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The MIT Marine Industry Collegium
Opportunity Brief #16

Towards Improved Techniques for Predicting Soil Strength in Offshore Environments

A Project of
The Sea Grant College Program
Massachusetts Institute of Technology
MITSG 79-17

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TOWARDS IMPROVED TECHNIQUES FOR PREDICTING SOIL STRENGTHS IN OFFSHORE ENVIRONMENTS

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Revised Edition

July 1, 1979

Marine Industry Advisory Services

MIT Sea Grant Program

Cambridge, Massachusetts 02139

Report No. 79-17

Index No. 79-717-Cim

PREFACE

This Opportunity Brief and the accompanying Workshop (held on March 29, 1979) were presented as part of the MIT/Marine Industry Collegium program, which is supported by the NOAA Office of Sea Grant, by MIT and by the more than 90 corporations and government agencies who are members of the Collegium. The underlying studies at MIT were carried out under the leadership of Professor Mohsen M. Baligh, but the author remains responsible for the assertions and conclusions presented herein.

Through Opportunity Briefs, Workshops, Symposia, and other interactions the Collegium provides a means for technology transfer among academia, industry and government for mutual profit. For more information, contact the Marine Industry Advisory Services, MIT Sea Grant, at 617-253-4434.

Norman Doelling

1 July, 1979

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1.0 A Business Perspective

To design economical offshore structures that will withstand the ravages of wind and wave, detailed knowledge of the engineering properties of subsea soil is essential. The problems are even more complex than those faced in designing large structures on land because the engineering properties of marine soils are less well understood and field measurements are extremely difficult and expensive to obtain. Acquiring "undisturbed" field samples for laboratory tests on a site several hundred feet below the ocean surface is almost impossible. New techniques and instrumentation are needed for estimating the engineering properties of subsea sites. This Opportunity Brief describes some emerging methods that are proving to be promising.

On land a wealth of information has been obtained which relates data from standardized tests (in the field or in the laboratory) to the *in situ* undrained shear strength of soils, so that this most relevant engineering parameter can be obtained from relatively straightforward field test procedures. At the Massachusetts Institute of Technology a group of researchers has been working on improved methods for estimating the undrained shear strength of subsea soils from *in situ* field measurements obtained from an instrument known commercially as a Dutch cone or Fugro cone. The cone is pushed into the soil at a fixed speed, and the force on the cone (penetration resistance) is recorded. This force can be related experimentally and theoretically to the shear strength. The pressure of the liquid content of the soil, pore pressure, is also measured to provide additional information to identify stratification of the soil, sandy "lenses" and the like.

This current work, which is sponsored by the M.I.T. Sea Grant College Program, Fugro, Inc., and the Instituto Technologico Venzolano Del Petroleo, draws on soil data developed during an eleven-year research program at M.I.T. sponsored by the Massachusetts Department of Public Works and the Federal Highway Administration.

The current research program involves:

1) Theoretical solutions for interpreting test results.
2) Modifications and variations of the geometry and transducers of the basic Fugro instrument.
3) Field measurements with various cones in soils of known properties and parallel measurements of pore pressure.
4) Interpretation of the test results on the basis of the theoretical models and the known soil properties.

For the offshore design and construction industry and for regulatory agencies, the techniques and analytical methods may lead to better understanding of marine soil properties and to safer, less costly offshore structures. For instrument companies, improvement in transducers and electronics, multiplexing of data from the transducers, and some "on-board" signal processing could result in a marketable new instrument. We do not imply that the results are complete. This Brief is the framework for a progress report to Collegium members which was held at MIT on 27 March, 1979.

This Brief draws heavily upon a paper to be published as part of the proceedings of BOSS '79, An International Conference on the Behavior of Offshore Structures, London, August, 1979 (Reference 1), and on the FY 1980-81 MIT Sea Grant Proposal submitted to the NOAA Office of Sea Grant.

2.0 Some General Background

Determining the engineering properties of soil layers for the design of structural foundations is usually done by making borings from which samples are taken for subsequent laboratory tests. Lab tests provide uniformity of test procedures and test conditions. However, the process of removal disturbs the soil sample and changes its properties. In addition, not enough information about the spatial variability of soil properties can be obtained from limited laboratory samples.

In the offshore environment, there exists a need to estimate soil parameters as reliably as possible since overdesign is extremely costly. Traditional sampling techniques are even less reliable at sea. Since most samples are obtained from surface ships, the samples are subjected to to variable, unknown stress from the pitching ship; and the samples are even more disturbed than on land. Furthermore, since very great stress relief occurs when samples are brought to the surface, the disturbance may be catastrophic. Thus new techniques are required for obtaining reliable data to estimate statistical risk.

3.0 The Research Program

The research program uses the Fugro cone and special piezometer probes as *in situ* testing devices. These are shown in Figures 1 and 2. Theoretical models of the interaction of cones and soils have been derived to relate the parameters obtained from the test devices to the engineering properties of soils. The models provide a framework for the analysis and "calibration" of field test data. Field tests were made at sites which have been studied in detail and whose engineering properties have been studied extensively by other means of field testing (field vane tests, for example), by laboratory sample tests, and also by "hindcasting" a planned and calibrated embankment failure.

3.1 The Cone Penetration Test: Theory

The load bearing capacity of clay soils is determined by the undrained *in situ* shear strength. Shear strength is not, however, easy to measure. Soil properties depend upon their stress history; hence samples taken from a field site to the lab are "disturbed" and properties measured in a laboratory may differ substantially from those *in situ*. The traditional U.S. *in situ* practice, "field vane" tests to measure shear strength, is time-consuming and expensive on land and is limited to soft to stiff clays.

The Dutch or Fugro cone was developed in Europe to estimate shear strength. The cone is relatively simple to operate. It is pushed into the ground at a constant speed (typically 2 cm/sec), and a transducer measures the resistive force *on the cone* per unit area of the cone base. This penetration resistance is then related to shear strength.

"Calibration" of such a cone, that is, the ratio of the cone resistance to the shear strength, has been done empirically by measuring

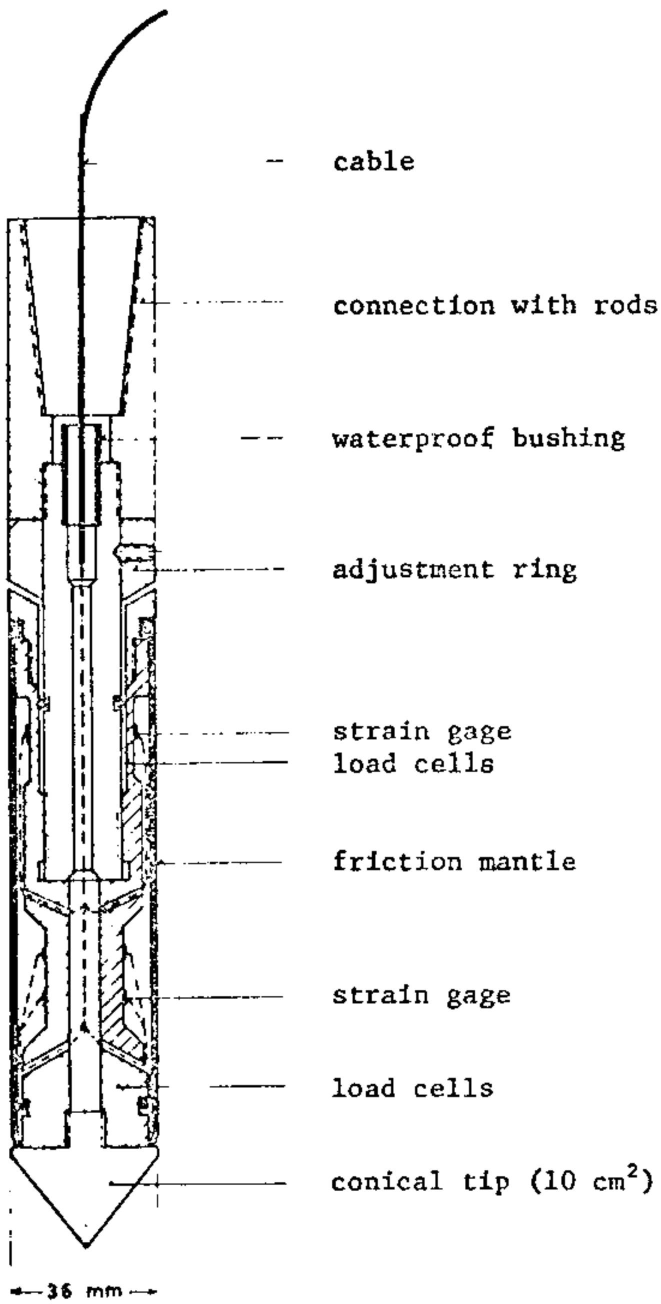

Fig. 1 Diagram of the Fugro electrical cone with friction sleeve (from Sanglerat (1972))

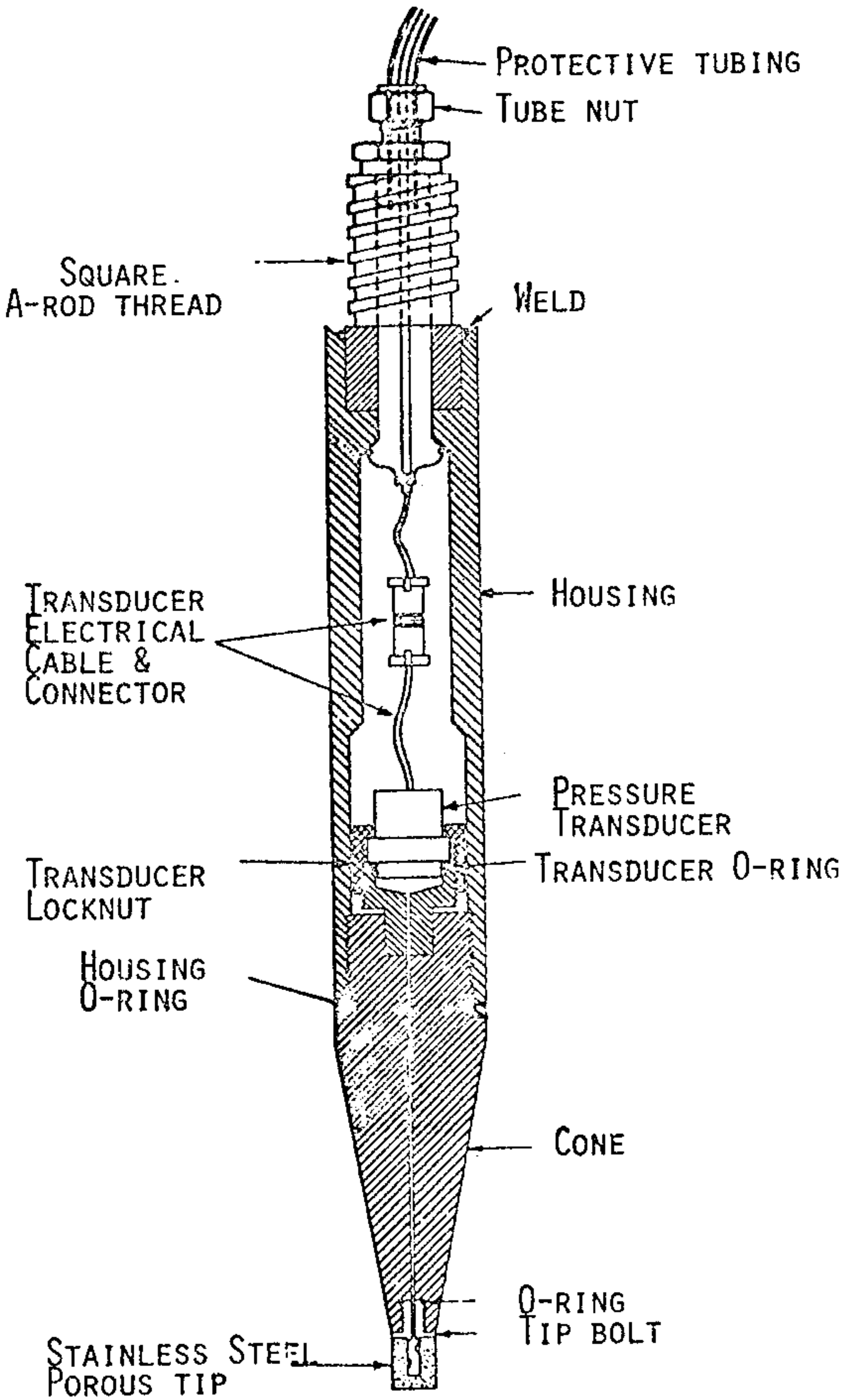

Fig. 2 Diagram of pore pressure piezometer probe (Wissa et al., 1975).

properties of known soils.

Professor M. M. Baligh of the M.I.T. Department of Civil Engineering investigated the ratio of resistance to shear strength for three simple geometries: a wedge, a cone pushed by a rod thinner than the cone base, and then a cone pushed by a rod whose diameter is equal to the cone base. These geometries are successively more difficult analytically, but each in turn is a better approximation to the Fugro cone.

For the wedge, a simple relationship is obtained. Under the assumptions that the soil is 1) homogenous, 2) isotropic, 3) massless, 4) rigid-perfectly plastic, 5) incompressible, and 6) with undrained shear strength, s_u, independent of hydrostatic stress, the penetration resistance, q_w, is directly proportional to the undrained shear strength, s_u, or

$q_w = N_w s_u$, where

N_w is a function of wedge angle, Θ, given by

$N_w = 5.71 + 1.67\Theta + 1/\tan(\Theta/2)$. ($\Theta$ in radians)

For a cone pushed by a small rod, whose diameter is significantly less than the diameter of the base of the cone,

$N_c' = 1.2\ N_w$.

In the case of a cone attached to a rod of the same diameter, the problem is more complex because the stresses on the vertical sleeve above the cone are unknown. When such stresses are negligible, the penetration resistance should be about equal to that given above. Baligh (References 2 and 3) has formulated an upper limit to N, for the case of significant horizontal stress, σ_h.

As an illustration, Baligh calculated cone penetration resistance coefficient $N_c = q_c/s_u$ for specific forces, which is given in Figure 3. The shaded area represents the range of possible values for a regular tip depending on the normal stress on the sleeve behind the cone. This considerable uncertainty can be resolved only by field tests.

3.2 Cone Penetration: Field Test

Real soils are not homogenous or isotropic, etc.; they are inhomogenous, unisotropic, etc. A field testing program was therefore undertaken to attempt to determine N_c in "real" situations. The on-going comprehensive field cone penetration test program was begun by M.I.T. in July, 1976. During this program, the cone penetration resistance (and the sleeve friction in some cases) was measured using cones with different tip angles and different ratios of shaft to cone diameters. At the same time a pore pressure measurement program was undertaken, since this parameter is also important in soil mechanics and is not well understood. It was expected that pore pressure measurements in conjunction with cone penetration tests would yield significant information for identifying types of soil in various strata. Detailed data were obtained of the pore pressure during steady penetration of conical pore pressure probes and of the pore pressure decay when penetration stopped. The probes were developed by Anwar Wissa, former MIT professor, were built by Geotechniques International, Inc., and were made available to MIT at no cost (Reference 4).

Cone penetration tests conducted by M.I.T. between July, 1976, and June, 1978, consisted of more than 5,500 feet of penetration in three clay deposits. The tests included cones with different geometries (tip angle, Θ, and ratio of cone to shaft diameter, D/d) and were performed at different penetration velocities to investigate the dependence of cone resistance and pore pressure on penetration velocity.

3.3 Representative Data

This section, taken with minor editorial additions from Reference

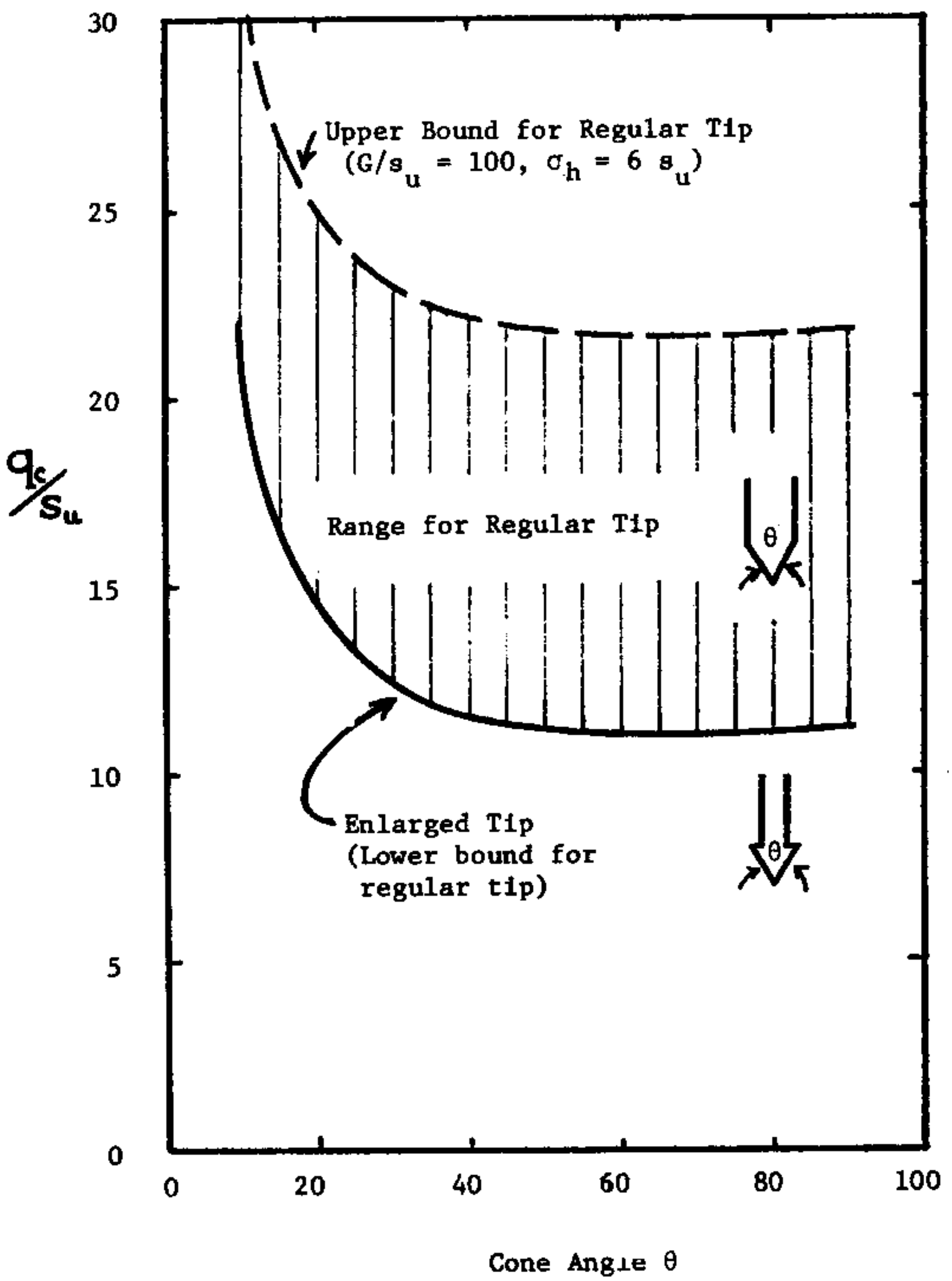

Fig. 3 Theoretical Cone Penetration Resistance in Clays

1, demonstrates the types of information obtained in field tests.

Figure 4 shows a typical record of pore pressure measurements at the tip of an 18^{o} conical probe in the Boston Blue Clay deposit studied by M.I.T. When steady penetration starts at a depth of 43.5' (13.3 m), the pore pressure increases rapidly and reaches the so-called penetration value, u, in less than 3". Steady penetration at a rate of about 2 cm/sec (0.80 in/sec) continues to a depth of 47' (14.3 m, indicated by the arrow) when another push rod (1 m long) is required. The installation of the rod takes 45 sec, and the pore pressure during this time decreases due to soil consolidation. Penetration is then resumed and the process repeated. Note the unmistakable sudden decrease in pore pressure at depths 47.2, 49.3 and 58.6' (14.4, 15 and 17.9 m) which suggests the presence of dense sandy layers. Subsequent results of cone resistance, q_c, show a significant increase in q_c at these locations.

Figure 5 shows the cone resistance, q_c, and the penetration pore pressure, u, (after eliminating the decay that takes place during push-rod installation) obtained from two separate tests 45' apart in the upper deposits consisting of peat, sand and heavily desiccated clay which contains sandy inclusions (called sandy lenses). Individually, the q_c and u records detect major changes in soil strata, but jointly they have an excellent potential for soil identification as well. For example, in the peat, q_c is low and u is high, whereas in the relatively clean sand, q_c is high and u is very close to the hydrostatic values, u_o, i.e. the hydrostatic pressure head which is equal to density times the depth.

Figure 6a shows the cone penetration resistance, q_c, of the standard Fugro cone in three holes located on a straight line 25' (7.6 m) apart. Plots of q_c are made after digitizing the continuous strip chart records

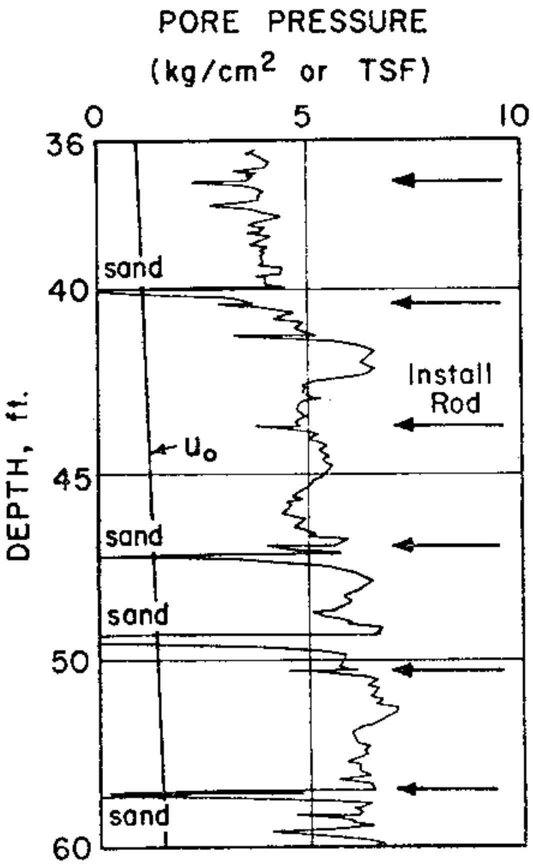

Figure 4. Typical Pore Pressure Recorded at the Tip of an 18° Conical Probe (1 ft = 0.305 m; 1 kg/cm^2 98.1 kPa)

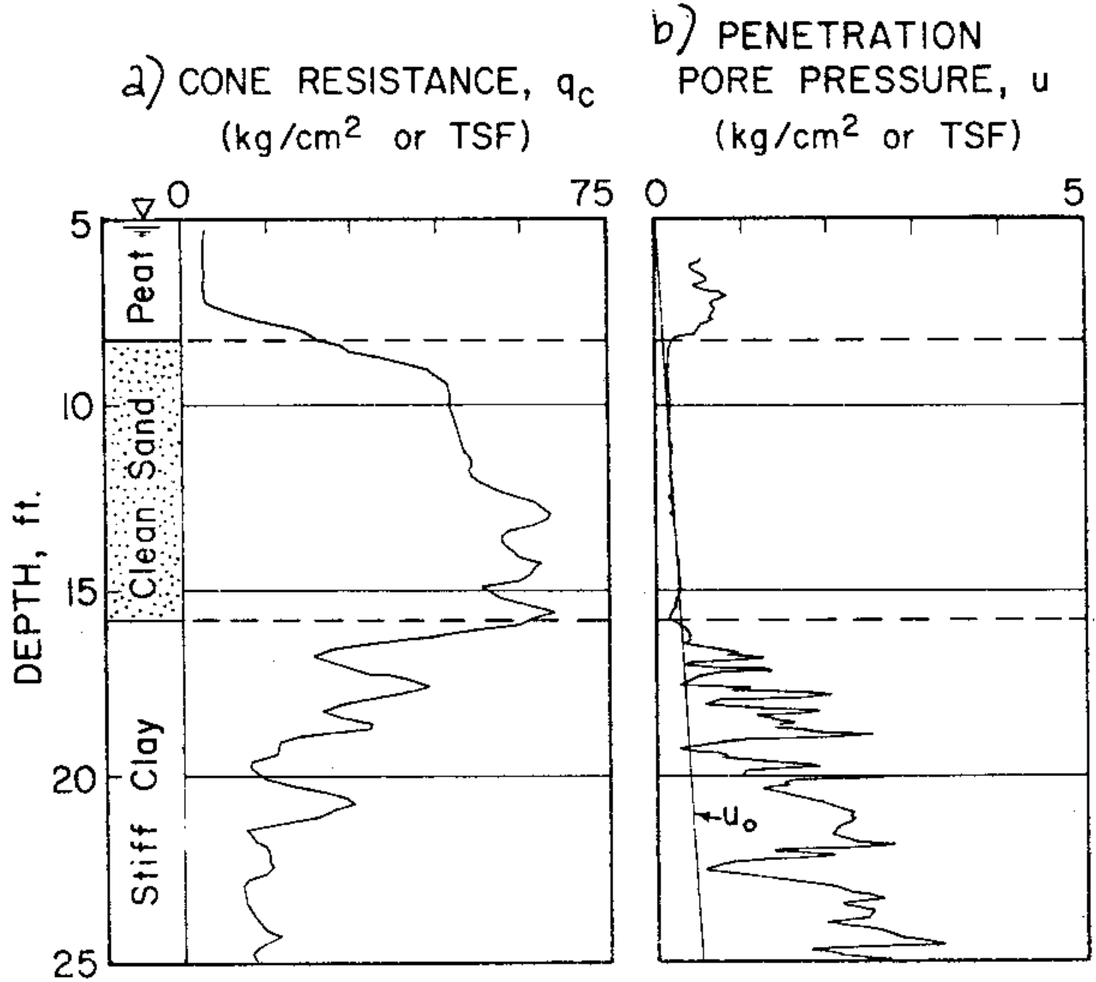

Figure 5. Cone Penetration in Soil Profiling (1 ft = 0.305 m; 1 kg/cm^2 98.1 kPa)

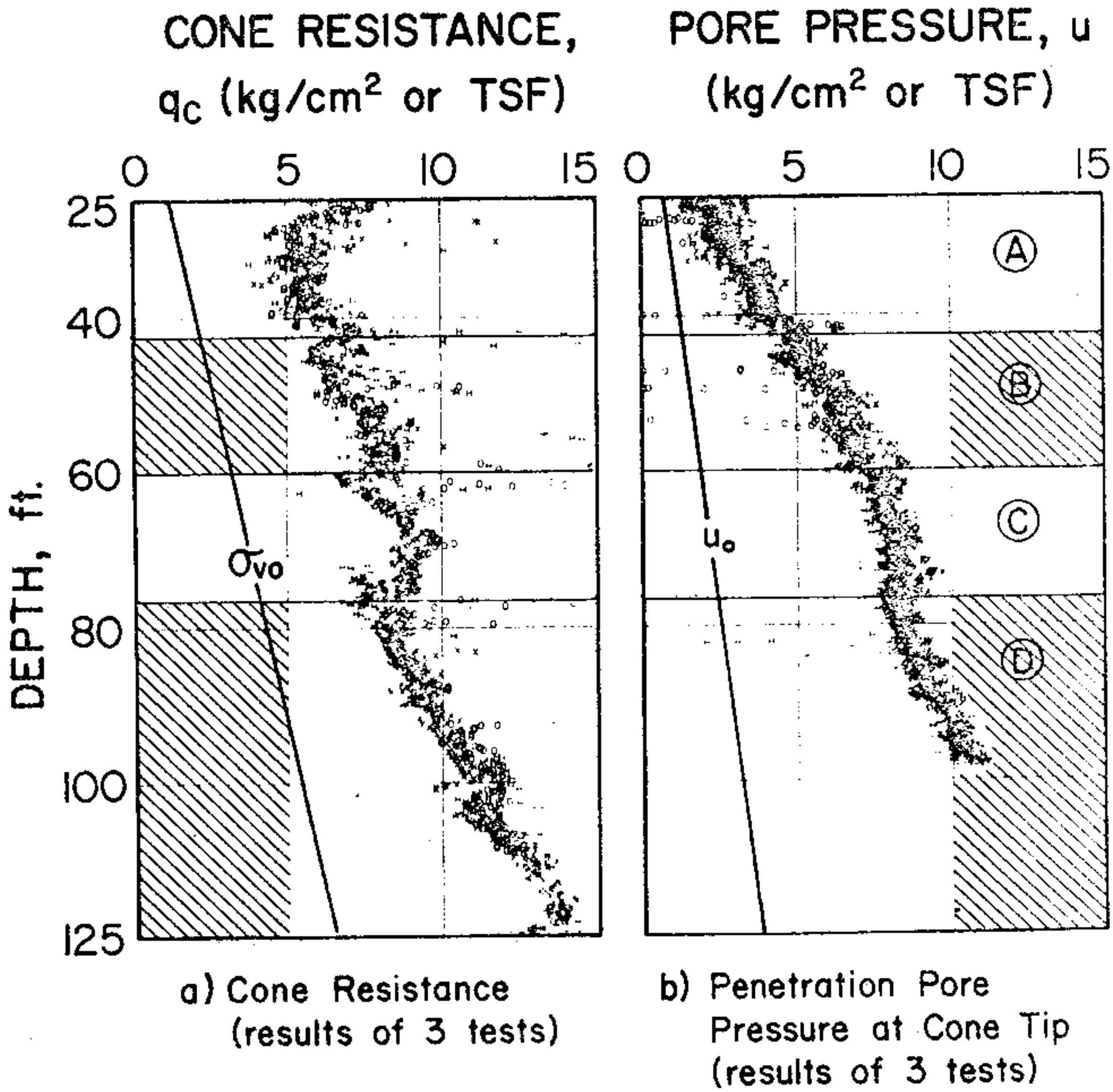

Figure 6. Cone Resistance and Pore Pressure
During Penetration
(1 ft = 0.305 m; 1 kg/cm^2 = 98.1 kPa)

every 3" (7.5 cm) interval such that Figure 6a contains more than 1100 data points. Note the following:

1. The cone resistance is very consistent. This is believed to be due to the simplicity of the test procedures, the uniformity (consistency) of soil disturbance, and the minor importance of human interference. Few other in situ tests provide the repeatability of cone penetration.

2. Since many more data points are obtained compared to, say, the field vane test, stratification and variability of the soil are very clearly defined, the possibility of missing weak layers is greatly reduced, and the temptation to apply engineering judgement to test results is virtually eliminated.

3. The soil profile between depths 25 and 120 feet (7.6 and 36.6 m) can be divided into at least four well defined layers (A, B, C, and D) within which q_c is more or less continuous.

4. At the interface between layers, the average value of q_c undergoes a jump, and q_c may have a sharp peak. Visual examination of samples and the results of piezometer probes (Figs. 4 and 5b) suggest these sharp peaks indicate sandy lenses. The lateral extent of these lenses appears limited because similar indications did not occur in all three penetration tests. Such information is essential in determining drainage layers for problems involving the dissipation of excess pore pressures.

5. Onshore, the cone resistance information in Figure 6a can be easily obtained in one day of field work at less than 10% of the cost of the vane data. The difference in cost might be less in offshore work.

3.4 Penetration Pore Pressure

Figure 6b shows the pore pressure, u, (after digitizing at about 2" - 5 cm - intervals), at the tip of 18^o conical piezometers (see Reference

4) during steady penetration at a rate of 2 cm/sec in three holes located within 50' (15.2 m) radius. We note in Figure 6b that:

1. The penetration pore pressure, u, is as consistent and exhibits roughly the same scatter as the cone resistance, q_c, in Figure 6a. The reasons behind the excellent repeatability of u are the same as discussed earlier for q_c.

2. The average value of u does not show the distinct jumps indicated by q_c between the different layers, except possibly between layers A and B. On the other hand, the average value of u increases linearly in layers A and B, remains more or less constant in C, and increases linearly in layer D. Therefore, based on average values of u alone, one can probably distinguish between three layers (A + B), C and D.

3. The locations where u shows a sudden drop in some tests can be traced to corresponding sudden increases in some q_c tests. This is believed to further confirm the presence of sandy lenses.

4.0 Estimating Strength from Cone Penetration Tests

In situ measurements by means of a field vane and cone penetration tests were compared to extensive laboratory tests and the results of an experiment involving an intentional "load-to-failure" test of a foundation in a medium-to-stiff deposit of Boston Blue Clay.

The method and detail of the test program were described in the Workshop and are reported in Reference 1. The conclusions from Reference 1 are summarized as follows:

1. "Crude" laboratory tests (e.g., unconfined compression test, miniature vane,...etc.) underestimate the undrained shear strength of Boston Blue Clay at depths due to excessive sample disturbance. This is likely to be the case with deep offshore samples.

2. In Boston Blue Clay, the field vane test (FV) provides an excellent s_u profile and good estimates of s_u to be used in circular arc stability analyses. More reliable correlations between s_u(FV) and pile performance are required, especially at the large depths encountered offshore.

3. Measurements of cone resistance, q_c, by means of electrical cones are repeatable and very valuable to: (a) detect the presence of thin soft layers and more pervious layers that might strongly affect stability or drainage, respectively; (b) distinguish between different strata even in the difficult case of consecutive layers of sandy lenses; and (c) provide a good description of soil variability (scatter) affecting design reliability.

4. The variability (standard deviation/mean) of the undrained shear strength, s_u, determined by the cone resistance, exhibits similar profiles and gives the same numerical values as the field vane test. In Boston Blue Clay, the standard deviation equals about 5% of the mean in the

deep clay with low consolidation and increases to 15% of the mean in the dessicated crust. Crude laboratory tests cause a much higher scatter because of sampling disturbance.

5. Conical piezometer probes can be used for identification and stratification of soil deposits by means of the pore pressures, u, developed during steady cone penetration and their subsequent decay after penetration stops. Measurements of penetration pore pressure, u, are repeatable, and provide the same advantages (mentioned in 3) as q_c in soil stratification. Pore pressure decay offers additional information regarding soil permeability.

6. The ratio u/q_c may eventually provide a promising method for estimating the consolidation of clays.

7. Estimates of s_u from q_c measurements are complicated by the difficulty of defining the undrained shear strength, s_u, of clays and selecting adequate values of the cone factor N_c. In Boston Blue Clay, N_c between 8 and 13 gives good estimates of s_u (field) for circular arc stability analyses when q_c is determined by means of "standard" electrical cones (of the Fugro type, 60^o angle pushed at a rate of 2 cm/sec). Compared to N_c obtained by others in four "similar" Scandinavian clays, results in Boston Blue Clay strongly suggest that N_c depends on consolidation and, possibly, on the sensitivity of the clay. Jointly, the results in the 5 clays give $N_c = 14 \pm 6$ at a depth of 25 ft (7.6 m) and $N_c = 14 \pm 3$ at a depth of 100 ft (30 m). Improved methods of interpreting q_c measurements are therefore required, especially at shallow depths, in order to provide estimates of s_u for stability analyses within the band of $\pm$ 20

to 25% provided by the field vane test. This can be achieved by means of more rigorous theoretical studies of cone penetration, the possible use of different cone geometries, and improved cone hardware including capabilities for pore pressure measurements. Reliable correlations with pile performance are also needed, especially for deep penetration offshore piles.

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