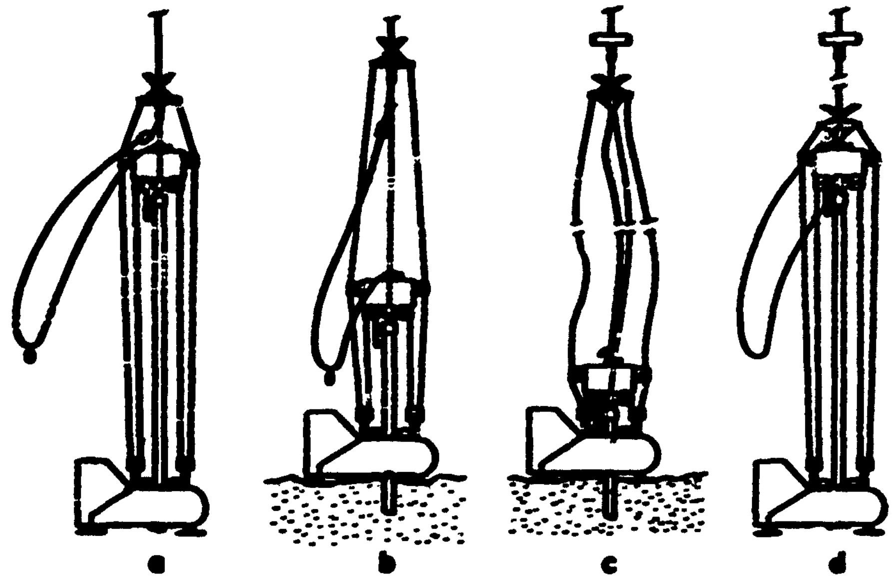
Fig. [illegible]. NGI gas-operated sampler (from reference [illegible]) penetrate the bottom have also been developed.[11,14,54,55] These have been especially useful in sampling noncohesive soils. The Russians, first to use vibratory sampling at sea, used an electric vibrator mounted on a bottom-rest sampling system composed of a bottom platform for stability and an upright framework along which the vibrator rides as it drives the core tube into the soil. The piston is immobilized by a cable attachment at the top of the frame. The system is lowered and raised by a cable attached to the frame, and electric power is supplied to the vibrator from the surface vessel.

40. Both Ocean Science and Engineering, Inc., and Alpine Geophysical Associates, Inc., manufacture vibratory corers. The Vibracore of Alpine operates in maximum depths of 200 ft of water and takes a 3-1/2-in.-diam sample up to 40 ft in length.[56,57] This is a bottom-rest system (fig. 15) composed of an H-beam tower that supports and guides the vibrator and the sampling tube and a four-legged bottom support framework. The air-powered mechanical vibrator drives the corer into the bottom. It takes about 120 sec to obtain a 20-ft core. The unit includes a meter which indicates the amount and rate



(Courtesy of The Society of American Military Engineers)

Fig. 15. Vibracore ready to be lowered to the bottom of the sea (from reference 56)



(a) Descending, (b) Coring, (c) Extracting, (d) Ascending

Fig. 16. Schematic diagram of USGS-WES sampler in operation

of penetration of the corer. The shipping weight for a 40-ft core model is 3000 lb. An umbilical cord delivers compressed air from the shipboard compressor to the vibrating unit and transmits electronic signals from the meter to the on-deck recorder. The core tube is withdrawn into the framework prior to raising the sampler. The U. S. Army Coastal Engineering Research Center has successfully used a Vibracore in its sand inventory work.[58,59]

41. The St. Louis District of the Corps of Engineers used a vibratory corer on the Mississippi River in 1966 (fig. 16). Built by the U. S. Army Engineer Waterways Experiment Station (WES) for the St. Louis District, it was patterned after the U. S. Geological Survey (USGS) model which had been used on the Columbia River.[60] The USGS version was designed to take a 6-ft-long sample; the WES model was designed to take a sample approximately 11 ft long, but only 5-ft lengths of samples were obtained by the St. Louis District due to the denseness and coarseness of the river bottom materials. This is a bottom-rest system using a fixed-piston corer. Through the unique arrangement of pulleys, the tension on

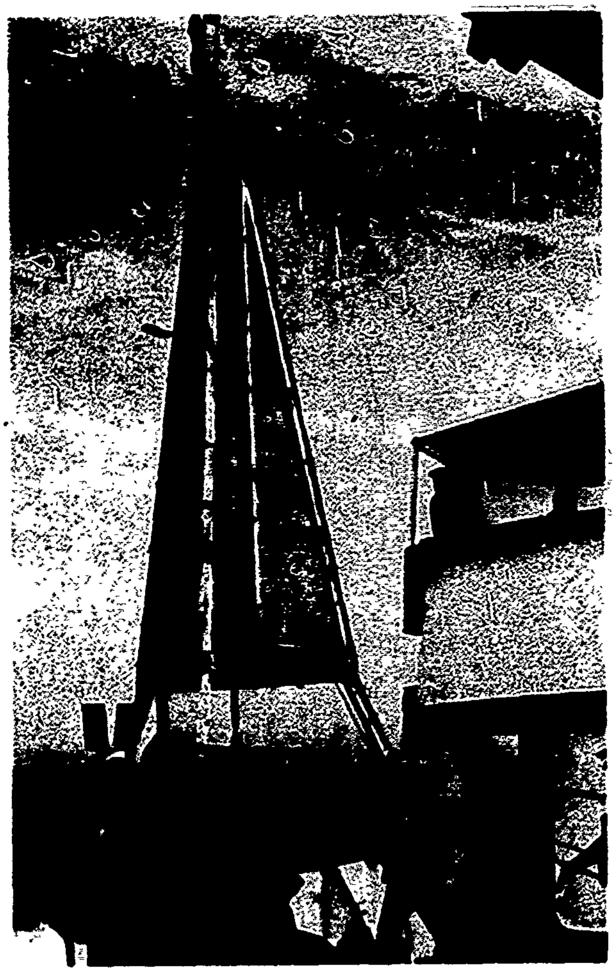
the cable is transmitted to the sampling tube as a downward thrust. Hence, the corer utilizes two principal means of penetrating the bottom: (a) vibration, and (b) conversion of tension on the main cable to a downward thrust on the sampler tube. The vibrator is a single-phase rotary type, delivering 135 kg at 60 Hz, and operates from a shipboard portable, gasoline-motor generator unit. The core tube is pulled back into the corer before the assembly is raised. Sampler characteristics are listed in table 4.

Other samplers

42. A pneumatically operated core sampler (the Mackereth sampler) has been developed for operating in water depths of 250 m (820 ft).[35] The bottom-rest system, which was designed for lake work, is held to the lake bottom by hydrostatic pressure acting on a 4-ft-high by 18-in.-ID-cylindrical anchor chamber embedded in the sediment. The corer sits on top of the cylindrical chamber and operates along the axis of the cylinder after it has become anchored. Upon contact of the cylindrical chamber with the bottom, water is pumped out of the chamber to the surface. As this withdrawal occurs, hydrostatic pressure drives the anchor into the bottom. When mud appears in the discharge aboard the surface vessel, the pump is shut off. The corer is then driven into the sediment by compressed air acting behind a piston, which pushes the core tube past another fixed piston into the soil. Using this corer, 1-1/2-in.-diam, 19-1/2-ft-long samples of glacial clay, said to have had no apparent evidence of disturbance, have been obtained. Compressed air may be supplied by bottled air or by a compressor. The entire assembly is lifted to the water surface with the core tube in the extended position. The sampler does not have a liner, and the sample is extruded by retracting the sampling tube into its housing and ejecting the soil sample into horizontal sample troughs. The characteristics of this corer are listed in table 4.

43. Richards[61] has developed a bottom-rest system which can take a 4.2-in.-ID, 9-ft-long sample. The system (fig. 17) is comprised of a bottom bearing platform and an upper tower. Electromechanical arrangements drive a sampling tube or a testing device from the tower into the soil. The system is capable of obtaining a soil sample, inserting a

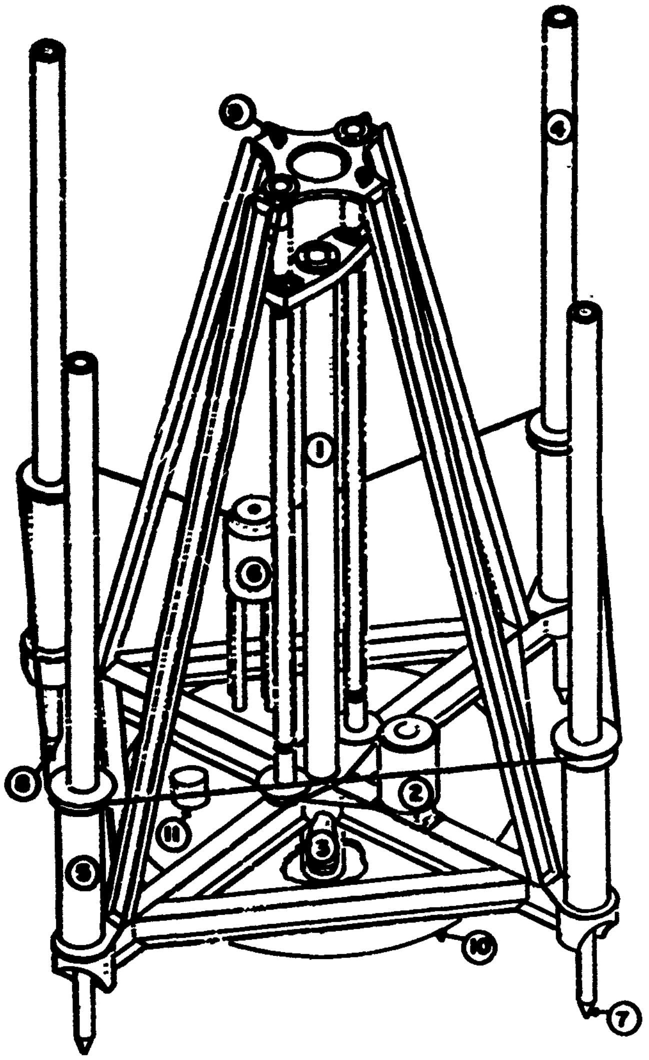
nuclear densimeter probe, or pushing and torquing a vane shear device. Each of these is a separate operation and requires raising the system, changing the device, and re-lowering. A wet-cell energy source and electric motors for powering the system and an electronics package for transmission and relay of commands and data are all mounted at the base of the tower and are either in pressure-compensating or pressure-protected enclosures. An umbilical cord from the surface vessel to the bottom system carries commands from the ship to the system and data from the system back to shipboard recorders. The corer characteristics are those of the Richards' hydroplastic corer described in table 4.



(Courtesy of Norwegian Geotechnical Institute, Oslo, Norway)

Fig. 17. Richards' bottom-rest system with vane shear equipment attached (from reference 61)

44. The University of Rhode Island has developed a bottom-rest system which is called the Deep Ocean Sediment Probe (DOSP) for the U. S. Navy Underwater Sound Laboratory (fig. 18). An electromechanical system is used to drive (without rotation) four test probes simultaneously, or to alternatively drive (without rotation) a thin-walled corer, all to a maximum penetration of 5 ft into the bottom. One probe houses a sound source, and the other probes house hydrophones to pick up the sound transmissions and thermistors to measure soil temperatures. The coring tube is made of acrylic



1. Sediment samples, 3" diam, 5' long, acrylic, 3/16" wall
2. Sampler drive motor and gear reductor
3. Core catcher-cutter assembly
4. Probe housing, 5' diagonal separation between probes, 5' long
5. Probe bearing support
6. Probe drive motor and gear reductor
7. Sound source probe tip, sparkplug in stainless steel
8. Hydrophone-thermistor probe tip, 3 each, pvc
9. DOSP support padeyes
10. Hydrostatic anchor skirt, 42" diam, 1/2" thick, 12" high
11. Hydrostatic anchor pump, 2 each pressure taps

NOT SHOWN

a. Electronics pressure cases (2 each)
b. Sing-around velocimeter
c. Pressure compensated automotive batteries (2 each)
d. Cable feed and takeup reel
e. Electrical cabling

(From preprints, Offshore Technology Conference Paper No. OTC 1290. 1970, SME/AIME)

Fig. 18. Deep Ocean Sediment Probe (DOSP)(from reference 62)

plastic and takes a 3-in.-diam core. The entire unit is held to the bottom by a 42-in.-diam by 12-in.-high hydrostatic anchor, which operates in a manner similar to the Mackereth hydrostatic anchor. Commands and data are transmitted via an umbilical cord running from the surface vessel to the DOSP.[62]

45. A multiuse sampling and test frame, DOTIPOS, has been developed to take 10-ft cores (discussed later in this report). The U. S. Naval Civil Engineering Laboratory has completed the design of a 50-ft bottom-rest corer which is expected to be constructed in the near future.

46. Various techniques for obtaining bottom samples from greater depths are suggested in the literature. Rosfelder[55] suggests the use of a mole-type corer which penetrates to the desired sampling depth by jetting and then pushes a sampling tube into the soil below the depth. With the development of a hole reentry system, this mole corer could provide incremental sampling. Another suggestion is to use electroosmosis to assist the penetration of samplers through thixotropic soils which often overlay consolidated formations at sea. This process could be used both on the outside of the sampler body to achieve greater sampling depth and on the inside of the core tube itself to reduce wall friction and sample disturbance. Rosfelder also proposes the use of a sampler which obtains continuous samples in flexible tubing which, in turn, is wound around drums and stored for later examination of the soil obtained. The drums could be located either above water or on the ocean bottom. All of these concepts require further investigation. Frohlich and McNary[63] are developing a hydrodynamically actuated hard rock corer. With this corer, ambient hydrostatic pressure would be used to accelerate a mass of sea water downward through a vertical pipe to drive a coring tube into the sea floor.

47. To summarize, propelled single-entry drive samplers are used to accomplish deeper bottom penetration and/or to obtain disturbed soil samples. A variety of energy sources are used or suggested: (a) electromechanical couplings, (b) rocket fuel, (c) vibratory systems, (d) compressed air, (e) water jets, (f) electroosmosis, and (g) hydrostatic pressures. Some of these are still conceptual, others are in prototype stage, and some are production models. They all require much improvement before samples can be obtained from sufficient depths for many foundation engineering purposes. Repeated sampling at various depths in a localized area depends on the ability of the surface vessel to hold a position over a point on the ocean floor and on the degree of drift of the corer on subsequent lowerings. To obtain deep samples at a fixed location, drill string operations which permit repeated entry into the same borehole are required.

Repeated-Entry Drive Samplers

48. In general, repeated-entry drive sampling requires the use of continuous drill string connections from the water surface to the bottom of the test hole. The major differences between this system and others are the ability to obtain samples at greater depths below the sea bottom and the capability of obtaining higher quality undisturbed samples of all material encountered in the borehole.

49. Ocean floor sampling using drilling equipment is primarily an adaptation of onshore drilling systems to the ocean environment of a continuously moving sea surface. Drilling equipment must be insulated from the rise and fall of the sea surface, and lateral motions must be held to less than those which would cause excessive bending moments in the drill string (or casing if casing is used). As recently noted,[64] the problem is one of providing an adequate base from which to conduct drilling operations. This might be (a) a barge, sunk at the site, then refloated upon completion of drilling; (b) a fixed platform, raised above the sea surface on legs jacked into the sea floor; (c) a fixed platform supported on piles; (d) an anchored barge; or (e) a self-propelled vessel using anchors and/or auxiliary thrusters to maintain a relatively fixed location. An elevated platform minimizes the effects of tides, waves, and currents, but is costly and is used for soil exploration only for major projects. Anchored barges are used until the distance between shore and the drill site becomes so excessive that an oceangoing tug is required for support purposes. A more flexible system is to place the drill rig on self-propelled vessels that can operate relatively independently of shore; a sophisticated development is the Global Marine Challenger used on the Deep Sea Drilling Project (DSDP), but this type equipment is hardly feasible for engineering projects because of its high cost. The choice of equipment for a particular drilling project is largely an economic one determined by depth of water, distance from shore, required depth of exploration beneath the bottom, and the size and importance of the proposed project.

50. Drill rigs used offshore are those commonly in use for

heavy-duty subsurface exploration onshore, and the sampling tools used on the end of the drill string are the same as those used onshore. These include standard split-spoon samplers, wire-line samplers, piston samplers, and rotary core barrels. The relative merits of these samplers for onshore work, as discussed in the literature, are applicable to offshore work also. The sample disturbance aspects of these samplers also are presented in the literature, and the Hvorslev ratios noted earlier in this report for drive samplers are applicable.

51. Drilling operations can be accomplished with or without casing and require the use of drilling mud. With casing, the mud can be returned to the sea surface, settled in a sump, and recirculated. The use of bottom- (sea floor) supported casing that is separated from the vertical motion of the surface platform is feasible until water depth becomes excessive, e.g., the column strength of some nominal-sized casing is not sufficient for it to be supported only on the bottom in water depths greater than 150 ft.[64] Drilling without casing results in the drilling fluid and cuttings being discharged on the sea floor.

52. WES has used casing from the water surface into the borehole while obtaining undisturbed sand samples from the subbottom of the Ohio River at the site of the Mound City, Illinois, lock and dam project and from the subbottom of Lake Pontchartrain off the mouth of the Inner Harbor Navigation Canal in New Orleans, Louisiana. Both sampling operations were performed in water depths of up to 40 ft. On the Ohio River project, a truck-mounted rotary drill rig was placed aboard and operated from an anchored barge. On Lake Ponchartrain, a truck-mounted rig was operated from a self-elevating platform which raised itself above the lake surface by jacking its legs into the lake bottom.

53. The sampling operation followed the WES procedure for undisturbed sand sampling below the water table.[65] In this procedure, a fishtail drill bit with baffles (fig. 19) advances the borehole. The baffles deflect the drilling mud jets upward and prevent the jets from disturbing the soil below the bit. When the sampling depth is reached, the borehole is cleaned out, the drill string and fishtail bit withdrawn, and the soil sampler lowered through the casing into the borehole.

Fig. 19. Fishtail bits with baffles

54. A Hvorslev-type piston sampler (fig. 20) is used to obtain undisturbed sand samples. The 16-gage steel sample tube has an inside diameter of 3 in. and is commonly 2.5 ft long. The angle of taper of the cutting edge is 10 deg, and the area ratio is approximately 11. Fig. 21 shows the relative positions of the piston and sampling tube during the sampling operation. The sampler is pushed continuously into the soil by the oil-operated hydraulic drive system of the drill rig. A rate of sample penetration that has given satisfactory results in the past is 0.16 ft/sec. Additional details on the drilling, sampling, and sample handling of saturated sands are presented in reference 65.

55. The drilling mud in the drill hole below the casing is considered partially responsible for the successful retention of cohesionless soil in the sample tube. The drilling mud forms a membrane on the bottom of the sample and has two effects that contribute to sample retention. The vacuum on the upper side of the sample due to the piston action causes the drilling mud to exert an upward pressure on the bottom of the sample. This pressure, coupled with the natural tendency of sand to arch, materially assists in preventing downward movement of the sample. The drilling mud prevents drainage of free water that otherwise would cause progressive erosion of the bottom of the sample. In addition, the viscosity of the drilling mud impedes the formation of turbulence around the bottom of the sample tube; turbulence frequently causes

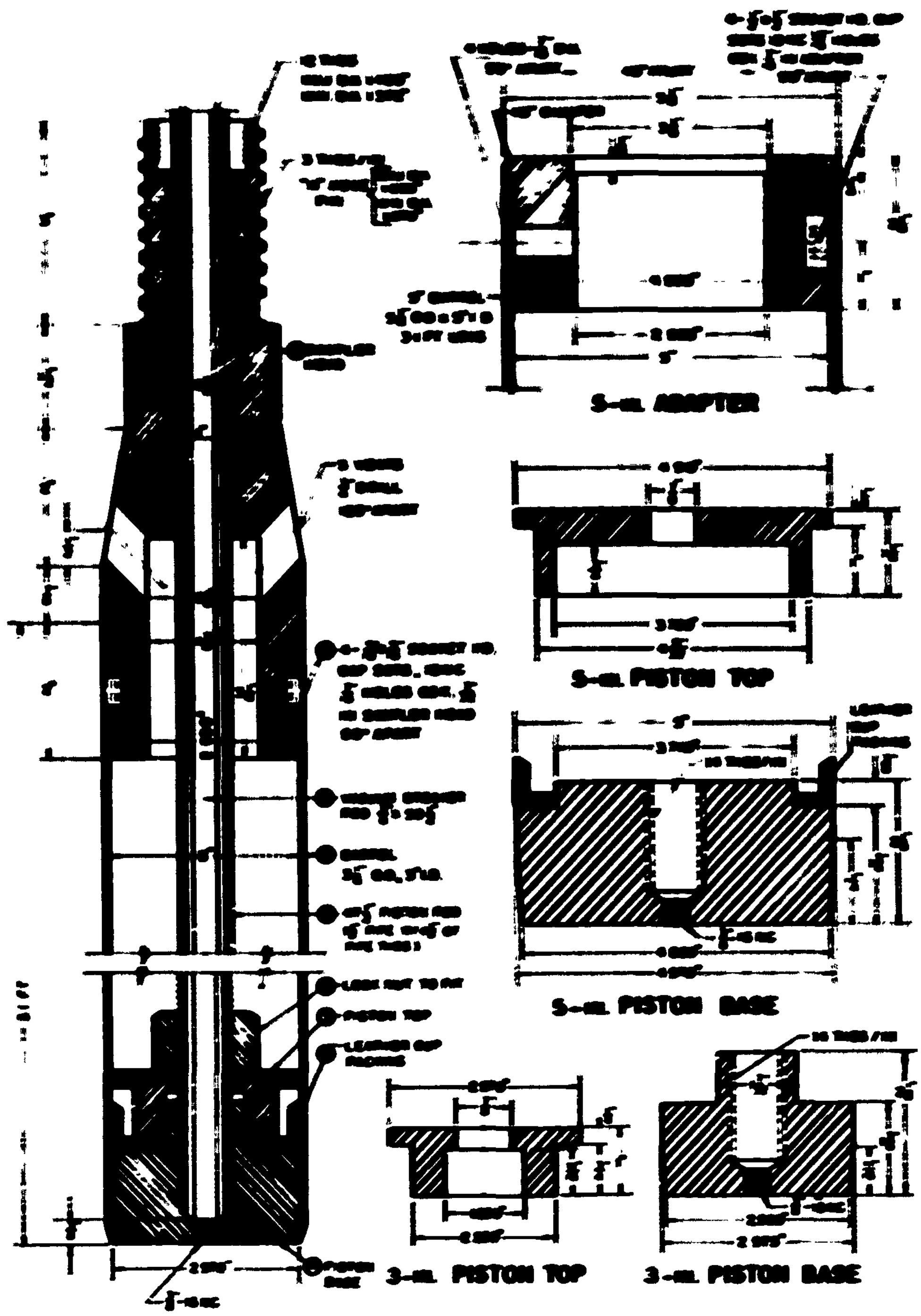


Fig. 20. Three-in. Hvorslev-type piston sampler

progressive erosion of sand samples during withdrawal of a sampler from borings drilled with clean water.

56. McClelland Engineers uses a wire-line system without casing

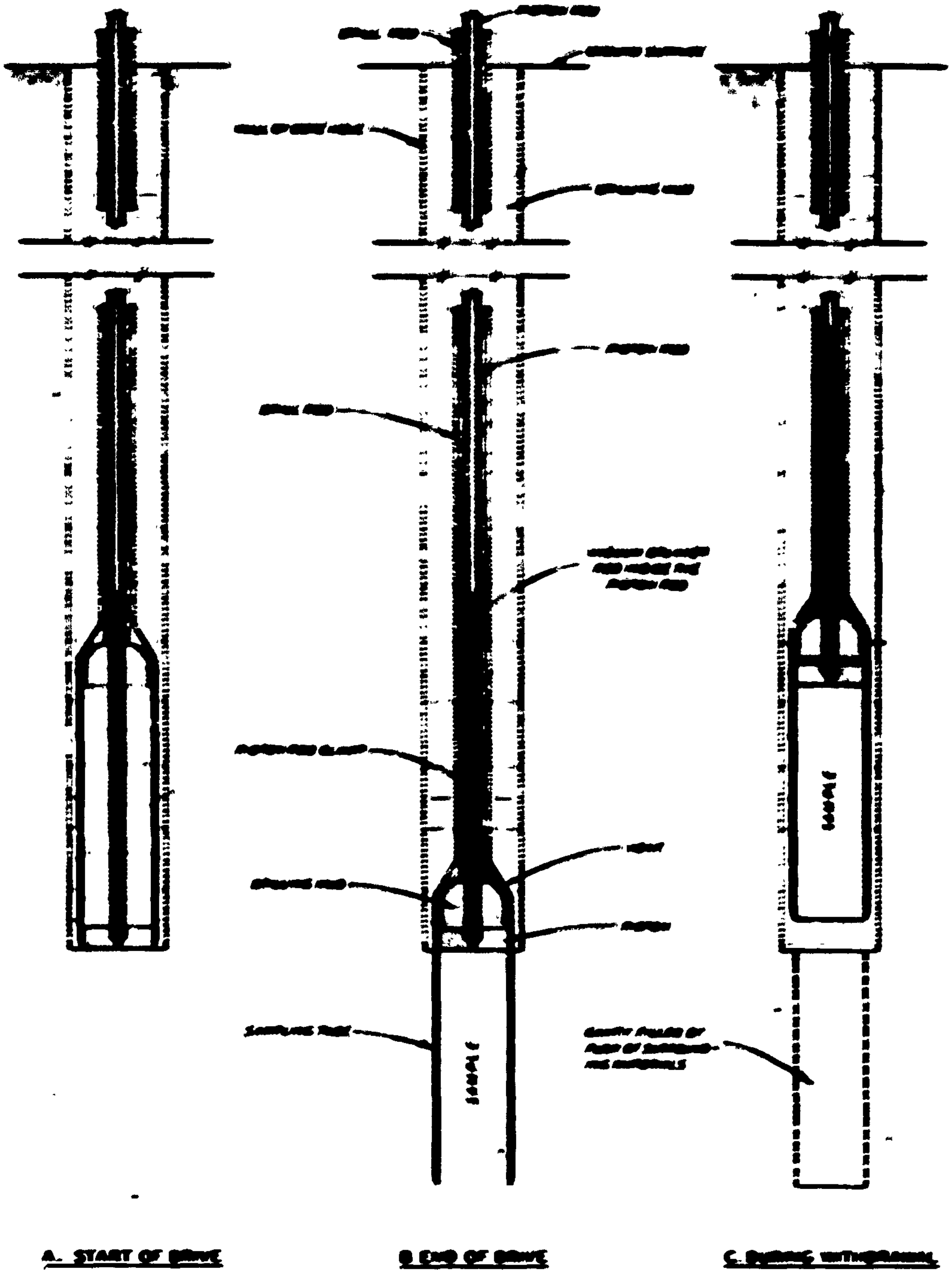


Fig. 21. Cross section through boring during Hvorslev-type piston sampler drive and withdrawal

(shown in fig. 22) to drive an open, thin-walled, 2.25-in.-OD sampler through 2.98-in.-ID drill pipe into the borehole bottom.[64] Mud and cuttings are wasted at the sea floor as the borehole is advanced to

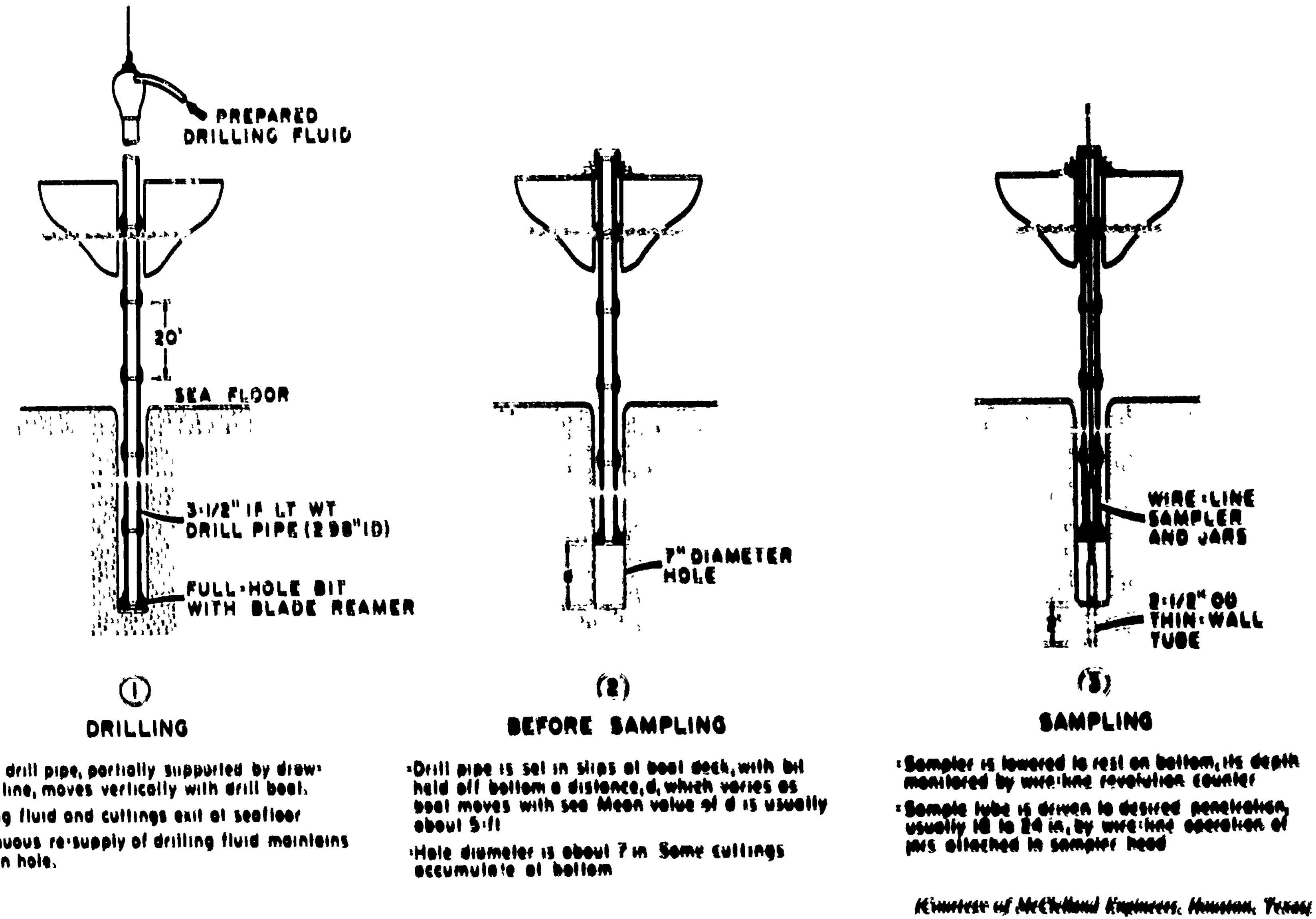


Fig. 22. Wire-line soil sampler (from reference 64)

sampling depths. The sampler is driven into the bottom by repeated blows, up to a maximum of 30, of a sliding weight (175 lb) dropped from a height of 5 ft, and the number of blows needed to achieve 24 in. of penetration is recorded. The 30-blow limit was set to prevent overstressing of the wire line during retrieval operations. Samples obtained using this method are generally somewhat disturbed.[6,64]

57. McClelland Engineers conducted a comparative onshore sampling program at Venice, Louisiana, in clay soils that were geologically and physically similar to those on the continental shelf in the Gulf of Mexico.[64] Both the 2.25-in. and the 3-in., wire-line samplers, a 3-in., open-drive push sampler, and a 3-in., fixed-piston sampler were used. The samplers selected permitted the relative evaluation of the effects of tube diameter, impact drive, push drive, and open-tube versus fixed-piston sampling on sample quality. Shear strengths were determined by unconfined compression tests and miniature vane tests on the soil samples and were compared with results of in situ vane tests.

58. A consistent trend of strength increase with depth was found in the formation investigated. This was true regardless of the combination of sampling and testing used. However, the numerical values of shear strength did reflect the effectiveness of the combinations used. Generally, the lowest strengths were obtained for the 2.25-in., wire-line soil samples, with progressively greater strengths occurring for the 3-in., wire-line samples; the 3-in. open-drive push samples; and the 3-in., fixed-piston samples. These findings confirmed Hvorslev's conclusions that: (a) sample disturbance decreases with increasing diameter of the sampler, (b) pushing a sampler causes less sample disturbance than hammering a sampler into the soils, and (c) a fixed-piston sampler causes less disturbance than an open-drive sampler. The unconfined compression tests generally gave lower shear strengths than the miniature vane tests. The in situ vane tests provided the highest strength values. The choice of the sampler and correct usage of shear strengths obtained are functions of the engineer's experience, and any applied correction factors are based on judgment.

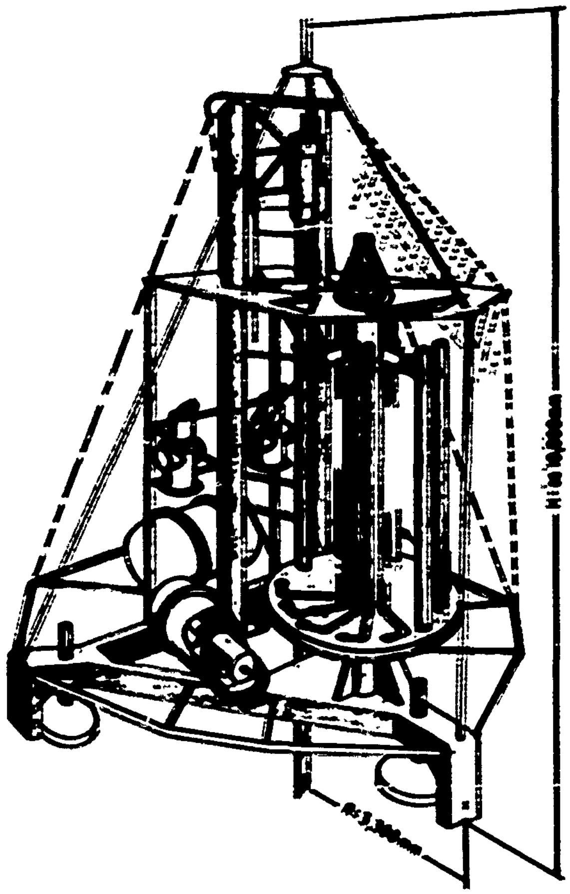
59. A major problem of repeated-entry drive sampler operations

from the surface without casing is the inability to reenter a hole after the drill string has been pulled up either to change drill bits or because of inclement conditions on the sea surface. To provide reentry capability, a system has recently been developed for use on the Global Marine Challenger DSDP.[66,67] However, this has not as yet (1972) been used in engineering soil exploration.

60. Another type of repeated-entry drive sampler is the bottom-rest Geodoff II, which was designed by Conrad-Stork, Haarlem, Holland, and the Dutch Geological Survey Department.[68] With this sampler, rotary drilling is used to advance the drill pipe. If the soil resistance is low, the core barrel is pushed into the soil ahead of the drill bit. The length of core barrel protruding in front of the drill bit decreases with increasing soil resistance. When the front of the core barrel is pushed back completely into the drill bit, i.e., when the soil resistance is high, the core barrel is locked to the drill pipe, and further sampling is achieved by rotary drilling. The Geodoff II can core to depths of 48 m (157 ft) in the sea floor, taking twelve 4-m- (13 ft) long cores. Each 4-m length of sample is brought up through the drill string and stored on a rotating supply disk before the next 4-m length of core barrel is inserted into the hole and the next 4-m length of drill pipe is attached to the drill string (see fig. 23). Upon completion of sampling and drilling, the drill string is broken down and also stored on the supply disk. The electric motor, hydraulic pumps, and computer are housed in drums on the main body. An umbilical cord to the surface vessel supplies electric power, electric signals, and drilling mud. Information is not available on the core barrel details or on results of any field tests. The advantages of this sampler over other bottom-rest samplers are its reentry capability for taking consecutive samples and its capability of sampling both hard and soft soils.

61. In summary, repeated-entry drive sampler exploration systems can obtain soil samples from greater depths below the sea floor than can the single-entry drive sampler systems. The McClelland Engineers' wireline system obtains disturbed samples, and the Geodoff II system appears to be capable of obtaining relatively undisturbed samples, but this has

GEODOFF III. The main body, its three legs positioned on the sea bottom, contains the rotary drilling device and a rotating supply disc on which the complete drill string is placed in 12 lengths of about 4 meters each. The electric motor driving the hydraulic pump, the computer, etc. are housed in the drums on the main body. An umbilical cord from the surface vessel supplies electric power, electric signals and drilling mud. The filled core barrels are stored by mechanical arms after every pipe length of drilling progress. The tubes are placed on the supply disc in special clamps. After the hole has been completed, the drill string lifting procedure is reversed and the drill pipes are returned one by one to their original positions on the supply disc.



(From Ocean Industry, Vol 5, No. 10, Oct 1970)

Fig. 23. Geodoff II repeated-entry drive sampler (from reference 63)

yet to be proved since details on the core barrel and results of field tests are not available. The wire-line sample tube and Geodoff II core barrel are similar to the devices used on land, and the Hvorslev criteria used to evaluate ability to obtain undisturbed samples are applicable. The need still exists for an undisturbed sampling system which can operate economically and efficiently in continental shelf water depths and obtain samples from greater subbottom depths. The system should be operable in the sea surface conditions that exist over the continental shelves.

Handling, Packaging, and Transporting Undisturbed Samples

62. Obtaining high-quality undisturbed samples by exacting drilling operations is not in itself adequate since the real purpose is to test and evaluate undisturbed samples. Often this cannot be done at the drilling location, but must be accomplished at a different location at a later time. Therefore, it is essential that samples of cohesive soils be removed from sampling tubes, properly packaged to prevent loss of

moisture and disturbance, and carefully handled during transport to the testing site. In the case of cohesionless saturated sands, tube samples must be given special treatment to ensure that determinations of in situ characteristics are not adversely affected. The procedures given in paragraphs 4-6 of the Corps of Engineers soil sampling manual[69] are appropriate for the preservation and shipment of undisturbed samples of marine soils. However, several considerations not common in land practices must be taken into account in marine samples. One is that cohesive marine soils near the surface of the ocean bed are frequently much weaker and of much lower density than soil deposits on land; therefore, undisturbed samples of such soils must be handled even more gently and carefully. Secondly, cohesive marine soils taken from sites where water depths are great may undergo considerable expansion when brought to the water surface because of the relief of high hydrostatic pressures. If there are no indications of disruption of soil structure, samples may still be suitable for strength and consolidation tests in the laboratory using back pressure, as discussed in Part III. Should severe disruption be evident, this must be taken into consideration in evaluating the validity of test results; presently, no procedure is known in which an undisturbed sample can be sealed under its in situ hydrostatic regime and tested in the laboratory in a simulated in situ environment.

Geophysical Techniques

63. Geophysical techniques are used extensively in the ocean to provide general information about the sea floor in a given area in a short time. Only a brief discussion is presented in the following paragraphs because of the vast scope covered in detail in other publications.[70-72] Geophysical techniques involve the detection and recording of the response of the sea floor and subbottom to various energy sources or the determination of gravitational or magnetic anomalies. Geophysical response data are recorded automatically on strip charts, magnetic tape, or punched-paper tape. Strip charts provide an immediate record of results which can be used to modify the exploration program as it

progresses, whereas magnetic-tape or punched-paper-tape records require further processing before results can be evaluated.

Acoustical techniques

64. Reflection acoustical techniques are commonly used for engineering work; they involve the generation of an underwater acoustical wave by explosives, gas guns, electrical discharges, or electromagnetic sources and the recording of the reflection waves from the bottom and subbottoms. Table 6 lists characteristics of a few acoustical systems. The "Sparker" creates a compressional sound wave by an underwater discharge of a high electric charge of 10,000 to 20,000 v across two electrodes, providing wave penetration of 1,000 ft or more into the ocean bottom.[74,75] The "Boomer" is another means of generating acoustical waves; through the manipulation of the electromagnetic field around two plates, the two plates are caused to alternately attract and strike and then repel each other. The Sparker and Boomer are used for continuous reflection surveys since they are able to generate acoustical waves repeatedly in quick succession. In both systems, the sound source (commonly referred to as a fish) and a series of hydrophones are trailed behind the surface vessel. The sound source could be attached to the vessel's bottom, but the trailing method provides flexibility in moving the equipment from vessel to vessel and permits the sound source and hydrophones to be spaced sufficient distances from the vessel to reduce significantly or eliminate noise effects of the vessel from the recording. The energy source to be selected would depend on the resolution and penetration desired (see table 6 and reference 73).

65. Records are obtained of reflections of sound waves from various soil or rock horizons in the form of automatically recorded output. An experienced geophysicist can identify various stratigraphic layers and the presence of rock subbottoms from such records. Correlation of these records with available logs of borings can provide much information on the lateral and vertical extent of specific soil horizons. Plots of time of travel versus distance between source and detector provide information on velocities of sound in the various layers. Correlations of sound velocities with soil properties have been attempted,

but much refinement is needed for them to be useful to the soils engineer.[76-78]

66. The refraction acoustical technique involves the recording of refracted sound waves from the bottom and subbottoms. Compared with the reflection technique, the refraction technique requires stronger sound sources and takes more time and the source and detectors must be spaced further apart. However, the refraction method provides deeper subbottom penetration. It is not commonly used in offshore engineering work, but is mentioned here since the results of an old refraction survey may provide the only existing data available in some areas.

67. Some characteristics of acoustical surveying are worth noting. High sonic frequencies give better resolution of the sea-bottom interface, but low sonic frequencies provide better penetration.[79] Acoustical impedence refers to the product of the density and acoustical velocity in the layer in question. Thin layers of soils are often masked out by stronger responses of the sandwiching layers. If a stratum has a lower acoustical impedence than the layer above it, this stratum generally will not be detected. These undetected layers are likely layers of soft material (low density), and thus are important in foundation engineering studies. Since hydrophones are simply receivers of sound energy, they also pick up noise and multiple reflections of bottom features, i.e., reflections that bounce back and forth between the mirror effect of the air-sea interface and a bottom reflector. By proper filtering, systems have been developed to minimize the effect of multiple reflections.

Gravity and magnetic field techniques

68. Other geophysical techniques include measurement of gravitational and magnetic anomalies. The gravimeter measures variations in acceleration due to gravity, and the magnetometer measures variations in the magnetic field. Both local gravitational and local magnetic fields reflect relative positions of rock in the subbottom. The gravimeter is sensitive to the increased gravitational attraction of denser masses along a traverse. The magnetometer is sensitive to the differing

magnetic properties of various rocks and their closeness to the surface. The information obtained using these techniques may be presented in profile and/or map form, showing lines of equal anomalies or lines of true strengths of the gravity or magnetic field. This corresponds in a sense to developing soil horizons in soil profiling and lines of equal elevations in topographic mapping. It is not implied that analyses of data from gravity and magnetic surveys are simple; the anomalies recorded are responses to three-dimensional fields, and the conversion to two-dimensional profiles and plans requires a good understanding of geophysics and field theory. The use of gravitational and magnetic data along with the engineer's more common tools may prove valuable in underwater soils engineering. Other geophysical tools, such as gamma ray and electrical resistivity systems, are discussed in the next part of this report.

69. In summary, geophysical tools may be a helpful adjunct to any marine soil study. The information presented in geophysical data sheets or maps can be useful in determining lateral and vertical continuity of soil strata, the location of bedrock, and intrusions of bedrock. Another consideration is the availability of geophysical data in various oceanographic institutions. Engineers may find valuable information in the geophysical surveys or tracks of oceanographic cruises which may have crossed areas being investigated. These may be the only existing sources of information on the area of interest, and they may be invaluable in planning further exploration work and in interpretating results of later exploration. The use of acoustical surveys should not be overlooked when sampling operations are conducted from an oceanographic vessel, for often the equipment is easily available and can be operated while traveling to and from the sampling site. The recording equipment can usually run continuously, and requires only intermittent attention to adjust the scales, change the paper from time to time, and record the cruise track and other items of interest on the printout.

PART III: TESTING MARINE SOILS

70. This part of the report presents the state-of-the-art of testing of marine soils. Both laboratory and in situ testing are discussed, and the presentation is generally limited to engineering tests. Tests are performed to obtain quantitative values which hopefully will be indicative of the condition of the in situ soil. The objectives of tests on marine soils are identical with those of tests on terrestrial soils, viz., (a) meaningful classification of the soil being tested, (b) evaluation of the consolidation characteristics of the soil, and (c) determination of the shear strength of the soil.

Laboratory Testing

71. A detailed discussion of laboratory soil tests is covered in soil mechanics literature and is beyond the scope of this report. This section discusses special considerations which are being given or should be given to testing marine sediments. The usefulness of numerical values from any test depends on how representative the test conditions were of the in situ conditions. Changes in the soil sample from the time of its removal from the sea floor to the time it is tested can significantly affect test results.

72. The removal of marine soil from its natural sea floor environment of generally cold temperatures and high hydrostatic pressures to laboratory conditions of room temperature and atmospheric pressure causes (a) an expansion of existing free gases, (b) the release of dissolved gases out of solution, and (c) the increased generation of gases by some biota that find the changes individually favorable. The increased volume of gas in the soil sample results in (a) a change in soil structure, (b) a decrease in the degree of saturation, and (c) a decrease in density. These changes influence (a) stress-strain, (b) shear strength, and (c) consolidation characteristics of the soil sample. The expansion pressures of gases in soil have been sufficient to cause the explosion of soil samples stored in plastic liners.

73. Many cohesive marine soils have thixotropic and sensitive characteristics, and many cohesionless soils have low density and loosely packed structures. Test specimens of these soils can be easily disturbed during trimming and other preparation processes. The transportation of soil samples from sea floor to the testing apparatus and the many rehandlings along the way provide other opportunities for sample disturbance.

74. Interstitial water in sea sediments generally have a salinity of 35 parts per thousand (ppt), and the reduction of salinity by the use of distilled water in tests may affect the liquid limit, plastic limit, shear strength, consolidation characteristics, and sensitivity of marine soils.

Gradation

75. Oceanographers in general have followed different procedures from those used by engineers in determining gradation and have used different grain-size scales for classification. This is particularly true in the grain-size analysis of fine-grained soils. Hydrometer analysis is the common tool of the engineer, but geological oceanographers have used the pipette technique.[80] Engineers use about 50 grams of fine-grained soil for the hydrometer analysis, but oceanographers use only about 25 grams of soil for analyses of both coarse- and fine-grained soils. Both disciplines use sieve analysis for grading the sand-size particles, but sometimes oceanographers also employ the rapid sediment analyzer (RSA). Fig. 24 shows the RSA facility at the U. S. Coastal Engineering Research Center (CERC). This unit has been adapted for direct punching of computer data cards as the sediment falls through the metering column. The RSA at CERC analyzes an 8- to 10-gram sample.[59]

76. Presentation of results also differs for the two disciplines. The soils engineer refers to percent passing a certain sieve size or precent finer than a given grain diameter. The oceanographer refers to various phi sizes, the median phi size, and uses a statistical approach to describe the gradation curve, i.e., phi skewness, phi kurtosis, phi deviation, etc. Krumbein defined a phi unit as the negative logarithm to the base two of the particle diameter in millimeters:

(Courtesy of U. S. Army Coastal Engineering Research Center)

Fig. 24. Rapid sediment analyzer. Settling tube and pressure tube are shown at left of photo; connecting tubes supply and drain water. At right is console housing digital voltmeter with timing and sampling circuitry; atop console is analog strip chart record for visual recording of pressure-time decay curve. In center is card punch for direct punching of data as sediment falls through metering column (from reference 59)

$$\phi = -\log_2 \text{ (D in millimeters)}$$

This notation works well with the Wentworth classification of soils, which is commonly used in ocean work, since the boundary between various Wentworth sizes may be expressed as whole phi units.[81] Table 7 relates the Wentworth scale to phi units, grain-size diameters, sieve sizes, and the Unified Soil Classification System (USCS).[82] Table 8 lists in detail various USCS soil groups and their identifying characteristics.

Plasticity

77. Atterberg limits for marine and on-shore soils are determined

by the same procedures. The plastic limit (PL) and liquid limit (LL) of a soil identify the specific USCS soil group to which a fine-grained soil belongs. Tables 8 and 9 delineate the various soil groups according to their PL and LL values. Reference 82 gives a full description of the use of the tables. The effect of using distilled water in such determinations on marine soils may be worth further investigation. Some investigations of the effect of leaching on the liquid limit have shown that the liquid limit remains unaffected by changes in the salinity of the interstitial water until the salinity falls below 15 ppt, after which the liquid limit decreases at an increasing rate with decreasing salinity. The plastic limit was found to be unaffected by variations in salinity. Because of the effect on liquid limit values, both the liquidity index and activity values are affected by the reduction of salinity.[83-85] Mixing distilled water with a marine soil does not cause leaching action but does reduce the salinity of the pore water.

Specific gravity, water content, and unit weights

78. Determinations of the specific gravity of solids, water content, and unit weight are generally made in the conventional manner used by soils engineers. Corrections for the salt content of the pore water are not usually made. Thus, the dry weight includes both the weight of the soil solids and that of the salts normally in solution in the pore water. For this reason, the water content of the soil may be a little lower and the specific gravity of solids and the dry unit weight may be a little higher than if corrections for salinity had been made. For example, uncorrected results of tests on a marine soil having a true specific gravity of solids of 2.70, a dry density of 60 pcf, and a water content of 68.5 percent would show values of 2.71, 61.4 pcf, and 64.7 percent, respectively, indicating that the differences are of no practical significance.

Consolidation

79. Consolidation tests on marine sediments can be performed in the conventional manner. However, consideration should be given to the use of back pressure to dissolve the gases which came out of solution

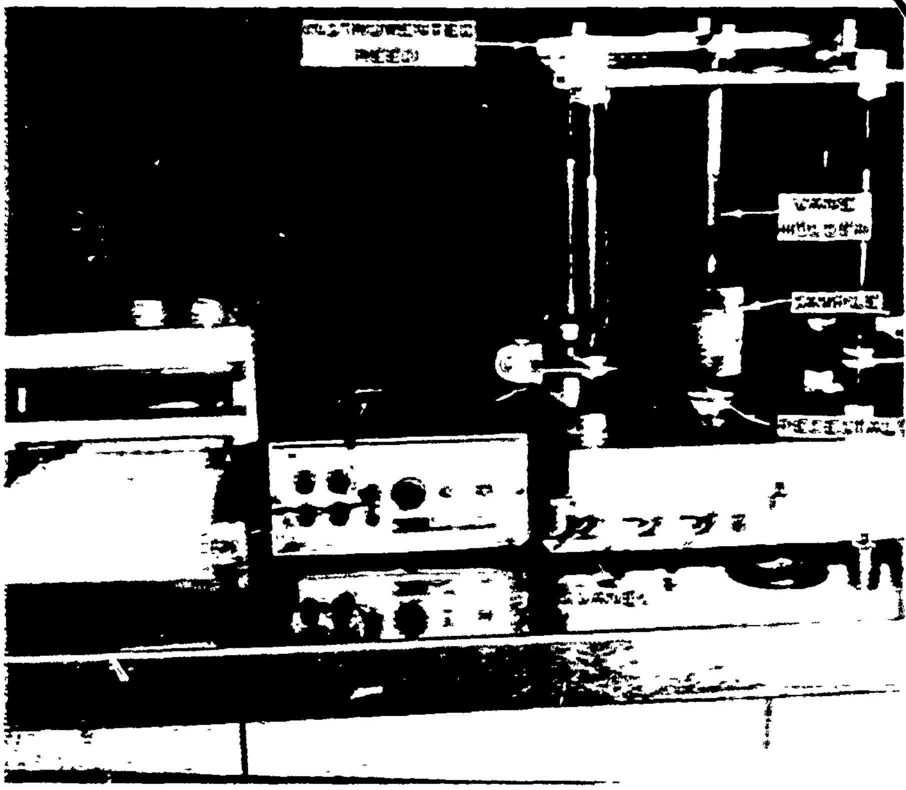
due to the change in ambient pressure that occurred when the sample was transferred from the sea bottom to the sea surface. In this case, a closed-system consolidation apparatus is needed. The use of salt water obtained from the sample area or prepared in the laboratory would minimize changes in salinity of the pore water, which otherwise might affect the consolidation results.

Shear strength

80. The laboratory vane shear test is a common means for determining the shear strength of marine soils. This test provides a convenient and quick way of testing soil samples while they are still in their liners or sampling tubes. It is important that the vane dimensions be sufficiently smaller than the inside diameter of the liner or tube to reduce sidewall effects.

81. Hironaka[33] of the Naval Civil Engineering Laboratory (NCEL) applied a stepwise regression analysis to laboratory test data and developed linear equations relating (a) vane shear strength to depth below surface, liquid limit, and median grain diameter, and (b) bulk wet density to vane shear strength and sensitivity (SN). The laboratory data were derived from tests performed on eight sediment cores taken in water depths of from 2300 to 2500 ft near San Miguel Island, California, with NCEL's gravity-type Ewing corer (see table 4). The soils were sands, clayey sands, and silty sands. (With these soil types the validity of the use of LL and SN becomes suspect.)

82. In performing vane tests, NCEL sample cores are sectioned into 3-in.-long intervals by cutting the plastic liner with the aid of a wire loop attached to a soldering gun. The loop essentially melts the circumference of the liner. A thin-wire saw slices the soil and completes the sectioning.[86] This eliminates the possibility of sample disturbance which might be caused by abrasive sawing of the plastic liner or attempting to extrude the soil from the liner. The vane shear test is then performed on these 3-in.-long sections. Instead of rotating the vane, which is the conventional method of performing vane shear tests, the NCEL vane shear machine (fig. 25) rotates the soil sample by rotating the pedestal upon which the 3-in. section of the core has been



(Courtesy of U. S. Naval Civil Engineering Laboratory)

Fig. 23. NCEL vane shear apparatus with sample being tested (from reference 31)

[illegible]. The torque transmitted to the vane through the soil is measured and recorded automatically.

83. Richards uses an arrangement whereby an entire 9-ft length of core can be tested in the laboratory without having to section and cut up the hard plastic core tube enclosing the sample. The tube is clamped to a vertical frame on top of which is mounted the vane shear machine. After each vane shear test, the vane is pulled up and moved out of the way. The core is then extruded out of the upper end by jacking from the lower end until the soil disturbed by the vane shear test is exposed. The extruded soil is cut off and used for other tests. The vane is reinserted into the core sample, and the operation is repeated. The extrusion operation undoubtedly disturbs the core.

84. Obviously, many arrangements other than the two discussed

above exist for the vane shear testing of soft marine sediments. McClelland Engineers performs miniature vane tests or unconfined compression tests on samples immediately after the thin-walled sampling tube has been separated from the wire-line sampler, i.e., the tests are performed on the vessel or platform as soon as the soil samples are brought to the surface and before they are sealed for shipment to the land laboratory.

85. Some characteristics of the laboratory vane shear test are important to consider in testing fine-grained marine soils. It is probably the simplest shear strength test to perform, since it requires a minimum of equipment and a minimum amount of handling of the soil, thus decreasing the extent of sample disturbance. For very soft soils, it may be the only way of obtaining a measure of shear strength. Essentially, it determines the average shear strength on a vertical cylindrical surface because its operation intrinsically forces this kind of failure. This may be appropriate for homogeneous soils, but many soil deposits are stratified with anisotropic shear properties. The vane shear test essentially provides an unconsolidated-undrained, or quick, shear strength. When this is commensurate with actual conditions and high-quality undisturbed samples are tested, the vane shear strength may give a good indication of the in situ strength.[87] The strength value obtained is related to the rate at which the vane is rotated. A rate of 0.1 deg/sec, prescribed by earlier users of the vane device, is still adhered to by many investigators.[11,33,88,89] Others have kept within the same order of magnitude, e.g., 0.5, 0.37, and 0.25 deg/sec.[61,90,91]

86. The next most commonly used test on fine-grained marine soils is the unconfined compression test. This test involves the application of an axial compression force to a soil cylinder which is unsupported laterally. Therefore, the soil must have sufficient intrinsic cohesion to support its own weight when it is extruded from the sample tube and when the specimen ends are being trimmed to fit properly between the platens of the test machine. Like the vane shear test, this is an unconsolidated-undrained test and is appropriate when this test condition corresponds to the loading condition in the field. It has been a

common practice of researchers in the field of marine soils to compare results of both laboratory and in situ vane tests with results of unconfined compression tests. In many cases, close agreement is obtained, but because of sample disturbance or variations in soil type between corresponding samples used in the two tests, large discrepancies can occur.

87. Triaxial and direct shear tests have also been used to determine the shear strength of marine soils. These offer an advantage over laboratory vane shear and unconfined compression tests because laboratory test conditions can be adjusted to approach in situ conditions through control of the confining pressure and drainage during testing. The direct shear test, which is used to determine shear strength of soils under drained conditions, has the same drawback as the vane test in forcing a failure along a predetermined plane. Triaxial and direct shear tests are more expensive to conduct than the vane shear or unconfined compression tests, and are justified only when high-quality undisturbed soil samples are available. Because of this, along with the relative ease and speed with which vane shear tests and unconfined compression tests can be performed, triaxial and direct shear tests have been infrequently conducted on marine soil samples from deep waters.

88. In some triaxial tests, distilled water is used in applying back pressure to the specimen to obtain 100 percent saturation of the soil by forcing into solution gases that are trapped within the soil. To avoid leaching, it may be preferable to use salt water taken from the vicinity of the sample location or prepared in the laboratory. Many researchers have noted the effects of leaching on the shear strength and sensitivity of clays.[83,84]

89. Another problem which can occur is leakage of chamber water into the soil sample due to the osmotic flow of water through the test membrane. Such flow would make it impossible to perform any long-duration tests at a constant water content.[48,92] The pore pressure would be affected in direct relation to the osmotic pressure. The use of salt water of the same salinity as the interstitial water in the soil sample in the triaxial chamber would eliminate both of these effects.

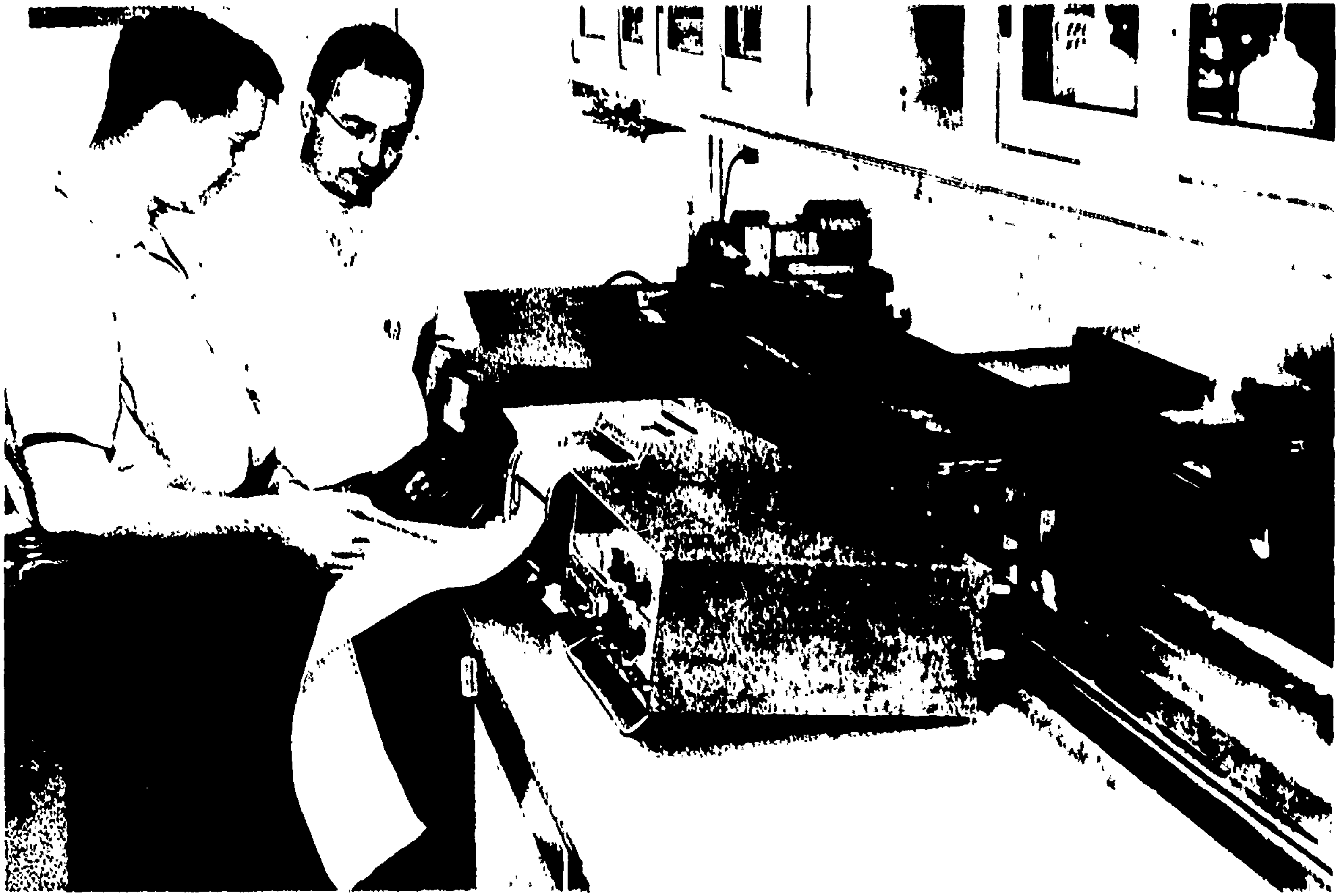
Other laboratory tests

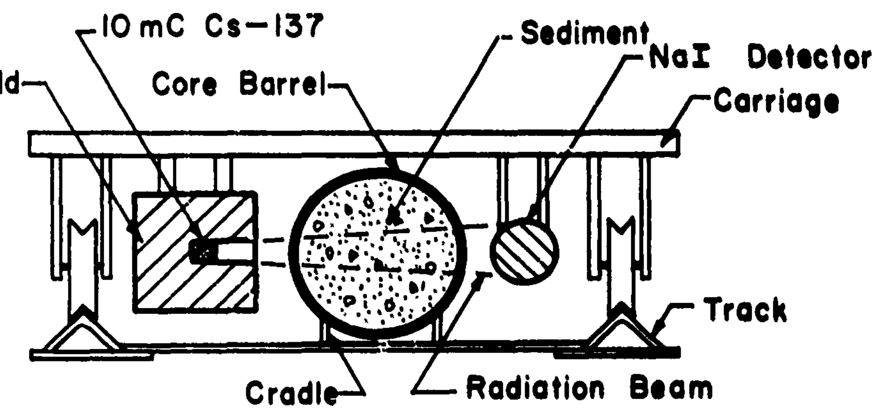
90. Three additional laboratory tests have proven useful: (a) x-ray diffraction, (b) gamma-ray densitometer, and (c) x-radiography. The diffraction test is used to identify the mineral composition of the sediment, and generally involves pulverizing a small sample of soil. The other two are nondestructive tests and can be performed on a sample that is still enclosed in its liner. The gamma-ray densitometer measures the density of the soil sample. One gamma-ray densitometer system uses cesium 137 as its radioactive source and the transmission method of measuring gamma ray density at the detector (figs. 26 and 27).[93,94] The sketch in fig. 27 shows that the core sits on a cradle between the source and detector. The gamma rays are transmitted through the core and excite the detector. The detector is wired to an electronic circuitry which permits the output to be recorded on a strip chart.

91. The x-radiography technique takes x-ray pictures of the full section of the sample while it is still in the core tube. This permits the detection of any disturbed portions or weak zones of the sample, e.g., cracking, large voids, intrusions, and edge bending along the length of the sample, giving the engineer guidance in selecting specimens for shear strength, consistency, and/or other tests. (Core shortening can cause a definite change in density, but x-radiography will not necessarily pick this up.)

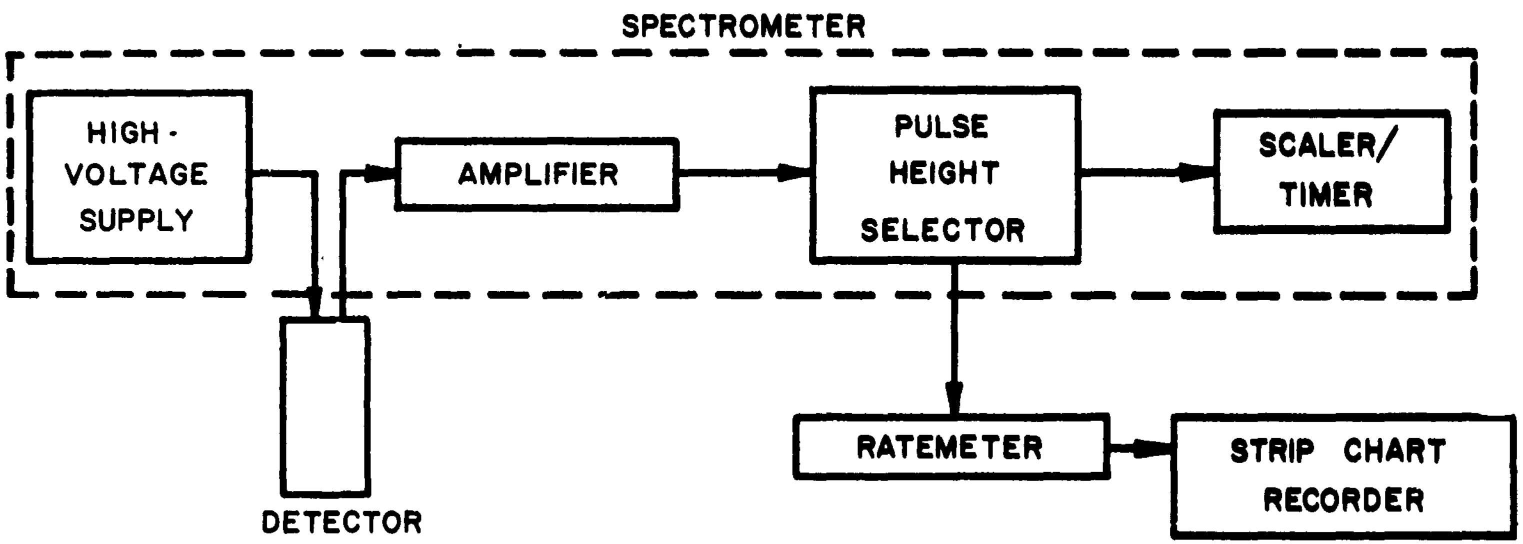
Summary

92. The significant factors to consider in laboratory testing of marine soils are the changes to which the samples have been subjected prior to testing. In performing any tests, consideration should be given to the maintenance of the salinity of the interstitial waters and to the duplication of in situ pressure and temperature. Current state-of-the-art takes a qualitative look at influences on test results due to changes of ambient conditions. Attempts to obtain quantitative values for the effects of changes from in situ to laboratory ambient conditions have not been totally fruitful. This is due in part to the current inability to successfully duplicate in situ environments in the laboratory. In addition, techniques for field determination of in situ properties





a. Arrangement of cradle, carriage, source, and detector

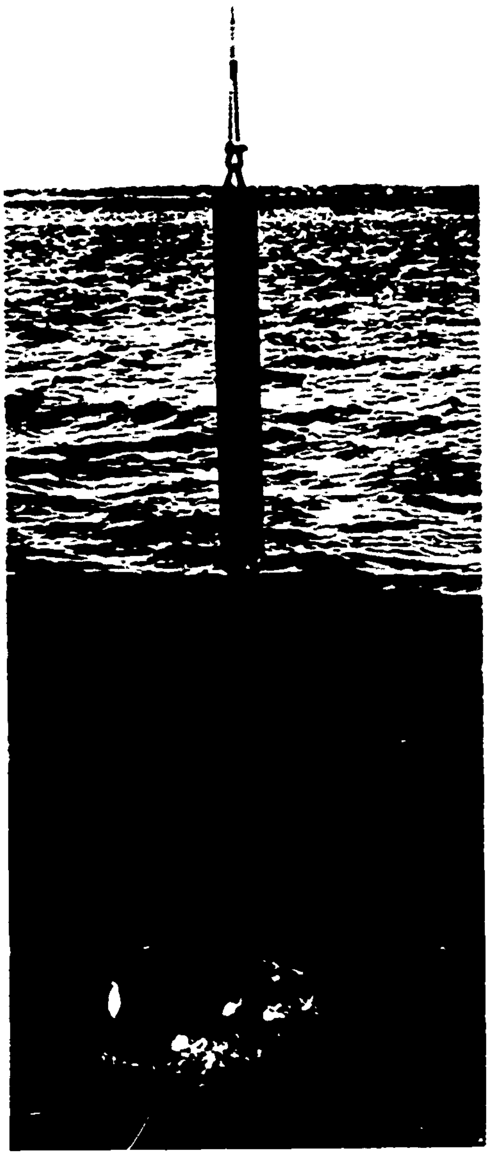


b. Block diagram of electronic components

(Courtesy of Professor A. F. Richards, Lehigh University)

Fig. 27. Transmission method of measuring gamma-ray density on Richards' γ-ray densitometer (from reference 94)

for comparison purposes have not been perfected.



(From Isotopes and Radiation Technology, Vol 6, No. 4, Summer 1969)

Fig. 28. Backscatter single-barrel γ-ray density probe (from reference 96)

In Situ Testing

93. In situ testing involves the placement of a testing device on the sea floor, applying known test conditions, detecting the response of the soil, and recording both the test conditions and the soil response. The intent of in situ testing is to assess the properties of soils in their natural environment. By so doing, the problems of sample disturbance associated with sampling and laboratory testing are bypassed. Results of in situ tests are also used for correlation with results from laboratory tests. The correlations obtained provide the engineer with a better understanding of the relationship of laboratory test results to the field conditions.

Test device systems

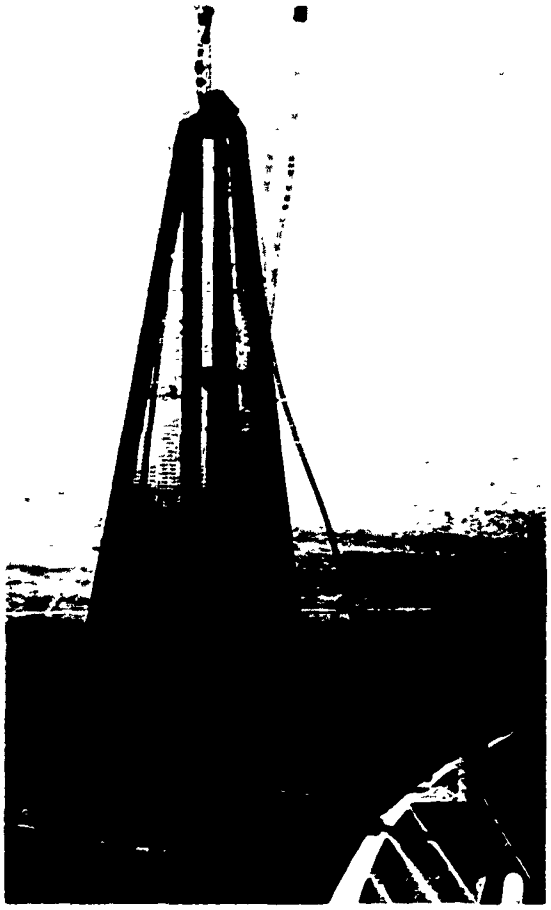
94. A variety of systems is used to house the test instruments. Some instruments, e.g., accelerometers, are attached to corers. For those tests requiring only a quick penetration into and withdrawal from the sea floor, a probe-type system tethered to a surface vessel and operated in a manner similar to that used in sampling is sufficient. The backscatter gamma-ray density, electrical resistivity, and pore pressure probes use such systems (figs. 28 and 29).[61,95-97] For those tests requiring

Fig. 29. Piezometer equipment designed to measure differential pore pressures. The instrumentation is housed in the tip of the probe. About 0.5 ton of lead weights is located above the piezometer (from reference 61)

(Courtesy of Norwegian Geotechnical Institute, Oslo, Norway)

observations of reactions to applied forces or sustained loadings, systems that can rest on the bottom facilitate the testing. These include the bottom-rest systems of Richards and Nacci which were discussed in paragraphs 43 and 44. Richards' system is capable of performing either vane shear tests or density tests by the gamma-ray transmission method (fig. 30) at desired vertical increments to a depth of 9 ft. This system is currently being modified to make tests down to 15 ft below the bottom. As noted earlier, these tests cannot be performed concurrently; the system must be raised and the test devices interchanged.

95. Nacci's system has acoustical probes for measuring in situ



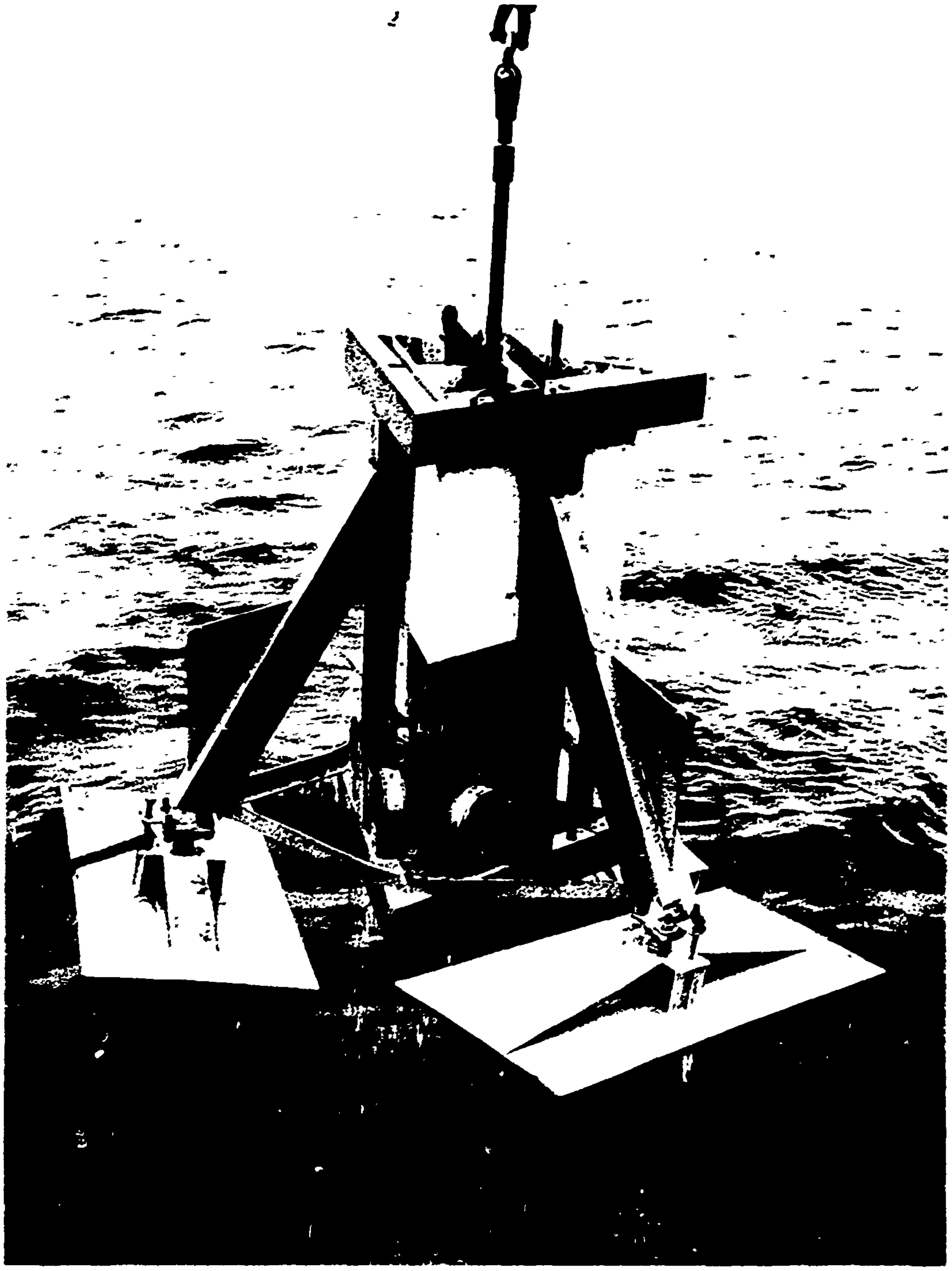
(Courtesy of Norwegian Geotechnical Institute, Oslo, Norway)

Fig. 31. Gamma-ray transmission equipment. The double-pronged probe contains a ^{137}Cs source in one part and a scintillation detector in the other part. The equipment on the platform above the probe includes a storage battery (protected against hydrostatic pressure), a DC to AC inverter, a relay-switching command module, and a container of electronic equipment (from reference 61)

sound velocities, and the probes are incrementally driven into the sea floor to a depth of 5 ft. Simultaneously, thermistors can be driven into the soil to obtain temperature readings at depth. Expanded capabilities for the system will provide 30-ft penetrations and will incorporate systems to measure electroresistivity, shear wave characteristics, gamma-ray density, and soil shear strength.[62]

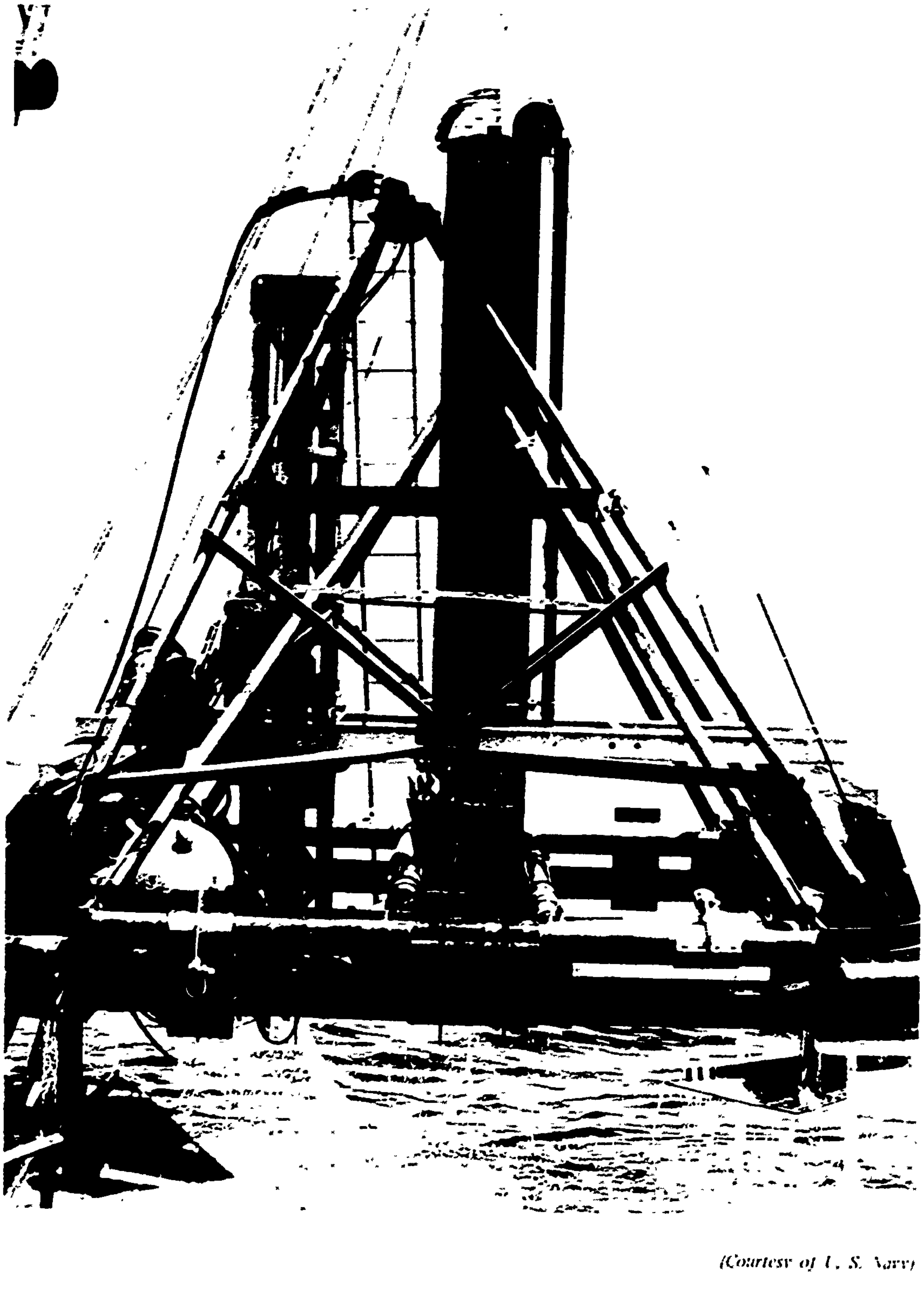
96. The U. S. Naval Civil Engineering Laboratory (NCEL) has a number of different in situ systems.[98] One of NCEL's earlier items was an in situ plate-bearing device for studying short-term behavior of foundation-type footings (fig. 31).[99,100] It has a tripodal arrangement with large bearing pads on the ends of the legs for support. The framework supports a movable weight whose movement is controlled by three closed-circuit, pressure-equalized hydraulic cylinders.

97. NCEL also has the Deep Ocean Test Instrument Placement and Observation System (DOTIPOS) (fig. 32) designed for a 6000-ft water depth. This system has already successfully completed intermediate proof tests at 1200-ft depths. This is a highly sophisticated multipurpose system which contains a closed-circuit television, a movie camera, and underwater lighting. Ten kilowatts of electrical power are



(Courtesy of U. S. Naval Civil Engineering Command, Port Hueneme, California)

Fig. 31. NCEL in situ plate-bearing device (from reference 99)

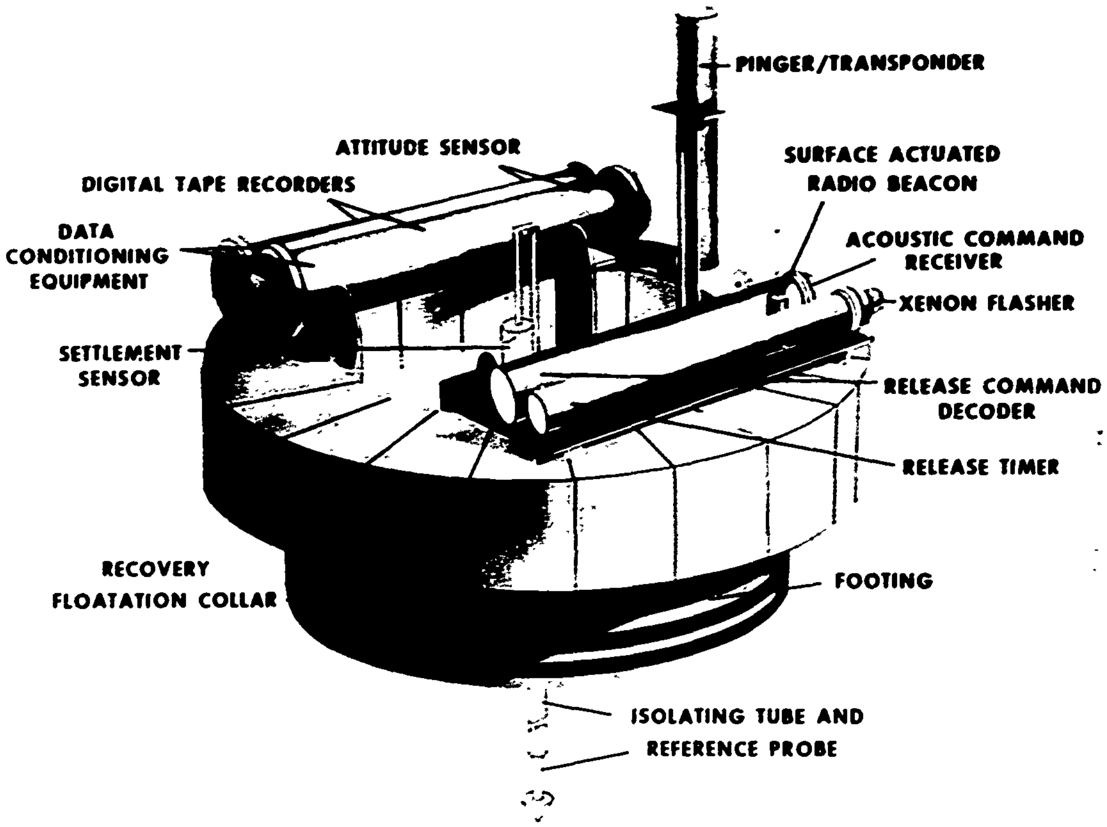


(Courtesy of U. S. Navy)

Fig. 5. The [illegible]

available at the platform to drive accessories, e.g., a vane shear device, a static cone penetrometer, or a sediment corer. Depth of penetration achievable is 10 ft.[98]

98. The DOTIPOS is also used for deploying other instrument packages on the sea floor. One such package is the Long-Term Ocean Bottom Settlement Test for Engineering Research (LOBSTER) (fig. 33) which is a 4-ft-diam footing used to study long-term behavior of footing-type foundations. It is designed for deployment down to depths of 6000 ft for durations of up to 400 days. The LOBSTER applies a stress of 100 psf to the sea floor. Proof tests in shallow water (depths to 120 ft) have been successful. The LOBSTER obtains data regularly on both total and differential settlement.

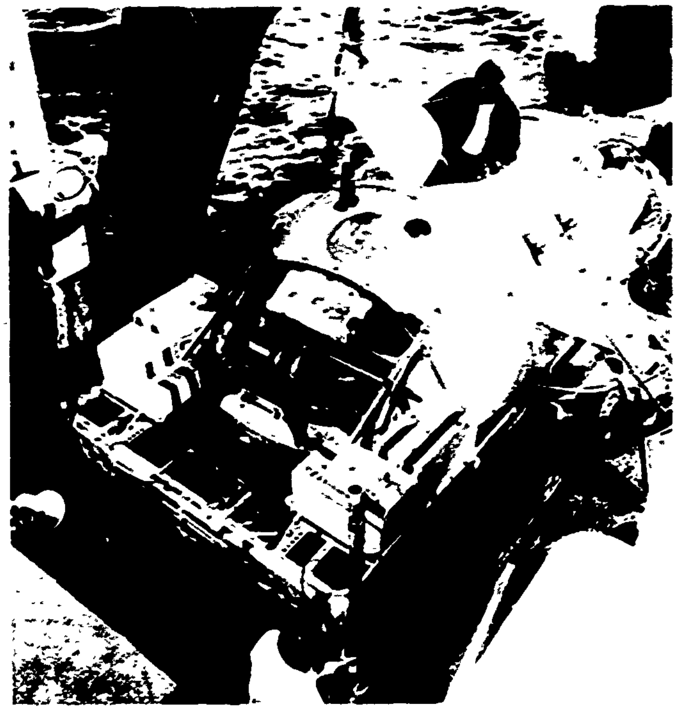


(From Ocean Industry, Vol 5, No. 5, May 1970)

Fig. 33. LOBSTER designed to study long-term footing foundation behavior at shallow-water or deep ocean and undersea construction sites (from reference 98)

99. It should be noted here that NCEL's efforts are directed toward achieving operational capabilities at a 6000-ft water depth. Proof testing of all equipment is conducted at various staging depths from shallow water to increasingly deeper waters.

100. Systems other than those described herein can also be used for obtaining in situ information on marine sediments. The U. S. Naval Undersea Research and Development Center has successfully measured acoustical properties of sea floor surface materials with the aid of the Westinghouse Deepstar 4000 submersible, using instrument packages mounted on a frame protruding from its bow end (fig. 34). Bottom crawlers and robot systems are other suggested possibilities.[101] Testing from the sea surface with a fixed or anchored base is also used.[102] This may be by means of oil company jack-up or fixed platforms when they are conveniently on location.[64] In shallow waters, divers may be used to perform some of the in situ testing.



(Courtesy of Naval Undersea Research and Development Center, San Diego, California)

Fig. 34. The research submersible DEEPSTAR on the afterdeck of R/V SEARCHTIDE during preparations for a dive. The compressional velocity-attenuation probes are attached to a "brow" on the front end of the vehicle. At the lower center of the figure, a clear, plastic-tube corer is in the retracted position (from reference 17)

101. Data acquisition, transmission, recording, storage, etc., components of in situ testing systems are either completely aboard the surface vessels, entirely contained in the bottom structure, or combinations of these. Power sources (motors and batteries) are generally pressure-compensated, e.g., motors

are immersed in oil in housings with a Plexiglas or other flexible closure that permits equalization of internal pressures with external ambient pressures. Any electronic circuitry at the bottom is usually pressure-protected, e.g., relay systems are placed in housings designed sufficiently strong and tight to withstand external pressures at design depths.

102. Direct transmission of all data acquisition to the command vessel has the advantage that the test can be monitored while in progress and remedial action taken as needed. Richards and Nacci's systems use this mode of transmission. The disadvantage is that electrical cable from the vessel to the bottom is needed. Accumulation of data at the bottom may be accomplished by a variety of storage systems (e.g., magnetic tape), and has the advantage that no connection to the vessel is necessary except the tether line needed for lowering and raising the instrument package. Even this can be eliminated for long-term tests. For example, the LOBSTER is completely self-sufficient, data being recorded on magnetic tape at the rate of once every 5 sec to seven times per hour, with a data storage capacity of 400 days on the sea floor. A disadvantage of the totally independent system is that malfunctions may not be detected until the instrument package has been retrieved. Start and stop commands can be initiated by transmission via a cable, by remote control, by a timer setup, or by contact switches.

103. Direct data transmissions to the command vessel can be stored on magnetic tape for later detailed review and concurrently displayed on continuous-strip charts, X-Y recorders, oscillographs, and other observational aids. Alternatively, to cut down on the number of receiving channels on the vessel, storage can be accomplished on the bottom platform, while just sufficient channels to the vessel are operated to permit monitoring of the bottom activities.

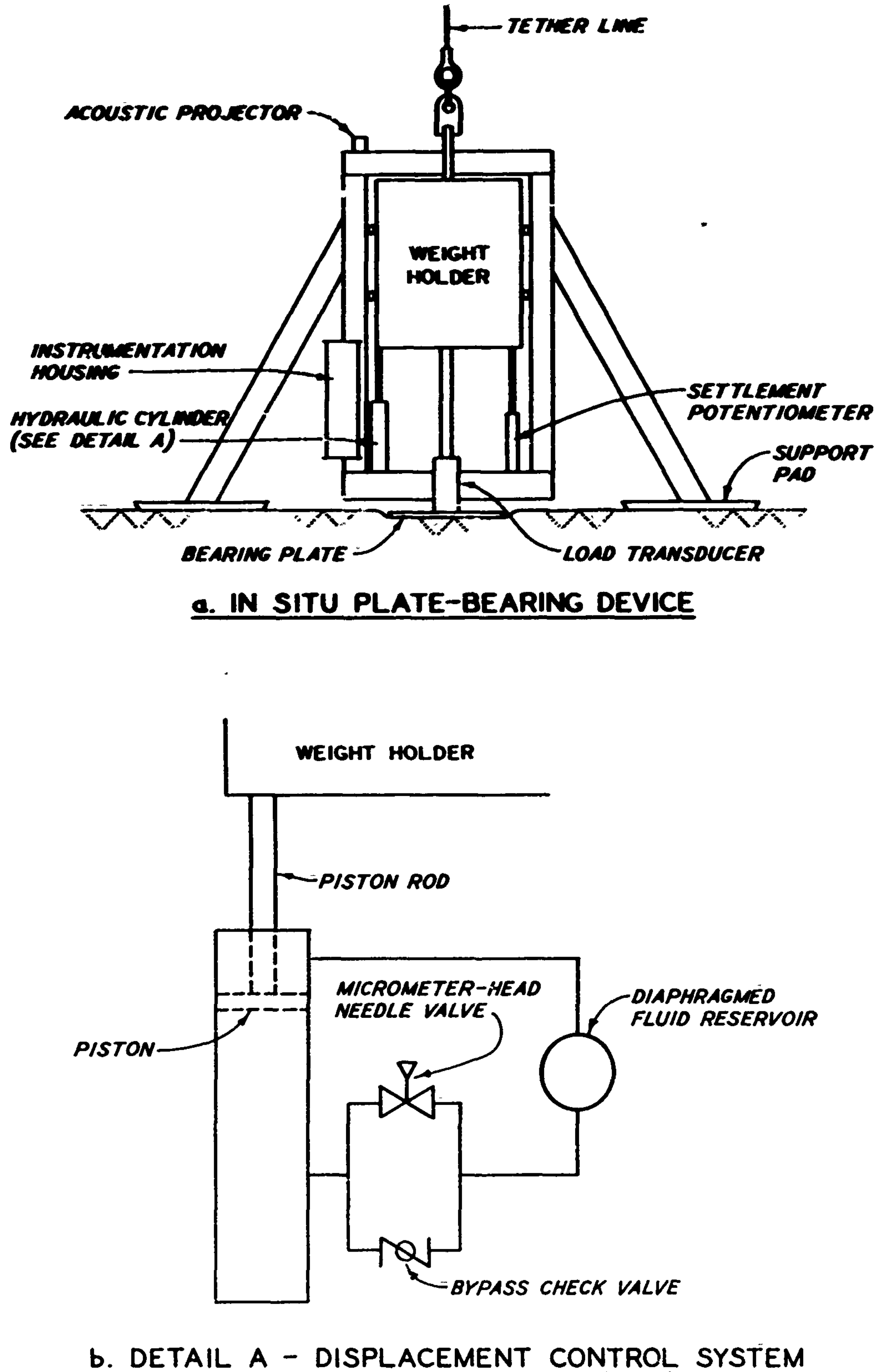
104. In conclusion, all modes of data acquisition are used. The choice of combinations depends on duration time of the instrument's stay on the bottom, depth of water and line lengths, subsequent use of accumulated data, and economics of the system.

Plate bearing

105. The plate-bearing test is used to determine empirically the bearing capacity of a soil as related to size and shape of the bearing surface and the loads applied to the bearing plate. On-shore procedures are generally load-control tests in which increments of load are added after deformations under previous increments have occurred. Kretschmer and Lee[100] considered it easier in a sea floor operation to preset the settlement rate and record the loading necessary to cause this settlement rate than to preset the load and record the settlement. The NCEL plate-bearing system operates in this manner. Fig. 35a shows a schematic of the NCEL system. The vertical movement of the weight holder and, therefore, the movement of the bearing plate are controlled by the flow of hydraulic fluid in the three hydraulic cylinders that support the weight holder. A schematic of the control system for one of the cylinders is shown in fig. 35b. On the downstroke of the piston, fluid flows from the underside to the topside of the piston through a micrometer-head needle valve and a diaphragmed fluid reservoir. The needle valve is preset at the surface for a particular flow rate, and thus controls the settlement rate of the weight holder and the bearing plate. The diaphragmed fluid reservoir, a pressure-compensating housing with a flexible membrane wall, compensates for and nullifies the effect of the large external pressures. On the upstroke of the piston to withdraw the bearing plate from the soil after each test, the fluid flows from the topside to the underside of the piston through the diaphragmed reservoir and the bypass check valve. The settlement potentiometer records the actual settlement, and the load transducer records the actual load required to push the bearing plate into the sea floor at the preset settlement rate. Load and settlement of the bearing plate and the vertical orientation of the device are transmitted acoustically to a hydrophone on the support ship. Plates ranging in size up to 1.5 ft in diameter can be loaded up to 6000 lb.

Field vane shear

106. The discussion of laboratory vane shear testing in paragraph 85 is pertinent to in situ vane shear testing. A characteristic



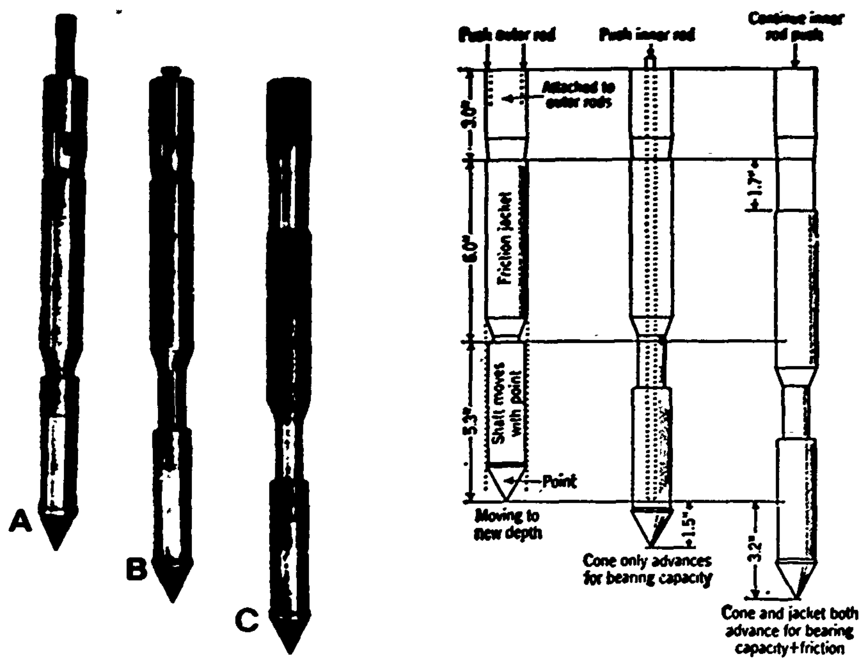
(Courtesy of American Society of Civil Engineers)

Fig. 35. NCEL plate-bearing and displacement control systems (from reference 100)

problem of field vane tests is accounting for the rod resistance during a test. Torque is generally measured at the top of the rod and thus includes soil resistance to rotation of the rod as well as that of the vane. This becomes more significant as the length of rod increases.[102] To minimize this, Richards' system allows an initial 50-deg rotation of the vane rod, independently of the vane, to permit calibration of rod resistance and eliminate it from the shear strength calculation.[61] Care must be taken to ensure that the soil in the test zone is not disturbed during the advance of the vane shear device.

Dutch friction cone penetrometer

107. The Dutch friction cone penetrometer (fig. 36) is not often used in the U. S. but is in Europe. McClelland Engineers personnel



a. Positions of penetrometer: (a) cone and friction sleeve retracted; (b) cone in extended position; (c) cone and friction sleeve both advanced (from reference 103)

b. Action of the friction cone (from reference 104)

(Courtesy of American Society of Civil Engineers)

Fig. 36. Dutch friction cone penetrometer

have indicated that they were impressed with its performance in the North Sea area. The cone penetrometer has been used on shore in the Netherlands for approximately 30 years and appears to be well-suited for pile foundation investigations.[105,106] The Dutch device measures the load required to push (at a predetermined penetration rate) (a) a cone-tipped probe into the soil, and (b) the cone plus a cylindrical sleeve. The load using the cone and sleeve is indicative of end bearing and skin friction. The difference in the two values is a measure of the frictional resistance of the soil.

108. WES has developed a cone penetrometer for determining the trafficability characteristics of surface soils (fig. 37). The WES cone penetrometer measures only the force required to push a cone-shaped probe into the soil; no friction sleeve is involved, but the shaft is smaller in diameter than the top of the cone to reduce the effects of friction during the test. In a WES study of sea floor trafficability,[101] Wiendieck concluded that the WES penetrometer would need to be modified for marine tests because of the very low strength of many sea floor soils. He further concluded that there is a need for a free-falling or driven instrumented sea floor-probing device (penetrometer) for the rational evaluation of large sea floor regions.

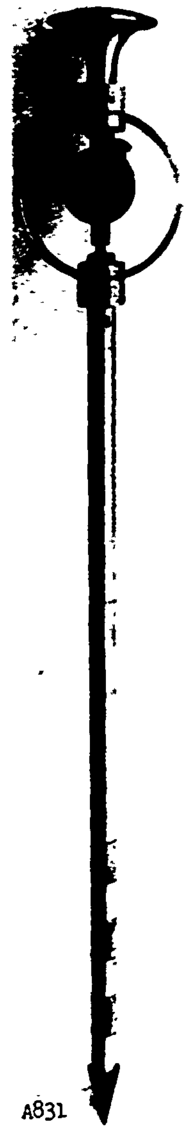


Fig. 37. Standard WES hand-operated cone penetrometer

Accelerometer

109. Accelerometers for measuring acceleration or deceleration of an object falling through the water and then penetrating into the sea floor have been used experimentally on the ocean floor.[107,108] The accelerometer might be attached to a sampling device or other penetration device. When used with a corer, the record obtained could also indicate whether the corer has malfunctioned. For example, the record could indicate if the corer entered at an angle and only partially penetrated the sea

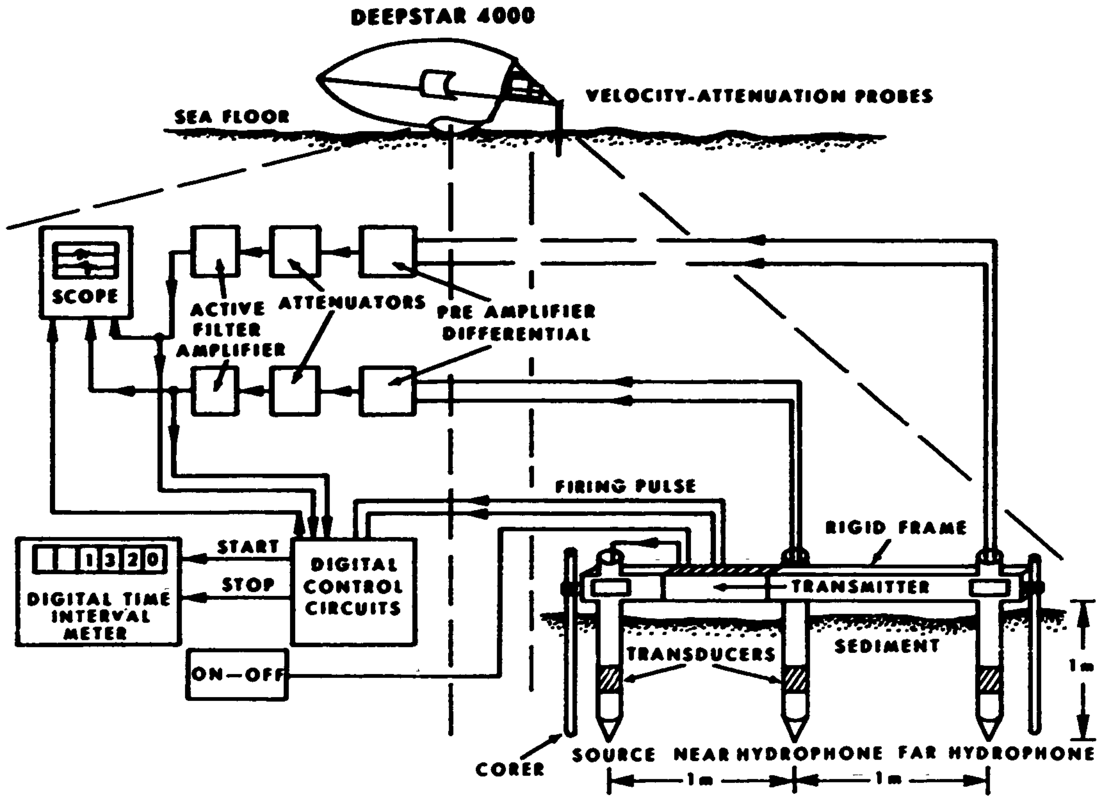
floor. By successive integration of the acceleration-time curve, a shear resistance versus depth curve can be plotted and evaluated. Consideration must be given to (a) the effect of cable motions due to stretching caused by the weight of the corer, (b) cable oscillation upon release of the corer for free-fall, and (c) proper inclusion of items a and b in force-displacement equations. The accelerometer appears to be potentially useful with any free-fall sampling arrangement. The information derived from the accelerometer may give a measure of the in situ strength of the soil.

Devices for determining porosity and density

110. The ability of a soil to reflect or transmit gamma rays, electric currents, or acoustic waves depends in part on the density of the soil. Related properties of porosity and void ratio have been correlated with gamma-ray and acoustic wave reflections or transmissions. Gamma-ray detection can be accomplished either by the backscatter (reflection) or transmission method.[109] The backscatter system counts the intensity of rebounding radioactivity as sensed by a detector on the same plane as the source at 90 deg from the source beam. The direct transmission system counts the intensity of radioactivity a set distance directly in front of and in line with the source beam from the radioactive source. The single probe units of the Coastal Engineering Research Center (formerly Beach Erosion Board) and the one shown in fig. 28 operate on the backscatter principle.[96,100] The transmission unit uses dual probes, one source and one detector (see fig. 30).

111. The in situ measurement of velocity of propagation of sound, either compressional or shear wave, is accomplished with multiprobe arrangements. Fig. 18 shows the DOSP with the sound source and three detectors. The separations between source and detectors are 4 ft on the sides and 6 ft on the diagonal.

112. The U. S. Naval Undersea Research and Development Center has used a three-probe (one sound source probe and two hydrophone probes) acoustical device attached to the front of the submersible Deepstar 4000. A schematic of the components involved is shown in fig. 38. The three



(Courtesy of Naval Undersea Research and Development Center, San Diego, California)

Fig. 38. Components in the compressional velocity-attenuation probe equipment, and placement of probes in the sea floor by DEEPSTAR (from reference 17)

probes attached to a rigid frame, are spaced 1 m apart, and can penetrate to a depth of 1 m. Core tubes can be attached to both ends of the rigid frame to permit simultaneous soil sampling and acoustical velocity attenuation tests. The block diagrams in fig. 38 represent the electronic components required for the firing of the sound source and the amplification and recording of the sound transmitted through the soil and detected by the hydrophones. This system can measure sound attenuation of three different frequencies (3.5, 7, and 14 kHz) without removing the probes from the sea floor to change frequencies of the source, thus ensuring that all measurements were made in the same soil.[17]

113. In situ tests alone cannot adequately meet the soils

engineer's need for knowledge of sea floor characteristics. Some in situ tests are expensive, lack flexibility, and yield results which are often difficult to analyze.[100] A comprehensive exploration program should include both in situ and laboratory testing.

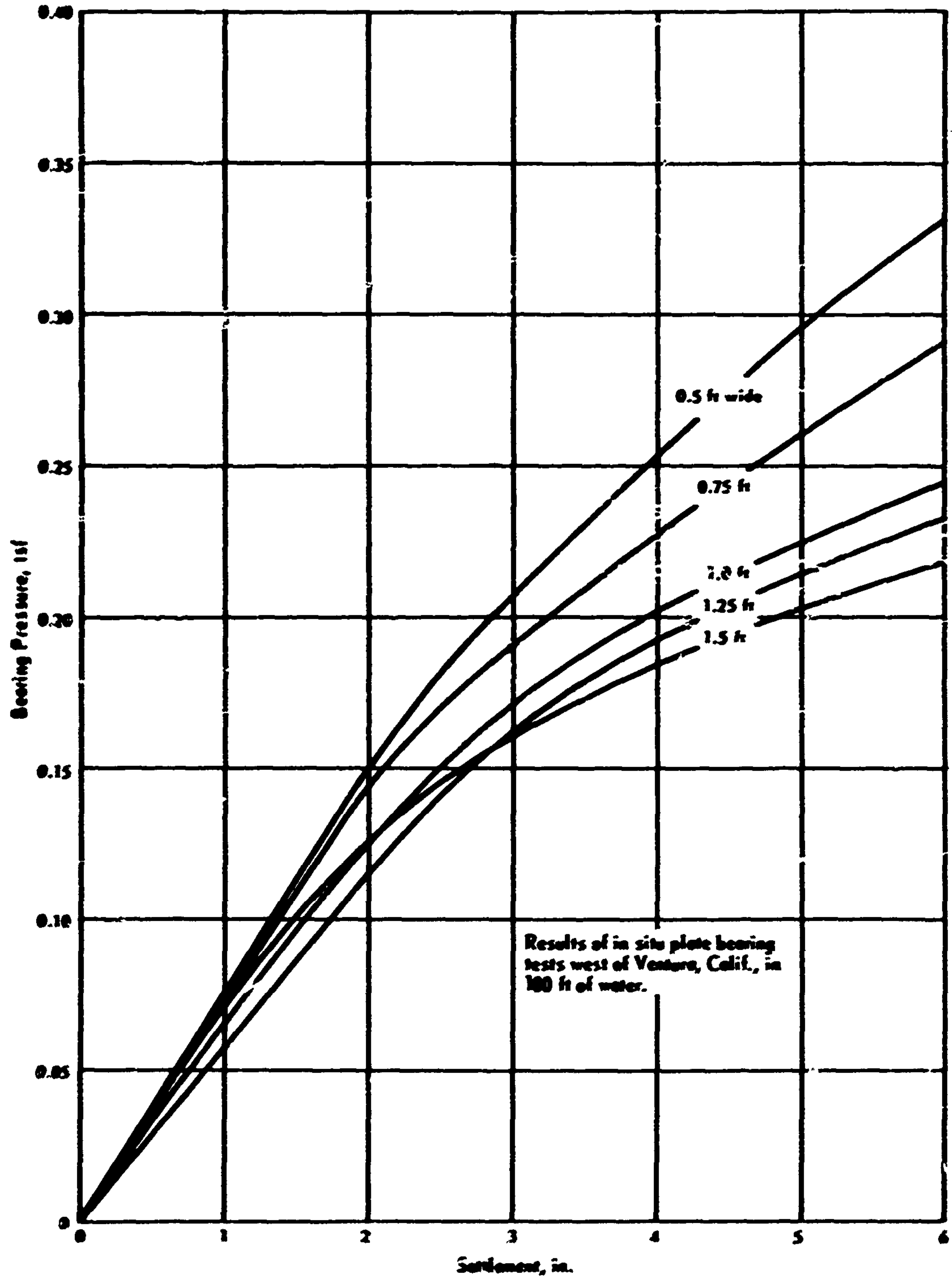
PART IV: PROPERTIES OF MARINE SOILS

114. Because of the dearth of soils data on continental shelf soils and because of the uncertainty as to the validity of data on "undisturbed" soils owing to lack of sufficient detail on sampling tools and procedures and test methods employed, only a broad view of the characteristics of continental shelf soils can be presented.

Off California Coast

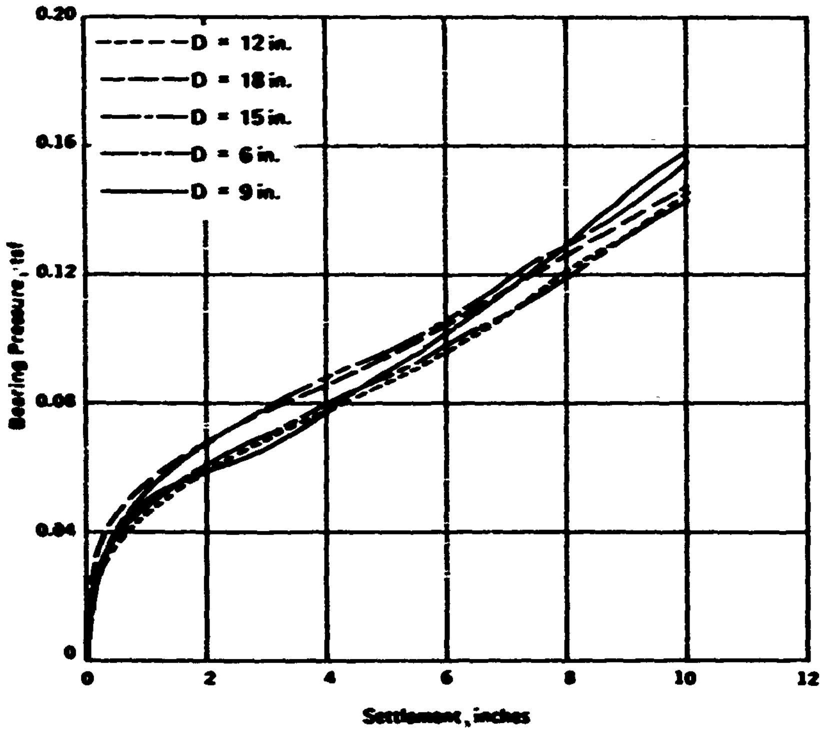
115. While much data on engineering properties of soils undoubtedly have been obtained in connection with offshore petroleum drilling operations, they have not been published in the literature, and may not be publicized for some time in the future for proprietary reasons. Consequently, the only data available are those obtained by the Naval Civil Engineering Laboratory and by others in oceanographic studies, and these data pertain to samples from less than 5 feet below the ocean bed. Table 10 shows the locations from which samples were taken in various investigations and indicates the types of tests performed. The soils sampled were largely silts. No test values are presented in table 10 for various reasons. In some cases, reported values for various parameters were inconsistent with each other; in other cases, the tests performed were inappropriate for the type of material being tested. In general, there was a lack of details on which to judge the quality of "undisturbed" samples tested and a lack of details on the test procedures used.

116. The U. S. Naval Civil Engineering Laboratory performed in situ plate-bearing tests at sites west of Ventura and Port Hueneme, Calif.[99,100,111] The in situ plate-bearing device and its operation are discussed in paragraphs 96 and 97. The nine bearing plates used included five circular plates of different diameters and four square plates of different sizes; settlement rates ranged from 0.005 to 0.1 in./sec. Test results indicated that (a) neither plate shape (circular or square) nor settlement rates significantly influenced the bearing pressure-settlement curves for soils at either site, and (b) a settlement increase occurred with increased plate size under the same pressures at the site west of Ventura (fig. 39), but plate size had no significant effect on bearing



(Courtesy of U. S. Naval Civil Engineering Command, Port Hueneme, Calif.)

Fig. 39. Bearing pressure versus settlement for various plate sizes for in situ plate-bearing tests west of Ventura, Calif. (from reference 99)



(Courtesy of U. S. Naval Civil Engineering Command, Port Hueneme, Calif.)

Fig. 40. Bearing pressure versus settlement for various plate sizes for in situ plate-bearing tests west of Port Hueneme, Calif., in 1175 ft of water (from reference 111)

pressure-settlement characteristics at the site west of Port Hueneme (fig. 40). Kretschmer and Lee suggested that the difference in the influences of plate size on bearing pressures at the two sites may be attributed to the differences in the soils at the two sites.[111] A comparison of the soil properties for the top 1.5 ft of soil at each site, given in table 11, indicates that this suggestion is reasonable. The soils from west of Port Hueneme have much higher (a) percent of particles finer than about . microns, (b) liquid limits, (c) plasticity indexes, and (d) water contents than the soils from west of Ventura.

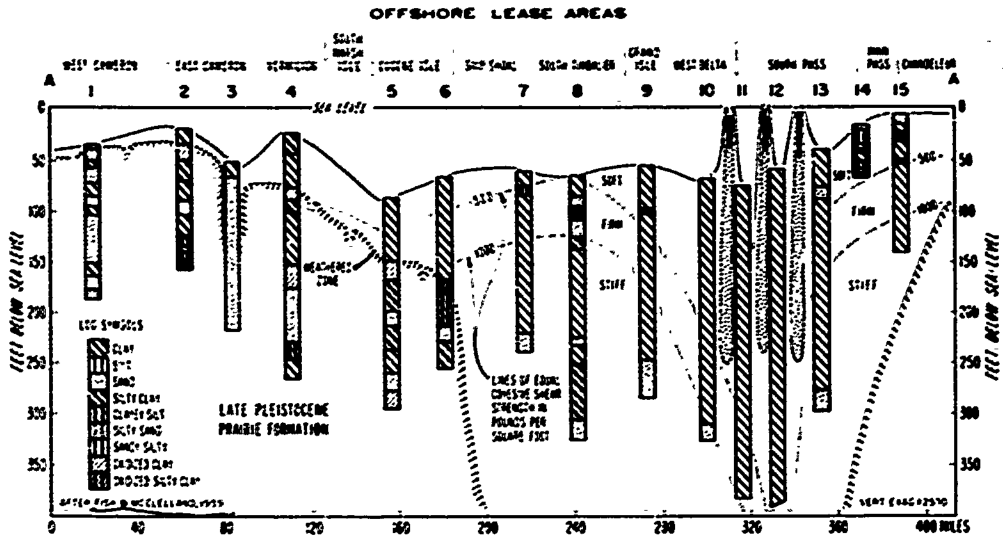
Gulf of Mexico

117. Because of oil company activities in the Gulf of Mexico continental shelf of the U. S. over the past two and a half decades,

more is known about this area than any other U. S. shelf zone. Much of this knowledge is proprietary and hence unavailable for public release. In addition, the information released into the literature is often five or more years old before the owners allow publication of the findings.

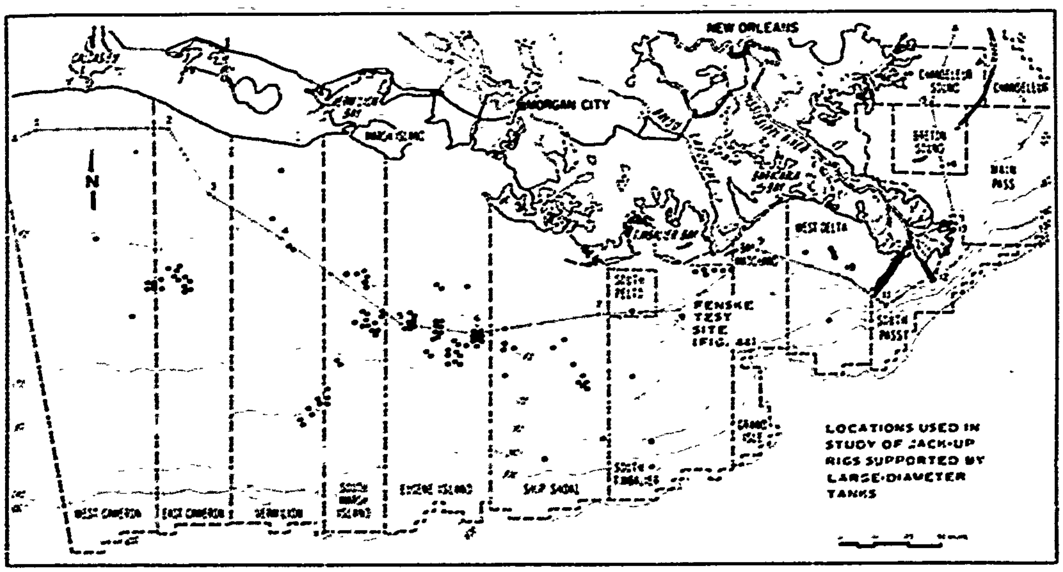
118. In the western Gulf of Mexico off Mexico and Texas, montmorillonite-rich silty clays predominate with median grain diameters between 2 and 4 microns (8 and 10 phi).[114] Laboratory vane shear strengths determined on specimens obtained with piston samplers using a motorized vane shear device operated at a constant rotation of 6 deg/sec ranged from 0.009 to 0.175 tsf for core depths of 0 to about 20 ft. One strength profile for soils from off the Texas coast showed shear strengths ranging from 18 to 170 psf, increasing nonuniformly from the mudline to 18 ft below and indicating the sediments to be very soft. Using an Anteus back-pressure consolidometer with back pressure equal to the in situ hydrostatic pressure, Bryant, Cernock, and Morelock found that the sediments on this shelf were generally slightly overconsolidated.[114] Very low-strength, overconsolidated, marine soils are not uncommon (unlike overconsolidated terrestrial soils, which usually exhibit high shear strengths because of much greater magnitude of preconsolidation pressures).

119. Clay is the predominate soil type on the continental shelf off the Louisiana coast due to its predominance in sediment transport by the Mississippi River.[115,116] The mineral composition of these clays, in order of decreasing abundance, is montmorillonite, illite, and quartz.[117] Natural moisture contents of the deposits generally lie between the liquid limit and plasticity limit except locally in some of the clays in the vicinity of the mudline where the natural moisture content exceeds the liquid limit.[91,102,117] A shear strength profile for an area offshore of and parallel to the Louisiana Coast is shown in fig. 41, with lines of equal shear strength. The locations of the 15 holes from which the profile was extrapolated are shown in the map of the area in fig. 42. Fig. 43 shows the plasticity chart for the soils from four of these boreholes and from Fenske's data,[102] all to be discussed in more detail in the following paragraphs.



(Courtesy of American Society of Civil Engineers)

Fig. 41. Shear strength profile (from reference 118). (See fig. 42 for locations of borings)



(Courtesy of American Society of Civil Engineers)

Fig. 42. Locations of 15 borings from which data were obtained to produce extrapolated profile in fig. 41. (modified from reference 118)

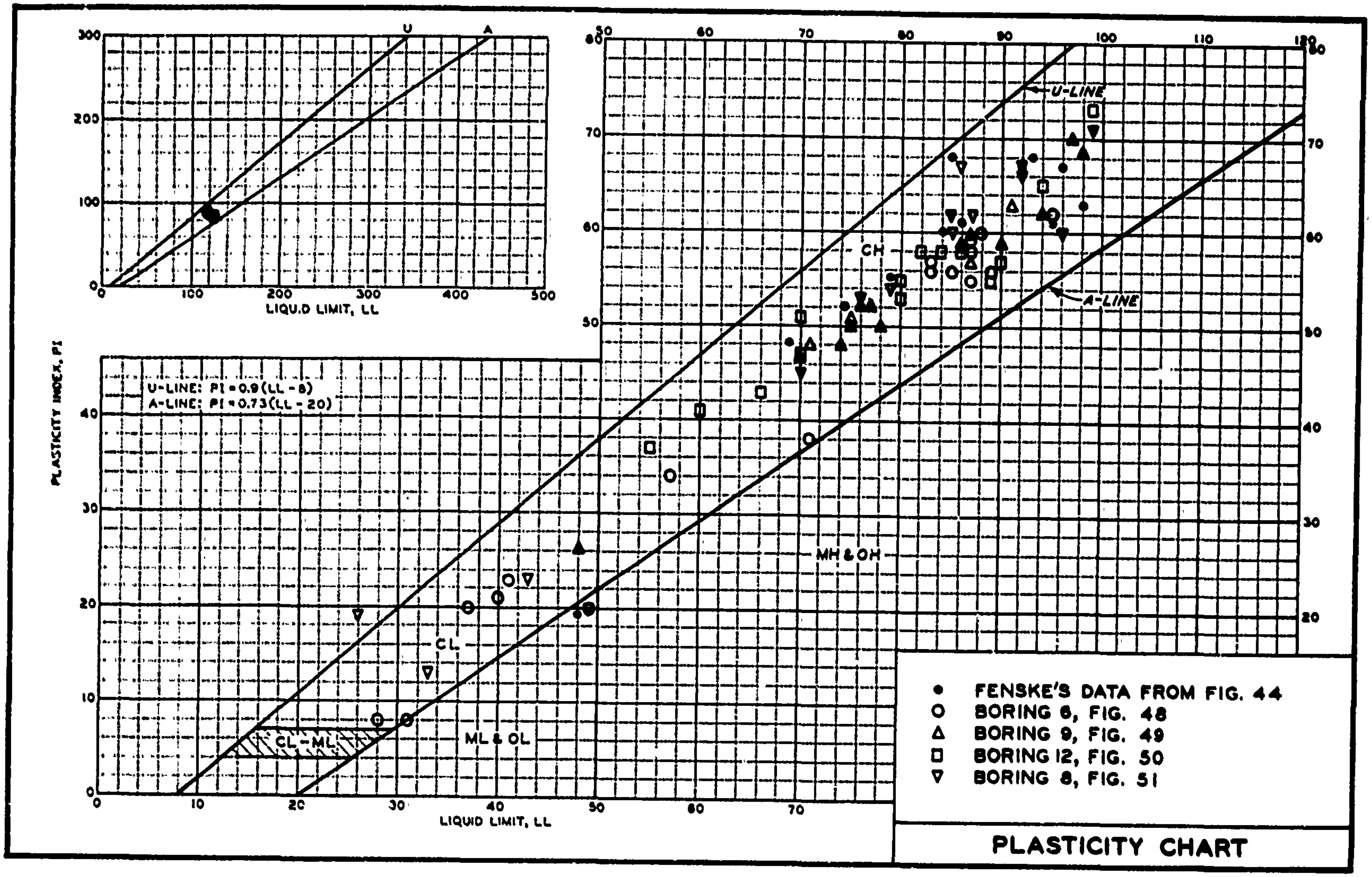
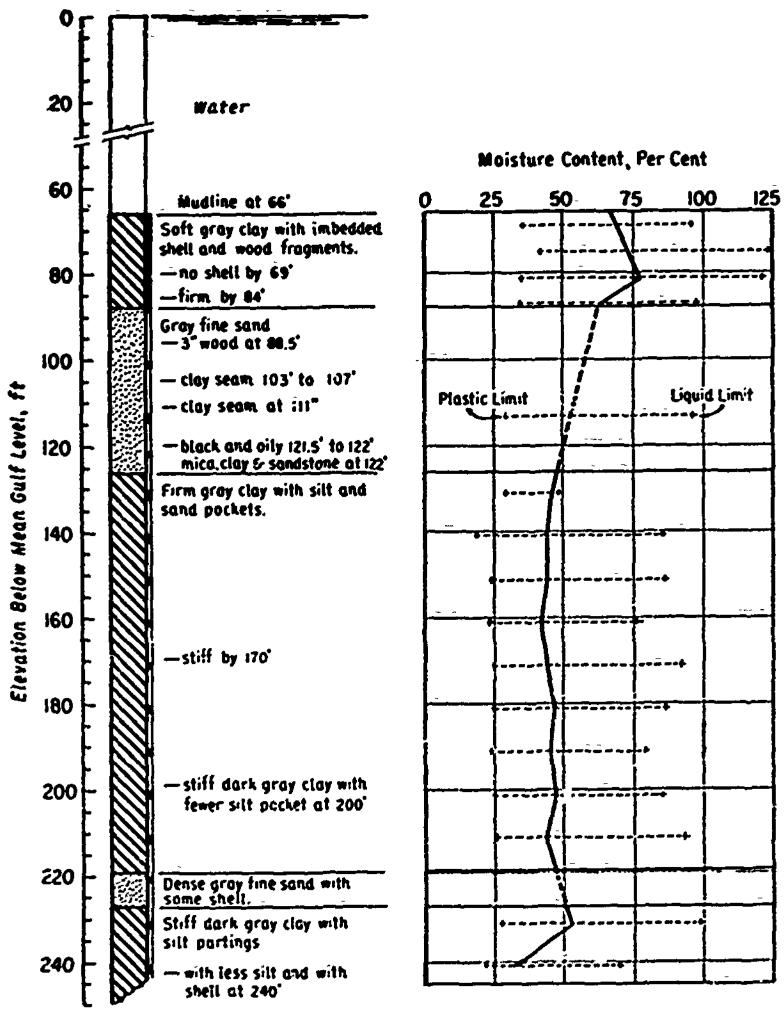


Fig. 43. Plasticity chart for soils from off Louisiana coast

120. In 1955, in situ vane tests were performed 15 miles off the Louisiana coast in 66 ft of water to a depth of 175 ft below the sea floor at the location shown in fig. 42.[102] Undisturbed 3-in. Shelby tube samples were taken in a boring 9 ft from the vane test site to a depth of 254 ft below the sea floor to determine soil conditions. A laboratory vane shear test was performed as soon as each sample was recovered before its removal from the sampler. Laboratory shear strengths were determined on undisturbed and remolded specimens by unconfined compression and consolidated-undrained triaxial testing. Fig. 44 shows the



(Courtesy of American Society for Testing and Materials)

Fig. 44. Soil conditions at Fenske's deep vane test site (from reference 102)

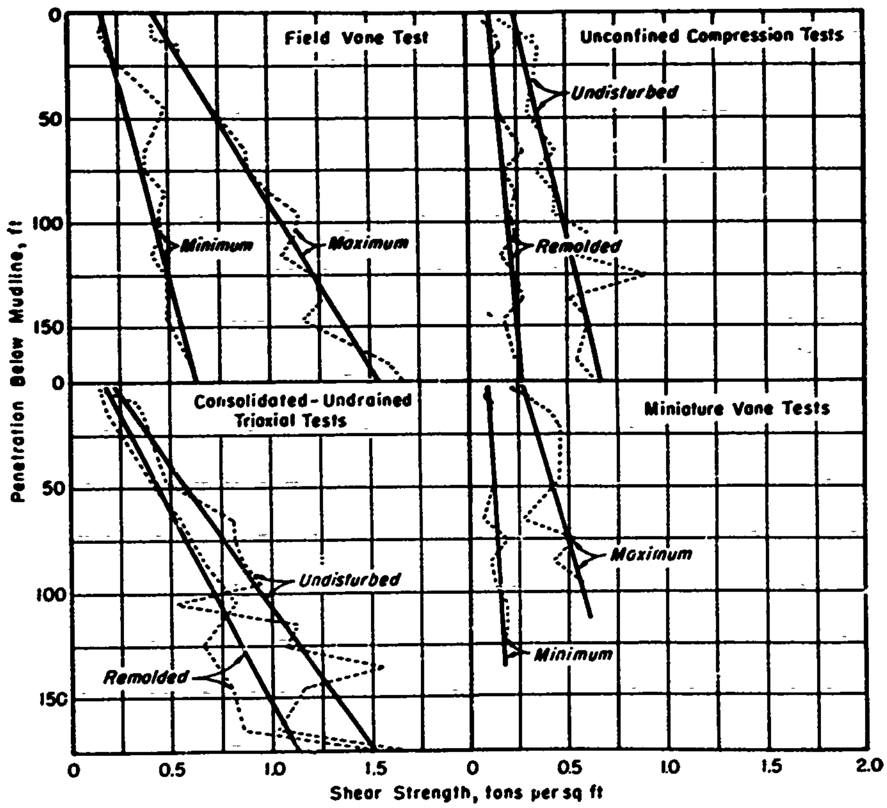
boring log and identification test results. The water content of these soils generally decreases with depth and lies between the plastic limit and the liquid limit.

121. In situ vane shear tests were performed using a 3-in.-diam by 6-in.-long (H/D ratio of 2) four-bladed vane rotated at a rate of 0.2 deg/sec. In situ vane test strengths were compared with strengths obtained from (a) laboratory vane (0.685-in.-long by 0.760-in.-diam) tests performed on the Shelby tube samples, (b) unconfined compression tests, and (c) consolidated-undrained triaxial tests in which the confining pressure was equal to the computed effective overburden pressure at the depth from which each sample was taken. Tests on both undisturbed and remolded material were performed. Fig. 45a shows the test results. Very shallow soils at this location are indicated to be substantially stronger than those off the coast of southern California. Strengths also tend to increase with depth.

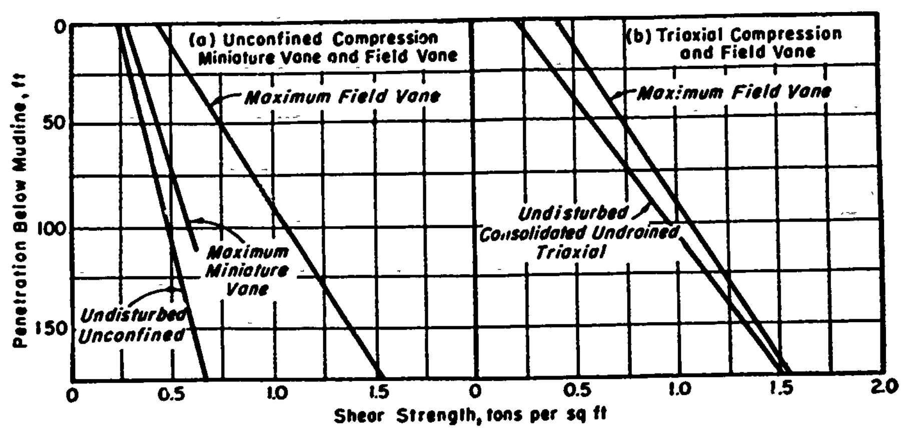
122. Fig. 45b compares the maximum field vane results with the undisturbed consolidated-undrained triaxial, maximum miniature vane, and undisturbed unconfined test results. According to Taylor,[119] strengths determined using consolidated-undrained triaxial compression tests should approximate in situ strengths more closely than those obtained using other types of tests. The undisturbed consolidated-undrained triaxial test results closely approximated the in situ maximum field vane test results. Relative strengths (in descending order) obtained using the various test procedures were (a) those obtained using maximum field vane (peak) tests, (b) those obtained using undisturbed consolidated-undrained triaxial compression tests, (c) those obtained using maximum laboratory vane (peak) tests, and (d) those obtained using undisturbed unconfined compression tests.

123. The sensitivity ratio was about 2.5 based on the field vane test results and slightly less than 2.5 based on the unconfined compression test results, indicating that the clays at this site are relatively insensitive.

124. Unconfined compressive strengths, q_u, ranged from about 0.5 tsf at the mudline to a maximum of about 1.3 tsf at a depth of



a. Plot of test data



b. Comparison of field vane and laboratory test results

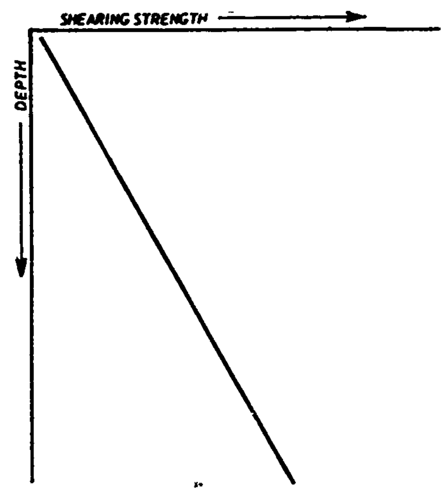
(Courtesy of American Society for Testing and Materials)

Fig. 45. Results of Fenske's 1955 deep vane tests off coast of Louisiana (from reference 102)

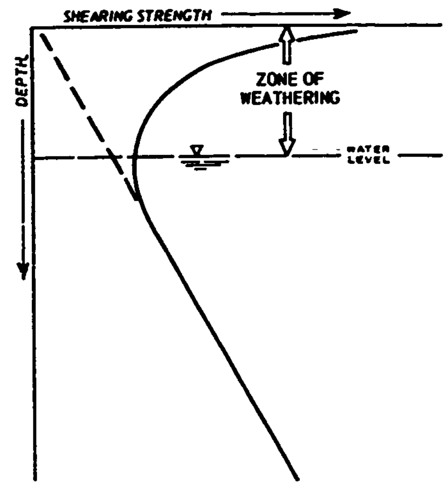
175 ft; thus, the soils vary in consistency from soft at the mudline to stiff at depth.

125. The eustatic rise and fall of sea level and its association with the discharge of the Mississippi River during the late Quaternary has controlled the depositional and erosional processes off the coast of Louisiana. This has affected the engineering properties and produced recognizable patterns in the sediments which can be used to predict local foundation conditions.[117] The effects of geologic processes on the strength characteristics of sediments are shown in fig. 46. Fig. 46a is an idealized shear strength-depth relation for a normally consolidated clay deposit, i.e., one in which the rates of deposition and consolidation have been sufficiently compatible that the soil at all depths has fully adjusted to the pressure of the overlying soil, and in which the deposit has no previous history of having been subjected to greater overburden or consolidation stresses than currently exist. The strength at zero depth is cohesion due to molecular interaction between solid particles at their points of contact; for clays, minimum cohesion is 0.05 tsf.[120] Figs. 46b, c, and d show the effects of weathering, erosion, and rapid deposition, respectively, on shear strength of a clay deposit. The strength-depth relations in figs. 46c and d will, of course, change with time.

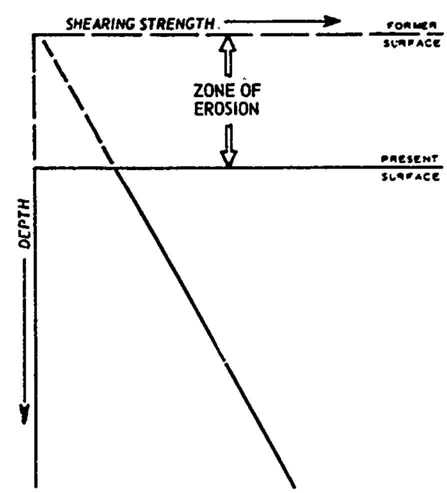
126. The ratio of shear strength (cohesion, c) to effective overburden pressure, p, (termed c/p ratio) is frequently used to define the variations of strength of a saturated clay deposit with depth. Terzaghi has noted that this ratio is independent of depth for a normally consolidated clay.[120] Both Skempton and Bjerrum have plotted, for normally consolidated clays, (see fig. 47) the c/p ratio versus the plasticity index, PI (which in turn reflects the type and quantity of clay particles in the deposit).[85,88,121] Bjerrum fitted a curve to his data, and Skempton fitted a straight line to his. In a qualitative sense, points plotted considerably above these lines indicate overconsolidated clays, and those plotted considerably below indicate underconsolidated clays.



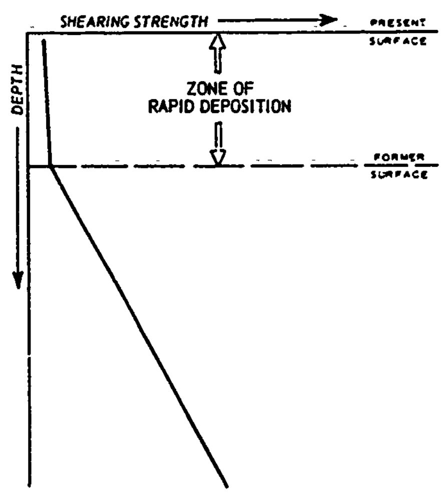
a. Typical strength profile for normally consolidated clay



b. Strength profile a. modified by desiccation and hardening of sediment elevated above water table



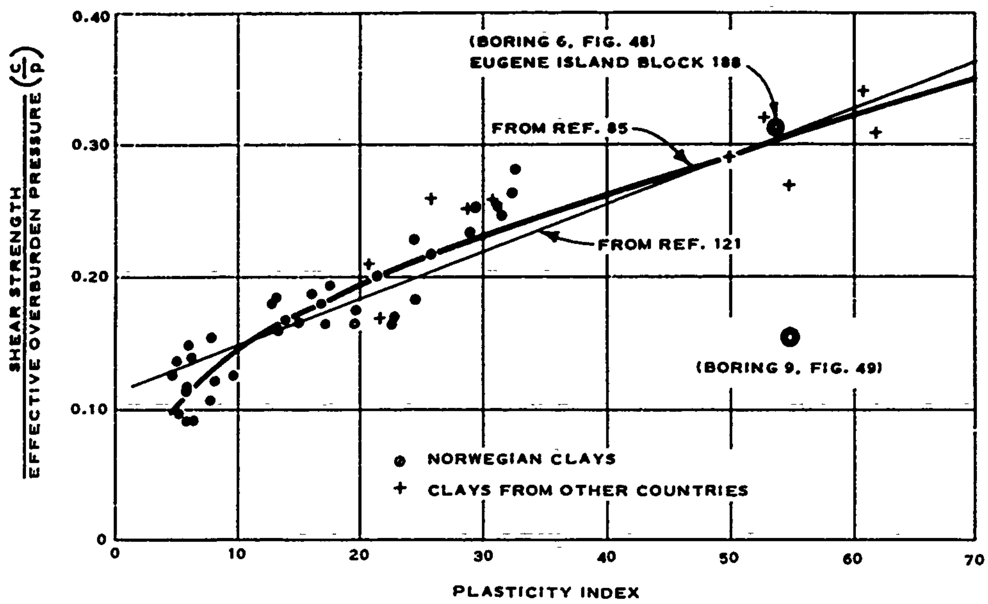
c. Strength profile a. modified by erosion of softer surface sediment



d. Strength profile a. modified by rapid deposition of new sediment

(Courtesy of The Geological Society of America)

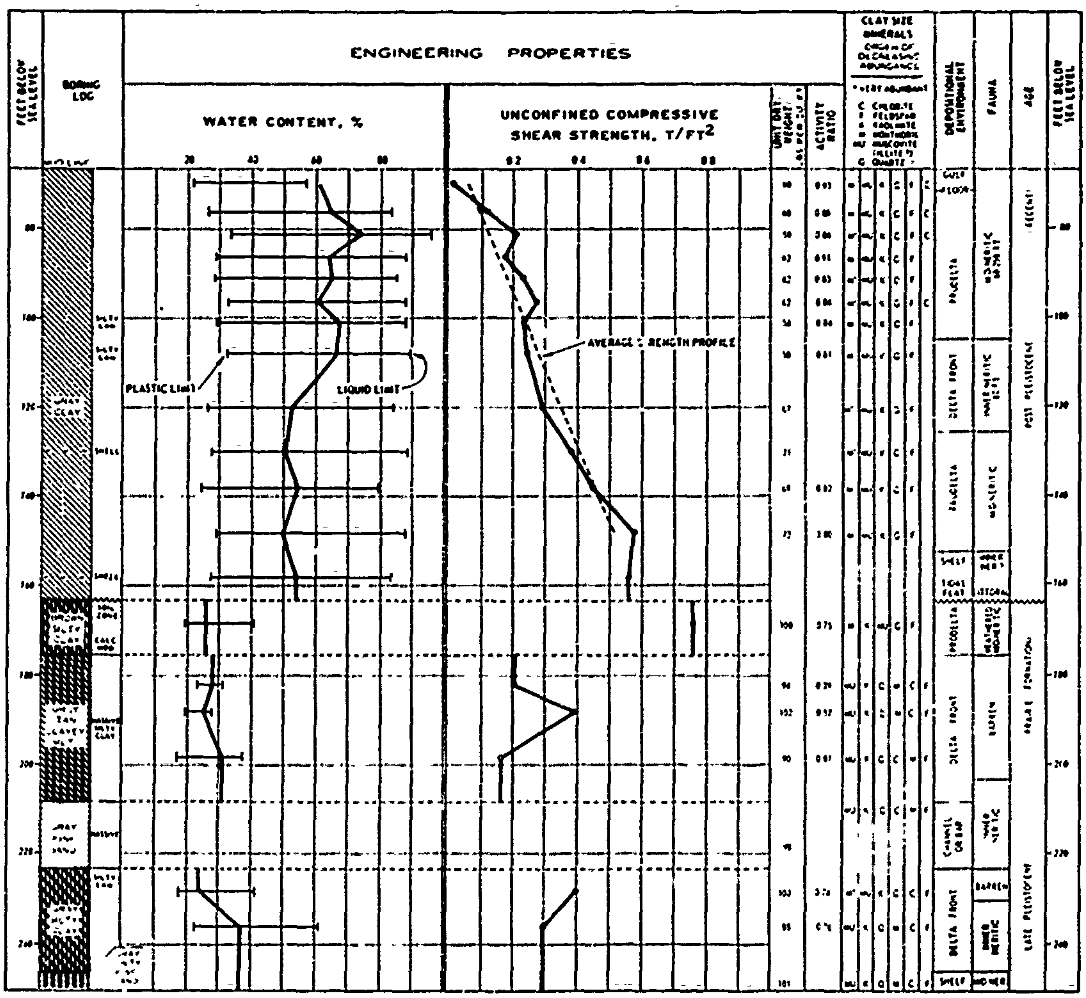
Fig. 46. Typical strength profiles for clay deposits, illustrating the effects of geologic processes (from reference 117)



(Courtesy of The Geological Society of America)

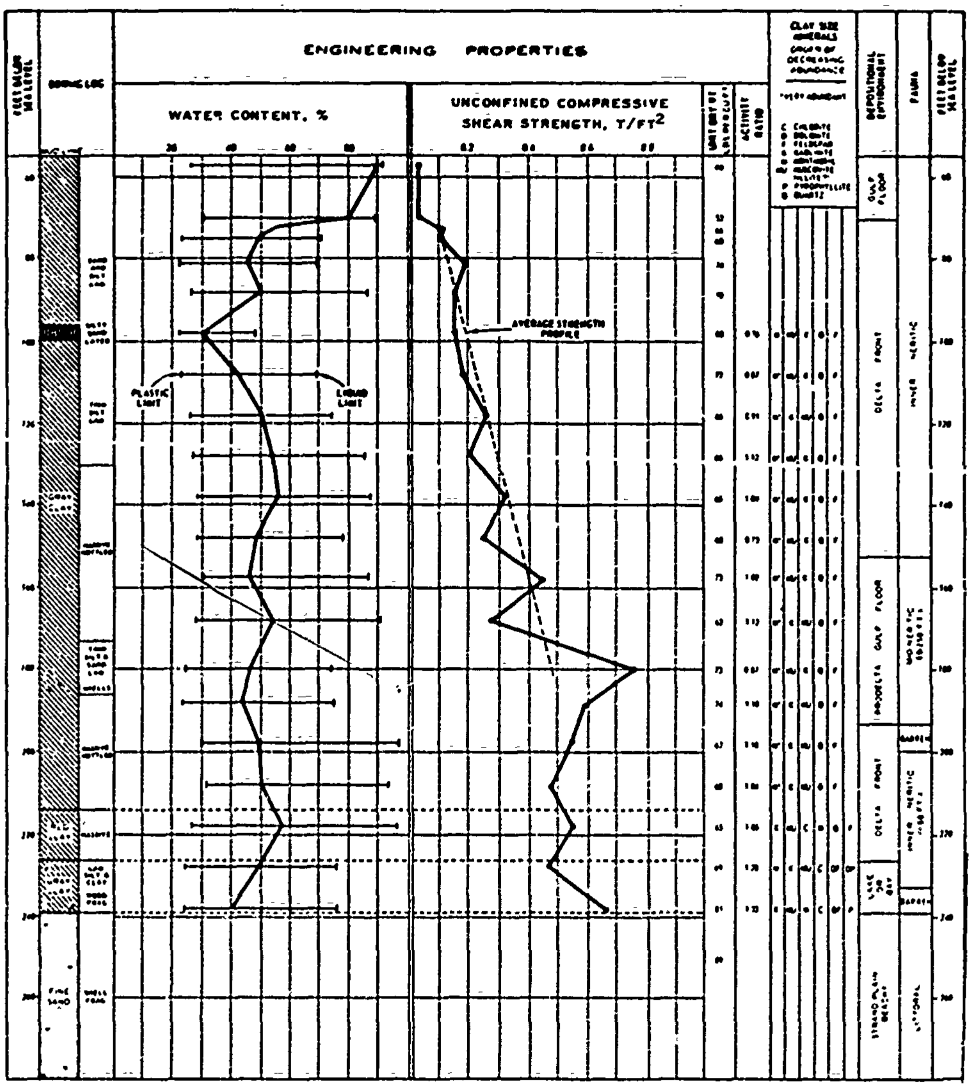
Fig. 47. Rate of change in clay strength with increasing overburden pressure data from Eugene Island area Block 188 boring compared with compilation by Bjerrum (reference 85) and Skempton (reference 121) (from references 117 and 121)

127. Figs. 48-51 show four boring logs with their respective properties which are indicative of the range of conditions encountered off the coast of Louisiana. Boring locations are shown in fig. 42. Fig. 43 also presents data from these four borings. Boring 6 (fig. 48) was made at a site 45 miles offshore in Atchafalaya Bay in 67 ft of water. The shear strength versus depth plot for soil from the mudline out to a depth of about 100 ft is indicative of a normally consolidated soil (see fig. 46a). The average c/p ratio and PI equal 0.31 and 53, respectively; when plotted on a chart of c/p versus PI, the point falls close to both Skempton's and Bjerrum's curves for normally consolidated clays (fig. 47). The consistency of the upper clays ranged from very soft to stiff. The upper clays are inorganic clays of high plasticity (CH). The soils immediately below the 100-ft level differ from the upper soils and reflect higher strengths probably due to weathering during exposure of this shelf during a lower sea level (see fig. 46b). The weathered zone is a stiff inorganic silty clay of low plasticity (CL),



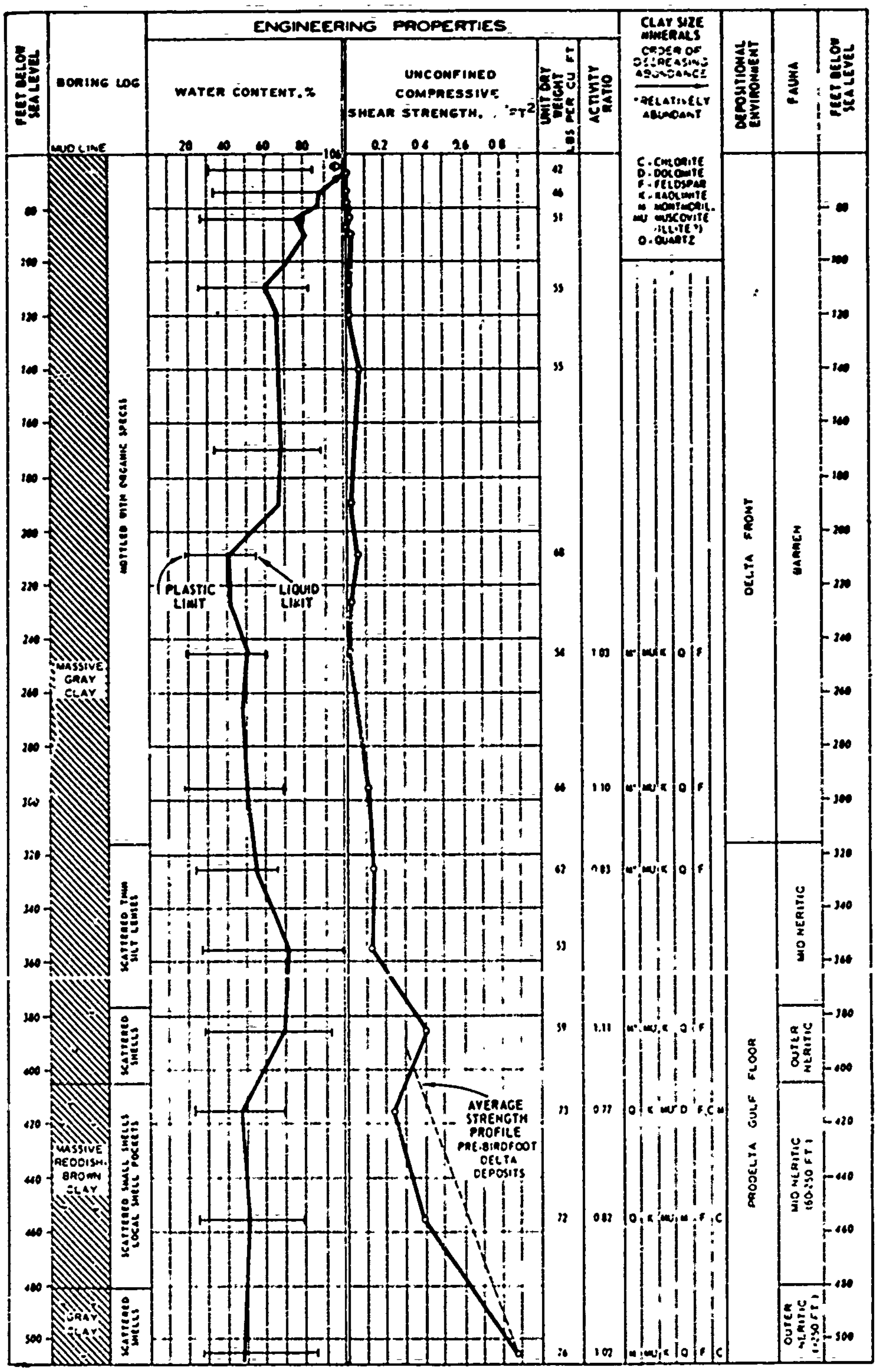
(Courtesy of The Geological Society of America)

Fig. 48. Engineering data from boring 6 off Louisiana coast (see figs. 41 and 42) (from reference 117)



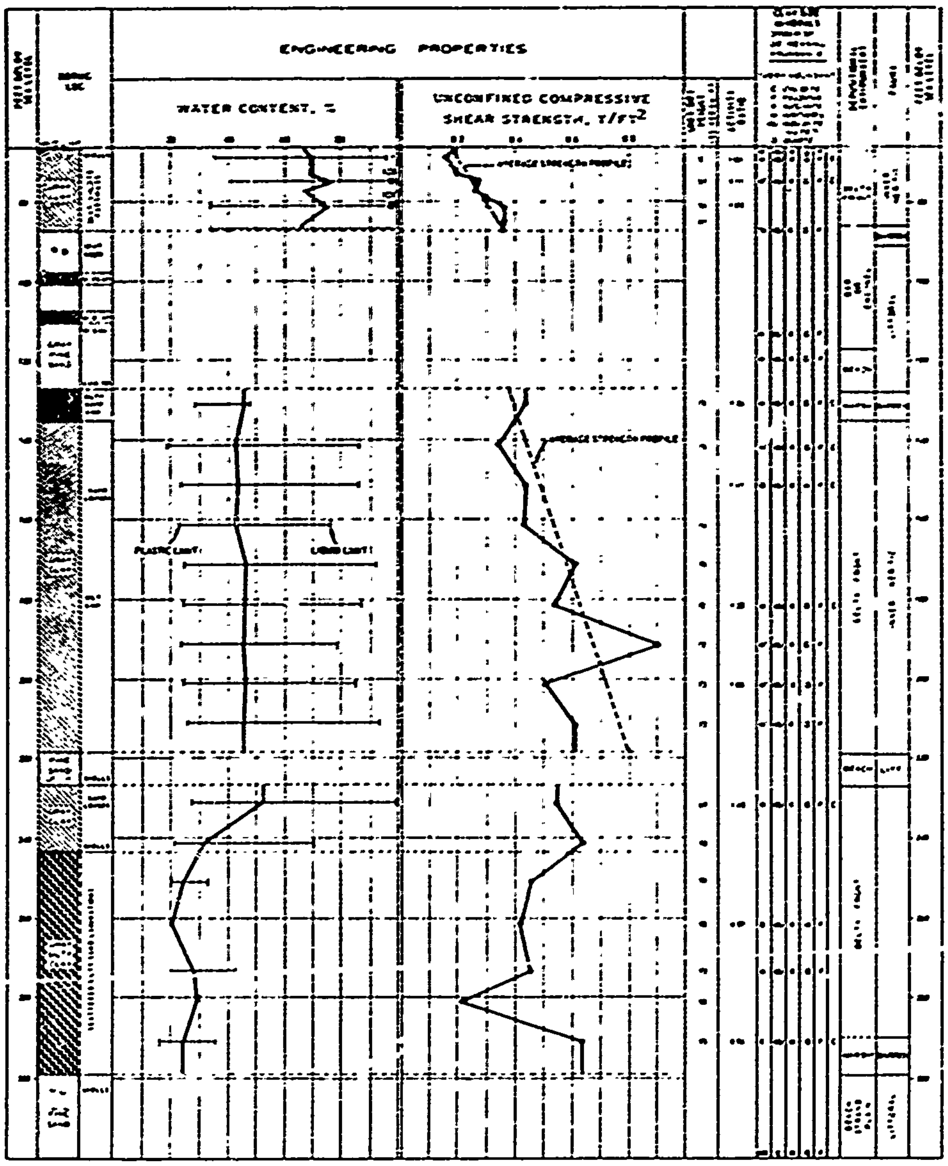
(Courtesy of The Geological Society of America)

Fig. 49. Engineering data from boring 9 off Louisiana coast (see figs. 41 and 42) (from reference 117)



(Courtesy of The Geological Society of America)

Fig. 50. Engineering data from boring 12 off Louisiana coast (see figs. 41 and 42) (from reference 117)



(Courtesy of The Geological Society of America)

Fig. 51. Engineering data from boring 8 off Louisiana coast (see figs. 41 and 42) (from reference 117)

and the soils below this are generally silts and clays of low plasticity, having interbedded silt and sand layers.

128. Fig. 49 shows data for boring 9, located in 55 ft of water approximately 8 miles from Grand Isle, La., near the frontal zone of the abandoned LaFourche-Mississippi delta. This boring shows 15 ft of very soft underconsolidated inorganic clay of high plasticity (CH), underlain by soft to stiff underconsolidated clay of high plasticity (CH). Plotting the c/p ratio of 0.15 and PI of 55 in fig. 47 shows that this deposit is still undergoing consolidation and that equilibrium has not yet been reached.

129. Boring 12 (fig. 50) was drilled near the South Pass frontal margin of the modern Mississippi birdfoot delta in 59 ft of water. The data show a deep underconsolidated clay deposit overlying approximately normally consolidated clay. This is an example of a case in which the rate of deposition far exceeds the rate of consolidation of the soil (see fig. 46d). Inorganic clay of high plasticity (CH) exists for the full depth of the borehole. To a depth of about 350 ft below the mudline, the CH soil is very soft, with strength generally less than 0.12 tsf. Soils in the lower 150 ft of the boring had consistencies ranging from soft to stiff.

130. Fig. 51 shows the existence of overconsolidated inorganic clays (CH) in the upper 20 ft of boring 8 (see fig. 46b). Consistency of these clays is soft to medium. Beneath this clay stratum lie about 30 ft of sands and silty sands, which in turn overlie about 85 ft of normally consolidated highly plastic clay (CH), having consistencies ranging from medium to stiff.

131. In summary, the U. S. Gulf of Mexico continental shelf consists of silty clays and clays. Underconsolidated, normally consolidated, and overconsolidated sediments may be encountered along this shelf. The consistency of the sediments ranges from very soft to stiff, there being generally an increase of strength with depth. In addition to references already noted, Noorany and Gizienski present detailed discussions on the shear strength and consolidation characteristics of these Gulf of Mexico soils in reference 1.

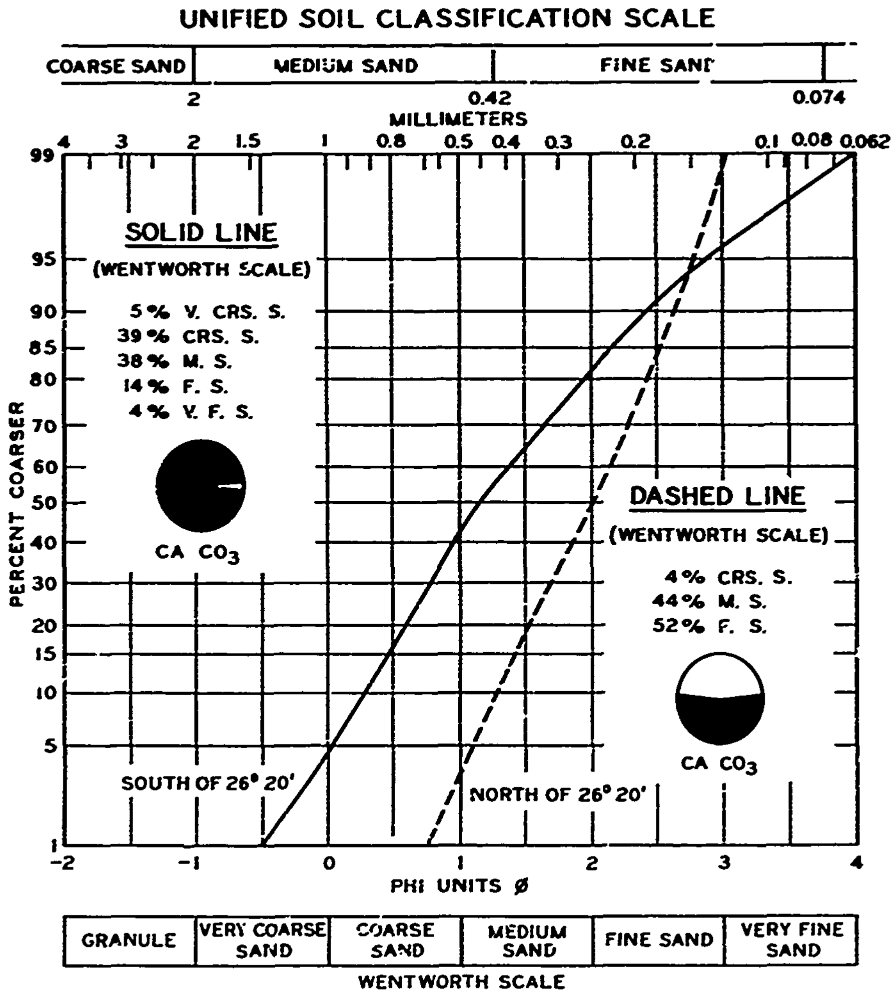
East Coast

132. In an effort to find sand for the replenishment of beaches, the CE has a continuing sand inventory program. One survey[59] under this program was performed off the southeastern coast of Florida from Miami to Palm Beach, latitudes 25°40' N to 26°48' N, in water depths of 15 to 350 ft from 0.1 to 4.5 nautical miles off the coastline. Seismic refraction survey techniques and a pneumatic vibratory corer were used to locate and sample potential sand deposits. The shelf south of Boca Raton, 26°20' N, was found to be generally rocky with only a thin veneer of sediments, though relatively thick deposits of sand-size calcareous skeletal fragments were found in troughs on the shelf surface generally paralleling the coast. North of Boca Raton, a deposit of homogeneous, fine to medium, gray sand overlying the shelf contained 60 percent quartz and 40 percent calcareous skeletal remains. The suitability of the sands for beach fill is apparently questionable because of the degradable nature of the calcareous sands south of Boca Raton and the fineness of the sands north of Boca Raton. Grain-size analyses were performed on representative 8- to 10-gram samples using a rapid sediment analyzer (see fig. 24). Some 31 core samples were taken, varying in length from 1 to 11 ft. The average size distributions of the sand south and north of Boca Raton are shown in fig. 52.

133. Information was obtained from the U. S. Coast Guard pertaining to foundation investigations by McClelland Engineers for five offshore light structures along the east coast located as follows:

- a. Frying Pan Shoals, 30 miles southeast of Cape Fear, N. C.
- b. Diamond Shoal, 23 miles southeast of Cape Hatteras, N. C.
- c. Entrance of Chesapeake Bay, 14 miles east of Cape Henry, Va.
- d. Scotland Light Structure, entrance to south channel in New York Harbor, 7 miles east of Sandy Hook, N. J.
- e. Ambrose Light Station, entrance to Ambrose channel in New York Harbor, 7 miles east of Sandy Hook, N. J.

134. A skid-mounted rotary drill rig placed aboard a self-propelled diesel-powered vessel was used at all sites. The drilling procedure used 4-in.-diam casing suspended from the drill deck level and



(Courtesy of U. S. Army Coastal Engineering Research Center)

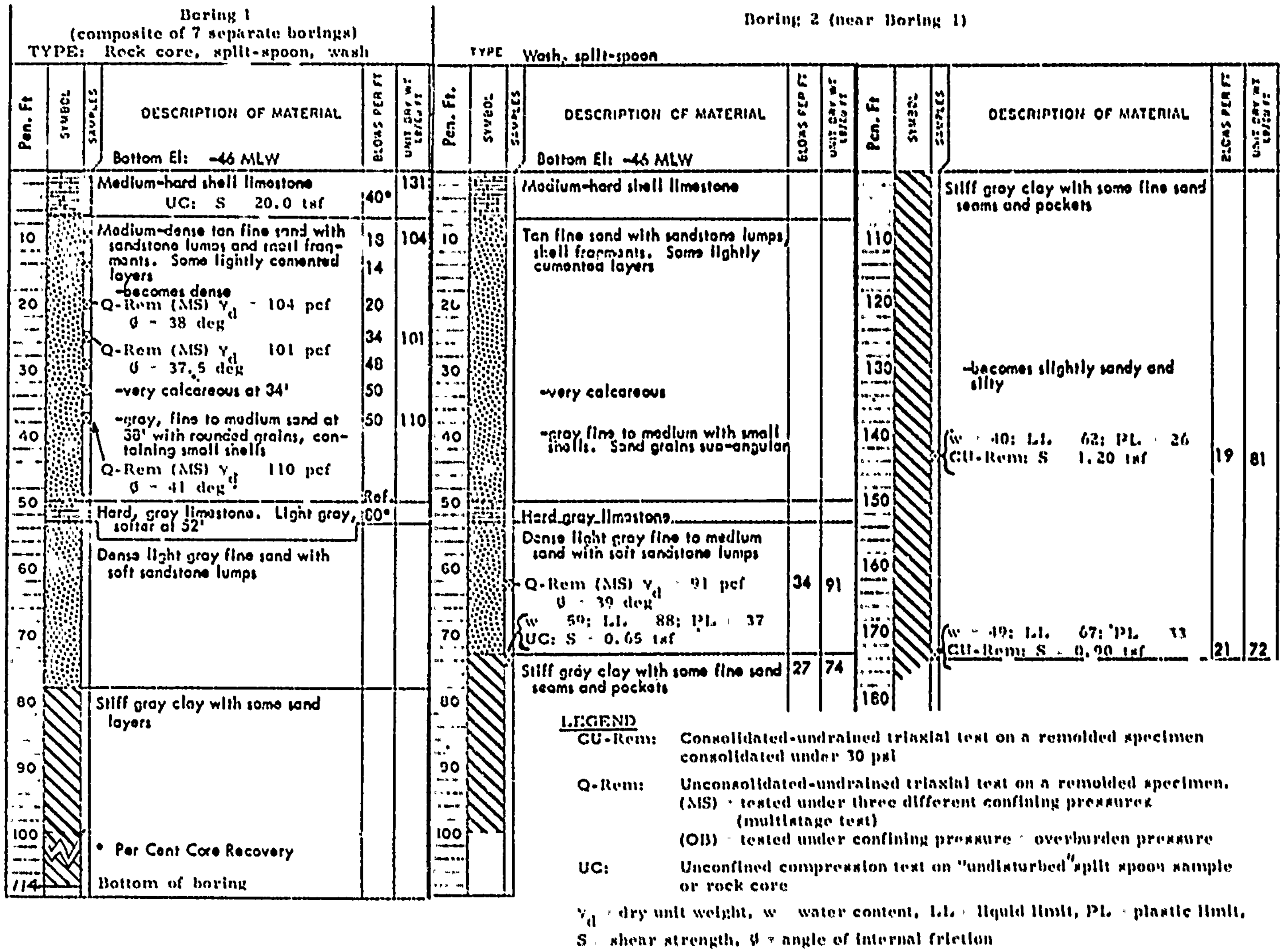
Fig. 52. Average size distribution of shelf sediments near Boca Raton. Note the difference in grain size and the concomittant difference in carbonate (shell) content (from reference 59).

penetrating a short distance into the sea floor. Drilling mud was used to stabilize the drill hole below the casing. At Frying Pan Shoals, conventional rotary drilling procedures advanced a 3-in.-OD, double-tube, rock core barrel into limestone deposits, and a 140-lb hammer with a 30-in. drop drove the 2-in.-OD standard split-spoon sampler into soil to provide standard penetration values and obtain soil samples. At both the Diamond Shoal and the Chesapeake Bay Entrance sites, a 140-lb hammer with a 30-in. drop drove 3-in.-diam Shelby tubes to obtain soil samples. At both the Scotland Light and Ambrose Light sites, samples of cohesive soil were obtained by hydraulically pushing 3-in.-diam Shelby tubes into the soil. Samples of cohesionless soil were obtained by driving either 3-in.-diam Shelby tubes or standard 2-in.-OD, split-spoon samplers into the soil with the 140-lb hammer and 30-in. drop. At the Ambrose site, a 680-lb hammer was required at times to obtain significant sample penetration, and a double-tube rock core barrel was used to sample shale.

135. Two boring logs for the Frying Pan Shoal site are shown in fig. 53. The water depth is 46 ft at this site. Other than a 9-ft cap and a 3-ft intermediate vein of limestone, the foundation soils generally consist of tan to gray, fine to medium sands (SP to SW) to depths of 72 to 78 ft. An unconfined compression (UC) test on a core from the limestone cap indicated a shear strength (S) of 20 tsf. Four unconsolidated-undrained, multiple-stage triaxial tests (termed Q-Rem (MS)) on remolded specimens of the sands indicated angles of internal friction, ϕ , ranging from 37.5 to 41 deg. A UC test on a split-spoon sample from the top of the clay stratum at a depth of 74 ft indicated an S of 0.65 tsf. Two consolidated-undrained triaxial tests (termed CU-Rem tests) performed on remolded clays below 144 ft indicated shear strengths of 1.2 and 0.9 tsf under confining pressures of 30 psi.

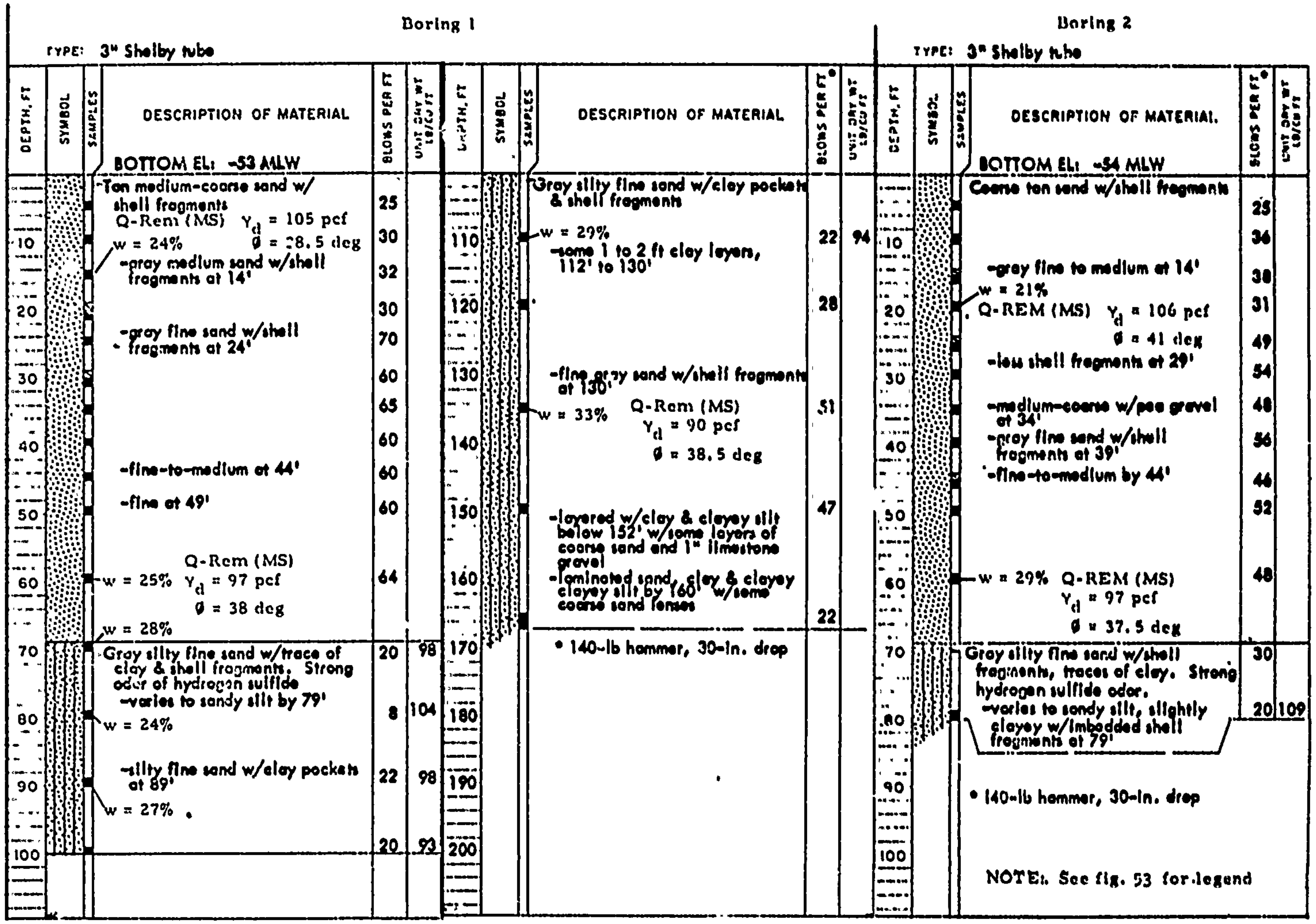
136. At the Diamond Shoal site in a water depth of 53 ft, the subbottom was comprised of medium and fine sands (SP and SP-SM) to a depth of about 69 ft below the surface (see fig. 54 for the logs of borings 1 and 2). Blow counts indicated medium to dense sands to 20 ft and very dense sands to 69 ft. Silty fine sands (SP-SM and SM) were found to extend below this depth to 167 ft in boring 1. Four Q-Rem (MS) tests on the sands indicated ϕ values ranging from 37.5 to 38.5 deg.

137. Sands (SP and SP-SM), and clays (CL and CH) underlie the Chesapeake Bay entrance site under 38 ft of water (see fig. 55 for logs of the two borings). Four Q-Rem (MS) tests on sands from 10- to 60-ft depths indicated ϕ values from 39 to 40 deg. Shear strengths in six UC tests on clays from 63 to 110 ft below the sea floor ranged from 0.24 to 1.16 tsf, indicating that these are soft to very stiff clays.



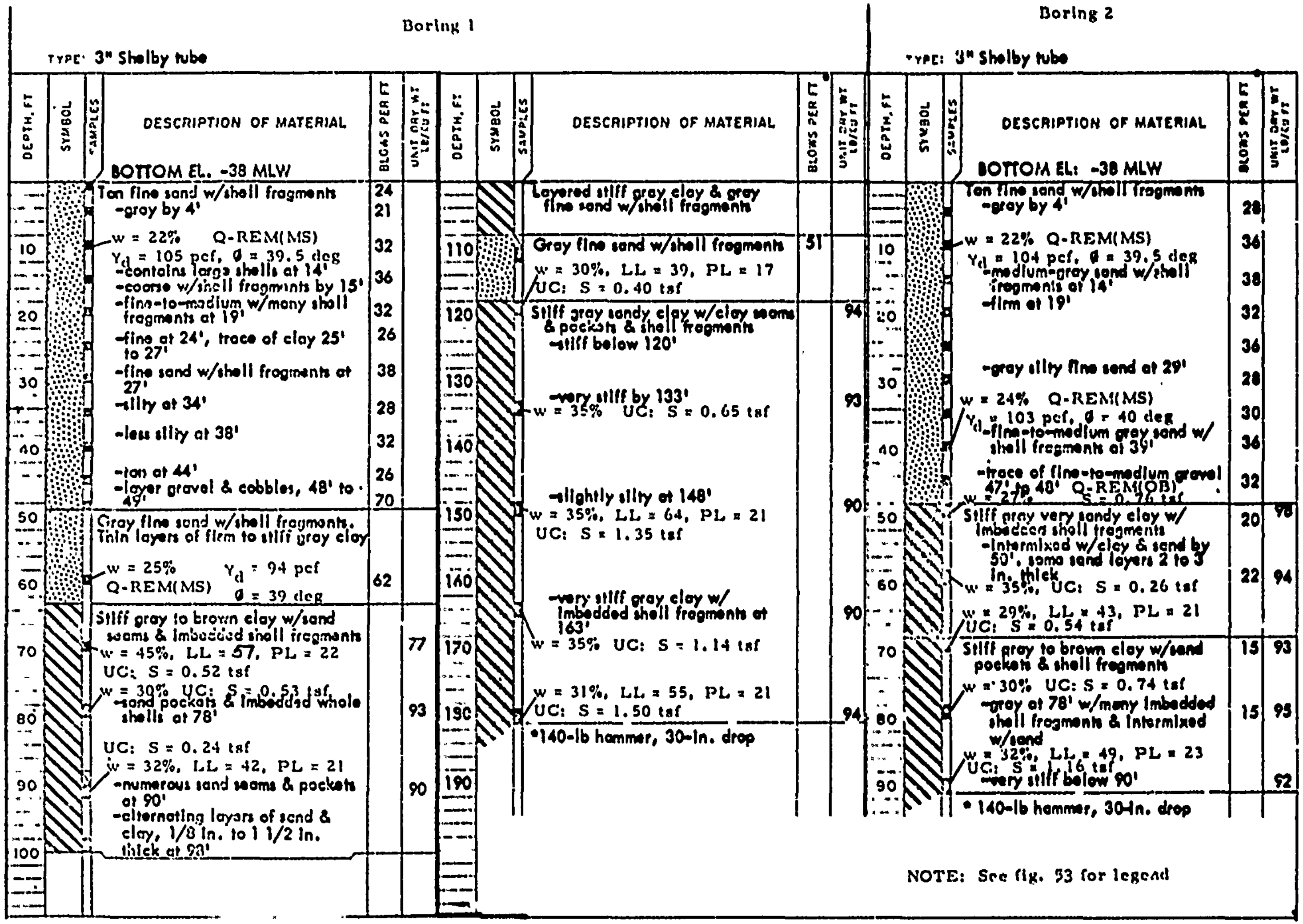
(Courtesy of U. S. Coast Guard)

Fig. 53. Boring logs for borings 1 and 2 made from Frying Pan Shoals offshore structure (from reference 122)



(Courtesy of U. S. Coast Guard)

Fig. 54. Boring logs for borings 1 and 2 made from Diamond Shoal offshore structure (from reference 122)



(Courtesy of U. S. Coast Guard)

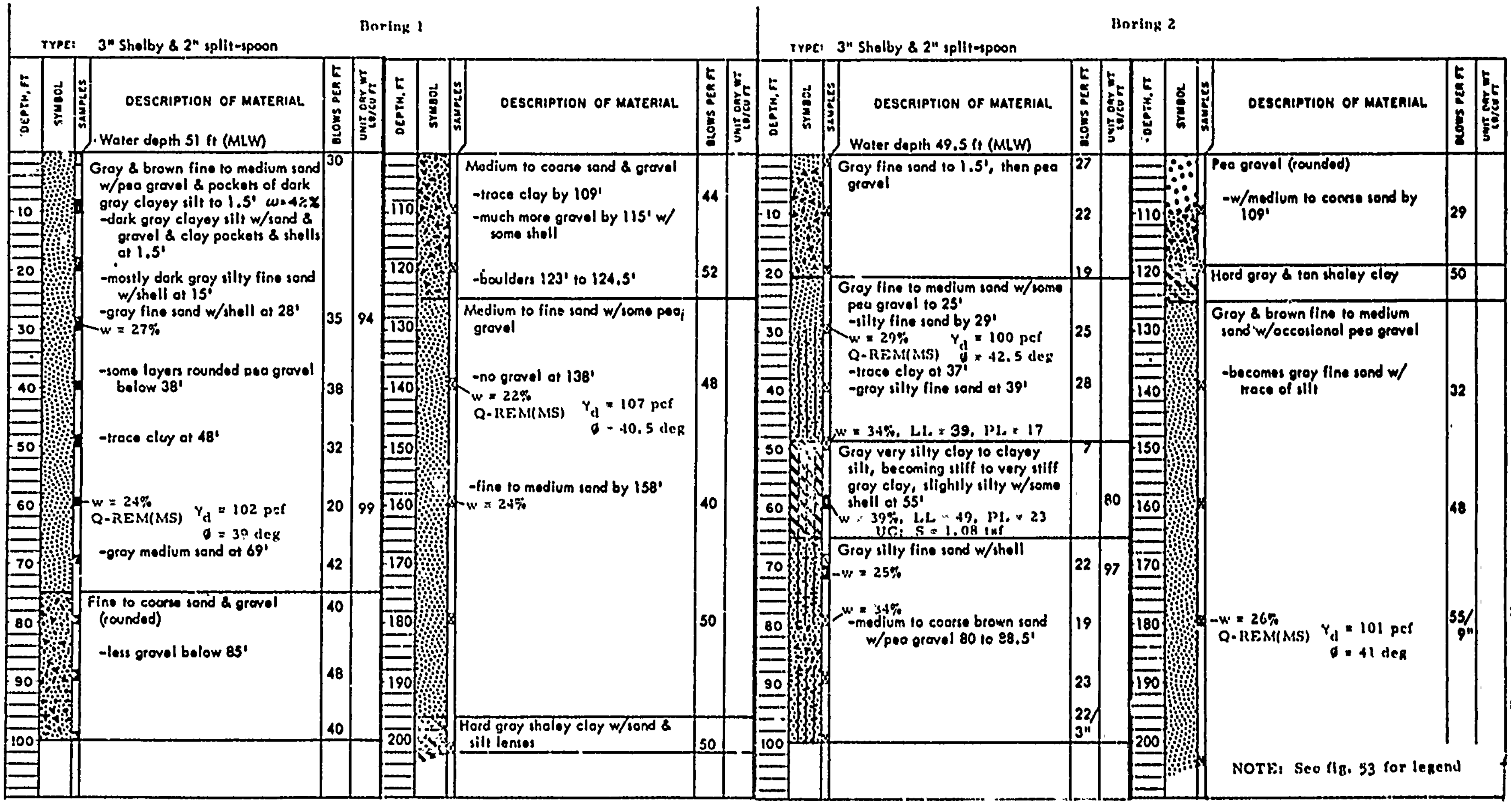
Fig. 55. Boring logs for borings 1 and 2 made at Chesapeake Bay entrance (from reference 122)

Five UC tests on clay strata from depths of 118 to 181 ft in boring 1 indicated shear strength values ranging from 0.40 to 1.5 tsf, indicating medium to very stiff clays.

138. At the Scotland Light Structure site, the water depth was about 50 ft. Soils were principally sands, sands and gravel, and pea gravel (see fig. 56 for logs of the two borings). One UC test on a silty clay with a water content of 39 percent at 60 ft below the mudline in boring 2 indicated a shear strength of 1.1 tsf. Q multistage triaxial tests of four sands at depths from 30 to 180 ft indicated angles of internal friction between 39 and 42.5 deg.

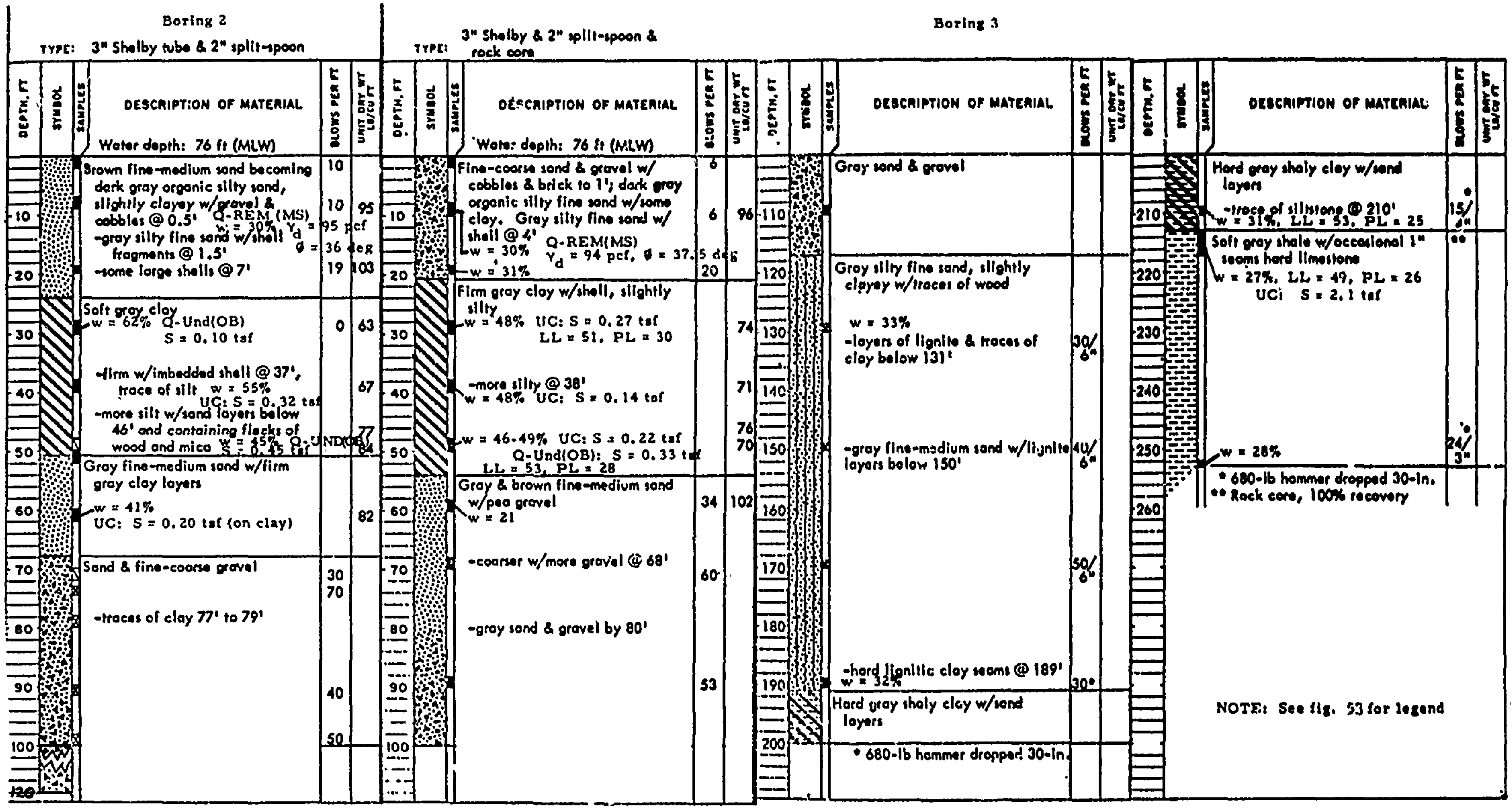
139. The Ambrose Light Station site, in 76 ft of water across the harbor entrance from the Scotland Light site, was underlain by strata of sand, clay, sand and gravel, and shale to a depth 253 ft below the sea floor. Logs of the two principal borings are shown in fig. 57. Six UC tests on specimens of the clay stratum from 20 to 50 ft below the mudline indicated shear strengths ranging from 0.14 to 0.37 tsf (i.e., soft to medium consistency). Three Q triaxial tests on single undisturbed specimens of clays tested with confining pressures equal to overburden pressures gave shear strengths ranging from 0.10 to 0.45 tsf (i.e., very soft to medium consistency). The underlying strata of sand and gravels were very dense, requiring more than 50 blows/ft with the standard 140-lb hammer for penetration. To facilitate sampling of these very dense sands below 190 ft, a 680-lb hammer was used. An unconfined compression test on a sample of the shale reached at the 190-ft depth indicated a shear strength of 2.1 tsf. The lack of microfossils at both the Scotland and Ambrose Light sites verifies the terrestrial origin of the soils at these two sites.

140. In general, the data above indicate that the soils in the East Coast continental shelf are coarser grained and stronger than those in the Gulf of Mexico. This should not be taken as a generalization for the entire East Coast shelf, because data for only a few borings are available and because the particular sites investigated were selected either as potential sand sources or as sites for navigation light structures, which typically would be located just off a channel area.



(Courtesy of U. S. Coast Guard)

Fig. 56. Boring logs for borings 1 and 2 made from Scotland Light Structure (from reference 122)



(Courtesy of U. S. Coast Guard)

Fig. 57. Boring logs for borings 2 and 3 made at Ambrose Light Station site (from reference 122)

Summary

141. There is an apparent lack of sands, gravels, and boulders along much of the U. S. continental shelf. Ice-rafted boulders and gravels can, of course, be found along the upper latitude shelf off the coast of Alaska. Sands, silts, and clays predominate in the shelf zones. Shells and microfossils are often dispersed within the deposits. The fine-grained cohesive soils are often very weak, e.g., the surface soils off the southern California coast and the underconsolidated soils in some areas off the Louisiana coast. Montmorillonite is the dominant clay mineral. Somewhat overconsolidated marine clays are often found to be weak and highly compressible. The continental shelf area off the Louisiana coast has been the most explored due to the exploitation of hydrocarbon fuel deposits during the past 25 years.

142. Information on the engineering properties of soils off the north slope of Alaska and of soils of the continental borderland off southern California (obtained in current exploration for and exploitation of hydrocarbon fuels in these areas) is still proprietary and probably will not be available in the literature for several years.

PART V: MARINE FOUNDATION ENGINEERING

143. Marine foundation engineering involves a variety of combinations of foundations and structures:

- a. Anchors are used to restrain the excursions of floating or submerged (but neutrally or positively buoyant) structures.
- b. Pile-type legs resting on or penetrating into the sea floor support platforms which are raised above the sea surface.
- c. When bottom materials are soft, pile-type legs resting on large mat foundations are sometimes used as an alternative to legs resting on or penetrating into the bottom.
- d. Footings and raft foundations can also be used to provide the necessary support.
- e. Combinations of these foundation types are used to meet the particular requirements of the pertinent structure and site.

144. Structures may be temporary or permanent and manned or unmanned. All of these factors, as well as the investment the owner is willing to make for a foundation study, influence the type of foundation selected. Cost is often the controlling factor in determining how extensive the soil investigation will be, since exploration offshore is far more expensive than on-shore exploration. Principal topics discussed in this part are anchors, piles, mats and footings, dredging, and underwater fills.

Anchors

145. Anchors are typically used to limit the vertical and lateral excursion of floating structures and of positively or neutrally buoyant submerged structures. Anchors may also be required for small instrument packages used to monitor various aspects of the water body or the air-sea interface and for large buoys used for navigational purposes or for data collection. On a larger scale, the structure requiring anchorage could be a floating bridge, e.g., the Hood Canal Bridge in Washington,

or a seadrome. Hood Canal Bridge is not strictly an offshore structure (it is located in one arm of the Puget Sound), but the depths of water and problems of construction encountered during its erection are indicative of what may occur offshore. Seadromes, or floating airports, were conceptualized shortly after World War I. Now with the advent of supersonic transports, the rising cost of land, and the surge of interest in the oceans, proposals for floating airports are again appearing in the literature.[123-128] With the current state-of-the-art, design and construction costs for a floating airport anchored to the bottom would make this the most expensive type of offshore airport. However, this may be the only feasible method of providing an offshore airport at a deepwater offshore site.[124,125,129]

146. Deadman anchors used in supporting bulkheads, drilled cast-in-place tieback or tiedown anchors, and mushroom anchors used to hold down power transmission lines are typical land anchors.[130-137] References 130-132, pertaining to mushroom anchors embedded in sands, present results demonstrating the relation of the pullout force to the geometry of the anchor, depth of burial, and strength and unit weight of the soil. Deadman anchors are discussed in many other soils texts besides those referenced. The mushroom anchor has its counterpart in the marine environment, but the deadman anchor does not have a direct counterpart. In all marine anchorage systems, cable length is critical: too short a cable can cause intermittent submergence of the structure or pullout of the anchor, and too long a cable might permit excessive excursion of the structure and possible kinking and fouling, with subsequent weakening of the cable between the structure and the anchor.

147. The anchor provides resistance, through soil-anchor interaction, to forces transmitted by the cable from the restrained structure. Some common types are the mass (or dead weight), mushroom, drag (or fluke), and pile anchors. A customary index used to indicate capability of a particular anchor is the holding power to weight-in-air ratio, HP/W_a. The holding power is the pulling force that an anchor can resist and is a function of (a) depth of embedment, (b) submerged weight of the anchor, (c) angle that the cable makes with the bottom, (d) the

angles subtended by the anchor fluke, the sea floor, and the anchor shank for fluked anchors, and (e) the soil properties at the particular site. Various anchors have different holding powers with respect to vertical or horizontal tractions, depending on the anchor's relationship to the soil.

148. A mass anchor can be made up of old car motors, welded rails, concrete, and similar inexpensive masses (fig. 58a). This type of anchor provides a vertical HP/W_a of less than 1 because its holding power, or resistance, is dependent solely on its submerged weight. Horizontal resistance is dependent on adhesion and friction between the contact of the anchor and the sea floor. Some very soft and weak marine soils may fail under the weight of a massive anchor; this in turn may fail the total anchorage system due to excessive tension on the cable, placing the anchored structure in jeopardy. Large mass anchors consisting of concrete boxes filled with tremie concrete and covered with riprap were used on the Hood Canal floating bridge.

149. The mushroom anchor is embedded usually by the force of its own weight; sometimes, however, the anchor is jetted to the desired depth (fig. 58b). Its holding power depends upon both its mass and the resistance of the soil above it. For anchors vertically embedded in sand, a horizontal sea floor, and a ratio of depth of embedment to diameter, h/d, less than 6, the failure surface is circular in plan. In cross section, it is tangent to the rim of the mushroom and then curves upward at a decreasing slope until it makes an angle with the surface of $\left(45 - \frac{\phi}{2}\right)$ deg. The dimensionless form of the pullout force equation, developed in model experiments using flat circular plates and air-dry sand,[130] is:

$$\frac{F}{hd^2\gamma'} = C_1 + C_2 \frac{h^2}{d^2}$$

where

F = pullout force

h = depth of embedment of the anchor

d = diameter of the anchor

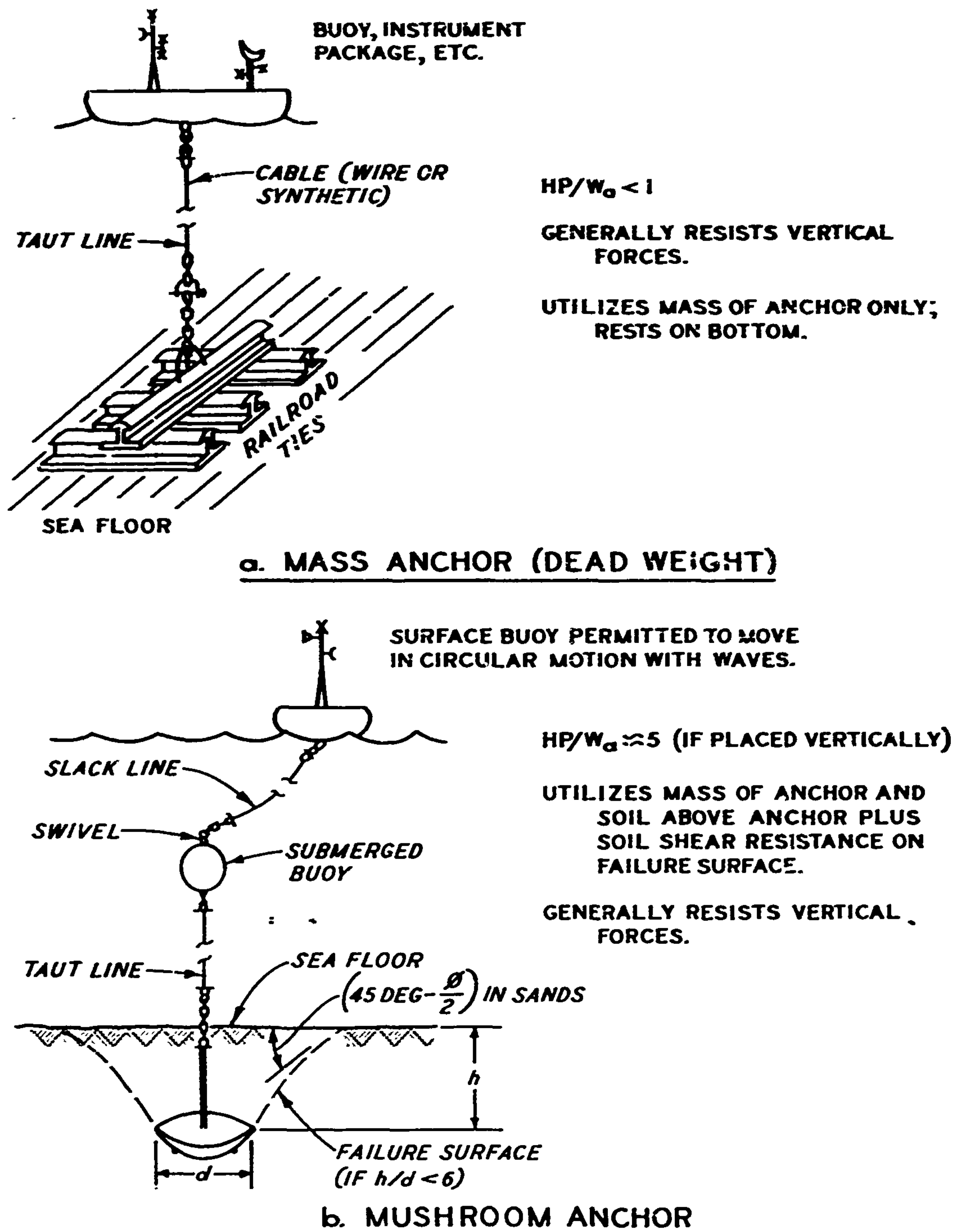


Fig. 58. Mass and mushroom anchors

γ' = effective unit weight of the soil

C_1 and C_2 = constants (presumably functions of the angle of internal friction ϕ and relative density)

For an h/d greater than 6, only a slight surface heave occurred in the vicinity of the shank of the anchor during failure, with the dimensionless equation becoming:

$$\left(\frac{F}{d^3\gamma} - 170\right)\frac{d}{b} = C_3 + C_4\frac{h}{d}$$

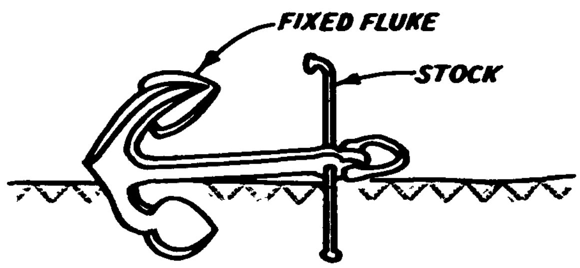
where

b = thickness of the anchor

C_3 and C_4 = constants

These equations have been, to some extent, confirmed in larger scale field tests in sand.[132] In the marine environment, mushroom anchors perform fairly well in soft mud bottoms, achieving embedment by sinking under their own weight or by jetting. An HP/W_a of 5 is attainable in a vertical direction, but, if the anchor should drag under load, it will probably come out.[138]

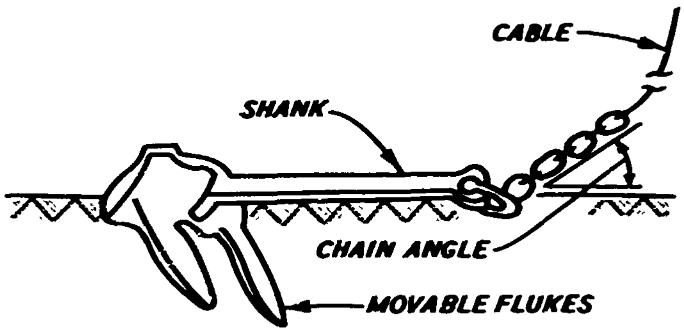
150. Drag anchors are typical ship anchors. The shape of the flukes on these anchors, as well as their relative dimensions, affects their holding capability. Embedment of fluke or drag anchors is accomplished by dragging the anchor along the bottom for a distance of 50 ft or more. Hence, their holding ability is dependent on how well the flukes dig into the sea floor. Some of the more common drag anchors are the Admiralty, Navy stockless, Danforth, and LWT (lightweight type) anchors (fig. 59). Characteristic of all fluke anchors is their ability to resist horizontal forces and their relative inability to resist vertical forces. For this reason, several shots of heavy chain (a shot is a 90-ft length of chain) are generally used between the anchor shank and the cable, and the scope (the ratio of the distance from the ship to the anchor divided by the depth of water) is made as large as possible (e.g., a ratio of 7) to keep some chain and the shank of the anchor lying on the sea floor. Drag anchors are susceptible to pullout when torque is transmitted through the cable to the shank. Some typical



$HP/W_a \approx 5$

FIXED FLUKE REQUIRES A DEFINITE PENETRATION ANGLE FOR BEST HP/W_a.

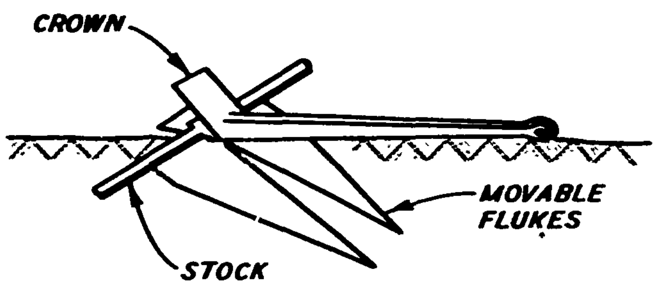
a. STOCK ANCHOR



HP/W_a ≈ 6 IN SAND
≈ 2 IN MUD

LACK OF STOCK ALLOWS FOR ROTATION AND SUBSEQUENT PULLOUT.

b. NAVY STOCKLESS (STANDARD) ANCHOR



HP/W_a ≈ 16 IN SAND
≈ 9 IN MUD

STOCK REDUCES TENDENCY TO ROTATE OUT.

CROWN FORCES FLUKES TO DIG INTO BOTTOM DURING DRAGGING.

c. LIGHTWEIGHT ANCHOR (LWT OR DANFORTH)

GENERAL COMMENTS: ANCHORS GENERALLY RESIST HORIZONTAL FORCES.

HP/W_a RATIOS GIVEN FOR CHAIN ANGLE = 0 AND 50' DRAGGING DISTANCE.

ANCHORS NEED TO BE DRAGGED $\approx$ 50' FOR FLUKES TO PENETRATE.

STRONGEST RESISTANCE IS IN DIRECTION OF INITIAL PULL.

Fig. 59. Stock, Navy stockless, and lightweight-type anchors

HP/W_a ratios are given in fig. 59, which also shows the anchors' relationships to the sea floor, chain, and cable; both higher and lower ratios are recorded in the literature.[138]

151. The STATO anchor is probably the most advanced design of the drag-type anchors.[139] Its designers have taken into consideration the composition of the bottom, the penetration required, and the problem of torquing. Fig. 60 shows the characteristics of the STATO anchor. The figure also shows the use of both STATO and mass anchors in an anchorage system. The angle subtended by the fluke and the anchor shank influences the holding capacity of the anchor; 34 deg was found best for sand and 50 deg was best for mud bottoms. The STATO anchor has a wedge insert to provide for presetting these angles (fig. 60).

152. Pile anchors may be precast or drilled cast-in-place concrete piling or steel piling. Of some 100 mooring locations using the drilled-in anchor piling, not one failure has been observed.[140] Another type of anchor is the embedment anchor with movable flukes, which penetrates the sea floor by free fall, by being driven, or by use of explosive propellants. The flukes are forced into an open position by either an upward lift or downward drive on the anchor after penetration is attained. Ideally, these units are placed vertically with a swivel arrangement between the anchor and the chain which assists in the transfer of only vertical forces to the anchor. HP/W_a ratios can be quite high on these anchors, depending on their embedment and the soil mass mobilized by the individualized anchors. HP/W_a ratios greater than 150 were obtained in installations of two driven fluked-type embedment anchors.

153. There has been little consideration given to the effect of shear strength or other soil properties of the bottom on the capabilities of various anchors, other than to distinguish generally between sands and cohesive soils. Some laboratory experiments have been reported in which a specific sand was used with anchors having different geometries to compare their holding power characteristics.[131,141,142] The literature on field behavior of anchors in cohesive soils generally contains no description of soil properties, and the literature search has

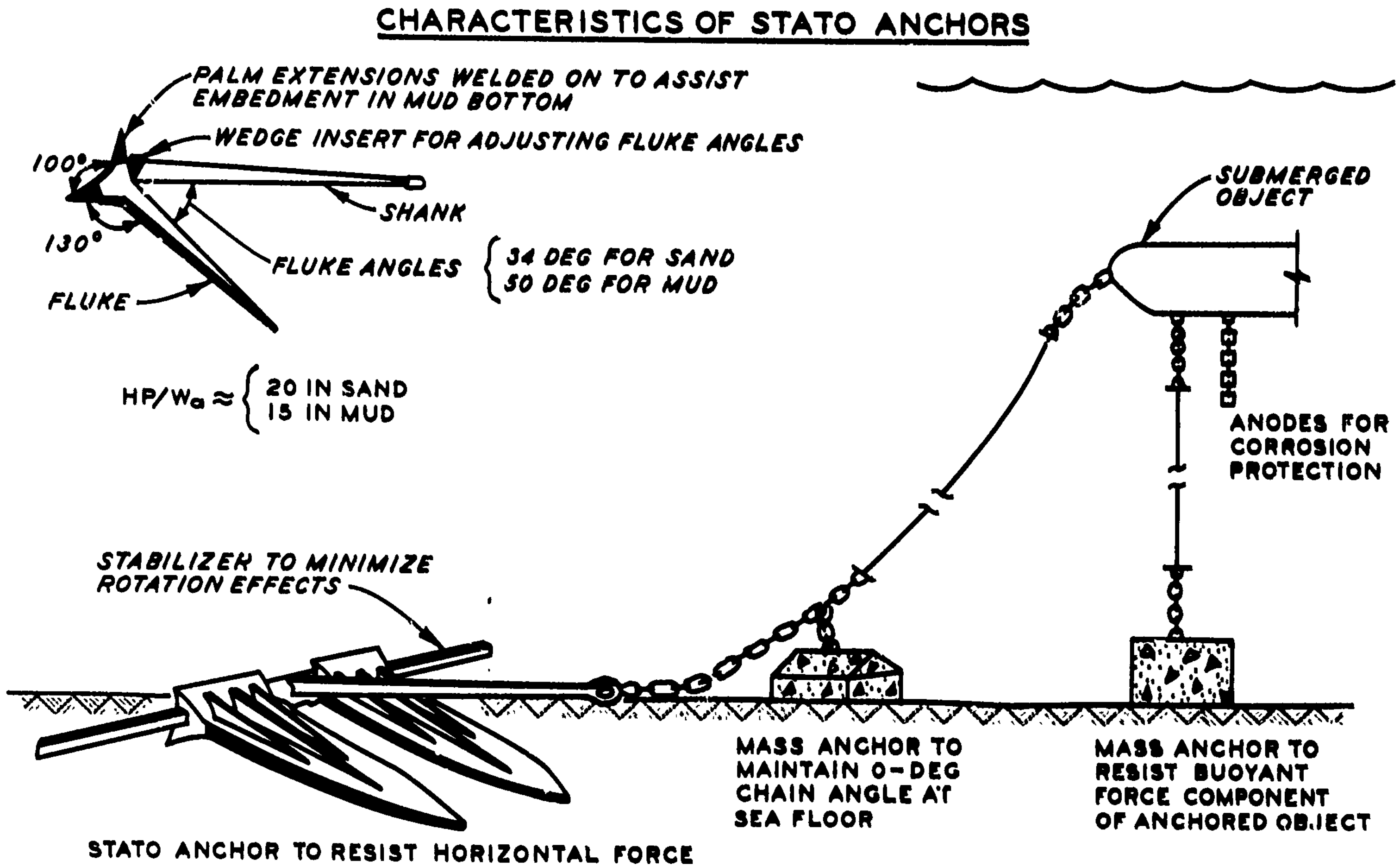


Fig. 60. The STATO anchor in a combination anchorage system

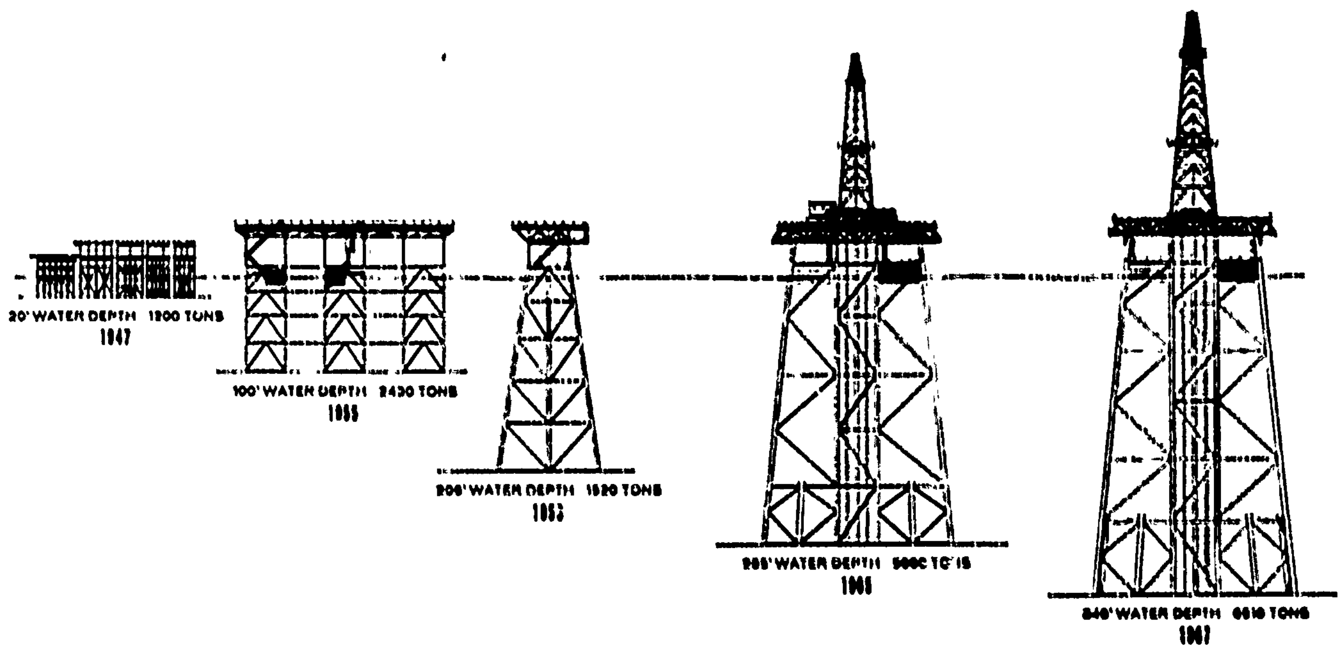
produced no information on any laboratory investigations of anchors in cohesive soils. Research along these lines may prove fruitful.

154. Anchors of substantial weight and size are used to hold floating and submerged structures within desired limits of excursion. Lightweight (15-ton) anchors are not uncommon equipment at offshore oil drilling sites. A 55-ton concrete mushroom anchor was used for a mooring installation at a Navy installation in Rhode Island.[138] Massive deadweight, drag, and pile-type anchors were used on the Lake Washington and Hood Canal floating bridges. Scouring occurred around nine of the 850-ton deadweight anchors on the Hood Canal floating bridge and riprap protection was placed.[143] Smaller anchors of special design are also used for restraining the movement of pipelines.[144]

155. Soil-anchor interactions need to be studied in more detail to arrive at the most efficient combination of anchor and soil for various applications. A similitude approach (e.g., along the lines of Baker and Kondner's work)[130] may be a way to reduce the variables studied to the more significant ones.

Piles

156. Offshore pile-supported platforms are the most prevalent man-made structures on the continental shelf. Fig. 61 shows elevation views and historical stages of the development of permanent platforms in the Gulf of Mexico. In 1970, Shell Oil Company constructed a 24-well platform in the Gulf of Mexico in 373 ft of water, a new record depth.[145] A pile design depends on (a) sea floor soil characteristics pertinent to end bearing, skin friction, and resistance to lateral loads; (b) loading conditions, including wave, wind, current, ice, and earthquake live loads and the dead loads of the structure and the pile itself; and (c) pile characteristics, including pile diameter, wall thickness, length, and composition. Part III has already indicated the low strength characteristics of many marine soils. This section of the report emphasizes the increased loading conditions, the increase in required pile dimensions, and other problems associated with offshore construction.



(From Offshore, Vol 28, No. 6, June 1968)

Fig. 61. Progress of offshore platforms (from reference 146)

157. Platforms in the Gulf of Mexico must resist hurricane forces. Of the more than 2000 permanent platforms built in the Gulf of Mexico from 1947 to 1967,[146] 22 collapsed and 10 others were severely damaged by hurricane forces. Of these 32 failures, 2 were temporary structures and 4 were platforms which were constructed in the early years before adequate design procedures and information were available. Of the remaining 26, 23 had been designed to resist 25-year probability storms, and 3 had been designed for maximum expectable storm conditions. The failures of 2 of the 3 platforms designed for maximum storm conditions were attributed to poor soil conditions.[145] There has apparently been no loss of life from the failures of the permanent platforms, owing to a well-developed hurricane warning system in the Gulf area and the practice of evacuating manned platforms upon the approach of a hurricane.[147] In the case of mobile drilling platforms, at least two pile-supported jack-up platforms tipped over while being prepared to move off location because of apparent soil failures.[148] Dollar value of property lost and production or exploration interrupted due to such failures generally runs into the millions. Even though hurricane mishaps involving permanent platforms have not resulted in loss of lives, failures of offshore platforms could result in loss of life, especially off the west coast of the United States where sudden storms or earthquakes are possible but not predictable. Loss of life has occurred during mobile platform mishaps.[149] One death is known to have occurred when a jack-up work-over barge with cylindrical legs tipped over in the Gulf of Mexico off the Louisiana coast due to inadequate soil bearing capacity.[148,150]

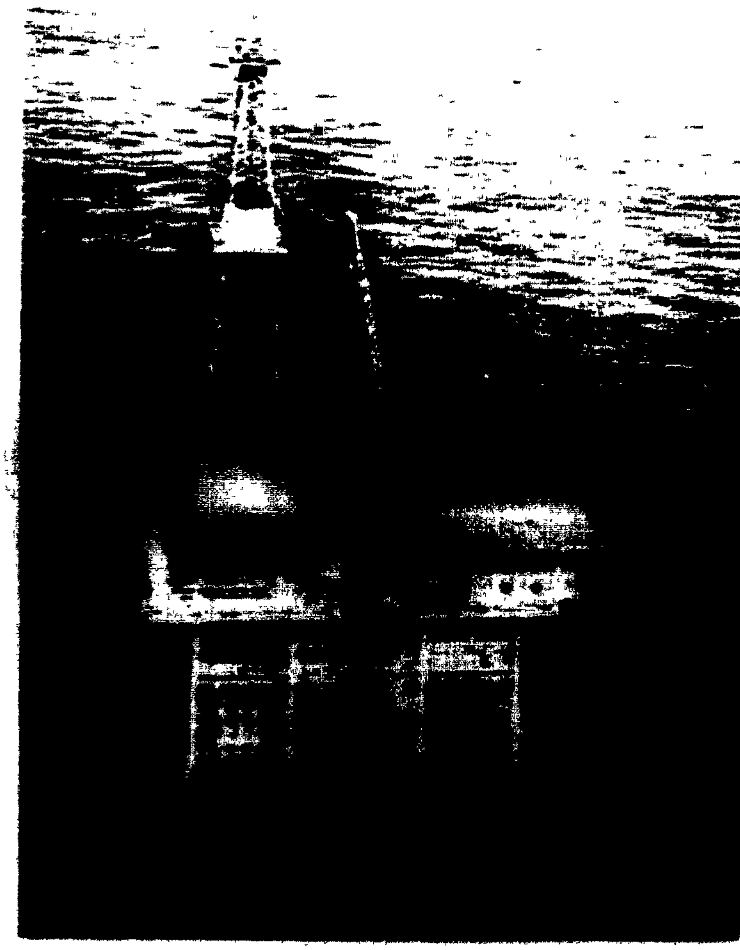
158. Offshore piles generally are subjected to higher vertical and lateral loads than on-shore piles.[151] To additionally complicate the picture, the soils offshore are often very weak, as previously noted. The design capacity for an individual pile of an offshore oil drilling platform might be 2000 tons, as opposed to a maximum capacity of about 200 tons for an on-shore pile. An offshore pile might weigh 110 tons, or about 35 times the weight of its on-shore counterpart.[151] Pile dimensions might be 48 in. in diameter by 600 ft long for an offshore pile, as compared with 16 in. in diameter by 100 ft long for an on-shore pile.

Obtaining pile hammers capable of driving these offshore piles is a problem. Individual pile loads on the U. S. Coast Guard light structures on the east coast were expected to be not less than 950 tons axial and 150 tons shear at the mudline,[122] and these are relatively light structures when compared with some of the massive oil platforms.

159. Off the North Slope of Alaska and in other offshore oil fields in the upper latitudes, lateral loads are even greater for marine structures due to high pressures from ice loads. For example, the 28.5-ft-diam cylinder of the monopod platform of the Union Oil Company in 90 ft of water in Cook Inlet, Alaska, can be subjected to a horizontal force of 7390 kips by a 6-ft-thick sheet of ice having a crushing strength of 300 psi driven by tidal currents.[152] Wind loads are practically negligible compared with possible ice loads. Ice loads off the North Slope area may be even greater than those in Cook Inlet.[153] With the discovery of oil off California and elsewhere in deeper waters, it is probable that the disparity between onshore and offshore piles will increase.

160. The development of offshore oil resources became so widespread that a publication concerning planning, designing, and constructing fixed offshore platforms was issued in 1969 by the American Petroleum Institute (API).[154] The foreword of this publication stresses the rapid advancement of offshore technology and encourages designers to use all research advances available to them. Foundation design is covered in more detail in conventional soil mechanics and engineering texts, particularly for marine work, in reference 20.

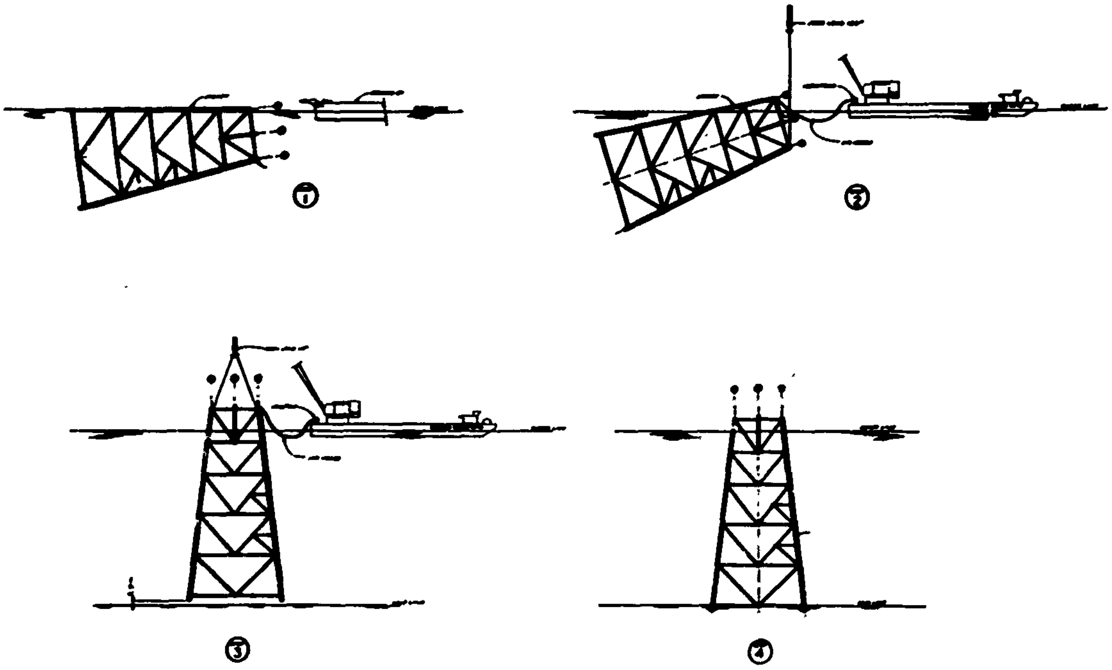
161. The operations of properly positioning, battering, and driving piles are particularly critical and difficult in offshore platform work because many moving parts depend on proper alignment and because free-swinging unbraced piles are subject to fatigue failure due to cyclic forces generated by pile-water interaction.[155] One system developed to meet offshore platform needs is referred to as the template technique. In this system, the platform substructure is prefabricated onshore, floated out to the site, and then sunk to the sea floor.[156,157]



(Courtesy of American Society of Civil Engineers)

Fig. 62. Offshore platform supported by piles (from reference 157)

Platform Holly (fig. 62) was erected using this technique in 200 ft of water in the Santa Barbara Channel off California. The legs of this template are bulkheaded and provide the necessary buoyancy for floating the entire unit to the site (fig. 63). Controlled flooding of the legs tips the template into an upright position and then sinks it onto location. Alternatively, the template can be placed on a barge and towed to the site. The hollow legs of the template, once positioned at the site, act as guides for the installation of the piles. In some installations, a temporary platform is then placed on top of the substructure from



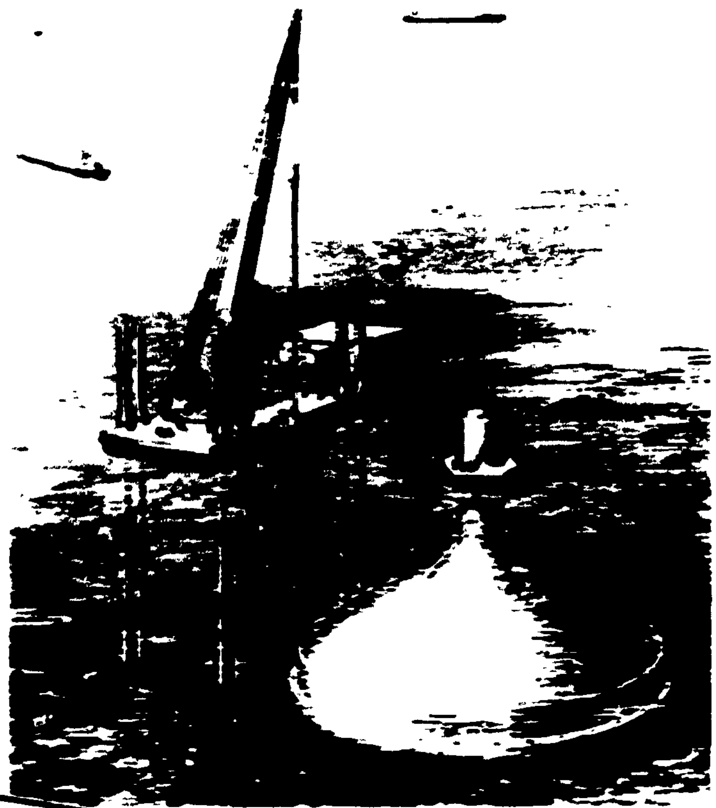
Step 1. Template launched and floating horizontally.
Step 2. Hook slings and pull 100 kips. Open flooding valves in top row of legs. Control flooding with air vent valves. Maintain approximately 10 kip hook load as template rotates.
Step 3. Stop flooding when template is vertical and still floating. Bring template on location and orient properly.
Step 4. Open all flooding valves and set template on bottom.

(Courtesy of American Society of Civil Engineers)

Fig. 63. Template installation sequence (from reference 157)

which pile driving operations are accomplished. In others, a derrick barge is used to facilitate the pile driving operation. Still others use a jack-up type barge or platform (e.g., a DeLong barge) to facilitate pile driving.[158] The completed piles are generally grout-connected to the substructure legs.

162. A jack-up rig was used in the installation of skirt piles around the Khazzan Dubai I, a 500,000-bbl storage tank located 60 miles offshore in the Arabian Gulf in 154 ft of water (fig. 64). Thirty 36-in.-diam by 90-ft-long skirt pipe piles were grouted into drilled 42-in.-diam holes.[159] The jack-up rig was so situated that it could drive six piles while at one location. The choice to use drilled grouted-in-place piles was made because of adverse soil conditions. The



(From Preprints, 1970 Offshore Technology Conference, 22-24 April 1970)

Fig. [illegible]. Placing piles for the Khazzan Dubai 1 underwater storage tank (from reference 158)

soil underlying the area to a depth of 120 ft consisted of a few feet of loose sand or mud overlying more or less cemented limestone (coquina) with interbedded layers of sand or silt. Pile pullout tests at a nearby site indicated an almost complete lack of soil adhesion to the piles until a depth of 180 ft below the sea floor was attained.[158,159]

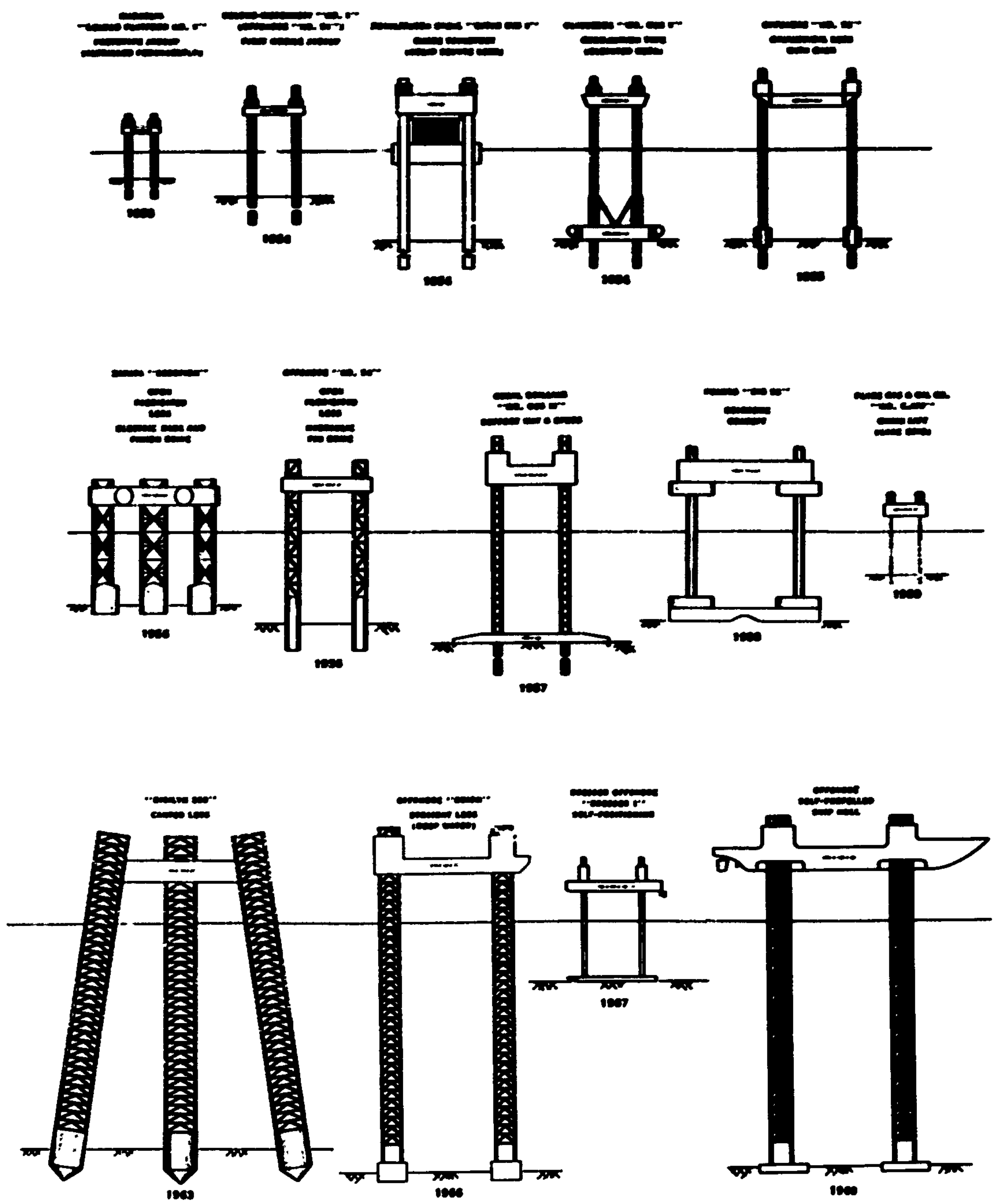
163. The tank has no bottom and operates on the water displacement principle. The piles are in compression when the tank is full of salt water and in tension when the tank is full of oil. For a 100-yr+ storm design, the estimated maximum forces on the structure are 36,500 kips downward when full of water and 33,500 kips upward when full of oil. The maximum horizontal force for the 100-yr+ storm design is 13,300 kips.[158] The piles were designed to support the entire weight of the structure with 5 ft of underscour below the tank walls. Movement of silt into the tank when the tank is discharging oil and taking in sea water would present a problem, and airlifting of sand or silt would be necessary should water flow be restricted or an excessive loss in capacity of the tank occur.

164. Legs of structures such as jack-up platforms are either of

the pile type, which attain adequate bearing capacity by penetration into the subsoil, or have enlarged bases, which must attain adequate bearing capacity with only nominal penetration. Fig. 65 shows the chronological development of jack-up rigs. The legs must support the weight of the structure, which is generally elevated above the worst expected sea state (about 35 ft above the sea surface in the Gulf of Mexico), and must withstand lateral forces of wind and any eccentric loadings imposed during operations or pullout of the legs when preparing to move off location. Experience has shown that jack-up rigs are most vulnerable to tipping over while moving on or off location.[148,149] Up to 1968, six rigs experienced failure due to inadequate soil bearing capacity and/or to mechanical failure of the legs while in the jacked-up position.[148]

165. Discussions with soil engineers and frequent statements in the literature disclose many uncertainties attending the design and installation of offshore piles. Uncertainties with respect to the behavior of heavily loaded large-diameter piles subjected to repetitive lateral loads of varying magnitudes force designers to specify conservative depths. Available soils data are usually from disturbed samples, and are sparse, and the designer must use such data conservatively. Development of adequate pile driving equipment has not met the need for driving piles which can carry increasing pile loads in offshore work.[153,160] In effect, overconservative design may lead to requirements for very large penetration, such that jetting and drilling may have to be done in advance of the pile; this results in additional uncertainties about the degree of soil distrubance caused by these procedures. Overconservative design resulting from the uncertainties mentioned can be both costly and time-consuming.[151]

166. Full-scale performance data on offshore piles are lacking in the literature and are greatly needed, since the extrapolation of onshore pile data to offshore work leaves much in doubt. Carefully planned investigations of the behavior of completed structures and programs of testing large offshore piles placed using the different available procedures and subjected to loads of the same order of magnitude as



(From Ocean Industry, Vol 3, No. 7, July 1968)

Fig. 65. Chronological development of jack-up rigs (from reference 148)

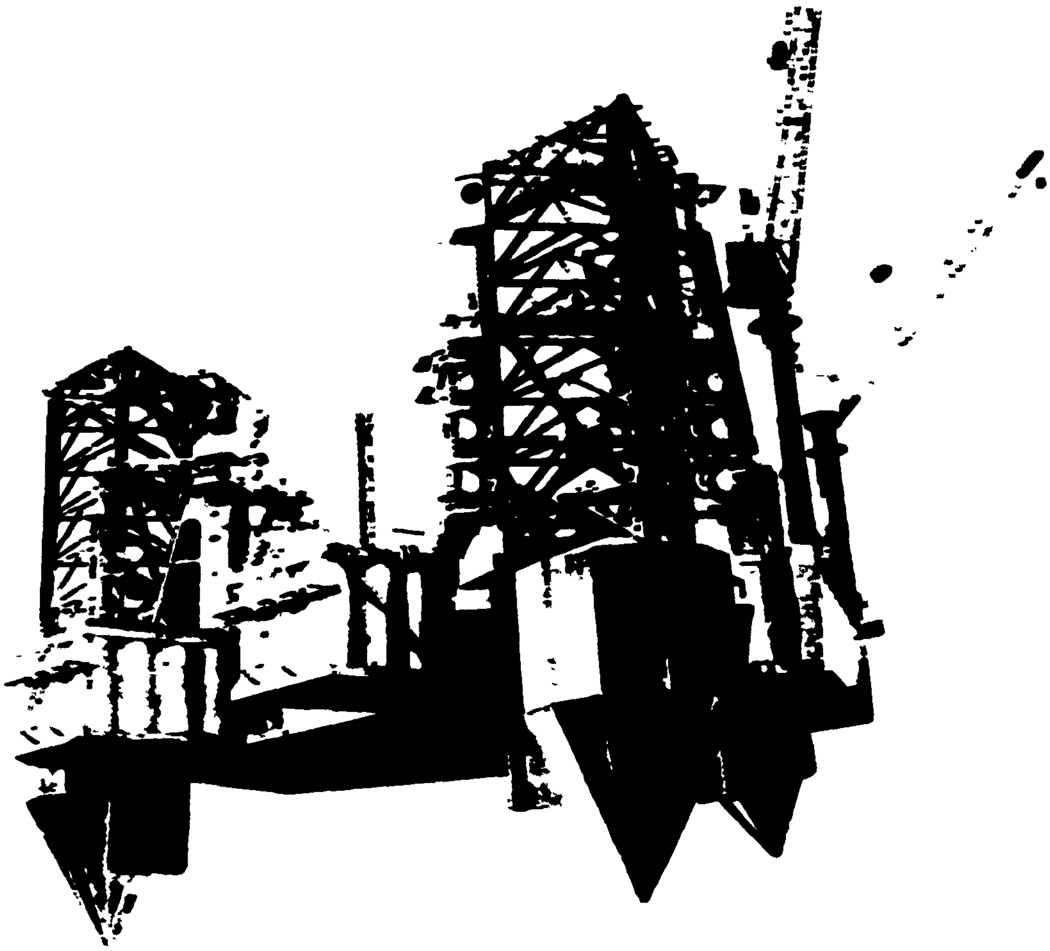
those encountered offshore would help minimize uncertainty and risk. Data of this sort have probably been developed in the petroleum industry, but are treated as proprietary, and, therefore, are not presently available for public use.

167. In addition to their use with offshore oil production and exploration platforms, piles are used in the construction of docking and mooring dolphins and artificial island transfer terminals in deep water and have been proposed as possible foundation elements for offshore airports.[20,124,125,129,155]

Mats and Footings

168. Mat or footing foundations are those in which the sea floor penetrations are quite small compared with the dimensions of the mat or footings. Some 30 percent of the mobile rigs in use in the Gulf of Mexico are partially mat-supported (fig. 65), and now even some permanent facilities rest on mat foundations, e.g., storage tanks.[161] In many instances, spud piles are installed below the mat to provide additional lateral resistance when shear resistance between the mat bottom and sea floor is considered insufficient. Protection against scouring is sometimes provided by placing riprap. Low resistance to lateral forces and vulnerability to scour are the main reasons that mats and other spread footings are not commonly used for permanent offshore installations. Without the supplemental use of piles, such foundations would not be able to resist uplift forces.

169. The legs of many mobile drilling rigs end in large-diameter, closed-bottom cylinders or tanks which function as large footings (figs. 65 and 66).[118,148] These footings decrease the penetration necessary to support the imposed loads. The footings must be designed to support the mobile platform on the weakest soils expected to be encountered at the various drilling sites. The number of footings per rig varies from 3 to 14, depending on the rig design, and footing diameters range from 16 to 45 ft. The geometry of these tanks varies from flat-bottomed, doughnut-shaped tanks near the end of the cylindrical pegs to

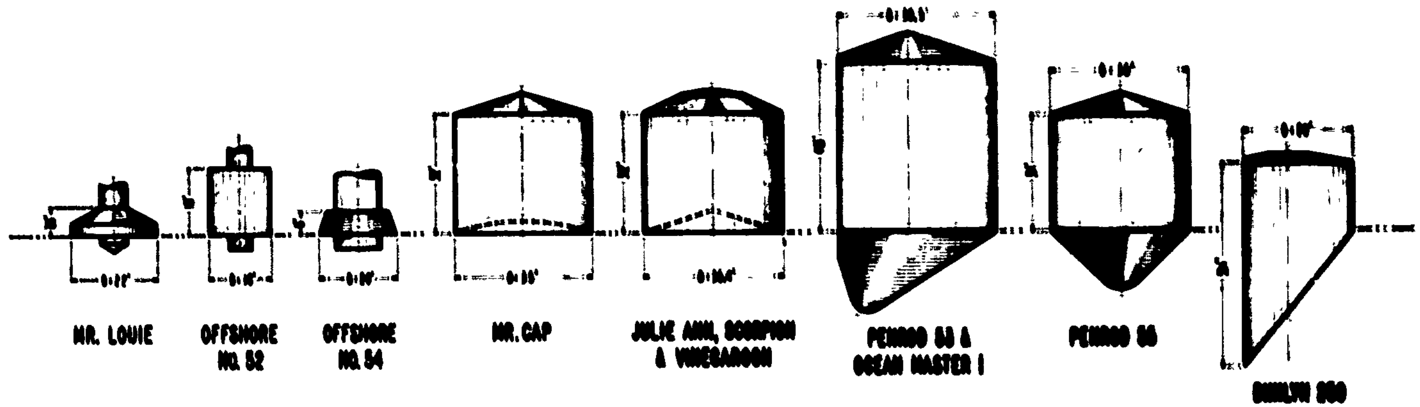


(Courtesy of Marathon LeTourneau Co.)

Fig. 66. LeTourneau three-legged jack-up rig

cylindrical cans with concave, conical, or truncated cylinder bottoms (fig. 67). Comparisons were made of the observed and theoretical performances of 11 jack-up rigs with individual tank-type footings at 120 clay bottom locations in the Gulf of Mexico (see fig. 42) in water depths from 80 to 300 ft.[118] The use of Skempton's equation for the bearing capacity coefficient, N'_c , and results of unconfined compression tests on good-quality soil samples gave reasonable predictions. The general bearing capacity equation for clays with $\phi = 0$ and Skempton's equation for N'_c are:[118,162]

$$q_d = cN'_c + \gamma'D$$



(Courtesy of American Society of Civil Engineers)

Fig. 67. Footing configurations (from reference 118)

and

$$N'_c = 6.0\ (1 + 0.2\ D/B),\ \text{respectively}$$

where

q_d = ultimate bearing capacity, psf

c = average cohesive shear strength of soil below footing level, psf

N'_c = dimensionless bearing capacity factor for circular footing areas on clay

γ' = submerged unit weight of soil, pcf

D = footing penetration, ft

B = footing diameter, ft

Skin friction on the sidewalls of the cylinders and the geometry of the surface of the footing base were not considered in this analysis because work done by other investigators had indicated that these factors are insignificant for the cylinder dimensions and bottom shapes involved.

170. Factors considered to affect the depth of penetration are the inclination of the legs, scour, and the presence of thin layers of sand or silt within the clay deposits. Penetrations of the footings of the mobile jack-up rigs studied reflect the equilibrium of structure loading with ultimate bearing capacity. Meyerhoff has shown that bearing capacity decreases slightly when the base of the footing is inclined.[118] The legs of the LeTourneau three-legged rigs can be inclined as much as 15 deg from the vertical. When the legs are inclined, deeper footing penetrations are required than when they are vertical. Scour can effectively reduce the depth of soil above the base of the footing (thus reducing the D/B ratio), causing a decrease in the bearing capacity and subsequent additional penetration of the footings. Thin sand and silt layers beneath the footings can contribute to the failure of some rigs. The presence of these layers may temporarily reduce actual penetration in comparison with that which would occur in a homogeneous clay foundation. A footing may punch through such layers under later increased loading after the drilling deck is jacked up into working

position.[113] The hazard can be reduced in some instances by applying temporary preloads in excess of the working loads to achieve penetration greater than that needed to support the rig in the jacked-up working position. One technique that has been used involves the temporary flooding of ballast chambers above the legs during jacking.

Dredging*

171. Dredging is performed (a) to make required excavations (for example, for buried pipelines or cables), (b) to remove materials unsuitable for foundations, and (c) to obtain material for fill construction. Dredging may involve (a) removal of bottom materials to a specified subgrade elevation or bottom condition, (b) cutting side slopes to specified angles, or (c) cleaning up soft bottom materials after primary dredging has been completed or as fill is placed.

172. Dredging in the United States for construction purposes has been done with (a) dipper dredges, (b) clamshell dredges, (c) hydraulic dredges, and (d), to a smaller but substantial extent, hopper dredges. Some work has also been done by dragline dredges. In other countries, ladder dredges are often used. A submersible crawler-type hydraulic suction dredge with cutterhead has recently been constructed; its designation is Crawl Cutter MKII, also called "Seapod."[164,165] A special suction dredge used at Akosombo Dam[166] dredged to depths of 226 ft in water as deep as 210 ft. The dredge was of the cutterhead type, supplemented by jet assists. Scows used to transport dredged materials are generally not self-propelled, although self-propelled hopper barges were used for Plover Cove Dam.[49]

173. While subsurface investigations of the site, as discussed in Part II, are necessary, the success of dredging operations depends more on visual observations and operational techniques than on control using conventional soil borings and tests. Some of the control factors in

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* Much of the material presented in this section is presented in reference 163.

dredging operations are given in table 12. When clamshell or dipper dredges are used, large blocks of cohesive soils are often obtained. Since some of the blocks are often relatively undisturbed, specimens can be obtained for unconfined compression tests for comparison with design boring and testing results.

174. Regardless of the purpose of dredging, frequent soundings to determine depths and side slopes obtained are necessary. For depth control as the work progresses, an ordinary recording-type echo depth sounder is adequate, especially one having a narrow dispersion of transmitted waves. This may not be satisfactory for payment purposes; therefore, lead line soundings are preferred for this purpose, with representatives of both owner and contractor present in the sounding boat. However, tidal or other currents may affect lead line soundings significantly, and it may be necessary to make soundings for payment purposes only at and near slack tide. Preconstruction soundings made in strong currents may show too deep a bottom, resulting in an apparent overrun of dredging quantities if postdredging surveys are made at slack tide.

Dredging to specified elevations
or to specified subsurface conditions

175. Dredging to specified elevations may not involve special inspection requirements for channel excavation, but this is not true when dredging for construction purposes. For example, in dipper dredging for Dry Dock No. 8 at the Norfolk Navy Yard, side decks of scows were at first washed with water prior to towing the loaded barges to sea for disposal of dredged material. This, combined with sedimented material resulting from dredging organic silt, caused accumulations of soft materials on the bottom which had to be removed by two suction dredges before a gravel layer, reinforcing forms, and tremie concrete could be placed. Accumulation of soft material should always be expected and should be investigated by divers, probings, or sampling. Inspection must ensure not only that unsuitable materials are entirely removed but also that excessive quantities of suitable materials are not removed. It is obviously impossible to remove all unsuitable materials without removing at least some suitable material, but good control will minimize excess removal of good material.

176. Effective control of dredging can generally be obtained by visual observations of dredged material by an inspector stationed at the dredge. This is generally true even if a hydraulic dredge is used because of differences in the sound and digging action of the dredge when passing from soft into firm materials. However, it may be desirable sometimes to have an inspector at the dredge discharge pipe with radio contact to the dredge.

177. In channel dredging, it is often permissible to dredge to steep slopes over a slightly wider area than needed and to allow the sides of the cut to assume their natural slope. However, where fill is to be placed in the excavation, this practice might result in unsuitable slide materials becoming mixed with fill materials on the cleaned-out bottom, and excavation to specified safe side slopes is desirable. It is practical to excavate slopes which are remarkably close to desired angles; however, since this type of dredging is not common, it is necessary for inspection forces to work closely with the dredge captain to secure the desired results. On dipper and clamshell dredging, specified side slopes can be achieved by controlling offset distances from the center line of the dredge and corresponding depth of excavation for various positions of the dredge.

Final cleaning of bottom

178. Where fill is to be placed after unsuitable materials have been removed, extreme care is necessary to ensure that soft materials do not remain after dredging and that soft materials do not accumulate at the toe of the advancing fill and become trapped in it. This requires dredging in several sequences such as (a) rough dredging to grade, (b) finish dredging to grade, and (c) supplementary or return dredging as fill is placed to remove accumulated fines.

179. Dredging generally causes fine materials to go into suspension and to escape back around the dredge into the dredged area. No matter how thoroughly unsuitable materials have been removed, some soft material can be expected to accumulate at the toe of advancing fills. In addition, even with relatively clean borrow materials, some fines will be washed out of the fill and accumulate ahead of it. Regardless

of the cause, accumulated fines must be periodically removed. This type of cleanup operation can best be done by small hydraulic dredges, but has also been successfully accomplished by clamshell dredging. For a highway embankment in a swamp at Wilmington, Del.,[167] simple probings were made using aluminum conduit as a sounding rod to determine if soft materials had been entirely removed; the limiting depth, with hand operations, was about 20 to 30 ft. Sampling with a dredge bucket is generally an effective construction control procedure. Unexcavated soft soil at Wilmington was displaced by advancing fill if its thickness was only a few inches, or at most a foot, but in almost all areas more effective removal was obtained during dredging and cleanup.

180. In some cases, e.g., Bremerton Dry Dock[168,169] and Plover Cove Dam,[49] a layer of suspended fines a few feet or many feet thick, remaining on the bottom after completion of dredging, could not be removed by sweeping the site with a sweep. In such cases, a gravel blanket 2 to 3 ft thick is often used to provide effective foundation-to-fill contact. This method, considered to be a last resort, should follow thorough cleaning and should not be used in lieu of thorough cleaning.

Underwater Fills*

181. There have been many nearshore operations involving underwater fills, such as land development projects, causeways, and jetties. Underwater construction of the lower portions of earth and rock-fill dams has become more prevalent in recent years, and some dams so constructed are listed in table 13.

182. A variety of equipment and procedures is generally possible in placing fill underwater and, at latter stages, above water, as indicated in table 14. Availability of equipment is often a controlling factor except on large jobs where specially designed equipment can be justified.

* Much of the material presented in this section is presented in reference 163.

183. Where fill is to be placed in a large area or in a dredged trench, lateral spreading of the fill is of little consequence if the bottom has been adequately cleaned. This is also true where fill is placed to have flat slopes and wide, level top surfaces. Under these conditions, bottom-dump scows are commonly used. On the work at Yonkers Sewage Treatment Plant (Hudson River), bottom-dump scow placement of clean sand fill resulted in underwater slopes of 1V on 5H to 1V on 11H, with the latter the more common. At the Southern Pacific Railroad crossing of Great Salt Lake in Utah, bottom-dump scow placement of a well-graded silty sand resulted in slopes with steepnesses varying inversely with water depths,[48,175-177] as summarized in table 15. Sands dumped suddenly from bottom-dump scows entrap air and appear to liquefy when they hit the bottom, whereupon they rapidly travel horizontally. This was observed at both Yonkers and at Great Salt Lake. At the Great Salt Lake crossing, sand traveled horizontally as much as 100 ft in 5 sec.

184. Steeper slopes can be obtained by placing fill materials slowly. Slopes of 1V on 2.75H were obtained at the Bremerton Dry Dock by hydraulically jetting materials off deck scows, and 1,300,000 cu yd of fill was placed in this manner. The fill was reasonably well graded from 3 in. down to a maximum of 10 percent finer than the No. 100 sieve. An alternative procedure for placing fill in shallow water is to unload deck scows using bulldozers, as done at the Great Salt Lake crossing, or using a clamshell. Deck scows are generally required when water depths are less than about 12 ft.

185. In constructing Plover Cove Main Dam underwater, the core (decomposed rock) was placed by clamshell and the bucket released just above the surface on which fill was being placed in order to minimize segregation. Shell material was dumped from bottom-discharge barges. Some redredging was necessary to remove pervious material from the core and fines from the shells. Maximum pore pressures in the core were about 53 percent of the weight of fill and dissipated in two months.

186. The construction of underwater rock dikes to retain fill materials is often necessary. Retaining dikes are commonly employed when

fill is placed by bottom-dump scows and must be retained in a restricted area. Retaining dikes also may be required to protect fill from erosion caused by (a) river or tidal currents, (b) ship movements, (c) waves, or (d) ice. In tidal areas and along rivers, ice may freeze around stone protection and shift the stones, indicating the need for larger stones than are needed in areas where ice is not a factor.

187. The construction of retaining dikes has been successfully accomplished on numerous occasions as, for example, at the Yonkers Sewage Treatment Plant on the Hudson River and at the Chesapeake Bay Bridge crossing near Annapolis. The upper portions of retaining dikes require filters, properly graded, on the fill side to avoid excessive loss of fill material into the retaining dikes and subsequent formation of sink holes on the fill surface, which may result from (a) tidal fluctuations, (b) wave action, or (c) rainfall and seepage from the fill into the stone dikes. Filters are normally provided only to a limited depth. Although filter layers on the fill side of retaining dikes have been used for a long time,[178] they have, unfortunately, been omitted on some projects. Contractor's claims for intrusion of fill material into the voids of stone retaining dikes can be avoided by fillside filter layers. Construction supervision and control should ensure that (a) the position and extent of stone dikes are known at all times, (b) stone dike material does not encroach upon possible work areas within the fill (especially where either sheet piling or bearing piles might be driven at a later date), and (c) required filter layers adjacent to dike material are actually in place.

188. While bottom-dump scow placement of rock for retaining dikes produces reasonably steep slopes, the slopes are much flatter than if placed by unloading rock by clamshell from deck barges. At the Great Salt Lake Crossing, bottom-dump scow placement of rock resulted in slopes that were dependent on water depth, as shown in table 16.

189. At Akosombo Dam,[166] rock fill placed underwater for cofferdams assumed a 1V on 1.25H slope; finer grained transition material on the upstream face assumed a 1V on 1.7H slope. Both the rock-fill and transition materials were placed by end dumping, the rock fill from

shore and the transition material from the top of the cofferdam.

190. Where rock is placed underwater and the voids are sluiced with sand to reduce compressibility and possible loss of material into the rock, a coefficient of "groutability" or "sluiceability" can be used. Such ratios can also be used in a qualitative sense when considering possible loss of fill into coarser materials.

191. The groutability ratio is defined in reference 179 as:

$$C_{gr} = \frac{D_{15} \text{ of coarse material}}{D_{85} \text{ of fine material}}$$

A ratio of 25 or more is required for the fine material to penetrate the coarser material, but this ratio has not been investigated intensively.

192. A coefficient of sluiceability developed for the High Aswan Dam[180] is defined as:

$$C_s = \frac{D_{10} \text{ of rock}}{D_{50} \text{ of sand}}$$

Sluiceability of sand into rock was dependent upon the mean rock size (see fig. 68). The extensive sluicing at this project provided valuable support for the use of fig. 68, but (presumably) values for C_s using other materials must be determined by tests.

Upgrading fill quality

193. For an interstate highway embankment in a swamp near Wilmington, Del.,[181] specification requirements for material after placement allowed a maximum of 8 percent finer than the No. 200 sieve, but the borrow area contained from 8 to 20 percent finer than this size. The borrow material was upgraded by dumping the material in a water-filled sheeted pit from which it was pumped to the work area by a 20-in. hydraulic dredge pump. An essential element of the operation was an 8-in. suction dredge located in front of the advancing fill operating continuously to remove fines carried in suspension beyond the satisfactory fill material.

194. The construction of hydraulic fill dams employs essentially the concepts described above for constructing pervious upstream and

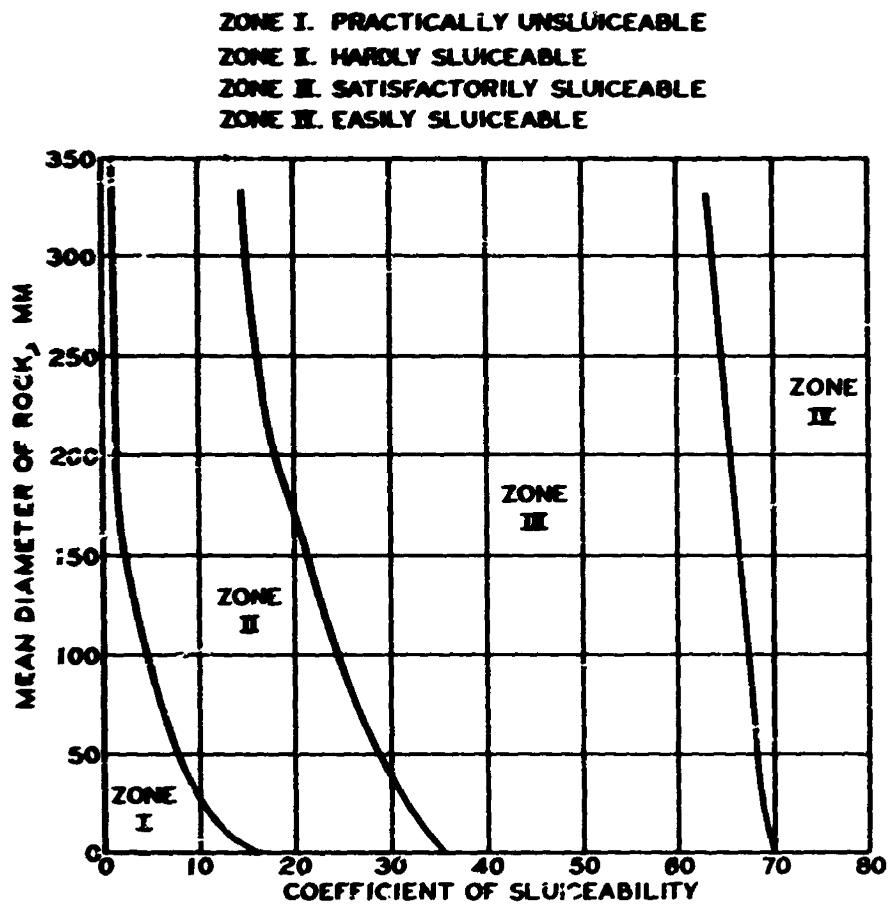

Fig. 68. Sluiceability of rock at Upper Aswan Dam

downstream shells. Experience developed in constructing such dams, summarized by Whitman,[182] indicates similar results as regards practicability of upgrading fill materials (also see reference 183).

195. A somewhat similar means for upgrading fill quality where truck haul is used is to end-dump the fill material, and suspended fines that are carried off ahead of the main portion of fill are removed by a small suction dredge. This requires careful control and virtually continuous dredging to remove the fines.

Underwater fill compaction

196. Clean sand fill materials placed underwater without compaction generally have moderate density, but relative densities of more than 50 to 60 percent should not be anticipated. While relative densities may average as high as 60 percent, they may range erratically from

10 to 70 percent. These may be adequate for highway embankments, industrial developments, and similar applications provided building or other loads are not heavy and that seismic or other dynamic loadings will not exist. Houses and other light buildings along the lakefront at New Orleans have behaved satisfactorily where supported on uncompacted hydraulic fill.

197. Compaction is necessary where underwater fill is to (a) become part of a dam, (b) be subjected to high surface loads, or (c) be subjected to seismic loadings for which consequences of liquefaction would be severe. For the few cases in which fills placed underwater have been compacted, fill materials were placed and then compacted in situ by deep vibratory methods such as vibroflotation or equivalent. This was done, for example, in cofferdams for the Bremerton Dry Dock[168] and for a dry dock at Baltimore,[184,185] and underwater for the High Aswan Dam.[171] Presently available vibratory methods for deep compaction include "Vibroflotation" and "Terra Probe." The former uses an eccentric weight vibrator that penetrates the soil; the latter uses a pile-driving vibrator clamped to a steel pipe shell which vibrates continuously as the pipe is driven into the soil to be compacted and is then withdrawn.

198. It has been proposed by Professor Arthur Casagrande that granular underwater fill be placed in layers and that each layer be compacted in a manner generally similar to that used in fill and compaction procedures on land. Professor Casagrande has designed a compactor (for which a patent application is in process) to compact rock fill in thick layers underwater in water depths of 300 ft or more. A method for constructing compacted island fills for bridge piers and other structures in water depths of 300 ft or more has been patented.[186] The feasibility of underwater surface compaction is suggested by the rapid development of underwater equipment, crawler and other types, and by experiences in compacting hydraulic fills as they are placed above water, as described below.

199. Although explosives have sometimes been used to compact saturated sand,[187,188] some engineers[189] believe that no adequate basis exists for evaluating the effectiveness of this method in reducing the

danger of liquefaction. Alternatively, other engineers consider compaction by explosives reliable, but only if properly planned, executed, and constantly supervised in the field by an engineer experienced with the method. A change in subsurface conditions may require immediate modification of an explosives compaction program. The concepts of in-place treatment of foundation soils, as reviewed by Mitchell,[190] apply also to underwater fills. This review includes a summary of cases in which deep vibratory and explosive methods have been used. The use of explosives appears attractive from the standpoint of cost, which is reported to be about half (or less) of that incurred using deep vibratory methods.[190] Deep vibratory methods appear less sensitive to changes in subsurface conditions such as thickness or density of loose soils.

200. The relative merits of vibroflotation versus compaction piles were investigated for a 30-ft sand fill at the Treasure Island Naval Station.[191] Both methods were effective but expensive; vibroflotation was apparently less costly and somewhat more effective. The effects of diameter of compaction pile at another site have been reported by Gupta.[192]

201. Where fill materials being used contain appreciable fines, neither vibratory, explosive, or compaction pile methods may be appropriate. These soils can be improved, however, by preloading, with or without vertical sand drains to accelerate consolidation. Such methods are discussed in references 167, 181, 182, and 193 and are basically no different from normal land operations.

202. Compaction of loose foundation materials underlying compact sands or clays may result in local voids at the contact between the loose and dense layers. This tendency may exist for all methods of underwater compaction.

203. Construction control for underwater compaction includes (a) measurement of settlement caused by compaction, (b) standard penetration or cone tests (or both) before and after compaction, and (c) undisturbed samples before and after compaction.

<u>Placement control</u>

204. Required construction control during fill placement must be

related to consequences of inadequate control, to the feasibility and cost of making corrective measures, and to time available. It is often assumed that dredging unsuitable materials and placing fill are relatively simple operations that are not demanding as regards inspection requirements. This is not correct, as demonstrated by numerous cases in which substantial quantities of soft materials have been entrapped within an advancing fill. Remedial operations in some cases have required the removal of all unsuitable materials and overlying good materials. At one job, 20 ft or more of good material overlying entrapped soft material was removed at the contractor's expense. In other cases, it has been possible to stabilize entrapped materials by installing vertical sand drains or surcharge fills to hasten consolidation. However, such procedures may be neither practicable nor desirable where the fill must have a high shear resistance. Therefore, to prevent costly remedial operations, continuous surveillance of the condition of the bottom should be maintained as fill material is placed.

205. Construction control considerations listed in table 12 are generally applicable for fill placement. Standard penetration test borings are generally sufficient, but undisturbed samples of sand can be obtained, where necessary, using procedures developed at WES.[65,69] The Swedish Foil Sampler was used successfully at the Great Salt Lake Crossing. As long as floating cranes or dredges are available, one of the most effective control measures is to obtain large samples with clamshell buckets. Small clamshell buckets operated by hand or from a small boat are useful. Recently developed samplers should also be useful for construction control. Simple procedures that can be followed by the contractor as part of his normal operations, using readily available equipment, are, of course, preferable to elaborate methods.

206. Check borings through the fill into original ground should be made as the work progresses to determine whether or not soft material is being entrapped. This may not be possible if slag fill is used. If slag fill is used, special care should be taken to ensure that all soft materials have been removed. This requires constant inspection and frequent sampling at the bottom. Probings are effective in shallow water

but not in water deeper than about 30 to 40 ft. Two-in.-diam aluminum conduit tubing has been used as a probe. When it could not detect a buildup of soft material at the toe of the fill, a clayey skin was left on the surface of the tubing that indicated the presence of soft material. Samples obtained with pipes driven into firm materials by the contractor are often sufficient. Recording depth sounders are very useful in showing mud wave accumulations at the toe of advancing fills.

207. When fill is being placed, special requirements include (a) removing partially buoyant or sunken debris that might be entrapped in the fill, (b) checking excavating dredge discharge pipe joints to avoid leakage and contamination of previously placed material, and (c) providing berms, baffle boards, and bulldozer channels at the discharge end of the dredge pipe to force water and suspended fines off and ahead of the area being filled in order that the fines can be removed by a small silt-removal dredge or carried outside the work area.

208. Permanent bench marks located outside the work area should be required, and surface settlement plates inside the work area should be provided so that payment can be made for fill material "lost" by foundation settlement during construction.

209. Divers are generally effective but expensive and are employed for fill placement control only when the stability and proper functioning of important structures must be ensured beyond reasonable doubt. Divers were used, for example, in the underwater construction phase at the Hugh Keenleyside (Arrow) Dam. Construction control procedures for this project are thoroughly described by Bazett.[170] Divers also inspected dredged areas at Akosombo Dam before rock fill was dumped for underwater construction of cofferdams. The current trend to construct cofferdams or lower portions of dams underwater will probably account for more cases in which divers are routinely employed for dredging and earth-fill placement control. Some engineers question the need for diver inspections and point out that poor visibility and other conditions under which a diver works restrict his usefulness and reliability.

Pollution control

210. Pollution of water from any source is receiving increasing

attention and must be considered when planning dredging and filling operations. Some examples are (a) avoidance of pollution that might have affected oyster culture in dredging for Plover Cove Dam in Hong Kong,[194] (b) preservation of a clean water supply for use by a pulp mill adjacent to the Hugh Keenleyside (Arrow) Dam,[170] and (c) reduction of turbidity downstream from hydraulic dredging operations for a causeway in Florida.[195] At the Florida project, an inexpensive untreated canvas filter, later replaced by a plastic screen, was suspended from lobster trap buoys and weighted at the bottom to trap suspended sediments. Turbidity was reduced from 2100 to 33 ppm, well below the level of 50 required by the Florida Air and Water Pollution Control Board.

211. Cleaning side decks of scows by water from the dredge can result in water pollution and deposit excess fines in the work area or elsewhere. Therefore, this practice should be prohibited.

Summary

212. Marine foundations support temporary or permanent, manned or unmanned structures. Foundation elements include anchors, piles, footings, and mats. Performance data on full-scale large offshore foundations are lacking. Construction equipment and technology are hard pressed to meet the requirements arising from operations in deeper and deeper waters. Many uncertainties exist in design input and construction capabilities.

213. Offshore foundation failures have occurred. Anchorages have failed due to excessive settlement of deadweight anchors into the sea floor and to rotation and pullout of fluke-type anchors. Bearing capacities of foundation soils have been exceeded, with the result that mat-supported structures have overturned and piles or spread footings have punched through layers on which they were founded. Sliding and lateral displacement of mat-type foundations have been caused by lateral forces generated by wave, current, tide, and wind action. All of these indicate the need for continued research and surveillance in the field of ocean soils engineering. A need exists for full-scale test data and

prototype performance data for all foundation types, i.e., stress and deformations in the soils associated with all types of anchors, piles, mats, and footings, or beneath embankments at sea. Preliminary physical and analytical models may point up the major factors causing foundation failures and will be helpful in developing full-scale test programs. Load test procedures are needed to test various foundation soil combinations on the sea floor to their ultimate capacities.

214. Construction technology and equipment capabilities require close study to develop capabilities of placing foundation elements more efficiently and at the proper lateral and vertical locations. Pile driving hammers of higher energy and efficiency are needed. Continued studies are needed to relate the hammer driving capabilities to pile and soil characteristics and attainable penetrations. Procedures for embedment of anchors at specific locations and depths need improvement. The blind dragging of anchors to cause embedment and then proof testing do not reveal the location of the anchor or its attitude with respect to the sea floor and the structure.

215. Studies arising from the nation's increasing concern over the environmental impact of dredging and spoil disposal, while primarily directed at the environmental aspects of polluted spoil and damage to estuarine and coastal areas by unpolluted spoil, will undoubtedly result in improvements in dredging equipment and operations from the engineering standpoint. The Corps of Engineers has initiated a comprehensive program of research, study, and experimentation relating to dredged spoil under the authority of the River and Harbor Act of 1970 (Public Law 91-611, approved 31 December 1970). Some of the research topics being considered are the development or enhancement of land using dredged spoil, and investigation of in-place spoil improvement (engineering characteristics) techniques using physical, chemical, and biological methods. Such studies will advance engineering design and construction capabilities for dredging and underwater fill operations.

PART VI: RECOMMENDATIONS

216. In the following paragraphs, recommendations are made for research to advance the state-of-the-art of marine soil mechanics. Obviously, parallel research has been and is being conducted on land, but the results of such research cannot often be directly extrapolated into the ocean environment. If the engineering profession is to minimize the uncertainties in design of offshore foundations, it must have information which is directly derived from research accomplished on marine soils and in the marine environment or in an environment in which conditions approach sea floor conditions.

Exploration

217. Improvement of apparatus and procedures to obtain high-quality undisturbed samples in water depths up to 600 ft should be continued. The problems of obtaining continuous undisturbed samples and attaining reentry into the drill hole inexpensively still have not been solved for water depths too great to permit use of jack-up barges. Sampling with single-entry drive and bottom-rest, repeated-entry samplers generally is less expensive and time-consuming than drilling from ships or jack-up rigs but lacks the capability of sampling at depth. However, these samplers are useful in shallow foundation exploration and in preliminary site exploration. Continued research and development are needed to minimize disturbance of samples obtained by these samplers and to increase the samplers' penetration capabilities.

218. Sampling of cohesionless soils is difficult in present sampling systems. The distance of travel through water from sea floor to sea surface and the difficulties involved in the procedure[65] using drilling fluid for obtaining undisturbed samples of saturated sand in great depths of water contribute to the problem of (a) retaining sand samples, and (b) keeping them in an undisturbed state. Research should be conducted to develop sampling procedures for obtaining undisturbed cohesionless soil samples from sea floors.

219. In the past, researchers have used supplemental energy sources and various means of reducing friction in the sampling systems to achieve improvements. These efforts should be continued with more in-depth investigation of (a) the use of propellants to provide a steady controlled drive on a piston-type sampler, (b) the use of lubricants which are either applied prior to the lowering of a sampler or are already an integral part of the sampling system, (c) the use of electro-osmosis to reduce soil-sampler wall interaction, and (d) the use of flexible liners which can both reduce friction and enhance sample retainment capabilities. All of these efforts should be correlated with determinations of engineering properties and supplemented by a study of how to measure disturbance and its effects on engineering properties.

220. The engineering usage of geophysical exploration techniques is a necessary adjunct to physical sampling of the sea floor. Greater involvement of the soils engineering profession is needed in the interpretation of geophysical output to obtain data meaningful to the design engineer. Further research should be conducted in relating geophysical data on ocean soils to their engineering properties. A primary problem is that of obtaining data of sufficient resolution to indicate differences in the engineering properties of the soils. Establishment of relationships of velocity and impedance to void ratio, unit weight, plasticity characteristics, shear strength, and other engineering soil characteristics would be extremely useful in preliminary surveys for selecting sites for construction.

221. It is necessary to do the above research from the perspective that the base of operations may be either at the sea surface, beneath the sea surface, or on the sea floor. Advantages and disadvantages exist in executing an exploration survey from any one base, both technically and economically, and these alternatives must also be evaluated and improved.

Laboratory Testing

222. The effects of changes in ambient conditions on a marine

soil when it is taken from its sea floor environment and tested under laboratory conditions must be evaluated to arrive at quantitative relationships which can be used in making best estimates of in situ properties. For example, the influences of dissolved air coming out of solution and of temperature changes must be evaluated.

223. Standard soil testing procedures for the determination of engineering properties of marine soils should be evaluated with respect to their applicability. The use of distilled water in testing should be examined to determine its effects on results of standard tests performed on marine soils. The effects of dilution of pore water in marine soil samples should be evaluated. Salt water of a certain salinity may need to be used instead of distilled water in laboratory testing to produce dependable values of in situ properties. The objective should be to develop a set of standard procedures which take into account the marine environment in the testing of marine sediments.

In Situ Testing

224. Continued effort must be made to improve and evaluate in situ testing. This includes the reduction in costs of engineer tests as well as the interpretation of the results in a manner usable by the design engineer. Studies similar to the in situ vane testing in deep waters by Fenske[102] must be accomplished for other continental shelves of the U. S. and including the use of other in situ test apparatus, e.g., the cone penetrometer, downhole pressure meters, and various borehole logging systems (gamma ray, electrical resistivity, and pore pressure). The use of accelerometers on free-falling or propelled probes and samplers appears to have merit and should be investigated along with the possible use of high-velocity-type penetrometers. In all cases, the in situ measurements must be effectively correlated with engineering properties.

Soil Properties

225. A more extensive investigation of existing and developing

soil data would be worthwhile from the viewpoint of extrapolating engineering characteristics, collating findings, and characterizing various geologic areas of the U. S. continental shelf with respect to these characteristics. This will provide a better understanding of the significant features of upper, middle, and lower latitude shelf soils or of west, east, Arctic, and Gulf of Mexico shelf soils similar to the general categorization of terrestrial soils as, for example, glaciated soils of northern U. S., arid soils of southwestern U. S., tundra of Alaska, and permafrost soils of Alaska and Canada. These terms suggest characteristics associated with them through experience which are useful in estimating engineering properties. Because of the high cost of determining engineering soil properties of subbottom ocean soils, a complete inventory of all explorations and test results should be established and periodic summaries and correlations prepared. Such a data bank should be computerized.

226. With the accumulation of further experience and field and laboratory test results, consideration might be given to the preparation of a report relating the Unified Soil Classification System to the characteristics of soils found in the sea floor with respect to underwater fills and various types of foundations.

227. Research should be continued to establish useful correlations of physical properties of fine-grained marine soils (such as shear strength) with index properties such as Atterberg limits and grain-size data. Extension and expansion of studies similar to McClelland's study[91] on consolidation of delta and prodelta clays and to Sherman and Hadjidakis' study[196] on Mississippi Valley fine-grained alluvial soils are examples.

228. The influence of cyclic changes of pore water pressure due to normal wave actions should be investigated, particularly with reference to metastable and underconsolidated low-strength marine sediments. The additional influence of hurricane waves or setup in the Gulf of Mexico and on the Atlantic Coast and of tsunamis off the Pacific Coast on pore water pressures in marine soils should also be studied. Such research could possibly explain the mechanisms creating turbidity currents,

sand flows, and liquefaction on continental shelves and slopes.

229. Scour is directly related to grain size and density in existing bedload and transport equations. Since scour is a common occurrence, research should be initiated to study the soil-water-foundation interaction from a soil mechanics viewpoint. The effect of shear strength on the scourability of various soil foundation combinations is an area of needed research.

Foundations

230. The contribution of soil to the overall resistance of an embedded anchor to pullout should be further investigated. (Work along these lines is being done by NCEL.) The failure surfaces in soils for various embedments of mushroom anchors have been determined and the soil resistance evaluated, but the work should be extended to involve more prototype studies. Similar studies are needed for other anchor types, i.e., drag, free-fall and propelled embedment, and pile anchors. The demand for greater anchor holding power to restrain larger floating offshore structural features in deeper waters requires more efficient design and usage of anchorage systems, including the coupling of the anchor configuration with the properties of the soil. Precision placement of marine anchors also needs study; haphazard embedment of an anchor by dragging leaves in question the depth of embedment, the attitude of the flukes and shank with respect to the bottom, and the location of the anchor relative to that of the floating structure. Jetting to depth or excavation, placement, and then backfill are positive control means, but these operations require evaluations both as to their effect on the holding power of the anchor and the feasibility of such placement methods.

231. Means of determining the resistance of the sea floor to loads transmitted to it through pile foundations should be investigated. The applicability of design equations and associated coefficients for skin friction and end bearing of piles commonly in use in onshore work should be investigated for the large-diameter, long-length, and heavily loaded offshore piles being used. Soil-pile interreaction to extremely high axial and lateral loads, as well as the effects of cyclic loadings

due to sea motion, requires more study. Load tests to determine ultimate capacities are needed. Data should be obtained on the performance of prototype pile foundations which have been subjected to large loadings. Such data could be obtained by instrumenting offshore piles to observe their behavior in the variety of soils which exist on the continental shelf, including not only individual test piles but also pile groups in actual platforms. A cooperative program between the Corps of Engineers, the oil industry, and other agencies would have much merit. Installation of pressure cells and deflection or strain indicators on piles and in adjacent soils to monitor the soil-pile interaction should develop some new capabilities in the state-of-the-art of instrumentation in the sea floor.

232. The design of mats and footings requires the proper knowledge of the shear strength characteristics of the soil, and recommendations pertaining to this have already been given under the subheading of soil properties. Research is needed on the resistance to lateral displacement developed between mat-type foundations of relatively small penetration and the sea floor. Though raft and mat-type foundations have not been used much, they are potentially inexpensive foundations which can distribute bearing loads sufficiently to support many structures on the sea floor surface.

233. The proper design of anchors, piles, mats, and footings to minimize scour requires the study of soil-water-foundation interaction. Feasible means of minimizing scour should be developed. The placement of graded filters may not be possible because of uncertainties associated with their construction in deep waters.

234. Other areas also require extensive study. These include foundations for pipelines to transport offshore oil production or perhaps to transmit fresh water along the shelf and foundations for utility-type transmission cables. The stability of flat natural slopes comprised of metastable and/or very soft underconsolidated soils requires investigation. This is particularly true because the current trend is to operate in deeper and deeper water offshore. The soils near the outer edge of the Gulf of Mexico continental shelf are of the weak

underconsolidated type, and the associated bathymetry in the area implies past slope failures.[120] Cyclic changes in pore pressures with the passage of waves over the sediments and their attendant influence on the stability of these natural deposits may explain some of the mystery which surrounds the development of turbidity currents.[197] A study to evaluate protective armor for fill slopes in the open sea is necessary to properly design these protective works. Alternatives to protective armor must also be developed. Excavation required for site preparation and the construction of underwater fills of soil or rock in the marine environment also require further study.

Summary

235. This state-of-the-art report covers in a broad overview the current position of marine soil mechanics and foundation engineering in water depths less than 600 ft.

236. A primary difference between marine and onshore soil mechanics and foundation engineering is the hostile and sometimes unpredictable marine environment with its deep waters and wind, wave, current, and ice forces. Present design procedures can take these forces into account, although the state-of-the-art on them needs much improving. For the soils engineer, the determination of the characteristics of the ocean bottom, evaluation of the forces that will be imposed on the soil, and actual construction on the sea floor within this hostile environment present primary challenges. It is found that improvements are needed in many phases of offshore soil mechanics work (i.e., in exploration, testing, foundation performance, and foundation design) and in evaluating the effects of the ocean environment on all these phases. Performance research, construction innovations, and cumulative experience from working at sea will all contribute to the future reduction in the cost and risk of offshore work.

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Table 1

Underwater Construction Engineering

Types of Structures and Uses

Types of Structures or Foundations	Typical Uses
Structural platforms consisting of fixed (pile or pier foundations) or floating (anchored) structures or foundations	Oil drilling Navigational aids Commodity handling Heliports and airports Oil storage Quays Causeways Wharves Bridges Buildings
Mat or footing foundations	Bridge piers Dry docks (tremie construction) Water intakes Quay walls Oil drilling
Earth fills and retaining dikes	Dams Cofferdams Causeways Jetties Offshore islands: Oil drilling Commodity handling Airfields Land development
Excavations	Channels Mining Borrow uses Aggregate sources
Underwater structures	Tunnels, floating Tunnels, cut and cover Pipelines

Table 2

Summary of Underwater Construction Experience

Factor	Construction-Inspection Experiences
Water depth, ft	
0-100	Range of frequent underwater construction operations
>100-400	Construction experiences mostly with oil drilling platforms, some with sewer outfalls
>400	Little or no experience. Projects are mainly speculative and subjects of research study
Wave action	Extensive experience with large ocean waves limited to: 1. Oil drilling platforms 2. Navigation and radar platforms 3. Unloading terminals 4. Bridge crossings 5. Island construction 6. Breakwaters
Pile driving	
Submerged piling	Extensive underwater pile driving experience. In WW II dry dock construction, more than 20,000 H-piles driven with tops 3 ft above subgrade in 70 ft of water
Large-diameter cylinders	Extensive experience in tower construction. Diameters to 6 ft, lengths of 400 to 600 ft, and loads to 3500 tons
Tremie concrete placement	Extensive experience in WW II dry dock construction (200,000 cu yd per dock) and on numerous construction projects. Water depths to 70 ft and greater
Dredging	Dipper dredging to 70-ft water depth; clamshell dredging to 80-100 ft; suction dredging to 200 ft. Controlled side slopes possible
Fill placement	Much experience in water depths to 80 ft; some experience to 200 ft
Prefabricated structures	Extensive experience on: 1. Drilling platforms 2. Prefabricated floor and wall trusses consisting of tremie concrete forms and reinforcing steel, from WW II dry dock construction 3. Bridge crossings 4. Ocean floor storage tanks 5. Offshore terminals 6. Tunnels and pipelines
Currents	Velocities of 6-8 knots encountered fairly often; higher velocities in river closures

Table 3

Characteristics of Two Surficial Grab and Two Surficial Scoop Samplers*

Type and Name	Characteristics
Surficial grab	
Orange peel	Four jaws close and take a hemispherical bite of the the sea floor. Samplers up to 1-cu-ft capacity and weighing 330 lb with a 22-in. diameter are available. Fig. 2 shows U. S. NOO 45-lb-capacity orange peel bucket which takes a 0.15-cu-ft sample. Canvas cover sometimes used to prevent sample washing
Van Veen	Two jaws are levered shut with long bars. U. S. NOO Van Veen grab shown in fig. 3 uses a trigger system to hold jaws open during descent. Contact of the weight with the bottom triggers the grab for closing. The U. S. NOO Van Veen takes a 0.15-cu-ft sample
Surficial scoop	
Pipe dredge	A length of metal pipe with the front end sharpened for cutting into rock and a grating at the aft end for trapping rock fragments while permitting fine sediment to pass through. The scoop shown in fig. 4a is 6-1/2 ft long and 1-1/2 ft in diameter
Frame or chain dredge	Steel mesh bag is attached to rear end of metal frame for trapping rock fragments entering through open front end. Front edges are sharpened for rock cutting. Front-end opening varies in size from 6-1/2 by 1-1/2 ft to 3 by 1 ft. Chain mesh bags have lengths of 5 to 10 ft (see fig. 4b)

* Additional information on surficial grab or scoop samplers is given in references 14, 18, 19, and 20.

Table 4

Characteristics of Marine Corers (Tube Samplers)

Identification	Inside Diameter in.	Length ft	Ratios* C_i, %	C_o, %	C_a, %	Reference
Kullenberg piston	1.9	6**	7.6	13.6	143.8	31
Hydroplastic (USHO)	3.2	8	1.6	13.4	56.8	29
Hydroplastic (Richards)	4.2	9	0	0	13	7
Hydroplastic (McManus)	6.0	20	1.4	3.5	31.6	30
USNEL spade corer	8 by 12	2	0	0	2.7	11
Boomerang	2.6	4	3.2	4.0	69.2	32
Univ. of Wash. piston corer	2.6	20**	2.2	13.5	137.8	Author†
Univ. of Wash. pilot corer	1.9	5**	5.4	22.2	123.4	Author†
USNEL Ewing	2.4	8**	1.9	18.2	97.9	33
NGI gas-operated	2.1	5.4	0.9	0	12	34
USGS-WES vibratory	1.9	11	0	0	29	WES Shop prints
Mackereth	1.6	20	0	0	51.5	35
Hvorslev's criteria[6] for comparison with above						
Short samplers (length < about 2 to 3 diameters)	--	--	0-0.5	--	--	--
Long samplers (length > about 8 to 10 diameters)	--	--	0.75-1.5	--	--	--
Cohesionless soils	--	--	--	0	--	--
Cohesive soils	--	--	--	<2-3††	--	--
General	--	--	--	--	<10	--

* Defined in fig. 1.
** Lengths may be increased by adding barrel extensions.
† From author's class notes taken in course at Univ. of Wash.
†† Unless taper of cutting edge α is very small.

Table 5

Ranges of Dimensions for Marine Sediment Tube-Type Samplers

	Gravity Sampler Min	Gravity Sampler Max	Piston Sampler Min	Piston Sampler Max
Diameter, D_s*	1 in.	12 in.	1.5 in.	6 in.
Length, L*	12 in.	15 ft	5 ft	80 ft
Maximum safe length, L_s, based on Hvorslev's criteria (see fig. 1)				
Cohesionless soil $\frac{L_s}{D_s} = 10$	10 in.	10 ft	15 in.	5 ft
Cohesive soil $\frac{L_s}{D_s} = 20$	20 in.	20 ft	30 in.	10 ft

* Data from reference 7.

Table 6

Characteristics of Acoustical Systems*

Description (Type, Manufacturer, Model)	Frequency kcps	Power joules	Resolution, ft	Depth of Penetration ft
Piezoelectric transducers				
Edo Western Corp				
Model 185	12.0	--	2-3	120-150
Model 415	7.0 or 3.5	--	0.5-1.5	30-150
EG&G International, Inc.				
Pinger probes	12.0 or 5.0	--	<1	45-90
Electromechanical transducers				
EG&G International, Inc.				
High-resolution boomer	1.2-0.8	200-500	±1	20-200
Standard boomer	0.6-0.04	13,000 (max)	±10	1,000 (max)
Sparkers				
Alpine Geophysical Assoc., Inc.				
Sparker (a)	5.0-0.5	50	5-15	300-400
Sparker (b)	0.6-0.25	100-400	20-25	1,200
EG&G International, Inc.				
Sparkarray	0.120-0.08	105,000 (max)	50-100	7,000+
Huntec Limited				
Mark 2A Hydrosonde System	2.3-0.1	--	--	150
Teledyne Industries				
Subot System	1.0-0.125	5,000 (max)	10-50	300-2,000
SSP System	0.125-0.02	160,000 (max)	--	15,000

* Information was obtained from table 1 of reference 73.

Table 7

Grain-Size Scales*

Wentworth Scale (Size Description)		Phi Units ∅**	Grain Diameter D, mm	U. S. Standard Sieve Size	USCS (Size Description)	
Boulder						
		-8	256		Cobble	
Cobble			76.2	3 in.		
		-6	64.0		Coarse	
			19.0	3/4 in.		Gravel
Pebble					Fine	
			4.76	No. 4		
		-2	4.0		Coarse	
Granule						
		-1	2.0	No. 10		
	Very coarse					
		0	1.0			
	Coarse				Medium	Sand
		1	0.5			
	Medium		0.42	No. 40		
Sand						
		2	0.25			
	Fine					
		3	0.125		Fine	
	Very fine		0.074	No. 200		
		4	0.0625			
Silt						
		8	0.00391			
Clay					Silt or clay	
		12	0.00024			
Colloid						

* Information was obtained from references 18 and 82.

** $\phi = -\log_2 D$.

Table 8*

Major Divisions			Group Symbols	Typical Names	Field Identification Procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights)			Information Required for Describing Soils
1	2		3	4	5			6
Coarse-grained Soils. More than half of material is larger than No. 200 sieve size. The No. 200 sieve size is about the smallest particle visible to the naked eye.	Gravels. More than half of coarse fraction is larger than No. 4 sieve size. (For visual classification, the 1/4-in. size may be used as equivalent to the No. 4 sieve size)	Clean Gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.			For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions, and drainage characteristics.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.			
		Gravels with Fines (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).			Give typical name; indicate approximate percentages of sand and gravel, max. size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses.
			GC	Clayey gravels, gravel-sand-clay mixtures.	Plastic fines (for identification procedures see CL below).			
	Sands. More than half of coarse fraction is smaller than No. 4 sieve size.	Clean Sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines.	Wide range in grain size and substantial amounts of all intermediate particle sizes.			
			SP	Poorly graded sands or gravelly sands, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.			Example: Silty sand, gravelly; about 20% hard, angular gravel particles 1/2-in. maximum size; rounded and subangular sand grains, coarse to fine; about [illegible] nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM).
		Sands with Fines (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).			
			SC	Clayey sands, sand-clay mixtures.	Plastic fines (for identification procedures see CL below).			
Fine-grained Soils. More than half of material is smaller than No. 200 sieve size. The No. 200 sieve size is about the smallest particle visible to the naked eye.					Identification Procedures on Fraction Smaller than No. 40 Sieve Size			
					Dry Strength (Crushing characteristics)	Dilatancy (Reaction to shaking)	Toughness (Consistency near PL)	
	Silts and Clays. Liquid limit is less than 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.	None to slight	Quick to slow	None	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Medium to high	None to very slow	Medium	
			OL	Organic silts and organic silty clays of low plasticity.	Slight to medium	Slow	Slight	Give typical name; indicate degree and character of plasticity; amount and maximum size of coarse grains; color in wet condition; odor, if any; local or geologic name and other pertinent descriptive information; and symbol in parentheses.
	Silts and Clays. Liquid limit is greater than 50		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	Slight to medium	Slow to none	Slight to medium	
			CH	Inorganic clays of high plasticity, fat clays.	High to very high	None	High	
			OH	Organic clays of medium to high plasticity, organic silts.	Medium to high	None to very slow	Slight to medium	Example: Clayey silt, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML).
Highly Organic Soils			Pt	Peat and other highly organic soils.	Readily identified by color, odor, spongy feel and frequently by fibrous texture.			

(1) Boundary classifications: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well-graded gravel-sand mi[illegible]

FIELD IDENTIFICATION PROCEDURES FOR FINE-GRAINED SOILS OR FRACTIONS

These procedures are to be performed on the minus No. 40 sieve size particles, approximately 1/64 in. For fi[illegible] screening is not intended, simply remove by hand the coarse particles that interfere with th[illegible]

Dilatancy (reaction to shaking)

After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens, and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.

Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

Dry Strength (crushing characteristics)

After removing particles larger than No. 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air-drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

* From reference 82.

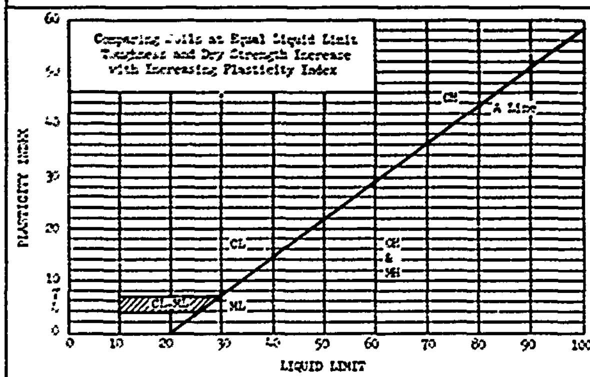
Table 8*

UNIFIED SOIL CLASSIFICATION
(Including Identification and Description)

	Field Identification Procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights)			Information Required for Describing Soils	Laboratory Classification Criteria	
	[illegible]			[illegible]	7	
... mixtures.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.			For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions, and drainage characteristics.	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
... sand mixtures,	Predominantly one size or a range of sizes with some intermediate sizes missing.				Not meeting all gradation requirements for GW	
... mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).			Give typical name; indicate approximate percentages of sand and gravel, maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses.	Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
... mixtures.	Plastic fines (for identification procedures see CL below).				Atterberg limits above "A" line with PI greater than 7	
..., little or	Wide range in grain size and substantial amounts of all intermediate particle sizes.				$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
... sands, little	Predominantly one size or a range of sizes with some intermediate sizes missing.			Example: Silty sand, gravelly; about 20% hard, angular gravel particles 1/2-in. maximum size; rounded and subangular sand grains, coarse to fine; about 15% nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM).	Not meeting all gradation requirements for SW	
...	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).				Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
...	Plastic fines (for identification procedures see CL below).				Atterberg limits above "A" line with PI greater than 7	
	Identification Procedures on Fraction Smaller than No. 40 Sieve Size					
	Dry Strength (Crushing characteristics)	Dilatancy (Reaction to shaking)	Toughness (Consistency near PL)			
...s, rock ...s or ...ity.	None to slight	Quick to slow	None	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.		
... plasticity, ...ty clays,	Medium to high	None to very slow	Medium			
...ays of low	Slight to medium	Slow	Slight	Give typical name; indicate degree and character of plasticity; amount and maximum size of coarse grains; color in wet condition; odor, if any; local or geologic name and other pertinent descriptive information; and symbol in parentheses.		
...aceous ...ic silts.	Slight to medium	Slow to none	Slight to medium			
..., fat clays.	High to very high	None	High			
...lasticity,	Medium to high	None to very slow	Slight to medium	Example: Clayey silt, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML).		
...s.	Readily identified by color, odor, spongy feel and frequently by fibrous texture.					

Use grain-size curve in identifying the fractions as given under field identification.

Determine percentages of gravel and sand from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse-grained soils are classified as follows:
Less than 5% GW, GP, SW, SP.
More than 12% GM, GC, SM, SC.
5% to 12% Borderline cases requiring use of dual symbols.



PLASTICITY CHART

For laboratory classification of fine-grained soils

...s are designated by combinations of group symbols. For example GW-GC, well-graded gravel-sand mixture with clay binder. (2) All sieve sizes on this chart are U. S. standard.

FIELD IDENTIFICATION PROCEDURES FOR FINE-GRAINED SOILS OR FRACTIONS

...dures are to be performed on the minus No. 40 sieve size particles, approximately 1/64 in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

...moist ...ecessary ...ng ...onsist... ...livery ...s, and ...ng ...the ...flour,

Dry Strength (crushing characteristics)

After removing particles larger than No. 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air-drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

Toughness (consistency near plastic limit)

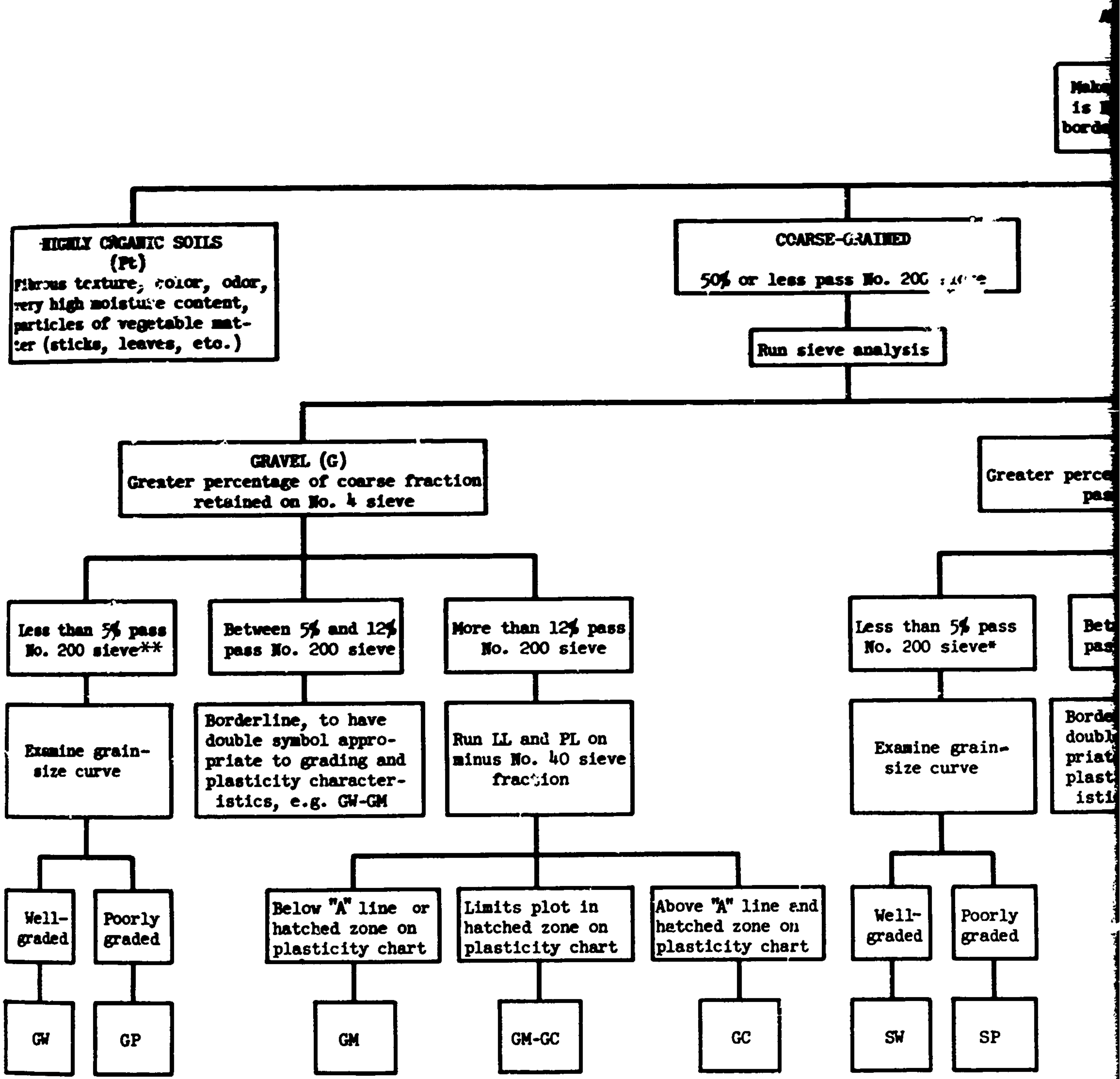
After particles larger than the No. 40 sieve size are removed, a specimen of soil about one-half inch cube in size, is molded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eighth inch in diameter. The thread is then folded and rerolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.

After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occur below the A-line.

Highly organic clays have a very weak and spongy feel at the plastic limit.

021260A



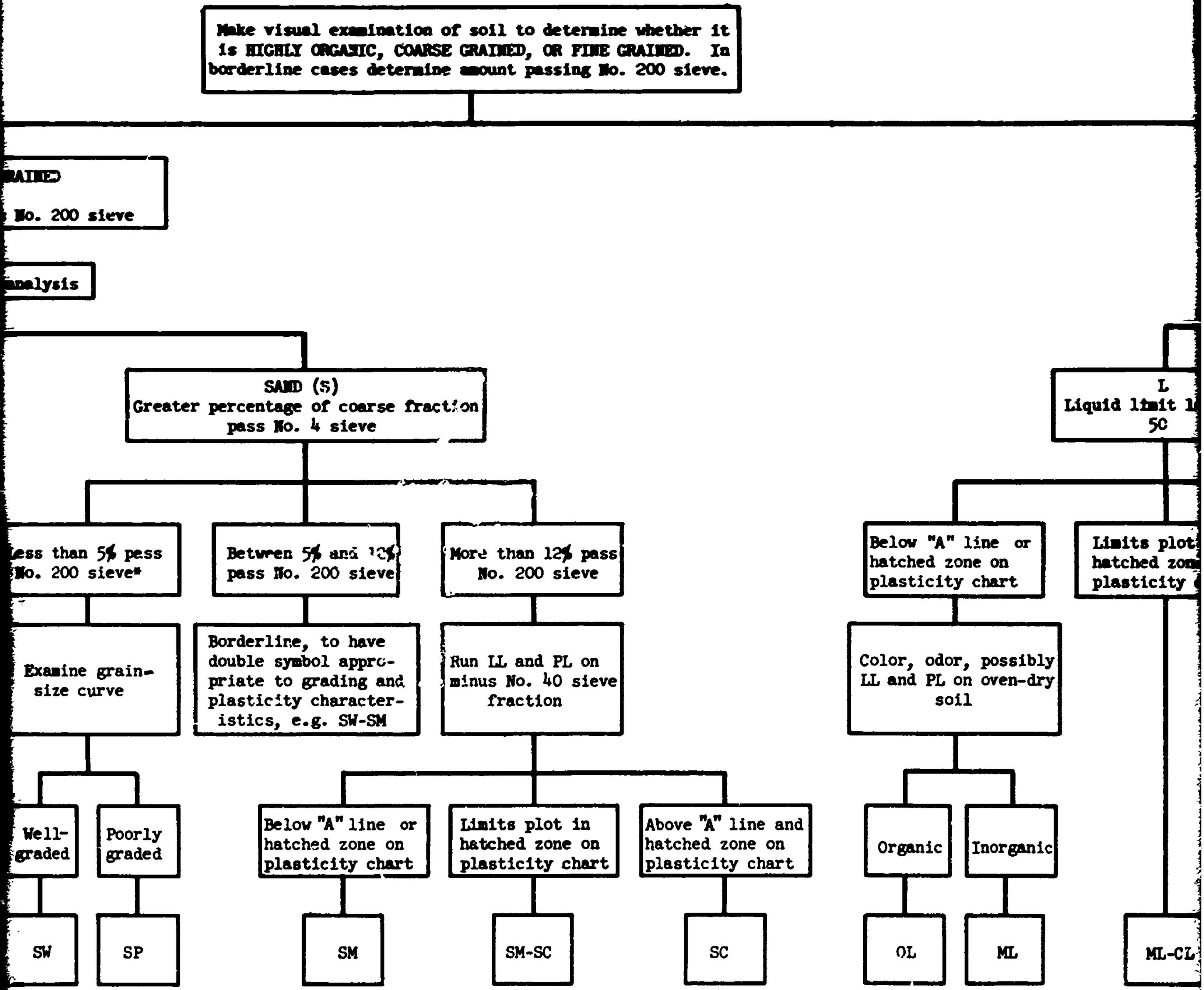
Note: Sieve sizes are U. S. Standard.

* From ref. 82.

** If fines interfere with free-draining properties, use double symbol such as GW-GM, etc.

Table 9*

AUXILIARY LABORATORY IDENTIFICATION PROCEDURE



h as GW-GM, etc.

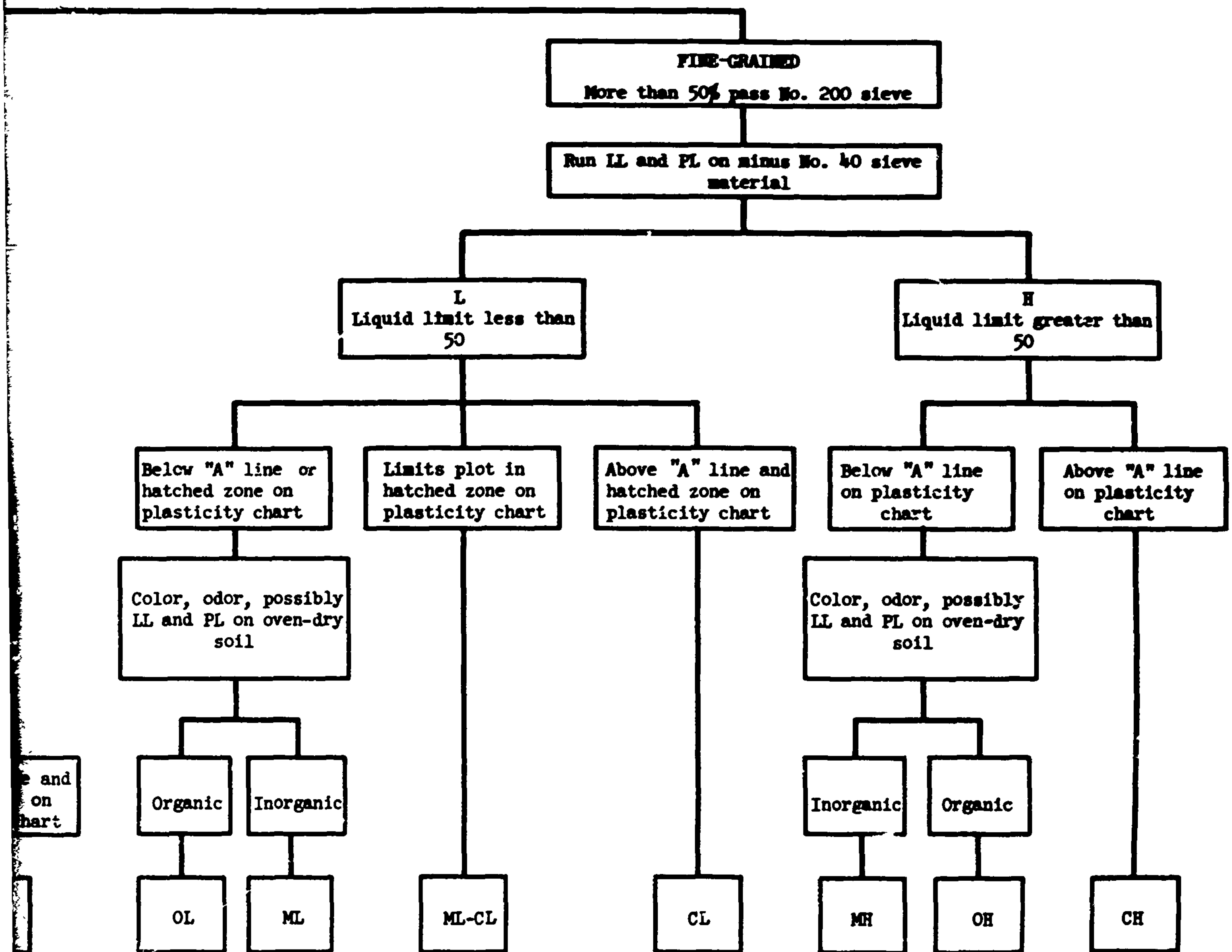

FINE-GRAINED
More than 50% pass No. 200 sieve
Run LL and PL on minus No. 40 sieve material
L
Liquid limit less than 50
H
Liquid limit greater than 50
Below "A" line or hatched zone on plasticity chart
Limits plot in hatched zone on plasticity chart
Above "A" line and hatched zone on plasticity chart
Below "A" line on plasticity chart
Above "A" line on plasticity chart
Color, odor, possibly LL and PL on oven-dry soil
Color, odor, possibly LL and PL on oven-dry soil
Organic
Inorganic
Inorganic
Organic
OL
ML
ML-CL
CL
MH
OH
CH

Table 10

Summary of Marine Soil Property Tests on Samples from off the California Coast

	Pertinent Data for General Locations Indicated						
Parameters	South of San Francisco	West of Ventura*	West of Port Hueneme*	Port Dume to Port Loma	Southwest of Los Angeles	West of Del Mar	West of San Diego
Reference	110	99	111	112	110	113	110
Latitude	37°18.8' to 37°01.5'	34°10.7'	34°09.6'	34°00' to 32°40'	33°47', 33°41'	32°59', 32°57'	32°40' to 32°36'
Longitude	122°42.2' to 122°26'	119°24.2'	119°45.3'	Not given	118°33.5', 118°22'	117°20' to 117°18'	117°35.5' to 117°18'
Water depth, ft	252-450	100	1175	<350-2500	2130 and 1660	830-955	2922-3996
Maximum sample depth below sea floor, ft	0.67	3.9	4.0	2.3	4.8	4.0	1.7
Type of sampler	No details	NCEL Ewing Corer**	NCEL Ewing Corer**	Box Corer†	No details	No details††	No details
Number of samples	12	2	3	18	2	7	18
Tests performed							
Grain size	x	--	--	x	x	x	x
Density	x	x	x	x	x	x	x
Water content	--	x	x	x	--	x	--
Liquid limit	--	x	x	x	--	x	--
Plastic limit	--	x	x	x	--	x	--
Consolidation	--	--	--	x	--	--	--
Direct shear	x	--	--	x	x	x	x
Laboratory vane shear	x	x	x	--	x	x	x

* Soil data obtained in connection with NCEL in situ plate-bearing tests.
** See table 4.
† Similar to USNEL Spade Corer (see fig. 9 and table 4).
†† Coring device designed for use with Submersible DR/V Deep Quest.

Table 11

Comparison of Soil Properties for Samples from Two Sites

Parameter*	Location: Site West of Ventura	Location: Site West of Port Hueneme
Soil classification	Silt (ML)	Silt (MH)
Particles finer than 3.9 microns, %	28	47
Liquid limit, %	44	105
Plastic index	16	56
Water content, %	56	138
Vane shear strength, tsf	0.028	0.011

* Values are mean values of soil properties of top 1.5 ft as determined by in situ plate-bearing tests (from references 99 and 111).

Table 12

Construction Control Factors for Dredging

Purpose of Dredging	Construction Control	Characteristics
Removing unsuitable materials	1. Visual inspection of dredged materials a. Dipper dredge--at dredge b. Clamshell dredge--at dredge c. Hydraulic dredge--at dredge or at discharge pipe or both	1. Basic inspection procedure--requires inspector at dredge when nearly to subgrade, or at discharge pipe with radio communications. Necessary for safe and economical construction
	2. Sampling of bottom	2. Checking of condition of bottom is required
	a. Grab samples (1) Small orange peel (2) Bucket of clamshell dredge	a. Generally applicable--do before placing fill
	b. Borings	b. A nuisance, interferes with construction. Seldom applicable
	c. Probings	c. Applicable in shallow water only (20-30 ft maximum). Insensitive in deep water
	3. Depth control--lead lines and acoustic surveys	3. Tidal or other currents can affect lead line surveys
	4. Diver inspections	4. Must be by feel only because of turbidity; varies in reliability
Cutting uniform side slopes of specified slope	1. Control of depth and radius of bucket or cutterhead--requires inspector at dredge 2. Soundings--lead line and acoustic surveys	1. Control of depth and distance out can produce acceptably uniform side slopes where required. Operation is unfamiliar to many dredge captains but is practicable when understood
Cleaning up soft bottom materials after dredging is complete or as fill is placed or both	1. Use of clamshell or dipper dredge	1. Generally satisfactory a. Very soft material may be difficult to remove--sometimes possible if material is allowed to consolidate for a few weeks b. Do not permit bucket loads of water or hoses to be used to clean decks of scows--cleaning decks results in excessive soft materials on bottom
	2. Use of hydraulic dredge	2. A small hydraulic dredge for bottom cleanup work is probably best method and is flexible--does not tie up principal excavating equipment. Disposal area availability is sometimes limited
	3. Use of airlifts	3. Suitable only for small areas--not always effective

Table 13

Dams Involving Underwater Fill

Dam and Reference	Maximum Water Depth, ft	Work Accomplished
Hugh Keenleyside Dam[170] (formerly Arrow Dam), Columbia River, British Columbia	55	Lower portions of earth dam constructed underwater on pervious foundation soils. Dam composed of sand and gravel zone, with upstream zone and 2200-ft-long upstream blanket of glacial till. Sand and gravel fill beneath blanket and in dam placed in water depths to 55 ft; till placed in water depths to 45 ft
Plover Cove Main Dam,[49] Hong Kong	89	Soft foundation materials removed and embankment constructed underwater. Core is decomposed rock; shells have interior horizontal sand blanket drains
High Aswan Dam,[171] Egypt	131	Portion of dam beneath normal river level constructed underwater. Underwater portion of core consists of coarse sand, grouted after placement. Adjacent underwater zones consist of dune sand compacted underwater after placement, by vibrators. Maximum height of dam is 364 ft; volume of fill is 55 million cu yd
Akosombo Dam,[166] Ghana	217	Dredging for underwater cofferdam construction
Wanapum Dam[172]	50	River closure section constructed underwater. Upstream and downstream dikes constructed in maximum of 50 ft of water; sand fill end-dumped between dikes and consolidated by vibroflotation. Slurry trench cutoff constructed through sand fill
Dalles Closure Dam,[173] Columbia River	180	--
Chain of Rocks[174]	—	Rock-fill dam built across Mississippi River without unwatering

Table 14

Underwater Fill Placement Methods

Methods	Characteristics
Bottom-dump scows	1. Fill assumes flat slopes unless retained
	2. Limited to minimum depths of about 15 ft because of scow and tug drafts
	3. Rapid--quick discharge entraps air and minimizes segregation
Deck scows	1. Usable in shallow water
	2. Unloading is slow--by dozer, clamshell, or hydraulic jets
	3. Steep side slopes of fill can be achieved
Hydraulic	1. Coarse materials drop out first--may cause shear failures in soft foundations
	2. Fines may collect in low areas and have to be removed
	3. Inspection of material being placed may be difficult
Dumping at land edge of fill and pushing material into water by bulldozer	1. Fines in material placed below water tend to advance and accumulate in front of advancing fill
	2. Work arrangement should result in central portions being in advance of side portions to displace sideways any soft bottom materials
	3. In shallow water, bulldozer blade can shove materials downward to assist displacement of soft materials

Table 15

Bottom-Dump Scow Placement of Fill Slopes

Great Salt Lake Crossing

Water Depth ft	General Slope (Very Erratic)
15-20	1V on 2H (some to 1V on 4H)
>20-30	1V on 3H (some almost horizontal)
40	1V on 4H to 1V on 7H (some flatter)
>40	1V on 5H to 1V on 10H and even flatter

Table 16

Bottom-Dump Scow Placement of Rock

Great Salt Lake Crossing

Water Depth ft	Rock Slope
15	1V on 1.5H
25	1V on 3H
40-50	1V on 5H or flatter

APPENDIX A: PERSONAL CONTACTS

Personal Visits

Organization	Individual	Subject
Lehigh University Bethlehem, Pa.	Prof. A. F. Richards	Bottom-rest sampler and test equipment; use of Prof. Richards' library; marine soils
Marathon LeTourneau Co. LeTourneau, Miss.	Messrs. C. Middleton, N. March, and J. Woods	LeTourneau jack-up rigs
McClelland Engineers Houston, Tex.	Messrs. B. McClelland and J. A. Focht, Jr.	Marine soil mechanics and engineering
USAE Coastal Engineering Research Center Washington, D. C.	Drs. D. Duane and C. J. Galvin, Jr.	Sand investory and continental shelf oceanography
Univ. of Rhode Island Kingston, R. I.	Prof. V. A. Nacci	Bottom-rest sampler and test equipment; marine soils

Conferences Attended

ASCE Conference on Civil Engineering in the Oceans, 10-12 December 1969, Miami Beach, Florida

Offshore Technology Conference, 22-24 April 1970, Houston, Texas

Correspondence

Organization	Individual	Subject
Alpine Geophysical Norwood, N. J.	Mr. W. C. Beckman	Vibracore sampler
Bechtel, Inc. San Francisco, Calif.	Adm. H. N. Wallin, USN, Retired	Offshore structures
Benthos, Inc. North Falmouth, Mass.	Mr. S. O. Raymond	Boomerang corer
Chicago Bridge and Iron	Mr. K. W. Lange	Dubai Kazzan storage tank
City of Long Beach Long Beach, Calif.	Dr. M. N. Mayuga	Thums Islands

Organization	Individual	Subject
Dames and Moore Los Angeles, Calif.	Mr. V. A. Smoots	Marine soil mechanics
Earl and Wright San Francisco, Calif.	Mr. W. R. Schmidt	Offshore structures and foundations
U. S. National Aeronautics and Space Administration, Goddard Space Flight Center Greenbelt, Md.		Orbital flight photos of ocean and interpretation
U. S. National Oceanographic Data Center Washington, D. C.	Dr. T. S. Austin	Apollo 9 photo of the ocean and instrument fact sheets
Univ. of Wisconsin Madison, Wisc.	Profs. J. R. Moore, R. P. Meyer, R. Harker, and G. Roderick	Sea grant, Green Bay study, and sand sediment sampler

Telephone Contacts

Alpine Geophysical Norwood, N. J.	Mr. G. Tirey	Vibracore sampler and geophysical survey techniques
Bechtel, Inc. San Francisco, Calif.	Adm. H. N. Wallin, USN, Retired	Design and construction of offshore structures and foundations
California Institute of Technology Pasadena, Calif.	Prof. R. Scott	Accelerometer in situ test equipment
Challenger Research, Inc., Rockville, Md.	Mr. C. L. Hayen	Challenger's state-of-the-art study
City of Long Beach Long Beach, Calif.	Dr. M. N. Mayuga	Thums Islands
Dames and Moore Chicago, Ill., and San Francisco, Calif.	Messrs. C. K. Au and J. Angemeer	Marine soil mechanics and engineering
Earl and Wright San Francisco, Calif.	Mr. W. R. Schmidt	Design and construction of offshore structures and foundations
E. D'Appolonia & Assoc. Pittsburgh, Pa.	Dr. E. D'Appolonia	Marine soil mechanics and engineering
Frederick R. Harris, Inc., New York, N. Y.	Mr. E. H. Harlow	Design and construction of offshore structures and foundations

Organization	Individual	Subject
Lehigh University Bethlehem, Pa.	Prof. A. F. Richards	Bottom-rest sampling and testing system and use of Prof. Richards' library on marine sediments, etc.
McClelland Engineers Houston, Tex., and New Orleans, La.	Messrs. J. A. Focht, Jr., and W. Emrich	Marine soil mechanics and engineering and sampling techniques
Mueser, Rutledge, Wentworth, and Johnston New York, N. Y.	Mr. P. C. Rutledge	Design and construction of offshore structures and foundations
Praeger, Kavanagh, and Waterbury New York, N. Y.	Dr. T. P. Kavanagh	Design and construction of offshore structures and foundations
Soros and Assoc. New York, N. Y.	Mr. L. Sugin	Design and construction of offshore structures and foundations
U. S. Army Engineers Coastal Engineering Research Center Washington, D. C.	Dr. D. Duane	Sand inventory and sampler
Office, Chief of Engineers, U. S. Army Washington, D. C.	Messrs. G. E. McWhite and J. C. Stillman	Techniques for construction in marine environment
U. S. Army Engineers St. Louis District St. Louis, Mo.	Mr. J. Fuhrmann	Vibratory sampler
U. S. Coast Guard Washington, D. C.	Mr. F. Gammon	Offshore lighthouse structures
U. S. Environmental Science Service Adm. Miami, Fla.	Dr. G. Keller	Marine soil properties, sampling and testing
U. S. Geological Survey Corpus Christi, Tex. and Woodshole, Mass.	Mr. H. Berryhill	Continental shelf soils