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by

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ABSTRACT

The measurement of soil properties in-situ offers the advantages of minimal disturbance, retention of the in-situ state of stress, temperature, chemical and biological environments, and cost effectiveness relative to many types of laboratory tests for evaluation of undisturbed soil properties.

This report is concerned with techniques for in-situ measurement of permeability, strength, stress-deformation properties, and volume change properties; property classes which are of interest in most geotechnical engineering problems. Emphasis is on test concepts, data analysis and interpretation, and advantages and limitations of methods, as opposed to details of apparatus and procedure.

Permeability (hydraulic conductivity) is often measured in-situ by means of inflow or outflow borehole pumping tests employing either constant or falling heads. Large scale pumping tests provide the most accurate results, but their high cost generally restricts their use to large projects.

Tests for determining shear strength include the standard penetration test (SPT), static cone penetration, vane shear, pressuremeter, and the Iowa borehole shear tests. The SPT has been the most widely used, but is least accurate. The static cone and pressuremeter provide more reliable data and are expected to be increasingly employed. The vane shear test, previously considered to be very reliable in soft clays, is now known to frequently overestimate soil strength. The borehole shear test, a rapid and low cost technique, is limited to soils with some cohesion suitable for stage testing.

In-situ stresses may be determined by pressure cells, hydraulic fracturing and the pressuremeter; deformation characteristics by the pressuremeter, plate load tests, seismic methods and back analysis of completed projects. The pressuremeter is seen as being of great promise. Hydraulic fracturing tests, while suitable for use in rock, provide results which are often extremely difficult to interpret in soils. Seismic methods of determining elastic moduli involve very small strains, so results need to be corrected before application in most cases. Plate load tests can yield accurate estimates of properties, but costs may be prohibitively high.

Volume change parameters have not often been measured by in-situ methods. However, the following techniques may be successfully employed: borehole permeability tests, penetration resistance (both dynamic and static), plate bearing tests, screw plate tests, and the pressuremeter. In-situ permeability tests are particularly well suited for the evaluation of the consolidation rate of fine grained soils. Correlations of penetration resistance with volume change characteristics are empirical and may yield misleading results. Both the load bearing and screw plate tests can provide reliable volume change parameters. However, because of practical time limitations, their use is mostly restricted to sands and slightly cohesive soils.

State of the art equipment, testing techniques and evaluation methods, as reviewed in this report, can provide many geotechnical design parameters with a degree of accuracy which compares favorably with conventional laboratory testing, often with substantial savings in cost and time. It is anticipated that existing methods for the in-situ measurement of soil properties will be even more widely accepted, further developed and supplemented by the introduction of new techniques in the foreseeable future.

I. INTRODUCTION

The determination of soil properties by in-situ measurement has assumed greatly increased importance in Geotechnical Engineering in recent years. Improvements in apparatus, instrumentation, measurement techniques and analysis procedures have been significant. It is probable that for many projects the determination of properties by in-situ measurement will assume an importance equal to, or greater than laboratory testing.

There are several reasons for in-situ testing, including:

1) To determine properties of soils, such as continental shelf and sea floor sediments and sands, that can't be easily sampled in the undisturbed state.

2) To avoid some of the difficulties of laboratory testing, such as sample disturbance and the

proper simulation of in-situ stresses, temperature, and chemical and biological environments.

3) To test a volume of soil larger than can conveniently be tested in the laboratory.

4) To increase the cost effectiveness of an exploration and testing program.

In-situ tests cannot be considered a panacea, however, for the following reasons:

1) Some of the tests may not be cost effective in all cases.

2) Uncertain empirical correlations between measured quantities and properties are often used.

3) Flow (in permeability tests) and stress directions cannot be independently varied in most cases. Principal stress directions in the test may differ from those in real problems.

4) The possible effects of future changes in environmental conditions cannot be readily determined.

This report considers the evaluation of soil properties that are needed for engineering analyses from the results of in-situ tests. Details of test apparatus and measurement methods are not considered except as they influence the values obtained for the properties of interest.

There are five property classes, one or more of which may be important in most geotechnical problems:

1) Permeability (hydraulic conductivity)

2) Strength

3) In-situ stress and deformation characteristics

4) Volume change characteristics

5) Durability or susceptibility to changes in properties due to time and changes in environmental conditions.

This report is concerned mainly with the first four of these property classes. In-situ tests are not generally suitable for prediction of the fifth property class, but they are very appropriate for monitoring changes with time. Some of the in-situ test types, such as pressuremeter and penetration tests, can be used to deduce information about more than one property. Similarly, some of the principles and considerations in testing relate to all the properties.

This report draws heavily on material presented at, and contained in, the proceedings of two recent conferences; namely the European Symposium on Penetration Testing (ESOPT), held in Stockholm, June 1974, and the ASCE Geotechnical Engineering Division Speciality Conference on In-Situ Measurement of Soil Properties, held at North Carolina State University, June 1975. It is organized in terms of property class rather than by test type. A list of pertinent references summarizing the current state-of-the-art is appended to this report.

II. PERMEABILITY

A. Introduction

Many difficult construction problems are directly related to the presence of ground water flow. Groundwater seepage has been responsible for slope and base instability in excavations, for face and roof instability in tunnels and for piping and erosion in earth/rock dams. Seepage losses in dam foundations or from reservoirs directly influence the safety and economics of water conservation projects. Large local variations of permeability are a common condition and lead to flow concentrations which may cause dewatering difficulties such as local piping. The success of many projects therefore depends upon a knowledge of in-situ permeability and its degree of variation.

All soil and rock deposits are permeable to some degree. The coefficient of permeability, k, probably exhibits the largest range of magnitude of any parameter indicative of geotechnical properties. The general range of in-situ permeability values for different soil and rock types is presented in Figure II-1.

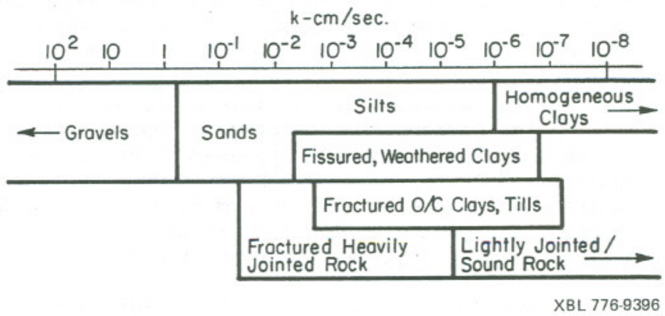


Fig. II-1 Approximate range of permeability (k) in soil and rock (from Milligan, 1975)

An analysis of dozens of dam foundations has shown that a conceptually correct, simplified approach based on reliable geologic data will yield equally good results as the most involved mathematical simulations (Milligan, 1975). In many cases, if the coefficient of permeability can be evaluated to within an order of magnitude a safe, economical design will be possible.

For many applications, the object of a permeability measurement is simply to determine if a problem exists. If so, a defensive design consisting of four steps may be appropriate (Gordon, 1975):

1) Assess the problem.

2) Design to accomodate a chosen range of permeability.

3) Monitor subsequent behavior.

4) Take appropriate action.

Defensive design may be applied to seepage control for dams and reservoirs, storage of contaminated water, disposal of solid wastes and prevention or correction of landslides.

In-situ permeability values may be utilized in fluid flow calculations or in conjunction with laboratory measurements of stress-strain behavior in order to deduce rates of consolidation.

B. Fluid Flow in Soils and Rock

In soils and rocks, the flow of water through pore space, voids, discontinuities or cracks is usually assumed to be laminar and to obey Darcy's Law, i.e.,

$$v = k \cdot i \tag{II-1}$$

where:

v = flow velocity

i = hydraulic gradient, head lost per unit length of flow path

k = coefficient of permeability (hydraulic conductivity), normally expressed in velocity units

Experience has shown that this relationship is valid for a wide range of soil conditions. It is possible, however, that the presence of preferred flow paths in an otherwise essentially impervious material may decrease the significance of an "average" permeability value.

In addition to being the governing equation for fluid flow in geotechnical problems, Darcy's law also provides the basis for the measurement of permeability. The procedure common to all direct methods of permeability measurement is to impose a hydraulic gradient and measure the resulting flow across a known cross sectional area. Darcy's law is then employed to calculate a value of permeability. Deviations from direct proportionality between flow velocity and gradient may sometimes develop when changes in soil structure occur during flow (Mitchell, 1976).

C. Variation of In-Situ Permeability

The in-situ permeability of soil and rock is influenced by both microscopic and macroscopic features. Microscopic properties such as void ratio and particle size, shape and orientation may, in some cases, be retained through undisturbed sampling.

The in-situ permeability of clean sandy soils, which lose their in-situ structure during even careful sampling, may be estimated in the laboratory by testing samples which have been carefully recompacted to the appropriate relative density. At best, such a procedure may provide an estimate of in-situ permeability, since laboratory compaction cannot be expected to reproduce a complicated in-situ soil structure which is the product of many physical and chemical processes.

The effects of important macroscopic features, such as sand lenses, fissures and clay seams, cannot usually be duplicated in laboratory testing. However, it is these macroscopic features which will most likely govern field behavior.

A comparison of corresponding field and laboratory permeability test results will usually show the field permeability to be considerably, but unpredictably, higher than the values measured in the laboratory. A reliable in-situ determination of permeability is therefore a necessity for many projects.

D. Direct Measurement of In-Situ Permeability

Methods which are commonly used for the direct measurement of permeability have been summarized by Milligan (1975) and are reproduced in Table II-1 of this report. The procedural details of these tests are thoroughly discussed in the references cited in Table II-1.

It has been noted that the approach which is common to all these direct methods is to impose a known hydraulic gradient and measure the resulting flow. If a constant hydraulic gradient is maintained during testing the test may be classified as a "constant head" test. If the imposed hydraulic gradient decreases or varies with time during testing, the test is classified as a "falling head" or "variable head" permeability test.

If the induced flow during testing proceeds from the measuring device into the soil to be tested, the test is described as being an "in-flow" test. Similarly, if the imposed gradient induces flow from the soil mass into the measuring device, the test is classified as an "out-flow" test.

In-flow tests tend to clog the soil voids with dislodged particles, resulting in the measurement of a coefficient of permeability, k_{in}, that may be lower than the true in-situ value. Outflow tests tend to erode particles from the soil skeleton, increasing soil porosity and resulting in the measurement of a coefficient of permeability, k_{out}, which may be higher than the true in-situ permeability.

The value of the ratio, (k_{out}/k_{in}) may be as large as 500 in extreme cases (Milligan, 1975). According to Milligan, the true in-situ permeability, k, can be estimated by:

$$k = \sqrt{(k_{out}) \cdot (k_{in})} \tag{II-2}$$

Use of this equation implies a judgment that the true in-situ permeability lies between k_{in} and k_{out}, but is closer to k_{in} than k_{out}. Therefore, the implicit assumption is that the erosion effect of outflow tests is more severe than the clogging effect of in-flow tests.

In the remainder of this section, the methods of in-situ permeability measurement are examined in more detail, together with their applicability to engineering practice.

The Borehole Permeability Test

Borehole permeability tests are performed by pumping water either out of a borehole (drawdown test) or into a borehole (infiltration test). The test may be performed as a constant head test in which the rate of pumping which is necessary to maintain a constant water level in the borehole is measured. Alternatively, a variable head test may be performed by observing the change in water level in the borehole after pumping has stopped.

A fully or partially cased borehole is usual-

TABLE II-1. DIRECT TESTING OF IN SITU PERMEABILITY IN SOILS
(from Milligan (1975))

METHOD	TECHNIQUE	APPLICATION TO: GRAVEL	SAND	SILT	CLAY	PROBLEMS	METHOD RATING	REFERENCE
A Augerhole	Shallow uncased hole in unsaturated material above G.W.L.	✓	Only where $k>10^{-3}$ cm/sec	?	-	Difficult to maintain water levels in coarse gravels	Poor	USBR, (1974)
A Test Pit	Square OR rectangular test pit (equivalent to circular hole above)	✓		?	-		Poor	Lacroix (1960)
B Cased borehole (no inserts)	i) Falling/rising head, Δh in casing measured VS time	✓	✓	?	-	Borehole must be flushed. Possible lines clog base (falling Δh).	Fair	Hvorslev (1951)
	ii) Constant head maintained in casing, outflow, Q VS time	✓	✓	?	-	Pumping (rising Δh) where WL lowered excessively.		USBR, (1974)
C Cased borehole (inserts used) i) Sand filter plug	i) Generally falling head Δh measured VS time only	✓	✓	?	-	Single tests only. Cannot be used as boring is advanced.	Fair	Hvorslev (1951)
ii) Perforated/slotted casing in lowest section	ii) Variable heads possible	✓	✓	?	-			
iii) Well point placed in hole, casing drawn back	iii) As for (ii) above	✓	✓	✓	-		Fair to Good	
D Piezometers/Permeameters (with OR without casing)	i) Suction Bellows apparatus (independent of boring) inflow ONLY measured VS time	-	?	✓	-	Restricted to fine sands, coarse silts, variable bellows required 'k' range 10^{-4} to 10^{-7} cm/sec.	Good (local zones)	Golder, Gass (1963)
	ii) Short Cell (Cementation) (independent of boring) Outflow ONLY measured VS time	✓	✓	-	-	Carried out in adit OR tunnel		Golder, Gass (1963)
	iii) Piezometer tip pushed into soft deposits/ placed in boring, sealed, casing withdrawn/pushed ahead of boring. Constant head, outflow measured VS time. Variable heads also possible	-	-	✓	✓	Possible tip 'smear' when pushed. Δu set up in pushing tip. Danger of hydraulic fracture		Gibson (1966) Wilkinson (1968) Hvorslev (1951) Bjerrum et al, (1972)
E Well pumping test	Drawdown in central well monitored in observation wells on, at least, two 90° radial directions	✓	✓	✓ (?)	-	Screened portion should cover complete stratum tested	Excellent (Mass permeability of foundation material)	Todd (1959)
F Test excavation pumping test(s)	Monitoring more extensive than E , during excavation dewatering (Initial construction stage)	✓	✓	✓ (?)	-	Expensive, but of direct benefit to contractual costing		--

ly employed in this method. The permeability of a localized zone of soil surrounding the base and uncased portions of the borehole is computed by the application of theoretically derived equations. The equations which are appropriate for a number of test configurations are reproduced in Figure II-2, from Hvorslev (1951).

The borehole permeability test is usually performed as an in-flow test, partially because inexpensive pumps are limited in their ability to affect drawdown at a constant pumping rate, and partially because it is usually difficult to accurately monitor the drawdown water level in the borehole. Constant head tests are preferred to variable head tests, because they are easier to perform properly and have been found to provide more reliable and consistent data, as discussed by Schmidt et al. (1976).

Permeability testing in a single borehole is likely to yield misleading results. Two simplified geologic profiles are presented in Figure II-3. It can be visualized that the results of permeability testing in these profiles will vary greatly with borehole location and testing depth. Permeability tests in Borehole #1 will consistenly yield the fairly low value of k exhibited by the silt stratum, but the important flow paths afforded by the sandy layers will remain undetected. Test results from Borehole #2 may, or may not, reflect the presence of the pervious jointing patterns. The probability of detecting a joint during testing will depend upon the relative dimensions of the test section and the joint spacings.

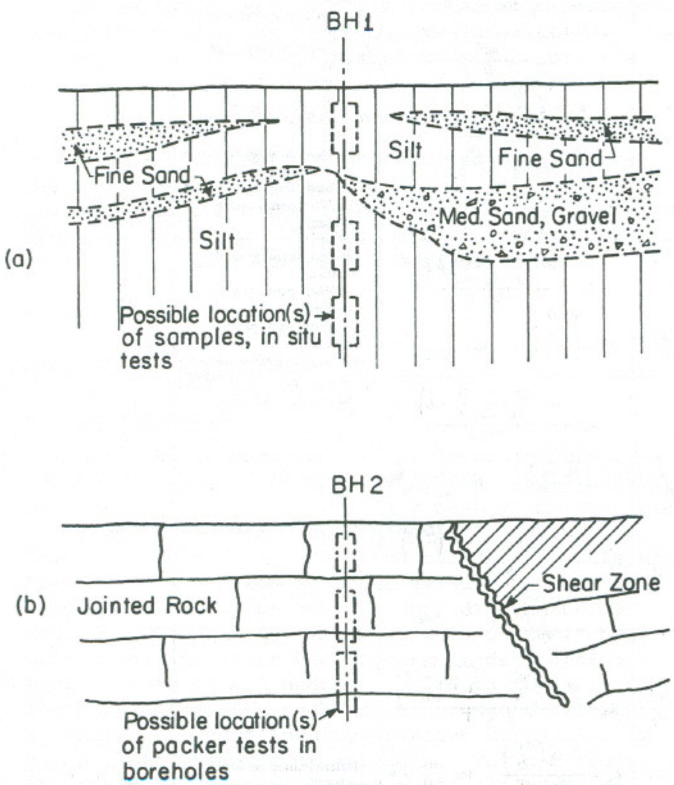


Fig. II-3 Limitations of sampling/in-situ tests in a single borehole: (a) in soils (b) in rock (from Milligan, 1975)

If such geologic discontinuities are encountered in a borehole permeability test, their presence may dominate the testing flow conditions and bias the measured value of permeability. However, the effect of geologic discontinuities on full scale performance cannot be directly predicted from their effect on small scale tests. The investigator must refer to a geologic site model, and, if necessary, perform more small scale tests or resort to full scale testing.

The primary advantage of the borehole permeability test is its simplicity. The results may be viewed as general indicators of the order of magnitude of in-situ permeability appropriate for the relatively small zone of tested soil. The uncertainties involved in a single test are significant, but their importance can be reduced by performing a large number of tests.

Large Scale Pumping Tests

Large scale pumping tests are performed by pumping water into or out of a screened well embedded below the natural groundwater table. As pumping proceeds, the resulting change of groundwater level is monitored in surrounding observation wells. Generally, four or more of these observation wells are employed. Pumping tests may be categorized as equilibrium (steady state) or non-equilibrium (transient flow) tests.

In an equilibrium pumping test, the groundwater level measurements are recorded once a constant water level has been attained in the observation wells. The time required for this equilibrium to occur may be very great, especially when the test is performed in soils of low permeability. An average value of permeability for the deposit under study can be computed from the test data by relationships such as the Thiem Formula, which are described by Lang (1967) and others.

In the non-equilibrium test, the rate of pumping, and the consequent rate of water level change in the observation wells are recorded. The test data can be analyzed by a procedure such as the one proposed by Theis (1935). The advantages of transient flow testing are that testing time may be reduced as compared with equilibrium testing, and groundwater drawdown may be controlled, or halted if necessary.

Large scale pumping tests with observation wells provide the most reliable, but most expensive, permeability data for relatively pervious deposits. Such a testing program was employed by Ahmad et al. (1975) in order to estimate seepage losses from an artificial lake. The soil deposit in question was an unconfined sand aquifer, samples of which exhibited a uniformity coefficient ranging from 2 to over 20. Surprisingly consistent values of permeability coefficients were obtained, ranging from 0.10 to 0.15 cm/sec.

Large scale pumping tests are currently the preferred method for determining the in-situ permeability for large construction projects which can justify the costs involved. The test conditions closely simulate full scale site dewatering. In fact, a functioning site dewatering system may be analyzed as a pumping test, and the

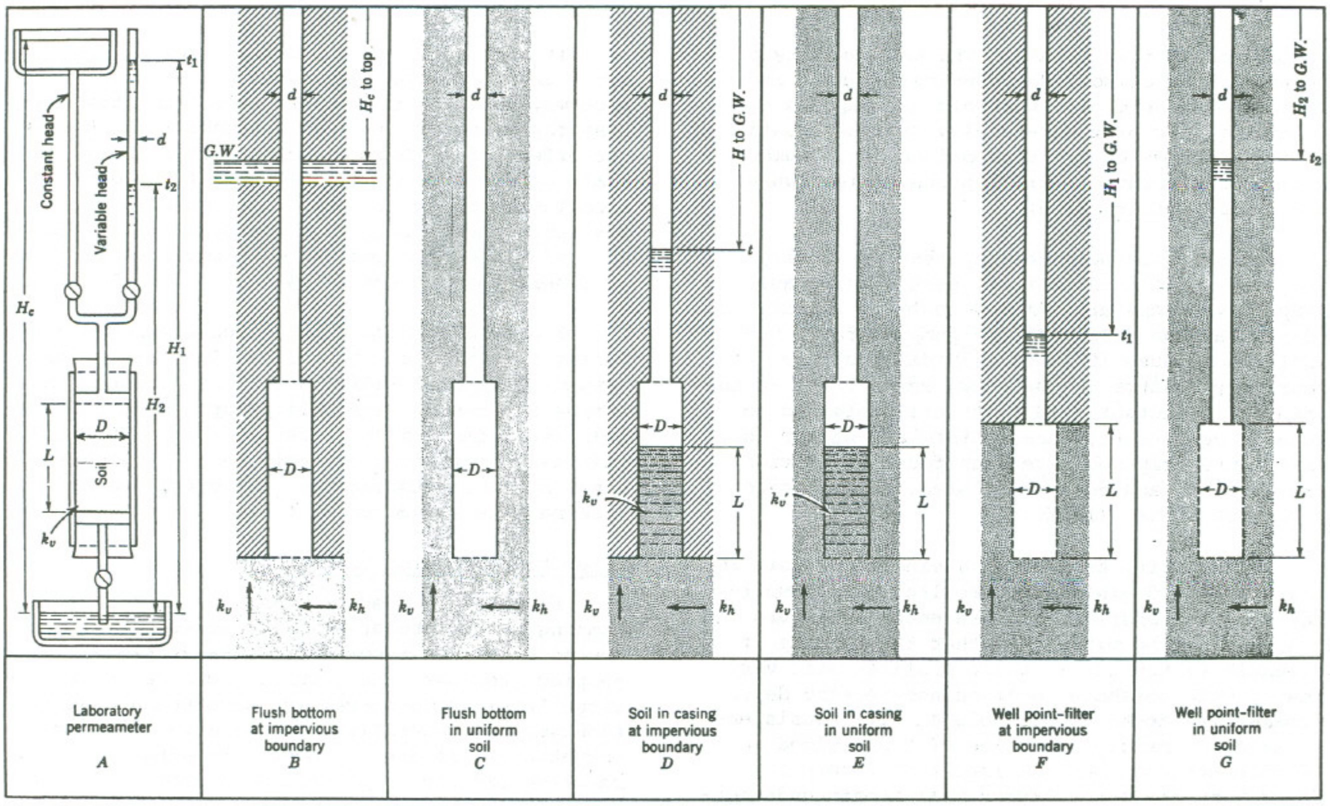


Fig. II-2. Configurations and appropriate equations for borehole permeability tests (after Hvorslev, 1951)

Case	Constant Head	Variable Head	Basic Time Lag	Notation
A	$k_v = \frac{4 \cdot q \cdot L}{\pi \cdot D^2 \cdot H_c}$	$k_v = \frac{d^2 \cdot L}{D^2 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_v = \frac{L}{t_2 - t_1} \ln \frac{H_1}{H_2}$ for $d = D$	$k_v = \frac{d^2 \cdot L}{D^2 \cdot T}$ $k_v = \frac{L}{T}$ for $d = D$	D = Diam, intake, sample (cm) d = Diameter, standpipe (cm) L = Length, intake, sample (cm) H_c = Constant piez. head (cm) H_1 = Piez. head for $t = t_1$ (cm) H_2 = Piez. head for $t = t_2$ (cm) q = Flow of water (cm^3/sec) t = Time (sec) T = Basic time lag (sec) k_v' = Vert. perm. casing (cm/sec)
B	$k_m = \frac{q}{2 \cdot D \cdot H_c}$	$k_m = \frac{\pi \cdot d^2}{8 \cdot D \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_m = \frac{\pi \cdot D}{8 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ for $d = D$	$k_m = \frac{\pi d^2}{8 \cdot D \cdot T}$ $k_m = \frac{\pi \cdot D}{8 \cdot T}$ for $d = D$	
C	$k_m = \frac{q}{2.75 \cdot D \cdot H_c}$	$k_m = \frac{\pi \cdot d^2}{11 \cdot D \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_m = \frac{\pi \cdot D}{11 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ for $d = D$	$k_m = \frac{\pi \cdot d^2}{11 \cdot D \cdot T}$ $k_m = \frac{\pi \cdot D}{11 \cdot T}$ for $d = D$	
D	$k_v' = \frac{4 \cdot q\left(\frac{\pi}{8} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right)}{\pi \cdot D^2 \cdot H_c}$	$k_v' = \frac{d^2 \cdot \left(\frac{\pi}{8} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right)}{D^2 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_v = \frac{\frac{\pi}{8} \cdot \frac{D}{m} + L}{t_2 - t_1} \ln \frac{H_1}{H_2}$ for $\begin{cases} k_v' = k_v \\ d = D \end{cases}$	$k_v' = \frac{d^2 \cdot \left(\frac{\pi}{8} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m}\right) + L}{D^2 \cdot T}$ $k_v = \frac{\frac{\pi}{8} \cdot \frac{D}{m} + L}{T}$ for $\begin{cases} k_v' = k_v \\ d = D \end{cases}$	k_v = Vert. perm. ground (cm/sec) k_h = Horz. perm. ground (cm/sec) k_m = Mean coeff. perm. (cm/sec) m = Transformation ratio $k_m = \sqrt{k_h \cdot k_v}$ $m = \sqrt{k_h / k_v}$ $\ln = \log_e = 2.3 \log_{10}$
E	$k_v' = \frac{4 \cdot q \cdot \left(\frac{\pi}{11} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right)}{\pi \cdot D^2 \cdot H_c}$	$k_v' = \frac{d^2 \cdot \left(\frac{\pi}{11} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right)}{D^2 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_v = \frac{\frac{\pi}{11} \cdot \frac{D}{m} + L}{t_2 - t_1} \ln \frac{H_1}{H_2}$ for $\begin{cases} k_v' = k_v \\ d = D \end{cases}$	$k_v' = \frac{d^2 \cdot \left(\frac{\pi}{11} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right)}{D^2 \cdot T}$ $k_v' = \frac{\frac{\pi}{11} \cdot \frac{D}{m} + L}{T}$ for $\begin{cases} k_v' = k_v \\ d = D \end{cases}$	
F	$k_h = \frac{q \cdot \ln \left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot H_c}$	$k_h = \frac{d^2 \cdot \ln \left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_h = \frac{d^2 \cdot \ln \left(\frac{4mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ for $\frac{2mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln \left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln \left(\frac{4mL}{D}\right)}{8 \cdot L \cdot T}$ for $\frac{2mL}{D} > 4$	
G	$k_h = \frac{q \cdot \ln \left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot H_c}$	$k_h = \frac{d^2 \cdot \ln \left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_h = \frac{d^2 \cdot \ln \left(\frac{2mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ for $\frac{mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln \left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln \left(\frac{2mL}{D}\right)}{8 \cdot L \cdot T}$ for $\frac{mL}{D} > 4$	Determination basic time lag T

ASSUMPTIONS

Soil at intake, infinite depth, and directional isotropy (k_v and k_h constant). No disturbance, segregation, swelling, or consolidation of soil. No sedimentation or leakage. No air or gas in soil, well point, or pipe. Hydraulic losses in pipes, well point, or filter negligible.

results used to refine the system as work proceeds.

E. Potential Errors in In-Situ Permeability Measurement

The sources of error in a typical in-situ permeability test include:

1) Inaccurate water quantity measurement
2) Inaccurate head measurement
3) Inaccurate test length
4) Inaccurate measurement of test section dimensions
5) Plugging or smearing of fractures or pores
6) Use of excessive pressures
7) Unknown flow resistance of measuring system

The effects of 1, 2, 3, and 6 may be mitigated by the use of careful procedures, but in general, the contributions of the other sources of error are difficult to isolate or interpret.

F. Conclusions

Most fine grained soils are relatively impervious, and fluid flow problems in these soils occur as the result of geologic discontinuities, such as fissures or sand lenses. A painstaking soils investigation is required to detect the presence and extent of these discontinuities. Coarser soils which exhibit a coefficient of permeability greater than about 10^{-5} cm/sec can be expected to cause problems in seepage control.

Conventional soil investigations, and many of the commonly used borehole permeability methods, can yield misleading permeability information. Consequently, large scale field observations are an essential supplement to any important permeability testing program.

If existing methods are applied with discretion and engineering judgment, the coefficient of permeability can often be predicted to within one order of magnitude, which is usually sufficient to enable a safe, economical design.

III. SHEAR STRENGTH

A. Introduction

Almost all geotechnical projects involve some consideration of soil shear strength. Soil strength is most commonly described as a peak shear strength in terms of the Mohr failure envelope parameters, friction angle ϕ, and cohesion c, but the residual, low strain and yield strength values may also be important in many projects. This discussion does not concern the many different ways of describing soil strength, other than to make a distinction between drained and undrained loading conditions. For clay soils, the peak undrained shear strength, s_u (half the unconfined compressive strength), is taken as the measure of strength. For non-clays, the angle of internal friction, ϕ', in terms of effective stresses is considered as the governing strength parameter.

Once an in-situ strength parameter has been obtained, it may be substituted in a limit equilibrium design analysis, used to classify the soil stratigraphically, employed as an index of another geotechnical property, or used as a quantitative basis for decisions regarding further testing.

In-Situ Strength Testing

The advantages of strength measurement in-situ are threefold: the large effects of sampling disturbance on soil strength can be minimized, the costs of undisturbed sampling and testing can be reduced, and in-situ measurement eliminates the need to reproduce complex chemical, biological, thermal and stress environments.

A disadvantage of in-situ testing is that the soil strength parameters must be measured "as is." Soil strength can be affected by the imposition of post-construction environmental conditions. Post-construction changes in the stress, thermal and chemical environments may often be simulated in laboratory testing. It is, however, not usually possible to measure the resulting "future" behavioral properties in-situ.

The most commonly used in-situ strength tests are listed in Table III-1, after Schmertmann (1975). The following sections describe the use of these tests in engineering practice.

TABLE III-1

Commonly Used In-Situ Strength Tests
(after Schmertmann, 1975)

Test	Abbreviation
Standard Penetration	SPT
Quasi-Static Cone Penetration	Q-CPT
Vane Shear	VST
Pressuremeter	PMT
Borehole Shear	BST

B. The Standard Penetration Test (SPT)

A simple method of obtaining at least some information concerning the degree of compactness of soil in-situ consists of counting the number of blows of a weight dropping a given distance required to drive a sampling spoon for a distance of one foot (30 cm). The resulting blow count, N, is at the present time probably the most widely used index of subsurface soil conditions in the U.S.A.

Although the details and advantages of the SPT, as it is defined in ASTM Standard 1586, are widely known, the effects of different factors on blow count are a subject of continuing study.

Advantages of the SPT include economy of use, simplicity of procedure and widespread acceptance and familiarity among practicing civil engineers. Disadvantages of the SPT include the uncertain effects of a large number of influencing factors, and the poor reproducibility and large variability of test results.

Estimating Strength of Sand from the SPT

A correlation between blow count and ϕ' for sands has been established by DeMello (1971) and is reproduced as Figure III-1. Use of this chart yields conservative values of ϕ', which are not applicable to shallow soil depths.

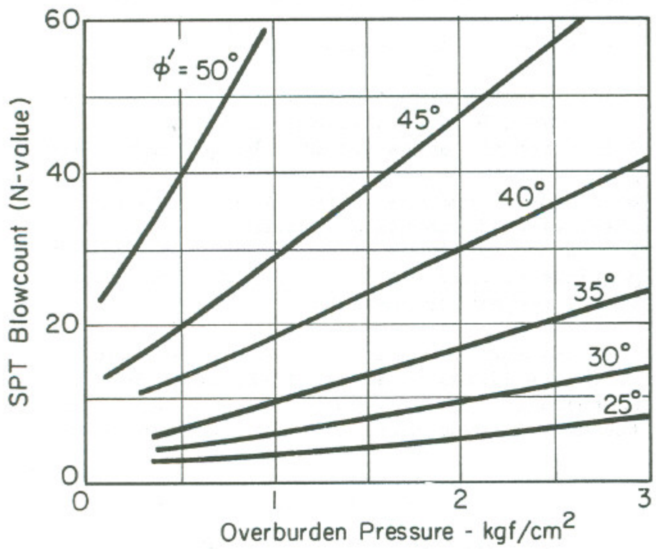


Fig. III-1 Method for estimating effective friction angle (ϕ') from SPT blowcount (N) (based on DeMello's 1971 analysis, USBR data)

Another common practice is to estimate ϕ', using relative density, D_r, as an intermediate parameter. A correlation of relative density, Gibbs and Holtz (1957), is often employed, although at high relative densities the relation-although at high relative densities the relationship suggested by Bazaraa (1967) may be more appropriate. The original Gibbs and Holtz relation is reproduced in Figure III-2. In Figure III-3, the Bazaraa, Gibbs and Holtz, and a third relationship suggested by Schultze and Melzer (1965) are compared. An inspection of this composite chart shows that an estimate of D_r from the SPT may easily involve a large uncertainty, even if the overburden pressure and "true" blow count are known with certainty.

Once the relative density estimate and split spoon sample identification have been obtained, a correlation such as that recommended for quartz sands by Schmertmann (1975) may be used to obtain ϕ'. This correlation is reproduced in Figure III-4. Because estimating D_r by the SPT can easily involve an error of ±20%, the error in ϕ' may be as large as ±5° (Schmertmann, 1975). Analyses, such as bearing capacity calculations, which are sensitive to variations in ϕ', must therefore include large safety factors if this approach is used.

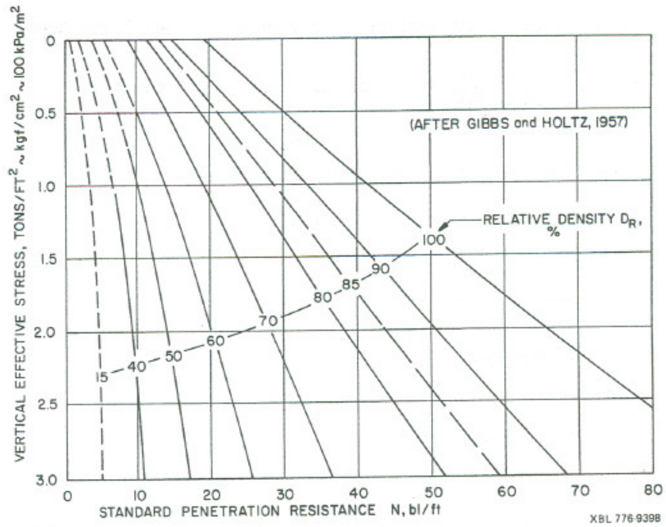


Fig. III-2 Correlation between relative density and standard penetration resistance according to Gibbs and Holtz (1957)

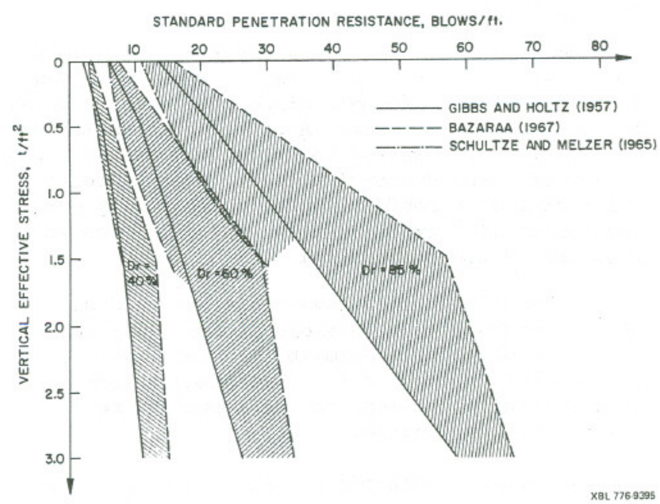


Fig. III-3 Three correlations between relative density and standard penetration resistance

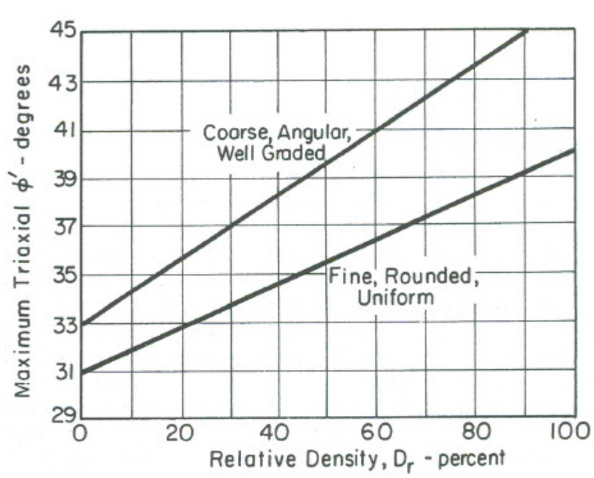


Fig. III-4 Approximate correlation between effective friction angle (ϕ') and relative density (D_r) in quartz sands (from Schmertmann, 1975)

Estimating Strength of Clay from the SPT

SPT results are occasionally employed to estimate the undrained strength of clays. A number of published correlations between s_u and blow count for insensitive clays are presented in Figure III-5. There is a wide degree of scatter in the correlations, and corrections for overburden pressure and overconsolidation ratio are not available. Clay sensitivity may decrease the blow count for a given undisturbed strength because of strength loss during penetration, in the manner depicted in Figure III-6.

One "rule of thumb" is that s_u, in tsf., is at least as great as N/15 (Schmertmann, 1975).

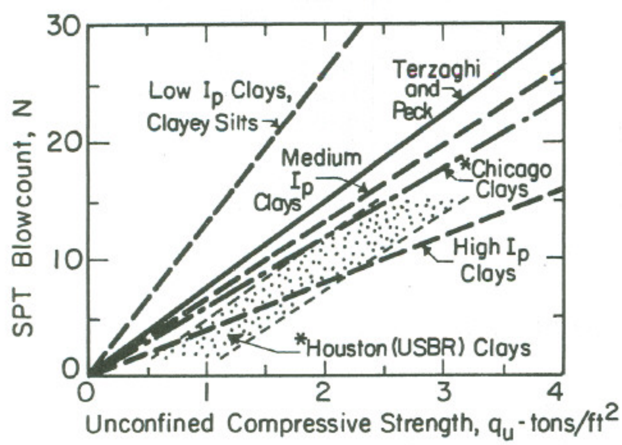


Fig. III-5 Some correlations between SPT blow count (N) and unconfined compressive strength (q_u) in clays (from DM-7 unless noted *) (from Schmertmann, 1975)

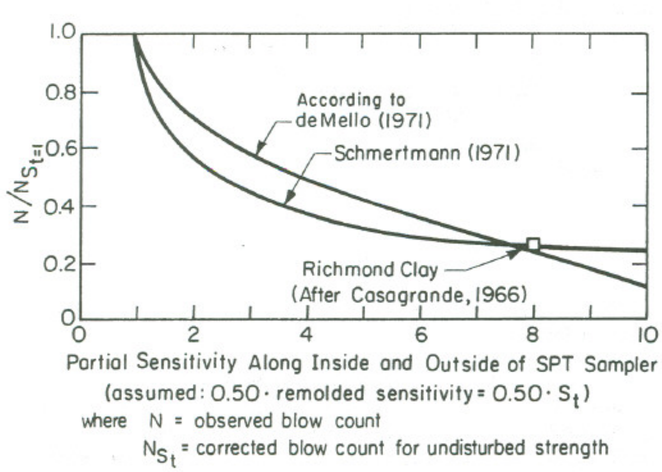


Fig. III-6 Estimated decrease in standard blow count with increasing clay sensitivity at constant undrained strength (from Schmertmann, 1975)

Factors Which Influence SPT Blow Count

The methods used for the prediction of ϕ' from the SPT allow variously for the effects of soil type, overburden pressure and relative density. SPT results are, in addition, affected to a significant degree by a number of other influencing factors:

1) State of Stress. Blow count is affected not simply by overburden pressure, but by the entire in-situ stress state, as well as stress history. The in-situ horizontal effective stress, σ_h' may actually have twice the proportional effect of vertical stress on blow count (Schmertmann, 1975). The difficulties of estimating horizontal stresses are discussed in Section IV of this report. At present, the possible influence of σ_h' on measured N values may be considered in only a very qualitative sense.

2) Pore Pressures. The generation and dissipation of pore pressure during the SPT may have a major influence on the resulting blow count. Soils, such as fine, loose sands, which generate significant positive pore pressures during driving, will yield decreased values of N, as a result of the decreased effective stress levels during driving. Dilatant soils will generate negative pore pressures during driving and increase the recorded blow count.

3) Sampler Side Friction. There is evidence that side friction between soil and the sampling spoon may contribute a significant fraction of the measured penetration resistance. The results obtained by Schmertmann (1975), reproduced in Table III-2 show that the contribution of soil-sampler side friction to penetration resistance increases dramatically with increasing soil friction ratio, FR. The friction ratio, in turn, will generally increase with increasing soil cohesion. For the insensitive clay in Table III-2, 86% of the measured penetration resistance originated from side friction.

Table III-2 CPT-Based N-Resistance Distributions (Schmertmann, 1975)

FR	Typical Soil Types	Part of Blow Count Due to	
		End Bearing	Side Friction
1%	Sand, above and below groundwater	56%	44%
2½%	Silty marl, silt, clayey sand	34%	66%
4%	Sandy clay, clay with S_t - 4	25%	75%
8%	Insensitive clay, peaty soils	14%	86%

FR = Friction ratio = unit side friction ÷ unit end bearing stress

St = Sensitivity

TABLE III-3. MEASURED INCREASES IN "N" WITH IMPEDANCE OF HAMMER FREE FALL (Schmertmann, 1975)

Investigators	Soil	Depth	GWT	N-range	N/N_{FF}	Notes
Frydman (70)	Natural	?	?	2 - 100+	1.4	In Israel, 2 turns sliprope over 10-12" cathead
Zolkov, (71, 72)	Dune sands SP, SP-SM	1-12m	above	7 - 60	1.8 @2m 1.5 @11m	do w = 1-6%
Serota & Lowther (73)	Dry sand compacted in a drum	At surcharged surface	Dry	10 - 20	1.06	In England, $D_r = 95\%$, 1 rope turn on cathead
					1.21	do., 2 rope turns
					1.4	do. but cathead hammer system weighed less

GWT = ground water table

TABLE III-4. SOME FINDINGS FROM WAVE EQUATION STUDIES OF SPT (from Schmertmann, 1975)

Variable	Relative Importance to N	Comments
1. Energy input = E thru hammer system	Very important	$N \sim 1/E$
2. Total resistance on sampler = R	Very important	$N \simeq 8R$ (kips) if $E = 70\% \cdot (350$ ft.lb) for $N \geq 15$
3. Distribution of R along sampler	Important	N increases c. 40% going from 100 → 0% end bearing
4. Rods: length	Minor	% change important at very low N
type (A,N)	Minor	N_N slightly larger than N_A
loose joints	Minor	
buckling	Minor	Maybe Impt. in 3D

4) Testing Technique. One of the most significant sources of uncertainty in the SPT relates to above ground apparatus and test conditions. A full discussion of these considerations may be found in DeMello (1971), Fletcher (1965) and Sanglerat (1972).

The usual cathead-slackened rope procedure of releasing the drive weight may result in a significantly larger blow count than that obtained if a "free fall" weight release mechanism is used. A summary of data on this effect is presented in Table III-3, from Schmertmann (1975). The unanswered question is: "How much drop friction is built into existing correlations between N-values and soil properties?"

Application of the one-dimensional wave equation to the SPT by McLean et al. (1975), has provided some important information concerning the relative importance of different details in SPT technique. Some findings of this study have been summarized by Schmertmann (1975), and are presented in Table III-4.

Conclusions

The SPT is, and will remain, an important geotechnical tool. The precision of the test is admittedly low, but many practicing engineers are able to apply judgment and local experience to derive satisfactory designs on the basis of personal or published correlations. The SPT is most properly applied to the "day to day" design projects which do not justify more sophisticated testing and in areas where the soil conditions are reasonably well known. Standards for equipment and procedure would make the SPT results more quantitatively meaningful. The general effects of many influencing factors in the SPT are becoming better understood; it is important that they be considered in the interpretation and application of test results.

C. <u>Cone Penetration Tests</u>

Many types of cone-tipped penetration devices are used in Europe and are receiving increasing acceptance in the United States because of the simplicity of testing, reproducibility of results and the greater amenability of the test data to rational analysis. The general types of cone penetration devices are summarized in Table III-5.

The quasi-static, Q-CPT or "Dutch Cone" method is the most commonly used in engineering practice. Dynamic cone penetration tests are subject to the same disadvantages as the SPT with an additional drawback of not providing simultaneous sampling. A combination dynamic-quasi-static system facilitates penetration through stiff layers which resist the static cone.

<u>Methods for Estimating Strength</u>

In the widely used Dutch cone test, a penetrometer of 10 cm^2 base area and 60° apex angle is advanced vertically at a constant rate (2 cm/sec) into the soil to be tested. The friction jacket advances simultaneously (electrical cones) or alternately (mechanical cones) with the tip. The resulting tip resistance, q_c, and side friction, f_s, are measured separately.

A conservative method has been developed de Beer for determining ϕ' from q_c on the bas: bearing capacity theory (Sanglerat, 1972, ESOI 1974). A less conservative, semi-empirical cc lation between ϕ' and q_c has been used in the and is presented in Figure III-7, after ESOPT

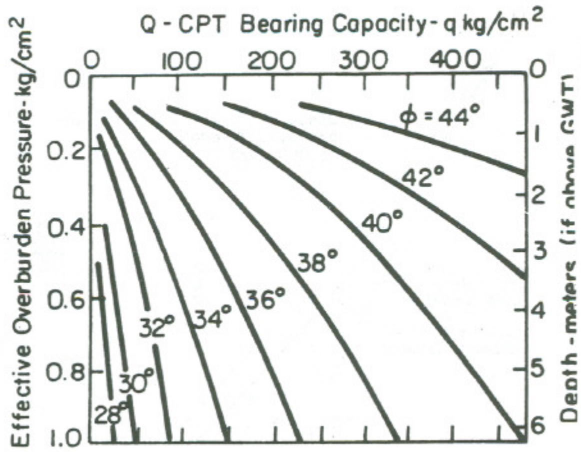


Fig. III-7 Method for estimating effective an of friction (ϕ') from static cone bearing resistance (q_c) reported i in USSR (ESOPT, 1974, p. 151)

Large scale model footing tests on sand (Muhs Weiss, 1971) have shown that:

$$q_c \left(\frac{kg}{cm^2}\right) = 0.80\ N_\gamma \qquad \text{(I:}$$

where N_γ is Terzaghi's bearing capacity factor dimensionless function of ϕ'. This correlatioı applies most directly to the bearing capacity

TABLE III-5. GENERAL TYPES OF CONE PENETRATION TESTS (from Schmertmann, 1975)

Type	Tip Advance: Method	Tip Advance: Rate	Where Used	Notes
1. Static	During increments of constant load	0	Research	Too slow for general field use
2. Quasi-static	Hydraulic or mechanical jacking	1-2 cm/sec	World-wide	Usually 10 cm^2 60° cone point
3. Dynamic	Impact of drive weight	variable	world-wide	Great variety of sizes, weights, etc.
4. Quasi-static & dynamic	Combines 2. & 3. using dynamic when Q-CPT cannot penetrate further		France Switzerland	Uses special penetrometer tips
5. Screw	Rotation of a weighted, helical cone	variable	Sweden Norway	
6. Inertial	Dropped or propelled into soil/rock surface	variable during measured deaccelera-tion	Offshore, Military	Useful for soils in inaccessible areas

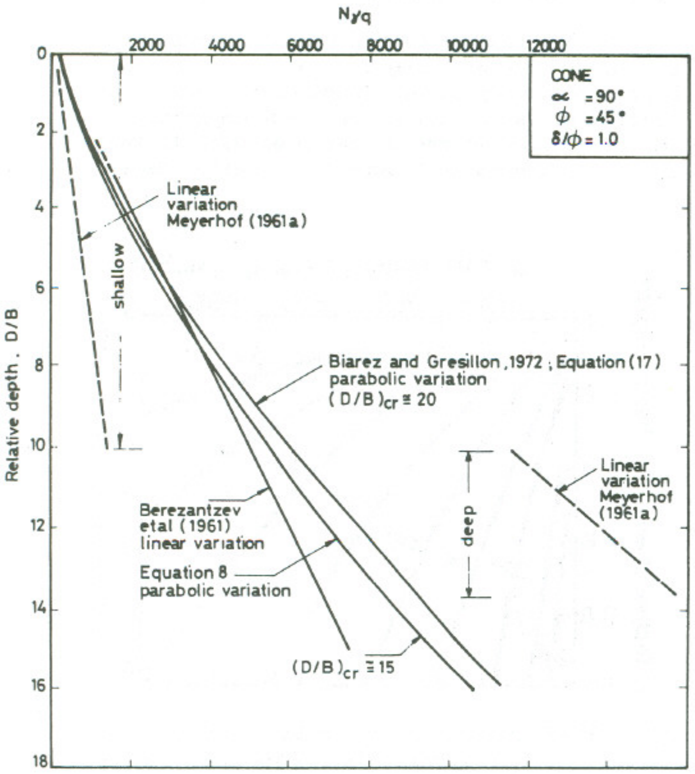


Fig. III-8 Effect of depth on resistance factor $N_{\gamma q}$ according to different theories (Durgunoglu-Mitchell, 1975)

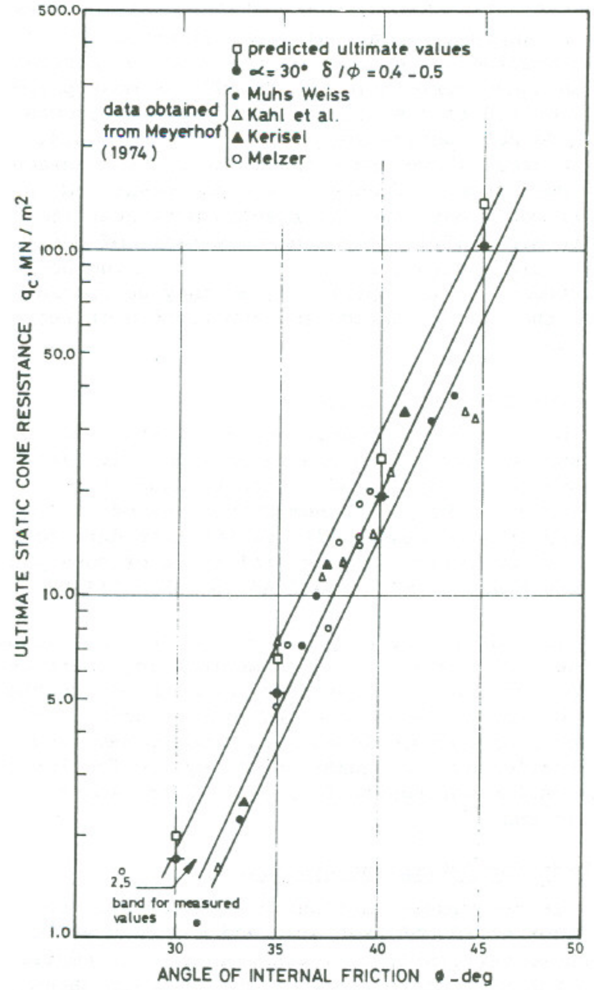


Fig. III-9 Ultimate cone resistance as a function of friction angle for several sands (from Durgunoglu and Mitchell, 1975)

putations for shallow footings. In reality, q_c is not uniquely correlated with ϕ'. The effects of other factors, including soil compressibility, stress state, and penetrometer base roughness have been recognized by Schmertmann (1975) and Durgunoglu and Mitchell (1975).

A procedure has been developed for estimating ϕ' from q_c on the basis of a rigid-plastic wedge displacement bearing capacity theory with empirical corrections for a circular cone shape by Durgunoglu and Mitchell (1975):

$$q_c = c\, N_c\, \xi_c + \gamma_s\, B\, N_{\gamma q}\, \xi_{\gamma q} \qquad \text{(III-2)}$$

For cohesionless materials, c = 0; and

$$N_{\gamma q} = F\,(\phi',\ \alpha,\ \delta/\phi',\ D/B)$$

where:

α = penetrometer base semi-apex angle

B = width of penetrometer base

D = depth of penetrometer base

$\xi_c, \xi_{\gamma q}$ = shape factors

δ = friction angle between penetrometer base and soil

γ_s = unit weight of soil

Bearing capacity factors, $N_{\gamma q}$, for the case of a Dutch cone and ϕ'= 45°, are presented in Figure III-8 as a function of soil depth, together with $N_{\gamma q}$ as calculated using other methods proposed by Berezantzev (1961), Biarez and Gresillon (1972) and Meyerhof (1961). Using equation (III-2), and the concept of a critical depth (Durgunoglu and Mitchell, 1975), ϕ' has been correlated with q_c as presented in Figure III-9. The data points show the $q_c - \phi'$ correlation presented by Meyerhof (1974) for actual field cases.

Sand profiles which possess a constant ϕ' and exhibit a linear variation of q_c with depth may be analyzed using a method suggested by Janbu and Senneset (1973):

$$q_c + a = N_q\,(p' + a) \qquad \text{(III-3)}$$

where:

a = penetration resistance intercept parameter, as indicated in Figure III-10

p' = effective overburden pressure

N_p = slope of $q_c - p'$ profile

N_q = F (tan ϕ') = N_p + 1, as indicated in Figure III-11

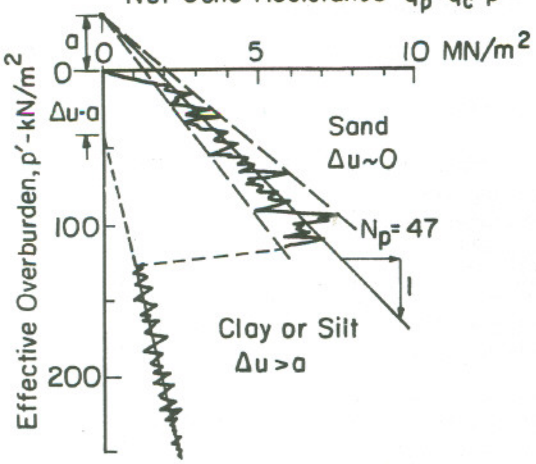


Fig. III-10 Principle of static cone interpretation, Janbu and Senneset (1973) method

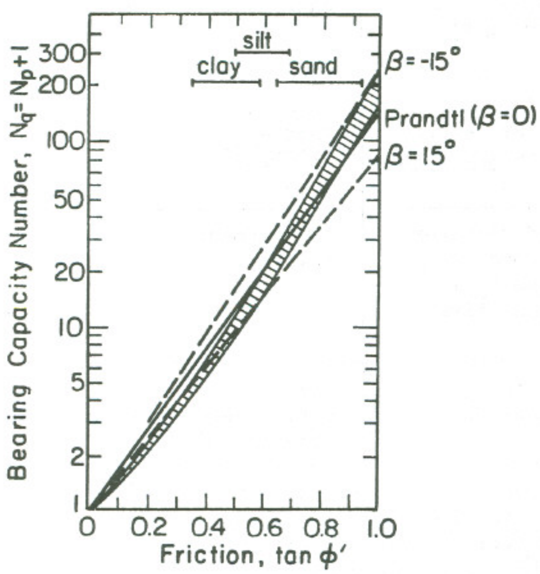


Fig. III-11 Values of N_q versus tan ϕ (after Janbu & Senneset, 1973)

An example of the use of this method is presented in Figure III-10. In practice, the investigator estimates the slope, N_p, and intercept, a, of the average linear q_c - p' profile. The relationship of $N_q = N_p + 1$ with ϕ', as presented in Figure III-11, is then used to estimate ϕ'. This method has the advantage of including overburden effects, and appears to provide reasonable results for appropriate profile conditions.

As with the SPT, relative density, D_r, may be employed as an intermediate parameter for estimating ϕ' from penetrometer data. A correlation of q_c, D_r, and overburden pressure for normally consolidated, uncemented, primarily quartz, saturated fine sands is presented in Figure III-12, after Schmertmann (1976)*. Use of this relationship

*The correlation applies when using the Fugro-type electrical cone, 10 cm², 60°, cylindrical tip, advanced continuously at 2 cm/sec.

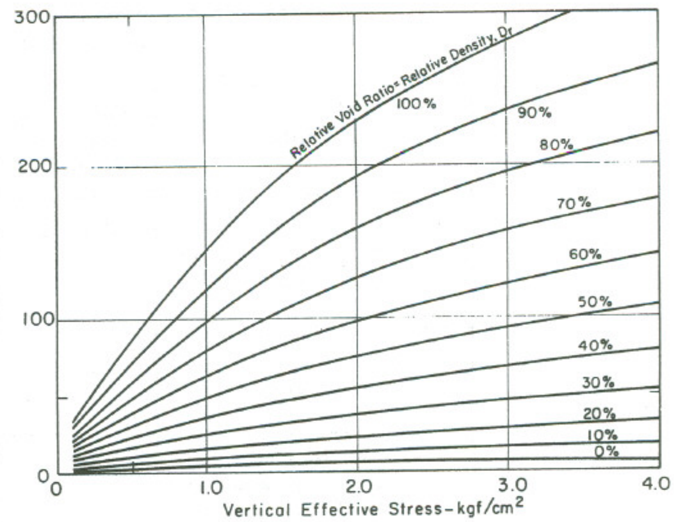


Fig. III-12 Q-CPT bearing capacity to estimate relative density in normally consolidated, silty fine to uniform medium sands (after Schmertmann, 1976)

requires a correction for overconsolidated sands, which Schmertmann (1974a), based on the results of chamber tests, suggested be taken as:

$$\frac{q_c}{(q_c)_{NC}} = 1 + 0.75\left(\frac{K_o'}{(K_o)_{NC}} - 1\right) \qquad \text{(III-4)}$$

where:

q_c = measured penetrometer tip resistance

$(q_c)_{NC}$ = penetrometer tip resistance for the normally consolidated case

K_o' = in-situ lateral stress coefficient

$(K_o')_{NC}$ = lateral stress coefficient for the normally consolidated case

Before applying the relationship presented in Figure III-12, the investigator must estimate $K_o'/(K_o')_{NC}$ and then employ Equation III-4 in order to convert the measured tip resistance, q_c, to the equivalent "normally consolidated" tip resistance, $(q_c)_{NC}$. If the overconsolidation ratio (OCR) is known, then $K_o'/(K_o')_{NC}$ can be estimated using $K_o'/(K_o')_{NC} = (OCR)^{0.42}$ and $(K_o)_{NC} = 1 - \sin\phi'$.

Strength of Clays from the CPT

The shear strength of clays, s_u, is most often estimated from penetrometer resistance by employing a relationship of the form:

$$s_u = \frac{q_c - p}{N_c} \qquad \text{(III-5)}$$

where:

s_u = undrained shear strength

p = overburden pressure

N_c = Bearing Capacity Factor appropriate for deep, circular foundations

TABLE III-6. SOME OF THE VARIABLES THAT INFLUENCE N_c IN EQUATION (III-5) (Schmertmann, 1975)

Variable	Approx. N_c factor potential	Direction	Notes
1. Changing the test method for obtaining reference s_u	2 to 3	Better sampling, thinner vanes, use of s_{uPMT} all decrease N_c	
2. Clay stiffness ratio = G/s_u	3	Increases with increasing stiffness	Vesic (1972)
3. Ratio increasing/decreasing modulus (E^+/E^-) at peak s_u	3	Decreases with decreasing ratio	Ladanyi (1967)
4. Effective friction, tan ϕ'	2 to 3	Increases with increasing ϕ'	Janbu (1974)
5. K_o', or OCR	3	Increases with increasing K_o' or OCR	Janbu (1974)
6. Shape of penetrometer tip	2	Clay adhesion on mantle of mechanical tips increases N_c	Example in Amar et al., (1975, Fig. 2)
	1.5	Reduced diameter above cone can decrease N_c in very sensitive clays	Schmertmann (1972)
7. Rate of penetration	1.2	Increasing rate increases N_c	Viscous, no pore pressure effects
8. Method of penetration	1.2	Continuous (electrical tips) penetration decreases N_c compared to incremental (mechanical tips) because of higher pore pressures.	

The bearing capacity factor, N_c, is, in fact, not a simple constant, but is affected by a number of factors as summarized in Table III-6, after Schmertmann (1975). Values of N_c ranging from 5 to 70 have been back-calculated by different investigators from measured values of q_c and values of s_u determined from other types of strength tests. Some values of N_c appropriate for different clay types are presented in Table III-7, after Brand et al. (1974). The best approach for design purposes is to experimentally determine N_c for a given clay type, penetrometer apparatus, testing procedure and reference s_u. If a friction-cone tip is employed, the measured soil-steel friction, f_s, is suggested by Schmertmann (1975) as a lower bound for s_u.

D. The Vane Shear Test

Introduction. In contrast with methods which derive strength parameters from intermediate variables, such as penetration resistance, the Vane Shear Test, VST, attempts to measure undrained shear strength directly. The test procedure is to advance a vane configuration to a desired soil depth and measure the applied torque as the vane is rotated at a constant rate. Shearing resistance is considered to be mobilized on a cylindrical failure surface of rotation, corresponding to the top, bottom and sides of the vane assembly.

Three common methods for installing the vane apparatus are illustrated in Figure III-13. As indicated in the figure, the vane may be installed at the bottom of a predrilled borehole, or pushed into the ground by means of an extension

TABLE III-7. CONE FACTORS DETERMINED FOR CLAYS (after Brand et al., 1974)

Reference	Clay	Cone Factor	Clay Properties				
			w, %	w_L, %	I_p, %	s_u, ton/m^2	Sensitivity
Thomas (1965)	London Clay	18	20-30	80-85	55	5-29$^+$	-
Ward et al. (1965)	London Clay	15.5	22-26	60-71	36-43	21-52$^+$	-
Meigh & Corbett (1969)	Arabian Gulf Soft Clay	16	30-47	38-62	20-35	0.5-4*	5
Ladanyi & Eden (1969)	Leda Clay (Gloucester)	7.5	50-70	50	23	2.5*	30-50
Ladanyi & Eden (1969)	Leda Clay (Ottawa)	5.5	72-84	40	20	5.7*	10-35
Pham (1972)	Soft Bangkok Clay (City)	16	60-70	70-80	40-50	1.3-2.9*	5-7
Anagnostopoulos (1974)	Patras Clay	17	30	35	18	3-7$^+$	1.5-3
Brand et. al. (1974)	Soft Bangkok Clay (Bangpli)	19	60-130	60-130	60-120	1.3-3.8*	5-7
Brand et. al. (1974)	Weathered Bangkok Clay (Bangpli)	14	100-130	100-135	60-80	1.3-3.2*	6-8

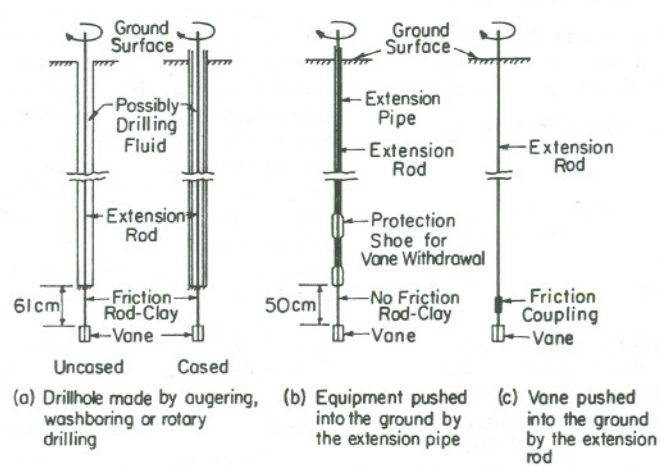


Fig. III-13 Methods for installing shear vanes

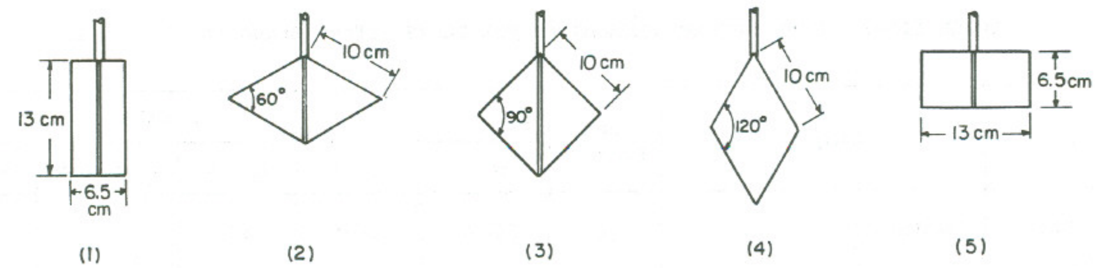


Fig. III-14 Some shear vane configurations in common use

rod, with or without the use of a protection shoe. A number of different vane configurations are in current use, as depicted in Figure III-14, but the preferred design, as specified in ASTM Standard D2573, is a four bladed vane with a height/diameter ratio of 2.

Determination of Strength from the VST

Undrained shear strength may be calculated from measured torque in the VST, provided that horizontal and vertical shear strengths are assumed equal, by employing the following equation:

$$s_{uv} = \frac{2T}{\pi D^3 \ (H/D + a/2)} \qquad \text{(III-6a)}$$

where:

s_{uv} = the undrained shear strength, from the VST

T = maximum applied torque

H = vane height

D = vane diameter

a = factor which depends on the assumed shear distribution along the top and bottom of the failure cylinder

= 2/3 if uniform shear is assumed

= 1/2 if triangular distribution is assumed (i.e., shear strength mobilized is proportional to strain)

= 3/5 if parabolic distribution is assumed

For a uniform shear strength distribution, and ASTM D2573 vane configuration, Equation III-6a reduces to:

$$s_{uv} = \frac{6}{7} \ \frac{T}{D^3} \qquad \text{(III-6b)}$$

Under these conditions, shearing resistance on the side (vertical) face of the failure cylinder contributes some 85% of the measured resistance to rotation, T. What is actually measured in the VST is therefore a weighted average of the shear strengths s_v on vertical and s_h on horizontal planes. It is possible to determine both s_h and s_v separately by repeating a test using a vane of a different shape or H/D ratio. This method was employed by Richardson et al. (1975) in order to determine the relative values of s_h and s_v in Bangpli Clay. The results of their investigation are reproduced in Figure III-15. The ratio s_h/s_v is seen to be fairly constant with depth, with an average value of 0.6.

Measurements of s_{uv} using different vane configurations will be influenced by different proportions of s_h and s_v, as noted in Figure III-14. It has been found that, in general, the ratio s_h/s_v is less than unity (Duncan and Seed, 1966; and Lemasson, 1974). It therefore seems that an accurately determined value of s_{uv} may be used as a conservative estimate of s_v.

Accuracy of the VST

Until recently, the VST was considered to be a most reliable means of measuring undrained shear strength in soft to medium clays, possessing the advantages of economy of use and reduced soil disturbance, compared to sampling and laboratory testing. A number of cases have been encountered, however, in which the use of s_{uv} leads to unconservative results in undrained stability analyses (Bjerrum, 1972; Pilot, 1972). Accordingly, a correction procedure was developed (Bjerrum, 1973) which attributes the discrepancy in field behavior mainly to strain rate effects. The true undrained strength is related to the measured shear strength as follows:

$$s_{u\ (field)} = \mu \ s_{uv} \qquad \text{(III-8)}$$

The Bjerrum correction factor, μ, is correlated with soil plasticity index, I_p, as shown in Figure III-16. An inspection of Figure III-16 shows considerable scatter in the data obtained subsequent to development of the correlation. This correction procedure is subject to the additional uncertainty inherent in the determination of I_p and s_{uv}, and in the formulation of stability analysis assumptions. Use of the Bjerrum correction factor may actually yield occasional unconservative results as noted by LaRochelle et al. (1974) and Ladd (1973). Schmertmann (1975) notes also that as I_p is determined using disturbed clay it cannot account for differences among clays having differing undisturbed structures.

Factors Which Affect the VST

It is now recognized that the VST is subject to the uncertain effects of a large number of influencing factors:

1) Disturbance. It is evident that the vane cannot be installed in the ground without causing some degree of disturbance in the soil around it. If predrilling is employed (see Figure III-13a), a zone of a certain width around the bottom of the borehole must be considered to be disturbed. In practice, it is commonly assumed that the disturbed zone does not extend more than 60 cm from the bot-

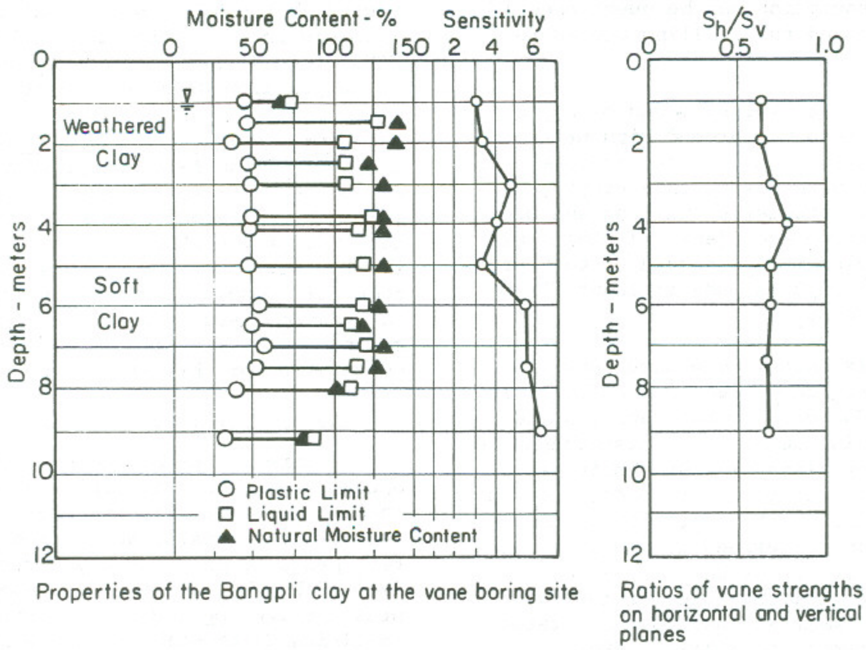


Fig. III-15 Ratio of strengths on horizontal (s_h) to vertical (s_v) planes for Bangpli clay (from Richardson et al., 1975)

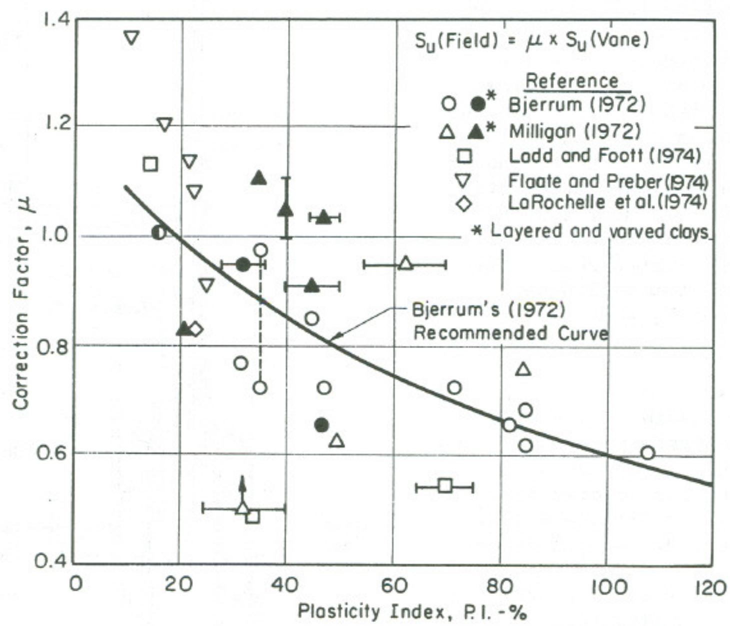


Fig. III-16 Field vane correction factor vs plasticity index derived from embankment failures (from Ladd, 1975) tom of the borehole. It has been noted (Flaate, 1966), that this assumption can be questioned if the borehole is wide and the drilling procedure unfavorable.

If the vane is installed by means of a protective shoe forced into the ground without predrilling (see Figure III-13b), a plastic zone will be formed around the vane head with a varying degree of disturbance. Studies by Cadling and Odenstad (1950) and Andresen and Bjerrum (1965) indicate no measurable disturbance caused by this procedure when the vane test is made at least 50 cm from the protective shoe.

When the vane is installed without any predrilling or protective shoe (see Figure III-13c), the disturbance is due only to the vane rod and the vane itself. This component of disturbance is also present if predrilling or a protective shoe is employed. In this regard, it is considered advantageous to minimize the "area ratio" of the vane. The area ratio is defined as the cross sectional area of the vane cross and stem as a percentage of the cross sectional area of the failure cylinder. It is recommended that the area ratio not be higher than about 15% (Flaate, 1966). Laboratory tests performed by Vey (1955) show that cohesive soils can stick to the vane surface, and, in effect, increase the area ratio of the apparatus. It has been suggested (Flaate, 1966), that this effect will probably depend upon the stickiness of the soil and be of less importance in sensitive clays.

It is recognized that the use of a damaged vane apparatus may greatly increase the magnitude of soil disturbance.

2) Mode of Failure. The stress distribution and stress strain behavior in the VST are little known. The soil is generally assumed to fail along the sides and ends of a circumscribed cylinder. It is also assumed that the shear strength is fully mobilized all along the surface at the same time, i.e., no progressive failure takes place. In the idealized case, the application of torque will produce large stress concentrations at the end of the vane blades (Flaate, 1966). The actual stress distribution will depend upon the degree of disturbance around the vane as well as the stress-strain properties of the undisturbed soil. The potential for progressive failure will depend upon the type of soil, its sensitivity and its stress-strain characteristics.

A laboratory program was carried out by LeBlanc (1975) in order to examine the actual failure pattern and stress-strain behavior during the VST. It was observed that the peak strength was obtained at very low angular deformations. Only for rotations in excess of 45° was a cylindrical failure surface observed. On the basis of these results, Roy (1975) concluded that it is questionable to interpret VST results in the classical manner which assumes a cylindrical shear failure surface at peak strength.

The failure mode in the VST may not correspond to the failure mode in a prototype situation. The effect of preferred failure plane orientations may govern full scale behavior, yet remain undetected in the VST. Because of the uncertainties involved in describing the failure mode in a given VST, it is desirable that VST results be used in conjunction with other strength tests.

3) Dimensions of Failure Cylinder. The actual diameter of the failure cylinder in a soft clay, and hence the moment arm of mobilized shear strength, has been observed to be some 5% larger than the diameter of the vane blades (Arman et al., 1975). An uncorrected discrepancy of this magnitude will result in an over-estimate of 16% in calculated shear strength. At present, there is not enough data available to enable a quantitative correction for this effect in different soil types.

E. The Pressuremeter Test (PMT)

A borehole pressuremeter, of the type developed by L. Menard, is depicted in Figure III-17. These devices are widely used in Europe and are receiving increasing acceptance in the U.S. The test proceeds once the apparatus is lowered to a desired borehole depth. Increments of hydraulic pressure are applied to the center cell, and the resulting deformation of the borehole wall, at set time intervals, is determined from fluid volume change measurements of the cell chamber. The two pressurized guard cells serve to stabilize the device within the borehole and to insure essentially axial plane strain conditions at mid-height

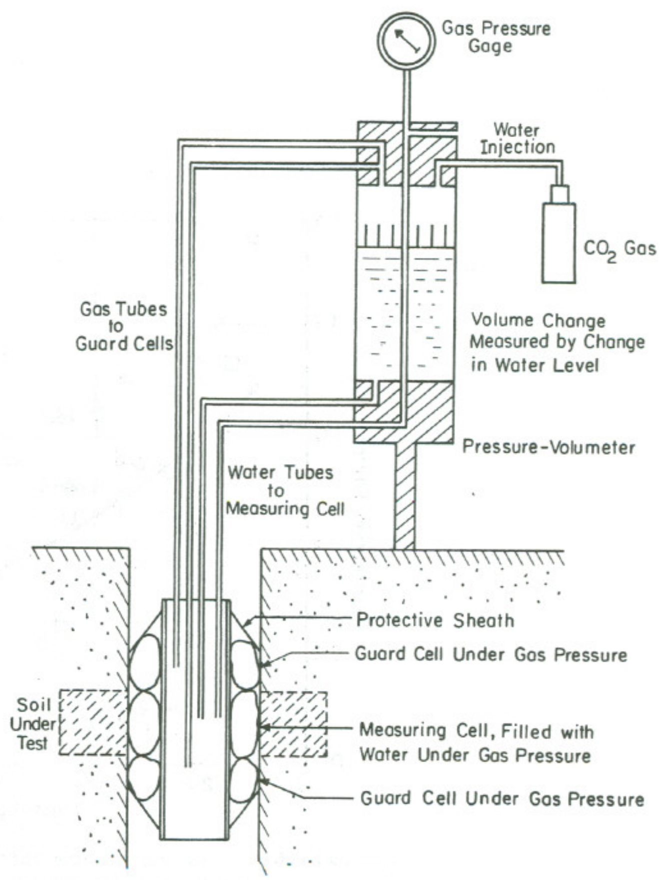


Fig. III-17 Classic Menard pressuremeter

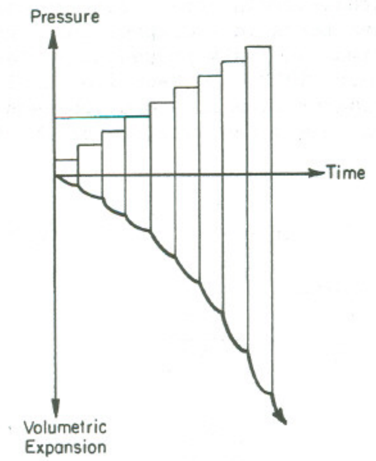


Fig. III-18 Pressure-volume change history during a pressuremeter test

of the apparatus. The relationships between pressure increments, volumetric expansion, and time are presented in Figure III-18 (Menard, 1975).

Important recent improvements of the PMT are the "Autoforeuse" and the "Camkometer" self boring pressuremeters, developed respectively by Baguelin et al. (1972) and Wroth and Hughes (1973). A self boring pressuremeter is schematically presented in Figure III-19a, and the Camkometer is depicted in Figure III-19b.

The PMT is seen to possess at least three distinct advantages:

1) The test models the axisymmetric expansion of an infinite cylindrical cavity -- a problem with well developed elastic and elasto-plastic solutions which are suited for application to soil mechanics.

2) The conduct of the test permits an estimate of the in-situ lateral stresses with no limits on K_o.

3) The test results provide not only strength data, but stress-strain soil properties applicable to the direction perpendicular to the axis of the borehole cavity.

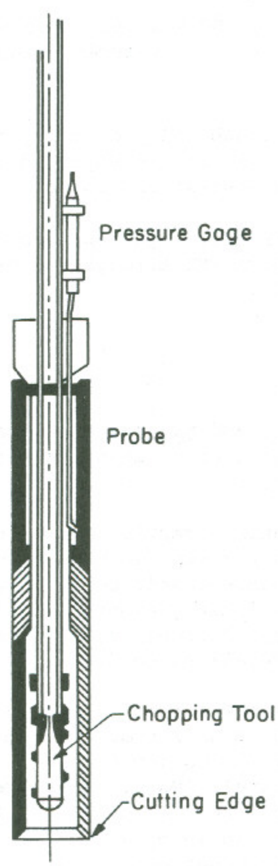


Fig. III-19a Sketch of "Autoforeuse" probe (from Baguelin et al., 1972)

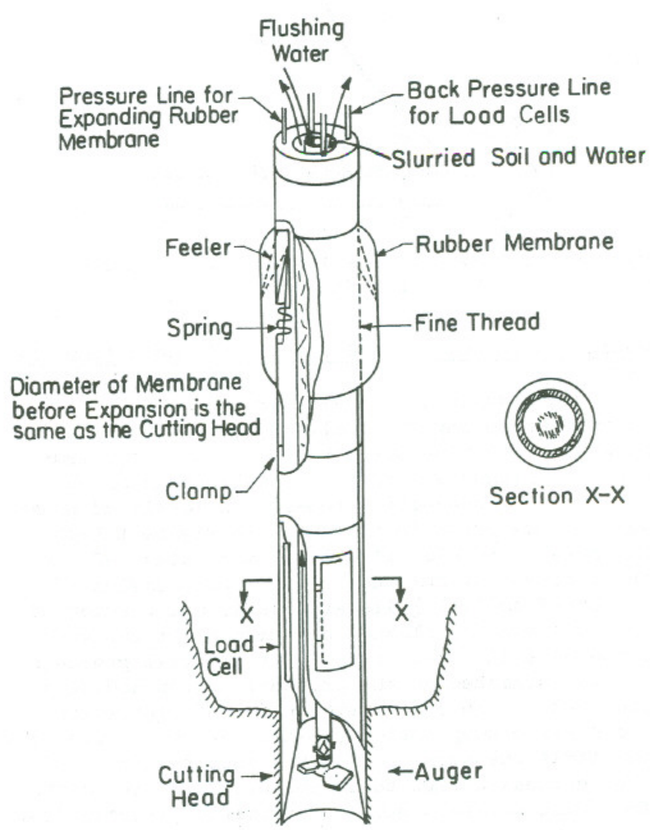


Fig. III-19b Main details of Cambridge in-situ instrument showing stress gages (from Wroth and Hughes, 1972)

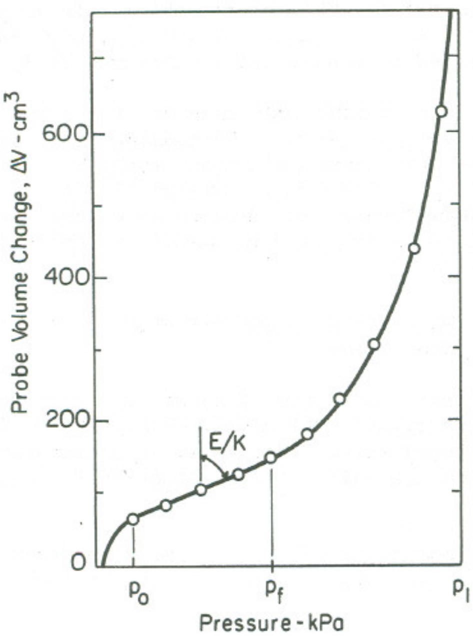


Fig. III-20 Theoretical pressuremeter curve

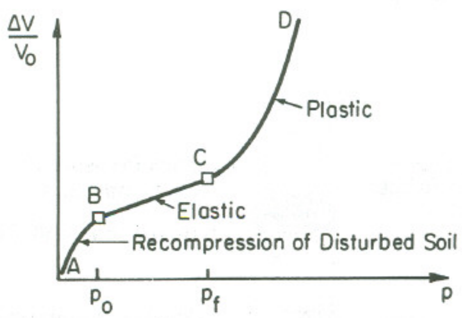


Fig. III-21 Typical result of a pressuremeter test in soft clay

Obtaining Strength and Stress-Strain Data from the PMT

PMT results are usually presented as a plot of chamber volume vs. applied pressure. Applied pressure should be corrected to account for membrane stiffness and the hydrostatic head of the fluid in the connecting tubes. An idealized pressure-volume curve is presented in Figure III-20. The results in Fig. III-21 for soft clay reflect three phases of the test: (1) recompression of disturbed soil, (2) linear elastic compression of the soil and (3) plastic deformation of the surrounding soil mass. The in-situ lateral pressure, p_o, is estimated as the pressure corresponding to the kink in the curve between the recompression and elastic compression phases. By definition, the the limit pressure, p_l, is the abscissa value of the vertical asymptote to the pressuremeter curve. The limit pressure may be determined directly from the curve, but more conventionally it is taken as the pressure corresponding to a volume increase ΔV equal to the initial volume of the borehole V_i (i.e., $p=p_l$ @ $\Delta V=V_i$) as volume increases for pressure increments at this point are normally relatively large.

Three independent theories for extracting a complete stress-strain curve from the results of pressuremeter tests in incompressible, nondilating soil were simultaneously presented by Ladanyi (1972), Palmer (1972) and Baguelin et al. (1972). Each investigator assumed $\phi = 0$, Poisson's ratio $\mu = 0.50$ and showed the validity of the following equations:

$$\tau_{ps} = \varepsilon_o \frac{d(p - p_o)}{d\ \varepsilon_o} (1 + \varepsilon_o)(1 + \varepsilon_o/2) \qquad \text{(III-9)}$$

For small strains:

$$\tau_{ps} \simeq \varepsilon_o \frac{d(p - p_o)}{d\ \varepsilon_o}$$

$$\varepsilon_1 = \frac{\Delta V}{\sqrt{3}\,(V_i + \Delta V)}$$

where:

ε_o = radial deformation of the probe, $\Delta a/a$, where a is the cell radius; computed from ΔV

τ_{ps} = equivalent plane-strain shear stress

ε_1 = equivalent axial compressive strain

p, p_o = corrected applied pressure and in-situ lateral pressure, respectively

V_i, ΔV = initial and differential chamber volumes, respectively

A graphical procedure developed by Amar et al. (1975), may be employed to deduce the complete stress-strain curve.

Shear strength is frequency determined from the Menard PMT through the semi-empirical relationship:

$$s_u = \frac{p_l - p_o}{N} \qquad \text{(III-10)}$$

where p_l and p_o are as indicated in Figure III-20, and N is a correlation factor generally taken to equal 5.5.

Until recently, ϕ' was determined from PMT results through "in-house" correlations of ϕ' with the pseudo-elastic modulus E_{PMT} (see section IV E) and the limit pressure p_l. The reliability of such correlations may be questionable when they are generalized to different soil types.

Cavity expansion theory has been applied to the determination of ϕ' from the PMT through the work of Gibson and Anderson (1961), Vesic (1972) and Ladanyi (1963). Using Vesic's theory, close agreement between ϕ'_{PMT} and ϕ' from triaxial tests has been shown by Winter and Rodrigues (1975), but investigations by Laier (1973) and Al-Awkati (cited by J. Schmertmann, 1975), showed a poor ability of the above theories to predict ϕ' from the results of pressurementer tests in large triaxial chambers.

The application of these theories to the determination of ϕ' is hindered by four factors:

1) Pressuremeters with ordinary length to diameter ratios require major correction factors to convert real PMT limit pressures to the limit pressure corresponding to infinite pressuremeter length (Laier et al., 1975).

2) At or near the limit pressure, the bulge in the pressuremeter cell seriously violates the plane strain assumption (Schmertmann, 1975).

3) Any evaluation of ϕ' requires a knowledge of in-situ stress conditions which may only be crudely determined in the conventional PMT (Laier, 1975; Hartmann and Schmertmann, 1975).

4) The initial soil modulus used in these theories may be underestimated when the conventional pressuremeter test is employed (Schmertmann, 1975).

A new method for determining ϕ' from pressuremeter tests in sand has been developed by Al-Awkati and reported by Schmertmann (1975). The method employs only the low strain portion of the PMT curve and thus avoids the uncertainty associated with evaluating p_1.

Al-Awkati's method proceeds as follows:

1) Plot PMT data as ln ($\varepsilon_o = \Delta a/a$) vs. ln (corrected pressure). A sample curve is presented in Figure III-22.

2) Measure θ as defined in Figure III-22.

3) Enter chart in Figure III-22 with tan θ and move vertically to measured or estimated ϕ_o' where ϕ_o' represents that part of ϕ' not due to dilatancy.

4) Read predicted triaxial ϕ' on horizontal scale.

The predicted triaxial ϕ' applies to the mean normal stress, σ_o', where:

$$\sigma_o' = \frac{p_f}{1 + \sin\phi'} \qquad \text{(III-11)}$$

and

p_f = chamber pressure when sand enters failure state.

One may determine ϕ_o' from a triaxial test in which the volumetric strain rate becomes zero at peak strength or by correcting for volumetric work in a test where this rate differs from zero. Alternately, Al-Awkati presents a correlation between ϕ_o' and the volumetric strain between 0 and 100% relative density for quartz sands which is reproduced in Figure III-23.

This method has been applied to corresponding PMT and laboratory triaxial test results, yielding a close agreement between predicted and measured values of ϕ', as summarized in Figure III-24 after Al-Awkati.

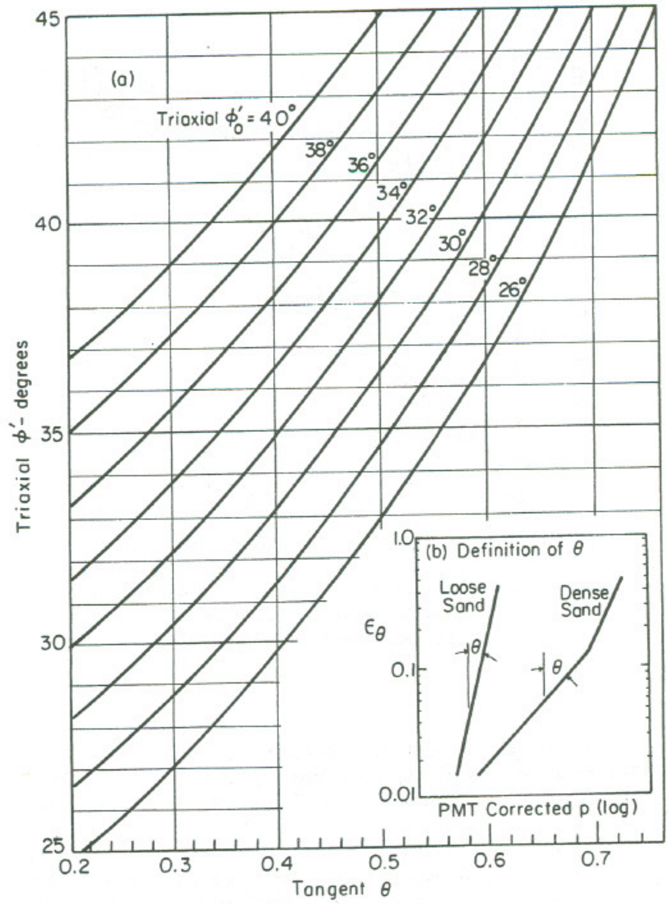


Fig. III-22 Al-Awkati's theoretical correlation between PMT data and triaxial maximum ϕ', using ϕ_o' (from Schmertmann, 1975)

Factors Which Affect the PMT

The standard Menard type PMT is highly sensitive to variations in test procedure.

A method of boring, and inserting the apparatus must be chosen which minimizes soil disturbance. Many methods will produce approximately the same

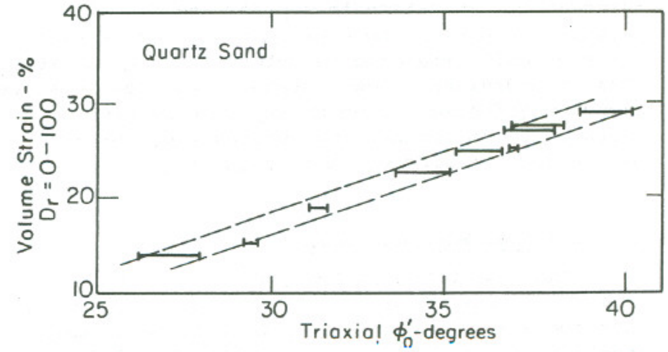


Fig. III-23 Correlation for estimating ϕ_o' using volume strain from relative density tests (suggested by Al-Awkati in Schmertmann, 1975)

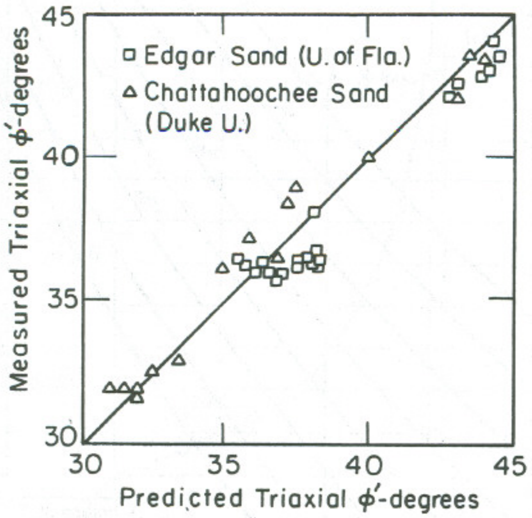


Fig. III-24 Al-Awkati's comparisons between measured and predicted ϕ' (Schmertmann, 1975)

limit pressure, but an accurate evaluation of soil deformation requires the best possible borehole preparation, especially in sensitive clay deposits.

During a standard PMT, the variation of pore pressure is not measured. As a result, the rate of expansion to assure essentially drained or undrained test conditions is not known. A common practice is to set an arbitrary time interval (1 - 2 minutes) for each pressure increment. Since the magnitude of soil volume change will increase as pressure increases in successive increments, a fixed time interval procedure may result in nearly drained conditions for the initial pressure increments, but essentially undrained conditions for the final pressure increments. It is likely that no part of the test will be fully drained or undrained.

This uncertainty in pore pressure behavior may be resolved by measuring pore pressure with a suitable transducer affixed to the face of the device. At present, very few pressuremeters are so equipped. Versions of the Cankometer, depicted in Figure III-19b, contain such a pore pressure transducer. An alternate approach to the pore pressure problem is to experiment with different rates of cell expansion to determine at what rates (fast = undrained, slow = drained) the test results become unaffected. Such an approach is generally impractical in engineering applications, because of the large number of tests required.

F. The Iowa Borehole Shear Test

The Iowa Borehole Shear Test is performed by lowering a shear head, consisting of two opposing horizontally grooved shear plates to an uncased section of a borehole (Figures III-25 and III-26). At the required test position the two shear plates are expanded until seated in the borehole walls at a preselected pressure. Some time is allowed for

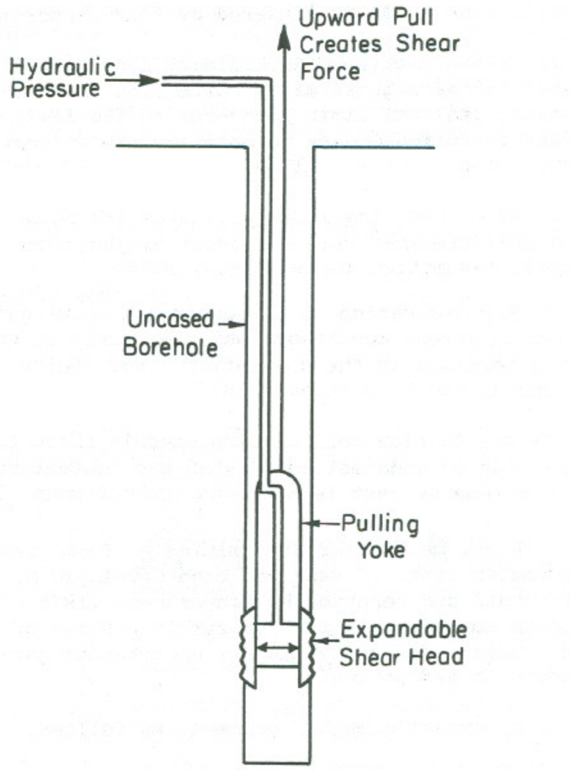


Fig. III-25 Iowa borehole shear test apparatus

consolidation to occur. When consolidation is complete the shear head is either pulled upwards, or pushed downwards at a steady rate of 2 mm/min.

The required forces for shearing are measured, and the shearing stress plotted against the normal pressure. At this point the shear plates may be contracted, the shear head lowered to its original position, rotated through 90° and the test repeated.

The shear head is then returned to the original position, another seating pressure selected and the test repeated.

By performing a number of tests at different seating pressures, a Mohr-Coulomb failure line and subsequently c and ϕ may be determined. In practice the number of tests that can be reliably repeated at the same position are influenced by the inherent friction of the soil. (In clays, the number may be as low as 2, whereas in sand the limit may be determined by how far the shear heads can be expanded.) c and ϕ values determined by this method show good correlation with those determined in triaxial and direct shear tests.

Major Limitations of the Method

i) Drainage conditions are not known, and, therefore, it is not possible to determine whether the strength parameters are those for drained, partially drained or undrained conditions.

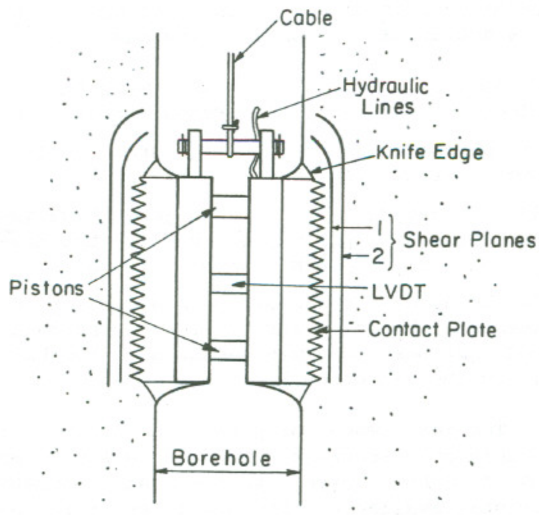


Fig. III-26. Schematic of borehole shear device (A) circumferential shearing resistance must be minimized for uniform distribution for contact pressure n_1, and (B) knife edges minimize and resistance

ii) The shear strength of unconsolidated portions of the borehole wall, not under pressure from the shear head plates but which extend along the ends of the plates, may add to the resisting shear force but is not accounted for in the calculations.

Advantages of Borehole Shear Test

i) The Mohr-Coulomb strength parameters may be determined in less than one hour for some soils.

ii) Erroneous shear values resulting from poor seating or remolding are usually apparent and may be systematically eliminated.

iii) Tests may be located at selected strata, as the tests are only performed after a hole is bored and logged.

G. Conclusions

Soil strength may be expressed in a number of ways, and the particular strength parameters to use in design will depend on the nature of a specific problem. In-situ strength testing methods and programs should be selected on this basis.

The Standard Penetration Test and Dynamic Cone Penetration Test suffer from a lack of theoretical understanding and controlled use, as well as uncertainties in interpretation. Use of these methods will continue, however, in the absence of economical replacements and because of the great number of available empirical correlations, especially for the SPT. In many cases SPT data are the only information available.

The quasi-static cone penetration test possesses many practical advantages and prospects for the development of a reasonable theoretical analysis of the test in the future are good. At present, best use of the method for strength evaluation requires the establishment of a cone factor for a particular soil deposit by means of a few separate strength tests by another method.

In spite of the several difficulties cited relative to the interpretation of vane shear test results, it can be anticipated that its use will continue for evaluation of the in-situ strength of soft clays.

The pressuremeter test has great potential for use in measuring soil strength, as well as stress-strain behavior. The potential accuracy of the self-boring PMT is promising, but the high costs of reliable results indicate that this approach may be most efficient if it is used in conjunction with simpler strength index measurements.

The borehole shear test is expected to increase in popularity as a result of its relative simplicity and the direct design value of the raw test data. Its use is limited to testing those soils which are suitable for stage testing. The applicability of stage testing to different soil types is a subject which needs to be studied further.

The different methods are not equally applicable in all soil types. The SPT and dynamic cone tests are most suitable for strength estimates in sands and stiffer soils. The static cone and vane shear devices cannot penetrate very stiff deposits. Pressuremeter testing, while suitable in principle for most soil types except gravels and rockfills, is not suitable in deposits containing interspersed cobbles or rock fragments. The borehole shear device has had its greatest applicability thus far in stiffer deposits. The vane shear test is best suited for soft clays; whereas greatest applicability of the static cone has been to sands, although it appears well-suited to soft and medium clays as well.

IV. IN-SITU STRESSES AND DEFORMATION CHARACTERISTICS

A. Introduction

The response of soil to the application of load stresses depends greatly upon the initial state of stress and the stress path experienced by the soil. Deformations and local yielding are consequently affected to a marked degree by the geologic stress history, the magnitude of the in-situ vertical and lateral effective stresses and the nature of the stress changes imposed during construction. The importance of stress path considerations have been discussed by Lambe and Whitman (1969) and Ladd and Foott (1974), among others.

Most limit analyses, as performed by geotechnical engineers, are independent of soil deformation characteristics and the magnitude of initial lateral stresses. On the other hand, the prediction of settlements depends upon the accurate determination of deformation parameters, which in turn depend upon the in-situ effective stress levels. Historically, deformation parameters have been evaluated from the results of laboratory experiments on nominally undisturbed samples and then substituted in a simple elastic analysis to yield values of stresses and displacement for use in analysis and design.

Some inadequacies of this approach are evident. Significant soil disturbance is inevitable as a result of the sampling and laboratory testing processes. A consequence of this disturbance is that predictions based upon laboratory test results may grossly overestimate the movements, resulting in overconservative designs (Marsland, 1973). Proper simulation of field conditions in laboratory tests requires knowledge of in-situ stress conditions, quantities that can best be determined by in-situ tests. In fact, Wroth (1975) suggests that correct evaluation of the in-situ lateral stress may not be possible from the results of laboratory tests.

B. The Estimation of In-Situ Stresses

The simplest concept of an in-situ stress regime is based upon several simplifying assumptions:

1) The ground surface is reasonably level, so that the vertical stress is a principal stress which may be calculated from the weight of overlying material.

2) Horizontal stresses are equal in all directions.

3) The pore pressure is known.

4) The coefficient of lateral earth pressure, K_o, is known.

In reality, the in-situ stress state may deviate significantly from these assumed conditions as a result of tectonic or geologic processes. Even if these assumptions hold true for a particular deposit, uncertainty will arise in the determination of the vertical stress, σ_{v_o}, and pore pressure, u_o.

The calculation of σ_{v_o} may be influenced by several sources of uncertainty, as considered by Massarsch et al. (1975b) and Tavenas et al. (1975):

1) Determination of total unit weight, γ_t, is subject to laboratory measurement errors.

2) The presence of a dry, fissured crust will produce horizontal variations in σ_{v_o}.

3) The assignment of an average unit weight to an interval of depth may not reflect the true average unit weight of that depth interval.

4) The determination of depth and layer thicknesses is subject to error. In conventional soil investigations, borehole depths are probably known only to the nearest 0.1 meter.

The measurement of pore water pressure is probably only accurate to within ±0.1 m of water head (±1.0 kPa), even with the best piezometers currently available. Sources of error in the measurement of u_o include:

1) Temperature effects

2) Fluctuations in atmospheric pressure

3) Calibration errors

4) Zero shift in electrical transducers

5) Uncertain depth of transducer placement

6) Seasonal fluctuations of the groundwater table

7) Failure to achieve complete equilibrium between the groundwater and the measuring system.

C. In-Situ Lateral Stresses

The nature of in-situ lateral stresses may be understood by examining the results of laboratory oedometer tests which model the one-dimensional consolidation experienced by an idealized soil deposit as the overburden pressure is varied (Wroth, 1975). The observed stress paths corresponding to one-dimensional laboratory consolidation and subsequent unloading take the form presented in Figure IV-1, where:

$$q = (\sigma_1 - \sigma_3)/2 \qquad \text{(IV-1)}$$

$$p' = (\sigma_1' + \sigma_2' + \sigma_3')/3 \qquad \text{(IV-2)}$$

$$\eta = q/p'$$

and

$\sigma_1', \sigma_2', \sigma_3'$ = major, intermediate and minor principal effective stresses, respectively

For the usual case where the in-situ horizontal stress is less than the overburden stress,

$$\sigma_v' = \sigma_1'$$

$$\sigma_h' = \sigma_2' = \sigma_3'$$

In Figure IV-1, the vector AB represents the stress path of a soil under a one-dimensional loading which models the increasing overburden pressure and normal consolidation during sedimentation. A constant ratio, K_o, is observed between σ_h' and σ_v'. Therefore, for a given soil, under these loading

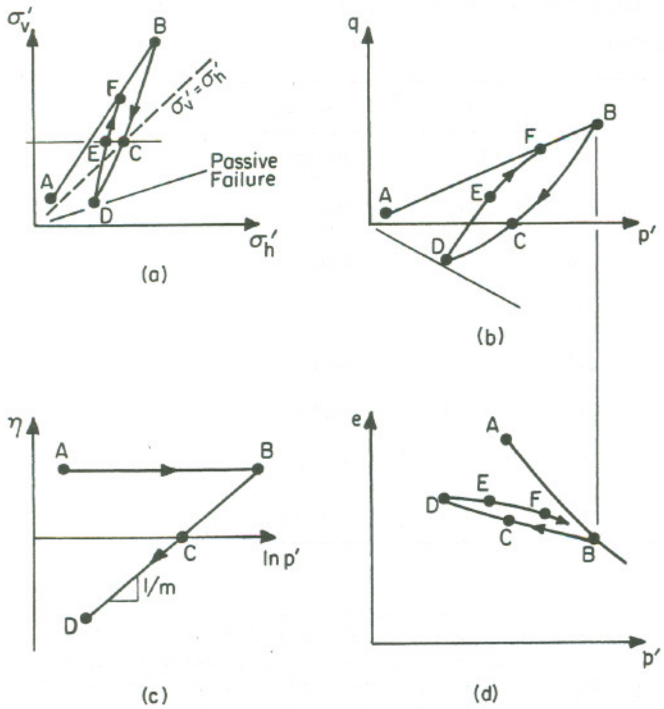


Fig. IV-1 Typical effective stress paths for soil consolidated one-dimensionally (from Wroth, 1975)

conditions:

$$K_o = \sigma_h'/\sigma_v' \text{ (a constant less than 1.0)} \qquad \text{(IV-4a)}$$

For normally consolidated soils, a theoretical relationship may be derived (Jaky, 1944):

$$K_o = (1 + 2/3 \sin \phi') \frac{1 - \sin \phi'}{1 + \sin \phi'} \qquad \text{(IV-4b)}$$

where ϕ' = soil friction angle, in terms of effective stresses.

A convenient approximation for this expression is often used that has been found to be accurate enough for most engineering applications (Wroth, 1972):

$$K_o \simeq 1 - \sin \phi' \qquad \text{(IV-4c)}$$

The derivation of these expressions is presented in Huck et al. (1974).

If a normally consolidated soil, at stress state "B" in Figure IV-1, is subjected to one-dimensional unloading, the stress path will follow a vector similar to $\overline{BC}$. At point "C", the horizontal and vertical stresses are equal, and K_o is equal to 1.0. For many soils (Ladd, 1975), this state corresponds to an overconsolidation ratio, OCR, ranging from 4 to 5.

If the stress path $\overline{BC}$ is assumed to be linear, the theory of elasticity may be applied to show that, for any point on the stress path $\overline{BC}$:

$$\Delta\sigma_h' = \frac{\mu'}{1 - \mu'} \Delta\sigma_v' \qquad \text{(IV-5)}$$

and,

$$K_o' = (\text{OCR})\,(K_{nc}') - \frac{\mu'}{1 - \mu'} (\text{OCR} - 1) \qquad \text{(IV-6)}$$

where:

K_{nc}' = coefficient of lateral earth pressure for a normally consolidated state

μ' = Poisson's ratio, in terms of effective stresses

By applying Equation IV-5 to the results of laboratory tests on 2 sands and 6 remolded clays, representative values of μ' have been obtained and correlated with Plasticity Index, I_p, by Wroth (1975). The test data appear in Table IV-1, and the resulting plot of μ' vs I_p is reproduced in Figure IV-2. It is emphasized that this correlation only applies to lightly overconsolidated soils. Heavily overconsolidated soils may exhibit much lower values of μ' as reported by Wroth (1975).

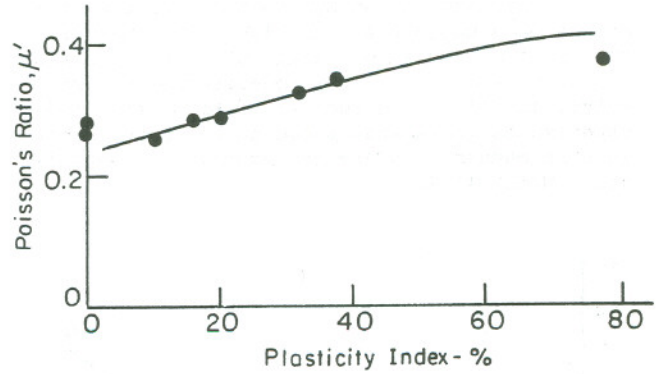


Fig. IV-2 Values of Poisson's ratio for lightly overconsolidated soils (from Wroth, 1975)

An important consequence of these findings is that for a lightly overconsolidated clay, K_o' may be readily estimated from I_p, ϕ', and OCR. A value of ϕ' is substituted in Equation IV-4 to calculate K_{nc}', and μ' is obtained from I_p using the plot in Figure IV-2. The best estimate of the OCR is made, based upon geologic information and/or laboratory test results. Equation IV-6 is then employed to determine K_o'.

Heavily Overconsolidated Soil

If a soil at stress state "C" in Figure IV-1 is subjected to additional one-dimensional unloading, the stress path takes on a curved shape similar to the arc segment CD. Along the stress path CD, the value of K_o' is greater than unity, and steadily increases with increasing OCR. The maximum value of K_o' occurs at point "D", where the soil is in a state of passive failure.

Along the "heavily overconsolidated" portions of the stress path between C and D, the horizontal stress, σ_h', is greater than the vertical stress, σ_v', and therefore becomes the major principal stress, σ_1'. For clarity in Figure IV-1, σ_h' is still identified as σ_3 in this range, however.

TABLE IV-1. Values of Soil Parameters Plotted in Figure IV-2

Wroth (1975)

No.	Soil	I_P %	μ'	m	Author
1	Pennsylvania sand	-	0.281	1.54	Hendron
2	Wabash river sand	-	0.267	1.60	Hendron
3	Chicago clay	10	0.254	1.36	Brooker
4	Goose Lake flour	16	0.282	1.60	Brooker
5	Weald Clay	20	0.287	1.58	Brooker
6	Kaolin	32	0.325	1.81	Nadarajah
7	London clay	38	0.346	2.26	Brooker
8	Bearpaw shale	78	0.371	2.92	Brooker

It has been shown by Wroth (1972) that the stress path segment BCD in Figure IV-1 may be transformed into a log-linear plot of the stress ratio, η, and the natural logarithm of the stress point, ln (p'). The results of Brooker's (1964) experiments on Bearpaw shale have been presented in this manner, and are reproduced in Figure IV-3, after Wroth (1975).

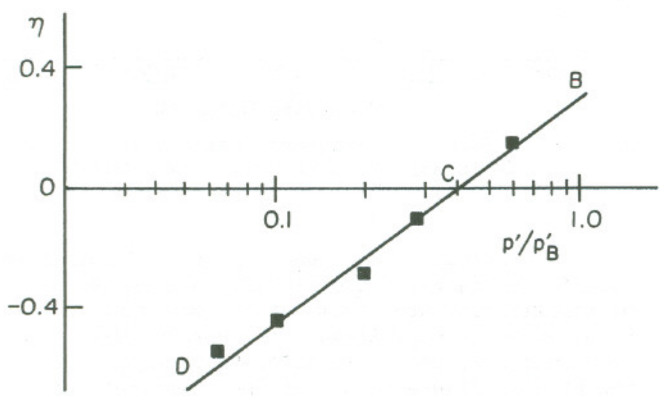


Fig. IV-3 One-dimensional unloading of Bearpaw shale (from Wroth, 1975)

The unloading stress path, BCD, may therefore be represented by the equation:

$$m\,(\eta - \eta_B) = \ln\,(p'/p'_B) \qquad \text{(IV-7)}$$

where:

p'_B, η_B = stress point, stress ratio at "B" in Figure IV-1

m = inverse of the gradient of BCD in Figure IV-1

Equation IV-7 may be rewritten as:

$$m\left[\frac{3(1-K'_{nc})}{1+2K'_{nc}} - \frac{3(1-K'_o)}{1+2K'_o}\right] = \ln\left[\frac{OCR(1+2K'_{nc})}{1+2K'_o}\right] \qquad \text{(IV-8)}$$

The values of the OCR corresponding to selected values of K'_o may thus be calculated for a given soil, if the parameters m and K'_{nc} are known. The parameter m has been obtained for the soils listed in Table IV-1, and correlated with I_p as presented in Figure IV-4, after Wroth (1975). The value of K_{nc} may be calculated from ϕ' using Equation IV-4.

The K_o' - OCR relationship predicted in this manner is compared in Figure IV-5 with the values of K_o' and OCR measured during one-dimensional un-

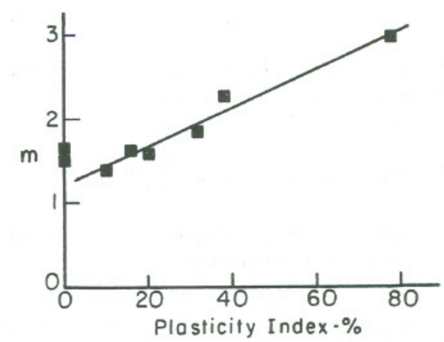


Fig. IV-4 Variation of the rebound gradient, m, with plasticity (from Wroth, 1975)

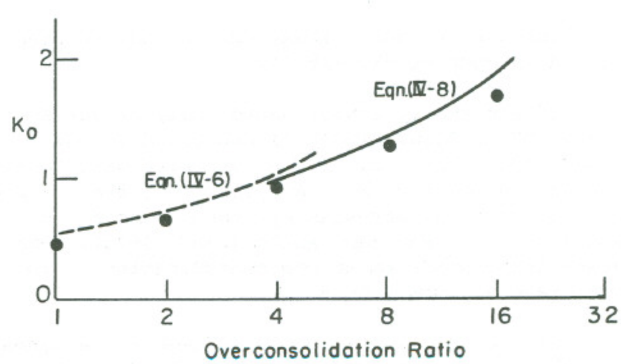


Fig. IV-5 Comparison of computed and experimental data for one-dimensional unloading of Boston blue clay (from Wroth, 1975) loading of Boston blue clay, after Wroth (1975). The predicted and observed values are seen to be in close agreement. The method described previously, applicable to lightly overconsolidated soils, is also seen to produce reliable predictions of K_o' in Boston blue clay, for low values of OCR.

Reconsolidated Soil

If the soil at stress state "D" in Figure IV-1 is reloaded one-dimensionally the effective stress will follow the approximately linear segment DE. The value of K_o' will decrease rapidly, as a large increase in σ_v' is accompanied by little change in σ_h'.

Stress states "E" and "C" correspond to the same vertical stress, σ_v', and preconsolidation pressure, $(\sigma_v')_B$, and hence possess identical overconsolidation ratios. The values of K_o' will differ, however, because stress state "C" is located on a path of primary unloading, while stress state "E" is located on a reloading path.

Therefore, the current stress state is unpredictable if a soil deposit has been subjected to more than one cycle of loading (deposition) and unloading (erosion).

Distribution of K_o' in the Ground

The distribution of K_o' in a natural soil deposit may be the result of a complex sequence of stress variations caused by deposition, erosion, tectonic movement, water table fluctuations and previous construction history. The effects of secondary compression, vegetation, capillary pressures and heterogeneous soil distributions will serve to complicate this situation even further.

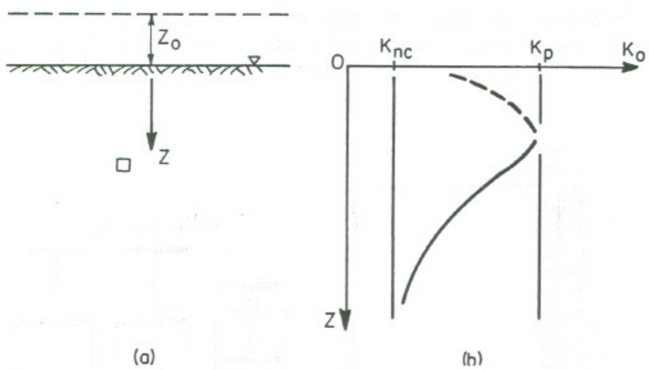


Fig. IV-6 Typical vertical profile of K_o' for an overconsolidated deposit

An idealized soil deposit in the normally consolidated state, with constant unit weight and horizontal ground surface, will exhibit a uniform distribution of K_o' with depth, as sketched in Figure IV-6(a). If a soil layer of thickness, z_o, were to be removed through erosion, the value of the overconsolidation ratio at any depth, z, would become equal to $(z + z_o)/z$. If the soil is only lightly overconsolidated, the methods presented previously could be employed to show that K_o' should decrease with depth as sketched in Figure IV-6(b). At a point near the ground surface, where the overconsolidation ratio is sufficiently high, the value of K_o' will be limited by a passive failure condition. An equivalent distribution of K_o' would also result from the effective stress increase and decrease caused by a fall and subsequent rise of the water table.

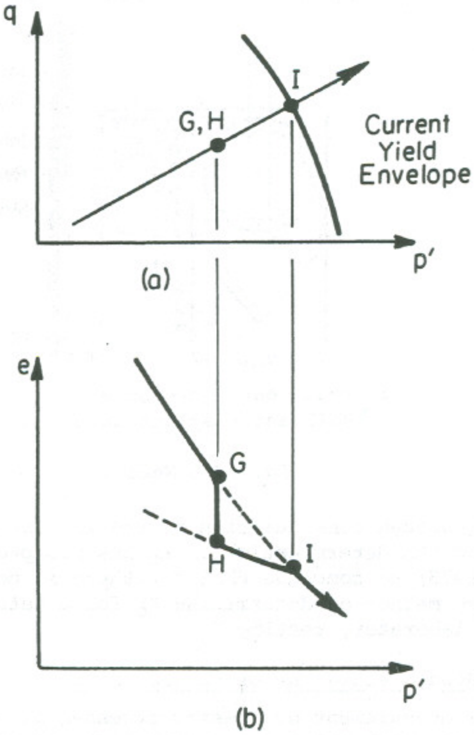
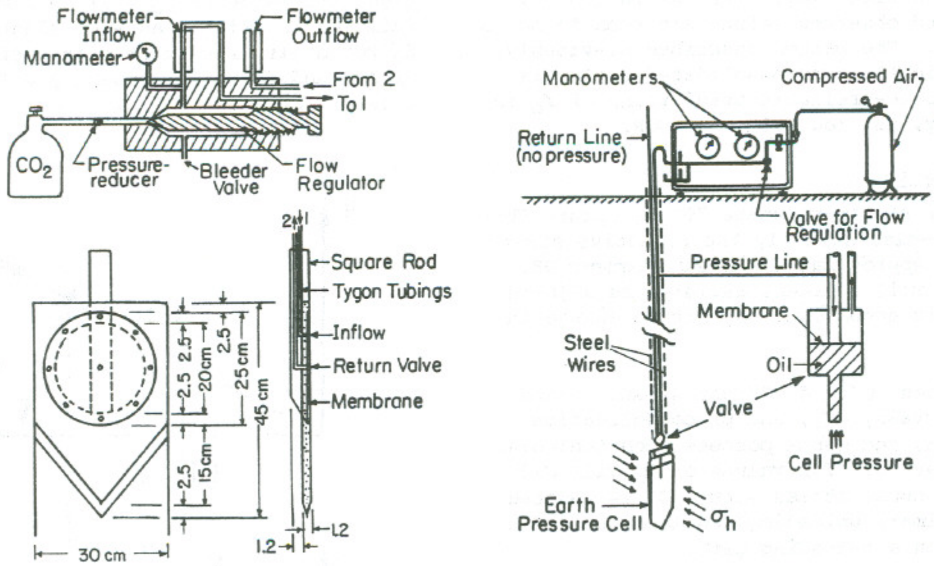


Fig. IV-7 Change of state caused by delayed consolidation

Delayed, or secondary compression, as described by Bjerrum (1967), is a common cause of apparent overconsolidation. If a soil element at stress state "G" and void ratio e_G in Figure IV-7 undergoes secondary consolidation, the void ratio will decrease to a value, e_H, while the stress state, H, remains unchanged from point G. At this state, the soil will behave as if it were lightly overconsolidated. If additional vertical stresses are imposed, the void ratio will decrease along the recompression curve "$e_H - e_I$", Figure IV-7. The vertical stress at state "I" would be interpreted as the preconsolidation pressure, and a value of K_o' would be estimated accordingly. In fact, K_o' is that of a normally consolidated soil, since the effective stresses have not changed during secondary consolidation. This phenomenon will produce an effect of apparent OCR which is independent of depth.

Laboratory Measurement of K_o'

The most widely used assumptions in the laboratory evaluation of in-situ lateral stresses are of questionable validity (Wroth, 1975). Even the most advanced techniques, such as those developed by Poulos and Davis (1972) or Tavenas et al. (1975), may be expected to produce accurate results only when applied to normally consolidated soils.



(a) Total earth pressure cell and read-out unit

(b) Schematic diagram of the earth pressure cell (Glötzl) method

Fig. IV-8 Measurement of the in-situ lateral pressure using Glötzl cells

A thorough consideration of the factors influencing the determination of K_o has prompted Wroth (1975) to conclude that "...there is no satisfactory method of determining K_o for a natural soil by laboratory testing."

D. In-Situ Measurement of Lateral Stress

The measurement of in-situ stresses is clearly an area which holds great potential for the successful application of in-situ testing methods.

The direct measurement of stresses in earth fills may be achieved through the installation of pressure cells during construction. Accurate results are difficult to obtain and verify, but the techniques of installation and measurement are fairly well developed.

Strictly speaking, it is not possible to directly measure stresses in undisturbed soil, since the insertion of a measuring device creates disturbance and modifies the local state of stress. In practice, in-situ measurement may be very successful if soil disturbance can be minimized and adequate time allotted for the re-establishment of equilibrium stress conditions. A number of imaginative techniques have been developed which attempt to realize these goals. These techniques are only summarized in the following paragraphs, and the reader is referred to the references for more details.

Stress Measurement in Soft Clays

Pressure Cells: Total pressure cells, which are pushed into undisturbed soil or placed in uncased boreholes, may provide reliable stress information if disturbance is kept to a minimum and adequate time is allowed for the dissipation of pore pressures. The Glötzl cell, depicted in Figure IV-8, has been found to be suitable, and is discussed in detail by Tavenas et al. (1975) and Massarsch et al. (1975b).

Hydraulic Fracturing

The hydraulic fracturing technique is frequently used to determine stresses in rock strata, and is experiencing increasing use in soil studies. A schematic of the operation is presented in Figure IV-9. Increasing water pressure is applied to an isolated section of a borehole. Soil fracture, in principle along the minor principal plane, is accompanied by a marked increase in water in-flow which is monitored at the surface. The applied pressure is gradually reduced, and when the water pressure is equal to the stress which is normal to the cracks, the cracks close and water flow drops significantly. The "closure pressure" is

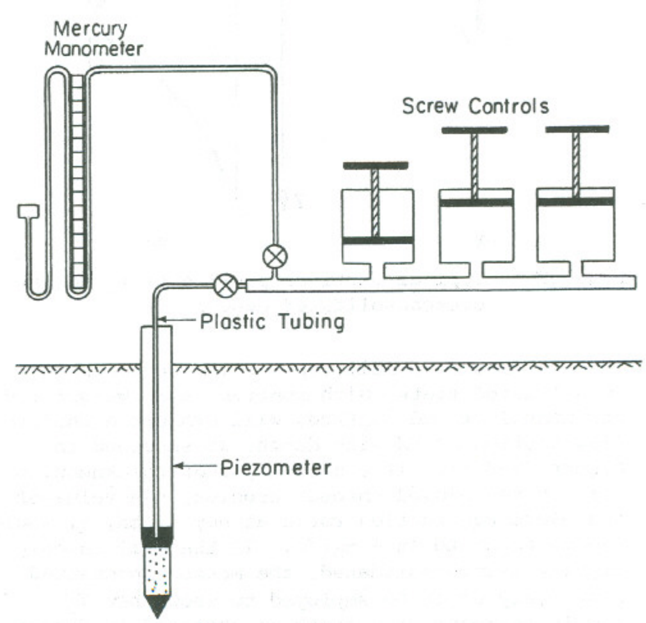


Fig. IV-9 Schematic diagram of the hydraulic fracture method recorded and is an estimate of σ_h.

In theory, hydraulic fracture can measure σ_h only in soils in which K_o is less than unity. In practice, the closure pressure often represents a weighted average of σ_h and σ_v, since the preferred crack orientation may be along horizontal stratifications. The closure pressure is not always clearly defined, and is subject to the uncertain effects of soil strength and deformation characteristics.

The theory of the fracture mechanism has been presented by Bjerrum et al. (1972). The details of the testing procedure are considered by Tavenas et al. (1975) and Massarsch et al. (1975b), who prefer the use of total pressure cells to the hydraulic fracturing method.

Pressuremeter Tests

The idealized results of a pressuremeter test in soft clay are presented in Figure III-21 and the details of the pressure-volume curve are discussed in Section III E. The pressure at point B (in Figure III-21) is generally assumed to represent σ_h, but in practice, point B is often poorly defined. Borehole advancement produces disturbance and stress relief in the adjacent soil, and small details of test technique have been found to introduce large uncertainties in test results. Tavenas et al. (1975) conclude that the conventional pressuremeter test is unsuitable for determining lateral stresses in soft clay because of this.

The self boring pressuremeter, depicted in Figure III-19, minimizes soil disturbance by limiting lateral soil movements to small values. Even this small disturbance produces excess pore pressures which must be allowed to dissipate before testing. The equilibrium values of lateral stresses are not greatly affected, however, once this dissipation has occurred.

Measurement of K_o in Stiff Soils

The measurement of K_o in stiff clays, sands, and soft rocks poses particular problems. Pushed-in-place total pressure cells generally cannot be used because of the high resistance of these soils to penetration. Hydraulic fracturing is effective only if K_o is less than one, and is not possible in previous sand deposits. Self boring instruments with load cells have been used, but may present problems of compliance, in that the stiffness of these soils may be significant when compared to the stiffness of the load cell itself.

The pressuremeter device may be the most suitable means of directly measuring lateral stresses in these materials according to Wroth (1975).

A great deal of development work needs to be done in this area of in-situ testing. It is anticipated that "secondary" methods, which deduce stress conditions from other parameters, will receive increasing attention.

E. In-Situ Measurement of Deformation Characteristics

It has been emphasized that soil deformation characteristics must be measured under the appropriate combination of effective stresses, since stress-strain behavior is stress path dependent. The estimation of these in-situ stresses may be a source of error in conventional laboratory testing. In-situ testing has the potential of "bypassing" this source of uncertainty.

Most current methods for predicting soil deformations model the soil skeleton as a linear-elastic material. Nonlinear stress-strain response is treated in a piece-wise linear manner. Newer methods, which predict ground response on the basis of elastic-plastic behavior, are receiving increased attention, but for this discussion in-situ measurement procedures will be considered in the framework of the theory of elasticity.

Application of the theory of elasticity necessitates a clear distinction between total and effective stresses. Terms which apply to total stress will be denoted with the subscript "u" while terms related to effective stress will be identified with a prime, '.

The shear modulus, G, is a convenient deformation parameter, because it is defined in terms of a stress difference which may be calculated from either effective or total stress measurements. For an isotropic elastic material:

$$2G = \frac{\sigma_1 - \sigma_3}{\varepsilon_1 - \varepsilon_3} = \frac{\sigma_1' - \sigma_3'}{\varepsilon_1 - \varepsilon_3} \qquad \text{(IV-9)}$$

where:

G = shear modulus

σ_1, σ_3 = major and minor principal stresses

ε_1, ε_3 = major and minor principal strains

Equation IV-9 may be transformed by applying relationships between the elastic constants, yielding a convenient relationship between the drained and undrained modulus of compression:

$$2G_u = 2G' = \frac{E'}{1 + \upsilon'} = \frac{E_u}{1 + \upsilon_u} \qquad \text{(IV-10)}$$

where:

E', E_u = Young's modulus for drained and undrained loading, respectively

υ', υ_u = Poisson's ratio for drained and undrained loading, respectively

It is re-emphasized that these moduli are dependent upon stress level, and should be measured at stress levels appropriate for the prototype situation.

Methods of Measuring Deformation Parameters

The methods currently used for measuring deformation parameters in-situ have been reviewed by Wroth (1975), and may be catalogued as follows:

1) Pressuremeter Tests have been found to yield the most appropriate values of parameters for the prediction of ground deformation caused by foundation and earthwork construction.

2) Plate Loading Tests are frequently employed, and yield reliable results if performed with care. However, the expense and complexity of the procedure may make use of this method impractical in many cases.

3) Seismic Methods possess unique advantages, but yield deformation parameters which correspond to levels of strain an order of magnitude or more smaller than the strains which occur in other testing methods. A correction must therefore be made before the results are applied to the prediction of ground deformations other than those caused by vibration.

4) Back Analysis of well documented case histories is an important means of understanding any aspect of soil behavior. This approach is not practical for "everyday" design problems, but does constitute the means of evaluating the "everyday" design methods.

In the following sections, each of these methods is discussed, with special emphasis placed upon the Pressuremeter Test.

F. Analysis of the Pressuremeter Test (Wroth, 1975)

The results of a two-cycle PMT in sand are presented in Figure IV-10 as a plot of applied pressure vs. radial deformation. The two major cycles of loading are identified by the paths between points A through H and H through M.

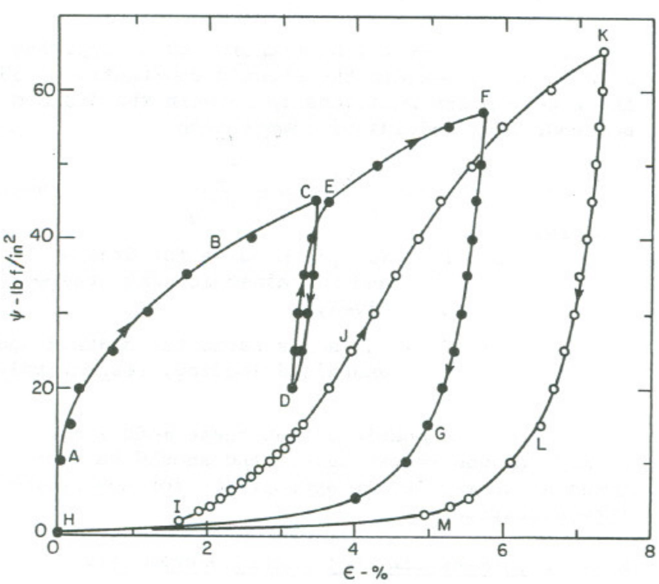


Fig. IV-10 Results of Pressuremeter test in sand at Landvetter, Sweden (from Wroth, 1975)
• 1st cycle of loading
o 2nd cycle of loading

The secondary unloading and reloading path, through points C, D and E in Fig. IV-10, illustrates that for small strains, the unloading behavior is linear and recoverable. When the sand is reloaded from point D, it responds elastically until the previous maximum pressure is reached at point E. The sand then yields, undergoing plastic strain as it rejoins the virgin compression curve BCF.

Upon unloading from point F to G, the sand once again exhibits elastic rebound. As the applied pressure is decreased from point G to H, the membrane of the pressuremeter deflates to its original size, and a loose annulus of soil collapses around the pressuremeter.

The reloading curve, $\overline{HIJK}$, exhibits an initial gradient which is an order of magnitude less than that of the original loading curve, $\overline{ABC}$, the difference being due to the extremely disturbed state of the soil at point H. The behavior of the disturbed sand, illustrated in segment $\overline{HIJ}$, serves as an example of the effects of disturbance on the measurement of deformation parameters in conventional pressuremeter tests. This phenomenon might well occur as the result of performing a PMT in an oversize borehole; a loose annulus of soil may collapse around the apparatus, and small pressure increments will induce inordinately large deformations in the disturbed material.

Interpretation of a Drained PMT

The interpretation of an undrained PMT is discussed in Section III-E. It was explained that the complete undrained stress-strain curve could be obtained from PMT results in an incompressible soil. That analysis has been extended by Wroth and Windle (1975) to take account of volume changes. The drained stress-strain curve may thus be derived from a PMT in permeable soil.

The soil surrounding the expanding cylindrical membrane is assumed to be deformed in axial symmetry and plane strain, all displacements therefore being radial in direction. These assumptions have been shown to hold true by Wroth and Hughes (1973) and Hartman and Schmertmann (1975). As a result, an analysis of the PMT need only consider one horizontal plane of soil at mid-height of the apparatus and consider the variation of stresses, strains, and displacements along a typical radius $\overline{ABC}$ shown in Figure IV-11(a).

A cylindrical coordinate system (r,z,θ) is introduced and the coordinate, r, is chosen to refer to the displaced position B' of the general point B. The displacement, $\overline{BB'}$, of a general point is indicated as ξ_B. Compressive strains are given positive values.

The circumferential strain, ε_θ, is everywhere tensile, so:

$$-\varepsilon_\theta = \frac{\xi}{r - \xi} > 0 \qquad \text{(IV-11)}$$

PMT results have been plotted in Figure IV-10 in terms of the dimensionless parameter, $\varepsilon = \xi_A/a_o$, which is equal to the strain, $-\varepsilon_\theta$, at the wall of the cavity in the soil.

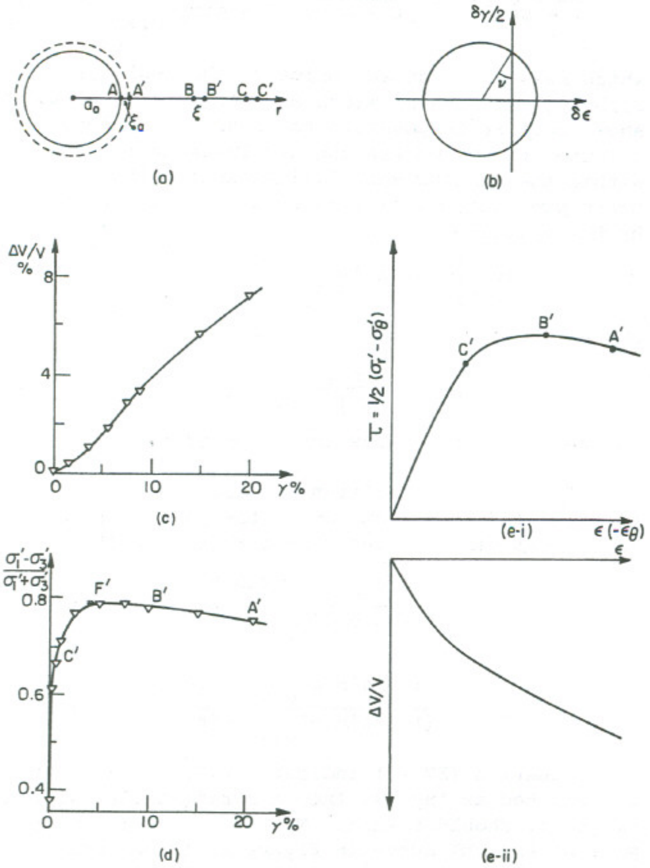


Fig. IV-11 Stress-strain behavior of dense sand during pressuremeter test (from Wroth, 1975)

As the test proceeds, the membrane expands, and an element of soil which was originally located at point A in Figure IV-11(a) will be displaced a distance ξ_A, and will now be located at point A'. Similarly, soil at point B will move some distance ξ_B to the point B'.

The radial strain in the soil, ε_r, is everywhere compressive, so that:

$$\varepsilon_r = -\frac{\partial \xi}{\partial r} > 0 \qquad \text{(IV-12)}$$

If some drainage is assumed to occur, a soil element of initial volume V_o will exhibit a volume change ΔV in response to the increased effective stresses. The volume change may be negative or positive.

The stress-strain and volume change behavior are illustrated in Figure IV-11. At any instant during the test, it may be said that, for the soil in contact with the membrane, the volumetric strain, ε_v, is equal to the tangential strain, ε_θ, times a factor, $-\ell$.

That is to say:

$$\varepsilon_v = \frac{\Delta V}{V_o} = -\ell\varepsilon_\theta\,, \qquad \text{(IV-13)}$$

where ΔV represents an increase in soil volume, and the factor ℓ is positive for soil expansion (dilation), and negative for soil contraction. In Figure IV-11(c), the volumetric strain is seen to vary linearly with shear strain for a large portion of the test. For that portion of the test, the factor ℓ is therefore a constant. The value of ℓ need not be constant, however, since the method is applied to the test data at specific points in time, and different values of ℓ may be substituted in each iteration.

The Mohr circle of strain for the soil element which has been distorted from point A to A' is presented in Figure IV-11(b). The angle of dilation, ν, is indicated in the figure, and is defined as:

$$\sin\nu = \frac{\partial \Delta V/V_o}{\partial \gamma}$$

The parameter ℓ may be expressed in terms of the angle of dilation. The plane strain assumptions imply that the vertical strain, ε_z, is zero, and that for small deformations:

$$-\frac{\Delta V}{V_o} = \varepsilon_r + \varepsilon_\theta = +\ell\varepsilon_\theta \qquad \text{(IV-14)}$$

therefore:

$$\ell = 1 + \frac{\varepsilon_r}{\varepsilon_\theta} \qquad \text{(IV-15)}$$

$$\ell = 1 - \frac{1 - \sin\nu}{1 + \sin\nu} = \frac{2\sin\nu}{1 + \sin\nu} \qquad \text{(IV-16)}$$

Alternatively, ℓ may be expressed in terms of the stress dilatancy parameter, D, introduced by Rowe (1962):

$$D = 1 + \frac{\Delta V/V_o}{\varepsilon_r} = \frac{1}{1 - \ell} \qquad \text{(IV-17)}$$

The detailed derivation by Wroth and Windle (1975) leads to the following expression for shear stress, $\tau = 1/2(\sigma_r - \sigma_\theta)$ in terms of the applied pressure p, and corresponding strain ε:

$$\tau = \frac{\varepsilon(1+\varepsilon)(2+\varepsilon-\ell)}{2(1+\ell\varepsilon)}\,\frac{dp}{d\varepsilon} \qquad \text{(IV-18)}$$

The case of no soil volume change (i.e., a completely undrained test in a saturated soil) corresponds to a value of 0 for the factor ℓ, and equation (IV-18) becomes

$$\tau = \frac{\varepsilon(1+\varepsilon)(2+\varepsilon)}{2}\,\frac{dp}{d\varepsilon} \qquad \text{(IV-19)}$$

Equation (IV-19) is identical to equation (III-9) presented earlier.

The important consequence of this analysis is that if the volume change in the annulus of soil immediately surrounding the membrane can be measured, ℓ can be calculated and the complete stress-strain curve may therefore be derived from equation (IV-18). It should be noted that ℓ need not remain constant during the pressuremeter test, as the analysis applies to each small increment of the expansion of the membrane, and ℓ may be calculated for each such increment.

Electrical resistivity methods have been employed to measure this volume change by Windle and Wroth (1975). In the laboratory, soil porosity was correlated with soil resistivity. Soil resistivity was measured during field pressuremeter tests by means of electrodes mounted on the membrane face. The field measurements indicated porosity changes which were consistent and reasonable. The approach suffers, however, from the fact that the measured field resistivity is actually the average resistivity of a fairly large volume of soil. The method of analysis calls for consideration of volume changes only with a narrow annulus of soil.

In practice, it is not yet feasible to measure soil volume changes during a conventional PMT. The "drained" analysis is useful nonetheless, in estimating the probable errors involved when the "undrained" analysis is applied to a test in which some drainage occurs.

Laboratory tests performed by Windle and Wroth (1975) indicate that a value of -0.3 for the volume change factor, ℓ, is appropriate for a normally consolidated clay in a fully drained PMT. If this value of ℓ is substituted in the "drained analysis, the computed values of shear stress will be somewhat higher than the values of shear stress calculated according to the "undrained" analysis. The "undrained" analysis is seen to underestimate shear stress by some 25% for $\ell = -0.3$ and $\varepsilon = .06$.

Use of the undrained analysis for a partially drained PMT in clay will therefore result in an underestimate of the peak shear stress. This error is partially compensated, however, by the fact that an undrained PMT measures a shear strength which is not the true in-situ undrained strength of the soil, but rather the strength of the soil after some degree of consolidation has occurred. The actual degree of this compensation is not known.

Analysis of a Pressuremeter Test in Sand

The original analysis of a PMT in sand, presented by Gibson and Anderson (1961), assumed that the sand failed at a constant stress ratio, with no volume change. These assumptions are reasonable only for the case of a very loose sand. The stress-strain behavior of a dense sand in plane strain is illustrated in Figure IV-11 d and e which indicate that for a substantial range of strain the sand deforms at a nearly constant stress ratio, $\sigma_1'-\sigma_3'/\sigma_1'+\sigma_3'$, and rate of dilation. The behavior may be described by the stress dilatancy relationship:

$$\frac{\sigma_1'}{\sigma_3'} = \left(1 + \frac{\Delta V/V}{\varepsilon}\right)\tan^2\left(\frac{\pi}{4} + \frac{\sigma_f'}{2}\right) \qquad \text{(IV-33)}$$

For plane strain:

$$\phi_f' = \phi'_{c.v.}$$

where $\phi_{c.v.}$ pertains to shear at constant volume.

The angle of dilation, ν, has been introduced in the analysis of a drained PMT. The definition of ν is recalled, and through triginometric substitution Equation (IV-20) can be transformed to:

$$\frac{1 + \sin\phi'}{1 - \sin\phi'} = \frac{1 + \sin\nu}{1 - \sin\nu}\;\frac{1 + \sin\phi'_{c.v.}}{1 - \sin\phi'_{c.v.}} \qquad \text{(IV-21)}$$

which relates ϕ' at any point to the angle of dilation, ν Hughes, Wroth and Windle (1975) have shown that it is possible to obtain a complete solution to the stress and displacement fields within the annular soil failure zone. The membrane pressure, p, is related to the strain, ε, by the equation:

$$\ln\frac{p}{p_f} = a\ln\frac{\varepsilon}{\varepsilon_f} \qquad \text{(IV-22)}$$

where:

$$a = \frac{\sin\phi'\,(1 + \sin\nu)}{1 + \sin\phi'} \qquad \text{(IV-23)}$$

and subscript f denotes the onset of failure.

If the stress dilatancy relationship in Equation (IV-20) is employed, the parameter, a, may be expressed in terms of either ν or ϕ':

$$a = \frac{\sin\phi'_{c.v.} + \sin\nu}{1 + \sin\phi'_{c.v.}} \qquad \text{(IV-24)}$$

$$a = \frac{(1 - \sin\phi'_{c.v.})\sin\phi'}{(1 - \sin\phi'_{c.v.}\sin\phi')} \qquad \text{(IV-25)}$$

Equation (IV-22) indicates that if PMT results are plotted as lnp vs. lnε, a straight line with a slope, a, should result. The virgin loading segment of the PMT curve in Figure IV-10 has been replotted in this manner in Figure IV-12. An average gradient, a = 0.455, is seen to fit the data closely. If the value of $\phi'_{c.v.}$ can be estimated, equations (IV-24) and (IV-25) may then be employed to determine ϕ' and ν.

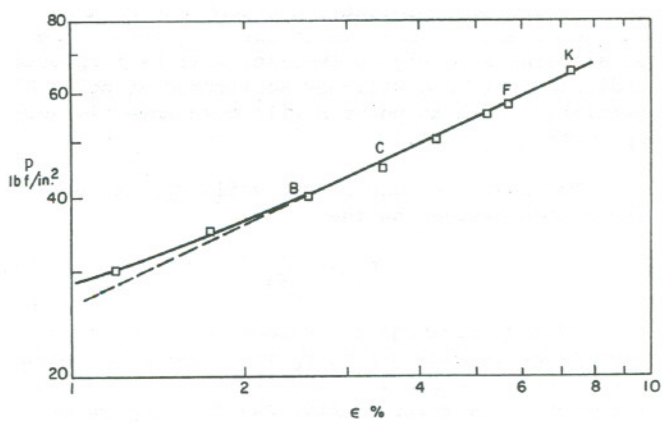


Fig. IV-12 Results of part of pressuremeter test on sand (after Wroth, 1975)

This method is not sensitive to small variations in $\phi'_{c.v.}$, and so for most purposes it is sufficiently accurate to estimate $\phi'_{c.v.}$ from drained triaxial tests on loose samples of the sand in question.

The calculation of the complete stress-strain from a PMT in sand proceeds according to this method as follows:

1) While the sand is failing, (along curve BCDEFG in Figure IV-10), the principal stress ratio, σ'_r/σ'_θ, is given by the value of ϕ' deduced from the experimentally observed value of a. For sand in contact with the membrane, the major principal stress, σ'_r, is equal to the applied pressure, p, so the minor principal stress, σ'_θ, and the deviator stress, $(\sigma'_r - \sigma'_\theta)$, may be calculated from the stress ratio.

2) For the "pre-failure" stage of the test, corresponding to segment AB in Figure IV-10, shear stress may be calculated by means of Equation (IV-18), if the rate of volume change, ℓ, is known. Application of this equation requires the selection of a value of ℓ for each data point. Since soil volume changes are small during this phase of the PMT, it has been found convenient, and not inaccurate, to set $\ell = 0.$ when applying Equation (IV-19) to the "pre-failure" stage.

This method has been applied to the PMT data in Figure IV-10. The resulting stress-strain curves are presented in Figure IV-13. It is seen that the calculated value of σ'_θ remains nearly constant throughout the test, providing a confirmation of the in-situ effective stress, p_o.

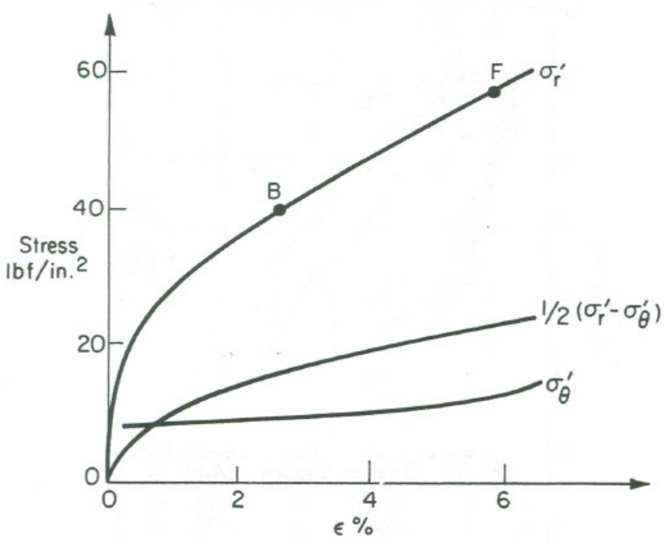


Fig. IV-13 Stress-strain curves derived from the pressuremeter test on sand (data in Figure IV-10) (from Wroth, 1975)

The importance of this method of analysis is evident. The complete stress-strain behavior of a sand may be derived from a PMT with a minimum of computation effort. Furthermore, the method does not require an estimate of the limit pressure, p_1, and thus permits the termination of a PMT after a small strain, of the order of 10%. The important parameters ϕ' and ν may be accurately evaluated, and deformation parameters may be obtained from the stress-strain curve.

The gradient of the derived shear-stress curve in Figure IV-13 may be related to the shear modulus G'. Recalling Equations (IV-13) and (IV-14)

$$\varepsilon_r - \varepsilon_\theta = (\ell - 2)\varepsilon_\theta = (2 - \ell)\varepsilon$$

therefore:

$$\frac{1}{2}\ \frac{d(\sigma'_r - \sigma'_\theta)}{d\varepsilon} = \frac{(2-\ell)}{2}\ \frac{d(\sigma'_r - \sigma'_\theta)}{d(\varepsilon_r - \varepsilon_\theta)}$$

$$= (2 - \ell)\ G' \qquad \text{(IV-26)}$$

Pressuremeter Tests in Sand

A number of considerations should be kept in mind when evaluating a PMT in sand, including the following.

The elastic modulus deduced from a PMT is appropriate for stresses and strains in the horizontal direction only. This may be important when testing anisotropic materials.

Finite Element studies performed by Hartman and Schmertmann (1975) have shown that non-linear soil behavior is required to produce ΔV vs. p curve of the usual type presented in Figure III-20.

The pressuremeter modulus, E_{PMT}, suggested by Menard (1975) is frequently used in practice as opposed to values deduced theoretically according to the methods just described and is defined as follows:

$$E_{PMT} = 2(1 + \mu)\ V_o\ \Delta p/\Delta V \qquad \text{(IV-27)}$$

where: V_o = volume of the cavity at the instant when $\Delta p/\Delta V$ is measured, provided $\Delta p/\Delta V$ is measured in the pseudo elastic phase of the test indicated in Figure III-20.

μ = Poisson's ratio

Non-linear soil behavior limits the validity of the assumption of a constant pressuremeter modulus, and hence E_{PMT} must be considered as a fictitious property.

The effect of a smear zone of disturbed soil on the value of the pressuremeter modulus is illustrated in Figure IV-14 after Hartman and Schmertmann (1975). In pre-bored holes, soil disturbance results in the measurement of a decreased modulus. On the other hand, the hysteresis of borehole unloading and reloading results in the measurement of an increased modulus. In practice, the two effects may compensate, resulting in the measurement of a more nearly accurate modulus value. The presence of a smear zone does not alter the general shape of the PMT curve.

An investigation by Laier et. al. (1975) has shown that the length/diameter ratio, L/D, of the pressuremeter apparatus will affect the limit pressure as indicated in Figure IV-15. A correction for the limit pressure is therefore required

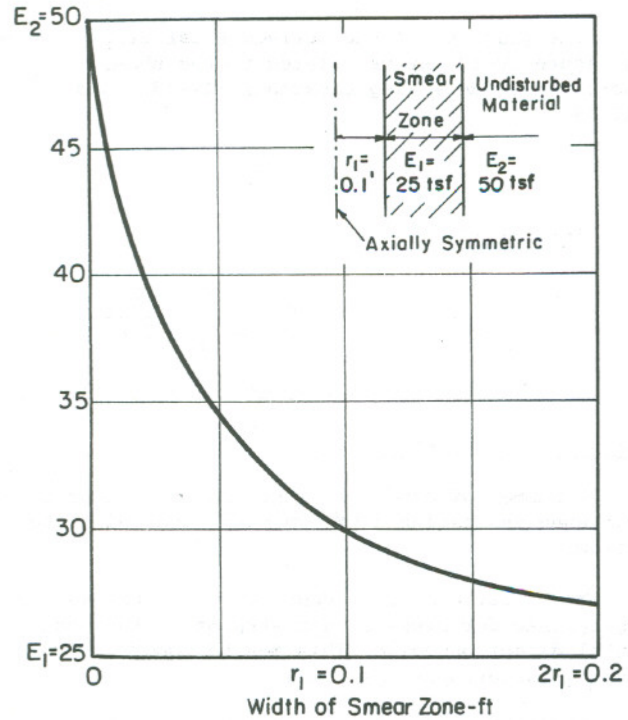


Fig. IV-14 Modulus calculations as a function of width of smear zone in linear plastic material with ν = 0.3 (from Hartman and Schmertman, 1975)

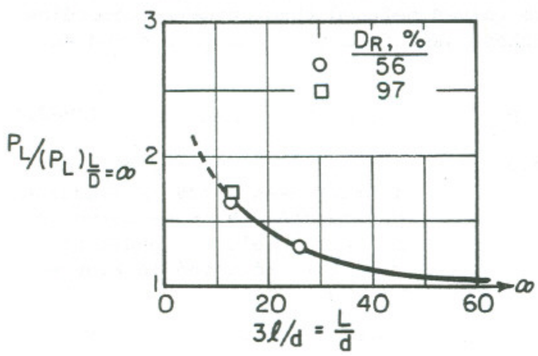


Fig. IV-15 Influence of PM length/diameter on measured limit pressure (ΔV measured along middle 1/3 of pressuremeter) (from Laier et al.,1975)

if the test interpretation is based upon an assumption of infinite pressuremeter length. As long as the ratio, L/D, is greater than about 4, measurements of elastic modulus do not need to be corrected for the infinite length assumption.

Pressuremeter Tests in Clay

An investigation by Roy et al. (1975) has shown that soil disturbance has a dramatic effect on the observed lateral stress and modulus for a PMT in clay. The effects of disturbance on the observed limit pressure were found to be minor. The effects of different borehole preparation techniques on observed lateral stress, modulus and limit pressure are summarized in Figures IV-16, 17, and 18.

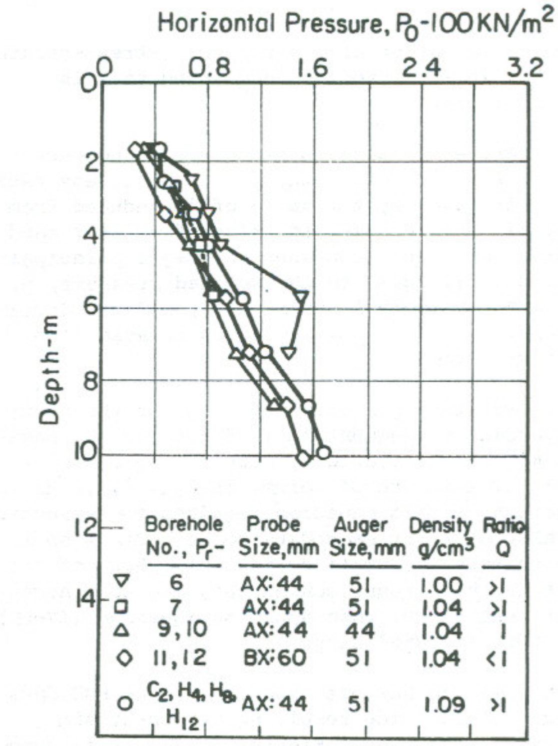


Fig. IV-16 Horizontal pressure measured at the Saint-Alban site (from Roy et al., 1975)

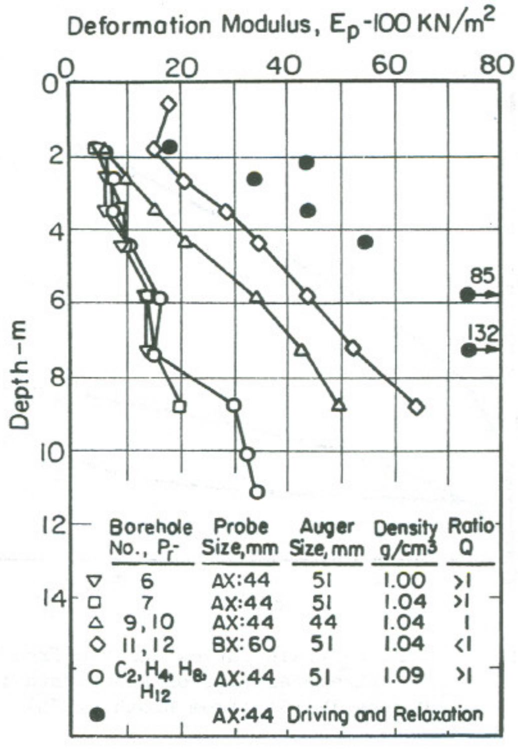


Fig. IV-17 Deformation modulus measured at the Saint-Alban site (from Roy et al., 1975)

At present, the most effective way of minimizing soil disturbance in the PMT is through the use of self boring devices, such as those sketched in Figure III-19.

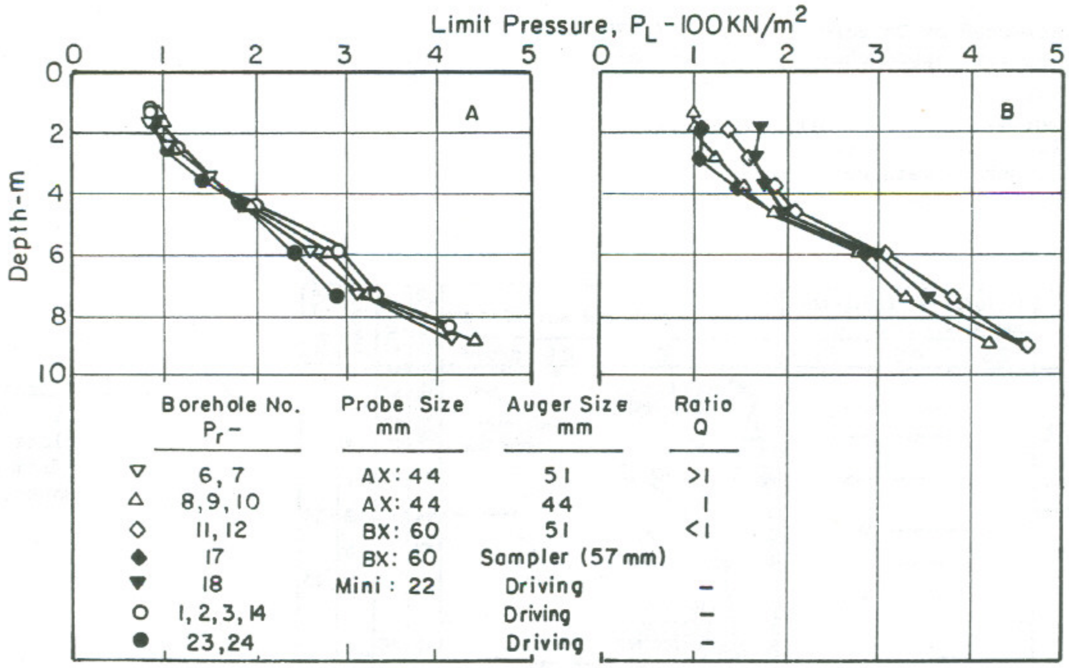


Fig. IV-18 Limit pressure measured at the Saint-Alban site: A-with different techniques. B-with different probes (from Roy et al., 1975)

In order to insure that a pressuremeter test is truly undrained, it must be carried out at a speed such that the rate of strain experienced by the soil is considerably higher than would normally be the case in good laboratory practice. Values of soil moduli determined in the laboratory may be expected to be considerably less than the corresponding values measured in an undrained PMT. Both the difference in strain rate between the two methods and the disturbance of laboratory samples contribute to this discrepancy.

G. Plate Bearing Tests

Plate bearing tests have been a traditional in-situ method for estimating the soil modulus, E, for the purposes of estimating the immediate settlement of spread footings. The vertical modulus of deformation is deduced from the load settlement behavior of a suitable bearing plate placed at a desired depth. A variation of the method is to measure the horizontal modulus using one side of a trench for the bearing test and the other side for the reaction. Experience has proven the reliability of plate tests, but the large costs involved have stimulated increasing interest in alternate methods.

Recent tests at the Division of Building Research Station in the U.K., as reported by Marsland (1973, 1975), have shown that an order of magnitude difference may exist between field and laboratory values of soil moduli. This discrepancy may be minimized by leaving the excavated surface exposed as short a time as possible, and by reestablishing the in-situ effective stresses before testing.

Shields and Bauer (1975) have undertaken a comparative study of full scale footing tests, plate bearing tests, pressuremeter tests and laboratory triaxial tests on sensitive clay in Ottawa, Canada. The soil profile is reproduced in Figure IV-19 and a schematic of the footing test arrangement is presented in Figure IV-20. The moduli determined from these tests are summarized in Figure IV-21.

Two series of pressuremeter tests were performed. The first series yielded low modulus values as the result of a large degree of soil disturbance. The more painstaking second series yielded higher modulus values. The calculations assumed a value of 0.33 for Poisson's ratio. Use of the correct undrained Poisson's ratio of 0.5 produces an increase of 13% in the calculated moduli. The PMT moduli in Figure IV-21 have been increased by this amount

The plate loading tests are seen to provide modulus values which most closely coincide with those determined from the model footing test. However, the plate load test is considerably more expensive than the other in-situ methods, particularly when it is performed with the degree of care which is required for accurate results.

H. Vibro-Seismic Tests

The basis of all seismic exploration techniques for evaluation of deformation characteristics is an interpretation of the time required for particular types of elastic waves to travel known or inferred distances through the subsurface. A variety of information of interest to the geotechnical engineer may be obtained from vibroseismic test results, and a more complete review of this method is planned for a subsequent report.

A state of the art review of vibroseismic methods for the determination of soil moduli has been presented by Ballard and McLean (1975). The following five approaches were discussed:

Refraction
Rayleigh wave - vibratory source
Rayleigh wave dispersion
Crosshole methods
Uphole/Downhole methods

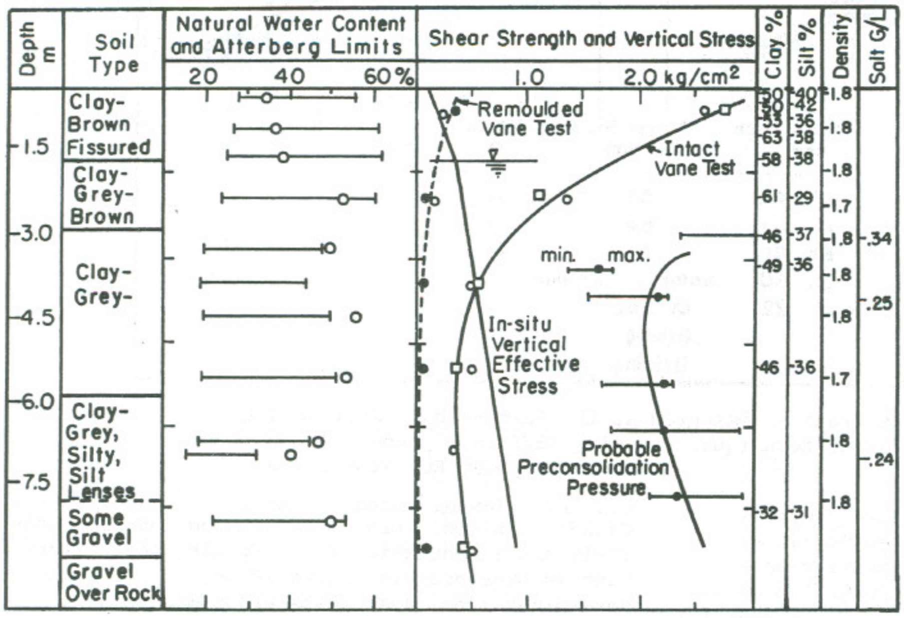


Fig. IV-19 Subsoil profile at site of large scale plate load tests (from Shields and Bauer, 1975)

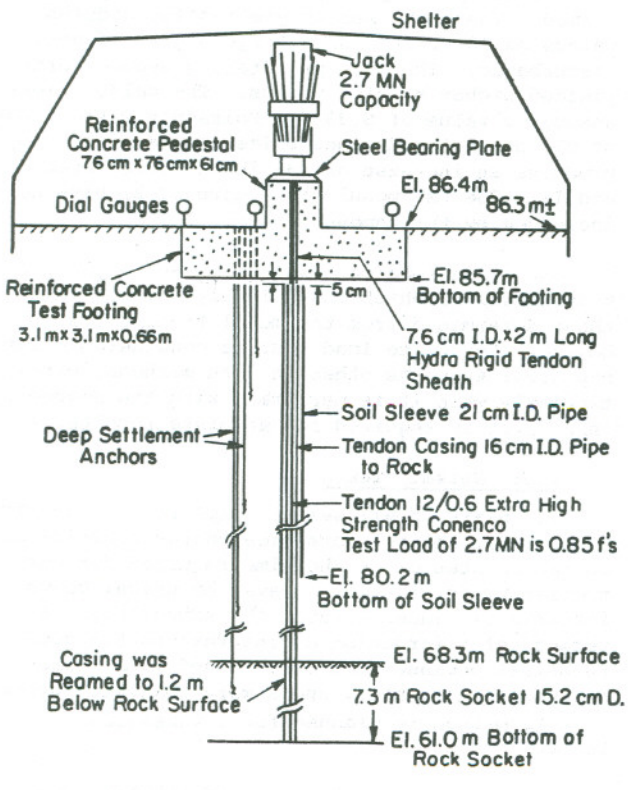


Fig. IV-20 Test arrangement for large scale plate load tests (from Shields and Bauer, 1975)

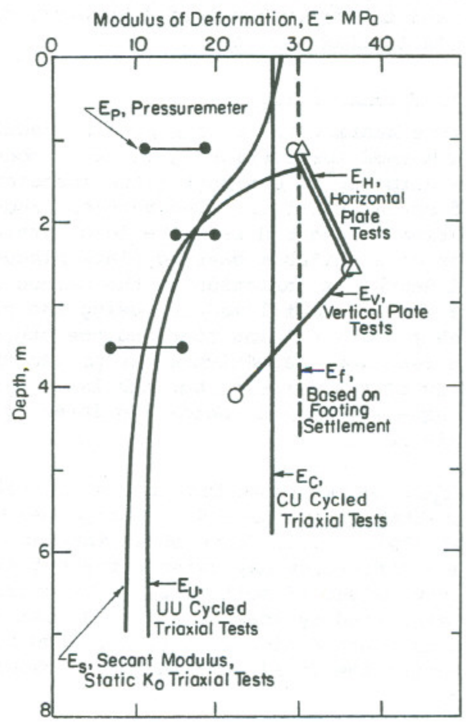


Fig. IV-21 Comparison of modulus values as determined by different techniques (from Shields and Bauer, 1975)

The study showed the crosshole method to be the most versatile and the most adaptable to techniques of signal enhancement. It should be noted, however, that under some conditions crosshole tests may not be possible, economical, or even reliable because of uncertainties in hole location and alignment. Seafloor testing is one situation wherein uphole-downhole tests may be more appropriate. It was also found by Ballard and McLean (1975) that the most common errors in the execution of vibroseismic methods lead to unconservative conclusions: i.e., velocities and moduli which are inaccurately **high**.

Corresponding field crosshole and laboratory resonant column test results have been compared by Anderson and Woods (1975). A schematic diagram of their crosshole set-up and triggering system is presented in Figure IV-22. A time dependent increase of the shear wave velocity, v_s, was observed in the laboratory and attributed to secondary consolidation. The measured variation of v_s with time is presented in Figure IV-23. For the deposits studied, 20 years corresponded to the time elapsed since the last major stress change due to excavation or construction. Values of v_s after 20 years of secondary consolidation were extrapolated from the lab data, and were found to be in agreement with the measured field values.

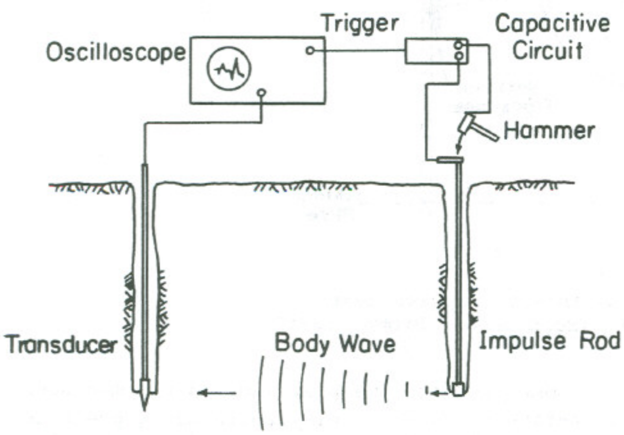


Fig. IV-22(a) Schematic of equipment used during cross-hole tests

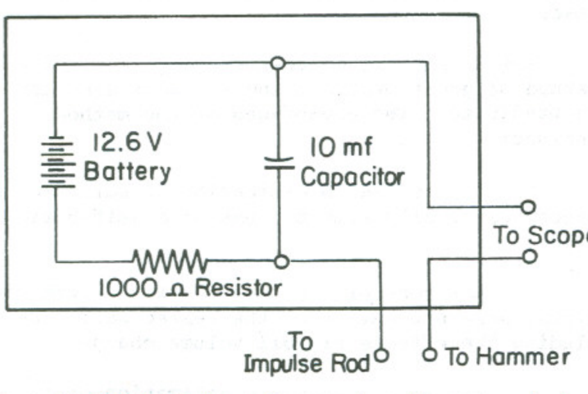


Fig. IV-22(b) Triggering system for cross-hole tests (from Anderson and Woods, 1975)

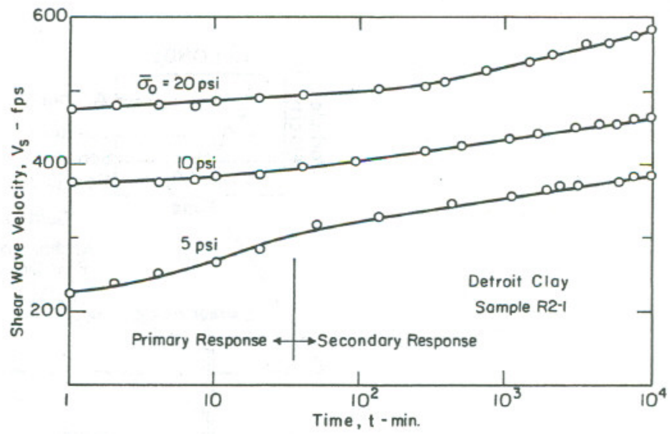


Fig. IV-23 Comparison of v_s with time of consolidation for Detroit clay (from Anderson and Woods, 1975)

The Hardin-Black (1969) empirical correlation of v_s with void ratio, OCR and p_o was also applied to the soil deposits, but yielded velocities which were generally higher than the field and laboratory velocities.

The in-situ impulse test shown schematically in Figure IV-24 (Miller et al., 1975) allows for the determination of shear modulus at varying shear strain levels (10^{-5} to 10^{-1}%). A plot of arrival time vs. distance of each sensor from the energy source allows evaluation of the shear wave velocity, v_s, vs. distance relationship. The transducer records give vertical particle velocity $\dot{v}$ vs. time. Shear strain γ can be calculated from:

$$\gamma = \frac{\dot{v}}{v_s} \tag{IV-28}$$

and the corresponding shear modulus G is given by

$$G = \rho v_s^{\,2} \tag{IV-29}$$

where ρ is the soil density.

I. Conclusions

The in-situ measurement of initial stresses and deformation characteristics is especially sensitive to the degree of soil disturbance caused by apparatus installation and testing procedure. Disturbance alters soil behavior by destroying natural structure or fabric and by modifying the system of stresses experienced by the soil.

To some extent, the effects of disturbance may be alleviated by the re-establishment of in-situ effective stresses, provided they are accurately known. Due to hysteresis effects, however, a reversible stress change will not result in a reversible change in initial deformation behavior. In many cases, the effects of changes in soil structure or fabric are irrecoverable. The most productive approach towards dealing with soil disturbance is therefore to try to avoid it in the first place.

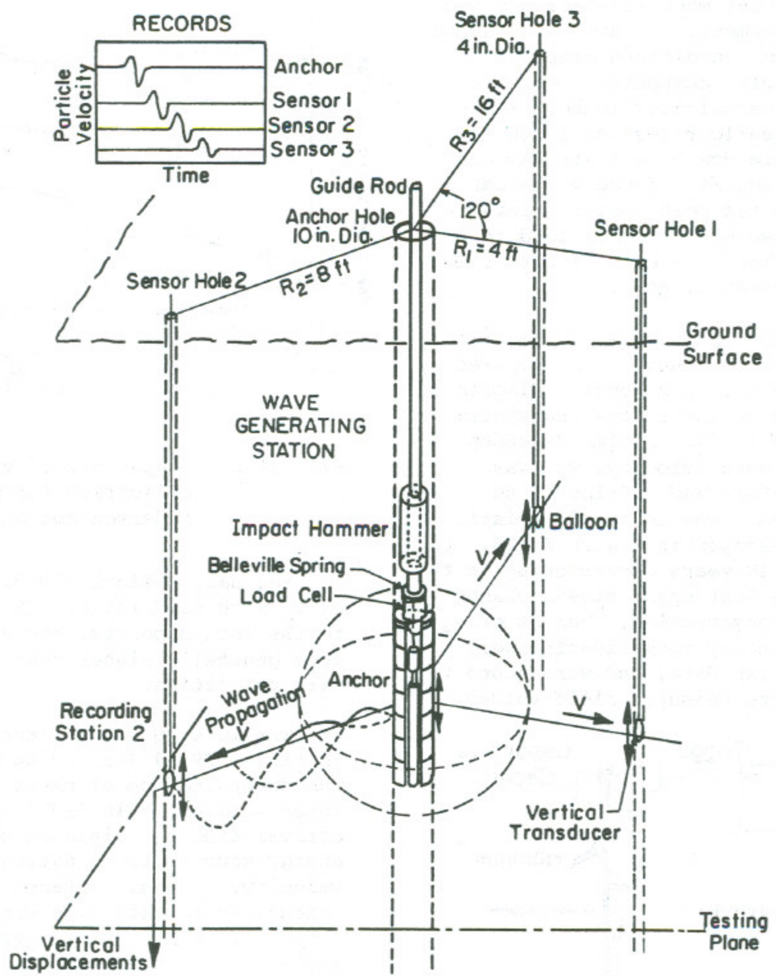


Fig. IV-24 Schematic representation of in-situ impulse test (from Miller, Troncoso and Brown, 1975)

With any test method, soil disturbance can be minimized only through the careful use of appropriate test procedures. Selection of a test procedure should involve a consideration of the particular soil type to be studied. The value of competent and experienced testing personnel is evident.

The self-boring method of advancing instruments into the ground is seen as being a very promising approach for minimizing soil disturbance. It is expected that increased experience with self boring devices in different soil types will help to define the effects of details of technique, such as cutting rate, instrument advance rate and drilling fluid pressure.

The accurate measurement of initial stresses in the ground is very difficult. Laboratory tests may provide useful bounds to the likely values of in-situ stresses, but the number of assumptions required precludes the accurate determination of in-situ stresses in the laboratory.

Some progress has been made in the measurement of in-situ stresses in clay soils using pressure cells, but there is a conspicuous lack of information on the measurement of stresses in sands, stiff clays and soft rock. The determination of in-situ lateral stress by means of the hydraulic fracturing method is not very reliable in many cases.

The pressuremeter test is seen as being a method of great promise, and its increased use is predicted. The advantages of the method include:

1) A potential minimization of soil disturbance, especially with the use of a self-boring device.

2) The possible derivation of the complete stress-strain behavior of the tested soil, including the effects of soil volume change.

3) The fairly accurate estimation of in-situ lateral stresses.

Vibroseismic methods of measuring shear and compression moduli are also of great value. The method avoids many of the effects of soil disturbance, and measures parameters which are representative of large volumes of soil. A disadvantage of the method is that the levels of induced strain are very small, and thus test results must be corrected before they are used in most predictions of ground deformation. Unlike the PMT, seismic methods do not yield estimates of soil strength.

The most accurate predictions of deformation behavior may be obtained from large scale field tests which closely simulate the prototype loading conditions. The expense of this approach, however, precludes its general use.

V. VOLUME CHANGE CHARACTERISTICS

A. Introduction

Of the four geotechnical property classes discussed in this report, soil volume change is probably the one least studied by means of in-situ measurements. The direct determination of in-situ soil volume change is difficult, and may require long time periods in the case of fine grained soils. In-situ measurement of volume change properties remains desirable, however, since difficulties of laboratory testing, such as sample disturbance and simulation of the in-situ environment, are partially avoided.

A number of empirical and analytical procedures have been developed to predict the rate and amount of soil volume changes in response to stress changes. These methods are summarized in the following section which is based on portions of the state of the art paper prepared by Mitchell and Gardner (1975).

B. Properties and Procedures

1. Properties

The specific properties that may be used to characterize the amount of soil compression and expansion which may be computed or estimated from the results of in-situ tests include the following:

C_c - One-dimensional compression index =

$$\frac{de}{d(\log_{10}p')}$$

where e is void ratio and p' is effective consolidation pressure

C_s - One dimensional swelling index =

$$\frac{de}{d(\log_{10}p')} \text{ on unloading}$$

C_r - One-dimensional recompression index =

$$\frac{de}{d(\log_{10}p')} \text{ on reloading}$$

a_v - Coefficient of compressibility = $-\frac{de}{dp'}$

m_v - Compressibility = $\frac{a_v}{1 + e_o}$

E_s - One-dimensional deformation (constrained) modulus $= \frac{1}{m_v}$

Δu - Change in pore water pressure

γ - Unit weight

w - Water content

D_R - Relative density

E' - Deformation modulus under drained conditions

μ' - Poisson's ratio under drained conditions.

p'_c - Effective preconsolidation pressure (maximum past effective pressure)

OCR - Overconsolidation ratio = p'_c/p'_o where p'_o is the present overburden effective stress.

The specific properties needed to describe the consolidation behavior (rate of volume change) that may be computed or estimated from the results of in-situ tests are the following:

k_h - Hydraulic conductivity (permeability) in the horizontal direction

k_v - Hydraulic conductivity in the vertical direction

c_{vv} - Coefficient of consolidation for vertical compression and vertical flow

c_{ch} - Coefficient of consolidation for vertical compression and horizontal flow.

2. Procedures

A variety of field tests are available that can yield data from which volume change properties can be deduced. These include:

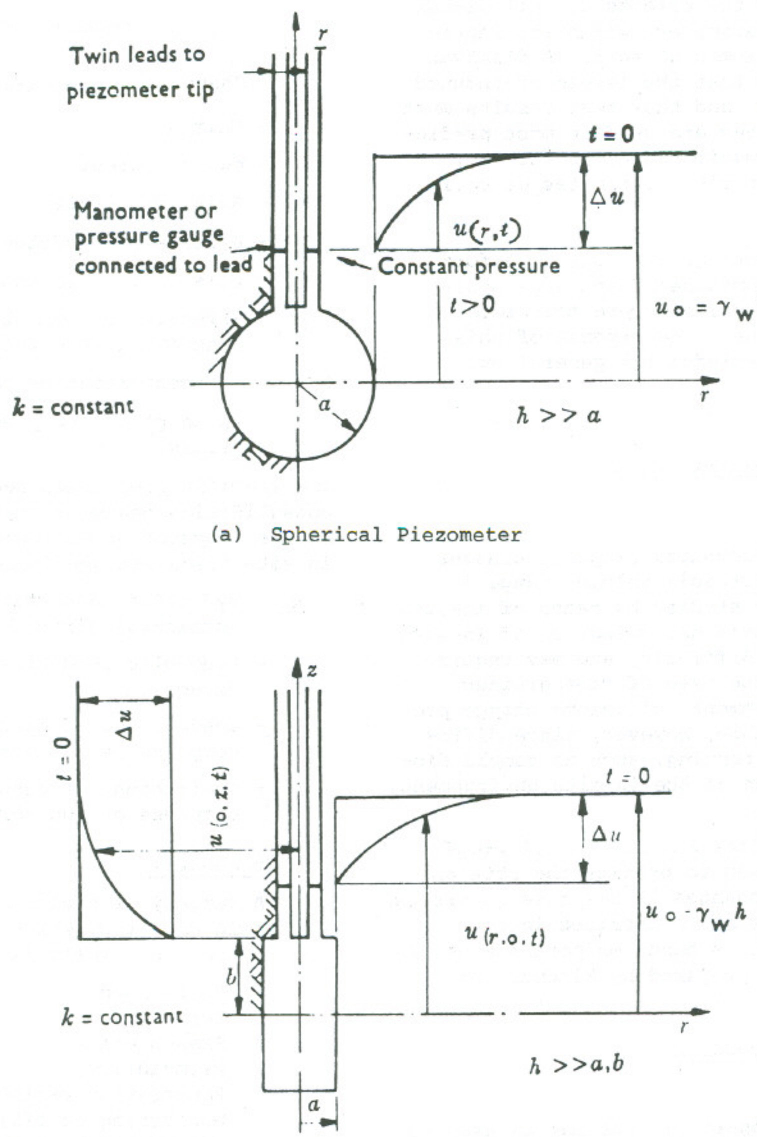
- Plate load
- Screw plate
- Pressuremeter
- Permeability
- Penetration resistance (dynamic and static)
- Monitoring of fills and excavations
- In-situ density and water content
- Special tests, including blasting, electro-osmosis, dielectric reponse properties, collapse measurements.

C. Consolidation Properties from Borehole or Piezometer Permeability Tests

Theory

If a hydrostatic head difference is applied between the water in a piezometer or borehole probe and the water in the surrounding compressible soil, consolidation or swelling will occur. If the applied pressure and rate of water flow into or out of the soil are measured, the permeability (hydraulic conductivity) and coefficients of consolidation or swelling may be calculated.

Spherical and cylindrical piezometer configurations are sketched in Figure V-1. A constant head difference, Δu, is maintained throughout the test. Variable head tests are also feasible, but their interpretation is more difficult.



(a) Spherical Piezometer

(b) Cylindrical Piezometer

Fig. V-1 Schematic representation of in-situ test for determination of consolidation properties from permeability measurement

For the spherical piezometer, the governing equation for consolidation or swell is:

$$c\left(\frac{\partial^2 u}{\partial r^2} + \frac{2}{r}\frac{\partial u}{\partial r}\right) = \frac{\partial u}{\partial t} \quad (r>a) \qquad \text{(V-1)}$$

where c is the coefficient of consolidation or swelling, u is hydrostatic excess pressure, r is radial distance and t is time.

The solution for the flow rate, Q, as a function of time is (Gibson, 1963)

$$Q(t) = 4\pi a\,\frac{k\Delta u}{\gamma_\omega}\left[1 + \frac{1}{\sqrt{\pi T}}\right] \qquad \text{(V-2)}$$

where: k = permeability

T = time factor $= \dfrac{ct}{a^2}$

The values of k and c may therefore be deduced from the intercept and slope of a plot of Q(t) vs. $1/\sqrt{t}$, in the manner illustrated in Figure V-2.

This solution has been extended by Wilkinson (1967, 1968) to consider the case of a cylindrical piezometer in anisotropic soil. The effects of a smear zone and the variation of k and m_v during testing are also considered. Large smear zones were shown to render the test invalid, but the influence of variation of k and m_v was found to be minor. The effect of soil anistropy is to impart a curved shape to the plot of Q(t) vs. $1/\sqrt{t}$.

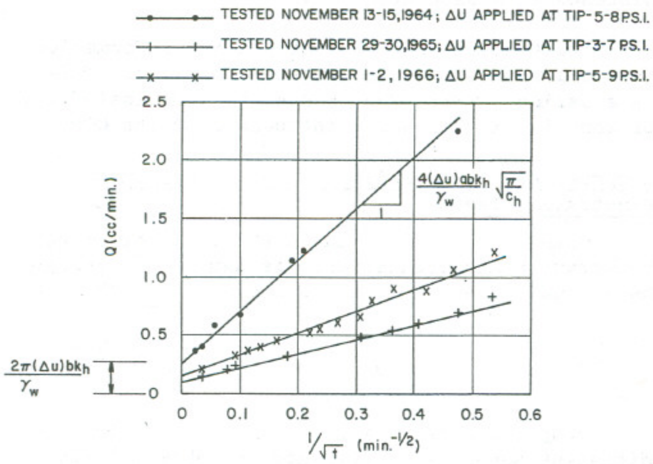


Fig. V-2 Constant head test results for foundation clay at fiddler's ferry ash lagoon embankment foundation illustrating basis for evaluation of permeability and coefficient of consolidation (data from Al-Dhahir et al., 1969)

For the cylindrical case:

$$c_h \left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r^2}\frac{\partial u}{\partial r}\right) + c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \qquad \text{(V-3)}$$

The solution for $T < 0.44$ is

$$Q(t) = ka\frac{\Delta u}{\gamma_\omega}\left[2\pi\ (N - 0.5) + \frac{4\sqrt{\pi}(N - 0.5)}{\sqrt{T}} + 12.6 + \frac{7.0}{\sqrt{T}}\right] \qquad \text{(V-4)}$$

$$\text{where:}\quad N = \frac{\text{length}}{\text{diameter}} = \frac{2b}{2a} \qquad \text{(V-5)}$$

For $T < 0.44$, water flow through the central section in Figure V-1 is purely radial in direction.

For a long permeameter, with a ratio, $N > 2$, water flow through the ends of the cylinder may be neglected, and the solution for the flow rate becomes:

$$Q(t) = \frac{2\pi\Delta u}{\gamma_\omega}\, b\, k_h\ \left(1 + \frac{2}{\sqrt{\pi T}}\right) \qquad \text{(V-6)}$$

$$T = \frac{c_h t}{a^2} \qquad \text{(V-7)}$$

If a short permeameter is used, the vertical permeability, k_v, may be computed according to a method prescribed by Jezequel and Mieussens (1975):

1. Obtain c_h and k_h using a long permeameter and Equations (V-6), (V-7)
2. Normalize short permeameter data, using:

$$QR = Q\frac{\gamma_\omega}{2\pi\ (\Delta u)\ b\ k_h} \qquad \text{(V-8)}$$

3. Enter Figure V-3 with QR and T to obtain $\overline{RK}$
4. $k_v = k_h/\overline{RK}$ (V-9)

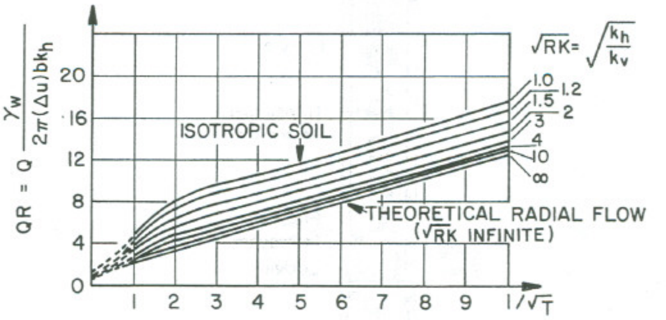


Fig. V-3 Normalized flow rate as a function of time factor for a short permeameter ($b/a = 0.2$) (from Jezequel and Mieussens, 1975)

Application of Methods

The methods described above have been applied to field tests in embankment soils as reported by Bishop and Al-Dhahir (1969). The field permeability values were in agreement with both the full scale field performance and the laboratory values. Field tests predicted values of c_v which were substantially less or greater than field performance values in some cases. The discrepancies in c_v were attributed to probable stress history effects.

Additional descriptions of the application of these methods are provided by Al-Dhahir and Morgenstern (1969), Al-Dhahir et al. (1969) and Jezequel and Mieussens (1975).

Discussion

The available evidence indicates that the values of k and c determined in the field are generally greater than the values predicted from laboratory tests.

The testing time required for a pressure increment in field testing is comparable to that of laboratory testing. The field testing time may, potentially, be greatly reduced through the use of a piezometer probe such as described by Wissa et al. (1975) and shown in Figure V-4 of this report.

Differences in the values of c calculated from field, laboratory and full scale performance measurements are almost unavoidable as the result of differences in flow direction, stress system and disturbance.

Borehole and piezometer permeability tests do not give soil compressibility, m_v, directly but it is possible to estimate m_v using the values of k and c:

$$m_v = \frac{k}{c\gamma_\omega} \qquad \text{(V-10)}$$

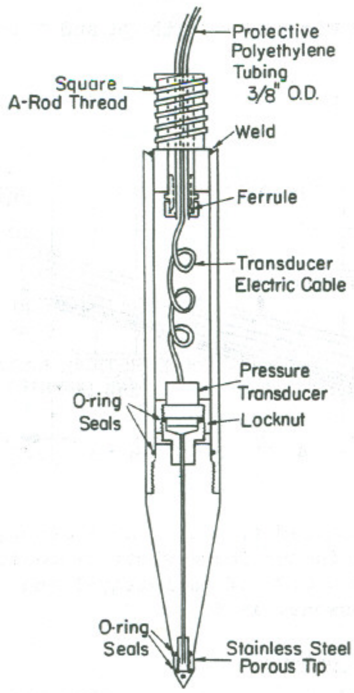


Fig. V-4 Schematic of the piezometer probe (from Wissa et al., 1975)

An approach for the evaluation of ċ that has given good results is to compute it by means of equation (V-10) using m_v determined from laboratory tests and k from field measurements.

D. Penetration Resistance as a Measure of Soil Compressibility

General Considerations

As noted in an earlier section the static or dynamic penetration resistance of a soil is a complex function of soil deformation, strength and compressibility characteristics. Analytical solutions for penetration resistance as a function of compressibility are not yet available, but a number of empirical correlations have been proposed.

Factors such as pore pressure behavior and possible grain crushing must be considered when comparing compressibility and penetration resistance. Contact stresses greater than about 100 kgf/cm^2 may cause crushing of soil grains, with susceptibility to crushing dependent on mineralogy, gradation, particle size and shape, void ratio and in-situ stress.

A wide variety of penetrometers are in current use, but this discussion will be limited to the SPT and Dutch Cone methods. The methods to be described apply only to dry or completely saturated soils.

Penetration resistance usually decreases when the ground water table is crossed, but the effects of pore pressure generation must be considered for individual tests. Soil dilation will induce negative pore pressures and increase penetration resistance in a saturated soil.

Penetration tests do not provide information on the preconsolidation pressure in sands. This is a serious limitation, since the compressibility of sand is significantly influenced by its OCR.

Compressibility as Indicated by the Standard Penetration Test

Compressibility based on the value of Standard Penetration Resistance, SPR, is usually expressed as an equivalent constrained modulus, E_s, that is,

$$E_s = \frac{1}{m_v} = \frac{1 + e}{a_v} \qquad \text{(V-11)}$$

Proposed correlations between E_s and SPR for different sands are summarized in Table V-1 and Figure V-5. A wide degree of variation is evident. The correlations are of little use unless it can be established which correlation, if any, holds true for a particular sand in question.

Correlations between SPR and the compressibility of cohesive soils have not been well-defined.

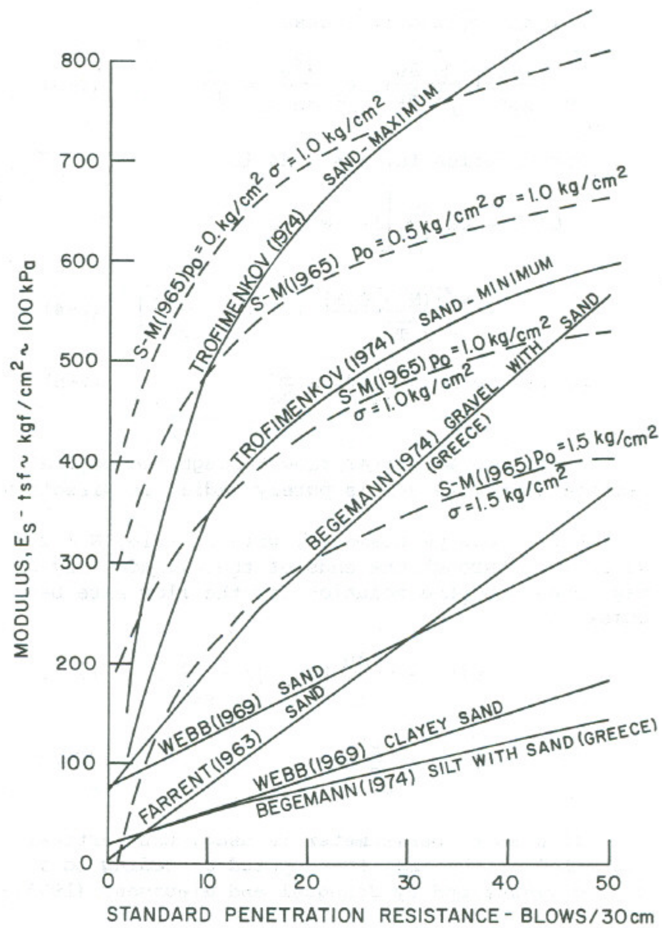


Fig. V-5 Relationships for compressibility modulus as a function of standard penetration resistance (from Mitchell and Gardner, 1975)

Table V-1

Compressibility as Indicated by Standard Penetration Resistance (Mitchell and Gardner, 1975)

Reference	Relationship	Soil Types	Basis	Remarks
Schultze and Melzer (1965)	$E_s = \nu\sigma^{0.522}$ kg/cm^2 $\nu = 246.2 \log N - 263.4\, p_o + 375.6 \pm 57.6$ $0 < p_o < 1.2$ kg/cm^2 p_o = effective overburden pressure	Dry sand	Penetration tests in field and in test shaft. Compressibility based on e, $e_{max.}$ and $e_{min.}$ (Schultze and Moussa, 1961)	Correlation coefficient = 0.730 for 77 tests
Webb (1969)	$E_s = 5(N+15)$ tons/ft^2 $E_s = 10/3(N+5)$ tons/ft^2	Sand Clayey sand	Screw Plate Tests	Below water table
Farrent (1963)	$E_s = 7.5\ (1-\mu^2)N$ tons/ft^2 μ = Poisson's ratio	Sand	Terzaghi and Peck loading settlement curves	
Begemann (1974)	$E_s = 40 + C(N-6)$ kg/cm^2 $N > 15$ $E_s = C(N+6)$ kg/cm^2 $N < 15$ C = 3(silt with sand) to 12(gravel with sand)	Silt with sand to gravel with sand		Used in Greece
Trofimenkov (1974)	$E = (350$ to $500) \log N$ kg/cm^2	Sand		USSR practice
Meyerhof (1974)	$S = p\sqrt{B}/2N$ inches $S = p\sqrt{B}/N$ inches p in tons/ft^2, B in inches	Sand and gravel Silty sand	Analysis of field data of Schultze and Sherif (1973)	Conservative estimate of maximum settlement of shallow foundations

Note: N is penetration resistance in blows per 30 cm. (blows/ft.)

Compressibility as Indicated by Static Cone Resistance

Many correlations between static cone resistance, q_c, and compressibility have been proposed. A number of them are listed in Table V-2. Correlations have been developed using both the constant of compressibility, C, and the compression index, C_c. For one dimensional compression:

$$\frac{\Delta h}{h} = -\frac{1}{C} \ln\left(1 + \frac{\Delta\sigma'}{\sigma'}\right) \qquad \text{(V-12)}$$

and,

$$\Delta e = -C_c \log\left(1 + \frac{\Delta\sigma'}{\sigma'}\right) \qquad \text{(V-13)}$$

thus, C and C_c are related:

$$C = 2.3\,\frac{(1+e)}{C_c} \qquad \text{(V-14)}$$

The parameter, α, may be used to relate the cone resistance, q_c, with C_c:

$$\alpha = \frac{2.3\ (1+e)}{C_c}\ \frac{\sigma'_o}{q_c} \qquad \text{(V-15)}$$

where

σ'_o = effective overburden stress.

The compressibility, m_v, may be expressed as:

$$E_s = \frac{1}{m_v} = \alpha q_c \qquad \text{(V-16)}$$

Recommended values of α range from 1.5 to 8 for normally consolidated soils, with higher values associated with softer more compressible soils. A consistent value of α may be exhibited by a given deposit, as illustrated in Figure V-6.

A general correlation proposed by Sanglerat et al. (1972) is presented in Figure V-7. In fact, for a given sand, α is not constant. The relationship

$$\alpha = 2\left[1 + \left(\frac{D_r}{100}\right)^2\right] \qquad \text{(V-17)}$$

has been proposed by Vesic (1970) for Ogeechee River sand.

In general, it is not clear whether α will increase or decrease with q_c. Conflicting results exist and have not yet been explained (Mitchell and Gardner, 1975).

It is evident that since α may vary by $\pm 100\%$, predictions of volume change based on q_c are rough estimates at best.

Table V-2

Compressibility as Indicated by Static Cone Resistance (Mitchell and Gardner, 1975)

Reference	Relationship	Soil Types	Remarks
Buisman (1940)	$E_s = 1.5\ q_c$	Sands	Overpredicts settlements by a factor of about two
Trofimenkov (1964)	$E_s = 2.5\ q_c$	Sand	Lower limit
	$E_s = 100 + 5\ q_c$		Average
De Beer (1967)	$E_s = 1.5\ q_c$	Sand	Overpredicts settlements by a factor of two
Schultze and Melzer (1965)	$E_s = \frac{1}{m_v} \nu\sigma^{0.522}$	Dry sand	Based on field and lab penetration tests compressibility based on e, e_{max} and e_{min}
	$\nu = 301.1 \log q_c - 382.3\ p_o + 60.3 \pm 50.3$		Correlation coefficient = 0.778 for 90 tests valid for p_o = 0 to 0.8 kg/cm^2
Bachelier and Parez (1965)	$E_s = \alpha q_c$		
	$\alpha = 0.8\text{–}0.9$	Pure sand	
	$\alpha = 1.3\text{–}1.9$	Silty sand	
	$\alpha = 3.8\text{–}5.7$	Clayey sand	
	$\alpha = 7.7$	Soft clay	
De Beer (1967)	$A = C \frac{A_{oed}}{C_{oed}}$	Overconsolidated sand	C from field tests A_{oed} and C_{oed} from lab oedometer tests $C_{oed} = 2.3 \frac{(1+e)}{C_c}$ $A_{oed} = 2.3 \frac{(1+e)}{C_s}$
Thomas (1968)	$E_s = \alpha q_c$ $\alpha = 3 - 12$	3 sands	Based on penetration and compression tests in large chambers Lower values of α at higher values of q_c; attributed to grain crushing
Webb (1969)	$E_s = \frac{5}{2}(q_c + 30)$ tsf	Sand below water table	Based on screw plate tests Correlated well with settlement of oil tanks
	$E_s = \frac{5}{3}(q_c + 15)$ psf	Clayey sand below water table	

Table V-2 (continued)

Reference	Relationship		Soil Types	Remarks
Meigh and Corbett (1969)	$E_s = \frac{1}{m_v} = \alpha q_c$		Soft silty clay	See Fig. 2 and text
Vesić (1970)	$E_s = 2(1+D_R^2)q_c$ D_R = relative density		Sand	Based on pile load tests and assumptions concerning state of stress
Schmertmann (1970)	$E_s = 2q_c$		Sand	Based on screw plate tests $\Delta\sigma$ = 2 tsf
Gielly et al. (1969) Sanglerat et al. (1972)	$E_s = \alpha q_c$			Based on 600 comparisons between field penetration and lab oedometer tests
	q_c < 7 bars	$3 < \alpha < 8$	Clays of low plasticity (CL)	
	7 < q_c < 20 bars	$2 < \alpha < 5$		
	q_c > 20 bars	$1 < \alpha < 2.5$		
	q_c > 20 bars	$3 < \alpha < 6$	Silts of low plasticity (ML)	
	q_c < 20 bars	$1 < \alpha < 3$		
	q_c < 20 bars	$2 < \alpha < 6$	Highly plastic silts and Clays (MH,CH)	
	q_c < 12 bars	$2 < \alpha < 8$	Organic silts (OL)	
	q_c < 7 bars:			
	50 < w < 100	$1.5 < \alpha < 4$	Peat and organic clay (Pt, OH)	
	100 < w < 200	$1 < \alpha < 1.5$		
	w > 200	$0.4 < \alpha < 1$		
	20 < q_c < 30 bars	$2 < \alpha < 4$	Gravel	
	q_c > 30 bars	$1.5 < \alpha < 3$		
	q_c < 50 bars	$\alpha = 2$	Sand	
	q_c > 100 bars	$\alpha = 1.5$		
	q_c > 12 bars, w < 30%	$C_c < 0.2$		See Fig. 9
	q_c < 12 bars, w < 25%	$C_c < 0.2$		
	25 < w < 40%	$0.2 < C_c < 0.3$		
	40 < w < 100%	$0.3 < C_c < 0.7$		
	q_c < 7 bars, 100 < w < 130%	$0.7 < C_c < 1$		
	w > 130	$C_c > 1$		

Table V-2 (continued)

Reference	Relationship	Soil Types	Remarks
Bogdanović (1973)	$E_s = \alpha q_c$		Based on analysis of silo settlements over a period of 10 years
	$q_c > 40$ kg/cm^2 $\quad \alpha = 1.5$	Sands, sandy gravels	
	$20 < q_c < 40$ $\quad \alpha = 1.5 - 1.8$	Silty saturated sands	
	$10 < q_c < 20$ $\quad \alpha = 1.8 - 2.5$ $5 < q_c < 10$ $\quad \alpha = 2.5 - 3.0$	Clayey silts with silty sand and silty saturated sands with silt	
Schmertmann (1974a)	$E_s = 2.5\ q_c$	NC sands	L/B = 1 to 2 axisymmetric
	$E_s = 3.5\ q_c$	NC sands	L/B $\geq$ 10 plane strain
De Beer (1974b)	$C > \frac{3}{2} \frac{q_c}{\sigma_o}$	NC sands	Belgian practice
	$A > \varepsilon \frac{3}{2} \frac{q_c}{\sigma_o}$	OC sands	$3 < \varepsilon < 10$, Belgian practice
	$E_s = 1.6\ q_c - 8$	Sand	Bulgarian practice
	$E_s = 1.5\ q_c\ ,\ q_c > 30$ kg/cm^2 $E_s = 3\ q_c\ ,\ q_c < 30$ kg/cm^2	Sand	Greek practice
	$E_s > 3/2\ q_c$ or $E_s = 2\ q_c$	Sand	Italian practice
	$E_s = 1.9\ q_c$	Sand	South African practice
	$E_s = \frac{5}{2}(q_c + 3200)$ kN/m^2	Fine to medium sand	
	$E_s = \frac{5}{3}(q_c + 1600)$ kN/m^2	Clayey sands, PI < 15%	
	$E_s = \alpha q_c,\ 1.5 < \alpha < 2$	Sands	U.K. practice
Trofimenkov (1974)	$E_s = 3\ q_c$	Sands	U.S.S.R. practice
	$E_s = 7\ q_c$	Clays	
Meyerhof (1974)	$S = pB/2\ q_c$ in consistent units S = settlement	Cohesionless soil	Conservative estimate, based on analysis of vertical strain
Alperstein and Leifer (1975)	$E = (11 - 22)\ q_c$	Overconsolidated sand	E_s determined by lab tests on reconstituted samples of sand
Dahlberg (1974)	$E = \alpha q_c$ $1 < \alpha < 4$	NC and OC sand	E_s back-calculated from screw plate settlement using Buisman-DeBeer and Schmertmann methods; α increases with increasing q_c; see text

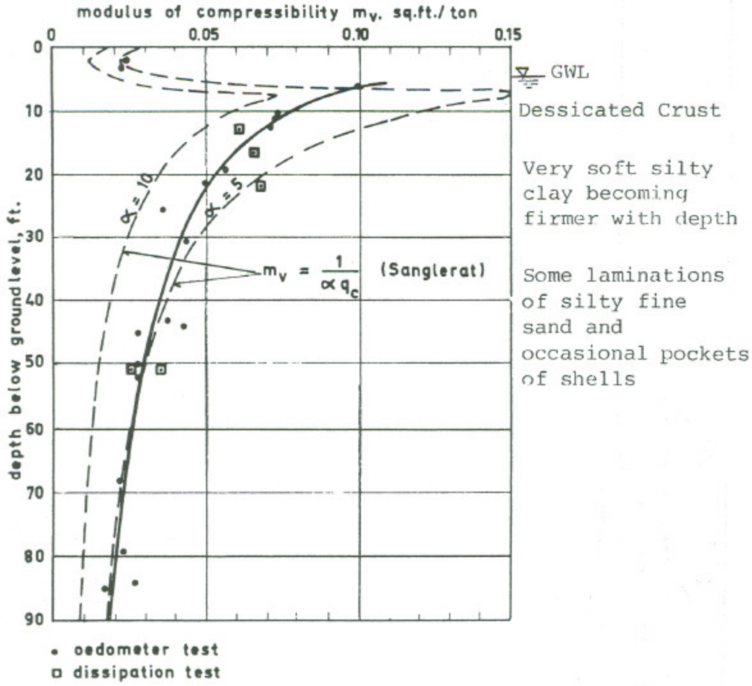


Fig. V-6 Comparison between compressibility of a soft silty clay as measured by oedometer tests and based on cone resistance (from Meigh and Corbett, 1969)

Overconsolidation Ratio and Maximum Past Pressure from Static Cone Resistance

A knowledge of the maximum past pressure at points in a soil deposit is needed for many analyses. A determination of this quantity by means of the static cone penetration test alone is not possible in cohesionless soils. An estimate of the OCR in clays may be obtained through the following method suggested by Schmertmann (1974a):

1) estimate s_u from q_c

2) estimate p_o' from profile information

3) compute s_u/p_o'

4) refer to the relationships between s_u/p_o' and OCR for different clays, as published for example by Ladd and Foott (1974).

Alternatively, a plot of q_c vs. depth may be extrapolated above the ground surface to where $q_c = 0$. The distance between this point and the existing ground surface corresponds to the depth of soil removed, assuming a constant unit weight throughout the profile.

E. Load Bearing Tests for the Determination of Compressibility Properties

The load bearing test, LBT, represents the first known application of in-situ testing to investigate soil compressibility. Factors of cost and testing time have increased the relative advantages of other in-situ methods, but the LBT remains a valuable test method, suited particularly to the testing of uncontrolled fills and stoney soils.

Basis of Load Bearing Test Interpretation

The load vs. settlement curve derived from the static LBT is usually interpreted to deduce the allowable bearing pressure, or more fundamentally, the moduli applicable to volume change prediction. Only the derivation of E_s will be considered in this report.

The approaches used for the evaluation of E_s from the LBT generally fall into three classifications:

1) Elastic Solutions are based on settlement of test plate, soil deformation at different depths, and deformations at surface points located nearby.

2) Statistical Methods use relationships between the settlement of bearing plates and large scale footings.

3) Finite Element Analyses consider both elastic and inelastic material response, employing the same measurements cited in 1).

Interpretation from Elastic Solutions

Foundation materials may be idealized as an isotropic, elastic half space. For a rigid LBT plate:

Fig. V-7 Correlation between compression index and cone resistance (from Sanglerat et al., 1972)

$$E_s = I_o \, I_1 \, k \, (1 - \mu^2) \, D \qquad \text{(V-18)}$$

where:

D = diameter of test plate

I_1, I_o = influence factors for surface loading and loading at depth

k = coefficient of subgrade reaction

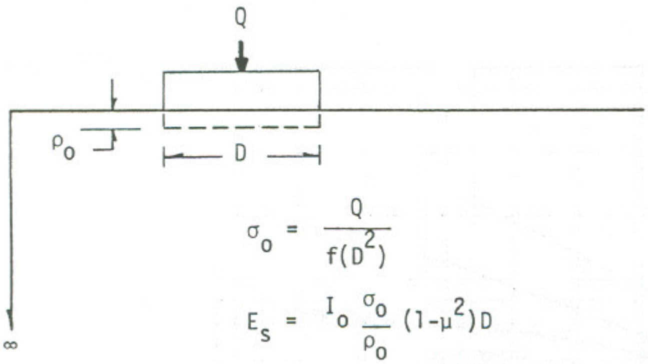
μ = Poisson's ratio

The simplest case is that of a rigid LBT plate located at the ground surface. The appropriate influence factors and the solution for this case are presented in Figure V-8. The influence factors for a number of other cases are given by Mitchell and Gardner (1975).

Interpretation from Statistical Relationships

The best known statistical relationship between settlement and footing size for a given bearing pressure is that proposed by Terzaghi and Peck (1948) for sands. Based on a statistical analysis of LBT results on footings and test plates, the settlement of a footing or plate of diameter D has been related to that of a smaller footing or plate of diameter D_1 by the following equation:

$$\frac{\rho}{\rho_1} = \left(\frac{2D}{D + D_1}\right)^2 \qquad \text{(V-19)}$$



$$\sigma_o = \frac{Q}{f(D^2)}$$

$$E_s = I_o \frac{\sigma_o}{\rho_o} (1-\mu^2) D$$

RIGID PLATE SHAPE	INFLUENCE FACTOR (I_o)
Circular	0.785
Square	0.815

Fig. V-8 Influence factors for LBT on surface

By recalling Equation (V-18), the ratio of the "equivalent elastic moduli," $\overline{E}_s/\overline{E}_{s1}$, may be expressed as follows:

$$\frac{\overline{E}_s}{\overline{E}_{s1}} = \left(\frac{D + D_1}{2D}\right)^2 \frac{D}{D_1} \qquad \text{(V-20)}$$

where: $\overline{E}_{s1}$ = elastic modulus determined from LBT, (through an equation of the form of Equation (V-18))

$\overline{E}_s$ = elastic modulus appropriate for a footing of breadth or diameter D

D_1, D = breadth or diameter of LBT or footing respectively.

For a LBT with D_1 equal to unity, Equation (V-20) may be simplified:

$$\overline{E}_s = D \left(\frac{D + 1}{2D}\right)^2 \overline{E}_{s1} \qquad \text{(V-21)}$$

An independent analysis of LBT data by Bjerrum and Eggestad (1963) has indicated that a single relationship, such as Equation (V-21), is not appropriate for sands at various relative densities. The wide range of observed settlement ratios, ρ/ρ_1, for different relative densities and footing sizes is illustrated in Figure V-9. The Terzaghi-Peck correlation is also sketched in the figure, and is seen to underestimate predictions of settlement in a number of cases.

An amended version of Equation (V-21) has been proposed by Bazaraa (1967):

$$\overline{E}_s = D \left(\frac{D + 1.5}{2.5\,D}\right)^2 \overline{D}_{s1} \qquad \text{(V-22)}$$

where: D is measured in feet.

A plot of settlement ratio vs. diameter which corresponds to that predicted by Equation (V-22) has been superimposed on the data in Figure V-9. The correlation is seen to be somewhat better than Equation (V-19), but one is persuaded to concur with Bjerrum and Eggestad in concluding that a unique relationship independent of relative density, or other parameters, does not exist.

Lambe and Whitman (1969) have noted that a LBT will be most successful if soil disturbance below the plate is minimized, and if the soil in question is homogeneous for a depth which is large relative to the size of the actual footing. A soil profile which will yield misleading LBT results is depicted in Figure V-10. The settlement of the test plate will be due primarily to strains in soil A, while the footing settlement will be due primarily to strains in soil B. If the two soils have different volume change characteristics, the settlement behavior of the plate and footing may be considerably different.

Interpretation of LBT from Finite Element Analyses

The finite element method (FEM) of analysis is readily adaptable to the interpretation of volume change parameters in the LBT. Complex test geometries, as well as non-homogeneous linearly elastic or non-elastic behavior may be accommodated.

Carrier and Christian (1973) have presented comprehensive FEM solutions for the settlement of a rigid circular plate on an isotropic linearly elastic and on a non-homogeneous linearly elastic half space. The solutions were based upon the three following variations of deformation modulus, E_s, with depth, z:

Case #1 $E_s = E_o$

Case #2 $E_s = n_v z$

Case #3 $E_s = E_o + n_v z$

The closed form isotropic elastic solutions of Case #1 and the non-homogeneous elastic solution for case #2 as presented by Gibson (1967) are in agreement with the FEM solutions. No solutions are available for comparison with the FEM solutions for Case #3.

F. Compressibility Characteristics from Screw Plate Tests

A flat pitch auger device of the type depicted in Figure V-11 is used in the screw plate test--termed a field compressometer by Janbu and Senneset (1973)-to observe the load-settlement behavior of soil at depth. The auger is screwed to the desired depth in the soil, increments of pressure are applied to the hydraulic piston, and the settlement of the screw plate is observed. The horizontally projected area over the single 360° auger flight is taken as the loading plate area.

Screw plate tests are well suited for use in sandy soils where undisturbed sampling is difficult. They are less suited for in-situ tests on clays except for estimating immediate deformations, because of the long consolidation time that would be required to define drained deformations.

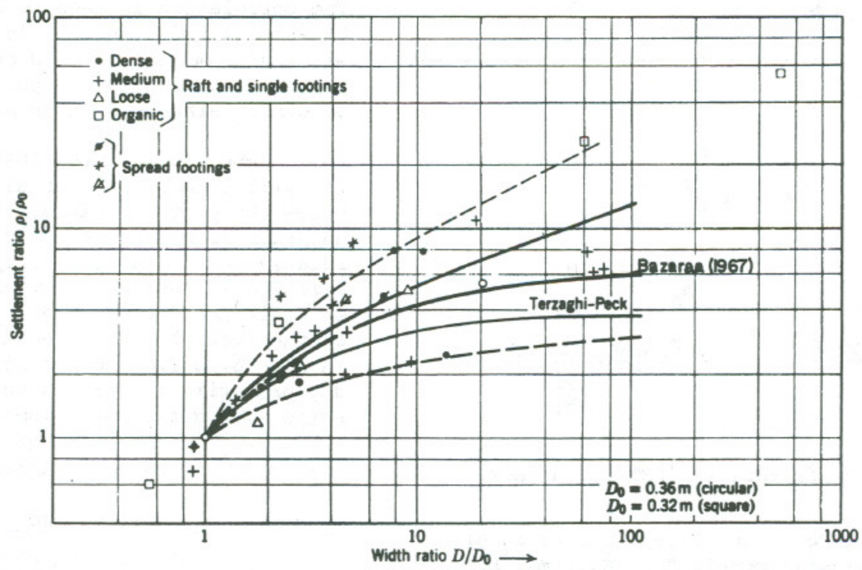


Fig. V-9 Comparison between settlement and dimension of loaded area (from Bjerrum and Eggestad, 1963)

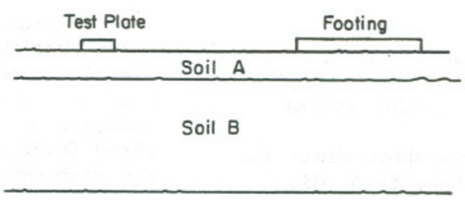


Fig. V-10 A soil profile which will yield misleading load bearing test results

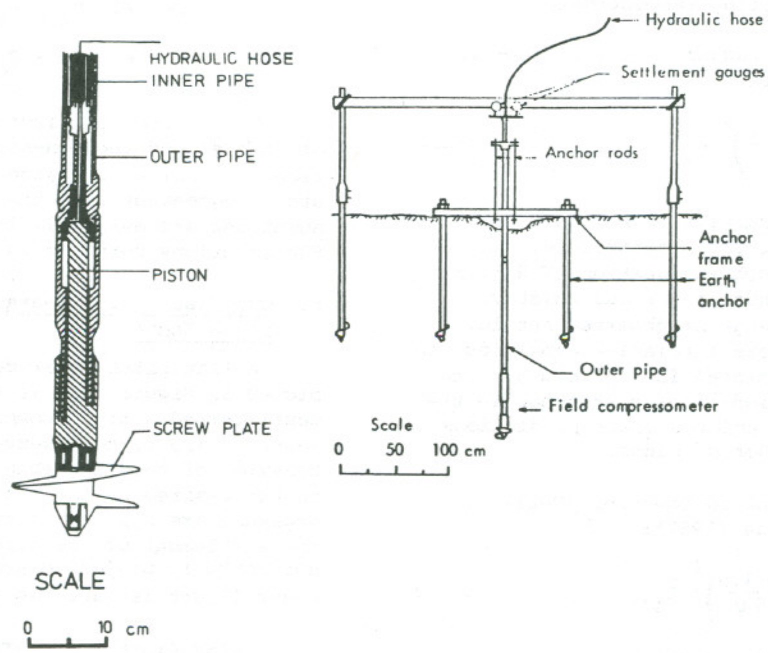


Fig. V-11 Screw plate test as developed in Norway (from Janbu and Senneset, 1973)

The modulus for one-dimensional compressibility, E_s, may be calculated from screw plate test results by employing a relationship proposed by Schmertmann (1970):

$$\rho = C_1 C_2 \Delta p \sum_{0}^{2B} \left(\frac{I_z}{E_s}\right)\Delta z \qquad \text{(V-23)}$$

where: $C_1 = 1 - 0.5 \left(\frac{p_o}{\Delta p}\right)$ (embedment correction)

$C_2 = 1 + 0.2 \log \left(\frac{t \text{ years}}{0.1}\right)$ (creep correction)

B = least dimension of rectangular footing or diameter of bearing plate.

I_z = settlement influence factor

Δ_p = increment in stress

The summation form of Equation (V-23) enables soil layering to be accounted for in the analysis.

Another procedure for determining E_s is described by Janbu and Senneset (1973). The modulus is related to a derived modulus number, m, as follows:

$$E_s = m\, p_a \left(\frac{\sigma'}{p_a}\right)^{1-a} \qquad \text{(V-24)}$$

where: a = 1 for OC clays and undrained loading of soft saturated clays

= 1/2 for many sands and silts

= 1 for NC fine grained soils

p_a = reference stress = 10 ton/m^2
1 atmosphere

Corresponding values of screw plate pressure and settlement are used as a basis for evaluation of m, following procedures given by Janbu and Senneset (1973) and summarized by Mitchell and Gardner (1975).

The coefficient of consolidation may also be deduced from the time settlement record obtained during a screw plate test. The method is illustrated in Figure V-12. Drainage during the screw plate test is essentially radial so the parameter measured is c_{vh}. The method has yielded results that compare well with results of oedometer tests as illustrated in Figure V-13.

The preconsolidation pressure may also be obtained from the screw plate test in the manner illustrated in Figure V-14.

G. The Evaluation of Volume Change Properties with the PMT

The procedure and apparatus of the pressuremeter test have been described in earlier sections. For an infinitely thick elastic cylinder corresponding to the tested soil in a PMT, it may be shown that:

$$E_s = \frac{2V_o\,(1 + \mu)\,\Delta\psi}{\Delta V} \qquad \text{(V-25)}$$

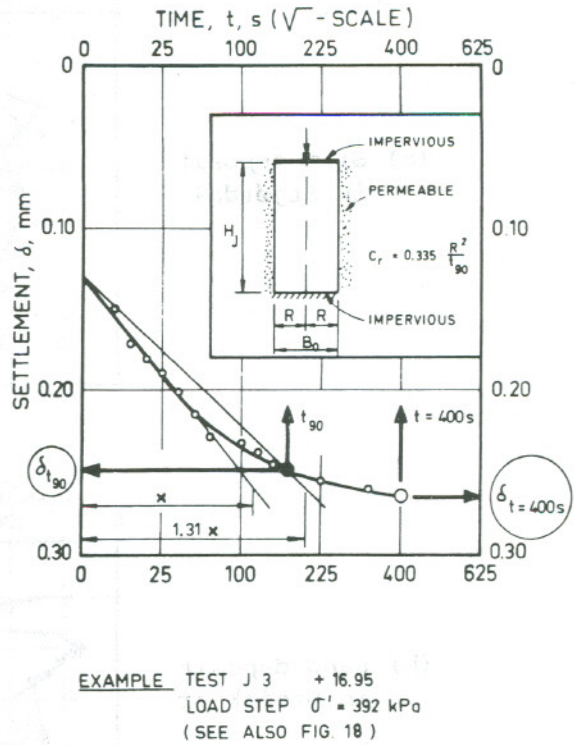


Fig. V-12 Determination of c_{vh} by screw plate test. Data shown are for a field test in a sand near Stockholm, Sweden $\Delta\sigma'$ = 392 kPa (from Dahlberg, 1974)

The terms of Equation (V-25) have been defined in Section (IV). The significance of the computed E_s depends upon the point of the test curve being considered. An idealized pressure-volume curve from a PMT is presented in Figure V-15. The moduli which are relevant to each phase of the test are indicated in the figure.

Time limitations and creep effects limit the usefulness of this method in clays for the measurement of volume change properties. The method has proved successful in the interpretation of pressuremeter tests in soft rock.

Windle and Wroth (1975) have suggested an electrical resistivity method for measuring soil volume change during a partially drained PMT. The method has been described in Section (IV-E). The basis of the test interpretation is a correlation which has been observed between soil resistivity and porosity:

$$\text{resistivity, } \rho = a\rho_w\, n^{-m} \qquad \text{(V-26)}$$

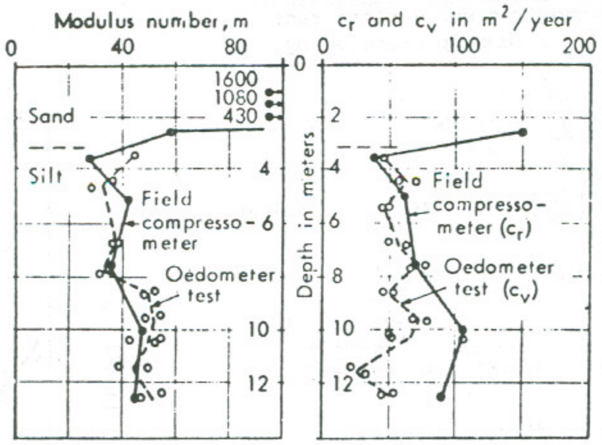
where: a = 1.0

n = porosity

m = 1.5 to 2.0 for marine soils

ρ_w = resistivity of pore water

(a) Silt deposit in Stjørdal



(b) Sand deposit in Steinkjer

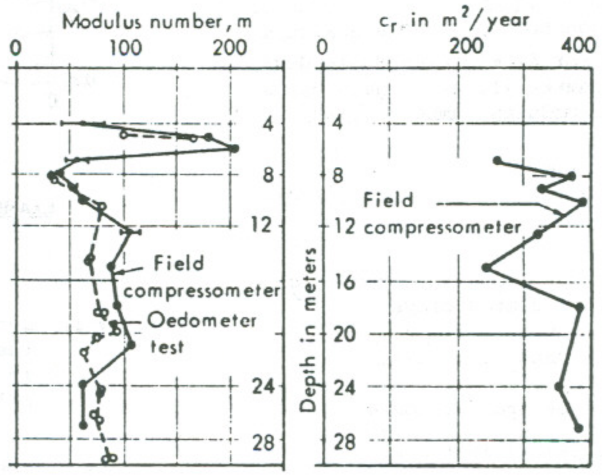


Fig. V-13 Comparisons between modulus numbers and coefficients of consolidation determined by screw plate tests and by oedometer tests (from Janbu and Senneset, 1973)

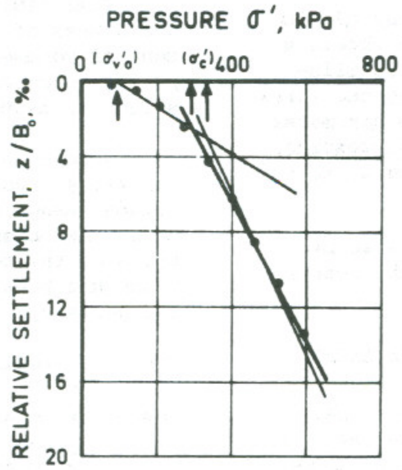


Fig. V-14 Evaluation of the preconsolidation pressure, σ'_c, from the results of a screw plate test

(from Dalberg, 1974)

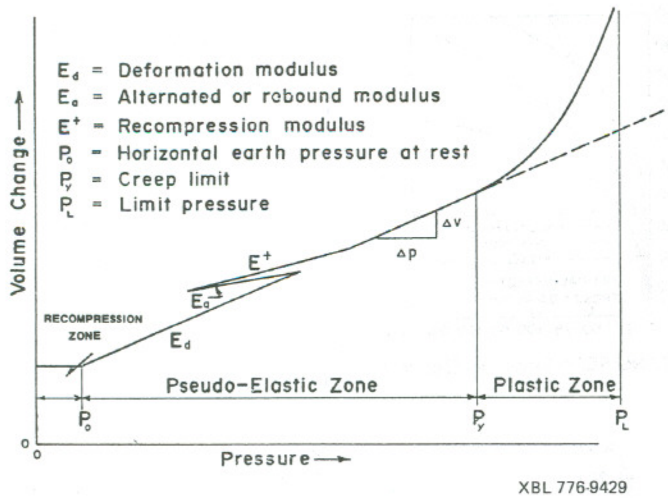


Fig. V-15 Idealized pressuremeter test curve indicating soil moduli which are applicable to each phase of testing

H. Monitoring of Earth and Earth Supported Structures

Monitoring during construction and after construction should be considered as much a part of in-situ measurement as is testing strictly for initial design purposes. Data gathered from an instrumentation program both shapes the design of future facilities and influences the construction rate and sequence of the facility under study. In a defensive design format, monitoring becomes an integral part of the design process.

Good quality field measurements are the first requirements of a "back calculation" procedure. The most commonly measured quantities are vertical or horizontal deformation, porewater pressure, and less frequently, soil pressure or load. A comprehensive description of field instrumentation is given by Cooling (1962) and more recent innovations are described by Dunnicliff (1971) and Selig (1974).

The proper evaluation of effective stresses and the influences of soil creep behavior constitute the largest obstacles to the implementation of back analyses. Nonetheless, if the appropriate data are available, a number of theories may be applied to deduce values of c_v, E_s, μ, C_s, C_r and C_c from the behavior of constructed facilities. The utilization of performance records from large scale instrumented earth loads and excavations has particular appeal because of the close approximation of conditions which will control the behavior of the as-constructed facility. There are, however, serious problems concerned with this approach, which include the following:

1) Unknown boundary conditions

2) Isolation of the volumetric component of measured deformation

3) Measurement and prediction of effective stress

4) Evaluation of loading and unloading rate effects

5) Cost considerations

I. In-Situ Density, Relative Density and Porosity as Indicators of Volume Change Properties

A knowledge of the in-situ density or relative density and water content provides an indication of the potential for, or magnitude of, several volume change phenomena including collapse, expansion, settlement under cyclic loading and liquefaction.

Undisturbed sampling, the Washington densometer, the sand cone and nuclear probes provide reliable means for evaluating in-situ water content and density. Penetration resistance, electrical resistivity and plate load tests have also been successfully employed to measure density.

1. Penetration Resistance as a Measure of Relative Density

Some correlations between relative density and blow count have been presented in Figures III-2 and III-3. Field data indicate that the effective overburden stress, σ'_v, should be multiplied by K/0.4 before entering these charts. This correction may alter the derived relative density considerably, since compacted sand fills may exhibit a coefficient of lateral pressure as great as 1.5.

CPT

Some suggested correlations between relative density and q_c are presented in Figure V-16. The relationships were developed from chamber test results and statistical correlations. Correlations of this type are subject to a number of uncertainties, including the determination of the reference relative densities themselves.

2. Collapse and Expansion Behavior

In-situ density is a useful indicator of potential expansion or collapse in partly saturated soils. Figure V-17 presents a general guide for assessing this behavior on the basis of natural dry density and liquid limit. Soils with combinations of liquid limit and natural dry density that plot below and to the right of the solid curved lines are susceptible to expansion; those plotting above and to the left are susceptible to collapse on wetting.

J. Volume Change Properties and Expansive Soils

Expansive soils pose very serious problems in many areas of the world, yet methods for in-situ measurement of their volume change properties are in a very early stage of development.

Satisfactory prediction of the amount and rate of heave in any case depends at least on the following factors:

1) Thickness of expansive soil layer

2) Depth to water table

3) Extent of soil dessication

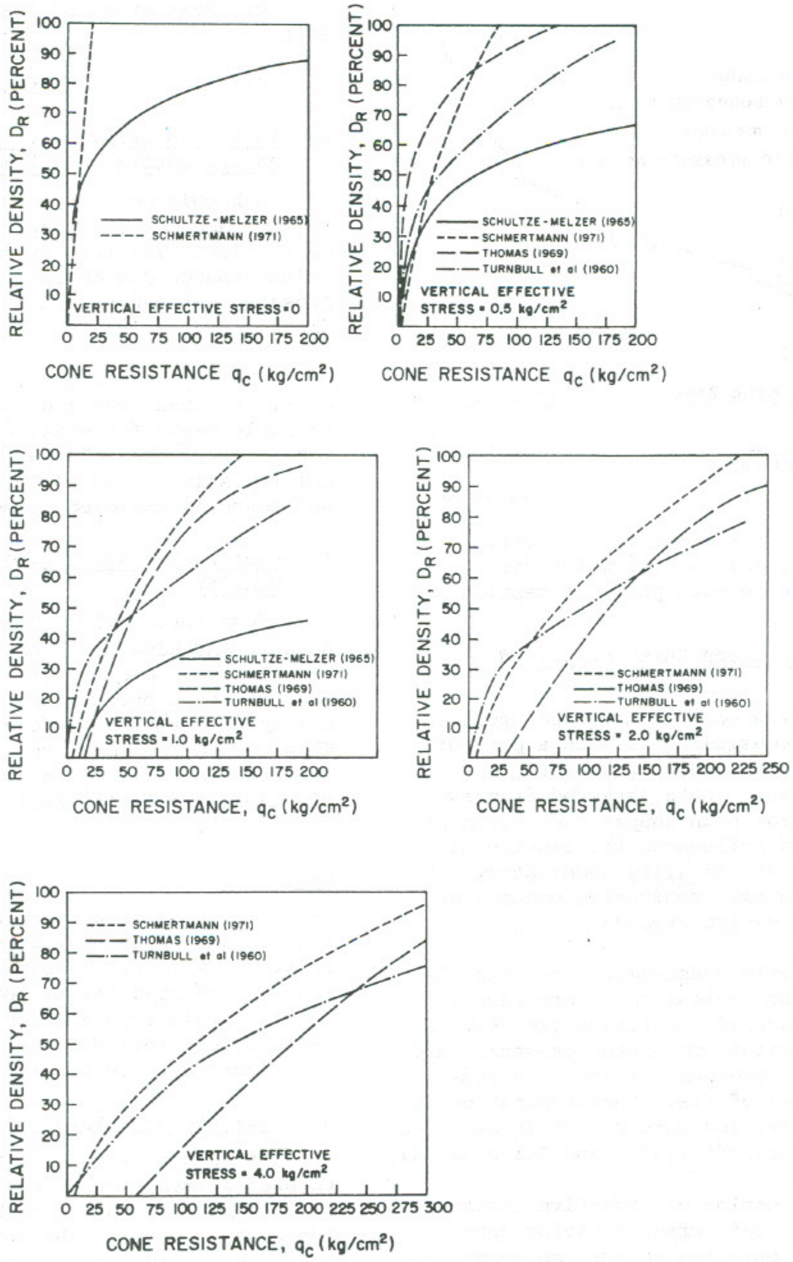


Fig. V-16 Correlations between static cone resistance, vertical effective stress, and relative density

4) In-situ stress

5) Applied stresses

6) Soil characteristics which determine the swelling index and coefficient of swelling

7) Depth of seasonal moisture variation

8) Chemical and biological environment

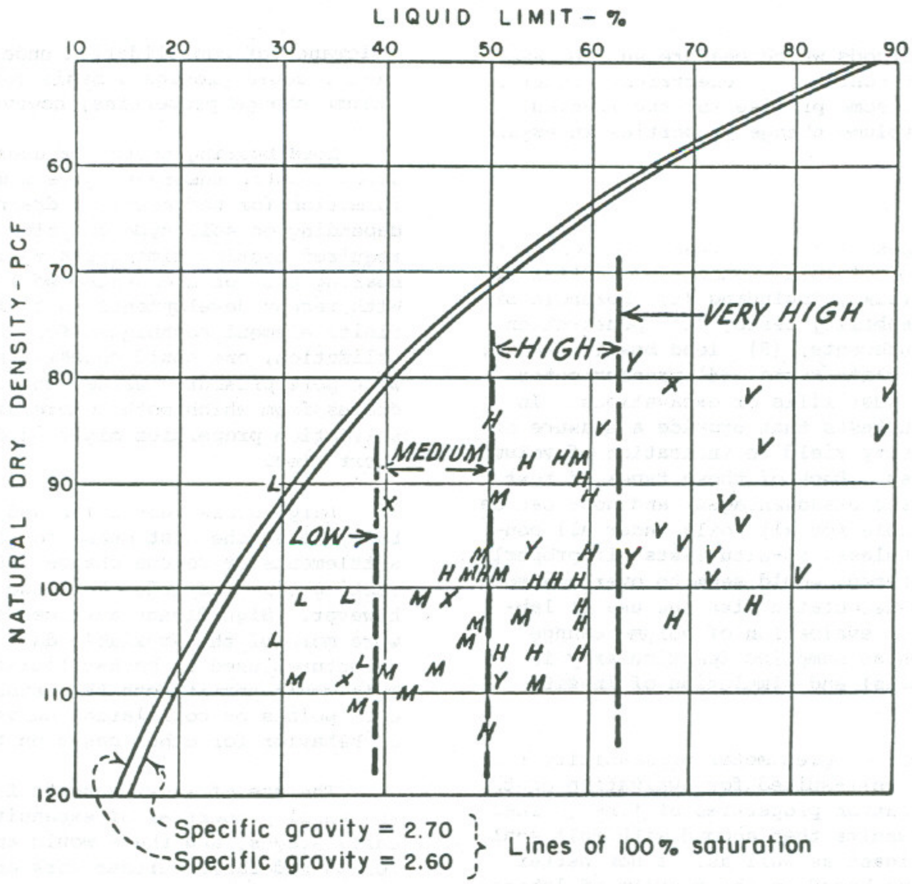
9) Temperature distributions.

In-situ methods are capable of providing information on each of these factors, but the techniques are not yet commonplace.

A distribution of soil swell which varies parabolically with distance above the water table has been considered by Vijayvergiya and Sullivan (1974). The distribution is sketched in Figure V-18. The surface heave, H, corresponds to the summation of soil swell over the depth, h, and is given by the equation

$$H = \frac{1}{3} h \left(\frac{\Delta h}{h}\right)_o \qquad \text{(V-27)}$$

where: $\left(\frac{\Delta h}{h}\right)_o$ is the unit swell at the ground surface



Symbol	Estimated Degree of Expansiveness
V	Very high
H	High
Y	Medium high
M	Medium
X	Medium low
L	Low

Fig. V-17 Guide to collapsibility, compressibility, and expansion based on in-situ dry density and liquid limit (adapted from Gibbs, 1969)

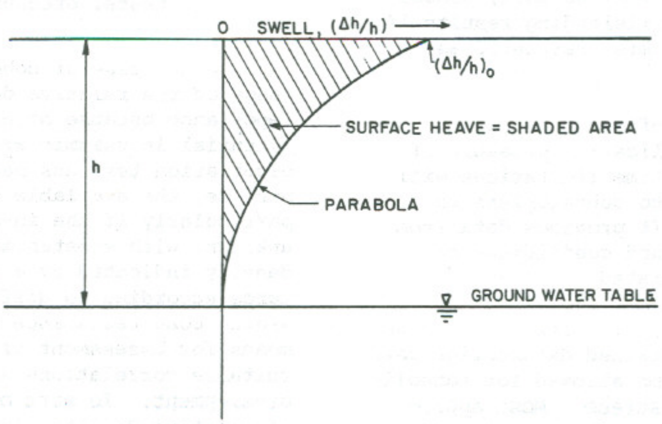


Fig. V-18 Variation of swell with depth according to Vijayvergiya and Sullivan (1974)

Indirect methods which measure such parameters as pH, salt content, or electrical properties are seen to hold some promise for the in-situ measurement of volume change properties in expansive soils.

K. Conclusions

Several types of in-situ tests are available for determination of volume change properties of soils and soft rocks, including (1) borehole or piezometer permeability tests, (2) penetration resistance measurements, (3) load bearing tests, including screw plate tests, (4) pressuremeter tests, (5) and test fills or excavations. In addition in-situ tests that provide a measure of density or porosity yield an indication of volume change properties. Each of these types of test has advantages and disadvantages, and none can be considered suitable for all soils under all conditions. Nonetheless, in-situ tests, if properly done and interpreted, would seem to overcome some of the problems associated with the use of laboratory tests for evaluation of volume change properties, such as sampling (particularly in cohesionless soils) and simulation of in-situ stress conditions.

The borehole or piezometer permeability test is particularly well-suited for evaluation of the rate of consolidation properties of fine-grained soils, giving results that accord with full scale performance at least as well as, if not better than, predictions based on the results of laboratory tests. Whether this type of test can be used for evaluation of compressibility as well (see equation V-10) remains to be seen. Problems because of disturbance, rapidly varying properties with distance from the probe and changing stress fields adjacent to the test device may be important, however.

Penetration resistance measurements, because of their simplicity, speed, and low cost are attractive. Correlations with volume change properties are empirical, and the test results in cohesionless soils are sensitive to in-situ stress conditions. The static cone penetration test can be well suited for evaluation of the compressibility and relative density of normally consolidated sands, but can give misleading results if overconsolidated sands are misinterpreted as normally consolidated.

The screw plate test offers a method for evaluation of the preconsolidation pressure of most soil types, although time limitations will probably restrict its use to cohesionless to slightly cohesive soils. It provides data from which the compressibility and coefficient of consolidation can be calculated.

Analysis of pressuremeter tests can be made in terms of undrained or drained deformation parameters depending on the time allowed for consolidation under each applied stress. Most applications to date in cohesive soils have been for determination of undrained properties. Analysis of the data in terms of effective stresses or the allowance of consolidation under each applied stress could provide a basis for determination of volume change properties, however.

Load bearing tests are usually interpreted using elastic theory to give a modulus of deformation for undrained or drained conditions, depending on soil type and time of loading. The required testing time for a fully drained load bearing test on a cohesive soil may be prohibitive. With recent developments in finite difference and finite element techniques for analysis of consolidation, one could conceive a load bearing test with pore pressure measurements and/or artificial drains from which both compressibility and consolidation properties might be determined in a short time.

Large scale test fills and excavations probably offer the best means for assessing potential settlements or volume change. High cost and long testing time preclude their use in most cases, however. Significant advances should be possible were more of the available data for full scale structures used to back-calculate properties. This would permit both the establishment of more data points on correlation curves and prediction of behavior for other cases on the same soil.

The use of in-situ tests for evaluation of the swell properties of expansive soils is in its early stages, and there would appear to be need for considerable further work on tests for use in these materials.

The most suitable currently available in-situ tests for evaluation of the volume change properties of different soil types are as follows:

Normally consolidated sands - static penetration tests, screw plate tests, pressuremeter tests

Overconsolidated sands - screw plate tests, load bearing tests pressuremeter tests

Soft clay - permeability tests, static penetration tests, self-boring pressuremeter tests

Stiff clay, shale - load bearing tests, pressuremeter tests

In the case of cohesionless soils, a knowledge of the relative density may be of primary importance because of concern over liquefaction potential in seismic areas. Although the standard penetration test has been widely used for this purpose, the available correlations are tenuous, particularly if the in-situ stress conditions are unknown, with substantial variations in relative density indicated by a given penetration resistance according to different correlations. The static cone resistance may offer more reliable means for assessment of relative density, although suitable correlations are still in early stage of development. To sort out the combined influence of relative density, in-situ stress and fabric will probably necessitate the combined use of more than one test type; e.g. static cone and screw plate.

VI. SUMMARY AND CONCLUSIONS

This report describes many of the methods currently used for the in-situ measurement of soil properties and indicates procedures for deduction of specific property values from the acquired data. The references cited provide more complete details of the procedures for each test.

An assessment of the applicability of the methods discussed is presented in Table VI-1. Each method is listed and its suitability for determining various different geotechnical parameters is indicated by a "grade" of A, B or C, with A indicating high applicability, C indicating limited applicability, and no mark indicating no applicability for determination of the property. On the basis of information obtained for preparation of this report, the potential for future development of the different methods for improved determination of different parameters has been assessed. These conclusions are summarized in Table VI-2 It is recognized that such an assessment is very subjective in nature.

It is believed that a number of techniques, such as the SPT, which enjoy widespread use at present, offer relatively little potential for future development. This is not to say that these methods will not see continued use and refinement. In many cases, what is necessary is not a drastic improvement, but rather a standardization of technique.

On the other hand, some methods which are infrequently used at present are expected to become increasingly popular, as their potential is recognized and exploited. The PMT is a prime example of a method which is expected to see increasing use. Similarly the static cone penetration test is likely to see greatly increased use in the U.S.

Sophisticated analytical methods for predicting soil behavior can now be used to evaluate complex boundary conditions and the effects of non-homogeneous, anisotropic and non-linear soil properties. In-situ testing constitutes a potential means of defining such conditions and characterizing such behavior, but at the present time the accuracy of analytical methods usually surpasses the accuracy attainable in measuring input parameters. In this context, the goal of in-situ testing research is two-fold: to improve our understanding of the behavior of undisturbed soil deposits, and to provide accuracy and relevant design parameters for specific engineering problems.

The potential advantages of in-situ measurement of soil properties are both fundamental and practical. The fundamental advantages include:

1) Decreased soil disturbance as compared to that incurred during sampling and laboratory testing.

2) Retention of the in-situ stress, temperature, chemical and biological environments.

The practical advantages include:

1) Increased cost-effectiveness of testing.

2) The potential ability to increase the intensity of testing for a given project and thus develop a data base which is suitable for statistical evaluation.

3) Decreased number of error sources.

The disadvantages of in-situ testing lie in the difficulty of attaining the realization of these potential advantages, as well as the lack of a sample (in many instances) for verification of the actual soil type being tested. The problem of test "loading" is a fundamental consideration of property measurement, i.e., "How much does the measurement of a given quantity affect the magnitude of that quantity?"

Future Developments

Possible improvements in in-situ testing may be categorized as follows:

1. Refinement of existing procedures and methods of interpretation.

2. Introduction of new methods.

3. Synthesis of existing methods.

The refinement of existing procedures and methods of interpretation is an on-going process. Some significant contributions such as the drained analysis of the PMT have been described in this report, and some important methods, such as an interpretation of the CPT in terms of cavity expansion, are expected to be available in the near future. A second report is planned concerning potential new approaches and methods for evaluation of soil geotechnical properties in-situ. Of particular interest in this regard are geophysical and remote sensing approaches.

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Table VI-1. Current suitability of methods

TABLE VI-1. CURRENT SUITABILITY OF METHODS	Soil Type	Vertical Effective Stress	Horizontal Effective Stress	Pore Pressure	Density	Water Content, Porosity	Saturation	Coefficient of Compressibility	Permeability Horiz. - Vert.	Coeff. of Consolidation (Vert. or Horiz. Drainage)	Young's & Shear Modulus	Angle of Internal Friction	Undrained Shear Strength	Drained Shear Strength	Gas - Fluid Chemistry	Soil Profile		
																Vertical	Horizontal	Obstructions
Dynamic Penetration	B				B	B		C			C	B	C			A		A
Static Penetration	B		C		B	C		C			C	B	B			A		A
Vane Shear		C	C										A			B		
Falling/Constant Head, Borehole Permeability				A				C	A	B						C		
Large Scale Pumping								C	A	C								
Hydraulic Fracture			B						C				C					
Piezometer Probe	A			A					A	B								
Pressuremeter			A					B		C	A	B	B	C		B		
Borehole Shear	C		C								C	B	B	B		B		
Direct Shear												A	A	B				
Plate Load											A	A	A					
Screw Plate	C	A									A	A	A					
Piezometer				A				C	A	B								

A = high applicability; B = moderate applicability; C = limited applicability; D = No applicability.

Table VI-2. Potential methods for further future development and improvement

TABLE VI-2. POTENTIAL OF METHODS FOR FURTHER FUTURE DEVELOPMENT AND IMPROVEMENT	Soil Type	Vertical Effective Stress	Horizontal Effective Stress	Pore Pressure	Density	Water Content, Porosity	Saturation	Coefficient of Compressibility	Permeability Horiz. - Vert.	Coeff. of Consolidation (Vert. or Horiz. Drainage)	Young's & Shear Modulus	Angle of Internal Friction	Undrained Shear Strength	Drained Shear Strength	Gas - Fluid Chemistry	Soil Profile		
																Vertical	Horizontal	Obstructions
Dynamic Penetration	C				B			C				C	C					C
Static Penetration	A	B	B	A	B	B		B			B	A	A			B		C
Vane Shear	B		C									C	B					
Falling/Constant Head, Borehole Permeability								B	C	B								
Large Scale Pumping				C				B	B	B								
Hydraulic Fracture		C	C															
Piezometer Probe	C	C	B	A	B			C	A	B								
Pressuremeter	C	B	A	A				A		B	A	A	A	B				
Borehole Shear	C		C								C	B	B					
Direct Shear											C	C	C					
Plate Load								C			C	C	C					
Screw Plate					C			A			A	A	A					
Piezometer				C				C	C	C								

A = high applicability; B = moderate applicability; C = limited applicability; D = No applicability.

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LIST OF SYMBOLS AND ABBREVIATIONS

a	Cell radius of the pressuremeter
a	Diameter of borehole for permeability tests
a	Penetration resistance intercept factor
a	Shear stress distribution factor, pertaining to the vane shear test
a_v	Coefficient of compressibility
B	Base width dimension
b	Length of borehole for permeability tests
BST	Iowa borehole shear test
C_c	One-dimensional compression index
C_r	One-dimensional recompression index
C_s	One-dimensional swelling index
C_1, C_2	Embedment and creep correction factors for the screw plate test
c	Cohesion
c_{ch}	Coefficient of consolidation for vertical compression and horizontal flow
c_{vv}	Coefficient of consolidation for vertical compression and vertical flow
D	Depth of penetrometer base
D	Diameter
D	Stress dilatancy parameter
D	Vane diameter for the vane shear test
D_R, D_r	Relative density
d	Diameter
E	Elastic modulus
E	Energy input through the standard penetration test
E_a	Alternated loading or rebound modulus
E_d	Deformation modulus
E_{PMT}	Modulus obtained by the pressuremeter test
E_s	One dimensional (constrained) modulus
$\overline{E}_s, \overline{E}_{st}$	Elastic modulus determined by the load bearing test
E_u	Young's modulus under undrained loading conditions
E^+	Recompression modulus
E'	Deformation modulus under drained conditions
e	Void ratio
FR	Friction ratio on the static penetrometer
f_s	Side friction on the static penetrometer
G	Shear modulus
GWT	Ground water table
H	shear vane height
H_c	Constant Piezometric head
$H_1 H_2$	Piezometric head at time t_1, t_2
h	Depth increment
I_p	Plasticity index
I_z	Settlement influence factor
I_1, I_0	Influence factors for surface loading and loading at depth for the load bearing test
i	Hydraulic gradient
K_{nc}	Lateral earth pressure coefficient for normally consolidated soils
K_o	In-situ lateral earth pressure coefficient
K_o'	In-situ lateral earth pressure coefficient
K_p	Maximum passive earth pressure coefficient
K	Coefficient of subgrade reaction
k	Coefficient of permeability (hydraulic conductivity)
k_h	Coefficient of permeability in the horizontal direction
k_m	Mean coefficient of permeability ($k_m = \sqrt{k_v \cdot k_h}$)
k_v	Coefficient of permeability in the vertical direction
k_v'	Vertical coefficient of permeability for casing
L	Length
ℓ	Factor utilized in pressuremeter test analysis relating volumetric to tangential strain
m	Transformation ratio, pertaining to horizontal and vertical permeability ($\sqrt{k_h/k_v}$)
m	Modulus number
m_v	Compressibility
N	Blow count for standard penetration test
$\overline{N}$	Correlation factor for strength in the pressuremeter test analysis
N_A, N_N	Standard penetration test blow counts for A, N rods
N_c, $N_{\gamma q}$	Bearing capacity factors
N_{FF}	Standard penetration test blow count for ideally free falling hammer
N_p	The slope of the point resistance versus effective overburden pressure profile for the static penetration test
N_{st}	Standard penetration test blow counts in sensitive clays
n	Porosity
NC	Normally consolidated
OCR	Overconsolidation ratio
P_o	Horizontal earth pressure at rest
P_y	Creep limit
p	Total overburden pressure
p_a	Applied pressure during screw plate test
p_f	Chamber pressure when soil fails during pressuremeter test

p Limit pressure during pressuremeter test

p_o In-situ lateral pressure in pressuremeter test

p' Effective overburden pressure

p' Stress point, average of the three principle effective stresses

p'_c Effective preconsolidation pressure

PI Plasticity index

PMT Pressuremeter test

p.s. Plane strain

Q Volumetric flow rate

q Deviator stress

q Volumetric flow rate

q_c Static cone point resistance

q_u Unconfined compressive strength

QR Normalized flow rate

Q - CPT Quasi-static penetration test

R Total resistance on standard penetration test samples

RK Ratio of horizontal to vertical permeability

s_h Horizontal shear strength in vane shear test

s_u Undrained shear strength

s_{uv} Undrained shear strength from the Vane Shear Test

s_v Vertical shear strength

SPR Standard Penetration Resistance

SPT Standard penetration test

T Maximum applied torque in the vane shear test

T Time factor

T Time lag in the borehole permeability test

t Time or elapsed time

V Chamber volume of the pressuremeter

V_i Initial chamber volume of the pressuremeter

v Flow velocity

v_s Shear wave velocity

$\dot{v}$ Particle velocity

VST Vane shear test

w Water content

w_L Liquid limit

Z Layer thickness

α Parameter relating cone resistance with constrained modulus

α Penetrometer base semi-apex angle

γ Shear strain during pressuremeter test

γ_t Total unit weight

γ_s Saturated unit weight of soil

γ_ω Unit weight of water

δ Friction angle between penetrometer and soil

ε_r Radial strain during pressuremeter test

ε_v Volumetric strain

ε_z Vertical strain

ε_o Radial strain of the pressuremeter cell

ε_1 Equivalent axial compressive strain during the pressuremeter test

$\varepsilon_1, \varepsilon_2, \varepsilon_3$ Major, intermediate and minor principal strains

ε_θ Circumferential strain during pressuremeter test

η Stress ratio between deviator stress and average principal effective stresses

μ Bjerrum correction factor during vane shear tests

μ Poisson's ratio

ν Angle of dilation

ξ_A, ξ_B Radial displacement of point A, B during pressuremeter test

ξ_c, ξ_q Shape factors

ρ Bulk resistivity during pressuremeter test

ρ Resistivity

ρ Settlement

ρ Soil density

ρ_ω Pore water resistivity

ρ_1 Settlement of one foot diameter bearing plate

σ_h Total horizontal stress

σ'_h Effective horizontal stress

σ_r Radial stress during the pressuremeter test

σ_v Total vertical stress

σ'_v Effective vertical stress

$\sigma_1, \sigma_2, \sigma_3$ Major, intermediate and minor principal stresses

τ Shear stress

τ_{ps} Shear stress, plane strain

ϕ Angle of internal friction in terms of total stress

ϕ' Angle of internal friction in terms of effective stress

ϕ_{cv} Angle of internal friction pertaining to shear at constant volume

ϕ_o Component of angle of internal friction, not due to dilatancy, in terms of effective stress

ϕ'_{PMT} Angle of internal friction, in terms of effective stress, derived by the pressuremeter test