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GEOTECHNICAL PROPERTIES OF DEEP SEA SEDIMENTS: A CRITICAL REVIEW OF MEASUREMENT TECHNIQUES

P J SCHULTHEISS

REPORT NO 134

1982

NATURAL ENVIRONMENT RESEARCH COUNCIL

INSTITUTE OF OCEANOGRAPHIC SCIENCES

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When citing this document in a bibliography the reference should be given as

SCHULTHEISS, P.J. 1982 Geotechnical properties of deep sea sediments: a critical review of measurement techniques. *Institute of Oceanographic Sciences, Report*, No. 134, [73pp.]

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ABSTRACT

This report presents a critical examination of the techniques employed for assessing the geotechnical properties of deep sea sediments. The review includes an introduction into some of the basic soil mechanics principles as well as highlighting the very important differences between terrestrial soils and ocean sediments. Central to the theme of this review is an analysis of the type of sample that is necessary for specific geotechnical tests or whether in-situ tests need to be performed. Emphasis is placed on the problems of obtaining relatively undisturbed samples from the sea bed together with a discussion of the various parameters which cause disturbance, including the problems of large reductions in hydrostatic pressures. Apart from destructive mechanical tests, the use of non-destructive testing, especially acoustic, on both samples and in situ have been examined.

It is concluded that, because all samples have suffered a significant amount of disturbance, very few laboratory tests can have high levels of confidence. Further studies are required to examine the effects of disturbance in a controlled manner before it is possible to evaluate the reliability of most laboratory tests. In-situ testing, while inherently expensive, can provide rapid and reliable answers providing that the tests are designed for specific purposes rather than attempting to use universal testing procedures.

PREFACE

Part of the Institute's programme for the Department of the Environment, investigating the feasibility of high-level radioactive waste disposal in the ocean, is to study natural pore water fluxes in the sediments. The importance of this topic is for the sub-seabed option where the existence of any pore water flow will influence the migration rate of radionuclides away from the canister.

One approach to this problem is to measure excess pore pressures, sediment porosity and permeability. These three parameters enable pore water movement to be assessed assuming Darcian flow. Sediment permeability and porosity is currently being measured in the laboratory on a suite of samples taken from the N. Atlantic and N.W. Pacific oceans. As part of this study it is essential to evaluate the reliability of these laboratory measurements. Consequently, the literature has been reviewed with this specific purpose in mind.

The problems relating to sampling disturbance caused by coring techniques and their effects on permeability and porosity are, to a large extent, similar to those involving other geotechnical properties of deep sea sediments. This critical review, therefore, has been generalised to include a brief analysis of other geotechnical measurements that may be relevant, not only to post emplac ement 'far field' measurements, but also to 'near field' emplacement considerations.

1. INTRODUCTION

Soil mechanics became a recognised science when its pioneer, Karl Terzaghi, published his famous book on soil mechanics in 1925. His influence on the subject is illustrated by the observation that his textbooks and papers provide to this day a continuing source of reference (Terzaghi, 1943, 1960; Terzaghi and Peck, 1948).

The mechanical properties of soils tend to be distinctly different from the mechanics of most other materials because of their particulate, multiphase nature. These characteristics necessitate the inclusion of a complete suite of parameters to describe soils which are generally inapplicable to other media. Using new parameters and new testing techniques, terrestrial soil mechanics has developed over the past fifty years into an integral part of all civil engineering projects. Taylor (1948) and Lambe and Whitman (1979) provide excellent coverage of the fundamentals of soil mechanics and its application to civil engineering.

In recent years, 'geotechnology' and 'geotechnique' have been used as descriptive terms to encompass the scientific as well as the engineering properties of soils and rocks, particularly for marine sediments. Man's exploration and exploitation of the continental shelves for hydrocarbon and mineral reserved has led to the application of soil mechanics theory to offshore sediments. This trend has probably accelerated the development of analytical techniques for soils because of the more demanding nature of many offshore projects. Sediments used as foundations to support massive structures on the continental shelves are often particularly weak when compared to their terrestrial counterparts. One major difference between conditions at sea and on land for foundations purposes is that of cyclic stresses. Although not absent in land based foundations (machine vibration and wind loading stress must often be considered) cyclic stresses on foundations resulting from wave loading, especially during storm conditions, are nearly always of prime importance. Additionally, the often harsh conditions (especially in areas like the North Sea) and the water column, make sampling and in-situ testing very difficult and much more expensive than on land. The analytical techniques used to predict foundation behaviour are only as good as the soil parameters, measured from laboratory tests on samples or from in-situ tests, that provide the information. Techniques of ascertaining foundation conditions in the North Sea are reviewed by Sullivan (1980) and continental shelf geotechnics are reviewed by Richards et al. (1976), Sangrey (1977) and Focht and Leland (1977).

Deep-sea geotechnics has been much less studied because of the lack of economic incentives. However, it has not been totally ignored. Military installations, manganese nodule mining and, more recently, toxic waste disposal provide examples of the diverse range of incentives that continue to provide both the reasons and funds to study the geotechnical properties of deep-sea sediments.

In this review, some of the more basic soil mechanics principles are examined with respect to their application to deep sea sediments. The important differences that exist between the sediments and their environments of the deep sea, the continental shelf and on land are noted and their effects on some of the geotechnical properties discussed. Techniques of obtaining data from both in-situ and laboratory experiments are reviewed with a strong emphasis on problems of sample disturbance from deep water locations. Both sampling and in-situ testing are very expensive operations in deep water. It is, therefore, imperative to determine which sediment parameters can be accurately assessed from sampling and laboratory testing and which need the added expense of in-situ testing techniques. Finally, because this review has been undertaken as a result of a particular need - that is, to examine the feasibility of high-level radioactive waste disposal on or in deep-sea sediments - this end has been kept in mind and it is purposely biassed towards some of the foreseeable associated problems. It is not intended that this report should be a comprehensive review of published techniques or data.

2. BASIC SOIL MECHANICS PRINCIPLES

This chapter reviews some of the more fundamental soil properties and characteristics, especially those that find an application to marine sediments. It is not intended to be an exhaustive account of the abundant terms used in the soil mechanics literature. Much of the information in this chapter has been obtained from Lambe and Whitman (1979) and the reader is referred to this textbook for a comprehensive treatment.

2.1 Volumetric and weight relationships

A soil is an assembly of discrete particles that are neither strongly bonded to one another, as are crystals in a metal, nor are they completely free to move, as are the molecules in a fluid. The spaces between the particles form an interconnecting array of voids that are filled with a fluid matrix. From the above description, it is apparent that although soils may exhibit both some solid and liquid characteristics (which, indeed, they do) the particulate nature and associated effects are dominant in their behaviour.

The volumetric ratio between the solid particle volume V_s and the volume of voids V_v (where the total volume $V = V_s + V_v$) is described by the porosity n or the void ratio e, where $n = V_v/V$ and $e = V_v/V_s$ therefore: $n = e/(1 + e)$ and $e = n/(1 - n)$. Porosity is often multiplied by 100 and thus values are given in per cent. The relative volumes of gas and liquid (usually water, V_w) present in the voids is expressed by the degree of saturation S where $S = (V_w/V_v) \times 100$. For most marine sediments S is normally considered to be 100%, i.e. saturated. The void ratio of any soil is not a fixed parameter but can vary over a range of values depending on how it is deposited. Consequently, one technique of characterising the packing of a soil is by calculating its relative density D_r, where $D_r = \dfrac{e_{max} - e}{e_{max} - e_{min}} \times 100\%$ where e_{max} and e_{min} are the maximum and minimum void ratios as determined by a given test procedure (Kolbuszewski, 1948). Such tests are only practically applicable to non-cohesive sediments such as sands.

Relative amounts of the different phases can also be expressed in terms of weight. The most readily obtainable and most often used is the water content w, given by $w = W_w/W_s$

where W_w is the weight of water

W_s is the weight of solids

The total unit weight of a soil γ_t is given by W/V

where W is the total unit weight

V is the total unit volume

The submerged unit weight of soil is given by

$$\gamma_b = \gamma_t - \gamma_w$$

where γ_w is the unit weight of water.

2.2 Particle Characteristics

Within a soil mass the individual components are the solid particles and the fluid matrix. In saturated sediments the nature of the water is relatively invariant compared to the variability of the mineral grains. The assemblage of many individual soil particles is described by their mineral compositions, their shapes, their sizes and the distribution of each. The size of an irregularly-shaped particle cannot be uniquely defined by a single parameter. Hence, particle size is dependent on the technique used in its determination. The commonly standardised techniques (e.g. British Standard 1377:1975) are a sieve analysis for particles larger than 0.06mm and a pipette or hydrometer analysis for smaller particles. It is common to classify particles by size using the following system:

>300 mm	Boulder
150 to 300 mm	Cobble
2.0 to 150 mm	Gravel
0.06 to 2.0 mm	Sand
0.002 to 0.06 mm	Silt
<0.002 mm	Clay

The distribution of sizes is normally plotted as the percentage (by weight) of particles finer than a given diameter as shown in Figure 2.1 (the arbitrary nature of the classification system is illustrated by the difference in the boundary between the gravel and cobbles when taken from different sources).

Particle shapes vary from being predominantly equidimensional as with most silts and sands to shapes such as plates which are far from equidimensional and common for clay size particles. Particle shapes can be assigned using visual estimation charts such as those of Powers (1953) which have been redrawn by Pettijohn et al. (1972).

Soil particles may be organic or inorganic. Inorganic particles may be either a mineral or a rock (an aggregate of minerals or glasses). The principles of mineralogy are beyond the scope of this review and the reader is referred to specialised texts, e.g. Deer et al. (1966) and Berry and Mason (1959). Deep-sea sediments are composed of particles which can be distinctly different from those found in other soils. In particular, large parts of the ocean floor are covered by oozes which contain large percentages of the calcareous and siliceous skeletal remains of marine organisms. It is not surprising to find that these sediments will often exhibit unique mechanical properties.

2.3 Void Characteristics

The special arrangement of the soil particles results in an interconnecting network of irregularly-shaped voids. Whether the fluid filling these voids is a gas or a liquid or a mixture can have a dramatic effect on the soil's mechanical properties. Marine sediments are virtually all 100% saturated and, hence, a discussion of dry soils is not central to the theme of this review. However, it is important to recognise that the water composition can have a strong influence on the structural arrangements and, hence, the mechanical properties of some sediments, especially clays. Electrolyte concentration, ionic type and pH all influence the structural arrangement during sedimentation (Silva, 1974). Clay particles are most affected by the ionised salts and form flocculated structures with higher void ratio than in fresh water (Monney, 1971). Other parameters which may need consideration in deep water applications are those of temperature and pressure which will alter the viscosity of the fluid.

2.4 Atterberg Limits

The Atterberg limits of a soil are arbitrarily-determined values of water content that define a limit between different soil states. Depending on the water content, a fine-grained soil can behave as a solid, semi-solid, plastic or liquid (Lambe and Whitman, 1979). The liquid limit, plastic limit and shrinkage limits define the intermediate boundaries between these states using specified tests defined by British Standard 1377. Although the Atterberg limits are arbitrary and do not lend themselves to any fundamental investigation they do provide a useful classification test for different soil types. The variation in the liquid limit for different sediment types is very wide, providing a meaningful indication of the relative interaction between water and particle surfaces. The liquid and plastic limits, W_l and W_p respectively, are supplemented by the plasticity index $PI = W_l - W_p$ and the liquidity index $LI = \frac{W_n - W_p}{W_l - W_p}$ where W_n equals natural water content. The plasticity index indicates the range over which the soil remains plastic and the liquidity index shows how close the natural soil is to the liquid limit. These limits and indices have proved to be particularly useful characteristics of remoulded soils. It should be emphasised that all these tests are performed on totally disturbed (remoulded) samples and provide, therefore, only a classification test.

For fine-grained marine sediments, virtually all have a water content above the plastic limit and most have water contents above the liquid limit (Monney, 1971). However, it is important to appreciate that this does not necessarily mean the natural state is that of a liquid, only that the remoulded state would be a liquid. If the remoulded sediment is allowed to stand its strength is regained. This phenomenon is known as thixotropy and seems to be due to the destruction and subsequent reconstitution of the molecular structure of the adsorbed layers which separate the mineral grains.

2.5 Permeability

The interconnecting network of voids causes soils to be permeable. Variations in the permeabilities of different soils is the primary cause of the wide range of stress-strain characteristics that exist.

Permeability is normally defined using Darcy's law which was determined empirically.

$$Q = kiA$$

where Q = the rate of flow ($m^3 s^{-1}$)

k = co-efficient of permeability (ms^{-1})

i = hydraulic gradient (no units)

A = cross-sectional area (m^2)

As defined by Darcy, k depends on both the sediment and permeant characteristics and has units of ms^{-1}. It is most often expressed this way in hydrology and soil mechanics because only one permeant (water) is usually considered.

The hydraulic gradient, i, is the pressure difference across the sample (expressed as a height of water) divided by the sample length. It is often useful to express the velocity, v, of an element of water approaching the soil in a water column

where $$v = Q/A = ki \ (ms^{-1})$$

the seepage velocity of an element of water in the sediment is given by

$$V_s = \frac{v}{n}, \text{ where n is the porosity,}$$

thus $$V_s = \frac{ki}{n}$$

An absolute permeability, K, of a material is independent of the permeant and is obtained using

$$K = k\mu/\gamma\,(m^2)$$

where μ = viscosity of permeant

γ = density of permeant

K is also sometimes expressed in darcys where one darcy is defined as the permeability of a substance if a pressure gradient of 1 atm will produce a flow rate of 1 $cm^3\,sec^{-1}$ of a fluid with a viscosity of 1 cP through a cube having sides of 1 cm in length.

$$1 \text{ darcy} = 0.987 \times 10^{-8} cm^2$$

The grain size, grain size distribution and void ratio are the most important among the many factors which affect the permeability of a soil. One well-known expression that relates these factors to permeability is the well-known Kozeny-Carmen equation

$$k = \frac{\gamma}{k_o T^2 S^2} \; \frac{e^3}{\mu(1 + e)}$$

where γ = unit weight of permeant

μ = viscosity of permeant

e = void ratio

k_o = factor depending on pore shape and ratio of length of actual flow path to soil bed thickness

S = specific surface area (area per unit weight of particles)

T = tortuosity (often assessed empirically)

Although the Kozeny-Carmen equation has been thoroughly tested for sands its inadequacy for saturated clays has long been recognised and improved empirical or theoretical formulations are needed for these sediments. Figure 2.2 illustrates the wide range of permeabilities that occur for different soils.

2.6 Stress-strain behaviour

The stress-strain behaviour of soils is significantly different from other materials. Although soils can exhibit elastic, plastic and fluid types of behaviour, none provides a fair description except over a very limited range of test conditions. In soils significant changes in volume can occur, if the soil is allowed to drain, caused by the expulsion of pore fluid. Soils generally have stress-strain-time relationships that are probably more complex than for most other materials.

2.6.1 The effective stress

If a load is applied to an element of saturated soil and the water is not allowed to drain freely then that load will be carried in part by the mineral skeleton and in part by the pore water. If we define that part of the load carried by the mineral skeleton as the effective stress, σ'

then $\sigma' = \sigma - u$

where σ = total stress

u = pore pressure

The effective stress is very important because it controls certain aspects of soil behaviour, notably compression and strength. It should be noted that at the zero effective stress condition cohesionless sediments are in a critical state and have zero shear strength.

2.6.2 One-Dimensional Consolidation

One-dimensional consolidation, or confined compression, has a wide range of applicability in soil mechanics. This arises because the sedimentation and subsequent consolidation of ocean sediments is essentially a one-dimensional process. Also, buildings cause compressions in the ground which can often (especially at depth) be considered to be one-dimensional consolidation. Consolidation refers to the gradual process of expelling pore fluid with the resulting compression of the sediment or soil framework; it does not refer to lithification as used in geology.

The parameters used in defining the consolidation characteristics of a soil are best illustrated with reference to a laboratory consolidation test. A diagrammatic illustration of a consolidation test is shown in Figure 2.3a. A soil specimen is sandwiched between two porous discs, which allow free movement of pore water, and is encased in a confining ring which prevents any horizontal strains developing. A vertical load is applied causing the total stress, σ, to act on the sample. This stress is initially carried entirely by the pore water as explained earlier, therefore, initially σ' is zero. As water is forced out of the voids, at a rate which depends on the permeability of the soil, the stress is gradually transferred to the soil skeleton. When the compression has virtually ceased then $\sigma' = \sigma$ and $u = o$. This is the end of the primary or hydrodynamic stage of consolidation. Traditionally, loads are applied in steps, each load doubling the previous value, when primary consolidation has ceased. Secondary compression due to the plastic nature of soils also occurs and, hence, the consolidation process can proceed for long periods of time for a given load. In practice, and to fit the consolidation theory of Terzaghi, consolidation is taken to be complete at a point when the excess pore pressure which has initially been increased due to the load, is zero. This is found by graphical construction techniques using a void ratio-time graph for each load (Taylor, 1948). Both primary and secondary consolidation obviously occur simultaneously. The distinction between the two is made because the graphical technique used to find the end of primary consolidation assumes that the processes tend to be independent at either end of the consolidation test.

2.6.3 Triaxial Test

The mechanical properties of many materials are tested by a compression test on a cylindrical sample. With soils, many of which cannot support themselves as a cylindrical sample, an all-round confining pressure in the triaxial test is applied before testing. A flexible rubber membrane isolates the sample from the confining fluid at a pressure σ_c as shown in Figure 2.3.b. In the standard triaxial test the axial stress σ_a is increased until the specimen fails while σ_c is held constant.

It is normal in soil mechanics to describe the stress state in terms of the three principal stresses which act on three orthogonal planes on which there are zero shear stresses. In order of decreasing magnitude there are the major σ_1, intermediate σ_2, and minor σ_3, principal stresses. For the case of the triaxial test $\sigma_2 = \sigma_3 = \sigma_c$ and $\sigma_1 = \sigma_a + \sigma_c$; $\sigma_1 - \sigma_3 = \sigma_a$ is known as the deviator stress.

In its wide range of uses the triaxial test can measure both the stress-strain properties and strength characteristics of soils both drained and undrained under a large range of stress conditions which may simulate the soil's natural environment. Bishop and Henkel (1962) provide a full account of the use of the triaxial test on soils.

2.6.4 Direct Shear Test

In the direct shear test a soil sample is held in a split container as illustrated in Figure 2.3.c. A confining force P is applied normally as shown. The shear stress S is then applied causing horizontal strains to develop. Either constant stress (stress controlled) or constant rate of strain (strain controlled) techniques are employed. Changes in the thickness of the sample also occur during shearing and the magnitude and direction of this change is of some importance. Although the behaviour of a soil in the direct shear test is similar in many respects to the triaxial test, the stress states are not generally well defined and only qualitative stress-strain data is usually obtained. Most of the distortion occurs in a thin zone along the plane of the split container and is of an unknown thickness.

3. DEEP SEA SEDIMENTS

Deep ocean basins occur both close to and far away from land masses, and at different water depths above and below the CCD (carbonate compensation depth). Consequently, deep ocean sediments are diverse in composition and texture. Two divisions are made for classification convenience; these are the terrigenous and pelagic deposits. Terrigenous deposits are derived from material eroded from land surfaces and are found in both shallow and deep water close to the land masses. Pelagic sediments accumulate slowly in the open ocean and consist of ooze and, below the CCD, of brown or red clay. Often, seafloor sediments contain appreciable amounts of terrigenous as well as pelagic sediments; these are called hemipelagic deposits. For a recent review of sedimentary processes and distribution the reader is referred to IOS Report No. 77.

Biogenous oozes are pelagic sediments composed of at least 30 per cent of empty shells and animal skeletons of tiny marine organisms. Calcareous and siliceous oozes consist respectively of calcium carbonate and silica skeletal remains. The most common clacareous oozes are formed from the remains of planktonic animals and plants such as foraminifera and coccoliths.

Siliceous oozes which are less prevalent than calcareous ooze and are most common at high latitudes, have two common kinds: diatom ooze, which is the remains of single-celled plants, and radiolarian ooze, which is the skeletal remains of microscopic marine animals.

The general distribution of these sea floor sediments has been given by Shepard (1963), Keller (1967) and Horn et al. (1974) among others. Calcareous ooze covers most of the Atlantic Ocean and much of the South Pacific. Siliceous ooze is found in a large belt around the Antarctic and north-east of the Japanese Islands. Red clay, which is the residual product of dissolution, is believed to come largely from atmospheric dust as well as land-derived deposits circulated by the major ocean currents. The major portion of the North Pacific and deeper parts of the Atlantic Ocean are covered in red clays.

4. SEDIMENT SAMPLING

4.1 Introduction

Information from sediment samples obtained from the sea-floor is required for a diverse range of purposes. These include biological, chemical, geological and engineering/geotechnical needs. Clearly, the type of sampling equipment necessary will be governed by the type of sample required and the depth from which it must be recovered (both water and sediment depth). In this brief summary emphasis will be placed on samples (especially those taken from deep water) which are potentially suitable for geotechnical purposes.

Soil sampling followed by laboratory testing is a widely used procedure in soil investigations. Laboratory conditions permit a degree of test control that can never be achieved in situ. On land, high quality samples can sometimes be obtained by cutting blocks from open pits. This is probably the best form of sampling that can be achieved as it renders the least possible disturbance to the soil sample. Most samples, however, are obtained from boreholes and considerable care must be taken to obtain high quality samples suitable for geotechnical testing especially from deeper layers. For marine sediments the problems are compounded by their often sensitive nature and the overlying water column. The need for reliable offshore samples is as great, if not greater, than on land yet the environment makes quality sampling far more difficult.

A survey of marine bottom samplers by Hopkins (1964) and Moore and Heath (1978) reveals that a very wide variety of devices has been invented to grab, scrape, suck or core samples from the sea bed.

4.2 Grabs

Grabs of one sort or another form a major class of sampling devices. They are equipped with jaws which are forced shut by weights, lever arms, springs or cords, enclosing a sediment sample. Hopkins (1964) describes nineteen of the different varieties existing at that time and, as pointed out by Taylor Smith (1969), there are likely to be many more in existence (probably hidden in different establishments throughout the world).

All types of grabs are intended to recover a representative sample of sediment from the surface of the sea floor. This can include much biological and some recent geological information. It is not usually intended that grabs should be used to retrieve undisturbed samples for geotechnical tests. The limited penetration and often highly disturbed nature of samples obtained with grabs generally limits their use to identification and classification tests of the surface material. Even grain size analysis from some grab samplers is liable to incorporate errors caused by finer material being partially washed out during recovery.

4.3 Coring Samplers

Apart from suction devices and dredges, which are obviously of little value for geotechnical sampling and analysis, the only other major class of sampling devices are the coring samplers. Hopkins (1964), Richards and Parker (1967), Bouma (1969), Taylor Smith (1969) and Noorany (1972) all list many types of corers and the reader is referred to these works for a comprehensive treatment and literature review of these devices.

Coring consists of inserting a hollow pipe into the sediment and withdrawing it so as to retain the cored material. To obtain the required penetration the core barrel can be pushed, rotated, vibrated, punched or simply dropped (or any combination) into the material.

Vibratory coring is usually the only technique available to obtain long core samples from dense sand layers. However, the disturbance caused to the grain structure is thought to be severe. Grouting (injection of a hardening fluid) has been used to preserve the structure of cohesionless soils (Bouma, 1969; Karol, 1970). Different grouts have been used, these include silica and polyacrylamide gel as well as various resins. The disadvantage with grouting is that the grout cannot be totally removed and, hence, the shear strength and deformation properties of the soil cannot be determined. Freezing is sometimes used to reduce the disturbance during sampling and transport. However, this can still alter the shear strength and deformation properties after melting has occurred. It appears that cohesionless soils cannot be satisfactorily sampled for the determination of deformation properties using techniques available at present. Results from laboratory tests must often be supplemented by seismic, plate load, pressure meter or penetration tests in situ (Broms, 1980).

Rotary coring is applicable only in very stiff lithified sediments and rocks where the shear stresses can be tolerated without significant disturbance to the sample. The normal technique for obtaining cores from unlithified material is to push or drop the core barrel into the sediment. Disturbance caused during this sampling procedure arises from many factors and is discussed in some detail in the following chapter.

The sampling methods which are used offshore, both on the continental shelf and in the deep ocean, are often primitive compared with techniques available on land. Samples are normally taken on the continental shelf with open drive samplers which can cause a large disturbance to the soil. The samplers are generally hammered down. Bjerrum (1973) and McQuillin and Ardus (1977) have reviewed soil exploration techniques used on the continental shelf. Coring techniques used in the deep ocean have been reviewed by Hopkins (1964) and Noorany (1972) among others.

4.3.1 Gravity Corers

These coring devices are driven into the sea-floor by their own weight which can usually be varied by using different numbers of lead weights at the upper end. They are generally lowered on a ship's winch and can either be driven straight in or allowed to free-fall from a predetermined distance using a trip mechanism (Hvorslev and Steton, 1946) as shown in Figure 4.1. As pointed out by Bouma (1969) the term 'gravity corer' is not strictly correct since other types of devices also rely primarily on gravity as far as penetration is concerned. 'Punch corer' or 'open barrel gravity corer' are probably more descriptive terms for this group of samplers. This type of corer usually has a non-return valve at the top to prevent flushing during retrieval, and a core catcher at the bottom which acts as a sediment non-return valve while still allowing easy penetration. Various forms of catchers exist for the cylindrical pipe, which is the most common form of barrel; the most often used is the 'tulip' type consisting of a hemispherical array of plastic or metal fingers.

Gravity corers are used for general sampling where core lengths less than about 4m are required. A well-designed corer with effective retainers and check valves can obtain relatively undisturbed sediments (lithologically) although the cores commonly suffer some degree of shortening.

4.3.2 Box corers

The original box sampler developed by Reineck has been adapted for deep water use (Bouma and Marshall, 1964). The essential part of the device is a large sampling box which is driven into the sediments by the weight of a frame and lead weights. A release mechanism on top of a central stem allows a blade to be driven under the box to retain the sediment on withdrawal. A much simplified box corer has been developed at IOS (Peters et al., 1980) which is shown in Figure 4.2. The large supporting frame has been dispensed with (which allows easier handling on deck) and two closely-fitting shovels are used to cut through the sediment. The corer (weighing 400-600 kg) has a stainless steel box, 30 cm square and 120 cm in length. For surface sampling, sediments collected by these types of corer seem to be of an exceptionally high quality. The usefulness of large diameter cores is well documented, especially by Hagerty (1974) and Rosfelder and Marshall (1967). However, the lack of penetration depth limits their use for geotechnical purposes.

A long box corer (Kastenlot) which is a development of the conventional gravity corer was designed by Kogler (1963) to obtain large volume core samples up to 10m long. The corer consists of two lengths of sheet metal, sometimes plastic coated, which are screwed together to give a barrel of square section with 15 cm sides. Barrels are normally available in 2, 4 and 6m lengths. The core catcher is of the trap-door variety and this, with the cutting-head attachment, allows unimpeded entry of the sediment. Results from these cores show them to be probably the best obtainable within the first 4m of sediment (Kogler, 1967).

4.3.3 Free-fall corers

The free-corer (Moore, 1961; Sachs and Raymond, 1965) is unique because it does not require a winch and a cable. The free-corer has an expendable weight and a short barrel with the core liner attached to a buoyant chamber mounted in a frame. The chamber assembly is released either on impact or by a time-activated release mechanism. On release the core liner is pulled out of the casing by the buoyant chamber and then freely ascends to the surface (Figure 4.3) where it can be located by radar reflectors, flashing lights or a radio beacon. It is an effective means of obtaining near surface samples in deep water because the fast rate of fall minimises lateral drift, hence, sampling of the bottom of canyons and the top of narrow peaks has been possible (Noorany, 1972).

4.3.4 Piston Corers

The main disadvantage of the gravity corer is its rather limited penetration. Piston coring has become the most widely used and successful means of sampling sediments in the deep sea because of its much increased penetration (up to 30m) which potentially makes it far more useful than the gravity corer, especially for stratigraphic studies.

The main difference between the gravity and the piston corer is the incorporation of a closely fitting piston within the core barrel. The piston should be held stationary at the sediment surface while the barrel moves past it penetrating the sediment column, as illustrated in Figure 4.4. Greater penetration can be achieved than with gravity corers because frictional forces between the inside of the barrel and the sediment are at least partly counteracted by the reduction in pressure which results beneath the piston. This helps to move the sample into the barrel and provides less overall resistance to penetration. Piston cores were first introduced into marine sampling by Kullenberg (1955) and it provided a major breakthrough in providing marine geological data. It was hoped that the quality of marine piston corers would be as good as the 'undisturbed' samples obtained on land using pistons. However, this has not generally been the case and marine piston cores often suffer considerable amounts of disturbance which will be discussed in the following chapter. With increased need for longer and larger diameter core samples, large piston corers have been developed. In particular, the Woods Hole Oceanographic Institution developed a 'Giant Piston Corer' (GPC) which had the capability of obtaining 30m cores (Silva and Hollister, 1973). Not all previous attempts at obtaining long cores have been successful and several GPC systems have been either broken or lost. Even when long cores have been recovered they frequently show regions of severe disturbance due to inadvertent piston motion. It is thought that the maximum depth achievable using this technique is probably 50m. Silva and Hollister (1973 and Silva et al. (1976), among others, report on geotechnical properties measured from cores obtained with the GPC. A system is currently under development in the USA to enable cores up to 50m in length and 11.4 cm in diameter to be obtained by conventional oceanographic ships (Driscoll, 1981). The system, called the Long Coring Facility (LCF), comprises a piston corer weighing 15 tonnes, a traction winch capable of pulling up to 90 tonnes and a synthetic fibre rope 7 cm in diameter. Design of the system should be completed by mid-1982 and manufacture is planned for 1983. One of the disadvantages of the system will be that only a limited number of oceanographic vessels would be able to accommodate such a large device.

4.4 Samples from the Deep Sea Drilling Project

The only routinely available technique for obtaining sediment samples from below about 30m is through the Deep Sea Drilling Project (DSDP). This project employs the ocean-going drilling ship Glomar Challenger to drill and recover sediment and rock samples from sub-bottom depths exceeding 1000m in water depths of up to 7000m. Some limited physical properties of the recovered sediments are measured on board as a matter of routine to complement the geological data. Sampling techniques used in this project, however, have not, until recently, yielded the high quality samples needed for many geotechnical tests. Kidd (1978) examines some of the visual disturbance features encountered in DSDP that could lead to geological misinterpretations. Bennett and Keller (1973) describe the case handling procedures which cause disturbance, while Demars and Nacci (1979) discuss the disturbance created by the rotary sampling procedure and how it effects the measurement of some physical properties. They concluded that the remoulding and swelling effects produce samples which are 'not representative of in-situ sediment properties'. Davie et al. (1979) report on the geotechnical properties of deep continental margin soils on a worldwide basis using DSDP cores.

Since December 1978 DSDP has been using a Hydraulic Piston Corer (HPC) which is lowered and retrieved through the drill string. The original version had a 4.4m long barrel and a 6.35 cm internal diameter. The quality of samples retrieved using the HPC are significantly better than those obtained previously by rotary drilling and it represents a major breakthrough for DSDP. However, core disturbance still remains a problem especially in the uppermost 30-50m where the sediments are less consolidated. In bad weather, motion of the ship is transmitted to the bottom of the drill pipe and, hence, the piston is not held stationary at the top of the core length. A variable length HPC has now been fabricated and was successfully tested in June 1981. The maximum core length has been increased to 9.5 metres to allow a degree of versatility when coring sediments of varying shear strength. It is hoped that improvements in this system will allow samples to be obtained throughout the sediment column which are suitable for geotechnical testing.

5. SAMPLING DISTURBANCE

5.1 Background

The engineering and physical properties of all soils depend to a very large extent on the orientation and distribution of particles in the soil mass. This soil structure or fabric, as well as the forces between adjacent soil particles, is extremely prone to disturbance during sampling procedures. Forces between soil particles exist as a result of both internally-generated forces (electro-chemical) and externally-generated forces (in-situ stresses).

In soils with clay particles, two extremes of soil structure exist; flocculated and dispersed. This is schematically illustrated in Figure 5.1. The differences that occur as a result of the salinity of the pore water in a natural and remoulded state are shown in Figure 5.2 Intermediate stages of soil structure are discussed in detail by Mitchell (1976). A totally remoulded structure is the worst case imaginable for a soil sample and the large differences in geotechnical properties resulting from remoulding are well established. The ratio of undisturbed to remoulded strength has been defined as 'sensitivity' and when the sensitivity is high, as in many marine sediments, the importance of obtaining a representative sample for laboratory testing cannot be overemphasised. Although a truly undisturbed sample is impossible to obtain, it is possible to produce high quality samples by cutting out blocks from excavations. However, this is impossible for marine sediments although some shallow box samplers can be sub-sampled in a similar fashion. Disturbance can arise from several sources - (a) disturbance of the boundary stress and pore water pressure conditions on removal from its natural environment; (b) disturbance of the physical structure and fabric of the soil caused by imperfect sampling and sub-sampling methods; and (c) sample transportation, storage and time-dependent effects.

5.2 Stress Relief

Even if a perfect sampling tool could be developed that produced no mechanical distortions, samples would still suffer a degree of disturbance caused by a change in the stress conditions. In a normal in-situ condition, a soil will experience different total vertical and horizontal stresses (Figure 5.3a). When a terrestrial sample is removed from the ground and is unconfined, it will tend to swell producing negative pore pressures (Figure 5.3b). In a laterally-confined terrestrial sample (Figure 5.3c), the in-situ, total horizontal stresses may be maintained but the total vertical stress is reduced to atmospheric pressure. The soil properties will depend on the amount of stress relief that has occurred, therefore, the problem is more apparent the greater the depth from which the sample is obtained. In general, the compressibility of soils increases and strength decreases when effective stresses are relieved in this way.

A complicating factor for samples recovered from the oceans is the expansion of pore water caused by the massive reduction in hydrostatic pressure. For example, a sample taken from an ocean depth of 5000 metres and brought to the surface experiences a pressure reduction of 51 MPa and a volume increase of about 2.4 per cent. This volume increase must take place vertically in a tube sampler and if the water cannot drain freely then structural disturbances and positive pore pressure must result (Figure 5.3d). The greatest effect will be on samples with low permeabilities raised rapidly from depth. Normally, cores are raised at about 1m/sec. It can be calculated that for a 2m core raised from a 5000m water depth at this rate, containing an ooze with a permeability of 10^{-9}m/sec the imposed hydraulic gradient during ascent is around 2000. In the laboratory, when conducting permeability experiments, low hydraulic gradients are desirable to prevent fabric disturbance. If a hydraulic gradient of 50 was not exceeded on the above example, it would require at least three days to raise the core to the surface. For a pelagic clay with a permability of 10^{-10}m/sec, using a 12m core and a maximum hydraulic gradient of 50, it would take up to 200 days to raise the sample to the surface! The author is unaware of any coring procedures that have deliberately taken more than the minimum time to raise the sample from depth. A further effect of large hydrostatic pressure reductions is the possible evolution of gasses from solution or from gas hydrates (clathrates).

Another disturbance factor caused by environmental change is the possible reduction in shear strength caused by marine bacteria growing in a core after it has been collected (Richards and Parker, 1967). To avoid this type of disturbance it is probably prudent to store cores in a refrigeration unit at the sea floor temperature (usually 2-5°C) until the samples are tested.

5.3 Sampler disturbance

Apart from the environmental changes which occur when a sample is collected, the process of inserting and removing a sampling device or sub-sampling device and trimming and mounting the specimen, will all have an influence on the structure of the soil. An extensive literature exists on the disturbance caused by the size and shape of the sampler. In particular, Hvorslev (1949) produced a treatise which is still of practical value. He discussed the forces and sediment deformations involved in drive samplers and defined the principal dimensions which are relevant to disturbance and shown in Figure 5.4. Richards and Parker (1967) recommend the following ratios and dimensions to minimise disturbance:

c_1	<20	
c_o%	0-3	This controls the outside friction
c_a%	<10	This relates the volume of displaced sediment to the volume of the sample
c_i%	0-1	This controls the inside friction
D_e	>5cm	

Cutting edge angle ≤5° and sharp

A look at the characteristics of various corers with reference to the above design criteria (Table 5.1) indicates a gross discrepancy between theory and practice.

Apart from the dimensions of the corer and cutter, it is essential to have well-designed non-return valves and sample retainers to minimise the resistance to penetration and prevent the loss of any sediment on recovery. Apart from the disturbance created inside the core barrel, the soil ahead of the advancing corer is subjected to stresses which influence the final core (Kallstenius, 1958). In marine sediments there is very little scope for the compaction inside the core tube because only a negligible amount of water can be forced out of the material during the short period required for penetration (Piggot, 1941; Hvorslev and Stetson, 1946). Therefore, it must be assumed that most of the core shorting occurs ahead of the cutter. In sediments consisting of thin strata of varying stiffness, the softer strata are likely to suffer a greater decrease in thickness than the firmer strata. When the friction inside the core barrel prevents further entrance of manterial, an advancing bulb or cone of sediment is then formed below the cutting edge (Horslev and Stetson, 1946) and further penetration merely pushes sediment aside as if the corer were a solid bar. This process can occur intermittently causing an interval of sediment to be totally absent from the recovered core (Weaver, 1981). It is obviously a mistake, therefore, to assume that the difference between corer penetration and core recovered can all be accounted for by a uniform compression of the strata. Richards and Parker (1967) discuss core shortening with respect to the suitability of samples for shear strength testing. A very large number of samples routinely taken with many coring devices are, therefore, not suitable for shear strength testing.

Parameter Corer Type	D_s cms	L metres	C_1 %	C_i %	C_o %	C_a %	cutting angle
Ideal	>5	-	10-20	<1	<3	<10	<5°
USHO Hydroplastic	8.18	3	37	1.6	13.4	56.8	5
USNEL Hydroplastic	6.70	3	45	1.2	5.5	33.0	8
Richards "	10.70	3	28	0	0	14.1	4
Lemont-Ewing	6.25	30	480	5.3	18.2	87.2	10
USHO-Kullenberg	4	3	75	1.8	10.1	105.5	8
UMEL Standard	6.0	1	17	0	0	64.7	?
UMEL Sargent	6.0	4	67	0	0	64.7	?
Saclanteen Sphincter	12	15	130	0.8	5.1	44.0	5
Benthos Boomerang	6.25	1	16	0	4	69.0	?
Kastenlot	15	6	40	0.6	7.7	22.4	8
Alpine Vibracore	8.9	6	67	0	0	65.3	?
Mackereth Pneumatic	4.1	6	146	0	0	54.8	?
NGI Torpedo	5.4	1.65	31	0.9	0	12	?
USNEL-Reineck	30	0.45	1.5	0	0	2.7	-
IOS Box Corer	30	1.20	4	0	0	14	-
Driscoll Piston Corer	6.6	15	227	0	17.7	152	30

Table 5.1 Core disturbance characteristics of various corers after Taylor Smith (1969).

With piston coring there is a disturbance factor which is unique to the system. It is imperative that the piston is held stationary during the driving stroke if relatively undisturbed cores are to be obtained. Although this is relatively easy on land, at sea with the corer suspended by a long wire from a ship which will be oscillating vertically, it is virtually impossible. Not only is the ship's motion a problem but the elastic rebound and vibration of wire when the weight is released makes keeping the piston stationary using the simplest type of corer an impossibility. Kullenberg (1955) has exhaustively discussed the theory and operation of the conventional marine piston corer. Using cameras mounted on the top of piston corers, McCoy (1980) has provided an analysis of the coring process. Corer recovery ratios (depth of penetration/length of sediment core) averaged 0.86 for 49 good stations (five recovery ratios exceeded 1.00) with no apparent relationship to the sediment type. Some of the sediment losses may be caused by loss of sediment from the surface ahead of the advancing corer as demonstrated by the photographs of large sediment clouds raised during penetration. Comparisons between the trigger weight corer and the piston corer deomonstrated that the piston corer could lose up to a metre of the upper sediment layer. Thinned laminae with sharper contacts indicated significant shortening on compression of the sampled stratigraphic section in piston cores as compared to equivalent horizons in the trigger weight cores. McCoy also demonstrates that substantial amounts of rotation occur during the sampling process (up to 150 degrees) and that most of the samples are taken with an inclination of greater than nine degrees.

Techniques do exist to prevent the piston from moving during core penetration. The Norwegian Geotechnical Institute's gas-operated, fixed-piston corer (Andresen et al., 1965) satisfies this requirement by taking cores in incremental stages of 1.65 metres. The corer has a torpedo shape and is allowed to penetrate into soft sediment under its own weight. When penetration ceases. a controlled ignition of gas drives a core barrel past a fixed piston into the sediment. By taking successive cores a complete profile can be established. However, it is limited in its use to soft sediments and in deep water the process would be extremely laborious. Another type of piston corer which minimises piston movement is the Mackereth corer (Smith, 1959). Here the piston is attached to a frame which rests on the sediment surface while compressed air forces the core barrel past the stationary piston into the sediment. It has been used successfully in lakes at up to 250 metres but not at sea (Richards and Parker, 1967). Kermabon et al. (1966) describe the sphincter corer which after modifications (Kermabon and Cortis, 1968, 1969) utilises thin pulleys and cables to immobilise the piston at the water-sediment interface during penetration (Fig. 5.5). A split piston is also incorporated which prevents suction of sediment up the core on retrieval should full penetration not be achieved. (Considerable disturbances can be caused if the piston is not immobilised during recovery [Bouma and Boerma, 1968]). The wide diameter sample (121 mm) also adds to the quality of the cores retrieved. The Swedish Geological Survey developed a foil corer which completely eliminates the sliding friction between the clay sample and the core tube (Kjellman et al., 1950). Steel foils, stored on spools behind the cutter, are drawn up through the core tube as it advances past a stationary piston. In this way the core is completely surrounded by the foils and, hence, protected from the moving core tube. Samples of varved clays taken using the foil corer apparently show no signs of disturbance (Noel, 1981). However, this type of corer is probably not a practical method for oceanic sampling.

5.4 Classification of Sample Disturbance

No sample is truly undisturbed; Ladd and Lambe (1963) define a "perfect sample" as one that has undergone no disturbance apart from that caused by stress release. The disturbance in addition to that caused by the stress release can be evaluated from the difference between the pore-water pressure in a "perfect sample" and the measured residual pore pressure. Ladd and Lambe (1963) and Okumura (1971) have proposed that disturbance could be defined from the residual effective stress for a perfect sample and that of the actual sample. Broms (1980) points out that these types of definitions are based on parameters which are difficult to measure and, also, it is not known if they would be a reliable index of disturbance. The volume change which occurs when a sample is reconsolidated in the laboratory to the in-situ stress has been used by Berre and Bjerrum (1973) as a measure of sample disturbance.

Schmertmann (1955) suggested that the degree of disturbance could be quantified by performing consolidation tests on both the "undisturbed" sample and a totally remoulded sample. Silva (1974) proposed that disturbance could be quantified more simply from the shape of the void ratio/log effective pressure (e - log p) curve. This technique is based on the rate of change of slope around the maximum past preconsolidation pressure. For a sample that has very little disturbance the change in slope is very rapid whereas for a sample with extreme disturbance,the change of slope is very gradual.

With sediment samples recovered from the deep oceans the primary problem of sample disturbance and its classification must be related to the relief of in-situ stress. For near surface samples the expansion of pore water will play a dominant role. With samples taken from depth (using hydraulic piston coring in the DSDP project) a combination of the release of in-situ effective stresses caused by overburden and the volume change effects need to be examined. The effects on the soil fabric caused by the pore water expansion have received little experimental attention in the past and this must be rectified before a proper evaluation of the usefulness of laboratory testing can be performed.

6. LABORATORY VERSUS IN-SITU TESTING

6.1 Background

The geotechnical properties of soils are determined in order to study their in-situ properties and fundamental physical characteristics. For this purpose samples are either recovered and tested or in-situ tests are performed. Generally speaking, the laboratory offers cheap and controlled conditions where a large range of different test techniques can be used. However, despite numerous precautions during sampling, transportation, sample preparation and testing, there will always be some disturbance which remains unavoidable; an unknown disturbance factor which can never be totally accounted for. For this reason, the determination of in-situ properties by laboratory techniques will always be difficult and often questionable. In-situ testing will always be preferable for many applications because there are no problems with sample disturbance. Also, in-situ tests can be applied to the material in a manner which can be more closely related to the scale of real application (e.g. foundation engineering) and it has the advantage of not having artificial boundaries, as occurs with laboratory samples. The main disadvantages of in-situ testing techniques are that they tend to be relatively expensive and only comparatively simple tests can be performed. For deep sea sediments the problems are magnified; laboratory testing is much less reliable than for terrestrial samples because of the increased problems of disturbance and because the sediments are often highly sensitive and in-situ testing is inherently more problematic in oceanic depths. Consequently, any investigation into geotechnical properties of deep sea sediments must involve careful consideration into the most appropriate technique to measure any given parameter.

6.2 Particle Characteristics and Atterberg Limits

Assuming that a representative sample of the sediment can be obtained (as is the case for most coring devices) in-situ measurements have no advantage over laboratory techniques for determining particle shape, size and mineralogy characteristics. The tests designed to measure the Atterberg limits have been specifically designed for use on remoulded material, hence, any in-situ evaluation is meaningless. It should be noted that it is necessary to use salt water and not fresh water for Atterberg limit tests on marine sediments and that the samples should not be sieved or oven dried.

6.3 Porosity/Void Ratio and Bulk Density

Volumetric and weight relationships between solid and fluid components of a sediment may be subject to change during the sampling procedure. Even core samples which apparently show no signs of disturbance may have suffered some compression which will provide bulk density measurements which are higher than in situ with correspondingly low values of porosity. Worse still, the softer strata in a sediment core may be compressed more than the firmer strata producing even greater discrepancies. This effect would tend to make the differences in volumetric and weight relationships throughout a core less than the real in-situ values. In practice, however, the fast rate of the sampling operation together with the low permeabilities of deep sea sediments probably result in little, if any, change in these parameters. Laboratory volume and weight measurements are probably most reliable on short wide-diameter gravity cores. Even these are still subject to pore water expansion effects and corrections for this must be applied.

Nuclear logging using gamma radiation, to measure the bulk density, and neutron radiation, to measure water content, are techniques used in many fields. Preiss (1967, 1968a, 1968b) describes how these techniques may be used for marine sediments both on core samples and in situ. These in-situ methods are particularly applicable to near surface sediments which can be extremely porous and tend to suffer the greatest amount of disturbance when sampled. It is difficult using these techniques to obtain accuracies better than one per cent due to the problems of calibrating the instruments with specimens of different chemical and mineralogical compositions.

The velocity of compressional waves in sediments has been found by many authors to be very closely related to porosity and bulk density (e.g. Hamilton, 1965; Taylor Smith, 1974, 1975) but generally the relationship is insufficiently determined to enable it to be used for accurate in-situ determinations. The same situation exists with electrical resistivity techniques. Although the relationship between resistivity and porosity is well defined, calibrations are necessary on each sediment type in the laboratory before in-situ results can be accurately interpreted (Kermabon et al., 1969; Jackson, 1975; Jackson et al., 1980).

6.4 Permeability

The permeability of a sediment depends on the pore structure and the permeant and covers several orders of magnitudes for different sediment types. It is to be expected, therefore, that structural disturbances caused by sampling procedures may have a large influence on the measured permeability. Even if sampling created no structural disturbance, small samples may be grossly inadequate for field permeability estimation. For laboratory tests to provide realistic values applicable to in-situ conditions the samples must be large enough to contain a significant distribution of features that affect the flow of water through the material. These may include changes in lithology, fissures and burrows. Anisotropy, even in relatively homogeneous sediments, is not normally measured in laboratory experiments where artificial flow paths and boundaries are imposed. Despite the potential inadequacies of laboratory measurements they still dominate the literature. Olsen and Daniel (1979) provide an excellent summary of both laboratory and field techniques for determining permeability together with a comparison of these values using a large amount of published data.

Standard laboratory tests for fine-grained sediments generally use cylindrical samples enclosed in a metal or plastic ring. A normal consolidation cell with suitable modifications can be used for permeability measurements. The use of back pressure is particularly important to provide complete saturation of the sample as the permeability of an almost saturated soil can be considerably different from its saturated value (Lambe, 1951). Back pressure reduces or eliminates the volume of gas in the pores by direct compression and by dissolving into the pore water. Lowe and Johnson (1960) discuss this use of back pressure to increase the degree of saturation. Permeability tests can also be performed in a standard triaxial cell (Bishop and Henkel, 1957) with the advantage of back pressure and applied stresses (both horizontal and vertical) being controlled.

In principle, maintaining a constant pressure difference across the sample and measuring the steady state flow rate permits the permeability to be calculated from Darcy's Law. However, in practice, it is usual to perform these tests at varying low pressure differentials and to plot them against flow rate. This should yield (according to Darcy's Law) a straight line passing through the origin. Deviations from Darcy's Law have been noted at both high and low pressure gradients. High gradient deviations could occur as the flow changes from laminar to turbulent. Nickerson (1978) shows that hydraulic gradients in excess of 10^{14} would be necessary to cause turbulent flow in very fine grained sediments. At low gradients deviations from Darcy's Law have been reported as non-linear and exhibiting a threshold gradient below which flow will not occur. Olsen (1965, 1966) reviews this data and critically examines the conventional experimental techniques of using air-water menisci and air bubbles in capillary tubes. He concluded that atmospheric contamination in the flow rate measuring capillary tubes caused spurious deviations in flow rate and pressure measurements which produced apparent anomalies in Darcy's Law. The other main contributing factor for non-Darcian behaviour is the plugging of flow channels by migrating particles during the test. It would seem from a survey of the literature that direct measurement of permeability on laboratory samples can be problematic and a great deal of care must be taken to ensure that the apparatus is properly designed. In particular, techniques for measuring low flow rates and hydraulic gradients must be completely reliable before accurate permeability measurements can be obtained.

The other main technique used to calculate laboratory permeability is from the normal consolidation test. Terzaghi (1925, 1943) used the permeability of a saturated sample consolidating under a load in a laterally-confining ring (one-dimensional consolidation) to express the dissipation of pore pressure with time. The coefficient of permeability can be expressed in terms of the coefficients of consolidation and volume compressibility which arise from the one-dimensional consolidation theory:

$$k = C_v \gamma_w m_v$$

where
k = coefficient of permeability (ms^{-1})
C_v = coefficient of consolidation ($m^2 s^{-1}$)
γ_w = unit weight of water (Nm^{-3})
m_v = coefficient of volume compressibility ($m^2 N^{-1}$)

Differences in permeability measured directly and computed from Terzaghi's theory are reported by Nickerson (1978) and Olsen and Daniel (1979) who both show that the direct method gives higher values. Nickerson suggests that the discrepancies are due to the variation of hydraulic gradient and/or the formation of a filter cake at the sediment-porous stone interface during consolidation. Olsen and Daniel attribute at least part of the discrepancy to the fact that Terzaghi's consolidation theory makes no adjustment for the structural viscosity (creep) of the soil.

6.5 Stress-strain behaviour

The complex stress-strain-time characteristics of soils have resulted in a wide variety of both laboratory and in-situ test techniques being developed. In laboratory tests stresses can be controlled which allow elaborate test procedures to be followed, whereas in-situ tests normally permit only simple test procedures. The advantage of the in-situ tests are that the soil has not suffered the disturbance caused by the processes of sampling and transportation to a laboratory. Engineering evaluation of soils generally involves either stability or foundation considerations although other considerations, for example, object penetration and breakout forces, are also sometimes required. Laboratory testing of marine sediments is much the same as for terrestrial soils whereas the environment makes in-situ testing (especially in the deep oceans) more problematic. Differences in the techniques used for laboratory and in-situ stress-strain parameters means that it is usually difficult to compare different sets of data realistically. Much of the effort of in-situ testing of marine sediments has been related to determining the 'strength'. This loosely-used term usually implies its shear strength. It must be appreciated that the shear strength of a sediment is highly dependent on the technique and conditions of the test for any particular sample. For example, a vane test at a given angular velocity may be closely related to an undrained triaxial test in a clay whereas it might more closely relate to a drained triaxial test if used in silt because of the differences in permeability. The in-situ reality is that there is normally always partial drainage which cannot be easily duplicated in the laboratory.

6.5 Penetrometers

Penetrometers of one form or another are widely used to obtain strength measurements. In terrestrial use the static cone penetration test is described (Dayal, 1979) as driving a cone-tipped cylindrical rod into soil at low constant speed whereby the cone thrust or both cone thrust and local side friction are measured either mechanically or electrically. The resistance to penetration is related in some way to the strength of the sediments and, because the penetration is continuous, a profile of resistance is obtained which is indicative of the relative strength with depth. The cone penetrometer system, Seacalf, developed by Fugro Ltd., now finds wide application in foundation investigations on the continental shelf. A hydraulically-propelled cone is mounted in a ballasted frame which is lowered to the sea floor and controlled remotely by cables from the sea surface. Zuidberg (1975) describes the Seacalf righ and Le Tirant (1979) describes other penetrometer devices used in the North Sea.

Dynamic penetration tests differ from their counterpart static test in that the strain rate is not controlled. In its simplest form on land a test consists of driving a cone-tipped rod into the ground by repeated blows of a standard hammer. The resistance of the ground is then quoted in n, the number of blows required to force the rod through a specific distance. A plot of n versus depth reveals a qualitative soil profile. It would seem that the results from dynamic hammer blow penetrometers can provide reasonable estimates of bearing capacity and settlement in sands but, for cohesive clays, the data can be unreliable and misleading.

Dynamic, impact penetrometers are those which record the deceleration of projectiles which penetrate the ground at some initial, terminal velocity. Scott (1967) and Preslan (1979) describe the use of an accelerometer mounted on a corer. This has provided not only information on the soil geotechnical profile but also valuable information on the behaviour of the coring device. Dayal et al. (1976) report on a specially designed impact penetrometer for use in marine sediments.

Dynamic penetration data is unlikely to yield valuable static stress-strain data useful to fields other than those involving dynamically penetrating objects except by empirical correlations. This type of data should, therefore, be borne in mind when considering the feasibility of free-fall disposal of canisters in sediments. Berry (1980) has provided a literature review on the types of penetrometers that have been developed and tested.

6.5.2 Vane devices

The vane test has proved to be a very useful method of determining the shear strength of soft clays and silts. Generally, a four-bladed vane is inserted into the ground and rotated at a slow constant angular velocity. The torque required to rotate the vane is measured and the shear strength calculated assuming a vertical cylindrical failure plane around the edges of the vane (Monney, 1974). Many marine sediments fall into the category of soft clays and silts and are, therefore, amenable to vane testing. Consequently, a considerable effort has been made to adapt this test for use in marine sediments. Taylor and Demars (1970) report on the performance of the vane used on a seafloor rig designed for both vane and cone testing. Richards et al. (1972) describe an in-situ device capable of obtaining vane shear strengths at penetrations of up to three meters. The in-situ strengths obtained were generally about twice the strength measured on gravity cores using miniature laboratory vanes. However, the results from laboratory vane tests are unreliable when compared with those from field vane tests. Baligh and Vivalrat (1979) compared field and laboratory tests with actual foundation performance and conclude that, while the laboratory vane shear strength underestimates the undrained shear strength, the field values provide excellent profiles. Perlow (1974) conducted a study to compare in-situ and laboratory shear vane strengths in various areas to determine the influence of sample disturbance caused by gravity coring. It was concluded that the differences in vane sizes caused large strength differences between the in-situ and laboratory measurement even when a constant angular velocity was used. This is concluded to result from the consequent difference in shear velocity at the edges of the vane blades caused by the different blade dimensions. It was proposed that the shear velocity should be standardised (at 15 mm/s) rather than the angular velocity.

Sherif et al. (1979) propose a combined laboratory vane shear and direct shear test which can describe the total stress failure envelope from only two samples. Selvadurai (1979) presents an analytical study of the vane tests which suggests that not only can strength tests be performed but that, by examining the initial torque-twist relationship deformation, moduli may be determined.

In general, although the vane test has limitations regarding the assumed failure plane, the lack of standardisation of strain rate and the degree of drainage associated with the test, it is usually considered to provide reasonable values of in-situ undrained shear strengths. Differences between values obtained with different size vanes at different shear rates should be examined very carefully before any conclusions are drawn.

6.5.3 Pressuremeter Tests

The Menard pressure meter (Gambin, 1971) has been used for onshore soil exploration for many years. Essentially, it consists of an expandable cylinder which is lowered to a predetermined depth in an open borehole and pressurised. Stress, measured by the hydraulic pressure, and strain, measured by the volume change, are monitored and plotted. When compared with assumed soil behaviour according to a given model, elastic or elastic-plastic, values of the soil modulus and yield strength can be determined. The normal criticism of the Menard pressuremeter is that it measures the stress-strain properties of soil which has been disturbed by the drilling procedure and suffered geostatic stress relief. To overcome the problems both self-boring pressuremeters and push-in pressuremeters have been developed. A self-boring pressuremeter developed at Cambridge University is described by Wroth and Hughes (1973) and is known as the Camkometer. It is pushed into the soil from the surface while the soil entering the hollow body is removed by a mechanical cutter and a flushing system. Load cells measure total lateral stresses and an expandable portion measures the stress-strain properties. A wire-line push-in pressuremeter, developed for marine soils at the Building Research Establishment, is described by Henderson et al. (1980). Although it has no device for removal of soil by a cutter and flushing mechanism (because of the limited space available for a wire-line tool) and it cannot measure total lateral stresses, the results apparently compare favourably with the Camkometer and with large plate tests on London Clay.

Although the pressuremeter measures horizontal mechanical properties, and hence is not a definitive tool for anisotropic soils, it is the only in-situ technique capable of directly measuring the deformability characteristics of marine soils as well as providing lateral stress and shear strength. Probably its greatest application is for providing data for piles subjected to horizontal loads.

Clough and Denby (1980) in a review of relevant data find that from the information available the self-boring pressuremeter yields undrained shear strength values that are higher than those from conventional laboratory tests. From their own data on San Francisco Bay mud they concluded that undrained shear strengths from the pressuremeter were generally 10-15 per cent above those from conventional triaxial compression and laboratory vane tests. They also found that undrained shear strengths from reconsolidated samples tested in triaxial compression were equal to or slightly above the pressuremeter values. Denby and

Clough (1980) critically examine different interpretation procedures for the self-boring pressuremeter and propose a new solution which they regard as being less subjective.

6.5.4 Plate-loading tests

In-situ plate-loading tests, while not a commonly used test, can be used to measure stress-strain and strength parameters. Marsland (1971) describes the use of a large plate-loading test on land (plates up to 865 mm in diameter have been used in 900-mm boreholes). Maximum loads of up to 150 tonnes can be applied on the plate with the reaction force being provided by a large loading frame and ten tension piles. Stress-strain-time diagrams are produced after measuring the vertical movement of the plate. Large-scale, in-situ testing has been shown to be particularly valuable especially when dealing with fissured clays which cannot be sampled representatively for small scale laboratory testing.

Underwater plate-load tests have not received a great deal of attention although some experiments have been performed. Harrison and Richardson (1967) report on two plate-loading tests on a sandy site in Chesapeake Bay. It was found that the strength values obtained by the plate test were in excess of those that would be obtained from triaxial tests and the results of varying plate diameter tests (300-600 mm in diameter) showed that the settlement under a given pressure decreased as the plate diameter increased. A plate-bearing device, developed at the Naval Civil Engineering Laboratory, Port Hueneme, is described by Kretschmer and Lee (1969). Small differences in settlement were observed for different size plates in the range 6-18 inches. But strength values were found to be more than 25 per cent greater than from cored samples in the laboratory. Another model testing device (1.83m in diameter) developed by the Naval Civil Engineering Laboratory, LOBSTER (Long-term Ocean Bottom Settlement Test for Engineering Research) has provided data of settlement over long time periods, up to two years. In general, the data has tended to confirm analytical predictions based upon the magnitude of primary consolidation tests (Anderson and Herrmann, 1971). However, the in-situ results also indicate a surprisingly large amount of continuing long-term settlement, apparently caused by secondary compression. Results also indicate that undermining of the foundation can occur by burrowing animals and hydraulic scour can cause large settlements and in some cases render the foundation useless. These effects would obviously not be revealed by laboratory testing. Simpson et al. (1974) monitored the settlement of various shaped concrete blocks in deep water (1240 metres) using a submersible for two years and concluded that predicted settlements approximated the total actual settlement (maximum difference 28%).

6.5.5 Borehole tests

Conventional borehole logging tools as used in the oil industry are extremely sophisticated and expensive. Although simpler tools have been developed, particularly for the coal and mineral industry, their design has not been optimised and their use not increased significantly for the offshore site investigation industry (Threadgold, 1980). Three types of geophysical logs form the main categories of borehole logging. Electrical or resistivity logging measures the conductivity of the soil which is strongly related to the porosity of the sediment or rock. Acoustic or sonic logs measure the velocity of compressional waves in the material. This is also strongly related to porosity but the relationships are probably not as well defined as for electrical resistivity. The value of the acoustic log would be greatly enhanced if both compressional and shear wave velocities were simultaneously recorded as they can provide values of elastic moduli. Although shear wave velocities can be measured in rocks, such measurements in unconsolidated sediments have proved to be more difficult. Radioactive techniques for borehole logging include the gamma-gamma back-scattering method. A gamma detector detects photons which have been back-scattered from the surrounding material. The response is related to bulk density. Porosity (or moisture content in unsaturated rocks) can be evaluated using the absorption of high energy neutrons by hydrogen in water. Natural gamma-ray logs are used for correlating different lithologies from their natural radioactivity. They are particularly useful for identifying clays.

Although not used extensively for geotechnical evaluation in the off-shore site investigation industry, geophysical logging techniques are used in the Deep Sea Drilling Project (DSDP). Apart from the electrical, sonic and radioactive logs, other devices such as televiewer, temperature, caliper and magnetometer logs have been used. More recently, borehole packing and pumping experiments have enabled estimates of basement permeabilities to be obtained.

Devices which measure mechanical properties directly, for example, vane tests and pressuremeter tests, have not yet been used in the DSDP project but have found application in shallower waters. Borehole pressuremeter tests have already been discussed; shear vane tests can be conducted using wire line techniques. A wire line test permits an instrument to be lowered through a hollow bit, eliminating the need to repeatedly retract it from the hole. Fenske (1957) first used the wire line vane test at the bottom of a borehole up to 77m deep in 20m of water. However, the early technique required a fixed platform for the operation. Doyle et al. (1971) and Kraft et al. (1976) describe the use of a remotely-controlled wire line vane test device which can be decoupled from the ship's motion. It has been used in 305m of water and penetrated the sea bed by more than 100m.

6.5.6 Tethered Platforms and Submersibles

Other techniques for obtaining in-situ geotechnical parameters are by the use of manned submersibles and unmanned, remotely-controlled, tethered platforms. The difficulties of operating equipment, mechanically tethered to a ship or fixed rig, have led to the development of apparatus which rests on the sea bed and is linked only by an umbilical to the surface support vessel. Tethered platforms of this sort have been designed to carry out a limited number of functions. Tirey (1972) describes the use of remotely-operated, submersible sampling equipment which is used to drill and core sediments mainly on the continental shelf. This technique eliminates some of the hazards when working in rough waters. Most samplers are designed to obtain a continuous core in a single lowering, but a few are designed to take samples at specific depths with a wire line through the drill pipe from the support vessel.

Tethered devices, used in continental shelf waters, for cone penetration were described earlier in this chapter. Rigs such as Seacalf (Zuidberg, 1975) and Stingray (Semple and Johnston, 1980) have found a wide use in site investigation for offshore platforms. A few tethered devices have been developed for use in deep water. Richards et al. (1972) describe a vane shear apparatus which will work in 4-6 km of water and penetrate nearly 3m into the sediment. The penetration and rotation of the vane is controlled from the ship using an electrical conductor within the mechanical cable but the power is supplied from a self-contained battery. Results from several locations indicate that the shear strengths are generally about twice the strengths measured on gravity-type cores using miniature laboratory vanes. This discrepancy may be attributed more to the uncertainties of vane testing using different rotation rates and vane sizes than to the disturbance caused by sampling.

One of the most versatile underwater in-situ testing units is the DOTIPOS

(Deep Ocean Test In-Place and Observation System) developed at the Naval Civil Engineering Laboratory, Port Hueneme (Taylor and Demars, 1970). DOTIPOS is equipped with both a vane and cone device which can penetrate up to 3m and will work in up to 200m of water. Taylor and Demars (1970) concluded that the ratio of cone resistance to vane shear strength is probably related to the sediment clay content. They also found only a slight discrepancy between their in-situ and laboratory vane strengths. This may be attributed to the low sensitivity of the materials tested.

Another versatile system that can be used for geotechnical evaluation of deep sea sediments is the Remote Underwater Manipulator (RUM) developed at the Marine Physical Laboratory, Scripps Institution of Oceanography. It is a tracked vehicle capable of operating on the sea floor to depths of 2000m. Its use when instrumented with a small corer, a vane shear test and a cone penetrometer is described by Anderson et al. (1972) and Gibson and Anderson (1974).

Only a few reports of geotechnical properties measured from manned submersibles appear in the literature. Manned submersibles have the advantage over remotely-controlled vehicles of more precise control over site selection but their limited penetration and susceptibility to movements by currents may limit their usefulness. According to Noorany (1972) the first in-situ vane shear measurements to be made from a subjersible were by Moore using a small vane attached to the submersible Trieste. The submersible Alvin has been used to measure directly bulk density using a nuclear transmission densitometer and penetration resistance of a static cone penetrometer to a depth of 1.52m (Perlow and Richards, 1972). An account of a vane test conducted from a manned submersible is that provided by Inderbitzen and Simpson (1972) who used Deep Quest to measure vane shear strengths in the San Diego Trough and took sediment samples with a corer designed specifically for Deep Quest. In these experiments, large differences between laboratory and in-situ strengths were observed. In-situ strengths were found to vary between 27 per cent above and 175 per cent below the laboratory strength values. The discrepancies between the values cannot be explained which highlights the difficulties of obtaining reliable strength measurements in sensitive near-surface sediments.

6.6 Acoustic Properties of Water-Saturated Sediments

6.6.1 Introduction

Although the stress-strain properties of terrestrial soils and marine sediments are non-linear, at low strains they tend to become linear and at very low strain levels they can be considered to be elastic. Stress waves can propagate in sediments at very low levels of strain and hence can be analysed using Hookean elastic theory, i.e. assuming stress is proportional to strain. In this way it is possible to define low-strain elastic moduli for sediments based on the velocity of stress waves.

For elastic materials the speeds of compressional and shear waves is related to the elastic moduli and bulk density by the following equations:

$$V_s^2 = \frac{G}{\rho}$$

$$V_p^2 = \frac{K}{\rho} + \frac{4G}{3\rho}$$

when V_p = compressional wave speed

V_s = shear wave speed

K = elastic bulk modulus

G = elastic shear modulus

ρ = bulk density

The compressional wave speed, V_p, in unlithified saturated sediments, is controlled by the pore water and varies from 1500 to 2000 m/s. The compressibility of the mineral skeleton has little effect on V_p and hence it is nearly independent of effective pressure. Compare this with undrained triaxial testing where the effective pressure controls the mineral skeleton compressibility and the moduli. Even in undrained traixial tests, the moduli obtained are much lower than those calculated from acoustic velocities. Values of moduli calculated from acoustic velocities are, however, directly useful for a variety of dynamic problems associated with earthquakes and machine vibrations.

Despite the apparent gulf between moduli measured by standard soil mechanics test and acoustic techniques, elastic moduli have received much attention. The moduli obtained using wave velocities represent maximum moduli at very low strain levels and these have become important parameters for dynamic foundation analysis. Consequently, it is relevant to review some of the techniques and data available for different soils, especially marine sediments.

6.6.2 Compressional Waves

Wood (1930) originally developed an equation to relate sound speed to sediment suspensions:

$$V_p^2 = \frac{1}{(n\beta_w + (1-n)\beta_m)\ (n\rho_w + (1-n)\rho_m)}$$

where $\beta = \frac{1}{K}$ = compressibility

n = fractional porosity

subscripts: w = water

m = mineral

Early laboratory measurements on low concentration suspensions of kaolinite made by Urick (1947, 1948) and Urick et al. (1949) demonstrated the validity of Wood's equation in these suspensions. In-situ measurements, using transducers inserted by divers (Hamilton, 1956) recorded surface sediment velocities three per cent lower in high porosity muds than in the overlying water. This phenomena agreed with Wood's equation. However, in all but very high porosity sediments, the velocity is found to be higher than Wood's equation predicted (e.g. Taylor Smith, 1974, Fig. 2). Wood's equation neglects the rigidity component caused by interaction between particles and hence only provides a lower limit for most sediments rather than describing all the data.

A large amount of data has been published on the physical/compressional wave interrelationships of saturated sediments (e.g. Hunter et al., 1961; Morgan, 1969; Buchan et al., 1972; Anderson, R.S., 1974; Akal, 1974; and Hamilton, 1970, 1972, 1976). Expense, practical difficulties and lack of control over experiments performed on in-situ sediments have resulted in many measurements being performed on artificially-sedimented material in the laboratory (Busby and Richardson, 1957; Shumway, 1960; Nolle et al., 1963; Hampton, 1967). McCann and McCann (1969) have used published results to study attenuation mechanisms. Acoustic measurements on retrieved core samples have been made (e.g. Hamilton, 1963; Buchan et al., 1967; Horn et al., 1968; and McCann, 1972). Measurements during coring, giving in-situ profiles, have been made by Shirley and Anderson (1975a).

Several generalisations can be made from the results of the above workers. Compressional wave velocity, over a wide frequency range, correlates more consistently with parameters such as the bulk density, porosity and void ratio than with other geotechnical parameters. Mean diameter also shows a correlation with velocity from natural sediments, but this can be attributed to the density as bulk density and grain size are also related in situ.

Despite variable bulk density for any given sediment being an important material property for soil engineers and geophysicists, little information is available on the variation of acoustic properties with changes in sediment structure under ambient conditions. Laughton (1957) and Brandt (1960) studied the variations that occur when high external loading causes porosity changes. Under ambient conditions sound speed and electrical formation factor are both very sensitive to variations in packing (Taylor Smith, 1975).

Hamilton (1971b) suggested methods of predicting in-situ compressional wave velocity from laboratory measurements. In unconsolidated marine sediments, sound speed variations with temperature and pressure are very similar to that found in sea-water. Using the ratio of laboratory sediment velocity to laboratory water velocity and multiplying it by the in-situ water velocity, a predicted in-situ ratio can be obtained.

Compressional wave velocities for saturated marine sediments from all sources exhibit velocities above those predicted by Wood's suspension equation (except for very high porosity muds). A finite rigidity within the sediment, caused by the structural frame, is generally the accepted explanation of the departure from Wood's equation (which, modelled on a true suspension, does not exhibit rigidity). Shumway (1960), among others, empirically modified the equation to include a rigidity term so that the discrepancy, between experimental and theoretical values, was reduced to a minimum. Using Gassmann's (1951) theory, Hamilton (1970) included a rigidity term so that the system bulk modulus included a frame component and shear wave velocities could then be utilised.

Hamilton (1971a, b; Hamilton et al., 1970) discusses appropriate elastic and visco-elastic models for saturated sediments, concluding that for small stresses (such as those of a sound wave) elastic equations can be used, but a linear visco-elastic model should be used to account for wave attenuation. Differences between acoustic and mechanical loading of sediments, in relation to elastic theory and its limitations, are discussed by Taylor Smith (1974). Elastic constants can be derived from a knowledge of bulk density, compressional and shear wave velocities, the latter being the most difficult to ascertain with confidence in sediments. The value of seismic techniques for dynamic testing in marine sediments is reviewed by Feldhausen and Silver (1971). The need for information on both shear and compressional waves in sediments for geoacoustic modelling of the sea floor is illustrated by Hamilton (1974, 1980).

6.6.3 Shear Waves and Measurement Techniques

In general, the paucity of shear wave, compared to compressional wave, data in sediments can be accounted for by the lack of reliable techniques for propagating, receiving and analysing these slow-moving, highly-attenuated body waves. Sources and receivers, for use in sediments, which are rich in shear characteristics and where the compressional wave form does not mask the slower shear wave, have proved to be difficult to devise. Consequently, the relative simplicity of producing and receiving compressional waves has probably detracted some attention from the shear wave problem. Nevertheless, as vast amounts of compressional wave data have now been collected and as geophysical modelling of sediments becomes more sophisticated, the necessity of shear wave data to complete these models has resulted in a substantial amount of recent effort being made to provide the required shear wave techniques.

Resonant column devices provide a controlled laboratory technique for dynamically testing soils at a range of strain levels. A solid or hollow cylindrical soil specimen is forced to vibrate at low frequencies in either a longitudinal or torsional mode. The resonant frequency is used to calculate the appropriate modulus at different strain levels. A comprehensive account of both the theory and data collected using this technique for a range of sediment types is provided by Hardin and Music (1965), Hardin and Drnevich (1972a, b) and Drnevich et al. (1978). Hardin and Drnevich (1972a, b) demonstrate that shear modulus versus strain is non-linear beyond the threshold strain where the modulus decreases hyperbolically with increasing strain. A maximum shear modulus, G_{max}, can be determined by extrapolation to zero strain. For practical purposes, $G = G_{max}$ when the strain amplitude is 0.0025 per cent or less.

While direct transmission of pulsed waves has been a highly successful technique for studying compressional waves in saturated sediments, similar reliable techniques for studying shear waves at low confining pressures have been, until recently, non-existent. Several techniques have been reported in the literature but only one provides high quality data. This has been developed by D. Shirley and others at the Applied Research Laboratories, University of Texas at Austin. It involves using highly-compliant piezo-electric bender elements as both transmitting and receiving transducers. Shirley and Anderson (1975b) and Shirley and Hampton (1978) describe measurements of shear wave velocities in high porosity kaolinite clay as low as 2 m/sec with an attenuation of 520 dB m^{-1}.

Using similar types of transducers, measurements of compressional and shear wave velocities can be made during conventional soil testing procedures. Schultheiss (1981) measured dynamic elastic moduli from wave velocities during both consolidation and triaxial testing, concluding that for a silt the dynamic constrained modulus was greater than equivalent static moduli by a factor of about 70. It was, however, suggested that a second, much slower, compressional wave might exist which may be more applicable for calculating sediment moduli as well as bringing static and dynamic moduli into close agreement.

Similar difficulties exist for measuring the shear wave velocity of marine sediments in situ. However, some techniques have been tried. Hamilton et al. (1970) (using divers and a submersible) and Davies (1965) have made Stoneley wave measurements using explosive detonator sources and an array of receivers on the sediment surface. Shear wave velocities can be computed from Stoneley wave group velocities dispersion curves.

Schwarz and Conwell (1974) developed and used successfully a shear wave refraction technique on the seafloor at a depth of 300 metres. The source consisted of a heavy (77 kg) iron slug acting as an electromagnetic hammer against the end plates of a tube. Two solenoids at each end of the tube, driven by discharges from a capacitor bank, enabled repeatable and reversible shots to be fired. An array of horizontal geophones on gimbals, extending over a 300-metre line, were used as receivers. Results showed velocities increasing from 38 m/sec at the sediment surface to 345 m/sec at a calculated depth of 90 metres.

More recently, a method using heavy weights as sources and ocean bottom seismographs as receivers has made shear wave observations possible over distances greater than 1 km on the ocean bottom (Whitmarsh and Lilwall, 1982).

In general, there is a severe lack of shear wave velocity data in ocean sediments both in the laboratory and especially in situ. The sensitivity of the shear wave velocity to disturbance probably means that accurate in-situ measurements are required for a detailed sediment assessment, certainly until the disturbance factor is properly quantified. It is interesting to note at this point that the changes in shear wave velocity from its in-situ value, which occur during the coring procedure as the core is raised through the water column, and during subsampling on deck would probably be highly indicative of the relative disturbance caused by sampling ocean bottom sediments. This in turn may throw some light on the validity of various laboratory measurements and their applicability to in-situ conditions.

Although there are many difficulties with obtaining reliable values of the acoustic velocities, V_p and V_s, as well as with the interpretation of the associated elastic moduli, they do have a major advantage in that they can be measured in situ (although probably with some difficulty in the case of V_s) and will represent a large volume of the sediment in question.

7. CONCLUSIONS

This review has attempted to illustrate how some of the basic principles of soil geotechnics can be applied to deep sea sediments. Problems associated with both types of test methods have been highlighted for the case of deep sea sediments and, consequently, it is appropriate to summarise the conclusions that have been drawn.

Laboratory tests obviously provide the best technique for determining those geotechnical sediment characteristics which only require a representative sample to be obtained. These include the particle mineralogy, grain size and shape distributions and the Atterberg limits. When testing for Atterberg limits in deep sea sediments, the samples must not dry out and water of the same salinity must be used in the test.

For most other tests of geotechnical interest, the sampling procedure will to some extent effect the property to be measured. The two causes of this are: (a) the structural disturbance casued by sampling, and (b) the effects of scale (unrepresentative sampling and the imposition of artificial boundary conditions). The crux of the problem is not whether the sample has been disturbed but by how much it has been disturbed with reference to the parameter to be evaluated. Possibly the least disturbed parameters are those involving volumetric and weight relationships, i.e. bulk density porosity, void ratio and water content. However, the disturbance caused by the expansion of pore water when samples are recovered from oceanic depths and the relative disturbances caused by different types of corers on varying sediment lithologies is still unknown.

Permeability is a very important geotechnical parameter that on land has been found to exhibit large differences in its value depending on whether laboratory or in-situ tests are performed. The major discrepancy arises as a result of unrepresentative sampling rather than physical disturbance to the sample. Although there is no directly comparable data for deep-sea sediments, it is possible that the situation may, at least in many areas, be different from the terrestrial experience. Many areas of deep sea sediments probably do not consist of the inhomogeneities, in terms of fractures and fissures, which drastically effect the permeability of terrestrial soils. Consequently, high quality sampling and laboratory determined permeabilities, covering the sediment lithologies to the depth of interest, may accurately represent the in-situ condition. However, it would be premature to draw this as a firm conclusion without direct experimental evidence.

The other main class of geotechnical parameters of interest are those which measure its stress-strain and strength characteristics. This area is littered with problems when trying to compare the data obtained in situ and from laboratory tests. Differences in the data can arise from two primary sources: (a) sample disturbance (both stress relief and physical alteration of the sediment fabric), and (b) interpretation of the different test techniques used for laboratory and in-situ testing. Samples obtained from small diameter, long piston corers cause the greatest amount of physical disturbance and are generally thought to be unsatisfactory for strength testing. Where the data from large diameter, short gravity core samples have been compared with in-situ tests, a large amount of scatter in the data precludes a meaningful comparison. Generally, the in-situ measurements provide higher stiffness and shear strengths compared to laboratory tests but this may be attributed to differences in test techniques as well as to disturbance. Even when a similar technique is used both in situ and on the recovered core samples, subtle differences in test procedures (for example, with the shear vane test either the angular velocity or vane size or both may differ) make it impossible to draw firm conclusions as to the validity of a given value. The consolidation test does seem to offer a quantitative evaluation of sample disturbance using the rate of change of slope on the e - log P plot around the maximum past preconsolidation stress. However, for near surface samples, this change of slope is much less distinct and cannot offer the same assessment of disturbance.

Much of the data available on shear strengths and large strain moduli of deep sea sediments as measured in the laboratory should probably still be regarded as unreliable. In-situ measurements of strength measured by methods such as the vane test are relatively crude and open to interpretation as to their applicability to real loading problems. They are, if carefully made, preferable to sophisticated laboratory tests using laboratory samples which may no longer accurately represent the sediment prior to being sampled.

Techniques such as the pressuremeter and large plate load tests would appear to offer the best technique for obtaining large strain moduli while acoustic tests provide the low strain moduli, both of which can be used for numerical modelling. These tests in deep ocean sediments are obviously expensive, hence, the end purpose of the tests should always be kept clearly in sight. Soils and marine sediments have particularly complex stress-strain-time characteristics which cannot be defined by a mere handful of parameters. Special tests may be more suited to many applications rather than the results from standard methods.

Assuming in-situ tests are required for a given purpose, it would seem prudent to conduct tests which most closely provide the information required rather than spending time and money on determining detailed parameters from more standard methods whose interpretation is uncertain. For example, if the depth of penetration of an object into the seafloor is required, by far the best and most reliable technique would be a simulated experiment to measure its penetration directly. To evaluate its penetration from in-situ measurements of moduli and strength would probably result in an equal, if not greater, amount of time and money being spent and provide an answer which would not compare in accuracy to the simulated test. This argument can be applied equally to many other aspects of sea floor engineering that are likely to arise in the future.

ACKNOWLEDGMENTS

I would like to thank R.B. Whitmarsh, M. Noël and T.J.G. Francis for their critical reviews of this paper and their enlightening comments. I am also grateful to P.P.E. Weaver for clarifying Chapter 3.

This research has been carried out under contract for the Department of the Environment, as part of its radioactive waste management research programme. The results will be used in the formulation of Government policy, but at this stage they do not necessarily represent Government policy.

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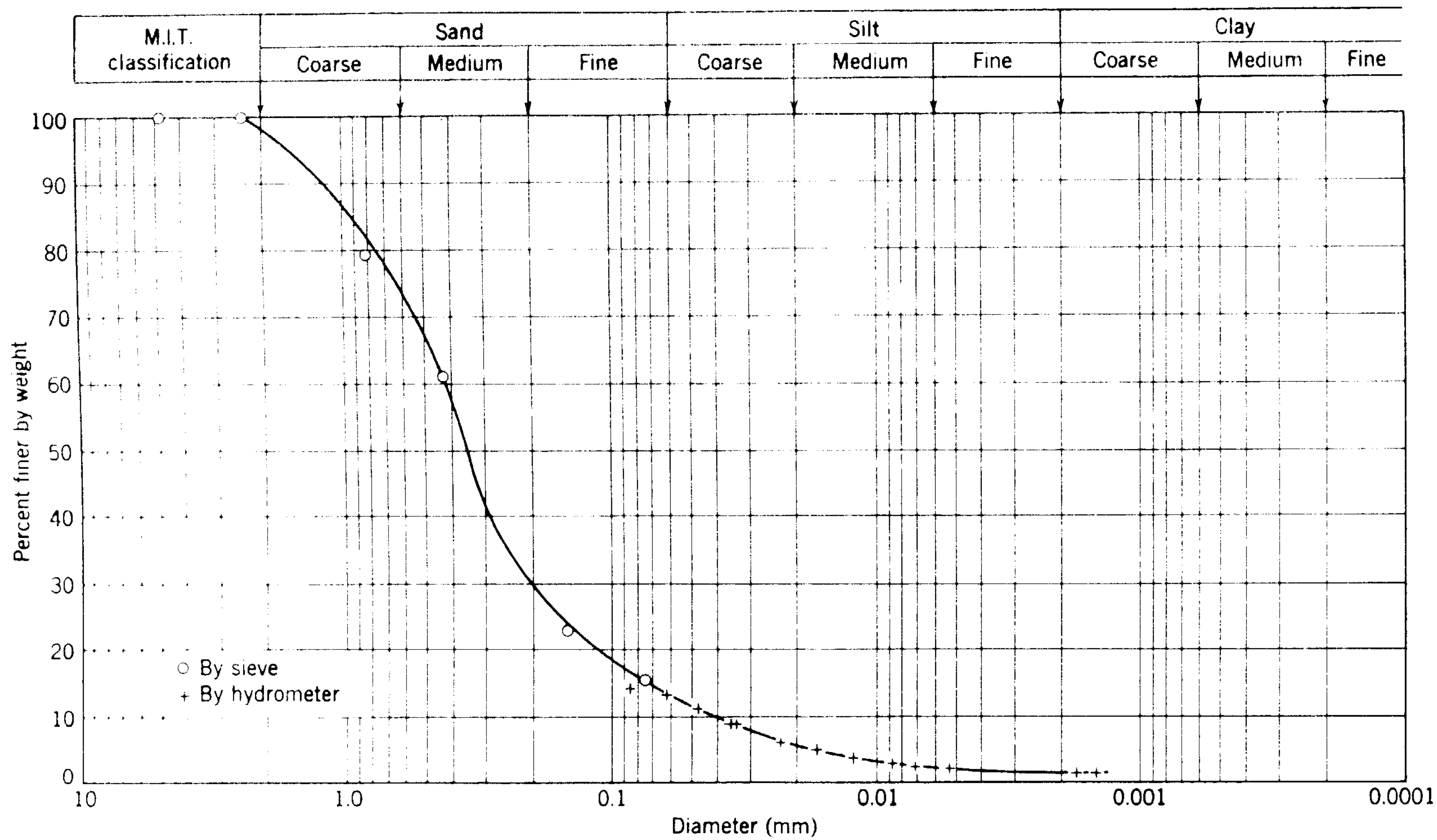
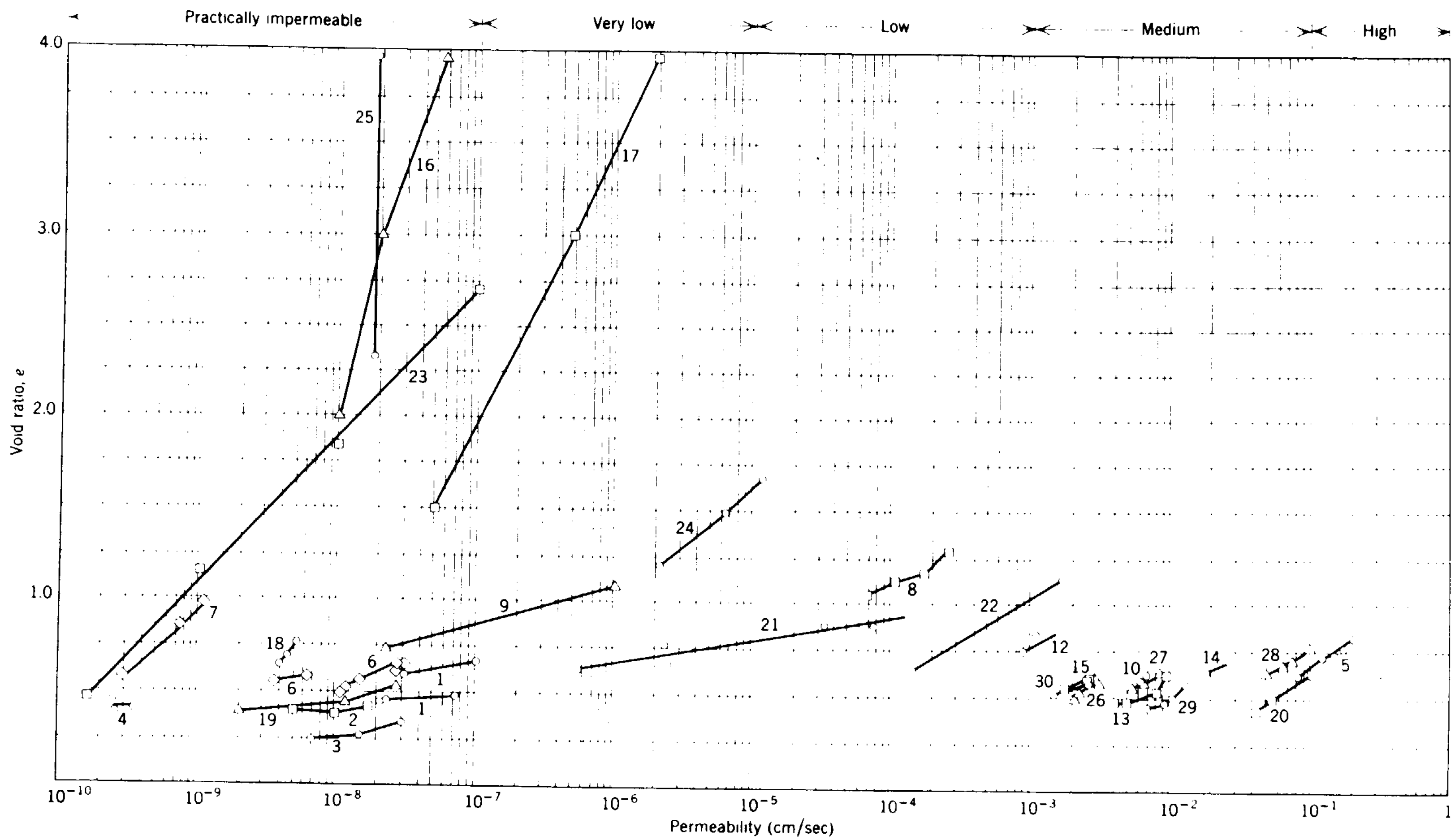


Figure 2.1 Particle size distribution curve (from Lambe, 1951)



Soil Identification Code

1 Compacted caliche
2 Compacted caliche
3 Silty sand
4 Sandy clay
5 Beach sand
6 Compacted Boston blue clay
7 Vicksburg buckshot clay
8 Sandy clay
9 Silt Boston
10 Ottawa sand
11 Sand Gaspee Point
12 Sand Franklin Falls
13 Sand Scituate
14 Sand Plum Island
15 Sand Fort Peck
16 Silt Boston
17 Silt - Boston
18 Loess
19 Lean clay
20 Sand Union Falls
21 Silt - North Carolina
22 Sand from dike
23 Sodium -Boston blue clay
24 Calcium kaolinite
25 Sodium montmorillonite
26 30 Sand (dam filter)

Figure 2.2 Permeability test data (from Lambe and Whitman, 1979)

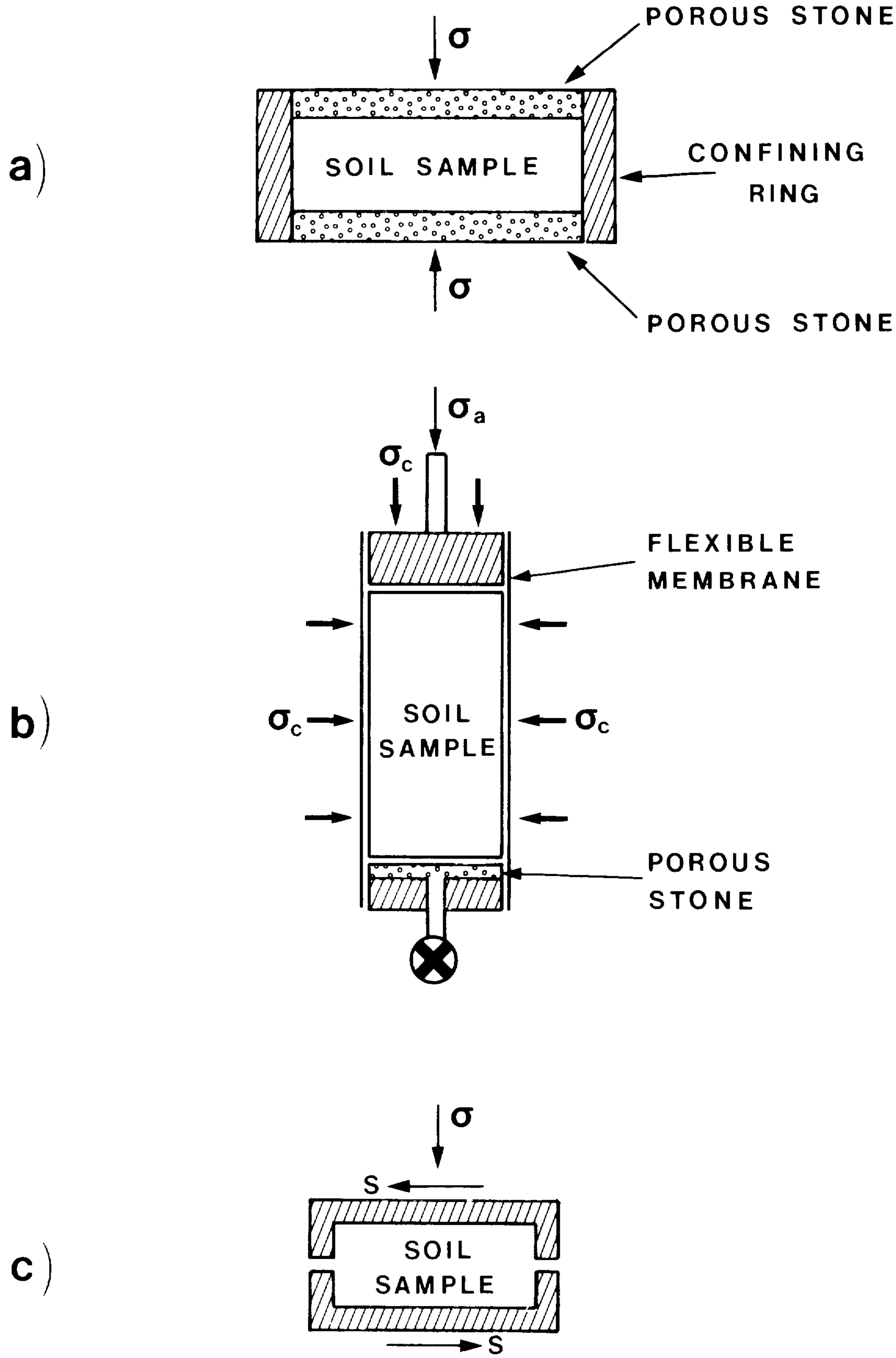


Figure 2.3 Schematic diagrams of soil tests,
a) the consolidation test, b) the triaxial test.
c) the direct shear test.

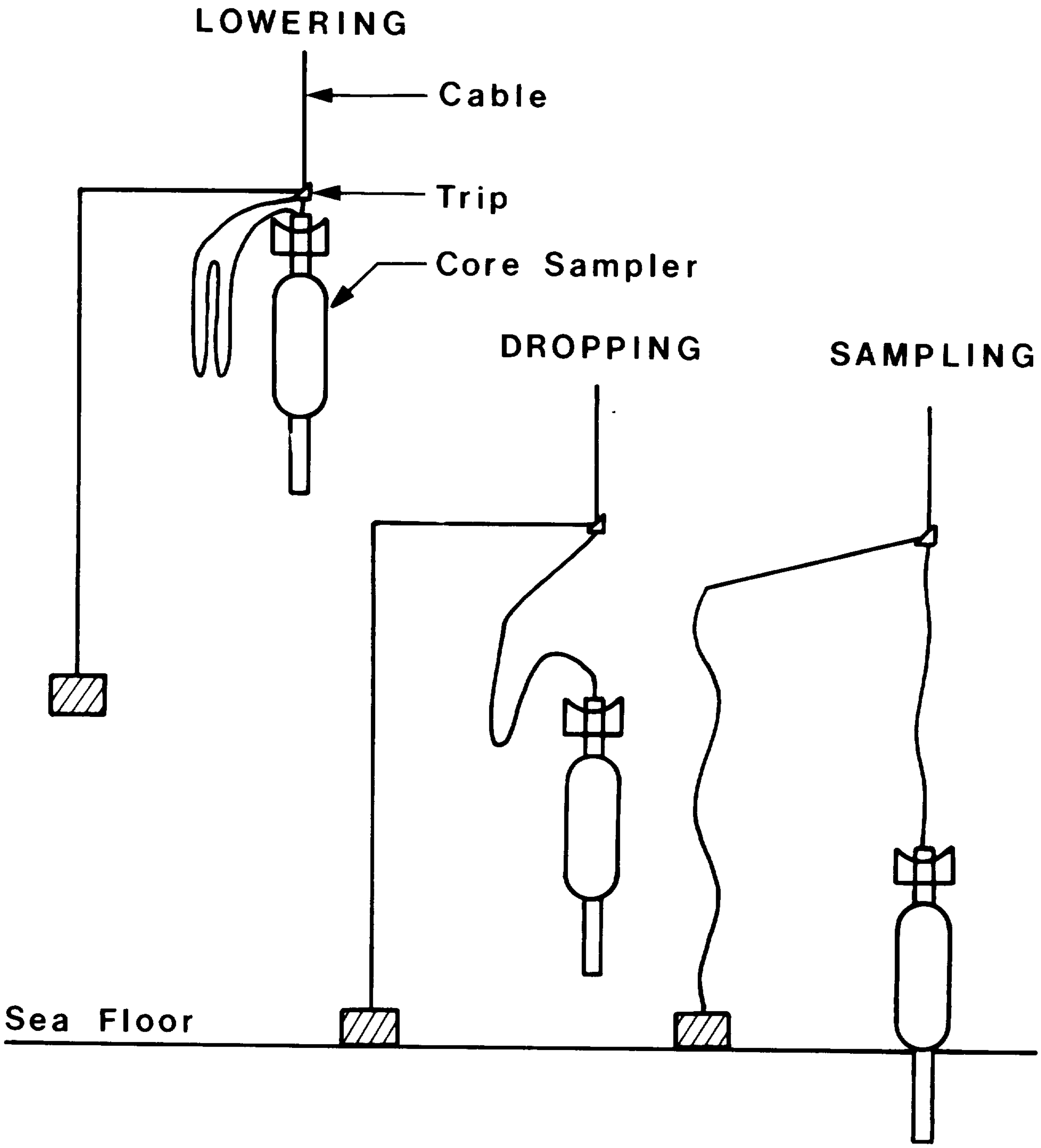


Figure 4.1 Operating principle of the gravity core sampler (redrawn from Le Tirant ,1979)

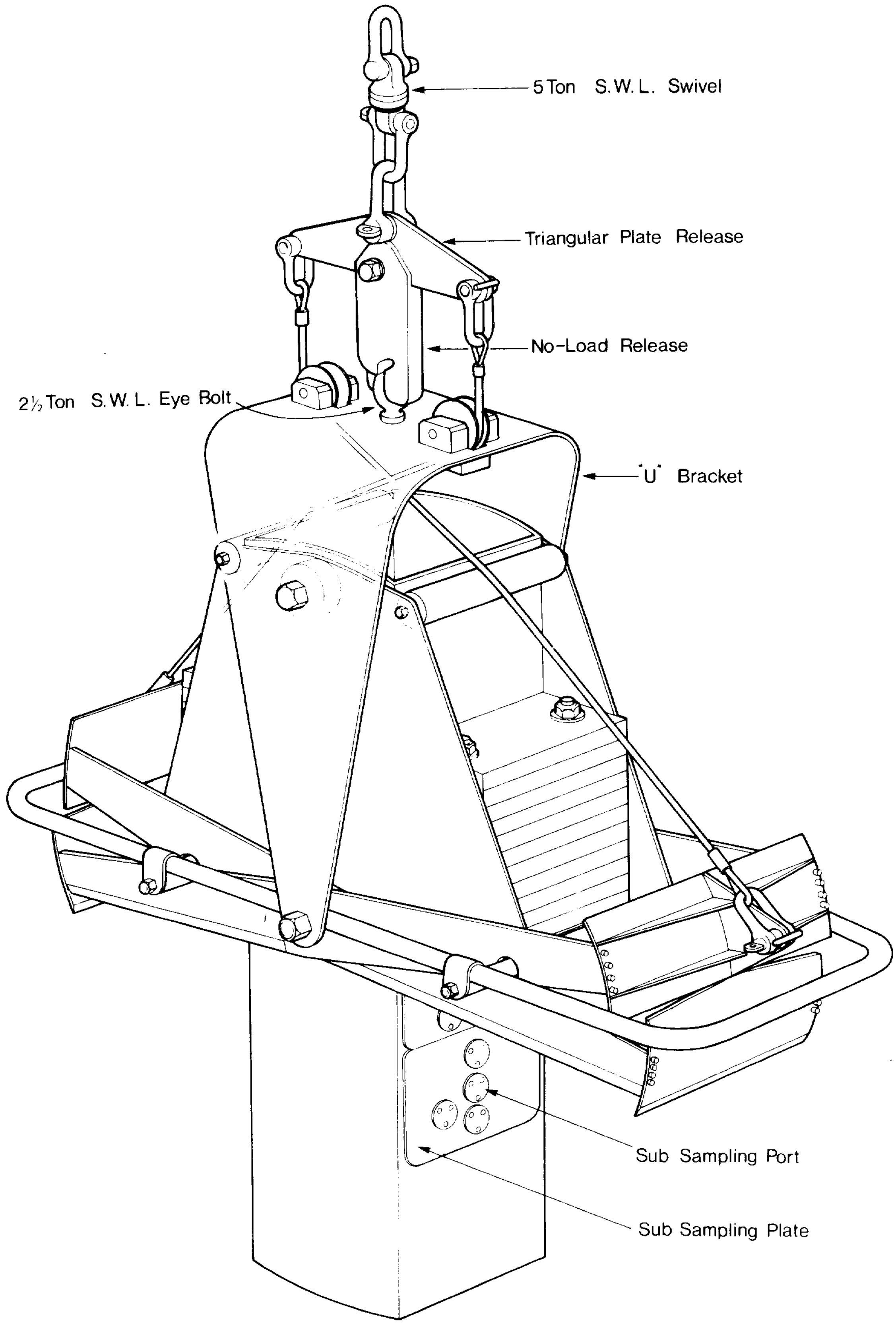


Figure 4.2 The IOS box corer (from Peters et al 1980)

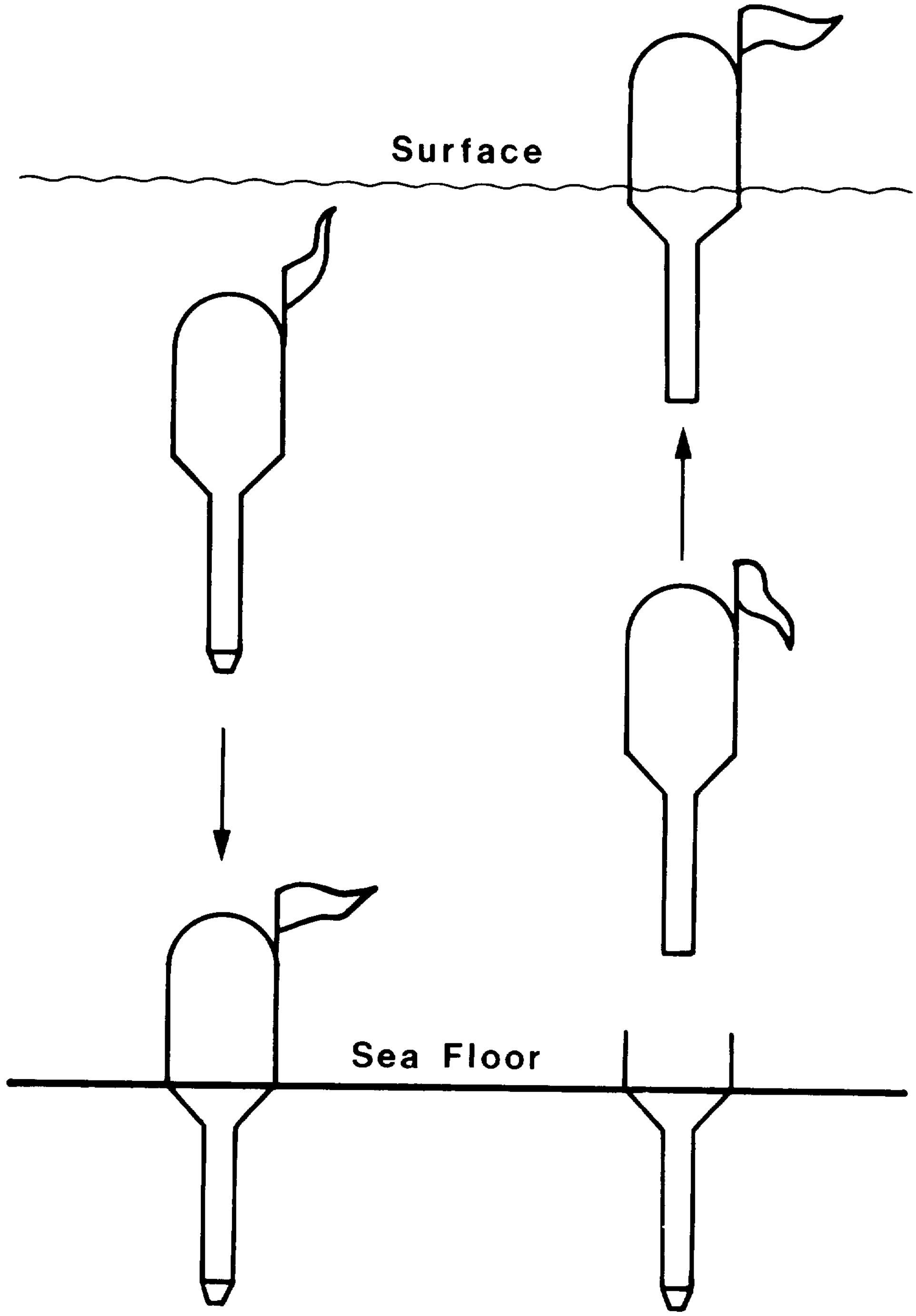


Figure 4.3 Operation of free corer (redrawn from Moore,1961)

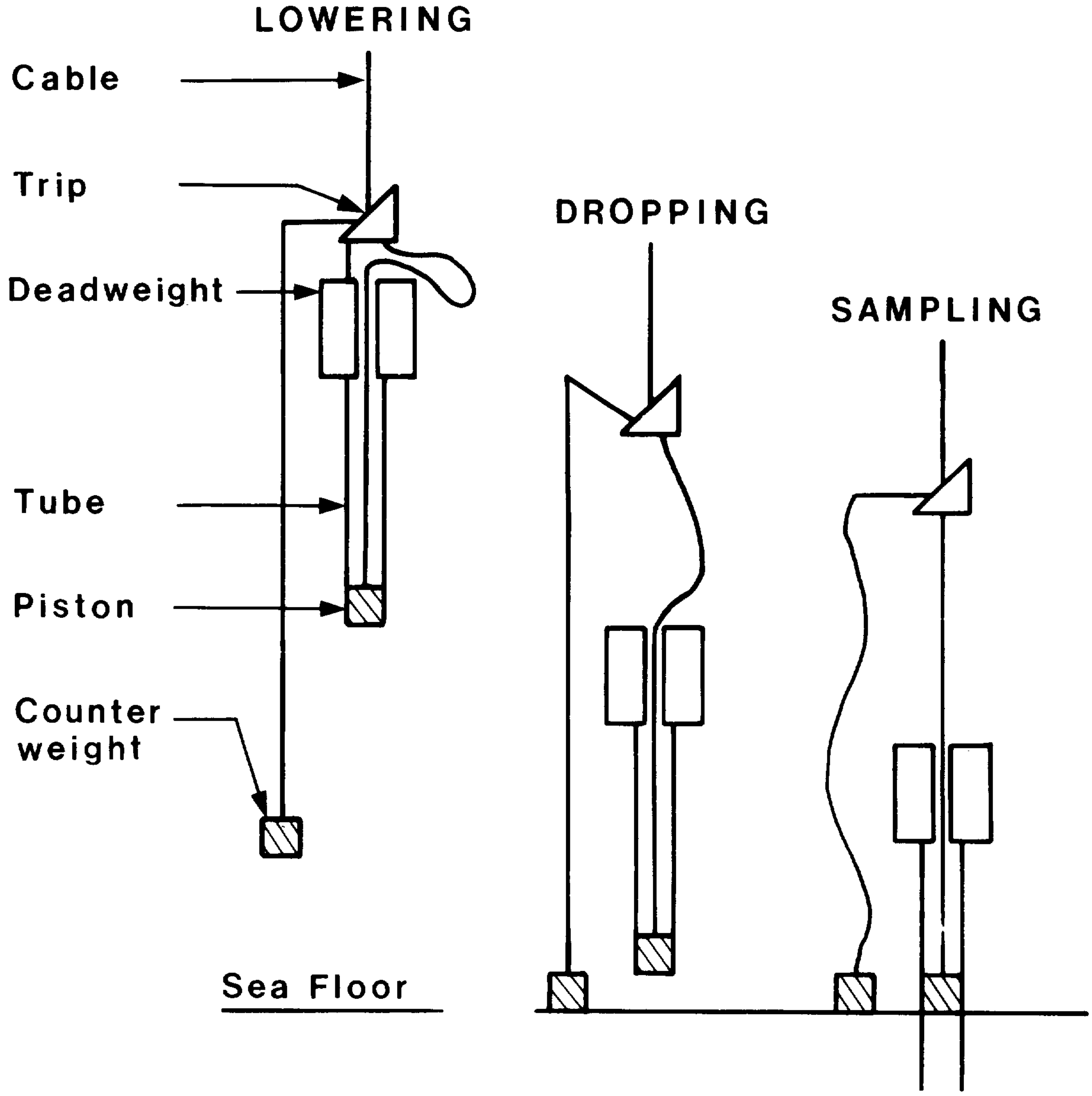


Figure 4.4 Operating principle of the stationary piston core sampler (redrawn from Le Tirant .1979)

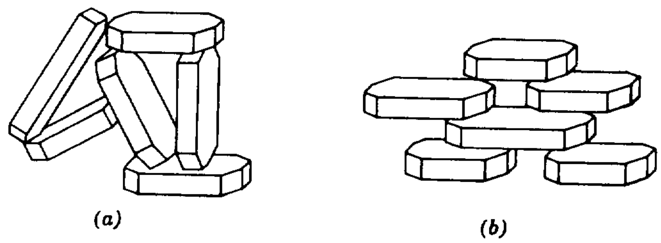


Figure 5.1 Types of soil structure. (a) Flocculated (b) Dispersed (from Lambe and Whitman, 1979)

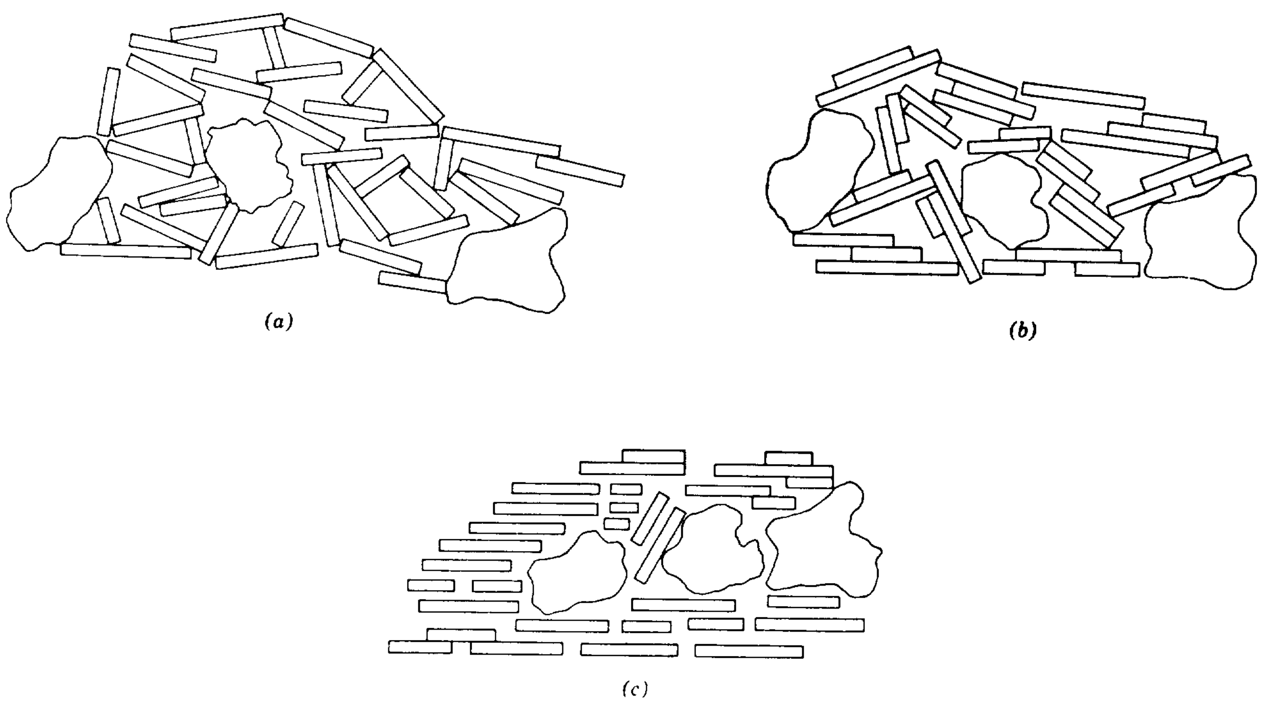
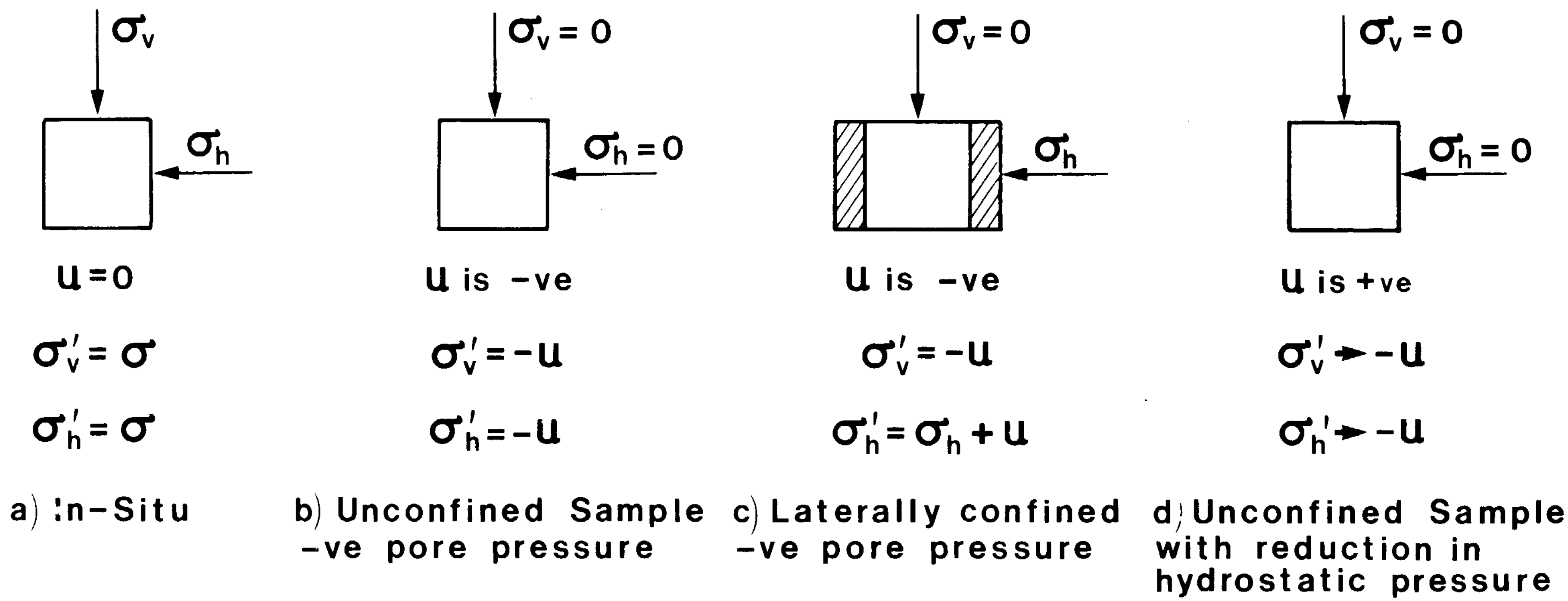


Figure 5.2 Structure of natural soil. (a) Undisturbed salt water deposit. (b) Undisturbed fresh water deposit (c) remoulded (from Lambe and Whitman, 1979)



σ_v = total vertical stress

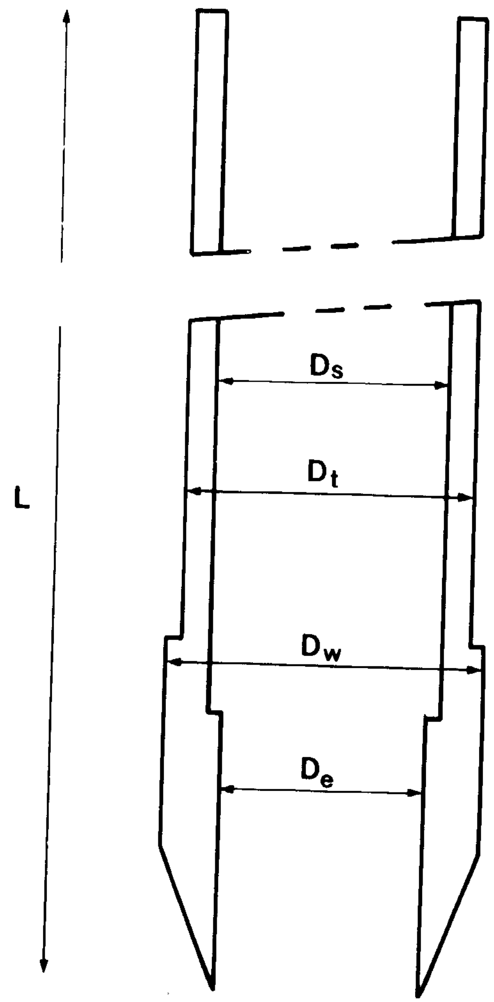
σ_h = total horizontal stress

σ'_v = effective vertical stress

σ'_h = effective horizontal

u = excess pore pressure

Figure 5.3 In-Situ and Sample Stress Systems



Inside clearance ratio,

$$C_i = \frac{D_s - D_e}{D_e} \times 100$$

Outside clearance ratio,

$$C_o = \frac{D_w - D_t}{D_t} \times 100$$

Area ratio,

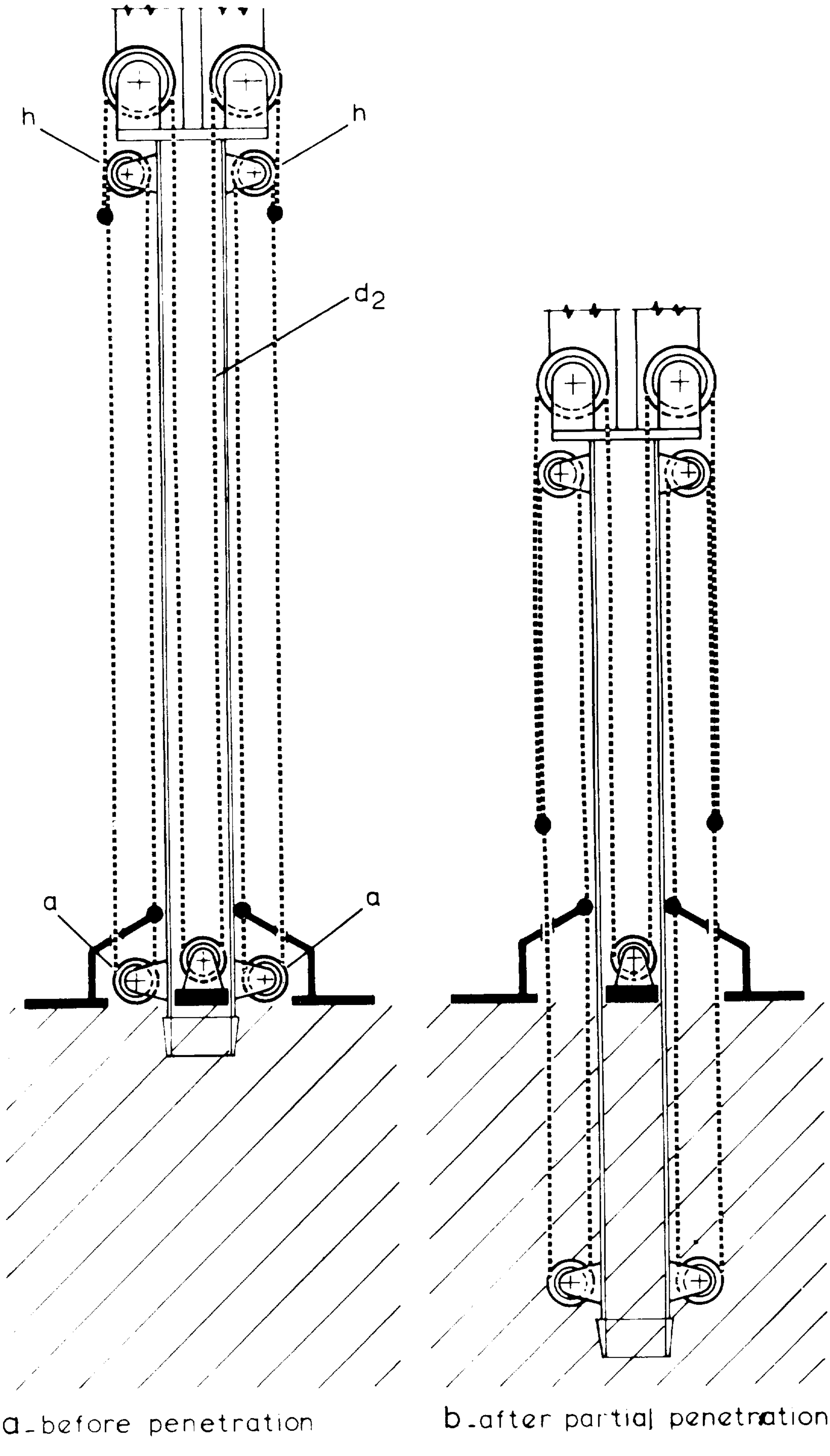
$$C_a = \frac{D_w^2 - D_e^2}{D_e^2} \times 100$$

Length - width ratio,

$$C_l = \frac{L}{D_s}$$

L = maximum sampling length

Figure 5.4 Dimensions and coefficients of coring device



PRINCIPLES OF THE ACTUAL DESIGN, INCORPORATING MODIFICATIONS

Figure 5.5 Sphincter corer with recoilless piston (from Kermabon and Cortis, 1968)