International Symposium on Cone Penetration Testing

Linköping, Sweden
October 4-5, 1995

Volume 2 • Technical Papers
Preface

In 1974, the first European Symposium on Penetration Testing, ESOPT was held in Stockholm, Sweden. The main objective of the symposium was to document the use of penetration testing, to outline areas of further research, to stimulate standardisation and to provide guidelines for future developments. After the second European Symposium, ESOPT-II in 1982, the ISSMFE Technical Committee on Penetration Testing decided to continue these successful speciality conferences on an international basis. Hence, in 1988 the first International Symposium on Penetration Testing, ISOPT-I was held in Florida.

Since ESOPT was held more than 20 years ago, the role of geotechnical engineering has changed significantly. New geotechnical areas have been developed, such as geotechnical off-shore, earthquake and environmental engineering. The cone penetration test, developed by Dutch engineers more than 60 years ago, has become the most widely used geotechnical field investigation method.

During the past two decades, the CPT has emerged from a simple, mechanical field investigation tool into a reliable electronic multi-purpose testing method. New types of cone penetrometers have been developed. A variety of sensors can now be incorporated in the cone, such as vibration and acoustic sensors, tilt meters, resistivity sensors, to mention just a few.

The International Symposium on Cone Penetration Testing, CPT'95 is jointly organized by the Nordic geotechnical societies. The theme of the symposium is the solution of geotechnical problems by cone penetration tests. Particular emphasis is placed on the exchange of practical experience and the application of research results. The aim of the symposium is to enhance the exchange of knowledge between researchers and practitioners from countries all over the world and to facilitate the interaction between experienced and younger engineers.

The technical programme comprises Theme Lectures by eminent international experts in the area of penetration testing, presentations of state of practice in Technical Reports and selected papers, a Poster Session, a Technical Exhibition and a Field Demonstration. Information regarding the symposium programme, as well as lists of all papers and respective abstracts were available on the Internet.

The symposium would not have been possible without the dedicated work and competence of the many authors which have submitted papers. The hard work and enthusiasm of many individuals and the support of many organizations and companies provided the basis for the planning and successful implementation of CPT'95.

Linköping, October 1995
Organizing Committee of CPT'95

K. Rainer Massarach
Chairman

Bengt Rydeell
Vice Chairman

Marius Tremblay
Secretary
Readers guide to Proceedings

In order to provide a sound basis for discussions and interaction between symposium participants, National Reports have been prepared by countries from all over the world which document the state of practice of cone penetration testing. Technical Reports have been prepared, covering the three sessions of the symposium. Theme Lectures were given on specific areas of CPT applications.

The symposium is documented in Proceedings consisting of three volumes:

Vol 1: National Reports

Vol 2: Technical Papers
  - Session 1: Equipment and Testing
  - Session 2: Interpretation of Test Results
  - Session 3: Solution of Practical Problems

Vol 3: Theme Lectures
  Technical Reports
  Key Note Addresses
  Summary Reports of Poster Session etc
  List of participants

Vol 3 will also include Technical Papers received after deadline of submission. At the beginning of each volume there is a list of contents and at the end of each volume there is an author index. In Vol 1 the National Reports are listed alphabetically by country. The Technical Papers in Vol 2 are listed for each session alphabetically according to the first author.

The proceedings are published by the Swedish Geotechnical Society in the SGF Report series.

Review of papers
The abstracts submitted were reviewed by the Advisory Committee. Accepted papers were placed in the most appropriate session by the Organizing Committee, as some papers cover more than one theme.

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When using material from these Proceedings full credit shall be given to the symposium and the author(s).
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Session 1 - Equipment and Testing
Cone Penetrometer Testing on North American Railroads

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SYNOPSIS: With the increase in maximum axle loads from 33 to 39 tons (290 to 350 MN) on North American railroads, sections of track with weak subgrade support are expected to require an increasing amount of maintenance. As part of a program to determine the nature of a track substructure problem and to recommend the best remedial activity, the Association of American Railroads has developed an on-track CPT vehicle.

The CPT vehicle is used to determine the strength distribution of the subgrade soil, the thickness of the stress reducing granular layer over it, and the extent of a weak soil deposit under and along the track. Based on the findings from CPT data, various alternative maintenance remedies may be considered. Also, an economic analysis can be applied to determine the most cost effective maintenance remedy. CPT data has been shown to provide invaluable information on track substructure conditions and maintenance requirements.

1. INTRODUCTION
Much of the uneven settlement of railroad track is caused by weak subgrade. In response railroads often attempt remedies without first determining the cause of failure and the substructure conditions. The selected remedial technique is often of limited effectiveness due to a mis-diagnosis of the problem. With the track-mobile CPT vehicle, the railroads now have a means to help determine the depth and longitudinal extent of the problem soil, its strength, the adequacy of the granular layer thickness above it, and the effectiveness of a given solution.

The cone can be pushed through the track substructure in the zones shown in Figure 1. The vehicle weight, approximately 30,000 pounds (135 MN), is used as the reaction mass while the cone is advanced using a hydraulic push frame mounted inside the vehicle. The frame can be moved laterally to position the cone between the rails.

Figure 1. CPT Vehicle and Probing Locations

2. CONE DESIGN CONSIDERATIONS
Early in the development of the test vehicle it was questioned whether commercially available cones were sufficiently robust for repeatedly pushing through the often hard, dense ballast layer. A
foiled ballast which has been compacted by years of train traffic may offer substantial resistance to penetration. After making inquiries with a few suppliers of cones and after some unsatisfactory trials with some commercial cones, it was decided to design and fabricate them at the AAR.

It was estimated that a pushing force of 10,000 pounds (45 MN) should be sufficient to penetrate most harder ballast layers. The corresponding tip resistance for the projected cone area of 1.55 m² (10 cm²) is about 6500 psi (45 MPa). While a cone with tip resistance only could have been made which would withstand substantially more stress, the sleeve resistance was also desired to estimate the soil type using the friction ratio.

While durability is the most important consideration during the push through the top granular layers, the critical issue suddenly changes to measurement sensitivity in low strength soils once the cone breaks through into the softer, weaker underlying layers. Some of the softer soils encountered in practice under track have tip resistance values of 50 psi (0.3 MPa) and below. The AAR cone design provided a reasonable compromise between these two conflicting criteria. The cone design feature which probably provides the most increase in durability over conventional designs is achieved through the increased wall thickness inside the sleeve.

3. CPT RESULTS AND THEIR APPLICATIONS

A typical test result is shown in Figure 2. In this location the track required repeated tamping maintenance to provide acceptable geometry. Tip resistance measurements, taken to 21 feet (6.4 m) below the ballast surface, show an eight foot (2.4 m) thick soft subgrade layer just under the ballast/subballast (granular layer). Under this soft soil layer was a very stiff layer. The granular layer thickness and the strength of the subgrade is of prime importance in assessing the stability of the track.

As an example of how the CPT results can provide a unique insight into the cause of track instability, consider Figure 3. The track deflection under a load of 40,000 pounds (180 MN) was slightly more than one inch (25 mm). Although the track surface was rough, the cause of the instability was not apparent from outward appearance. The ballast was clean on the surface and seemed to be relatively thick. However, the tip resistance measurements illustrate that the clean ballast layer is very thin with a soft layer just under the ties. Excavations in the track...
revealed that the clay had pumped up from the subgrade, migrated through the ballast voids, and was now mixed with the ballast just under the ties.

CPT data can be used to determine the likelihood of the two most prevalent soft subgrade failure modes of (1) progressive shear and (2) excessive plastic deformation (Li and Selig, 1995). Progressive shear is shown in Figure 4a where the soil is squeezed out under the ties. The resulting subgrade profile often has the largest depression just under the tie ends where the shearing stresses are usually the largest. For this subgrade failure mode, the subgrade strength just under the granular layer is of primary concern.

Whereas progressive shear is concentrated in the upper few feet (1 m) of subgrade, excessive plastic deformation (Figure 4b) can result from 25 foot (7.6 m) depth in a subgrade (Li and Selig 1994). To assess the potential of this failure mode, the CPT should be able to penetrate to this depth. It is not necessary to determine if the soft subgrade extends beyond this depth or if a harder layer is just beyond it since this does not significantly affect resilient or permanent strain.

Another use of CPT data is the prediction of track stiffness or modulus. The modulus of the subgrade largely controls that of the track. Research has shown that tip resistance often correlates well with subgrade modulus, as shown in Figure 5 (Ebersohn and Selig, 1992). This relationship was determined from four investigations with widely varying track super- and substructure conditions. With an estimate of subgrade modulus from this correlation, models such as GEOTRACK can be used to estimate the track deflection and modulus.

The placement of hot mix asphalt (HMA) as an underlay beneath the ballast but above the subgrade has been gaining acceptance as a way to reduce the stresses on the underlying weaker materials. However, research has shown HMA to be of little benefit in reducing stresses if the weaker layer is more than about 3 feet (one meter) under it (Chismer, et al, 1995). The CPT may be used to first determine if such a weaker layer is present and within this distance.

Lifting the track and tampering ballast under the ties is the standard method used to restore the track to a smoothed profile. However, in some
cases the displacement of weak subgrade may cause tamping maintenance cycles to be so frequent that removing the soil and replacing with a stronger material to a certain depth is less costly than repeated tamping. This determination can be aided by the CPT which can be used to find the extent of the weak soil deposit with depth and distance along the track. If the weak soil deposit is relatively shallow and/or short, removal and replacement may be an attractive option.

The CPT can also be used to rule out the subgrade as a source of excessive track settlement. Often it is not known beforehand whether the ballast or subgrade is to blame for poor track geometry. High tip resistances in the subgrade can rule out soft subgrade as a cause and allow the track engineer to focus on other potential causes. Much expenditure has occurred in addressing the wrong problem such as attempting to increase the strength of an already stiff subgrade, while a degraded ballast is to blame for the rough track.

As mentioned, all of the alternative maintenance techniques, tamping is the most commonly used to obtain smooth track geometry. Raising the track and tamping more ballast under the ties is often attempted to increase the depth of ballast between the tie and the weaker underlying layer. However, the question seldom asked is, how much more ballast is required to reduce stresses on the subgrade to an acceptable level. A design method to limit the shear stress on the subgrade by designing the granular depth is available (Li and Selig, 1994) and has been shown to provide reliable results. However, using the CPT tip resistance to provide a measure of design shear strength ($S_u$) as in the equation

$$S_u = \frac{q_t - \sigma_k}{N_s}$$

is fraught with too much uncertainty since the factors which affect the empirical cone factor, $N_s$ (subgrade plasticity index, stress history, and in-situ lateral stress) are usually unknown and their determination is not a part of the investigation at this time.

Therefore, it was decided to develop the more empirical approach of comparing the CPT results in areas which are stable and where the track is rough. In this way a database is developed which can then be used to determine the value of $N_s$ with more confidence. With a reported range of $N_s$ from 10 to 25 or more (Lunne and Kleven, 1981), and considering the repeated loading environment and highly variable drainage conditions of the subgrade along the track, it seems prudent to use a value of about 20 until more data shows that a lower value is warranted.

CONCLUSIONS

Although the CPT is not the only tool needed to assess track substructure condition and cause of failure, it has provided a significant advance. The CPT, used along with borings, shallow excavations, and possibly more exotic techniques such as ground penetrating radar, can provide a comprehensive picture of substructure performance with a minimum of disturbance to the track and train traffic.

The CPT vehicle can provide information on subgrade conditions to allow assessment of alternative remedial techniques. Further developments of the CPT vehicle are being contemplated which will allow it to sample the subgrade at desired depths, and possibly to measure pore water conditions.

REFERENCES


Slot-type pore pressure CPT-u filters. 
Behaviour of different filling media.

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SYNOPSIS. In the CPT-u cone, there is a so called filter section, the purpose of which is to distinguish the generated pore water pressure from the mechanical earth pressure existing between the soil particles.

The measurement of generated porepressure outside CPT probes has been practised since 1974. The filter section used has often been constructed using some porous material. Another type of filter is constructed from steel only, with a tiny slot acting as filter. (Vlaarbloom 1975) and (Sidce 1984). This type of filter offers simplicity in the field, and some technical advantages. As with the porous filters, the absence of air is crucial but can be easier controlled, however it is obvious that the filling media in the slot and in the interior of the cone is of great importance. This paper describes some laboratory tests on different filling media in slot filters.

1. INTRODUCTION

The function of a slot filter while performing the task of distinguishing the pore water pressure from the total pressure is to break the structure of the soil so that only individual particles floating in the liquid can enter the slot. A typical slot filter appears in picture 1.

The geometry of the slot filter comprises two separate parts, the cone and the ring, and two O-rings. The material is normally hardened Stainless steel. Prior to filling, the cone and the ring are pressed together. The size of the slot will then be 0.3 mm. Note that the thread will be cut off by the two O-rings. If any bubbles are present in the thread, they cannot influence the pore pressure readings. The cavities A and B can be filled with the same or with different types of medium. Until now users have filled it with water or silicon oil. I have used water with approx. 1% Gelatine, Petroleum grease and water or Petroleum grease and Hydraulic oil. Since Gelatine has to be prepared in advance by boiling and cooling the filters, the grease fillings have become more popular by the users.

Experience from the field has however indicated that grease filling would introduce some hysteresis in the pore pressure measurement. A series of tests were performed in order to study the behaviour of these different combinations of filling media.

Fig 1. Slot filter (ENVI Memocone)
2. PRESSURE CHAMBER
The pressure chamber is constructed from steel, and capable of 10 Bar overpressure. The chamber is oil filled, but around the piezocone there is a rubber bag containing water. A reference pressure transducer is placed at the bottom. A dead weight pressure balance of type WIKA having an accuracy of better than 0.1% is used as pressure reference, and both the piezocone and the reference transducer are calibrated against this balance.

3. THE PIEZOCONE
A piezocone of type MEMOCONE has been used for the tests. The pressure sensing element consists of a stainless steel membrane with a strain gauge bridge inside, and a chopper stabilized amplifier that will amplify approx. 100 times. This type of amplifiers are very stable, but unfortunately they produce some noise on the signal. In order to reduce the noise, a low-pass filter is placed after the amplifier. The time constant is 0.05 sec.

Fig. 3. Calibration results
Note: all tests are made at 7°C

Fig. 4. Piezocone amplifier
This will cause a delayed reaction for pressure changes. However considering that the piezocone, penetrating at the normal rate of 2 cm per second, will travel only 1 mm during this time it will not affect the measurements in the soil significantly. Note the difference
between a reaction delay and hysteresis. In order to reduce influence from the delay, the low-pass filter has been removed in the tested piezocene.

![Graph showing pressure and hysteresis](image)

**Fig. 5. Amplifier delay**

4. STATIC MEASUREMENTS

The static measurements are made by pumping oil into the chamber until the weights of the balance are raised. The weights are rotated to overcome friction, and a stable pressure is achieved. The static hysteresis has been measured by altering the pressure in a sequence of 0-5-10-5-0 Bars. The different readings at 5 Bar, while increasing or decreasing the pressure, divided by 2 is then expressed as hysteresis +/-.

5. DYNAMIC MEASUREMENTS

The dynamic measurements are made by operating the piston by hand. This will cause pressure variations inside the chamber. These variations are picked up by the reference pressure transducer and by the piezocene. Data collection is made in a GEOPRINTER 60. This instrument can be programmed to make synchronised measurements on two channels exactly at the same instant. The sampling rate was 10 readings per second, and the resolution of measurements 1/1024. Data is collected in the internal memory, and the files can be downloaded into a diskette afterwards.

Before measurement, zero readings and scales are checked against the pressure balance. The datalogger is started, and a pressure wave is generated by pressing the piston.

The results are saved on a diskette and processed in a PC for presentation.

5. FILLING MEDIA

The different filling media investigated here are:

1. GREASE Type STATOIL LK 62
2. GREASE Type BP LT 2
3. GREASE Type ARIENS moly No 1
4. GELATINE MIXTURE 10 gram/litre
5. OIL, hydraulic oil

Type 1 grease was used earlier by ENVI and is relatively stiff. Type 2 grease is temperature stable, and is currently used by ENVI. Type 3 grease is very soft, almost liquid, and is currently used by NGI.

6. RESULTS FROM STATIC TESTS

The static pressure test results are shown in table 1:

Ref to fig 1.

<table>
<thead>
<tr>
<th>A cavity</th>
<th>B cavity</th>
<th>Hysteresis KPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water</td>
<td>Grease 1</td>
<td>+/- 12</td>
</tr>
<tr>
<td>Oil</td>
<td>Grease 1</td>
<td>+/- 1</td>
</tr>
<tr>
<td>Water</td>
<td>Grease 2</td>
<td>+/- 7</td>
</tr>
<tr>
<td>Oil</td>
<td>Grease 2</td>
<td>&lt; +/- 1</td>
</tr>
<tr>
<td>Water</td>
<td>Grease 3</td>
<td>+/- 1-2</td>
</tr>
<tr>
<td>Oil</td>
<td>Grease 3</td>
<td>&lt; +/- 1</td>
</tr>
<tr>
<td>Water</td>
<td>Gelatine</td>
<td>&lt; +/- 1</td>
</tr>
</tbody>
</table>

7. RESULTS FROM DYNAMIC TESTS

For swift changes of pressure, the hysteresis is greater. Fig 6 is an example where grease in combination with water causes a clearly visible hysteresis in the order of +/- 25 KPa. However, with the same grease in combination with hydraulic oil, the hysteresis is reduced to +/- 2-3 KPa. (Fig 7).

Grease 3 shows a better response. In combination with water +/- 12 KPa and with oil +/- 2 KPa. Gelatine and water shows excellent response also in the dynamic tests. See Fig 10. The hysteresis is less than 1 KPa which is the accuracy of the test equipment.

8. CONCLUSIONS

At any normal use of a piezocene, it should, in view of full scales of 1 - 2 MPa, be acceptable to use a grease/oil filling in slot filters. When...
ultimate accuracy is needed, there is the possibility to use Gelatine filling for these cases.

9. REFERENCES


Interpretation of Piezocone Results from Centrifuge Testing

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SYNOPSIS: A new miniature piezocone has been developed for the centrifuge testing center at University of Colorado at Boulder, to perform penetration testing on centrifuge soil models. In recent research, a series of penetration tests was performed in centrifuge models of soft clay. Shear vane tests were performed simultaneously with the cone penetration tests for assessing the undrained shear strength of the soil. From this research, correlations between undrained shear strength and penetration resistance, and correlations between the excess pore pressure generated during the penetration and penetration resistance are suggested.

1. INTRODUCTION
In centrifuge modeling, even after proper initialization has been followed, it is still necessary to verify that the correct soil profile in terms of strength and stiffness has been obtained. Thus, there exists the need for in-flight soil characterization, which can be carried out by performing cone penetration tests or shear vane tests on the soil model being spun in the centrifuge (Ko, 1988). In the last decade, much attention has been devoted to the piezocone (PCPT), which represents a new generation of in-situ testing devices. Piezocones are regarded as the most efficient tools for stratigraphic logging of soft soils, and this fact was the motivation for the development, design and construction of a miniature piezocone suitable for use in centrifuge testing.

In recent research carried out at the University of Colorado, several penetration tests were performed on normally consolidated and moderately overconsolidated centrifuge soil models, which were prepared to achieve certain property (strength, stiffness, overconsolidation, etc.) profiles. Shear vane tests were performed simultaneously with penetration tests, in order to obtain the undrained vane shear strength thus obtained for use as a reference. The goal of this research was to evaluate the results obtained from piezocone tests using existing methods of interpretation, and to find new methods of interpretation for piezocone tests (Esquivel, 1995).

2. CENTRIFUGE MODELS
Scale models in combination with theoretical analyses are frequently used in engineering. Scale modeling is often used when theoretical solutions have to assume major simplifications and approximations, or when numerical solutions are too lengthy or not feasible. This is the case for many geotechnical problems. Geotechnical scale models are also used when building and testing a full scale model is too difficult, dangerous or expensive.

In geotechnical problems, the soil behavior depends on the stress levels and on the stress history. Consequently, the stresses at a point in a
model should be the same as the stresses at the corresponding point in the prototype. With the aid of a centrifuge, it is possible to test models under conditions that approximate those in the field in terms of the in-situ stress profile. This is one of the reasons why centrifuge testing has been firmly established as a viable method of performing scaled modeling of earth structures, under conditions that approximate those in the field in terms of the in-situ stress profile (Ko, 1988). In centrifuge models, using different preparation procedures, it is possible to build different soil property profiles to simulate normally consolidated as well overconsolidated conditions. This technique is very promising in piezocone calibration tests.

In contrast to calibration chamber tests, in centrifuge models it is possible to incorporate many aspects, such as the effect of overburden pressure and the vertical stress gradient, and obtain results that realistically simulate prototype conditions.

3. EXPERIMENTAL EQUIPMENT

The experimental equipment used in this research consists of a miniature piezocene, a miniature vane and a penetration testing apparatus.

The design of the piezocene CUB1 was inspired by a penetrometer developed at University of Cambridge (Almeida, et al., 1985). The piezocene CUB1, schematically shown in Figure 1, is 305mm long and 12.7mm in diameter. The cone has a projected area of 127mm² and an apex angle of 60°. Two load cells are mounted at its extremities. The primary load cell, located inside the shaft, measures the point resistance. The pore pressure transducer located inside the primary load cell measures the excess pore pressure developed during the penetration, and its dissipation after the penetration. The secondary load cell, mounted at the base of the shaft, measures the total load applied to the piezocene. The secondary load cell allows an evaluation of the skin friction $f_s$ along the piezocene shaft.

Since one of the major applications of piezocone testing is assessing properties of soft and very soft clays, a main concern is the sensitivity of the load cells. Considering this fact, the CUB1 primary load cell was designed to measure the relatively low penetration resistance of soft clays. It has a resolution of 0.0005 kN for the range 0-0.5 kN. The high sensitivity is achieved by using semiconductor strain gages.

The pore pressure transducer used in the piezocene is a Druck pressure transducer (model PDCR81) with a silicon diaphragm, which ensures very fast response time. It is placed inside the primary load cell, as shown in Figure 1. The filter element is located at the base of the cone.

![Figure 1. Schematics of the piezocene CUB1.](image)

![Figure 2. Different filter configurations.](image)
Although they were not used in this study, cone configurations with the filter element located at the tip and at the face of the cone, respectively, also could be considered (Fig. 2). The cones can be easily replaced because they are simply threaded to the primary load cell. Whereas the piezocone shaft and the load cells are made of 7075-T6 aluminum, the cone is made of stainless steel, and the 10-micron filter element is made of sintered stainless steel.

The 400g-ton centrifuge at University of Colorado is equipped with a vane shear test apparatus for testing in flight. The dimensions of the blades are 12.7mm long and 6.3mm wide, giving a total width of 12.7mm.

The penetration testing apparatus has dual purposes. In a first phase during the tests, the setup serves as a slurry consolidometer. In a second phase, it supports the equipment necessary for penetration tests. The basic framework of the penetration testing apparatus consists of a steel base, two welded steel support frames and a rotary stand which supports the steel container. The steel frames are used for supporting the pneumatic loading system during the consolidation process, for helping to place the whole apparatus on the centrifuge platform, and for supporting the piezocone driving device.

The piezocone driving device consists of a Duff Norton linear mechanical actuator, model PKM-2500-16. The actuator is driven by an Electo-craft servo motor (model 64436-012). The position and the motion of the piezocone are controlled from a computer located in the centrifuge control room.

The purpose of the container rotary stand is to allow different penetration tests at different sites in the same tub of soil, without stopping the centrifuge for changing the position.

4. TESTING PROCEDURES

Soil specimens were prepared in such a way as to obtain a specified overconsolidation ratio at the gravity level under which the penetration tests are performed. Specimens were prepared by consolidating a slurry made of Speswhite fine china kaolin. This consolidation process consists of a preconsolidation phase at normal g-level and a reconsolidation phase in the centrifuge. The preconsolidation phase is performed by applying a combination of a surcharge load and a seepage gradient to the specimen. During the reconsolidation phase in the centrifuge, the pore pressures are continuously monitored. The specimen is considered reconsolidated when variations on the output voltage of pore pressure transducers are less than 1% per hour.

5. SPECIMEN DESCRIPTION

The specimens were prepared in order to represent typical soil profiles. Three specimens were preconsolidated in such a way as to result in normally consolidated conditions during spinning, and three specimens were prepared in such a way as to obtain moderately overconsolidated conditions during spinning. Table 1 summarizes the specimen characteristics.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Initial height (mm)</th>
<th>Final height (mm)</th>
<th>Nominal g-level</th>
<th>OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>562</td>
<td>300</td>
<td>20</td>
<td>6</td>
</tr>
<tr>
<td>E</td>
<td>590</td>
<td>295</td>
<td>50</td>
<td>4.5</td>
</tr>
<tr>
<td>L</td>
<td>498</td>
<td>293</td>
<td>50</td>
<td>1</td>
</tr>
<tr>
<td>M</td>
<td>463</td>
<td>290</td>
<td>75</td>
<td>1</td>
</tr>
<tr>
<td>N15</td>
<td>555</td>
<td>316</td>
<td>15</td>
<td>4-8</td>
</tr>
<tr>
<td>N50</td>
<td>555</td>
<td>316</td>
<td>50</td>
<td>1</td>
</tr>
</tbody>
</table>

6. PENETRATION TESTS

When the piezocone moves during a penetration test, the cone is subject to different g-levels. This means that the cone's own weight increases during the penetration, causing a decrease in the compression force acting in the piezocone load cell. Thus, the penetration force $F_p$ has to be corrected by adding a force $\Delta W$ equal to the variation in the cone's own weight variation. Corrections for this effect can be obtained by performing penetration tests in just water.
Penetration tests were performed at seven different sites along a circumference with a radius of 180mm and 45° apart. The distance between two adjacent penetration sites was 138mm, which corresponds to 10.8 piezocone diameters. At the end of each penetration test, a dissipation test was also performed.

The penetration resistance profiles showed good repeatability, meaning that any eventual boundary effect or interference from adjacent tests can be neglected while interpreting penetration test data.

7. RESULTS

Penetration test data have systematically shown that when the difference between the corrected penetration resistance and the total vertical stress \( q_r - \sigma_0 \) is plotted against the undrained shear strength \( s_u \) in log-log scale, the data points show a linear pattern. Performing regression analyses, the following correlation was found for normally consolidated specimens (Fig. 3):\[
\log(s_u) = -1.64 + 1.11 \log(q_r - \sigma_0) \tag{1}
\]

For moderately overconsolidated specimens, the following correlation was found (Fig. 4):\[
\log(s_u) = -1.24 + 1.05 \log(q_r - \sigma_0) \tag{2}
\]

Similarly, when the effective cone resistance \( q_e \) is plotted against \( s_u \) in log-log scale, the data points also showed a linear pattern. Performing regression analyses, the following correlations were found for normally and moderately overconsolidated specimens, respectively (Figures 5 and 6):\[
\log(s_u) = -1.62 + 1.13 \log(q_e) \tag{3}
\]
\[
\log(s_u) = -1.11 + 0.989 \log(q_e) \tag{4}
\]

For normally consolidated specimens, the following equation was found to correlate the excess pore pressure \( \Delta u \), generated during the penetration, and the quantity \( (q_r - \sigma_0) \) (Fig. 7):\[
\log(\Delta u) = -0.0726 + 0.868 \log(q_r - \sigma_0) \tag{5}
\]
The data corresponding to the moderately overconsolidated specimens resulted scattered and no correlation could be suggested. Therefore, further investigation is required.

8. CONCLUSIONS
This research has shown that piezocone testing in centrifuge models is feasible, producing consistent and reliable data. The performance of vane tests simultaneously with the penetration tests allowed the calibration of the piezocone in terms of the shear strength of the soil.

With the aid of a centrifuge, it is possible to test models under conditions that approximate those in the field in terms of the in-situ stress profile, including the effective vertical stress gradient. In calibration chambers, the state of stress is practically constant along the height of the sample. Due to this fact, penetration tests performed in centrifuge models can more readily generate a much larger data bank than penetration tests performed in calibration chambers. In
addition, the container rotary stand allows the performance of many penetration tests in one single specimen, while in a calibration chamber, the number of penetration tests is limited by the chamber size and cover constraints.

The large number of piezocone tests performed on centrifuge soil models allowed the assessment of correlations between the undrained shear strength $\phi_u$ and the quantities $(q_1 - \sigma_v)$ and $q_v$. Also, a correlation between the excess pore pressure $\Delta u$, generated during the penetration, and the quantity $(q_1 - \sigma_v)$ was suggested.

9. ACKNOWLEDGMENTS
The financial support provided by Coordenação de Aperfeiçoamento de Pessoal de Nível Superior (CAPES) and National Sciences Foundation was essential to the success of this research and is gratefully acknowledged.

10. REFERENCES


New ASTM Standard for Electric Friction Cone and Piezo Cone Penetration Testing of Soils

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SYNOPSIS: The American Society for Testing and Materials (ASTM) subcommittee D-18.02 on Sampling and Related Field Testing has been developing a new cone testing standard. The standard is currently in a society ballot and should be published by 1977. The standard is to replace existing standard D-3441 developed in the 1970s. The old standard included both mechanical and electric cones. The new standard is for right cylinder Fugro type electrical cones, finding almost exclusive use in North America today. The standard is overall agreement with the proposed international standard presented at the International Symposium on Penetration Testing held in 1988. In the new version there is guidance on manufacture, calibration, and maintenance. An annex gives details on calibration procedures. The draft has passed a main committee ballot.

1. INTRODUCTION
ASTM has been working on a new electric cone testing standard. As leader of the task group on cone penetration testing, I felt obliged to report on the progress and content of the standard. This standard will be much more useful, especially for those specifying cone testing under contract. It contains useful information on how quality cone operators conduct business. ASTM standards are consensus standards, and therefore there was significant participation from manufacturers, practitioners, and users.

2.0 BACKGROUND
The old ASTM standard was developed in the 1970's primarily through the efforts of John Schmertmann, of Schmertmann Craps, Gainesville, Florida. John was instrumental in introducing the cone penetrometer to North America. His cone penetrometer report for the Federal Highway Administration stimulated significant interest in our office and we had begun using mechanical penetrometers at that time.

ASTM standard D-3441 contained information on the use of mechanical cones and electric cones. Mechanical cones were the basic Delft cone and friction-mantle cone designs. The electric cones were either independent or subtraction design. The standard did not provide specific guidance on calibrations. The old standard simply stated to maintain measurement accuracy of within 5%. After experience with the calibration of electrical and mechanical penetrometers, the profession had developed specific calibration techniques for equipments. Some found that it was quite a challenge to build ones own penetrometer because of such factors as temperature compensation of the system. Electronic cones with downhole amplifiers also required special compensations. The calibration techniques adopted were in general accordance with those proposed by Schapp and Zuidberg (1983) at ESOP72.

3.0 DIMENSIONS AND TOLERANCES
The dimensions and tolerances adopted are in general conformance with those proposed in the International Reference Standard, ISOPT-1, (1988). This standard addresses only friction cones and piezocones. Variations in cone tip dimensions are given in Figure 1. The standard supports both 10 and 15 cm² cones.

Given below are some examples of tolerance requirements for the friction sleeve:

The friction sleeve should be located within 5
to 15 mm behind the base of the cone. The outside diameter of the manufactured friction sleeve and the operating diameter are equal to the diameter of the base of the cone with a tolerance of +0.35 mm and -0.00 mm. The friction sleeve is made from high strength steel of a type and hardness to resist wear due to abrasion by soil. Chrome plated steel is not recommended due to differing frictional behavior. The surface area of the friction sleeve is 150 cm² +/- 2 percent, for a 10 cm cone. If the cone base area is increased to 15 cm², as provided for in subparagraph 7.1.1., the surface area of the friction sleeve should be adjusted proportionally, with the same length to diameter ratio as the 10 cm cone. With the 15 cm tip sleeve area of 2.0 to 3.0 x 10⁻⁴ mm² have been used successfully in practice. This indicates that acceptable sleeve length to tip diameter ratio ranges from 3 to 5.

4.0 PIEZOCONE MEASUREMENTS

Three pore pressure measurement locations are supported. Most cone testing in North America is performed with the piezo element in the u₁ location as defined by Campanella and Robertson (1988). Within the standard there is guidance on typical practice for fluid preparation and saturation procedures. Below are some excerpts from the standard regarding piezo element;

The pore pressure measurement location of the porous element is limited to the face of the cone, u₁, directly behind the cylindrical extension of the base of the cone, u₂, or behind the sleeve, u₃. In the μ location a minimum 2.5 mm cylindrical extension of the cone tip, h₂, should be maintained for protection of the cone. Typical element thickness in all locations in the horizontal plane ranges from 5 to 10 mm.

5.0 OPERATIONS

Within the standard there is significant guidance on conditioning and technical precautions. Conditioning for piezoelements is addressed. There are many technical precautions to warn the user of potential unreliable data or impending damage to the equipments. Due to length restrictions they cannot be mentioned.

6.0 CALIBRATION REQUIREMENTS

In the standard there is guidance on calibration requirements for the penetrometers. Newly manufactured penetrometers should meet the requirements in the annex. These requirements are given in Table 1. For penetrometers in active use it is required to have some form of periodic load range calibration checks. For most small projects, it would be satisfactory to calibrate the penetrometer before and after a project. The requirements for penetrometers under active use are given in Table 2. The requirements listed on Tables 1 and 2 are normally easy to achieve with substation type penetrometers finding common use today.

7.0 REPORT AND CALCULATIONS

In the reporting section, optional pore pressure parameters such as the pore pressure ratio and pore pressure parameter ratio, Bₚ as defined by Wroth (1988) can be calculated and reported.

8.0 SUMMARY

This standard had many contributors, for which, I am grateful for their assistance. The following is a partial list of contributors: R. Campanella, M. Jeffries, R. Yilmaz, J. Axon, T. Nolan, R. Yilmaz, H. Zuidberg, J. Schmertmann.

We will need volunteers to write mechanical, seismic, and resistivity cone standards in the future. This standard should be published in 1997 of the ASTM volume 4.08. Revisions can be balloted as the need arises. Please contact me to participate in future standards development, or to get advance copies, or to suggest future revisions. J Farrar, U.S. Bureau of Reclamation, PO Box 25007, Denver CO, 80225, USA, (303)-236-3730x416, FAX 236-4679, jfarrar@do.usbr.gov.

9.0 REFERENCES

"Reference Test Procedure for the Cone Penetration Test (CPT)," Proceedings of the First International Symposium for Penetration


**Table 1 - Summary of Calibration Tolerances for Newly Manufactured Cones**

<table>
<thead>
<tr>
<th>Calibration Parameter</th>
<th>Element</th>
<th>Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zero-load error</td>
<td>Cone and sleeve</td>
<td>≤ ± 0.5% FSO</td>
</tr>
<tr>
<td>Zero-load thermal stability</td>
<td>Cone and sleeve</td>
<td>≤ ±1.0% FSO</td>
</tr>
<tr>
<td>Nonlinearity</td>
<td>Cone</td>
<td>≤ ± 0.5% FSO</td>
</tr>
<tr>
<td>Nonlinearity</td>
<td>Sleeve</td>
<td>≤ ± 1.0 % FSO</td>
</tr>
<tr>
<td>Hysteresis</td>
<td>Cone and sleeve</td>
<td>≤ ±1.0% FSO</td>
</tr>
<tr>
<td>Calibration error</td>
<td>Cone</td>
<td>≤ ± 1.5 % MO at &gt;20% FSO</td>
</tr>
<tr>
<td>Calibration error</td>
<td>Sleeve</td>
<td>≤ ± 1.0 % MO at &gt;20% FSO</td>
</tr>
<tr>
<td>Apparent load transfer</td>
<td>Sleeve transfer</td>
<td>≤ ± 1.5 % FSO of Sleeve</td>
</tr>
<tr>
<td>Apparent load transfer</td>
<td>Cone transfer</td>
<td>≤ ± 0.5 % FSO of Cone</td>
</tr>
</tbody>
</table>

**Table 2 - Summary of Calibration Tolerances for Cones Under Production**

<table>
<thead>
<tr>
<th>Calibration parameter</th>
<th>Element</th>
<th>Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zero-load error</td>
<td>Cone</td>
<td>≤ ± 0.5% FSO</td>
</tr>
<tr>
<td>Zero-load error</td>
<td>Sleeve</td>
<td>≤ ±1.0% FSO</td>
</tr>
<tr>
<td>Nonlinearity</td>
<td>Cone</td>
<td>≤ ± 1.0 % FSO</td>
</tr>
<tr>
<td>Nonlinearity</td>
<td>Sleeve</td>
<td>≤ ± 2.0 % FSO</td>
</tr>
<tr>
<td>Calibration error</td>
<td>Cone</td>
<td>≤ ± 2.0 % MO at &gt;20% FSO</td>
</tr>
<tr>
<td>Calibration error</td>
<td>Sleeve</td>
<td>≤ ± 3.0 % MO at &gt;20% FSO</td>
</tr>
<tr>
<td>Apparent load transfer</td>
<td>Sleeve transfer</td>
<td>≤ ± 2.0 % FSO of Sleeve</td>
</tr>
<tr>
<td>Apparent load transfer</td>
<td>Cone transfer</td>
<td>≤ ± 1.5 % FSO of Cone</td>
</tr>
</tbody>
</table>
Figure 1. Manufacturing and Operating Tolerance for Cone Tips
Subdivision of Soil Massifs into Types of Soils

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SYNOPSIS: The most important feature of soil massifs is lithology and the character of soils bedding. The new technique of lithological subdivision of soil massifs based on 3 parameters, two of which are the traditional ones: the soil resistance to the probe cone \( Q_a \) and the skin friction \( f_s \) as well as the natural gamma-phon of the soil \( l \) is being proposed. The equipment and the technique of soils investigation are being developed on the basis of the CPT. The complex criterion \( X \) is obtained which allows to redefine the process of lithological subdivision of soil massifs.

One of the methods of estimate engineering geological properties of soils is static probing which determines soil resistance under the cone of the probe \( Q_a \) and on the coupling of friction \( f_s \). Each of these parameters and their relationship are used for the estimate of composition, physico-mechanical properties of soils and their heterogeneity as well as for the calculation of bases and foundations. N.K. Begeman /1/ determined the criterion which on relationship between the parameters \( Q_a \) (soil resistance to the cone of the probe) and \( f_s \) (soil friction on the coupling of the probe) allowed to subdivide sands, silty sands, loams and clays. V.I. Ferenskov /2/ proposed the new technique of obtaining these parameters \( Q_a \), \( f_s \) and \( l \) (natural gamma-phor of the soils) by means of static probing. This technique allows to determine the type of the soil.

The technique of subdividing soil massifs on the basis of the \( Q_a \), \( f_s \) and \( l \) parameters has been further developed in NIIGSP /3/. The field test measuring set of apparatus PIRA-10 has been designed for measuring these parameters. There was also redefined the criterion of \( \frac{f_s}{Q_a} \) and proposed the complex criterion \( X \) for determining the type of soil with the use of the abovementioned parameters. The PIRA-10 set can be applied with any equipment for static sounding as well as with any other sinking apparatus without principal modification of their design or technique of work.

The PIRA-10 set consists of the probe with measuring elements \( Q_a \), \( f_s \) and \( l \), registering amplifier-transducer, the cable line between them and the measuring element of the depth of sinking. The probe design meets the requirements of the 20069-81 GOST (the State Standard) /4/. The results of measuring are being transmitted on the digital display of the
recording device. The electronic unit of the probe provides the high accuracy of measurement as well as the operation with a multichannel cable of any length, the normalizing response of the probe due to which it became possible to change probes in field without any additional calibration. The range of soil resistance measurements:

- $Q_s, \text{M}\Omega - 0.5 \ldots 60$;
- $f_s, \text{kPa} - 5 \ldots 600$;
- the principal error
  - $t(5+0.5P_{20}/P_x)%$;
- the indications are being read in 10 s after forcing the probe down.

The range of measurement $I$, imp/sec - 1...999; the error - $+1\text{imp/sec}$; the measurements are made in each 0.5 or 1.0 m of sinking while halting the probe for 30 or 60 sec.

The regime of measurement of gamma-phon depends upon the required accuracy of investigations. For extending the $Q_s$ and $f_s$ measurements range the principal decisions are provided which allow to increase the response of the low part of the range as well as to boost the upper limit of measurements. The sinking rate of the probe - $1 \pm 0.3$ m/min., the depth of sinking - up to 30 m.

The experimental operations had been performed by an apparatus for static sounding C-652 of NIIPromstrov design equipped with the PIRA-10 set at a site in Volgograd city.

For testing the repeatability of obtained measurements the trial sounding at 3 points at a distance of 1.5 m from each other had been carried out. The figure 1 shows that the obtained results are in good agreement. The depth of sounding was usually 10-17 m. The modern technogenesis deposits, the deposits of the Chvalyn and Chazar stages of the Quaternary, had been driven.

![Diagram of sounding](image)

Fig.1 The diagrams of sounding, obtained at 3 points of the surface, located at a distance of 1.5 m from each other: a - for the $Q_s$ parameter; b - for the $f_s$ parameter.

Chvalinsky deposits up to 10 m in thickness are presented in the upper part by clayey sands of brownish-green colour, fluid; they are underlined by the clay of chocolate colour, tightplastic, with the pockets of fine sand. The lower part of the Chvalinsky deposits is presented by sandy, tightplastic loam of grey or light-grey colour.

Chazar deposits are presented by the sand, grey, fine, dense or dusty, about 2 m in thickness. In some places the sand is absent. The sand is underlined by the tight-or softplastic clay of grey colour with brownish shade. The thickness of clay by the results of sounding accounted for 3-4 m.
The given description shows that all the principal types of soils are presented among all the investigated dispersed soils: sands, clayey sands, loams and clays. Thus, the technique and the sounding results testing had been carried out on the basis of rather representative and various soil material. The points of sounding were located in close proximity to the boreholes having been bored with the core chiseling, by which the types of soils have been determined in accordance to SNIP 11-15-74 /5/. This was the test for determining the soil type by the results of sounding with the use of PIKA-10 set. The example of sounding in Volgograd city is shown in figure 2.

![Diagram](image)

**Fig. 2.** Geological-lithological cut carried out on the basis of static sounding (Volgograd city).

I - stratigraphic index;
II - the depth of the foot layer, m;
III - the layer thickness, m;
IV - the lithological description (A - the earth fill, crushed rock, clay; B - clay sand, brown-green, fluid; C - loam, tightplastic; D - clay of chocolate colour, tightplastic with layers of...
sand, E - loam of light-brown colour, tightplastic with sands inclusions; F - clay of brown-grey colour, tightplastic with pockets of sand; from the depth of 11 m it's fluidplastic).

V - the cut; V1 - the depth, m.

On the basis of static sounding parameters, the criterion $t = f(r/Q)$ is usually being determined. According to the instructions on sounding soils for the construction purposes with the use of the criterion the sandy and clayey soils are being distinguished. The soils with $Q > 10$ MPa by $L < 0.005$ are related to sandy soils; the soils with $L > 0.01$ irrespective of the $Q$ value are related to clayey soils. The conducted investigations save the sufficient statistic material (about 300 determinations of $f$ and $Q$) for redetermining the $t$ criterion and allowed to distinguish clayey sands and loams besides sandy and loamy soils (fig. 3). The use alone with the $Q$ and $L$ parameters the data on the natural gamma-phon for the $I$ soil (about 100 determinations) allowed to determine the type of the soil by the sounding results even more well-grounded.

Figure 3. The relationship between the specific soil resistance under the pirobe cone $V$ and one on the coupling of friction $f$ for different types of soils (the upper- and the middle Quarantine deposits in Volgograd Povolzye): 1 - sand, 2 - sandy loam, 3 - loam, 4 - clay.

The natural gamma-phon of the $I$ soil can be expressed as a dimensionless relative value $F = 1/I_{10}$, where $I_{10}$ - any initial gamma-phon of the soil which is considered to be equal 10 imp/sec for the given region.

For the purposes of mathematical processing and the most complete characteristic of the soil the results of static sounding and the natural gamma-phon determination can be expressed as one value while considering it as a complex criterion $X = t(I/10)$ (table 1).

<table>
<thead>
<tr>
<th>The type of the soil</th>
<th>Criterion $t = f(r/Q)$</th>
<th>Natural gamma-phon of the soil $I$, imp/sec</th>
<th>The complex criterion $X = t(I/10)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>sand</td>
<td>$L &lt; 0.012$</td>
<td>13 $&lt; I &lt; 19$</td>
<td>$t &lt; 0.0156$</td>
</tr>
<tr>
<td>clayy sand</td>
<td>$0.012 &lt; L &lt; 0.020$</td>
<td>19 $&lt; I &lt; 25$</td>
<td>$0.0156 &lt; t &lt; 0.0380$</td>
</tr>
<tr>
<td>loam</td>
<td>$0.020 &lt; L &lt; 0.030$</td>
<td>25 $&lt; I$</td>
<td>$0.0380 &lt; t &lt; 0.0823$</td>
</tr>
<tr>
<td>clay</td>
<td>$L &gt; 0.030$</td>
<td></td>
<td>$0.0823 &lt; t$</td>
</tr>
</tbody>
</table>

The determination of the soil type on the basis of the criterion $t$ and the natural gamma-phon which is expressed as a dimensionless value $I$ is being performed according to a diagram (fig. 4) (the fields of distribution of different types of soils are formed by the rectangles limited by the
It should be stressed that the values of relationships between the parameters of sounding, the natural gamma-phon and the type of the soil having been obtained for the Quaternary deposits of the Volga region (Table I) can't be considered as universal. These relationships may be different in regions where soils have different mineral composition, different content as well as the composition of organic matters. That's why the given relationships between the parameters of sounding, the natural gamma-phon and the type of radioactive elements in the soils determining their gamma-phon is in a close connection with the organic matter content in the soils and their composition as far as the organic matters favour the accumulation of radioactive elements. One of the most widely spread radioactive elements is uranium. V.A. Maximovsky and V.N. Solantsev /7/ showed the close correlation between the content of uranium and organic carbon (fig. 5, 6).

\[ U, 10^{-4}_g; \text{Corg, } \% \]

\[ C_{org}, \% \]

Figure 5. The content of uranium and organic carbon C org. in rocks of different types of precarboniferous silic-shale formations in Sikhote-Alin mountains /7/ a - in percent of initial rock; b - in % of silicless matter; I - sandstones; II - alveol-to-clayey shales; III- silic-clayey shales; IV- phillites; V - jaspers.

Figure 6. The relationship between the uranium content and the organic carbon content in the rocks of precarboniferous formations of Sikhote-Alin /7/.
This correlation is clearly seen both for natural rocks of different types and for their silicious matter. Above that there is the correlative connection between uranium and organic carbon of different origin (for example of humus and sapropelic origin) existing in soils. The uranium content is significantly more in organic matter of humus origin rather than of sapropelic one. (fig.7.)

The organic matter of humus origin is characteristic for peats and peaty soils; sapropelic organic matter is characteristic for freshwater and sea silts.

Figure 7. The relation of content of humus carbon (1) and sapropelic (2) organic matter as well as uranium /μγ/ 7.6./.

The close connection of uranium with organic matter for most types of sedimentary and igneous rocks for all stages of the Earth geological history is shown in the work by S.G. Moruchov /9/. The content of radioactive elements in soils also depends upon their mineralogical composition /5/.

Organic matters not only favour the accumulation of radioactive elements but also influence significantly on plastic and strengthening properties of soils /10/. These properties influence greatly the parameters of static sounding. That's why the organic matters as well as the mineral composition of soils determine all the relationships between the data of sounding, the natural gamma-phon and the type of the soil. This should be taken into account by the establishment of the values of the soils type criterion by the results of sounding with the application of the FKA-10 set for each new region of investigations.

The Conclusions:

1. The conducted investigations allowed to redefine the criterion of the soil type what gave an opportunity to distinguish sandy loams and loams in addition to sandy and clayey sands having been distinguished earlier.

2. The complex criterion is proposed for the determination of the type soil with the using of static sounding parameters 4a and 4b and the natural gamma-phon of the t soil.

3. The mineral composition of soils as well as the content and composition of organic matters existing in them influence on plasticity, strength and gamma-phon of the soils. In this connection the value of the criterion for the same type of the soil may be different in different regions depending upon the features of the soils composition. In the area of one region the values of the criterion are practically constant for each type of the soil.
The Use of the Laser Induced Fluorescence Cone for Environmental Investigations

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SYNOPSIS: The combination of two proven technologies, the Static Cone Penetrometer and Laser Induced Fluorescence, provides a powerful geo-environmental investigation tool. In addition to the standard cone penetrometer sensors, hydrocarbon contamination of the ground is detected in situ using laser light. A case history of the use of the tool to investigate an infilled void is presented. The infill comprises variable mixes of soil, ash and tar wastes. The data collected is validated against the information gathered from a sampling programme. It is shown that calibration of the laser for site specific conditions is important. There is a need to consider all three CPT sensor signals when developing a ‘signature’ for tar contaminated zones. The degree of contact at the laser window/soil interface is also important. A tentative inverse relationship between CPT friction ratio and fluorescence is presented. The tool is shown to provide excellent data on the presence of hydrocarbon contamination and has also exhibited potential in providing details on the concentration and hydrocarbon type.

1. INTRODUCTION
Environmental legislation within the European Union is becoming more stringent. Part of this legislation relates to the protection of soil and groundwater resources. Many countries have a policy of sustainable redevelopment and the need to build on ‘brown-field’ sites. Hence there is an increasing emphasis to evaluate potentially contaminated land. Until recently investigative techniques have been adapted from standard geotechnical site investigation practices, essentially involving drilling, sampling and laboratory testing. This paper describes the use of a new probe technique the laser induced fluorescence (LIF) cone penetrometer which is used to investigate land with potential hydrocarbon contamination. It is used to give a rapid evaluation and allow focus and/or reduced scope for the necessary invasive sampling work for validation by chemical analyses. The LIF cone penetrometer system is the combination of two proven technologies.

2. TECHNOLOGY
Laser Induced Fluorescence (LIF) is an established analytical technique used to determine a substance’s molecular make up. When a material is bombarded with laser light (photons) of a certain frequency (wavelength) it’s molecules absorb some of the photons. This causes the molecules to go into an unstable excited state which cannot be maintained. The molecule reverts to a stable state by emitting one of it’s photons. This emission is known as fluorescence. The particular molecular type is encoded in the fluorescing photons. The concentration of the molecules is encoded in the measured fluorescence intensity (FI) being the number of photons.

Recent developments in laser technology have permitted the deployment of a tuneable and portable laser system into the field since early 1994. Originally a more simple nitrogen laser unit was first used in the field in the USA in the late 1980s. The present system was
developed in the early 1990s and has been used in the field to accurately map hydrocarbon (HC) contamination such as petrol, aviation fuels, diesel, heating oils and coal tars. Earley and Rapp (1995) illustrated the technical and cost benefits of the LIF cone when compared with soil gas and drilling with sampling surveys.

The Static Cone Penetration Test (CPT) has been used for over half a century to investigate in situ soil conditions and foundation design parameters. There is a wealth of technical data covering the use of the CPT in soils and interpretation of the data collected (Meigh, 1987). The power of the CPT is its applications in fill materials and non-textbook soils (Lunne et al. 1995). The combination of the standard CPT and the LIF equipment has produced a powerful geo-environmental investigation tool, the LIF cone penetrometer. Apart from the additional hardware mounted in the CPT vehicle the only additional equipment pushed into the ground is a steel module with a sapphire window mounted behind the standard cone. Two optical fibre cables are strung with the normal electrical CPT umbilical cable. One transmits the laser light via a prism to the sapphire window and the second returns any fluorescence to the detection systems in the CPT vehicle. The detection system accurately measures the fluorescence intensity, wavelength and the lifetime of the photons.

3. SITE DESCRIPTION
In December 1994 the LIF CPT was used as one of the evaluation techniques for a contaminated UK site. The location was a former mining area where subsidence of shallow worked seams had created a large void in the ground. This had been back-filled and it was evident from seepage that tar type hydrocarbons were present. Contamination dated from works which operated in the mid 19th century.

A historical review was undertaken first. Evidence from aerial photography showed the void to be at least 7m deep with an area of some 4,500m². The underlying geology of the area is an interbedded sandstone and shale bed-rock overlain by glacial till deposits, typically 2m thick. The natural rock-head slopes gently south eastwards and the groundwater table lies close to the till/rock boundary. Close to the infilled void, the area is under drained by the mine system.

4. PREVIOUS INVESTIGATIONS
Initial studies were undertaken using a percussive ‘window-sampler’ to investigate the desk study information and the lateral extent of the contamination. The window sampler holes confirmed the presence of buried tars but due to the limited penetration the vertical extent of the contamination required further definition.

5. PROPOSED INVESTIGATION
The information collected from the desk study and limited ‘window sampler’ work was used to plan the LIF CPT work. An irregular grid of 16 probe holes at 20m spacing penetrated the void. The LIF cone penetrometer was pushed to depths up to 15m. In addition to the CPT work, 2 sample holes were sunk using the CPT vehicle’s hydraulics to push the ‘Mostap’ discrete sampler. This type of sampler recovers samples from known depths without risk of cross contamination occurring during sampling. The sample locations were chosen using the data produced on site with the LIF cone data. Figure 1 shows a plan of the investigation extent.

6. THE LIF CONE SYSTEM
The technical and fieldwork advantages of the LIF CPT probe technique over the conventional drilling/sampling approach include:

- real time in-situ data on HC contamination
- minimise human contact - no cuttings or soil disposal
- high resolution of stratigraphic and contamination data ~2cm
- rapid delineation - up to 100 metres of profiling in a day

In addition to the normal CPT data collected from the selected cone penetrometer type the LIF data is collected simultaneously at 2cm depth intervals. During the penetration of the LIF cone, the fluorescence data is presented as fluorescence versus depth (FVD).
Prior to each test the LIF system is standardised against a known hydrocarbon source placed against the sapphire window. This standardisation takes into account variations in the laser energy being delivered to the window. It also permits the data from all the tests at a site to be directly comparable. Details of the type of hydrocarbon product can be determined by temporarily stopping the penetration (for 1 to 2 minutes) in a contaminated zone and collecting detail on the fluorescence wavelength, lifetime and intensity as a wavelength lifetime matrix (WTM). The WTM signature is unique to particular hydrocarbon and typical signatures are shown in Figure 2.

7. LIF CPT FINDINGS
This section discusses the findings of the investigation based on the LIF and 'Mostap' sampling work. It draws upon all analytical results from this and previous work. The investigation revealed that the void was up to 12m deep. Figure 3 shows an interpreted longitudinal cross-section of the filled void, and shows that the backfill is very variable. It comprises ash and clay together with a tarry organic fraction. Analysis suggests that this tarry material can be divided into two broad types shown in Table 1.
Table 1. Composition of tars

<table>
<thead>
<tr>
<th></th>
<th>Solid Fraction (%)</th>
<th>Organic Fraction (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type 1</td>
<td>20 - 30</td>
<td>80 - 70</td>
</tr>
<tr>
<td>Type II</td>
<td>70 - 95</td>
<td>30 - 5</td>
</tr>
</tbody>
</table>

For both material types the hydrocarbon proportion of the organic fraction is typically 15% with the remainder comprising polar compounds, being hydrocarbon with combinations of nitrogen, sulphur and oxygen. The hydrocarbon fraction is essentially heavy end deposits with the majority of compounds being C20 to C35 material and comprising polycyclic aromatics, PCA’s. These PCA’s and associates polar compounds have a strong fluorescence potential with the LIF cone.

Tuning of the laser wavelength for site specific conditions is important. An initial understanding of the potential contaminant chemistry allows selection of the optimum laser excitation wavelength. Optimisation of this with site specific samples was found to be beneficial. While this is not particularly critical for compounds which display strong fluorescence potential, such as PCA’s, it is likely to become more important as fluorescence potential decreases.

Interpretation of the data from this site showed that it was important to consider all three CPT sensor signals cone end resistance (qc), sleeve friction (Fs) and FI, when developing a ‘signature’ for contaminated zones. This allows better identification of the contaminant type and of clean soil. For this case study the Type I and II tar material signatures were different. This is simply illustrated by Figure 4 which plots friction ratio (fr) against FI and shows an inverse relationship. The fr is the quotient of the Fs and qc. It is considered representative of the physical properties of the materials at this site.

The Type I tarry material is characterised by high friction ratio and relatively low fluorescence intensity. The FI measured is apparently at variance with its known high
organic carbon content and strong fluorescence potential. However, consideration must be given to the material physical properties. It has a low degree of mobility and is extremely viscous. Values of qc in this material are low (typically <1MPa).

The Type II material contains tar which is physically stable due to a high solids fraction comprising mainly ash. This dominates the CPT signature giving variable qc (peaks typically 5MPa) and relatively low fr. The relative density of the tar/ash mix is judged as medium dense. FI is generally in the range 50 to 100%.

For a possible explanation of the different material signatures an investigation into the physical behaviour was made. The lower friction ratio of the Type II material is due to the high granular (ash) content and is consistent with established CPT behaviour. The material particles 'collapse' into pore space as part of the shearing process associated with the penetrating cone. This displacement creates a 'film' of contaminant and water at the cone/soil boundary as viscous effects do not allow the displaced volumes to drain away quickly enough. Consequently a relatively smooth contaminant surface is present at the laser sensor window. This minimises the amount of laser light scatter resulting in high FI values. The Type I material is much more viscous. the cone sleeve friction in this high tar material appears similar to that of the Type II material since limiting values apply. However, the lower granular solids content and pliable nature of the material gives a lower qc. This combination results in a higher friction ratio. The highly viscous nature of the tar results in it not flowing back tightly around the cone immediately after shearing. Hence, with less water present and tar material which does not flow readily, a relatively rough surface is presented to the laser window. This allows more scatter of the emitted laser light giving low FI values. Hence, for the tarry type hydrocarbons FI alone may not be a good indicator of contaminant concentration.

The results are consistent with other work using the LIF CPT, Bratton et al. (1993) have shown that the calibration and limits of detection of the LIF CPT is affected significantly by the soil type, grain size, oxidation and the presence of natural organic compounds. For a given contaminant
concentration, FI values are generally greater in a granular soil than a cohesive soil. This is due to pore space volume effects. Al-Sana et al. (1995) examined the effect of oil contamination on the geotechnical properties of sand. It was shown that while oil contamination led to decreased permeability and strength and increased compressibility, the changes were not significant. It was also demonstrated that the lighter the hydrocarbon fraction, the less the effect. Thus the tarry materials described here are likely to affect the cone penetrometer and fluorescence behaviour more than most of the other contaminants. With further data it should be possible to modify the CPT interpretation guide (Meigh, 1987), where soil type is given as a function of cone point resistance and friction ratio, by the addition of a third axis representing fluorescence intensity.

In addition to contaminated soil de-lineation the CPT identified the stratigraphy and material types. It allowed the boundaries between different overlying fill types to be detected. Some of these were shown to be the lines of weakness along which the ‘mobile’ Type I tar migrated to the surface. It also showed that the majority of the void was underlain by intact glacial clay which aided qualitative risk estimation.

8. CONCLUSIONS
The combination of traditional techniques and the LIF CPT has allowed an evaluation of the probe technology. Potential advantages of the tool are rapid de-lineation of soil properties and any contamination, while minimising human contact. The LIF CPT data are validated using the CPT vehicle mounted discrete sampler. Tuning of the laser wavelength for site specific conditions is important, especially for those compounds with limited fluorescence potential.

Interpretation has shown the need to consider all three CPT sensor signals (cone end and sleeve resistance plus FI) when developing a 'signature' for contaminated zones. The degree of contact at the laser window/soil interface is also important. Here the amount of scatter of the laser light affects the measured FI. Based upon the limited data presented an inverse relationship between CPT friction ration and FI is presented. For tarry type hydrocarbons FI alone appears a poor indicator of contaminant concentration due to the material physical properties. The LIF CPT successfully profiled the hydrocarbon contamination of the infilled void allowing a cost beneficial corrective action strategy to be devised. Although the present database is limited the tool shows much promise in providing insitu quantitative data on hydrocarbon contamination.

9. REFERENCES
Use of a thin slot as filter in piezocone tests

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SYNOPSIS: A new method for measuring the pore pressures using a thin slot instead of a porous filter has come into use. As pressure transmitting media between the pore water in the soil and the pressure transducer inside the cone, various gels and fluids have been used. The method has become popular and gained widespread use because of its relative simplicity and also because the results in measured pore pressures appeared to be similar to those obtained with more elaborate methods. However, the demands in the standards for accuracy of the measurements and particularly of the pore pressure measurements are high for various reasons. A detailed investigation into the function and accuracy of the "slot filter" and its possible shortcomings and advantages has therefore been made at the Swedish Geotechnical Institute. This investigation has resulted in recommended guidelines and certain limitations for using the method.

1. INTRODUCTION
The use of piezocone tests for routine investigations has earlier been hampered in Scandinavia, partly by problems associated with use of sensitive electronics in the field, and partly because of the extensive preparations required in connection with the demands for saturation of the pore pressure measuring system. This has especially been the case in regions with difficult climatic conditions, long distances to base facilities and limited access for heavy vehicles in the terrain.

The problems associated with use of electronics and lightweight unprotected equipment in difficult conditions in the field have to a large extent been solved. The comprehensive preparation work for deaeration of filters and fluids and saturation of the measuring system in the ordinary way of measuring pore pressures, often in combination with cold and bad light, has largely remained even if it has also been possible in certain respects to simplify this procedure.

2. METHODS OF PORE PRESSURE MEASUREMENT IN PIEZOCONES TESTS
The pore pressure measurement is made by a pressure transducer inside the cone, which is to be brought into stiff contact with the pore water in the soil. This contact is normally provided by a non-compressible fluid in the cavities inside the cone and a saturated filter. The filter prevents soil particles from intruding into the cavities and separates the pressure in the pore water from the effective stresses in the soil. Great care has to be taken to first deaerate the fluid and filters and to keep them free from air and then to completely saturate the cone at assembly and to keep the system saturated during the penetration test. The first is time consuming and the second can be very difficult in soils where negative pore pressures are created during the penetration and the fluid tends
to be sucked out of the cone. The latter always results in incorrect pore pressure measurements in a large depth interval below the level where the full saturation is lost. In many cases all further pore pressure measurement will be impossible because the filter becomes blocked by remoulded soil which is pressed into its emptied voids.

A new method, the "slot filter", was introduced in the 80's (Sidery 1984, Elmgren 1987). In this method, the porous filter is replaced by a thin slot, which must be wide enough to transmit the pore pressure and thin enough to prevent soil particles from intruding into the slot during penetration. Usually, the slot is about 0.3 mm wide. Gelatine or grease were suggested as pressure transmitting media. In most cases grease has been used. This is normally free from air and firm enough not to flow out of the slot and cavities. Because of the design of the cone, grease has not been used in all the cavities but only inside the removable tip and in the slot between this and the other parts of the cone. The innermost cavity at the transducer, where extra care and a viscous fluid may be required to remove all the air bubbles, has usually been filled with water. To fill the inner cavity with water and to press grease into the cavities in the cone tip is a simple operation and the cone automatically becomes fully saturated when the tip is screwed on and excess grease is pressed out through the slot. The cone can thereafter be handled on the ground without extra arrangements for maintaining saturation because the stiffness of the grease prevents both grease and entrapped water from flowing out. Also during the penetration tests, this type of system often appeared to be better for maintaining saturation when passing unsaturated zones and layers where negative pore pressures are created.

3. INVESTIGATIONS ON THE FUNCTION OF THE "SLOT FILTER"

An investigation has been performed at the Swedish Geotechnical Institute in order to study the function of the slot filter and possible effects on the measured parameters, (Larsson 1994). This investigation has mainly concerned the most commonly used method with a combination of grease and water as pressure transmitting medium. The investigation comprises laboratory tests in a calibration chamber to study the behaviour of the measuring system as regards principle and comparative field tests in very homogeneous deposits with soft clays.

3.1 The function of a slot filter filled with grease according to laboratory tests

The slot filter was tested in a special calibration chamber for piezocones, (Malabadié et al 1990), in which the cone can be tested regarding responses to both static and transient changes in external pressure. In the tests, the slot was varied between 0.1 and 0.3 mm and greases with varying firmness were used. No recommendations for what type of grease should be used had been given previously and in practice any available type had been used.

Most of the tests in the laboratory were made with a so called "universal grease", which is a relatively soft grease and easy to press into the cavities in the cone tip. In static tests, it was found that the pressure transducer inside the cone did not respond to increasing external pressures until a certain threshold value had been exceeded. Thereafter, it registered values which corresponded to the applied pressure minus this threshold value. At a following decrease in external pressure, the decrease was not fully registered until the external pressure was correspondingly less than the internal pressure whereupon values corresponding to the external pressure plus the threshold value were registered. When the external pressure returned to atmospheric pressure there was a remaining excess pressure corresponding to the threshold value inside the cone. If a temporary suction was applied on the outside, a corresponding remaining negative pressure was also observed.

At increasing and decreasing external pressures a hysteresis loop was thus found with a measurement error of ± the threshold value.
The average of this loop could also be displaced in parallel to the calibration curve depending on at what stage of assembly the zero reading of the internal pressure transducer was taken since remaining pressures of ± the threshold value could be built in during the assembly, Fig. 1.

![Diagram](image)

**Fig. 1. Schematic relation between external pressure and recorded pressure when a slot filter filled with grease is used.**

From the results, it becomes obvious that when using grease or any other type of semisolid gel a certain pressure difference between the outside of the cone and its interior is required for the grease to move in a sufficient degree to transmit the pressure changes to the pressure transducer. This pressure difference was about ±20 kPa for the actual universal grease at room temperature. This value could be diminished by using a softer grease and increased considerably when a firmer grease was used. The resulting measuring error can be up to twice the threshold value depending on what pressure is built into the system during assembly before the zero reading is taken.

Comparative tests with slot widths varying between 0.1 and 0.3 mm did not show any significant differences.

The function of the system when the cone is subjected to negative pressures was also studied. In corresponding tests with water and glycerine, gas (boiling) usually starts to develop at a pressure of 70 ± 80 kPa below atmospheric pressure whereby the saturation is lost; the boiling being considerably more violent in water. The corresponding limit when using slot filter and grease becomes lower because boiling does not normally occur in the grease unless air bubbles have been entrapped at assembly. Boiling occurs in the water when the external pressure transmitted through the semi-solid grease has created a negative pressure of about 70 to 80 kPa in this part. For the particular universal grease, this means that boiling did not occur until the external pressure was close to vacuum. The study thus shows that a cone with a grease filled slot can be subjected to larger and more long-term negative pressures than with a fluid filled filter before loosing its saturation. This advantageous effect can be increased by using a firmer grease, but then the measuring accuracy decreases to a corresponding degree.

The response of the pore pressure measuring system for transient external pressures was also tested. Here too, it was found that the external pressure change had to exceed the threshold value before any change in pressure was registered and that the subsequent measurements during the pressure rise were correspondingly too low. The external increase in pressure was somewhat irregular but this was not registered and an evened out time-pressure curve was obtained. When the external pressure decreased, the measured values in the cone changed from being the threshold value too low to being correspondingly too high and this was again reversed at the next increase in external pressure, Fig. 2. The observed behaviour is thus that when using a grease filled slot an evened out response is obtained, in which minor variations are not registered and the measured amplitude is ± the threshold value too low.
3.2 The function of grease filled slot filters according to comparative field tests

Comparative tests in soft clay were performed in three test fields in Sweden. These fields have also been used in a larger investigation concerning piezocone tests in clay (Larsson and Mulabdic 1991) and a comprehensive data bank for comparisons is available. In each field four parallel tests were made with slot filters, two starting directly from the ground surface and two with pre-drilling through the dry crust.

In the first test series in Norrköping, the same universal grease as in the laboratory tests was used. The first two metres of the profile consists of dry crust where negative pore pressures are created in the test. No problems occurred and all four tests showed very similar results. A prerequisite of this agreement is the use of zero values read off before the cone is assembled and after the cone tip has been unscrewed after the tests. Otherwise, large and generally unacceptable differences could be found between zero readings taken before and after the tests and the results became strongly dependent on what zero value was used. A comparison with pore pressures measured with conventional pore pressure measuring systems showed that the results were similar.

However, a closer study reveals that the pore pressures measured by use of the grease filled slot in general are about 25 kPa lower and that the recorded variations when passing different soil layers are subdued, Fig. 3. The threshold value of 25 kPa in the field obtained at about 8°C can be compared to the corresponding laboratory value of 20 kPa at 20 °C.

In the following test series in Skå-Edeby and Lilla Mellösa, a softer grease was used to study the possibility of obtaining better agreement between the pressures measured with the two types of system. As a probable result of this, saturation was partly lost when the cone penetrated the dry crust in the tests which started from the ground surface although the crusts were only about a metre thick. The tests with pre-drilling yielded measured pore pressures that were very similar to those obtained by ordinary filters and fluids.

In the comparative tests, no influence from the type of filter used could be observed in the values of tip resistance and sleeve friction after correction of the measured values for pore pressure effects according to standard procedure.

From the investigations, the following conclusions can be drawn:

- When a grease filled slot is used, the pore pressure is in principle measured with a measuring error of ± the threshold value for the particular grease.
- This measuring error becomes larger if the zero readings are not taken in unstressed conditions with the tip unscrewed.
Variations in the generated pore pressure become partly evened out because of the firmness of the grease and details cannot be observed when a firm grease is used.

- When a firm grease is used, zones and layers of unsaturated soil or where negative pore pressures are generated can often be passed without loss of saturation.
- When a softer grease is used, the aforementioned advantage is lost and the same problems with saturation as with normal fluid saturated filter will occur.
- The corrected tip resistance and sleeve friction are essentially unaffected by the types of filter and fluid used.

### 3.3 Comparative tests in soil profiles where problems have occurred when using conventional filters

Problems with loss of saturation are often encountered in zones and layers of unsaturated soil or where negative pore pressures are generated when conventional filters and fluids are used. This can be avoided by pre-drilling through these zones and, at larger depths, more or less successfully by more elaborate preparation techniques.

An alternative to this is to use slot filters filled with firm grease and to accept the associated measuring errors. Numerous examples exist of profiles where the conventional technique has failed and where the new technique has made it possible to register relevant profiles of generated pore pressures. Such an example from SGI files is shown in Fig. 4. In the conventional tests, a vacuum was created in the pore pressure measuring system at 4.5 m depth, whereupon it ceased to function. The negative pore pressures registered with the slot filter where lower and the system continued to function. A certain offset from "true" values can be observed in highly permeable layers but the trend is representative for the main layering of the soil.
4. DEVELOPMENT TENDENCY FOR THE USE OF SLOT FILTERS
The current trend in the use of slot filters is that specifications are being given for the type of grease that may be used. Furthermore, tests are being made with replacement of the commonly used water in the innermost part of the system by other substances such as glycerine.

A further area, where slot filters have been reported to be advantageous is in very stiff boulder clays where abrasion tends to completely clog the filters. Good results have here been obtained here by using slot filters and also by using them in combination with glycerine in all parts of the system.

5. CONCLUSIONS AND RECOMMENDATIONS
In many cases, a slot is often a practical and economical alternative to conventional filters. The hitherto most common technique with grease in the slot and water in the innermost part of the cavities in the cone causes certain measuring errors, which have to be taken into account in choice of technique, handling of the equipment and evaluation of the results.

For tests aimed at determination of stratigraphy and at evaluation of properties based on tip resistance and sleeve friction only, the errors are normally not large enough to significantly affect the evaluation. However, the type of grease used and its properties should be specified to enable an estimation and a limitation of the errors.

In tests where the measured pore pressures are to be used as direct parameters for evaluation of properties, studies of pore pressure dissipation or establishment of in situ pore pressures in various layers in the soil profile, significant measuring errors created by use of a semi-solid gel in the measuring system should be avoided.

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Cone Penetrometer Development and Testing for Environmental Applications

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SYNOPSIS: This paper summarizes the U.S. Department of Energy (DOE) research and development activities involving the cone penetrometer and the use of various sensors and sampling tools for environmental applications. These activities include enhancement of thrust capacity necessary for penetrating through gravelly soils, development of tools to determine local soil properties (moisture, hydraulic conductivity, pore pressure), to sample soil gases and groundwater for analysis, to determine sampling position, and to characterize subsurface geology and hydrology; and development of sensors and/or instrumentation to locate, identify, and quantify contaminants in soils and groundwater. The major focus is on developing these sensors and tools to fit inside a push rod to obtain real-time, depth profiles of subsurface characteristics during a cone penetrometer push or to be emplaced at a discrete depth for long-term monitoring. Many of these cone penetrometer technologies have been demonstrated at DOE and Department of Defense sites as part of site characterization activities.

1. INTRODUCTION

In the United States, nuclear weapons production resulted in many sites with contaminated soils and groundwater. Of the known 10,500 hazardous substance release sites, only one-fourth have been fully characterized (Report No. DOE/EM-0232, 1995). The hazardous substances at these sites include organics, heavy metals, radionuclides, and anions. The subsurface stratification consists of layers ranging from hard-to-penetrate cobbles and gravels to fine-textured sand, or a combination of them. The existence of confined aquifers and the extent of natural infiltration from rainfall or snowmelt also vary significantly from site to site. It is of great importance to fully characterize these sites, better assess the contaminant flow and transport, and provide a remedial strategy based on the imminent nature of contaminant migration, which affects human health and safety.

The DOE Environmental Management (EM) program is responsible for the cleanup of these contaminated sites as well as treatment of wastes for safe disposal and decontamination/decommissioning of surplus facilities. These waste management activities will cost an estimated $200 to $350 billion in constant 1995 dollars, from 1995 to approximately 2070. Among the reasons cited for the uncertainty of this cost estimate is the lack of characterization of waste/site problems. Innovative technologies are needed for effective and efficient waste characterization and cleanup operations to achieve significant savings in costs and time.

Within EM, the Office of Technology Development (OTD) is responsible for developing innovative technologies for environmental restoration and waste management activities. The “expedited site characterization” (ESC) methodology, supported for implementation by OTD, promotes the use of
innovative technologies for conducting site investigation studies. The ESC approach focuses on use of field analysis instrument/methodology to analyze the majority (80-85%) of samples on-site, and minimally-intrusive tools, such as a cone penetrometer, to deploy sensors and instrumentation for real-time determination of subsurface characteristics, i.e., chemical, physical, and geophysical parameters. The use of a cone penetrometer to obtain samples (soil gases, soils, and groundwater) for analyses and to cost-effectively “probe” the subsurface for depth-profile information at various locations, supplemented by results obtained from field analytical techniques, allows timely, informed decisions to be made on-site regarding where to take samples and when to stop sampling.

This dynamic sampling strategy is in contrast to the traditional approach to sample and install monitoring wells in a predetermined grid matrix, and to await laboratory results before proceeding to the next step. In a recent case study, the ESC methodology was demonstrated to have realized a total cost savings of $4 M at a DOE site, including the reduction of needed monitoring wells from 16 to four (Purdy, et al., 1995).

The use of a cone penetrometer has become an integral part of DOE site characterization operations. Earlier development activities and commercial offerings have been reviewed and summarized in a report, “Review of Instrumentation and Sensory: Cone Penetrometer Applications” (Report No. DOE/HWP-149, 1994). Additional information regarding 1994 and 1995 development activities can be found in the DOE “Rainbow-series” booklets that summarize OTD’s technological activities. These can be found at the URL: http://cmst.ameslab.gov/cmst/homepage.html.

2. CHEMICAL SENSORS/INSTRUMENTATION
2.1 Determination of Organics
The organic contaminants at DOE sites consist primarily of chlorinated volatile organic compounds (VOCs), i.e., trichloroethylene (TCE), perchloroethylene (PCE), carbon tetrachloride (CCL4), and 1,1,1-trichloroethane (TCA), as well as some spills of petroleum, oil, and lubricants. Because of the prevalent nature of VOC contamination, development of technologies for their determination has been a major focus within the OTD. Several VOC analyzers were demonstrated in April 1993 to analyze soil gases, in real time, during a cone push at the DOE Hanford Site in Washington. These analyzers included direct sampling ion trap mass spectrometer (DSITMS) instrumentation, an infrared (IR) spectrometric
technique, a portable detector incorporating an acoustic wave sensor, and a Halon wet total organic chloride sensor. A Teflon® transfer line was used to transport the soil gases to the surface for analysis. One end of the transfer line was attached to the cone penetrometer probe and the other to a gas-sampling manifold that splits gas flow to each analyzer. All analyzers tested at the site were shown to be capable of measuring low, single-digit parts-per-million (ppm) levels of CCl₄. Fig. 1a provides a real-time, depth profile of the CCl₄ concentration obtained from the DSTM during cone penetrometer pushes; Fig. 1b provides results from the Briel & Kjerd Model 1301 portable IR spectrometer, indicating that the CCl₄ concentration varied with the change of pressure and temperature that occurred during the measurement period.

Other notable techniques that are viable tools for cone penetrometer applications include the RCL Monitor from Transducer Research, Inc., (Buttner, et al., 1994), and a high-speed gas chromatograph (GC) system equipped with a surface acoustic wave (SAW) detector from Amerasia (Staples, et al., 1994). Both demonstrated parts-per-billion (ppb) range sensitivity for VOCs in well gases.

Much ongoing developmental work emphasizes the "in situ" aspect of sample preparation or sample analysis. Milanovich, et al., 1994, developed a fiber-optical sensor system with an optimized reagent chemistry based on the colored formation of the Fujisawa reaction product of pyridine with organochlorides. A flow-cell design with reagent reservoirs coupled with fiber optics allowed the probe to be deployed by a penetrometer cone (32 mm i.d.) and operated remotely for an extended period of time. This colorimetric detection system has demonstrated a sensitivity of 5 ppb, by weight, for TCE and chloroform. Fig. 2a gives the schematic design of a cone "probe," and Fig. 2b depicts a

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**Fig. 2a** The high-aspect ratio reagent delivery system provides fresh reagent to the sensor for repeat measurements without increasing pressure within the sensor body.

**Fig. 2b** Calibration or working curve for TCE in water at 0-100 ppb.
determination of chlorinated VOCs (TCE, TCA, and chloroform) and copper ion has been developed in a collaborative effort between Sandia National Laboratory and the Center for Process Analytical Chemistry at the University of Washington. This probe features a small diameter (13 mm) and a very small reagent reservoir (10 milliliter) allowing as many as 500 analyses before the need to replenish the reagent reservoir.

Microsensor Systems, Bowling Green, Kentucky, is under contract to develop a miniature GC, equipped with a solid-state microsensor as the detector, that will fit inside a cone penetrometer rod (38 mm i.d.) for in situ characterization of VOCs in soil gases or a headspace of groundwater. Argonne National Laboratory and Tufts University are working together to develop sample preparation devices to fit in a cone rod, allowing remote in situ purging/trapping of VOCs from groundwater and in situ thermal desorption of VOCs and semi-VOCs from soils, followed by their quantitative transfer to the surface for on-line GC/MS analysis. Field prototypes of both the purge device (Fig. 3a) and the thermal desorber (Fig. 3b) have been designed and are being tested.

A Raman-cone penetrometer system (Carrabba, 1995) was built to test the feasibility of locating and identifying nonaqueous phase liquids (NAPLs). This system includes a truck-mounted, compact, Raman spectograph (approximately 60 lbs.) and an air-cooled argon ion laser (514 nm excitation, 50-75 mW) connected through a 100 m fiber-optic cable to a sapphire window mounted on the cone or a liquid sampling cone. The ruggedness of this system was shown during a push to a depth of 20 feet. However, strong fluorescence background from soils with 514 nm excitation (Fig. 4a) was encountered, which, in turn, masked the presence of TCE and PCE. The optimum Raman wavelength was found to be dependent on soil type. As evidenced in Fig. 4b, the presence of a PCE band at 1570 cm⁻¹ can be easily detected with 364 nm excitation.
Many cone penetrometer technologies developed through non-DOE sponsorship have been adapted to meet DOE characterization needs. A notable example is the cone penetrometer-mounted, laser-induced fluorometry (LIF) sensor for in situ, real-time, subsurface determination of petroleum hydrocarbons. The LIF sensor coupled through a fiber optic to a sapphire window was originally developed by the U.S. Army Corp. of Engineers Waterways Experiment Station, Vicksburg, Mississippi. Two advanced systems are undergoing regulatory verification/validation/certification by the U.S. Environmental Protection Agency (EPA). The first of these, developed by the U.S. Navy NRaD Division, employs a pulsed nitrogen laser with 337 nm excitation. The second, Loral’s ROST™ system, employs a neodymium-doped yttrium aluminum garnet pump laser tunable in the range from 280 to 300 nm (Report Title: Laser Induced Fluorometry/Cone Penetrometer Technology Demonstration Plan, 1995).

2.2 Determination of Inorganics

The inorganic contaminants at DOE sites consist mainly of heavy metals (e.g., mercury, lead, and chromium⁵) and radioactive substances (e.g., plutonium, uranium, technetium-99, strontium-90, and tritium). For heavy metals, an x-ray fluorescence technique recently measured chromium, copper, and bromine contaminants in a borehole to about 50 ppm level (Reeves, et al., 1994). An improved probe (32 mm o.d. by 1 m long) incorporating an x-ray tube and a high resolution liquid nitrogen cooled silicon detector, is being built with onboard pulse processing electronics. The projected performance of this probe is 10 ppm level of detection for chromium in less than two minutes.

Applied Research Associates (ARA) has developed and is marketing a Gamma Radiation-Cone Penetrometer Technique (Shinn, et al., 1994) for locating radioactive contamination. This technique uses a NaI(Tl) detector (25 mm diameter by 152 mm long) that can be deployed in two ways: (1) lowered through a rod string, and (2) lowered down a 32 mm PVC well after its installation by use of a cone penetrometer.

An integrated gamma probe incorporating a 25 mm diameter by 76 mm long NaI(Tl) crystal detector in a cone penetrometer push rod was developed and field tested for determining uranium in soils by detecting the presence of gamma-ray peaks from the 234mPa daughter of 238U at 766 and 1001 keV. Test results

![Graph](image)

*Fig. 4a Soil fluorescence masks TCE and PCE with 514 nm excitation (11 mW, 10 sec. int.).*

![Graph](image)

*Fig. 4b Spectra for PCE saturated Savannah River Site soils collected with 364 nm excitation - high wavenumber region.*
indicated an unsatisfactory lower level of detection for $^{239}$U on the order of 60-80 pCi/g (Brodzinski, 1995). This higher than expected detection limit is largely caused by a significant zero and/or gain shift resulting primarily from failure to reach temperature equilibrium during cone pushes, and to some extent from interfering radionuclides such as $^{210}$Pb. Future activities being planned include better quantifying the effect of temperature on zero and gain shifts, and the development of a cryogenically-cooled germanium detector for cone penetrometer applications.

3. PHYSICAL SENSORS/ INSTRUMENTATION

Soil properties (moisture content, pore pressure, and hydraulic conductivity) are needed for flow and transport modeling leading to risk assessment, and for better remedial strategies. Sandia National Laboratory developed a time domain reflectometry (TDR)-cone penetrometer probe that achieved a sensitivity of ±1% moisture by volume. The TDR measures signal response from a pulsed electromagnetic wave to determine the dielectric constant of the soil, which can be correlated to moisture content. This prototype probe design is undergoing acceptance testing at a user site. The same Sandia research team in collaboration with GeoCenters is developing a porous, polymer-based optical sensor to measure relative humidity (RH) in unsaturated zone soils. The pore pressure can then be calculated from the RH values. A long equilibration time (~1 hour) was required to get a stable RH reading. The authors are unaware of any ongoing study to measure hydraulic conductivity using a cone penetrometer technique.

Cone probes that measure pH, temperature, soil resistivity, and seismic P and S waves are commercially available. A new collaborative activity between Lawrence Livermore National Laboratory and ARA will use cone penetrometer equipment to deploy electrodes for subsurface mapping by the Electrical Resistance Tomography (ERT) technique.

4. CONE PENETROMETER ENHANCEMENT

Many DOE sites contain gravelly soils or cobbles that "standard" cone penetration equipment with a hydraulic force of 20 tons is unable to push through. A notable example is at DOE Hanford Site, Washington; the 20-ton equipment typically met refusal there at 10 to 20 feet. In a similar gravelly formation, however, a heavy-duty, 40-ton system reached an average of 150 feet (Timian, et al., 1992). The heavier systems also use larger push rods that provide much needed space for sensor integration. The sizes of push rods are typically 36, 44, and 57 mm o.d. for 20-, 30-, and 40-ton capacity trucks, respectively.

A sonic cone penetrometer is another means being developed by ARA to provide the necessary penetration force through hard layers. The prototype design incorporates a small sonic head that attaches to the push rod above the head clamp. During the sonic-cone penetrometer operation, the push rods will be vibrated using a counter-rotating weight mechanism while the push cylinders maintain a static load and monitor penetration depth. The sonic head is expected to operate only during periods of pushing through hard materials. The useful operating lifetime of the bearings in the sonic head is projected to be about 200 hours.

UTD has fabricated and field-tested a strain gage-equipped probe, the POLO$^{TM}$ system, that determines the location of sampling devices or integrated sensing probes. The POLO$^{TM}$ system defines the trajectory of the hole in the ground by navigating from point to point along the penetration path. A mapping algorithm describes the borehole from strain-gage measurements. This system was demonstrated to have a total error, in position location, of less than 0.5% of the distance traveled (the longest distance tested to date is 70 meters). Continuing development involves the integration of the POLO$^{TM}$ system with a vibratory drive and a specially designed probe wedge to provide a steerable, enhanced delivery system.
As to enhanced sampling devices, various transfer line materials (stainless steel, nickel, aluminum, Teflon®) have been studied by Argonne National Laboratory to quantify the outgassing properties and the transfer loss of VOCs under varying temperature, flow rate, and latent moisture content. The stainless steel materials, type 304 and 316, were found to be superior to other materials studied. The Westinghouse Savannah River Company (McCarty, et al., 1995) in Aiken, South Carolina, is developing means for rapid transport of groundwater to surface for analysis (as compared with baseline samplers: bailer, BAT™, and Hydropunch® systems). Two devices have been fabricated. The Cone Sipper™, a gas lift pump with its schematic shown in Fig. 5a, uses check valves and alternate vacuum and pressure to draw and transport water samples to the surface. The double-piston displacement pump, Fig. 5b, uses alternate hydraulic pressures for up and down strokes. The sample recovery studies of both devices are being extensively evaluated. The Cone Sipper™ is also being evaluated by a commercial partner for integration with its own probe.

5. CONCLUSIONS
Although cone penetrometer techniques have existed for more than 50 years, most earlier efforts focused on geotechnical applications, e.g., oil exploration, construction engineering, etc. Only recently has the emphasis on environmental characterization, monitoring, and sensing stimulated a new wave of development of devices, gadgets, sensors, and sensing equipment for integration into the cone penetrometer system. Many areas still need to be explored or further developed. For example, the use of fiber optics with a cone penetrometer offers many opportunities for remotely sensing metals, radionuclides, explosives, and organics (NAPLs). Use of the cone penetrometer as a tool for installing monitoring wells and for emplacing sensors and sensing equipment is another area likely to increase in applications.
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The application of acoustic emission testing with penetration testing.

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SYNOPSIS: When penetrating a soil with a CPT cone, soil particles around the cone are displaced and crushed. These mechanical actions generate acoustic emissions (AE), both in low and ultrasonic frequency range. Because of its origin, this AE contains information concerning the soil characteristics such as soil type, shear resistance, compressibility, soil fabric and ageing.

This paper describes the results of a research program aimed to examine the possibilities of using the AE signals which can be recorded during penetration of the soil with a CPT cone. Two test setups and their results are described: a needle-sounding device, developed to study the parameters influencing the AE signal in a laboratory setup, and an acoustic CPT cone which was also used in laboratory conditions as prototype AE CPT cone in order to prepare a field test campaign.

1. INTRODUCTION

The use of sound with CPT testing is already known for a long time. CPT operators can hear from the sound generated via the CPT rods when course materials are met during the test. Feeling the vibrations on the rods by hand even gives a wider range of soil types to be distinguished roughly. In the past, researchers used this phenomenon by introducing a microphone in the CPT cone (Muromachi et al, 1974; Villet et al, 1981; Tringale and Mitchell, 1982).

The term AE stands for elastic waves generated internally in the material from very low frequencies to ultrasonic level. Depending on the frequency one is interested in, measurement equipment is chosen. Massarsch (1986) was the first to use AE in the ultrasonic frequency domain with CPT testing. He introduced a resonance piezoelectric AE transducer (resonance frequency 210 kHz) with preamplifier in the tip of a CPT cone. This high frequency level was chosen for two reasons:

- High frequency vibrations are easily damped in soil material, so one can be sure the measured signal is generated close to the sounding device.
- The use of the ultrasonic frequency range prevents noise and other signals generated at ground level (normally of low frequency) to interfere with the measurements.

Massarsch reported that with this technique seams of sand and silt material with a thickness of a few mm could be detected within a clay deposit of a few m thickness while this could not be seen with a piezocone.

2. RESEARCH PROGRAM

Two AE test setups were built: a small test cylinder in which an AE needle sounding device was used, and a prototype AE CPT cone which was used in a laboratory test container. Both test setups were aimed to get used with the equipment, the AE signals generated in the ground, their source mechanisms and the way the signals should be analysed.
Parameters affecting the AE signals such as soil type, penetration rate, relative density, stress level, sample preparation technique and ageing were investigated.

All tests are performed on dry sand samples for the preparation of which 3 different types of sand were used: Molisand, Calibrated Sand and Rhine Sand. The sieving curves are given in figure 1. Some general characteristics of these sand types are given in table 1.

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Molisand</th>
<th>Rhine Sand</th>
<th>Calibrated Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_{50}$ (mm)</td>
<td>0.168</td>
<td>0.387</td>
<td>0.945</td>
</tr>
<tr>
<td>$d_{10}$</td>
<td>1.47</td>
<td>2.44</td>
<td>1.49</td>
</tr>
<tr>
<td>$d_{10}$ (mm)</td>
<td>0.128</td>
<td>0.187</td>
<td>0.680</td>
</tr>
<tr>
<td>Roundness $c$</td>
<td>0.36</td>
<td>0.43</td>
<td>0.49</td>
</tr>
<tr>
<td>Sphericity</td>
<td>0.75</td>
<td>0.80</td>
<td>0.75</td>
</tr>
<tr>
<td>$\rho_{s}$ (g/cm$^3$)</td>
<td>2.650</td>
<td>2.651</td>
<td>2.652</td>
</tr>
<tr>
<td>$\gamma_{sua}$ (kN/m$^3$)</td>
<td>16.387</td>
<td>17.834</td>
<td>16.685</td>
</tr>
<tr>
<td>$\sigma_{c}$ (kN/m$^2$)</td>
<td>13.553</td>
<td>14.898</td>
<td>14.334</td>
</tr>
<tr>
<td>$\sigma_{sma}$</td>
<td>0.585</td>
<td>0.458</td>
<td>0.559</td>
</tr>
<tr>
<td>$\psi_{c}$ (°)</td>
<td>31.5</td>
<td>33.15</td>
<td>30.92</td>
</tr>
</tbody>
</table>

![Sieve curves of the three sand types](image)

Fig. 1. Sieve curves of the three sand types

More details of the full test program are described by Mengê (1994).

3. AE MEASUREMENT EQUIPMENT AND SIGNAL ANALYSIS

The AE measurement equipment consists of a piezoelectric AE sensor, a preamplifier and a high speed digitizing board plugged in in a portable personal computer. The sensor has a flat response in the frequency range between 100 kHz and 1 MHz. This allowed us to perform frequency analysis on the measured signals. A bandpass filter with the same frequency characteristics is included in the preamplifier. The amplification factor is 60 dB.

The amplified signal is digitized and stored on harddisk for analysis later on. The signal is measured as it is generated at the output of the preamplifier. During the penetration, a continuous signal is generated. This could not be measured completely; only small ‘shots’ of the signal are measured and stored for further analysis (see figure 2).

![Waveforms](image)

Fig. 2. Measuring of the waveforms

The signal analysis is based on these shots or waveforms. Each waveform can be analysed both in time and frequency domain. In order to have a clear presentation of the results, individual waveforms are not studied but following characteristics are defined:
- RMS value of the waveforms, which is a value for the energy in the waveform;
- amplitude distribution: the number of pulse-maxima in different amplitude intervals. The result of an amplitude distribution also allows counting. With ‘countings’ one means the number of threshold crossings when a certain threshold (here fixed by the amplitude intervals) is set.
- frequency distribution: the energy available in frequency intervals (for example intervals of 100 kHz can be chosen).

4. TESTS WITH THE NEEDLE SOUNDING DEVICE

Samples prepared by means of dynamic compaction (tamping), sand raining and vibrating the sand were made at different relative densities. They were sounded with different penetration rates and after different time intervals. These tests and their results are extensively discussed by Mengé and Van Impe (1995). Only the most important conclusions will be repeated here.

The needle sounding device, developed to examine the parameters influencing the AE signals when penetrating the soil, seems to be a highly sensitive laboratory measuring equipment. It was concluded that this equipment measures phenomena influencing the AE at grain-size level. Figure 3 shows part of a typical waveform measured with the AE needle sounding device (penetration rate 0.5 mm/s). It is clear from this graph that the signal mainly consists of two parts: a general noise type pattern with bursts or ‘events’ exceeding the noise level. From this study, it was concluded that these individual bursts are generated in the contact points between the grains through friction or crushing, the main source mechanisms of AE in granular media.

penetration depth seems to be rather unaffected by penetration rate;
- relative density: AE intensity increases with higher $R_a$-values; this is obvious when one considers the source mechanisms of the AE;
- soil type: when examining rather similar soil types (various sand types) the frequency distribution (which seems to be influence solely by soil type) gives most information; this is also a powerful tool to get more information about the mineralogy of the sandtype; when crushable minerals are present, more energy in the higher frequency range is found;
- preparation method: AE generated in samples prepared with the pluviation method are clearly lower intensive than the AE signals generated in dynamically compacted samples; as explained for the influence of relative density, the number of contact points between the soil particles could be an explanation;
- ageing is clearly visible from samples with a relative density $R_a\approx75\%$; from these tests it could be concluded the AE parameters are more sensitive to ageing than the mechanical penetration resistance $q$ is;
- as with $R_a$-values, there seems to be a ‘critical depth’ from which the depth or stress condition does not influence the intensity of the AE signal any longer.

5. TESTS WITH THE AE CPT

The large test container consisted of a concrete cylinder 1.00 m in diameter and 1.00 m in height. The container has rigid boundaries ($u=0$) and the vertical stress is applied through a top plate with membrane and air pressure. Side friction could be neglected. The overburden pressure to be applied was limited to 50 kPa and no vertical deformations were measured during pressurizing. The sample preparation was done using a raining device with constant falling height for each layer ($\leq100$ mm thickness) or by means of tamping. A general overview of the container and test setup is given in figure 4.

Detail of the AE CPT cone is given in figure 5. As suggested by Massarsch (1986), a needle

![Fig. 3. Typical AE waveform (RM-55-30/21)](image)

Following parameters are discussed:
- penetration rate: within the tested range of 10 to 50 mm/min, the RMS value per unit

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tip was used. This design is more sensitive and better suited to detect thin seams.

Most tests with the AE CPT cone were performed in homogeneous samples of Molsand. These introductory tests served to examine the AE response with a larger sounding device (compared to the needle sounding device) and to test methods of analysis.

Figure 6 shows a typical waveform obtained with the AE CPT device (penetration rate 20 mm/s). It is clear that a strongly different signal is measured compared to figure 3. This is mainly due to the penetration speed. Individual events can no longer be distinguished. The scatter in RMS values and other characteristics from waveform to waveform is less significant than with the needle sounding device. Figures 7 and 8 give the RMS value in function of $q_t$ and in function of $I_d$ respectively. From both graphs a clear influence from relative density and soil strength on the AE RMS value can be seen. It is also clear that samples prepared in different ways (rained and tamped) show a clearly different behaviour.

Most information can be found from the amplitude distribution. Figure 9 shows the frequency distributions (in 0.1 V intervals) for samples with $I_d=65\%$, one prepared by means of raining, the other prepared by means of
In the case of the tamped samples, the amplitude distribution seems to be evenly distributed over the intervals while this is not the case with the reamed samples. By means of this method, sample preparation can clearly be distinguished.

The power spectrum of the waveforms shows clearly different peaks at various frequency levels. This is shown in figure 10 where the mean power spectrum for a sample prepared by tamping with I_d=65% is given. Analysis of all the tests performed with Molsand) however shows a clear repetition of the pattern given in figure 10. From this it is concluded that also with the AE CPT cone the power spectrum is depending mainly on soil type and mineralogy.

In order to examine the possibility of identifying soil layers, some tests were performed with layered samples. Figure 11 shows the RMS values and q_s values in function of depth for a test performed in a layered sample (top to bottom: Molsand, silt, Molsand, gravel, Molsand). Apart from some boundary influence at the top of the sample, the different layers can clearly be identified from the RMS values, while q_s values almost continuously increase with penetration.

CONCLUSIONS

The application of the AE technique with penetration testing was examined using two laboratory test setups. The laboratory needle sounding apparatus, used with low penetration rates (0.1 to 0.5 mm/s), proved to be a very sensitive apparatus. Various influencing parameters were examined. Apart from the material itself, it is mainly the relative density which influences the AE behaviour.
Fig. 11.a. RMS-measurements from a test in a layered sample

Fig. 11.b. q, measurements from a test in a layered sample

The influence of sample preparation technique and ageing was tested and could be recognized clearly. This illustrates that the soil skeleton structure, the orientation and the number of contact points between the grains are parameters influencing the source mechanisms of AE.

However single AE events can not be recognized anymore the tests with the prototype AE cone show that, at the higher penetration rates used for CPT testing, the AE is still an important source of information. The sand type clearly influences the pattern of the power spectrum and sample preparation can be recognized by means of the amplitude distribution. The density of the sample on the other hand influences the AE level (RMS value).

Finally, it was shown that the AE RMS value offers a more sensitive way to detect and identify soil layering.

ACKNOWLEDGEMENTS
The author wants to thank the Belgian National Fund for Scientific Research for the financial support given to this research program.

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SYNOPSIS: Radio-isotope cone penetrometers (RI Cones) have been developed at the Kyoto University. These cone penetrometers make use of neutrons and γ-photons to measure the water content and the density of soil respectively. Application of RI-Cones in the sandy deposits were strongly desired, especially in Japan, to know the in-situ conditions for the assessment of the liquefaction during earthquake loading. Frozen sampling along with several in-situ testings were carried out. RI-Cone measured data were compared with the laboratory test results on the frozen sand samples. RI-Cone measured water content and wet density were found to be in good agreement with those obtained on the frozen samples. Finally, the CPT based conventional procedures to evaluate geotechnical parameters using empirical relations are also discussed based on the RI-Cone performance. From these results it is concluded that the RI-Cone penetrometers are versatile tool in sand deposits.

1. INTRODUCTION
Cone penetration testing has been increasingly gaining popularity owing to the simplicity, repeatability, economy as well as the least interaction with the human. The variability of tests and the interpretation of the data and the versatility of CPTU tests have been demonstrated by many researchers over a period of time. CPTU has now been accepted in many countries as a part of overall soil investigation.

Various modifications and/or adaptations have been done in cone penetrometer to suit the specific purpose at hand, e.g., grouting cone, seismic cone, cone pressuremeter etc.

With the ever graying population in Japan, which leads to the ever decreasing number of qualified personnel to do the sampling, and ever increasing environmental concerns has led to the development of the RI-Cones. The versatility of RI-Cones in clay foundations has now been well established (Shibata, et. al., 1992, 1994).

The desire to evaluate the performance of RI-Cones in the sandy foundations has long been felt. Recently, we have been able to carry out penetration tests in sandy deposits. The site (Higashi Ogishima) where the tests were carried out has been described in details in following paragraphs. Frozen sampling was also done at the Higashi Ogishima site, as it is believed that the freezing of soil preserves the soil structures, to compare the results.

2. DESCRIPTION OF RI-CONE PENETROMETERS
Fig. 1 and Fig. 2 show the schematic diagramme of the NM- and ND-Cones, for detailed description see Shrivastava (1993). Basic features of these two cone penetrometers can be summarized as follows:

2.1 NM-Cone Penetrometer: It makes use of the fact that the neutrons are slowed down in the presence of the hydrogen and this very fact can be used to measure the water content of the
soil if all the hydrogen present in the soil is in the form of water. NM-cone uses $^{252}$Cf as the fission source of the neutron having half life of 2.65 years. The detector used in the cone penetrometer is the $^4$He-filled proportional tube.

2.2 ND-Cone Penetrometer: Principle of using gamma-ray to measure the density of the soil is well established. Gamma-ray interacts with the soil predominantly in three ways. All the interaction is dependent on the energy level of the gamma-ray. They are at low energy (< 600keV) photoelectric absorption, between 600keV to 1.2MeV it is Compton scattering and above 1.2MeV it is electron-positron pair production. If the detector is so designed that it measures only the gamma-photons between 600keV and 1.2MeV then the incoming gamma-photons are only function of the density of the material and it can be given by the following equation:

$$I = I_0 \exp (-\mu_{t} \rho dx)$$

(1)

where; $I_0$: incident radiation intensity; $I$: transmitted radiation intensity; $\mu_t$: total mass absorption coefficient; $\rho$: density of the absorbing material; and $x$: thickness of the absorber.

The gamma source used for the ND-Cone is $^{137}$Cs and detection is done by means of the sodium iodide activated with thallium (NaI(Tl)) scintillator mounted on the photomultiplier tube (PM-tube).

3. FIELD ASSESSMENT OF RI-CONES

The results of the RI-Cone penetration tests carried out at the Higashi Ogishima site is presented. Frozen sampling was also carried out at this site. Various organizations participated in the testing program under the auspices of Port and Harbor Research Institute, Ministry of Transport. Various in-situ tests were carried out to study the site comprehensively. Some of the tests carried out, beside RI-Cones, are CPTU, seismic cone, dilatometer, cross hole etc. Data for Higashi Ogishima, Kawasaki City has been presented.

The SPT profile of Higashi Ogishima is shown in Fig. 3. This is a reclaimed site. Reclamation was done by pouring sand obtained from neighboring region. From a depth of 5m to 11m the $N_{SP}$ varies from 3 to 5, a very loose sand deposit. Underlying this deposit is silty sand with a thickness of about 5m and beyond 16m we encounter clay deposit.

Frozen sampling ($r = 150mm$) was done for the sand layer upto a depth of 11m to evaluate the accurate values of the wet density as well as the natural water content profiles. Profile of the specific gravity, $G_s$ is shown in Fig. 4. In spite of a little scattering, specific gravity of Higashi Ogishima sand is found to be 2.70 irrespective of depth. Fig. 5 shows the distribution of $e_{min}$, $e_{max}$ and the current void ratio ($e$) with depth. $e_{min}$ and $e_{max}$ show two different pattern for the deposit above and below 6m. The $e_{min}$ and $e_{max}$ in the upper portion is 0.697 and 1.125 respectively and for the lower portion it is 0.661 and 1.058. The current void ratio is determined to be 0.9. Fig. 6 shows $q_c$ and $u$ profile obtained through NM-Cone. As the profile of pore water pressure in the reclaimed sand deposit is almost equivalent to the hydrostatic pressure, with a low $q_c$ profiles, this sand deposit is found to be clean and loose. Fig. 7 shows the distribution of $w_c(\%)$ and wet density profile as a function...
of depth. Also plotted are the data obtained from laboratory testing on frozen samples. In order to determine the wet density of the frozen sample, water content of the frozen sample was determined for the melted state by using a simple expression \( w_m = w_i / 0.917 \), in which \( w_m \) is the water content at the melted state and \( w_i \) is the water content in the frozen state. Thereafter, the wet density of the soil was determined using following equation:

\[
\rho_t = \frac{(1 + \frac{w_m}{100}) \rho_w}{\rho_w / G_s + w_m / Sr}
\]

where; \( \rho_t \) is the wet density, \( \rho_w \) is the density of the water, \( G_s \) is the specific gravity and \( Sr \) is the saturation in percent. In spite of some scattering, the data are well matched. The water content measurement done in the laboratory on the frozen sample in fact does not represent the actual in situ water content as the effect of freezing and thawing etc. are not well defined. However, in case of NM-Cone what we measure is the actual in-situ water content of the soil with the minimum of distortion.

\[\text{Fig. 3} \quad \text{Fig. 4} \quad \text{Fig. 5} \quad \text{Fig. 6}\]
4. LIQUEFACTION POTENTIAL OF SANDY DEPOSITS BY CPT

4.1. General framework
Liquefaction is a condition where the pore pressure in a cohesionless soil sand builds up to such a level that the effective stress becomes zero and the soil loses all its strength. This may be due to static or cyclic stress. Hence, the most appropriate method for analyzing soil liquefaction potential is to evaluate, using in-situ methods, the pore pressure generation by the earthquake. The in-situ method chosen should be such that the soil is not disturbed during the test and can only be achieved by non-destructive testing, such as CPTU.

To assess the liquefaction potential, the cone resistance \( q_u \) is modified to account for the overburden stress effect in a manner, similar to, as suggested by Seed and Idriss (1971) for SPT. The correction factor, \( C \), as a function of effective overburden stress, is obtained by the procedure outlined by Robertson and Campanella (1985) and later modified by Shibata and Terakusa (1988). The modified cone resistance is expressed as \( q_u' = C \cdot q_u \), where \( q_u \) is the measured cone resistance. Thereafter, the cyclic stress ratio, \( \tau_{cy}/\sigma'_{vo} \), is developed during the earthquake is estimated. Several correlations can be used, such as the one proposed by Tokimatsu and Yoshimi (1983):

\[
\tau_{cy}/\sigma'_{vo}=0.1(M-1)\sigma_{max}+0.015\sigma_{vo} \quad (3)
\]

where, \( \tau_{cy} \) is the amplitude of uniform shear stress cycles equivalent to actual seismic shear stress time history, \( \sigma_{vo} \) is the total vertical stress in situ, \( \sigma_{vo} \) is the in situ effective vertical stress, \( M \) is the earthquake magnitude, \( \sigma_{max} \) is the maximum horizontal acceleration of ground surface, \( z \) is the depth in meters (\( z < 25m \)), and \( g \) is the acceleration due to gravity. The chart presented by Shibata and Terakusa (1988), which also includes data from Robertson and Campanella (1985), requires the knowledge of mean grain size, which can be obtained by the chart suggested by Robertson and Campanella (1985).

4.2 RI-Cone Based Assessment of Liquefaction Potential
As is already shown, RI cones can quantitatively evaluate the in situ natural water content as well as wet density for the loose reclaimed sand deposit. The accurate measurement of those components can lead to know the in situ overburden pressure and void ratio, which are indispensable to the assessment of liquefaction potential. Therefore, in situ assessment of
liquefaction potential can be done by carrying out RI cone penetration tests.

Relative density profile for the sand deposit based on NM cone measurement is shown in Fig. 8, together with the plots determined from the laboratory measurement through frozen samples. The set of values for \( \epsilon_{\text{min}} - \epsilon_{\text{max}} \) (Fig. 5) are used to get Dr (%). Although the laboratory determined Dr values show some scattering, but the Dr (%) values determined by RI-cones data show almost no scattering, prompting us to conclude that the RI cones can evaluate the Dr profiles well. Having put the general framework, let us investigate the possibility of liquefaction at Higashi Ogishima reclaimed sand deposit. Shibata (1985) proposed the critical cone resistance for liquefaction, \( (q_c)_{\text{cr}} \), based on the normalized critical cone resistance, \( (q_c)_{\text{cr}} \), which is expressed as follows:

\[
(q_c)_{\text{cr}} = \left[ \frac{0.7 + \sigma_v}{1.7} \right] \cdot (q_c)_{\lambda_c}. \tag{4}
\]

Profile of direct measurement of \( q_c \) is compared with \( (q_c)_{\text{cr}} \) in Fig. 9. The acceleration intensity, \( \alpha_{\text{max}}/g \), is assumed to be 0.2. It is observed from Fig. 9 that measured \( q_c \) is much smaller than \( (q_c)_{\text{cr}} \) for the whole range of reclaimed sand deposit. It is concluded that the reclaimed sand deposit at Higashi Ogishima will liquefy if an earthquake with the acceleration intensity of \( \alpha_{\text{max}}/g \) equal or greater than 0.2 hits the area.

![Graph showing comparison of measured \( q_c \) and computed \( (q_c)_{\text{cr}} \) at Higashi Ogishima.]

5. CONCLUSION

RI cone penetration tests are carried out at Higashi Ogishima site characterized by loose reclaimed sand deposit. Natural water content \( (w_s) \) and wet density \( (\rho_w) \) profiles are measured together with the conventional CPTU triple components. The RI-cone measured values of \( w_s \) and \( \rho_w \) are validated by the laboratory test results on the frozen samples obtained from the nearby site. From the above going results it is concluded that RI cone penetrometers are versatile tool and well applicable to loose sand deposits.

Simplified assessment of liquefaction potential is also performed based on RI cone measurements. The overburden pressure, \( \sigma_{\text{ov}} \), which is one of the most influential factors on the liquefaction potential assessment, can be derived precisely from \( \rho_w \) profiles obtained through RI cone penetrometer. The measured cone resistance, \( q_c \), at Higashi Ogishima sand deposit is found to be much smaller than the critical cone resistance, \( (q_c)_{\text{cr}} \), which is defined as the boundary values for liquefaction, resulting in that the sand deposit at Higashi
Ogishima will liquefy if a strong earthquake occur at this site.

6. ACKNOWLEDGMENT
Authors would like to express their sincere gratitude to Dr. Hiroyuki Tanaka, Chief of Geotechnical Survey Laboratory, Port and Harbour Research Institute, Ministry of Transport, who conducted the frozen sampling of sand as well as series of in situ testing projects at Ogishima. Thanks are also extended to Mr. Masao Ohtake, President, SRE; Mr. M. Yoshimura and other members of R & D Div, SRE for their help and encouragement during the preparation of this manuscript.

7. REFERENCES


Comprehensive Presentation of Raw CPT data

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SYNOPSIS: A new format for plotting raw CPT data is presented. A single log scale is used for all CPT measurements (cone resistance, sleeve friction resistance and dynamic/static pore pressure). Also, the vertical axis is in terms of log vertical effective stress rather than linear depth below ground surface. An additional vertical axis is also included which shows the equivalent depth in metres. This new data plotting format allows for better evaluation of uniform strength soil layers than is currently possible using traditional linear scales.

1. INTRODUCTION
Raw CPT data (i.e., cone resistance, sleeve friction, and dynamic pore pressure) are presented using software programs developed by field CPT contractors, CPT equipment vendors, or CPT consultants. Various plotting formats are used worldwide. Some CPT data plots clearly represent the data, while others are poor in quality and inflexible in terms of scaling. This paper introduces a fresh new format for presentation of raw CPT data which solves several of the problems associated with current CPT data presentation methods. A discussion is included specifying the utility of using log scales and multiple stress units.

2. ISSUES CONCERNING PLOTTING OF RAW DATA
When presenting raw CPT data for a project, the CPT contractor must select plotting scales to show all the data for each CPT measurement of interest. For sites in which strong and weak soil layers exist this can result in unreadable plots for the low strength layers. For example, deep sand layers may have cone resistance of 300 atm (30 MPa) whereas near-surface normally consolidated clay layers may have cone resistances less than 3 atm (300 kPa). A linear scale forces the low strength soil traces too close to the zero to be easily discernable from the plot. There are two solutions: 1) use of log scales, or 2) use of low and high scales (dual scales). Log scales emphasize the low range and de-emphasize the high value, which is important because delineation of the low strength layers is or greater interest. In the past, geotechnical engineers have resisted the use of log scales. As a result, engineers have often resorted to use of dual scales (low scale = 0-10 atm and high scale = 0-500 atm for cone resistance).

The major problem with the use of a log scale measurement versus linear depth scale is that all relationships become curved. This difficulty can be resolved by replacing the depth scale with log(at) vertical effective stress scale. As a result, linear relationships such as cone resistance with vertical effective stress in clay will plot as a straight line on a log-log plot. Any uniform strength soil layer will have a constant stress exponential relationship and will plot as straight lines on this log-log plot (Olsen, 1994). A constant stress exponential
relationship can be defined (Olsen, 1994, Olsen & Mitchell, 1995); 

\[ q_{c,t} = \frac{q_{c} - q_{n,m}}{\sigma_{v}^{c}} \]  

(1)

where;

- \( q_{c} \) = vertical effective stress (atm units) 
- (i.e., 1 atm = 100 KPA, = 1 tsf)
- \( c \) = cone resistance stress exponent
- \( q_{n,m} \) = measured cone resistance (atm)
- \( q_{c,t} \) = normalized cone resistance
  (to an equivalent value at \( \sigma_{v} = 1 \) atm)
- \( \sigma_{v} \) = total vertical stress (atm)

Log-log cone resistance versus vertical effective stress plots can assist in the identification and evaluation of uniform soil layers. If a soil layer is thick (for example 2 metres) and has a uniform strength (constant friction angle or clay c/p ratio), then the \( q_{c,t} \) will be constant throughout the soil layer and the relationship between \( \log_{10} \) net cone resistance and \( \log_{10} \) vertical effective stress will be linear. This is shown conceptually in Figure 1 and by example in Figures 2 and 3. These examples graphically show the limits of the uniform soil layers, the equivalent \( q_{c,t} \) value \( q_{c} \) at \( \sigma_{v} = 1 \) atm, and the log-log slope (i.e., stress exponent) as \( c \). The stress exponent is equal to the linear measured slope over one log vertical effective stress cycle as shown in Figure 1. A stress exponent of 0 corresponds to a vertical line while an exponent of 1 (linear scale) represents a line with a slope of one horizontal log scale to one vertical log scale. Normally consolidated clay have stress exponents of 0.9 to 1.0 and decrease with over consolidation. Loose sands can have a stress exponent as high as 0.8, medium dense sand about 0.65 and very dense sand as low as 0.3 (Olsen, 1994).

3. SELECTION OF STRESS UNITS

Cone resistance is typically plotted using MPa units while sleeve friction values are plotted using kPa units. For very weak clayey soils kPa units are used for cone resistance. Use of different stress units for both measurements makes direct comparison of the measurements more difficult. If measurements must be converted to atmospheric pressure units for normalization calculations (i.e., Equation 1), then atmospheric stress units should at least be shown on the plots. Atmospheres as a unit of measurement are within 4% of the following stress units; tsf, bars, and kg/cm² force. Also, kPa units can be thought of as one percent of atmospheric pressure, 1 atmosphere (atm) is 100 kPa or 100 percent of one atm. A combination of metric and atmospheric units can be used, to plot all CPT measurements on a single log scale, starting at 0.01 atm (1 kPa) and ending at 1000 atm (100 MPa).

4. EVALUATION OF CPT MEASUREMENTS

Raw CPT data plots should provide a means for evaluation in addition to presentation of the data. Identification of uniform soil layers from \( \log_{10} \)-\( \log_{10} \) plots is one such example. Comparison of cone and sleeve friction resistance data at soil layer boundaries (and at rod breaks) can assist in the identification of soil mixtures. Using a single \( \log_{10} \) scale for all CPT measurements (i.e., from 0.01 to 1000 atm) allows the dynamic pore pressure level to be compared to cone and sleeve measurements. For example, dynamic pore pressures close to the cone resistance level indicates normal consolidated clay. A low dynamic pore pressure compared to the sleeve resistance can indicate a dilative behavior of clayey sand. Comparison of cone resistance to sleeve friction resistance is important in differentiating between overconsolidated and normally consolidated soil layers. Overconsolidated deposits have sleeve friction resistances which are higher than for those of normally consolidated deposits. However,
Figure 1. Determining the cone resistance normalization value ($q_{n,1}$) and corresponding stress exponent, c, for thick uniform soil layers (Olsen, 1994)
slightly higher cone and sleeve friction measurements can only be observed if both are presented on the same scale.

In addition, use of the same scale for all CPT measurements (cone, sleeve, and pore pressure) can be important for evaluation of non-classical soils. For example, at rod breaks, pore pressures partially dissipate in silty deposits. When probing is restarted, the reduced sleeve resistance will momentarily increase for 5 to 25 cm of probe travel. There may also be a minor cone resistance increase. This can best be observed using the same scale for all CPT data.

Figure 2. Uniform unstable silty clay layers for a Hong Kong Bay deposit (airport replacement at Chek Lap Kok)

5. THE NEW RAW CPT DATA PLOT
The final raw CPT plot shown in Figure 4 is designed to be self explanatory. A uniform clay layer can be observed from 6 to 11 metres based on the nearly constant log-log trace, high $B_v$ value, and SCN trace indicating clay. The stress exponent (e.g., log-log slope) of this layer is approximately 1 but the projected $q_{c,v}$ is higher than 10 atm (indicating over consolidation). This is a river based CPT sounding where recent dredging work has created an apparent over consolidation effect for the deposit.

Figure 3. Uniform sand layer at Holmen, Norway (NGI, 1985)

The only unusual part of Figure 4 is the calculated soil classification (Olsen, 1988, 1994, Olsen & Mitchell, 1995). Typically, an engineer viewing raw CPT data must estimate soil type based on friction ratio, an potentially problematic task without computer evaluation. Inclusion of a CPT calculated soil classification (i.e., using the Soil Classification Number (SCN)) goes along way toward isolating critical layers for better overall evaluation.

6. CONCLUSIONS
A new format for plotting of CPT data is presented. Using a single log$_{10}$ scale for all CPT measurements against log$_{10}$ vertical effective stress allows for better confirmation of uniform layers. When a uniform layer is
observed, the data trend log-log slope is the stress exponent required for cone resistance normalization. This in situ determined stress exponent can be used as confirmation of the chart based determined stress exponents (Olsen, 1994; Olsen & Mitchell, 1995). A single scale for plotting of all measurements also allows a better comparison of how each measurement effects the others.

8. REFERENCES
Figure 4: New format for plotting of raw CPT data (using a CPT sounding from a test investigation).
New Concepts for CPT Standardisation in The Netherlands

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Fugro Engineers B.V., Leidschendam, MOS Grondmecanica, Rhoon, The Netherlands
Rotterdam Public Works, Rotterdam, Delft Geotechnics, Delft, The Netherlands

SYNOPSIS: Recent developments affecting Cone Penetration Testing (CPT) in the Netherlands include: (1) standardisation of geotechnical practice, (2) ISO 9000 quality system requirements and (3) tight accuracy requirements related to input parameters for advanced numerical models. These developments lead or will lead to adjustments to the industry practice for cone penetration testing. Preferably, standardisation should anticipate and fit the practice adjustments.

The draft version of the Dutch Standard NEN 5140 introduces new concepts for standardisation of electric cone penetration testing. The most important feature is a classification system for the accuracy of the CPT results. The classification system allows the designer to select a CPT class that suits the project requirements. The accuracy classes include uncertainties of 50 kPa, 250 kPa and 500 kPa for the cone resistance $q_c$.

The new concepts encourage, but do not specify, the accreditation of the quality system of a CPT company.

1. INTRODUCTION

Cone Penetration Testing (CPT) is the most important ground investigation technique employed in The Netherlands. Recent developments affecting CPTs include: (1) standardisation of geotechnical practice, (2) ISO 9000 quality system requirements and (3) tight accuracy requirements related to input parameters for advanced numerical models. These developments lead or will lead to adjustments to the industry practice for cone penetration testing. Preferably, standardisation of CPTs should anticipate and accommodate the practice adjustments.

Development of a national building code took place in the early 1990s. It led to standardisation of geotechnical practice within a legal framework. The principal geotechnical standard is NEN 6740 (NNI, 1991a), which is similar to Eurocode 7 (CEN, 1994). NEN 6740 refers extensively to cone penetration tests for derivation of partial factors and design parameters for a range of geotechnical models (Peuchen et al., 1995). The current standard on cone penetration testing is NEN 3680 (NNI, 1982). It covers mechanical and electric cone penetration testing. The link between NEN 3680 and NEN 6740 is relatively poorly defined, in particular with respect to parameter interpretation for soft soil conditions. Another development is the application of ISO 9000 quality systems in geotechnical practice. In this area, The Netherlands lags behind countries such as the UK and Hong Kong (Kramer, 1995). Accreditation of ISO 9000 quality systems for companies operating...
CPT equipment started in 1994. Further developments are likely as a result of a general responsibility/risk shift from client to supplier. Also, the wide availability of fast computers leads to a rapid increase in the use of advanced geomechanical models. This development requires accurate parameter interpretation which in turn relies on accurate and reliable determination of soil parameters.

In view of these developments, the Subcommittee on Geotechnical Investigation Techniques of the Nederlandse Normalisatieinstituut (NNI) started work in 1994. A draft version for a new standard on electric friction cone testing, NEN 5140 (NNI, 1995), was ready in December 1994. The formal period for comments by industry started in April 1995.

### 2. NEW CONCEPTS

The draft version of the NEN 5140 standard introduces new concepts for standardisation of electric cone penetration testing. Table 1 presents a summary.

<table>
<thead>
<tr>
<th>Table 1. New concepts</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Description</strong></td>
</tr>
<tr>
<td>Equipment</td>
</tr>
<tr>
<td>Size of cone penetrometer</td>
</tr>
<tr>
<td>Cylindrical extension of cone</td>
</tr>
<tr>
<td>Net area ratio</td>
</tr>
<tr>
<td>Friction reducer</td>
</tr>
</tbody>
</table>
Table 1 Continued

<table>
<thead>
<tr>
<th>Description</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Electronic heave compensator</td>
<td>This is a common feature on CPT units in The Netherlands. It compensates for small movements of the thrust machine and elastic compression of the push rods. Electronic heave compensation is permissible as long as it is within the penetration accuracy requirements.</td>
</tr>
</tbody>
</table>

**Procedure**

Classification system  
Four classes consider accuracies of cone resistance, sleeve friction, cone penetrometer inclination and penetration depth.

Calibration  
There are no specific requirements other than reference to NEN 2649. This describes general requirements for measuring and calibration systems.

Location survey  
Determination of coordinates and ground surface elevation are requirements, but agreement of specific criteria is subject to contractual arrangements.

Data recording  
Digital recording is an implicit requirement for Class 1 and Class 2. The records are time-based, which allows a check on penetration time.

Sensor offset  
Determination and recording of absolute sensor offsets apply to Class 1 and Class 2, at start and end of test.

Sensor drift  
For Class 1 and Class 2, sensor drift between start and end of the test should be less than the allowable uncertainty.

CPT termination  
CPT termination criteria are subject to agreed contract conditions, except for an inclination limit for the cone penetrometer of 20°.

Hole backfill  
This is subject to agreed contract conditions.

**Data Processing**

Data presentation  
Tabular or digital data presentation applies to Class 1 and Class 2. This includes time.

Cone penetrometer geometry  
The use of measured areas, rather than theoretical areas, of the cone base and the friction sleeve is permissible for calculation of q_c and f_r.

Corrections  
Data processing commonly includes data adjustments for sensor offsets and penetration interruptions. Class 1 and Class 2 requirements include explanation and reporting of such adjustments.

Thrust machine  
The mass and reaction system of the thrust machine can affect CPT parameters for the upper ground layer. Description of the thrust machine and its reaction system is a reporting requirement for Class 1 and Class 2.

3. CLASSIFICATION SYSTEM

The most important concept is the classification system for the accuracy of the CPT results. This permits the geotechnical designer to select a CPT class that suits the project requirements.

Classification systems are not unusual for geotechnical practice. A well known example is sample-quality formalised by, for example, NNI (1991b) and BSI (1981). ISSMFE (1989) introduced a classification system for CPTs by means of the CPT Reference Test Procedure. In Clause 5, ISSMFE specifies:

- taking into account all possible sources of error (parasitic frictions, errors of the measuring devices, eccentricity of the load on the cone with respect to the sleeve, temperature effects, etc...), the precision of the measurement shall not be worse than the following whichever is the greater:
  - 5% of the measured value
  - 1% of the maximum value of the measured resistance in the layer under consideration.
The precision shall be verified in the laboratory or in the field taking into account all possible disturbing influences."

Clause 10.10 specifies:

"When testing deviates from the Reference Test two classes of precision are defined:
- Normal precision: see Section 5
- Lower precision: the precision obtained shall not be worse than the following whichever is the greater
  - 10% of the measured value
  - 2% of the maximum value of the measured resistance in the layer under consideration.

In all such cases, the class of precision of the tests shall be indicated in the report and on the test graphs".

A particular problem for standardisation is the "measured resistance in the layer under consideration". The draft version of NEN 5140 gives a classification system that allows a more practical and commercial selection of a CPT class on the basis of the project requirements. Table 2 presents the classification system.

### Table 2. Classification system

<table>
<thead>
<tr>
<th>Class</th>
<th>Measurement</th>
<th>Allowable Uncertainty</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Cone resistance</td>
<td>0.05 MPa or 3%</td>
</tr>
<tr>
<td></td>
<td>Sleeve friction</td>
<td>0.01 MPa or 10%</td>
</tr>
<tr>
<td></td>
<td>Inclination</td>
<td>2°</td>
</tr>
<tr>
<td></td>
<td>Penetration depth</td>
<td>0.2 m or 1 %</td>
</tr>
<tr>
<td>2</td>
<td>Cone resistance</td>
<td>0.25 MPa or 5%</td>
</tr>
<tr>
<td></td>
<td>Sleeve friction</td>
<td>0.05 MPa or 15%</td>
</tr>
<tr>
<td></td>
<td>Inclination</td>
<td>2°</td>
</tr>
<tr>
<td></td>
<td>Penetration depth</td>
<td>0.2 m or 2 %</td>
</tr>
<tr>
<td>3</td>
<td>Cone resistance</td>
<td>0.5 MPa or 5%</td>
</tr>
<tr>
<td></td>
<td>Sleeve friction</td>
<td>0.05 MPa or 20%</td>
</tr>
<tr>
<td></td>
<td>Inclination</td>
<td>3°</td>
</tr>
<tr>
<td></td>
<td>Penetration depth</td>
<td>0.2 m or 2 %</td>
</tr>
<tr>
<td>4</td>
<td>Cone resistance</td>
<td>0.5 MPa or 3%</td>
</tr>
<tr>
<td></td>
<td>Sleeve friction</td>
<td>0.05 MPa or 20%</td>
</tr>
<tr>
<td></td>
<td>Penetration length</td>
<td>0.1 m or 1%</td>
</tr>
</tbody>
</table>

The following comments apply:
- The allowable uncertainty (or inaccuracy) is the larger value of the absolute uncertainty and the relative uncertainty. The relative uncertainty applies to the measurement; not to the measuring range.
- Classes 3 and 4 are identical, except for the measurement of inclination. Class 3 requires determination of "penetration depth" relative to a datum. "Penetration length" applies to Class 4.
- The accuracy requirements for the sleeve friction include the uncertainty of the geometry of the friction sleeve. In particular, the diameter of the friction sleeve is equal to or greater than that of the cone base. Estimation of the "end resistance" on the friction sleeve is difficult.

The accuracy classes include uncertainties of 50 kPa, 250 kPa and 500 kPa for the cone resistance q_c. The uncertainty of 500 kPa applies to common (good) practice. The accuracy requirements cover factors such as variations of the geometry of the cone penetrometer, calibration accuracy, drift of electronics resulting from temperature effects, etc. The classification system also covers accuracy of sleeve friction, depth below datum and inclination from vertical.

The classification concept relies on:
- company-specific detailing of procedures
- uncertainty analyses.

An example of a specific procedure is a temporary penetration interruption below the transition zone of dense sand to clay. This reduces sensor drift resulting from transient heat flux through the penetrometer (Post, 1995). Another example is compensation for heave of the thrust mechanism by incorporation of an electronic heave compensator.

Schaap and Zuidberg (1982), Zuidberg (1988) and Post (1995) present background information on uncertainty analyses. The draft standard presents informative guidance, in particular about potential errors that are not directly measurable in a calibration laboratory. Such errors include (Dunnicliff, 1988):
- gross errors, for example due to inexperience, misrecording and computational errors
- systematic errors, for example due to improper calibration, loss of calibration, hysteresis and non-linearity
- random errors, such as caused by electrical and digital noise
- conformation errors, for example inappropriate design limitations and incorrect test procedures
- environmental errors, such as due to vibration and shock waves, eccentric loading of sensors, water and air pressure variations, temperature shift, variations in electric source, corrosion and interference.

4. PRACTICE
Penetration rates of 60 mm/s and cone penetrometers worn to pencil size are not unattractive to CPT contractors operating in a competitive environment. From the Client's perspective, it is often difficult to enforce strict adherence to a CPT standard. The new concepts for CPT standardisation do not directly address this problem. However, the new concepts suit the accreditation of the quality system of a CPT operator while retaining a measure of freedom and allowing room for innovation. This meets the change in industry practice, which already shows progress with self-regulation of the bona-fide companies in The Netherlands.

In The Netherlands, cone penetration tests primarily serve to establish stratigraphy. In addition, they are the basis for direct (CPT) design of end bearing piles in sand (NNI, 1991f). Classes 3 and 4 are generally adequate for these situations. Accreditation systems for Classes 3 and 4 are largely in place.

Class 2 permits derivation of geotechnical parameters for stiff clays and sands. The Class 2 requirements correspond to specific foundation design situations and are currently applicable to some advanced CPT contract in The Netherlands. It generally requires rigorous procedures, but is feasible with currently available apparatus.

Class 1 suits soft ground engineering, including derivation of geotechnical parameters such as angle of internal friction, undrained shear strength and soil stiffness. Another example is the application of direct (CPT) design methods for friction piles in firm and stiff clays. Testing to the accuracy requirements of Class 1 is currently very difficult. It requires a very high standard of care, "well-tuned" apparatus and favourable ground conditions. The expected price tag is considerable and the commercial market is yet to be explored.

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Uncertainties in Cone Penetration Testing

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SYNOPSIS: The general increase in quality awareness also increased the need for more awareness and quantification of uncertainty in electric Cone Penetration Testing. A number of sources of uncertainty contribute to the overall uncertainty of the CPT results. Two important sources of uncertainty in a modern CPT system are discussed, namely the measurement of depth and the variation of recorded cone penetrometer signals due to temperature variations. Current practice of careful application of proper procedures results in an overall uncertainty that is sufficient for the majority of CPTs. However, special procedures or dedicated equipment may be required to meet the specified uncertainty for special applications.

1. INTRODUCTION
Cone Penetration Testing (CPT) has been a well established soil investigation technique for several decades. While the principle of the test has not changed, the equipment used to perform the test improved drastically over the last decades. With the increasing need for quality management, a key issue emerges with respect to the quality of CPT results, i.e. what uncertainty has to be assigned to the measured CPT parameters.

This paper discusses the overall uncertainty in CPT results, as a result of the complete CPT process. Focus will be on a state-of-the-art electric CPT system as currently in use throughout the Fugro Group.

The CPT process is divided into several components that are discussed individually. Keywords include penetration depth and verticality, cone penetrometer, data transmission and recording, operator and procedures and data normalisation. Influence of temperature on the electronics system (load cells, transmission and recording), e.g. heating up of the penetrometer tip during penetration in sands, will be discussed in detail. Results of dedicated laboratory and field measurements are presented. Suggestions are given to avoid unnecessary uncertainties and how to deal with the inevitable remaining uncertainty.

This paper focuses on the measurement of the cone resistance (qC). Similar considerations are applicable to other measured parameters.

2. SOURCES OF UNCERTAINTY
The discussion on uncertainty is limited to properly executed CPT’s following proper procedures and using proper equipment. Uncertainty caused by negligent execution of the test and operator errors is ignored.

The total uncertainty in CPT results can be grouped into three categories:
A - The uncertainty in measured data. This uncertainty includes calibration uncertainty, influence of ambient variables on the measurement system and influence of applied operational and measuring procedures.
B - The uncertainty in the position where the data are measured. This uncertainty is a result of the penetration process and applied operational and measuring procedures.
C - The uncertainty about representativeness of the data. Usually, measured data are transformed into more general design
parameters. The uncertainty associated with a design parameter is related to the uncertainty in the individual measurements and the amount of measurements. Furthermore, the required parameter may have been influenced by the measurement process, e.g., a CPT performed close to a previous borehole.

3. COMPONENTS OF THE CPT PROCESS

3.1 Penetration depth and verticality

Various phenomena influence the determination of the cone penetrometer position, besides the registration of the penetrated CPT rod length. The phenomena include start position, non-vertical penetration, thrust machine heave, elastic compression of CPT rods and connections, and CPT rod bending. Where start position and non-vertical penetration are an issue at all loads, the influence of other phenomena increases with increasing load. The sensor which measures the length of the penetrated test rods does not have to contribute significantly to the overall uncertainty. Current systems have an uncertainty of less than 0.1% of the measured length.

Heave of the CPT thrust machine increases the measured CPT rod length, but not the cone penetrometer penetration. Deformation of the CPT thrust machine may be relevant for positioning of the penetration recorder and possible heave compensation systems. Heave and deformation of modern heavy CPT trucks may be negligible, but for less sturdy CPT systems both heave and deformation must be considered.

The required start position of a CPT test is usually marked with a survey peg with a known position and elevation. The (visual) positioning of the cone penetrometer at the start of a CPT test relative to the survey peg is an obvious source of uncertainty.

Non-vertical penetration will cause a difference between CPT rod length and true vertical distance to reference level. Also a horizontal difference between position of entry point and actual position occurs. For example: a gradual increase in inclination to 20 degrees at 40 m penetration length results in a difference between depth and penetration length of 0.45 m and a (maximum) horizontal shift of 4.4 m (!). The application of an inclinometer in cone penetrometers allows the operator to monitor the deviation from the vertical. The operator can terminate the CPT in case of excessive inclination to limit the possible uncertainty. An accurate measurement of the inclination allows correction of depth.

Elastic compression of CPT rods and connections under true vertical loading at maximum capacity (200 kN) is approximately 0.1% of the length of the rods. True vertical loading however only occurs in theory. Some form of bending will occur. A combination of helix and curve type deformation will occur at high load and may shorten the distance between the cone penetrometer and the top of the CPT rods by an additional 0.1% of the length of the rods. When the load is reduced, the string will rebound to its most “straight” position. Reduction of load occurs when load applied at the top is reduced, or when the reaction force decreases, e.g., when penetrating from sand into clay. In the former case the top of the CPT rods will move upwards, in the latter case the rebound of the CPT rods may result in a more rapid penetration of the cone penetrometer into the clay. During high thrust CPTs at significant depths, (upward) rebound lengths of over 0.1 m have been observed.

The above overview of possible uncertainties in the determination of the position of the cone penetrometer shows that great attention must be given to careful registration of penetration length and careful application of corrections and compensations for the above phenomena.

Frequent verification of proper working of all systems by an independent measurement of the penetrated CPT rod length, directly related to the ground surface, is good practice.

3.2 Cone penetrometer

The uncertainties affecting the output of the cone penetrometer include: calibration uncertainty, geometrical variations, errors in load transfer, non-centric loading, inclined loading, ambient pressure variations and temperature variations. A description of Fugro cone penetrometers can be found in Schaap and Zuidberg (1982) and Zuidberg (1988).

Calibration uncertainty here covers the
difference between the assumed linear load cell output and the actual load cell output, so the
 calibration uncertainty includes non-linearity and hysteresis. Current standard practice results
in a calibration uncertainty of better than 0.5% of the measured value.

Standards like NEN5140 (1995) allow geometrical variation of the cone diameter of about 2%. The uncertainty in \( q_c \), is 2% of the measured value, if the nominal cone area is used to translate the measured force in \( q_c \). Measurement and application of the actual cone diameter can reduce the uncertainty and may be
required.

The uncertainty caused by improper load-transfer and non-centric loading is not significant in well-designed cone penetrometers. The uncertainty caused by inclined loading due to non-vertical penetration is very difficult to determine and is usually not considered. It is obvious that the measurement of \( q_c \) is influenced by the relative orientation of the penetration direction. By convention CPTs must be performed vertically.

Variations in water pressure during a CPT influence the \( q_c \) measurement and hence cause uncertainty. If required, the use of a piezo-cone penetrometer enables measurement of water pressure and correction of \( q_c \). The resulting parameter is the total cone resistance \( q_t \).

Usually, some uncertainty remains because the water pressure is not measured in the groove behind the cone but immediately below the groove.

Influence of temperature variations is discussed in Section 4.

### 3.3 Data transmission and recording

Data are recorded in the cone penetrometer by strain gauges. Strain gauge output is amplified in the cone penetrometer and subsequently transferred to the data acquisition system at surface level. The use of amplified signals between cone penetrometer and surface ensures that the noise on the signal and tensile forces on the cable do not contribute significantly to the overall uncertainty. At surface level, the data are converted from analogue to digital data. The digital resolution must be such that the uncertainty is not influenced. Usually a

ratio resolution/uncertainty larger than ten is sufficient. The current 16-bit systems meet this
requirement. The standard sampling rate is 1 record/sec. Slower rates should only be
allowed after careful consideration of the consequences for resolution in depth of the
CPT parameters.

### 3.4 Operator and procedures

The operator is the key factor in the CPT process. Correct and clear procedures are
necessary, but proper application depends on the operator.

An experienced operator who works according to current company operational
procedures, will not contribute significantly to the uncertainty.

Current procedures already focus on minimising uncertainty and maximising

### 3.5 Data normalisation

Data normalisation includes subtraction of zero-load output. In case of over-water CPTs the
zero-load output includes output due to water pressure. Determination of the appropriate zero-
load output is crucial. Since the zero-load output is measured data, the uncertainty in the
normalised data must take both uncertainties into account.

Application of depth correction for non-verticality on the basis of measured cone penetrometer inclination (NEN5140, 1995) is sometimes required to meet low uncertainty
requirements. The uncertainty in inclination measurement influences the quality of the
correction. Usually rebound and heave are compensated automatically during recording.

### 4. TEMPERATURE VARIATIONS

Temperature variation in components of the electronics system is a crucial variable that
determines the uncertainty in CPT data, as with most other electronic systems. A variation in
temperature of a component may shift the zero load output. Under laboratory circumstances,
ambient temperature is kept within small margins to avoid influence of temperature variations on measurements. In CPT testing, however, temperature control is impossible.
Where theoretically the electronics components in the CPT truck could be kept at a constant temperature, the cone penetrometer temperature will be a result of the ambient temperature in the soil, the heat generated by friction acting on the cone penetrometer and heat generated by the strain gauges inside the cone penetrometer.

Temperature correction systems are applied in the current generation of cone penetrometers, but some influence of temperature variations remains. Uniform (= slow) temperature variations are compensated accurately, but non-uniform (= rapid) variations in temperature create heat gradients through the cone penetrometer body that cannot be compensated fully.

The complexity of the temperature regime in the cone penetrometer makes it difficult to assess the uncertainty caused by temperature variations. Therefore a simplified empirical approach was followed to enable quantification of uncertainty caused by temperature variations.

The following experiments were conducted:

Experiment A - Performing CPT tests with a special cone penetrometer, instrumented to measure internal temperature at 3 specific positions (figure 1). Output of the cone penetrometer is monitored. Temperature variations in the data acquisition unit are minimised.

Experiment B - Applying controlled temperature variations to a cone penetrometer, while the data acquisition unit is kept at laboratory conditions.

Experiment C - Applying controlled (ambient) temperature variations to a data acquisition system. Output is monitored, while the input is an accurately stabilised value. Input device and data acquisition unit are kept at a stable (laboratory) temperature.

![Diagram of temperature sensors in cone penetrometer](image)

Figure 1 - Location of temperature sensors in cone penetrometer

![Graphs showing CPT results with temperature measurement](image)

Figure 2 - Results of CPT with temperature measurement
The results of the experiments are specific to the geometry of the cone penetrometer used and should not be used other types of cone penetrometer construction.

Figure 2 shows a typical result of experiment A. It demonstrates the rapid increase in temperature while penetrating in sand, and the decrease in temperature when penetration is stopped. The different responses at different positions in the cone penetrometer clearly show that a heat flux exists in the cone penetrometer body. Furthermore, the dependency on the duration of the stops between individual strokes is clearly illustrated during the prolonged stop at 3 m. Heating of the cone during penetration in sands was estimated at 1°C/MPa(\(q_c\)), assuming a linear relation between \(q_c\) and heating of the cone.

Experiment B was conducted to simulate the process during the actual test, to assess the influence of the temperature variations on the output of the cone penetrometer. The inside temperature and temperature gradient as measured during the CPT was reproduced by manipulating both ambient temperature around the cone penetrometer and the temperature of the cone itself. The apparent shift in zero load output was monitored. Results of a typical test result are shown in figure 3, which shows clearly the effect of the temperature compensation system. After 5 minutes the cone penetrometer output is fully adapted to the new temperature. Within this experiment, the maximum shift in apparent zero load output is about 130 kPa during the rapid fall in temperature.

Figure 4 - Results of temperature variations on output of data acquisition unit

The results of a typical test in experiment C are shown in figure 4. Figure 4 shows that the data acquisition unit adapts relatively slowly to ambient temperature variations. Assuming the unit was switched on well in advance of the start of the test, the temperature variation in the unit depends on the ambient temperature variation. If a 5°C variation over the duration of a CPT is considered to be representative, then the data acquisition unit contributes about 10 kPa to the uncertainty in the \(q_c\) measurement.

5. PROPOSED PROCEDURE
Since uncertainty is an implicit property of measurements, an allowable uncertainty must be specified. The allowable uncertainty must be determined depending on the expected use of the data. To meet specified uncertainty levels, operational procedures must aim at minimizing temperature differences, especially the temperature difference between measurement of the zero load output and measurement in the zone that is crucial for the uncertainty. This includes a penetration stop to allow temperature stabilisation when entering a soft clay stratum below a sand.
Electric systems must be switched on well in advance of the start of the CPT, to ensure a stable internal temperature. Operational procedures must include regular checks on system performance, such as an independent measurement of penetration length.

Since the measured data are not used directly in calculations, different uncertainties may be defined for an individual CPT data point and for the engineering input. This statistical approach can only be applied in case the uncertainty in each individual CPT data point can be treated as stochastic. Where (a part of) the uncertainty of an individual data point is systematic, the generalised parameter will have the same (systematic) uncertainty, plus the remainder of the stochastic uncertainty. Most of above described uncertainty sources result a combination of stochastic and systematic uncertainty.

6. CONCLUDING REMARKS

Above considerations are important parts of the overall assessment of uncertainty in CPT testing. Including the above considerations, the overall uncertainty in q, and depth for Fugro cone penetrometers is estimated as follows:

1 - during standard adverse operations, following normal procedures: 200 kPa or 5% of the measured value and 0.2 m or 2% of the measured depth.
2 - following special procedures: 100 kPa or 3% of the measured value and 0.1 m or 1% of the measured depth.

Above uncertainties are low enough for most engineering applications.

Special procedures and equipment may be required to meet the uncertainty levels required for specific applications. Examples of these specific applications are CPTs in very soft clays and CPTs with a small uncertainty in verticality. Special equipment may include measurement of temperature inside the cone penetrometer.

Favorable conditions include measurements in soft clays, where no heating occurs and offshore CPTs, usually performed in a stable temperature environment.

Resolution during profiling by CPT depends on relative inaccuracies and is therefore far better than the absolute measurements discussed herein.

7. REFERENCES

Seascout Mini CPT System

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SYNOPSIS: A miniaturised Cone Penetration Test (CPT) system has been developed that utilises a 1.0cm² cross section cone penetrometer rather than the conventional 10cm² or 15cm². Considerable experience has now been gained with it in the marine environment and results indicate that it produces data comparable with full size cones and that it is robust and adaptable enough to become much more widely used in both the marine and terrestrial environments.

1. INTRODUCTION
Over the last 25 years CPTs have become the oil industry’s most favoured method of testing seabed soils in situ. As described by Lunne and Powell, (1993), the test is rapid, accurate and repeatable and can be adapted to measure a variety of additional parameters.

The main drawback of seabed CPT units such as the ‘Seacat’, (Zuidberg et al. 1986), or ‘Seaspire’, (Power and Eastland, 1985) is their size. Using conventional 10cm² or 15cm² cones typically requires a unit weighing between 10 tonnes and 20 tonnes to provide sufficient thrust and reaction to achieve acceptable penetrations in “hard/dense” seabed soils. Smaller units such as the ‘Seabed WISON’ (Kolk and Power, 1982) weighing around 5 tonnes are available but in very dense sands for example this may limit achievable penetration to two to three metres or less.

To deploy such units therefore requires a dedicated geotechnical drilling vessel with a central moon pool, or vessels with large cranes or substantial ‘A’ frames. Such vessels are expensive and cost has therefore tended to limit an even more widespread use of the seabed CPT as a means of soil profiling and parameter measurement.

With the launch of the ‘Seascout’ system which weighs around 1 tonne in air, yet has the penetration capability of an equivalent 10 tonne unit, this problem has been overcome. This paper describes the equipment and its evolution, experience with it to date and how the results compare with “full size” cone tests. It also considers the wider applications of such a system.

Fig. 1. Seascout being deployed
2. HISTORY
For many years Fugro and its subsequently merged competitor McClelland had been looking at the possibilities of using a 1.0cm³ cone for a number of applications. These included truck mounted terrestrial systems for “environmental” investigations and a hand held unit for use on snow slopes to aid avalanche predictions (Schaap and Föhn, 1987). A small marine unit was eventually developed for use in shallow inland waters. However several factors limited the possibilities for deep water marine applications. These included:
(a) the ability to apply sufficient thrust to a thin cone rod without crushing or buckling it
(b) sufficient miniaturisation of electronic components to allow instrumentation to be built into the cone to measure all of the desired parameters i.e. cone tip resistance sleeve friction and excess pore water pressure, as well as an inclinometer to ensure verticality of the test
(c) Suitable material for the coiled cone test rod (see 3.1 below) that could be coiled and straightened enough times for a cost effective number of cone tests to be performed.

Many years of research and development and prototype tests followed and in early 1994 a trailer mounted unit using a straight rod was ready for trials on land in the Netherlands. The results were very promising and in August of that year the first ‘wet’ trials were performed in a Dutch canal using a coiled rod system. The total length of the coiled steel rod was approximately 7.5 metres, allowing a test penetration of up to 5.8 metres. This was the maximum length that could be manufactured as a continuous rod to the required specifications. A source of longer rods has now been located which should permit penetrations of 10 metres to be achieved in future.

The first commercial contract with the equipment was successfully performed in September 1994 and to date (July 1995), 6 contracts have been awarded, amounting to over 100 successful tests. Two units are currently in operation.

3. SYSTEM DESCRIPTION
3.1 SEASCOUT Apparatus
The SEASCOUT apparatus in its present configuration, Figure 2, comprises 5 main components, namely: a deployment/reaction frame, an hydraulic power pack, a thrust mechanism, a cone penetrometer on a coiled steel push rod and an electronic data acquisition system. The tubular steel deployment frame is an equilateral triangle in plan view with sides of 2.0m in length and the height is 2.4m. It weighs approximately 1 tonne in air but additional ballast can be added if required. The thrust mechanism is a patented development of the Fugro “Wheeldrive” system (Power and Eastland, 1986) in which articulated rather than solid drive wheels grip the push rod and move it up or down when rotated. The steel push rod is wound in a coil to make the apparatus more compact and easy to handle. It passes through a mechanical straightening device before entering the thruster. The cone has a 60° apex, a cross sectional area of 1.0cm², a friction sleeve behind the cone with an area of 15cm² and an internal 10kN load sensor and inclinometer. Pore pressure measurement is an option and is made via a porous element in the ‘face’ or at the base/shoulder of the cone.

Test data are transmitted in real time via an umbilical to the surface where they are displayed graphically on the operators computer screen and also stored digitally for subsequent processing and presentation.

4. OPERATIONAL PROCEDURES
The unit is transported fully assembled and can typically be mobilised on a vessel and ready to sail within an hour. At the test site the unit is lowered to the seabed and the test activated via the operator’s portable computer. Currently two power/lifting configurations are available: in one the unit is deployed on a standard steel lifting cable and there is a separate power/signal umbilical to the unit. For deep water (current system limit is 600m) the seabed unit is powered by a battery pack on the frame and
5. EXPERIENCE TO DATE

5.1 Projects Completed
Projects that have been performed with the Seascout system so far, include:
- an investigation into seabed compaction offshore Holland
- the evaluation of anchor locations for a Floating Oil Production Unit in the Irish Sea.
- a geotechnical survey of the Qatar - Pakistan pipeline route - with tests performed in water depths of up to 430 metres
- a number of major North Sea pipeline route surveys
- and as a standby tool on a pipeline installation post lay survey spread, intended to immediately investigate ground conditions at sites where trenching difficulties occurred.

5.2 Results Achieved
In 90% of tests performed the maximum achievable penetration of around 5.8 metres was obtained in soil conditions ranging from Very Soft Clays to Very Stiff Clays and Very Dense Sand. Most tests stayed within a verticality tolerance of ±10° and the number of cones and test rods lost or damaged in the ground was comparable with conventional CPT units. On a number of these projects tests were performed, as a calibration exercise, adjacent to previous test sites where “full size” cones had been used. Figure 3 shows the comparison between a Seascout test in Firm Clay/Dense Sand and a adjacent 10 tonne thrust “Seacat” test using a 15cm² cone. The tests were within 20 metres of each other in the Irish Sea and the main variations in the respective profiles can be attributed to the natural variability in local ground conditions.

Figure 4 shows the results of two comparative tests performed within metres of each other onshore in the Netherlands in Dense silty fine Sand, the “full size” test was performed with a 15cm² cone and a conventional 20 tonne truck-mounted CPT system. Again, the small differences between the test results were typical of the variability seen between adjacent full size tests at the site.
pore pressure. Sleeve friction measurements have been omitted for the sake of clarity but these exhibited the same level of agreement between tests. The Seacout results are shown in a typical report presentation format with interpreted soil profile and estimated shear strength (s_0) range. The estimated s_0 range has been arrived at by dividing the cone resistance by “N_s” factors of 15 and 20 which gives a good correlation with high quality laboratory test results in this type of clay. As can clearly be seen the results show remarkably good agreement. What they also illustrate is a potential advantage of the smaller cone which is the sensitivity of the cone tip resistance and pore pressure measurements to this interbedded layers. The correlation between results obtained with 1cm² and full size cones has been consistently high in all of the tests performed by Fugro to date. This level of agreement is also supported by a growing weight of commercial and academic research such as that reported by Cordosa de Lima and Tumay (1992).

6. SYSTEM APPLICATIONS

As described above, applications so far have included:
- pipeline route surveys
- anchor site evaluation
- seabed compaction control
- trenching problem investigation

Future applications are expected to include:
- predredge surveys
- river crossing investigations
- foundation investigation for subsea structures
- Jack-up drilling rig site evaluations
- seabed penetration predictions for dropped objects
- sewerage outfall investigations
- cable route surveys
- “difficult access” site investigations, offshore and on land.
7. SYSTEM ADAPTATIONS

7.1 ROV Option
Seabed CPTs have been performed in the past from ROVs (Geise and Kolk, 1983) but the size of the equipment and the reaction force available have limited test penetration to approximately one metre. However there is increasing demand for in situ tests in difficult access locations such as the back-filled trenches of laid pipelines (Power et al, 1994) where the assessment of the soil's ability to provide uplift resistance and thermal insulation can be critical. Another application is the engineering and environmental assessment of drill cuttings accumulations beneath and around offshore production platforms.

Design work has now been completed for a version of 'Seascout' that can be attached to an MRV (Multi Role Vehicle) as shown in Figure 6. Test reaction in this configuration can be provided by downward thrusters, water ballasting or suction anchors.

7.2 Trencher Option
A number of companies are currently investigating the possibility of incorporating a version of 'Seascout' on pipeline ploughs, jetting machines and other forms of trenching machine. The intention being to collect extra soils data in areas of operation to enable more detailed correlations between progress rates and soil types to be made and to perform measurements where unexpected difficulties are encountered.

7.3 Towed Sledge Option
Another design under consideration is a towed sledge version. This could be used with other tools and sensors to provide a continuous sub-

Fig. 6. Seascout - ROV version
7.4 Onshore Version

The equipment is also likely to have numerous onshore applications, mounted either on a small trailer or directly onto lightweight four wheel drive vehicles for difficult/limited access sites.

8. CONCLUSIONS

Considerable experience has now been gained with the Seascout Mini CPT system and the following conclusions can be drawn:

1. the unit is much easier to handle and deploy than conventional systems.
2. the design is robust enough to operate commercially in the marine environment.
3. the test results compare very well with those obtained with "full size" cones.
4. the versatility of the system design will enable it to be adapted for a wide variety of applications offshore, nearshore and onshore.

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ADDITIONAL RESULTS OF THE AMAP’sols

STATIC-DYNAMIC PENETROMETER

by:

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ABSTRACT: The new high performance AMAP’sols static-dynamic penetrometer has been performing well in France for the last three years. It was designed in Lyon in cooperation with Van Den Berg in Holland. It allows the use of various types of cone tips for static penetration, such as standard electronic 10 cm³ cone equipped with the friction sleeve as well as the piezocone, the Van Den Berg environmental cone and the cone with mechanical, telescopic transmission with surfaces from 12 to 50 cm². This penetrometer is capable of penetrations over 100 meters and can withstand cone tip resistance up to 140 MPa. Whenever the static cone meets refusal, further penetration may be achieved by dynamic penetration generated on the rods by the action of a very powerful hydraulic hammer. This has provided penetrations through layers which had never been completely traversed with the heaviest of other types of static-dynamic penetrometers. This paper presents results obtained with the AMAP’sols penetrometer adapted with different types of cones in various soil types commonly encountered in areas in and around Lyon and Le Havre, France.

1. INTRODUCTION

A summary of the development of the penetrometer was presented in 1994 (ref.1). Emphasis was placed on the static-dynamic penetrometer, invented in France in 1950. Evolutionary developments in 1967, in Lyon, permitted ever deeper penetrations of dense or gravelly soils (ref.2). Both the purely static and purely dynamic penetrometer types have advantages and drawbacks (ref. 3). The French developed the static-dynamic penetrometer in order to combine the advantages and eliminate the short comings.

In 1992, the AMAP’sols, truck mounted, static-dynamic penetrometer was developed which went into field trials, thereby providing its great advantage, in terms of deeper penetration, for use in many different soil types and conditions. Preliminary test results obtained in the city of Lyon were then presented and showed a penetration of 15 m into dense sandstone substratum (ref.1). This penetration by penetrometer of any other kind had never before been achieved.

Since then, at another site in Lyon, penetration was achieved through 21 m of dense, sandy gravel alluvium followed by 24 m into dense grey-tan sandstone. This stands as a penetrometer record in this kind of soil. This type of soil investigation is less expensive than the usual method of sampled bore hole.

2. THE AMAP’sols STATIC-DYNAMIC PENETROMETER

2.1 Principle of operation

In 1992, geotechnical engineers in Lyon and Saint-Etienne conceived a new static-dynamic penetrometer equipped with mechanical cone tip to improve this type of soil exploration. The important improvements brought about consisted of:

(1) Espey agréé par la Cour de Cassation, Lyon
(2) FONDAVICE, Lyon
(3) P.-D.G. d’AMAP’sols, Saint-Héand, Loire
(4) SETSOIL, Velux, Bouches-du-Rhône

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**Static mode**: totally automatic operations, penetration at 2 cm/sec with continuous numeric recording every 2 cm on memory board with simultaneous drawing of the cone resistance diagrams in real time. This allows for instantaneous control of the penetration.

The records and evaluations are transmitted by modem to the office which permits rapid engineering interpretation of the test results to evaluate the soil parameters needed for the determination of soil bearing capacities and settlements (ref. 4, 5 and 6).

**Dynamic mode**: the old fashion way of driving a probe by a free falling hammer was replaced by a very powerful, fast-action hydraulic hammer (see fig.1), with adjustable energy, capable of going through extremely dense layers and penetrate into altered bedrock.

**Installation**: the penetrometer is mounted on a 6x6 Mercedes truck, of 260 kN.

The leveling of the truck, to insure true vertical penetration, is done automatically by five hydraulic jacks controlled by electronic sensors. Four of the jacks bear on retractable track of the truck which is an auxiliary propulsion mechanism (see fig. 2).

Two of the tracks are fast and easy mobility on soft soil terrains where conventional trucks would bog down. They are a patented device of Van Den Berg of the Netherlands who has a great experience with the static penetrometer over many years. His high degree of technical expertise in hydraulic systems and the recording of data has greatly contributed to the success of the device.

The name AMAP'sols means "Ateliers Mobiles d'Ancullation par Pénétration des Sols" (Mobile Soil Testing Unit by Penetration).

### 2.2 Characteristics

This penetrometer offers the possibility of all manners of penetration into soils for specific purposes. It can push all the known measuring tips varying from 10 to 50 cm², be they of the mechanical or the electronic types such as piezoces and others.

Usually, static penetration is done either with a 50 cm² mechanical tip equipped with a skin friction sleeve of 250 mm in length, or with a 44 cm² tip and a 200 mm sleeve.
In the static mode, the following measurements are made:

- \( q_c \): cone resistance (up to 30 MPa)
- \( f_s \): skin friction, which permits calculation of the friction ratio, FR
- \( Q_{PD} \): total resistance to penetration (up to 220 kN)

When refusal is met at 30 MPa with the 44 or 50 cm\(^2\) tip in a hard soil layer, static penetration may be continued with a 12 cm\(^2\) tip not equipped with a friction sleeve. The static cone resistance of the 12 cm\(^2\) tip can reach 140 MPa. When refusal is met, dynamic penetration is used.

It is obvious that at this level of stress, the evaluation of the ultimate shear strength of soils no longer translates into significant physical meanings.

So, dynamic penetration is used only to get through dense soil layers.

For additional information, every 25 cm, a static test is performed up to 140 MPa (which is very high and well above the capacity of any other static penetrometer). Each time that \( q_c < 140 \) MPa, the static penetration mode is employed. A sound alarm is triggered every time any of the load limits are reached for each of the tubing configurations.

In the event of a sudden drop of tip resistance, the dynamic driving mechanism automatically stops instantaneously at the upper boundary of the less resistant layer. This prevents the penetration to occur without any measurements made of the softer layer.

### 2.3 Other uses

The penetrometer accepts different cone tips, such as the piezocone and the envirocone of Van Den Berg (Ref. 7).

In each of these two cases, the computer programs used are those defined by Van Den Berg.

Depending on its manner of use, the piezocone can measure \( q_c, f_s \) as well as pore water pressures.

With the envirocone, the following measurements, besides \( q_c \), are made possible:

- Soil conductivity
- \( H^+ \) and \( O^+ \) concentration
- Redox potential
- Temperatures
- pH and porewater pressure

This range of utilization brings about a considerable improvement of the resources available to study environmental and waste management problems, thanks to the quality of the data obtained.

### 3 - LE HAVRE HARBOR

The operation of the AMAP'sols penetrometer 12 cm\(^2\) cone tip, in its static mode, is different than that of the classical electric cone of 10 cm\(^2\). Therefore it was necessary to prove the reliability of the cone tip calculated values by comparing them with the 10 cm\(^2\) electrical and 50 cm\(^2\) mechanical cone tips.

The comparative tests were made easy due to the fact that any one of the three cone types could be used for a soil foundation study in Le Havre (France).

These interesting results were obtained in the sedimentary deposits at the delta of the Seine river, in the Le Havre autonomous harbor.

Fine sandy and silty soils with some peat and gravelly layers are present.

Initially, two soundings with the AMAP'sols were made to 37 and 31 m. Down to a depth from 12 to 15 m, the large 50 cm\(^2\) cone tip was used followed thereafter by the 12 cm\(^2\) when denser soils had to be penetrated.

Subsequently and for comparative testing purpose, AMAP 1 was replaced in AMAP 4 using the traditional electrical 10 cm\(^2\) cone. The results are superimposed on the diagram presented in fig.3. Correlations are very good between the electric sounding method and both the large and small AMAP'sols cone tips. Therefore, the following conclusions can be drawn:

For all practical purposes, the same results are obtained whether the 10, 12 or 50 cm\(^2\) cone is used. It is obvious that the larger cone has the tendency to smooth out the crests of the diagram due to the presence of gravels or the dips showing thin layers of soft soils.

These results prove the validity of the method, using the small AMAP'sols cone (12 cm\(^2\)).

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Fig. 3 - Comparison of penetration diagrams in Le Havre harbor
4 - BRIDGE NEAR CHAMONIX

A three-span bridge was constructed in 1993 over the Arve river, in Cluses Marnaz, near Chamonix. The hyperstatic spans were 19, 32 and 19 meters. Following a soil investigation made with two pressuremeter soundings of 30,50 and 33 meters, it was decided to resort to metal piles deriving their support in lateral friction. Their length were calculated at 25 m into loose sandy soils interspersed with silt lenses, at times very organic. An important artesian condition had been detected between 20 and 26 m deep in a medium dense gravelly sand layer. During the pile driving operation, it was found necessary to increase the pile length by 13 m and subsequently, by an additional 20 m after two additional pressuremeter soundings of 55,50 and 65 m. This gave a total pile length of 58 m. The initial soil exploration had only penetrated to 33 m. Whereas the added pile length proved satisfactory for the support of the right abutment as well as for the two middle supports, it turned out inadequate for the left bank which underwent important deformations. A subsequent investigation initiated by a lawsuit was made to determine what remedial measures had to be undertaken.

Consequently, four static-dynamic penetrometer tests were made with the AMAP’s sols penetrometer in order to determine the geotechnical characteristics of the soils through which the piles had been driven as well as those existing below the pile tips. These new tests reached the following depths:

on the left bank:
AMAP 1 = 70 m, AMAP 2 = 82 m
on the right bank:
AMAP 3 and AMAP 4 = 75 m.

None met refusal.

They were made with the large cone tip (50 cm³) with lateral friction measurements on a special friction sleeve down to the depths listed below:

20 m in AMAP 1
42 m in AMAP 2
38 m in AMAP 3
39 m in AMAP 4

Figure 4 presents the diagram of the AMAP 2 penetration.

Based on the information thus collected, the bridge was closed for a period of 4 months while improvements were made to the left embankment. These consisted of removing 6 m of soil fill and replacing it with a lightweight expanded polystyrene fill material and the installation of 8 vertical drains extending into the artesian aquifer.

5. DYNAMIC PENETRATION

It has been known for a long time that dynamic penetration into saturated cohesive soils should be avoided (ref.8). In other soil conditions however, this type of penetration is admissible even though the major and delicate problem then is to translate the penetration data into penetration resistance values of the conventional dynamic type, such as obtained by the Dutch formula, for example, which may be open to further discussion. This interpretation is made unnecessary with the data from the static-dynamic AMAP’s sols penetrometer. Thanks to its great capacity and when used with the 12 cm³ cone tip, soils with static resistances of up to 140 MPa may be penetrated. The dynamic mode therefore is only used to get through extremely dense soils which, once traversed, can be abandoned to return to the static mode of penetration, as soon as the resistance drops back down below 140 MPa. Consequently whenever the static penetration is over 140 MPa it is no longer of concern to evaluate the dynamic resistance or to translate the data into allowable bearing stresses. This is an important feature of the AMAP’s sols penetrometer.

6. COST OF AMAP’s sols TESTS

Practical experience over the last two years of operation of the AMAP’s sols penetrometer shows that soil investigations made with this manner are less expensive than any other methods.

In France, the unit rate per meter of penetration of the typical AMAP’s sols investigation is half as expensive as that of a good quality pressuremeter sounding and its cost is only about 30 to 33 % that of soil boring with sampling or drilling parameters.
Fig. 4 - Penetration diagram of the AMAP'sols static-dynamic penetrometer for the Cluses Marnaz bridge near Chamonix
7. CONCLUSIONS

This paper presents data which lead to the following three important conclusions, observed in France.

- Record penetrations in dense or hard soils are achieved.
- The important details of stratigraphy in dense and medium dense and soft and medium stiff soils remain in evidence.
- The static-dynamic method of soil investigation with the AMAP'sols device is considerably more cost effective than any other traditional method of investigation soils.

REFERENCES


Innovations with CPT for Environmental Site Characterization

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ABSTRACT: The Cone Penetrometer Test (CPT) represents an important new tool for use in environmental site characterization. Its role as both a primary investigation technique and a part of a synergistic exploration strategy are explained. The types of sensors either available, under development, or that have potential for near-term development that could be used with the CPT are listed.

Among the most promising sensing technologies are fluorescence detection systems. This approach begins with a laser mounted in the CPT vehicle that provides a strong ultraviolet light source. The light is launched down a fiber optic cable through a sapphire window in the CPT probe and onto the soil. The induced fluorescence is collected in a second fiber optic cable and returned to an optical detector located in the CPT vehicle. If petroleum hydrocarbons are present, a strong fluorescence will be detected, with the intensity of the fluorescence dependent upon the degree of contamination.

1.0 INTRODUCTION
The cone penetrometer test (CPT) represents an important new tool for use in environmental site characterization. It has significant roles as both a primary investigation technique and as part of a synergistic exploration strategy.

2.0 TRADITIONAL SITE CHARACTERIZATION
Environmental site characterization approaches, even to this date, are usually based on traditional drilling, sampling and laboratory testing. Characterization plans are formulated on informed judgment that considers site geology, the characterization requirements, and perhaps, prior information available from previous wells or drilling logs. The traditional approach involves laying out an array of drill holes, often in a rectangular pattern, with spacing determined from judgment or rules of thumb. Cuttings are examined in the field and samples are taken to the laboratory for testing and analysis. After periods ranging from a few weeks to several months, the scientists/engineers are able to assess the results of their plan. They then can order infill drill holes and monitoring wells. Plume characterization, because of its three dimensional nature, usually requires significant infill sampling and well monitoring, adding further time and cost to the investigation. The traditional environmental site investigation process is nearly stalled by the slowness of drilling, the lengthy times for laboratory testing and, overall, the lack of real-time decision-making capability. The DOD and DOE have recognized that to meet their cleanup time tables, the traditional Remediation Investigation and Feasibility Study (RI/FS) approach is inadequate.

2.1 Characterization Research Efforts
Research programs by the DOD and DOE in this area are in direct response to the need to identify and characterize contaminated sites
in a more timely and economical manner than is currently possible with traditional drilling, sampling and laboratory testing techniques (Bratton, et al., 1993, and EPA, 1988). The ultimate goal of these research programs is to develop a more complete contaminant distribution and geologic model that can be used to design more cost-effective remediation plans. As a result of this work, significant limitations in available site characterization equipment and analysis methods have been identified and programs have been created to develop the much needed improvements. The Tri-Service agreement to develop the Site Characterization and Analysis Penetration System (SCAPS), and the Air Force developed tunable dye laser system are examples of the DOD and DOE efforts to develop advanced field characterization tools (Bratton, 1993). A list of sensors and sampling tools that have been developed for the CPT by the DOD, DOE and private industry are listed in Table 1.

These research programs have significantly reduced the time required to acquire data and have greatly improved the data density.

However, the methods to analyze these data are essentially the same as were used 10 to 15 years ago. In the worst case, the engineer/scientist manually sifts through reams of data and develops contours of the contaminant plume using contouring programs. In the best case, the engineer/scientist has access to a database and couples the database to a contouring program to “automate” the process. In addition, drilling, sampling and analytical testing plans are still typically laid out on a rectangular grid with samples taken at set depth intervals (typically every 1.5 m), or changes in lithology. These rigid plans, although typical of the industry, often miss significant areas of contamination and are wasteful of the limited financial resources. It is common to discover that a high percentage of the monitoring wells at a site are in the area of non-detect, are screened at the wrong depth interval or are too shallow. While the well may be in a non-detect region, it still must be monitored on a quarterly basis (EPA, 1988), at high cost, yet adding no useful additional data.

<table>
<thead>
<tr>
<th>CPT SENSORS</th>
<th>CPT SAMPLERS</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Laser Induced Fluorescence</td>
<td>• Sampling Points for GWT Fluctuation and Periodic Samples (¼&quot; to 2&quot; temporary or permanent monitoring wells)</td>
</tr>
<tr>
<td>• Electrical Resistivity</td>
<td>• Soil Gas</td>
</tr>
<tr>
<td>• Temperature</td>
<td>• Soil Samples, Confirm CPT Soil Classification and Contaminant Information</td>
</tr>
<tr>
<td>• pH</td>
<td></td>
</tr>
<tr>
<td>• Oxidation-Reduction Potential</td>
<td></td>
</tr>
<tr>
<td>• Raman Spectroscopy</td>
<td></td>
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<tr>
<td>• Seismic Wavespeeds and Damping</td>
<td></td>
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<tr>
<td>• Soil Gas</td>
<td></td>
</tr>
<tr>
<td>• Piezo-CPT for Groundwater Flow Detection</td>
<td></td>
</tr>
<tr>
<td>• Ground Penetrating Radar</td>
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</tr>
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</table>
2.2 Intelligent/Adaptive Site Characterization

The authors, under contract to the Air Force, have developed an intelligent/adaptive site characterization methodology to meet this need. This methodology incorporates real time data analysis methods into the site characterization process. An adaptive site characterization methodology is defined as a method that can optimize the use of surface geophysical and cone penetrometer probe measurements, as well as traditional drilling, sampling and monitoring techniques. An intelligent methodology is one that involves interaction between predictive mathematical models and site measurements to both improve the predictions and make decisions optimizing the site measurements. The methodology incorporates four basic software functions that include: 1) site data manager, 2) graphic display, 3) geostatistical decision analysis, and 4) groundwater flow-based decision analysis models.

A schematic of the Intelligent/Adaptive Site Characterization process is shown in Figure 1. This flow chart begins with data collected by a variety of means, consisting of but not limited to, site reconnaissance, cone penetration testing with associated sensors, drilling to obtain samples for analytical testing, and surface geophysical methods. This data is entered into the database manager that creates a virtual site model. The database manager uses the virtual model to create site maps and fence diagrams. In addition to the these graphics, three-dimensional graphics can be prepared based on data contained in the virtual site model. Geostatistical modeling is conducted to perform parameter estimates (contour plots) and error estimates using data contained in the virtual site model. Information produced by the geostatistical model can be used in either of the two independent modes, both of which involve Bayesian decision analysis. In the first mode, the error maps are used to evaluate the benefits of additional measurements against

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**Figure 1. Schematic of adaptive/intelligent site characterization methodology.**

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the goal of the characterization plan and the cost of performing these measurements. If the costs outweigh the additional benefits of new data, then the measurement process stops. The other mode can also arrive at this decision by using a flow code to estimate if the modeling errors are sufficiently small that all parameters (i.e., plume shape, contaminant concentration) are known to the appropriate confidence level. For either mode, if additional uncertainties exist, additional measurements are collected and the characterization process is continued.

3.0 CONE PENETROMETER TESTING

The heart of a modern site characterization approach is the cone penetrometer test (CPT). The CPT provides a rapid, minimally invasive method of obtaining a detailed description of the site stratigraphy, ground truth data to enhance the analysis of surface geophysical data, samples for analysis of contaminant identification and concentration, and the ability to place short-term monitoring wells.

Of these CPT capabilities, real time sensing of contaminant concentrations is of the highest value in the site characterization process. Recently, many new sensors have been developed to determine chemical contamination in both vadose and saturated soil environments. Many of the sensors have been listed in Table 1.

3.1 Advanced CPT Probes

Among the most promising new contaminant sensors for the CPT are fluorescence detection systems. These sensors use a laser mounted in the CPT vehicle which provides a strong ultraviolet light source. It is launched down a fiber optic cable through a sapphire window in the CPT probe and onto the soil as depicted in Figure 2. The fluorescence excitation is collected in a second fiber optic cable and returned to an optical detector located in the CPT vehicle. If a fuel is present, a strong fluorescence will be detected, with the intensity of the fluorescence dependent upon the degree of contamination.

![Figure 2. Schematic of Laser-Induced Fluorescence CPT probe.](image)

The first generation of these systems used laboratory lasers that were not ideal for field applications. New systems being developed that use smaller lasers and/or downhole light sources which eliminate the laser in the CPT truck.

A typical fluorescence spectrum for one of the new downhole light sources is presented in Figure 3. The spectrum was obtained from a sand sample contaminated with various levels of jet fuel using an excitation wavelength of 254 nanometer (nm). The peak fluorescence is at 358 nm, which is near the peak naphthalene emission. By preparing laboratory samples or by obtaining the resultant spectrum, calibration curves can be developed which relate the emission spectrum to soil contamination, as shown in Figure 4. The calibration curve is then used to relate field-determined, CPT-obtained laser induced fluorescence (LIF) profiles to fuel contamination.
methods of turning the light along with having diverging or converging light at the sample. Results indicated that an optical module which focused (or converged) light on the sample and also filtered the return signal produced significantly better signal-to-noise ratios than other diverging methods. Although total signal strength at the sample is slightly reduced due to the additional optics, the energy density applied to the sample is increased because the light is focused to a small spot size. The optical filters also provide a reduction in the background noise, creating a better signal-to-noise ratio.

A major advantage of CPT based fluorescence detection systems is that the thickness of the contaminated soil can be determined more rapidly and more thoroughly than with conventional laboratory sampling. The data is obtained and displayed during the sounding. Having the CPT-based detection system available in real time allows the on-site analyst to make timely decisions regarding the investigation program, unlike the laboratory-based methods which may require several mobilizations to fully characterize the site.

4.0 SUMMARY AND CONCLUSIONS

With the rapid advances being made in CPT technology, there are a variety of new ways that it can be applied to environmental site characterization. Recent sensor advances have significantly enhanced our abilities to detect contamination in-situ. CPT-installed wells and CPT samples allow users to confirm the results from the sensor and meet regulatory requirements. Recent fluorescence detection systems have been effectively demonstrated in this aspect and confirmed with numerous site samples.

The new CPT technologies not only provide improved methods but also significantly more detailed data at a reduced cost. Real-time continuous sensors now provide sufficient data volume for the application of geostatistics and volume rendering analysis methods. Geostatistics can be applied to both improved modeling...
programs and assisting with selection of the next sampling location. This intelligent/adaptive approach to site characterization ensures efficient use of site characterization funds while obtaining near optimal testing locations. It is this type of approach that will allow governments to meet their remediation goals within their limited budgets.

5.0 REFERENCES


Advanced deep cone penetration testing and backfilling in overconsolidated clay

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SYNOPSIS: The maximum depth of conventional cone penetration testing (CPT) in stiff, overconsolidated clay is often limited by the maximum thrust that can be applied with the available equipment. The cause is the high friction between the CPT rods and the clay. This problem occurred during a preliminary soil investigation for a bored tunnel in The Netherlands. The CPT’s resulted in refusal after only 5 m of penetration in the overconsolidated Boom clay.

Consequently an advanced method of friction reduction has been developed and applied successfully in a detailed soil investigation for the tunnel. During 35 m of continuous penetration in stiff, overconsolidated clay (cone resistances between 4 and 6 MPa), the required thrust was limited to 20 kN only. CPT’s up to 62 m depth have been performed without reaching the (maximum) thrust of 200 kN. By using the same method, backfilling of the CPT holes during pulling of the rods was applied.

1. INTRODUCTION
Cone penetration testing in stiff, overconsolidated clay can be rather frustrating. Due to the cumulative friction between CPT rods and clay, the maximum thrust (in The Netherlands occasionally about 200 kN) is then already encountered, before the planned CPT depth is reached. The CPT results in a refusal. Part of the soil conditions remain unknown, unless alternating drilling and cone penetration testing is applied. But these techniques are often time-consuming and expensive.

In 1994 a detailed soil investigation had to be carried out for a bored tunnel under the river Western Scheldt. During the previous preliminary site investigation, the CPT’s resulted in refusal after only 5 m penetration in the stiff, overconsolidated Boom clay. Therefore a sophisticated method of friction reduction between the CPT rods and the clay has been developed and applied. As the Client ordered proper backfilling of the CPT holes, the method has also been used for backfilling of these holes.

In this paper the method of friction reduction and backfilling is described. The detailed soil investigation for the bored tunnel will act as a case study. A project description will be presented, followed by a brief overview of the geological setting. The soil investigation and the results will be discussed, followed by the conclusions and final remarks.

2. FRICTION REDUCTION
The developed method of friction reduction is based on controlled injection of a mud between the CPT rods and surrounding soil during penetration. This basic idea of friction reduction is not entirely new and applied in several different configurations of the cone and rods (for instance Diggle, 1983).

The first rod above the cone is perforated with openings, to provide the mud injection between the rods and surrounding soil. The small openings are located above the zone of influence of the conetip and friction sleeve. Consequently, the CPT results (measured cone resistance, sleeve friction and pore pressure) are not affected by the mud injection. A pump provides controlled mud injection during the test, creating a lubricating mud film between the rods and the surrounding soil.
The presence of the mud film results in a considerable decrease of friction between the CPT rods and the soil. A schematic configuration is shown in Figure 1.

**Figure 1. Method of friction reduction**

3. METHOD OF BACKFILLING

Through the mentioned injection openings it is possible to backfill the CPT hole directly after completion of the CPT. While pulling the CPT rods, a bentonite grout mixture is injected under high pressure, thus backfilling the CPT hole. The same pump is used.

At least twice the theoretical volume of the CPT hole is injected. The injected volume per m pulled CPT rod is measured, to control the backfilling process. The required stiffness of the backfilling depends on the surrounding ground conditions and can be regulated by the type of bentonite grout.

The sealing of the backfilling method has been verified. A backfilled CPT hole has been sampled by the Delft continuous sampler. This sampling system has been developed by Delft Geotechnics as well and provides continuous undisturbed samples of class 1 quality (BSI, 1981). Analysis of the backfill samples confirmed a very effective sealing of the CPT hole.

The procedure of friction reduction during penetration and backfilling while pulling the rods has only little effect on the execution time of the CPT. Before the test starts, the mud has to be prepared and mixed. The mud drum and the pump can be placed in a restricted working environment, for instance inside a CPT truck or on a small working platform.

4. PROJECT DESCRIPTION

The most south-western part of The Netherlands, Zeeuwisch-Vlaanderen is separated from the rest of The Netherlands by the river Western Scheldt. Zeeuwisch-Vlaanderen can be reached by ferries, or over land via Belgium. Because the ferries need to be renewed in the near future, a fixed link between Zeeuwisch-Vlaanderen and Zuid-Beveland is proposed. It concerns a bored tunnel between the city of Terneuzen and the town of Borssele. The map of Figure 2 shows the project area and the tunnel route.

The Western Scheldt has a width of about 6 km. Two deep trenches, with a maximum depth of 35 m, are extensively used shipping routes. In the middle of the Western Scheldt a mudflat is situated. The tidal variation is about 3 m.

**Figure 2. Project area**
5. GEOLOGICAL SETTING

5.1 Regional Geological Setting

Regarding the depth and the location of the tunnel, Tertiary and younger sediments are of interest. During the Tertiary period, times of sedimentation were alternated by periods of erosion, due to tectonics and sea level fluctuations. Marine sands and clays were deposited.

During the Quaternary period, the geological setting was influenced by climatic conditions, in particular several glaciations. During these glaciations strong erosion occurred. Deposition of marine sands in the early Quaternary period was followed by deposition of fluvial sands. Due to the sea level rise during the Holocene epoch, marine influences controlled the sedimentation of sand and clay and the development of peat. Because of tidal flow, erosion resulted in deep trenches in the actual Western Scheldt. These trenches reach into the Tertiary layers (RGD, 1992).

5.2 Boom clay

The Boom clay was deposited during the Oligocene (Rupelian) epoch in the Tertiary Period. These deposits are found in The Netherlands and Belgium, but also in Germany, Denmark and Poland, according to Declee et al. (1983).

The Boom clay is a detritical, marine deposit. The clay minerals in the Boom clay are mainly kaolinite, illite and smectite. Ferricarbonates (for instance pyrite) were formed, due to the presence of organic material. Calcareous nodules (septaria) were developed by redistribution of carbonates (Vandenberge, 1978).

The main part of the originally deposited Boom clay layer (with an estimated thickness of over 100 m) was eroded at the end of the Oligocene epoch (Schiteeet al., 1983). The remaining Boom clay is thus overconsolidated, which explains its (very) stiff consistency.

6. SOIL INVESTIGATION

6.1 Objectives and programme

For the design of the bored tunnel a detailed site investigation was necessary. Previous site investigations in the area showed the presence of the stiff, overconsolidated Boom clay. The Boom clay appears to have rather good geotechnical conditions for tunnel boring without a supporting fluid. The main objectives of the detailed soil investigation were therefore to locate the top and (very important) the bottom of the Boom clay in the proposed tunnel route, and also to obtain detailed information about its geotechnical characteristics.

An extensive soil investigation programme was carried out, consisting of 66 CPT’s, 44 borings and a few pressuremeter and dilatometer tests. Because the traditional CPT’s showed refusals after 5 m of penetration into the Boom clay, application of the friction reduction method was necessary to reach and identify the bottom of the Boom clay. The stiff Boom clay was sampled by core drilling techniques. Many samples were tested in the Delft Geotechnics’ laboratory on index, strength and deformation parameters.

6.2 CPT’s

The CPT’s were performed in the Western Scheldt and on both shores. In the Western Scheldt, the CPT’s were performed from a jack-up platform and by using a vessel. Because of the combination of the tidal variations of about 3 m and maximum waterdepths of about 35 m, strong currents had to be countered by using heavy support casings.

All CPT’s were performed by application of the presented friction reduction method. The Dutch standard for static cone penetration tests NEN 3680 (NNI, 1982) was applied. Piezocones (with the filter element just behind the cone tip) were applied, to provide a maximum of data for soil classification purposes. Also insight in the existing hydrological conditions was obtained, for instance indications about the in situ waterpressures in the deep sandlayers underlying the Boom clay.

The CPT holes have been backfilled by using the presented method, in order to avoid
possible difficulties caused by open CPT holes, during boring of the proposed tunnel.

7. RESULTS
A typical CPT result in the Boom clay is presented in the graph of Figure 3. The top of the Boom clay is located at a depth of 24 m - NAP. The bottom of the Boom clay is situated at 60 m - NAP. The graph shows the presence of sand above and below the Boom clay.

The measured cone resistances in the Boom clay are typically between 4 and 6 MPa, indicating a (very) stiff clay. There is no general increase of cone resistance with depth measured, which confirms the overconsolidation. Below 45 m - NAP, peaks in cone resistance (up to 10 MPa) reflect the presence of interbedded sandy layers.

Both excess pore pressures and negative pore pressures were measured during penetration in the Boom clay, indicating both contractive and dilative response of the soil (Campanella and Robertson, 1988). Negative pore pressures were also measured in the mentioned interbedded sandy layers within the Boom clay.

The sleeve friction in the upper part of the Boom clay is typically between 0.15 and 0.20 MPa. A decrease in friction is occasionally measured at 1 m intervals. This decrease is apparently due to dissipation during stops in penetration. These pauses are necessary for the installation of the next CPT rod after 1 m of penetration. In the lower part of the Boom clay, below 40 m - NAP, the sleeve friction decreases (with increasing depth) and varies generally between 0.07 and 0.10 MPa. The friction ratio decreases (with increasing depth) from 5 to 2%.

The CPT results show that the Boom clay can be divided in an upper and a lower layer. The upper layer (BK1, up to 45 m - NAP in Figure 3) can be considered as more or less pure clay, while the lower layer (BK2, starting at 45 m - NAP in Figure 3) is more sandy. Correlation with laboratory results confirms this subdivision, as can be concluded from the average values of the index parameters shown in Table 1.

Table 1. Average indexparameters of the Boom clay layers

<table>
<thead>
<tr>
<th>Boom clay layer</th>
<th>WC (%)</th>
<th>PL (%)</th>
<th>LL (%)</th>
<th>PI (%)</th>
<th>&lt; 2 μm (%)</th>
<th>c_u (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BK1</td>
<td>26</td>
<td>25</td>
<td>76</td>
<td>51</td>
<td>58</td>
<td>140</td>
</tr>
<tr>
<td>BK2</td>
<td>27</td>
<td>22</td>
<td>68</td>
<td>45</td>
<td>41</td>
<td>90</td>
</tr>
</tbody>
</table>

WC = water content
PL = plastic limit
LL = liquid limit
PI = plasticity index
< 2 μm = clay fraction
c_u = undrained shear strength
Figure 3. Typical CPT result in the Boom clay
8. CONCLUSIONS AND FINAL REMARKS

The presented method of friction reduction and backfilling by injection proved to be very useful in an extensive soil investigation in (very) stiff, overconsolidated clay. Deep (over 35 m) penetration through the Boom clay layer was easily performed, without refusals due to friction. The method did not slow down the entire cone penetration process and was applied in a restricted working environment. Piezocones were used, application of other special cones is very well possible.

Since its development, both the friction reduction and backfilling method have been applied successfully in many other geotechnical and environmental soil investigations. In environmental investigations the possibility for backfilling is of particular interest, because it minimises the risk of flow of possible contaminated material through the CPT hole.

From the experiences gained so far it is expected, that the method can be applied successfully in other difficult geotechnical environments as well, such as stiff London clay or tropical residual soils.

9. ACKNOWLEDGEMENTS

The author appreciates the permission of Kombinatie Middelplaat Westerschelde v.o.f. for publication of this paper.

10. REFERENCES


The ROTAP: a useful tool for the execution of cone penetration tests.

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SYNOPSIS: One of the most important limitations on the use of cone penetration tests is the presence of stone concretions in the subsoil. When these concretions have a cone resistance of more than 70 MN/m² an "ordinary" CPT can not penetrate them. As a consequence a normal CPT can not give any information about the underlying layers.

The ROTAP is a cost-effective tool to overcome this major limitation of cone penetration tests. This "ROtating APParatus" transforms the hydraulic force of the CPT-rig into a rotating movement. This rotation is used to drill through the concretion. It can also be used to drill holes through existing foundations or to reduce lateral friction on CPT-rods.

1. INTRODUCTION
As constructions become bigger and heavier and people demand more security on the stability, geotechnical engineers need more information about the characteristics of increasingly deeper lying layers. Cone penetration tests are accepted worldwide to obtain some of the most important geotechnical parameters in a cost-effective manner. However one of the most important restrictions on the use of cone penetration tests is the limited depth that can be reached. The ROTAP system is an accurate tool to overcome this restriction.

This paper will explain what a "ROTAP" is and how it works. Furthermore two case-studies are given. In both cases ROTAP-interventions were a useful and cost-effective tool to reach the demanded depth.

2. DESCRIPTION
The "ROtating APParatus" (Fig. 1) is a device that can be mounted on the penetrometer

Figure 1. The ROTAP
and transforms the hydraulic force into a rotating movement. The system generates a torque of 65 kgm at a speed of 225 r.p.m. Water is used to cool the drilling bit and to transport the cuttings to the top of the hole. To avoid damage to the drilling bit the pressure in the penetrometry cylinders is limited to a maximum value of 10 kN. The bit pressure and the drilling speed depend on the type of soil, the diameter of the tubing and the type of drilling bit.

Drilling can be destructive as well as non-destructive. In hard layers cores ø21.5 or ø45 mm can be taken using an open bit (ø38 or ø60 mm). In soft layers a fishtail bit or tricone can be used to drill destructively.

3. WORKING PROCEDURE
The working procedure is the following:
- Execute a cone penetration test until the maximum capacity (cone resistance or lateral friction) of the CPT-rig is achieved.
- Remove the measuring head and mount the ROTAP on the CPT-rig and drill a hole through the concretion.
- Remove the ROTAP and continue the CPT until the maximum capacity of the CPT-rig is achieved again.

4. CASE HISTORIES
Recently two jobs have been executed in the centre of Brussels. Each site had its own specific problems and ROTAP interventions were the most effective solution in both cases.

4.1. AN EMBASSY BUILDING
In the heart of Brussels, near the Berlaymont building, a new embassy for a European country has to be constructed. Because the building has several subsoil floors, the foundations will be put on approximately eleven meters below street level.

Previous "normal" cone penetration tests on this particular site indicated the existence of a dense sand layer of tertiary age starting at variable depths below street level. These CPT's however could not penetrate any deeper than between seven and fourteen meters below street level because of the presence of sandstone concretions. Those concretions have a cone resistance of more than 70 MN/m².

There were two main reasons why further information about the deeper lying sand was indispensable:
- Previous experience with this tertiary sand learned that decalised zones occur locally. These decalised zones have low cone resistances and can be very compressible. The decalisation process is probably caused by groundwater fluctuations throughout the centuries.
- During the Middle Ages the sandstone concretions were used for the foundations of new buildings. Therefore they were exploited in subsoil mine shafts. Afterwards some of these mine galleries collapsed with disastrous consequences.

In order to check for the presence of both of these possible problems five cone penetration tests to a depth of more than twenty meters below street level were demanded. Only with the use of ROTAP interventions to drill through the sandstone concretions this depth could be guaranteed.

Figure 2 shows the result of a mechanical CPT where cone resistance (q_c) and total lateral friction (Q_a) are measured. Three ROTAP interventions were necessary to achieve the required depth of more than twenty meters. At the depths where the interventions are done no measurements of cone resistance or lateral friction can be made. On the diagrams one can find zero values at those depths.

None of the five executed cone penetration tests showed any indication for the presence of mine galleries or decalised zones. By
consequence the foundations could be constructed as planned.

4.2. THE MARTINI TOWER

As a part of the geotechnical investigation for the renovation of the International Center Rogier - the Martini Tower - in Brussels, six electrical cone penetration tests had to be executed to a depth of forty-five meters in order to reach the top of the tertiary "Landenian" sand.

Based on the regional geotechnical information the lithology was expected to be as follows:
- Disturbed zone (±4 meters).
- Alluvial clay from quaternary age with sandy and peaty zones.
- Alluvial sand and gravel from quaternary age with flint pebbles and locally some clay layers.
- Tertiary Ypresian sand-clay complex, alternately consisting of sand, clayey sand to sandy clay and clay.
- Tertiary Ypresian clay. This clay is known as relatively homogeneous and overconsolidated.
- Tertiary Landenian sand. The top of this layer was expected to be at a depth of approximately forty-five meters.

The major problem to achieve the demanded depth was the high lateral friction that would occur while penetrating the Ypresian clay. This clay layer was expected to have a thickness of more than twenty meters. The most economical way to reduce the lateral friction in this layer turned out to be the use of several friction reducers. Previous tests learned that the most effective distance between two friction reducers was four meters.

However, there was one practical problem complicating the use of several friction reducers on this particular site: the risk of breaking the CPT-rods while pushing the friction reducers through the alluvial gravel was too high. The solution for this problem was drilling a hole through the gravel with the ROTAP.

The working procedure was as follows:
- A hole was drilled manually through the disturbed zone to avoid damaging utility pipes and cables.
- In this hole a cone penetration test was executed as deeply as possible without friction reducer. With this procedure depths of between twelve and sixteen meters could be reached.
- The CPT-rods were withdrawn and large casing ø80 mm was drilled into the ground by use of the ROTAP up to one meter above the level achieved with the CPT.
- The hole inside the large casing ø80 mm was cleaned out by drilling destructively inside of it using "normal" casing ø60 mm.
- Next the ROTAP and the casing ø60 mm were removed. The electrical cone penetration test was continued inside the large casing ø80 mm. But this time every four meters a friction reducer was added to the CPT-rods.
- Once the last friction reducer had passed the alluvial gravel casing ø60 mm was installed and pushed as deep as possible.
- Further the cone penetration test could be continued through the tertiary Ypresian clay without any more problems.

It was only at a depth of fifty-two meters that the top of the Landenian sand was achieved. Afterwards this was confirmed by one drilling to a depth of sixty-two meters and an extra mechanical CPT that was executed inside a hole that was drilled to a depth of fifty-five meters.

5. CONCLUSIONS

The ROTAP is a useful and cost-effective tool for the execution of cone penetration tests in difficult soil conditions. It can be used to drill holes through hard layers or existing foundations to reduce lateral friction.
Figure 2. Mechanical cone penetration test with three ROTAP interventions
Figure. 3. Electrical cone penetration test. The ROTAP is used to install double casing in the alluvial gravel.

Proceedings CPT '95
The results of elaboration of hydrocompensated piezocone for investigations of the sea soils.

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SYNOPSIS: The results of design and elaboration of hydrocompensated piezocone for the use in sea are briefly described. In the article a special attention is paid to discussion of piezocone design, intended for direct measurement of the excess pore pressure. Experience of the use hydrocompensated piezocone for soil studies in the sea conditions are described. The system of collection of measurement data are considered from the viewpoint of error minimization. The problems of improving of the program ensuring for the interactive system for data control are discussed.

INTRODUCTION
At present time the cone penetration testing is extensively used in geotechnics - on the onshore as well as in aquatories.
CPT is one of the preferable methods according to criteria of economy and information content. Since 1990 the authors commenced to work out modernization of CPT - equipment in installations of company "Koken Boring" MDS 200/10 and "Koken Boring" MDS 100/100, which are exploited on the board of the geotechnical vessel "Tras".
This vessel is a property of the geotechnical company FEMGE (Uzno-Sakhalinsk, Russia. The installation MDS 200/10 was equipped with computerized apparatus with cable communication line "Zond-2", where the piezocones with compensation of hydrostatic pressure were used, but the installation MDS 100/100 was equipped with the Memory cone "Zond-3", which allows to read the information, which is acquired according the CPT results, into computer.
During the next stage the versions of "Zond-2" and "Zond-3" were modernized into devices "Zond-M" and "Zond-3M", respectively.
The program ensuring of computer was perfected. Further the results of the performed works are listed, and the principles, which are laid on the ground of realization of this apparatus, are substantiated.

GENERAL
More and more often marine geotechnical studies are carried out in the deep-water offshore regions.
If for performing the CPT-studies was used the apparatus with the measuring sensors, being located in the inner hermetically cavity of piezocone, then for determining of effective values of cone resistance Qc and side friction Fs for taking into account the influence of hydrostatic pressure, the known ratios are used (Konrad, 1987). The value of excess pore pressure dU during cone in-
trusion is determined here, using the equation:
\[ \text{d}U = \text{U} \cdot \text{h}, \]
where \( \text{U} \) - the measured value of the pore pressure, \( \text{g} \) - specific weight of the sea water, \( \text{h} \) - water depth. Before the beginning of probing the hydrostatic pressure component can be eliminated by means of balancing of the measuring apparatus, however, in certain cases the range of electrical balancing is limited.

When the probing depth is sufficient, then the change of hydrostatic pressure could be commensurable with the values of the parameter changes. In this way, for the piezocones with hermetically inner cavity, the sensitivity of sensors decreases relative to the parameter, which is being measured. This has an especially powerful effect when determining the excess pore pressure, because in certain cases the value of the excess pore pressure is insignificant relative the value of hydrostatic pressure.

At the same time the value of the excess pore pressure \( \text{d}U \) is an important feature and a characteristic of the soil, and on the basis of this value one can determine the soil filtration characteristics, as well as to determine the soil type, using the ratio \( \text{d}U/\text{Qc} \) (Parez, 1988), and to apply in various correlation relations (Lunne, 1990).

CONSTRUCTION
The authors of the present article have proposed and realized the design of piezocone with compensation of hydrostatic pressure and used it by carrying out actual geotechnical works.

The design of piezocone with compensation of hydrostatic pressure is shown in the fig. 1. The proposed piezocone has following distinctive features: - the presence of demountable plug with diameter \( D_p = 1.4 \text{ Dc} \) (\( \text{Dc} = 35.7 \text{ mm} \) - the cone diameter), above the friction jacket; - the presence of the compensatory filter and the movable compensatory element.

The compensatory filter and movable compensatory element are intended for transmission of hydrostatic pressure to dielectric liquid inside the piezocone.

The demountable plug in intended for soil condensation and providing of reduction of soil friction relative to penetration far, which follows the piezocone, as well as for admission of hydrostatic pressure to the compensatory filter.

In clays the dissipation time for the excess pore pressure can reach some tens of minutes, therefore, in the case of absence of demountable plug in the design of the piezocone, the pressure of the pore liquid the area of the compensatory filter would had been approximately equal to the pressure value at the filter of the measuring sensor, and in such case the differential sensor would
had not shown anything. The sensor of the excess pore pressure was located at the distance from the cone base, being 1/3 of its height, since our experimental results showed, that the excess pore pressure reaches its maximal value at the condensation core of soil, having outstripped the cone, at the distance from the basis of instilling cone, being approximately equal to 1/3 of its diameter.

The compensation principle of hydrostatic pressure was proven in the altitude chamber by supply of external pressure in the piezocone up to 50 MPa; and the readings of the sensors of cone resistance, side friction and excess pore pressure showed no alterations.

The graduation procedure for the measurement channel of the excess pore pressure was carried out using original graduating device. Using the piezocone with compensation of hydrostatic pressure permits us to measure directly the value of excess pore pressure dU and the efficient value of side friction F_s and in order to obtain the efficient value of cone tip resistance Q_c, it is necessary to introduce an insignificant correction to the value of the excess pore pressure dU.

\[ Q_c = Q_t \cdot \left( 1 - \frac{d}{D} \right) dU, \]

where \( Q_t \) - actual total tip resistance, \( D \) - cone diameter, \( d \) - diameter of load cell support.

By carrying out the geotechnical works, piezocones were applied for geotechnical investigations in a series of the oil and gas fields of western part of the offshore of Okhotsk sea, near by the north - eastern coast of isle Sakhalin (Astonkh, Bau'lin, Lun', Chayvo oil and gas fields).

**PRACTICAL APPLICATIONS**

The apparatus was applied in the work at sea depths from 19 to 140 meters, at the probing depths to 70 meters.

At these grounds there were investigated the sandy and the clayey soils, as well as specific soils, such as satu.

rated soils with two granulometric peaks.

For example, the 1-st peak 0.007 mm - 22%, 2-nd peak 0.09 mm - 36%, or 1-st peak 0.005 mm - 31%, 2-nd peak 0.04 mm - 44%.

According to the results of the classification and triaxial tests, these soils had sufficiently high plasticity index Ip = 9 and 11 and low permeability coefficient Kp = 1.07 x 10^-7 cm/sec. Closeness was equal 1.5 kPa, but the internal friction angle = 36-42 degrees.

Following the method, used by Bishop (1967) for calculation of brittleness index

\[ \text{as } lb = \frac{(f_f - tr)}{tr}, \]

where th - peak shear stress and tr - residual shear stress, we estimated in a similar manner the change in pore pressure, as dU assuming the excess peak pore pressure and dU - excess residual pore pressure.

Therefore, for the given soils this parameter, which characterizes the dilatancy properties of soil, lies in the range 0.06 - 1.10.

Let us emphasize, that for purely sandy grounds in the aforementioned region the value Ub lies in the range 1.20 - 1.60, but for clayey soils - 0.15 - 0.30. Typical curves of behavior of pore pressure show on Fig. 2.

As the areas of the isle Sakhalin offshore are located in seismic region, including the fact of immediate vicinity from epicentre of earthquake in May 28, 1995 in Neftegorsk with M=7.5 (which totally destroyed the city), FEMGE Co. carried out investigations in seeking for possibilities of rarefaction of sea soils in the foundation of gravity structures.

The investigations were carried out by the method of cyclic triaxial tests, similarly to the method (Andersen et al., 1988), as well as by means of analysis of the CPT data (Seed, 1979). The results, obtained by using apparatus "Zond", confirmed the peculiarity of physical and mechanical properties of those "two-peak" soils, and this succeeded to evaluation of possible danger.

Fig. 3 shows an example of representation of CPT results for this marine soils.
In the field conditions, with permanent time limits, the standardized registration forms are not convenient.

SOFTWARE
The use of small personal computers of the type "Note book", operating from low voltage power sources (12 V) of direct current, provided the rise of novel solutions of realization of the apparatus for SPT.

Experience of development and of the use of computerized apparatus "Zond-2" and "Zond-3", being mentioned in introductions of this article, permitted to carry out evaluation of the results of these works, as well as to formulate the main requirements, which further have been realized by the authors in the apparatus "Zond - M" and "Zond - 3M".

Accomplishment of piezocones' graduation in the laboratory conditions before the preparation for field activities has been provided.

The results of graduation of the corresponding piezocones are stored in the PC memory, which permits, when necessary, to perform operative change of piezocones in field conditions.

On the basis of graduation according to the PC program, the parameters of the linear regression are calculated, using the Gauss method. The graduation results are following: the graduating dependences, coefficients of linear regression are correlation coefficient, which are portrayed on PC display and printer. The zero readings and estimation of correction are carried out automatically according to the PC program.

$$d = A - A_0$$

where A - the value of the free term of linear regression in graduation, Ao - the value of the zero reading for a corresponding measurement channel before starting of probing process.

As an actual problem of SPT one should consider the determination - with high accuracy level of soil characteristics in broad range of their variability. Authors have made solution and measures, which provide
measurement of soil parameters in two measurement ranges for each parameter.

Here the PC program provides automatically choice of the measurement range. The range of representation of information for each measurement channel in the probing procedure can be changed, depending on the operator's desire.

For the purposes of providing for the relative error, being brought into relative to the measurement range, to be less than 1%, the piezocone graduation is carried out for each of the chosen limits of the measurement. In the process of apparatus realization the following ranges of parameter changes were admitted:

Q - the actual total tip resistance - from 0.008 MPa up to 8 MPa for the first measurement range and from 0.04 up to 40 MPa for the second;

Fs - the efficient side friction - from 0.0002 up to 0.2 MPa for the first measurement range and from 0.001 up to 1 MPa for the second;

dU - the surplus pore pressure - from minus 0.1 to 0.5 MPa for the first measurement range and from minus 0.5 to 2.5 MPa for the second range.

The results of graduation and static probing can be portrayed to printer in the forms of figures and tables, and the operator by means of the PC taste - panel can choose the gauge of representation of graphical information.

For the purposes of information input into PC the port typ RS232C seems to be as the most rational.

The consecutive (or "in series") communication channel has applications as in the cable version of the apparatus "Zond-M" as well as in the version of the autonomous probe "Zond-3M" in the process of reading of information from the operative memory device (OMD) of the microprocessor system (MP - system).

Realization of the autonomous probe "Zond-3M" was performed on the basis of all aboveconsidered conceptual solutions.
CONCLUSIONS
The use of the piezocone with compensation of hydrostatic pressure permits a direct measurement one of the most important soil parameters - the excess pore pressure, elevating sensitivity of the sensors as well as accuracy of detection of Qc and Fs, renders the possibility to elaborate a computerized system with automatic choice of measurement ranges and accomplishment of zero readings before the start of the probing process, which increases the productivity of these activities.

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The development of the Laval piezocone.

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SYNOPSIS. A piezocone has been developed at Université Laval to characterize the soft sensitive clays and measure their mechanical properties. For that purpose, it is equipped with a very sensitive load cell to measure the tip resistance, and a particular effort has been made to minimize or eliminate the errors often encountered in this tool, such as the shape factor, the lack of accuracy of the load cells, the cross-talk between the cells and the temperature drift. To allow a more realistic interpretation of the measurement of the friction, the friction sleeve has been moved to 5 diameters behind the cone. A calibration cell is also presented which allows a more reliable interpretation of the measurements.

1. INTRODUCTION

Ever since Janbu and Senneset (1974) suggested the idea of pushing a piezometer in the soil to help in the interpretation of the static cone penetration test, considerable progress has been accomplished in the development of the piezocone which combines the tip resistance (q_t), the friction (f_s) on a sleeve behind the cone, and the pore pressure (u) generated by the penetration of the cone. When linked to an efficient data acquisition system, this equipment becomes a powerful tool for the profiling and characterization of soil deposits. However, in soft soils where the tip resistance is very low and the pore pressure high, special precautions must be kept in mind at the design stage, and also in the testing procedure, in order to minimize the errors inherent to this equipment. The shape factor at the junction between the cone base and the shaft, the sensitivity of the load cells for the tip resistance and the friction sleeve, and the temperature effects, are amongst the factors which are discussed as they may result in major errors in piezocones having a low capacity. This equipment needs to be carefully calibrated, and for that purpose, a special cell is presented. The recommended standard position of the friction sleeve immediately behind the cone tip where u varies markedly presents some difficulty of interpretation which can be easily circumvented by moving the sleeve up the shaft, as the Authors suggest, where the pore pressure has equalized. Finally, some aspects of the testing procedure, such as the rate of penetration need to be re-evaluated.

2. CHARACTERISTICS OF THE PROBE

2.1 General layout

The general layout of the Laval piezocone is given on figure 1. The pore pressures are measured in two places, i.e. immediately behind the cone tip, and just before the friction sleeve which has been moved to 5 diameters behind the tip. With this arrangement, it is easy to position the load cells for q_t and for the friction component f_s, so that they are quite independent from each other, thus avoiding the inconvenience of a subtractive set up.

2.2 Sensitivity of the load cells

The sensitivity of commercially available piezocones is generally not sufficient for soft sensitive soils, for example, a specified sensitivity of 0.05% of a nominal tip load capacity of 100MPa, may be considered acceptable for a piezocone intended for use in sand or stiff clays, but it would result in an unacceptable error of ±4 kPa, for the strength of a soft clay layer. It was then decided to develop a piezocone with load cells better adapted to soft soils.
and 0.8 MPa for the pore pressure measurements respectively at the tip and at the friction sleeve (Fig. 1), these transducers have generally a sensitivity of about ± 0.5 kPa.

2.3 Pore pressure measurements

The excess pore pressures u generated during the penetration of the piezocene in fine soils are known to undergo quick variations around the tip of the probe as reported by many authors (see for example Robertson et al. 1986). The choice of the position of the porous element for measuring u is then somewhat problematic. It is generally agreed that in order to best define the stratigraphy of the soil, the porous element should be at the tip. On the other hand, there is a need to measure u close to the joint between the cone unit and the body of the probe so that it can be used to determine the shape factor (1-a) of the cone (Fig. 2).

In the Laval probe, the pore pressure is measured in two places, i.e. immediately behind the cone, and just before the friction sleeve to allow the interpretation of the friction measurements (Fig. 1). Indeed, the experience has shown that the measurement of pore pressure behind the cone and close to the joint gives good indications on the stratigraphy of the soil, and at the same time, the u data obtained can be used to calculate qe, which is the tip resistance value qe corrected with the shape factor (1-a). Placing the porous element behind the cone rather than on the tip has the further advantage of minimizing the problem of compressibility of the porous element and of its abrasion. The pore pressure element located near the friction sleeve will be discussed below. The porous elements are made of aerolith 20 which is a material with a good resistance to plugging.

2.4 Shape factor

The shape factor (1-a) which depends essentially on the geometry of the joint between the cone unit and the body of the apparatus (Fig. 2) is a very important design detail which has a significant implication on the accuracy of the measures. Balagh et al. (1981) have shown that when a piezocene probe is subjected to a hydraulic pressure u, the recorded pressure on the tip is not equal to u, but rather to qe = a u, where “a” is the net area ratio between A1 and A (Figs. 1 & 2). The tip resistance has thus to be corrected with the shape factor (1-a) and becomes:

\[ q_e = q_a + (1-a)u \]
Some values of shape factors reported in the literature vary from 0.20 to 0.58 (Lunne et al. 1986). In the case of soft sensitive soils where the measured pore pressure may be as high or even higher than the recorded tip resistance $q_u$, the magnitude of the correction may be very important relative to the value of $q_u$. Hence it is quite important to keep the "a" value as close to unity as possible, i.e. to minimize the difference between $A_u$ and $A_T$ (Fig. 2).

For the Laval piezocone, a special attention has been devoted to this design detail. As recommended in the "reference test penetrometer" in ISSMFE (1988), the watertightness has been insured by placing a O-ring in the space between the cone unit and the cylindrical sheath above the joint. A quad ring is placed in the gap of the joint to prevent dirt from entering that space. In this way, the cone is free to move relative to the cylindrical sheath (Fig. 1). All the mechanical components were machined with a small margin, but also with enough clearance to prevent interference with the measurements of $q_u$. By means of the calibration cell described below, the shape factor $(1-a)$ obtained with that design varied from 0.06 to 0.12. Trying to obtain values lower than 0.06 resulted in overtightness of the mechanical parts with possible transfer of load to the sheath of the probe.

The same observations are relevant to the end joints of the friction sleeve. In this case, the net area ratio "b" is given by the ratio of $A_{10}$ to $A_T$ (Fig. 2) and can be designed to be the same at both ends of the sleeve. In that case, the correction to be applied to $q_u$ will be a function of the difference in the pore pressures at both ends of the sleeve. In the case of the Laval probe where, as discussed below, the friction sleeve is placed at 5 diameters behind the tip, $q_u$ has about the same value at both ends of the sleeve so that the correction of $q_u$ for the end effect becomes negligible.

2.5 Friction sleeve

The ISSMFE Committee on Penetration Testing (ISSMFE 1988) recommended that the friction sleeve in the penetrometer be placed at less than 15 mm behind the base of the cone tip. This location is definitely not the best for the friction sleeve. According to a model presented by Teh and Houlsby (1991), the stresses in that zone undergo important variations of magnitude, and such is the case particularly for the stress normal to the sleeve. The excess pore pressure is also subjected to some major variations as shown on Fig. 3 which presents the theoretical distribution of the pore pressures obtained with the strain path method by Baligh and Levedouch (1986). Experimental results obtained in a homogeneous deposit of sensitive clay in Louisville (Quebec) are in very good agreement with the theoretical curve and confirm the important variation of $q_u$ up to a distance of more than 5 diameters behind the cone tip. Konrad (1987) has also shown that in sensitive clays, the effective stresses
behind the cone and for a distance of about two diameters are very small and close to zero. In the light of all this evidence, it was decided to move the friction sleeve at 5 diameters behind the cone where the pore pressure becomes nearly constant along the length of the sleeve. As illustrated on Fig. 1, a second porous stone was placed just ahead of the sleeve to allow a more reliable analysis of the measured friction.

3. CAUSES OF ERRORS.

By comparing many commercial piezocones, different authors (Lumme et al. 1986; Bruzzi and Battaglio 1989) have identified a certain number of characteristics which may be the causes of errors and need to be checked.

- Reproducibility and hysteresis. Although few probes were appreciably affected by the lack of reproducibility or hysteresis, there is a need to make checks in this respect. The Laval piezocone has negligible hysteresis and a reproducibility within the range of the sensitivity.

- Cross-talk between cells. Surprisingly enough, it is not infrequent that the load applied on one cell influences a neighbouring cell. This is especially the case for a pore pressure reaction resulting from a load increase on the tip (Bruzzi and Battaglio 1989). It is usually caused by the lack of rigidity of the chamber enclosing the pore pressure transducer and the porous element. Hence, the absence of cross-talk between different measuring cells needs to be checked, no cross-talk has been detected in the Laval piezocone.

- Temperature drift. When the probe is introduced into the soil, its temperature varies and the zero may shift by many degrees depending on the magnitude of the temperature and on the characteristics of the measuring cells. This is by far the most important cause of error, and one that is very difficult to minimize and impossible to eliminate completely. In their study of different piezocones, Lumme et al. (1986) have measured zero drifts varying from 18 kPa to 760 kPa for a 25°C drop in the temperature. This is due not only to a problem of temperature compensation of the electronic board and gauges, but also to the thermal contraction of the different metallic components. In spite of a significant effort invested in trying to correct this problem in the Laval piezocone, a variation of 25 kPa on the value of $p_0$ is observed when the probe is immersed in melting ice, which means a drop of about 22°C. The soil temperature in the area of the Champlain clay deposits varies between 7°C and 9°C, so, in the testing procedure, the probe is first placed in a hole filled with water at the surface of the soil, and when the readings have stabilized after 10 to 15 minutes the test is started. There is no appreciable time drift in any of the cells.

4. CALIBRATION CELL

Considering all the sources of error which may affect the performance of the piezocone, one can readily come to the conclusion that it is of paramount importance to proceed to a meticulous calibration in the laboratory before going on the field. For that purpose, the calibration cell illustrated on Fig. 4 has been designed; it allows both static and hydraulic calibrations. It consists of an aluminium cylinder with a joint below mid-height and with a piston inside which is fixed to the friction sleeve through grip holes (Fig. 1) and maintains the probe in the axis of the load. In position “a” (Fig. 4), the tip load cell is calibrated under static load. In position “b”, the piston rests on a support and the tip does not touch the bottom of the cell so that the load cell of the friction sleeve can be calibrated independently of other loads. When proceeding...
to these calibrations, readings are recorded on all four measuring units to ascertain that there is no cross-talk between them. For the hydraulic calibration, the cell is filled with water (Fig. 4c) and the two pore pressure transducers can be calibrated by increasing the hydraulic pressure; at the same time, the area ratio $a$ is determined as there is no static load on the tip, and the equality of the area ratio $b$ at both ends of the sleeve can be ascertained. Finally, to simulate the test conditions when loading is applied on the friction sleeve and on the tip simultaneously, a rubber membrane has been fixed between the piston and the central joint so that load on the friction sleeve is a function of the hydraulic pressure in the cell, while a static load can be added to the piston. With this calibration cell, all the characteristics of the cone can be determined in the laboratory. On the field, the calibration was checked by means of a hydraulic calibration cell every 2 or 3 soundings to ensure that the piezocene is working properly and that the constants have not changed.

5. PROCEDURE

The piezocene has the advantage over the penetrometer that it can measure the pore pressure in order to derive full benefits from it, certain precautions must be taken during the testing procedure.

5.1 Saturation. It is well known that in order to obtain a good reaction of the pore pressure with a piezocene, and at the same time a good definition of the stratigraphy of the soil, it is essential that the porous stones, the small conduits to the pore pressure transducers and the cavity of the transducers be well saturated. This operation is more easily made in the laboratory. The saturation of the porous stones is carried out by first de-airing the stones under vacuum in a dry state, and then immersing them in glycerine while keeping the system under vacuum. The best results were obtained with this technique (Bruzzi and Battaglio 1989). The small conduits and the cavity near the transducer were saturated with a syringe. The stones were transported on the site immersed in glycerine. On the field, all the operations for installing or changing the stones are carried out under glycerine. With these precautions, the probe reacts quickly to changes in pore pressure, which indicates that it is well de-aired.

5.2 Rate of penetration

The recommended standard for the rate of penetration of the penetrometer is 20 mm/s, although it is not meant to apply officially to the piezocene, it is usually adopted as such. If the purpose of the sounding is to determine the stratigraphy of a soil, such a high rate of penetration becomes questionable because, even though the probe is well saturated and the $q_s$ load cell is very sensitive, these instruments may not react quickly enough to differentiate strata when travelling through stratified soils. An example of the influence of the rate of
penetration on the differentiation of the strata is given on Fig. 5. These data were obtained in a deposit of clay containing thin strata of sand 0.5 to 4.5 cm thick in Sainte-Anne de la Pérade, Québec, (Diène 1989). The results show clearly that when the standard rate of 120 cm/s (2mm/s) is used, the variation of qe and u would indicate that the deposit is fairly homogeneous, however, when the rate of penetration is decreased by a ten-fold, the presence of strata is revealed. This aspect of the procedure would need more consideration before adopting the standard rate. Indeed, if the main purpose of the sounding is to define the stratigraphy in fine soils, it may be preferable to use a rate of penetration slower than the standard recommended.

6. CONCLUSION
For soft sensitive soils, the Laval piezocone has proven to be a very reliable and powerful tool which has been used on many sites for many investigations and research projects as discussed in a companion paper by Leroueil et al. (1995). We can foresee that new applications will emerge as the geotechnical engineers will become more and more familiar with the use of this tool.

ACKNOWLEDGEMENTS
The authors wish to acknowledge the contributions of Messrs. S. Paré, J.-C. Prince, J.-Y. Julien and M. Bégin in developing the probe and its data acquisition system. Financial support was obtained from the Natural Science and Engineering Research Council of Canada, du programme des Fonds pour la Formation de Chercheurs et l'Aide à la Recherche (FCAR) du Québec, le Ministère de l'Équipement, du Logement, du Transport, et de la Mer de France.

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Numerical analysis of boundary effects of calibration chamber on electrical resistivity measurements

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SYNOPSIS: Measuring the electrical resistivity of a soil by a cone penetrometer is a widely accepted technique for geotechnical and geo-environmental investigations. The measured electrical resistance is affected by the soil resistivity, cone diameter and geometry of the electrode array. Therefore, it is always necessary to determine the shape factor of the cone before the electrical resistivity of the soil can be determined from the measured resistance. The shape factor is usually determined by laboratory measurements made in a calibration chamber. The chamber has to be large enough relative to the cone diameter to eliminate any boundary effects. Thus, it is very cumbersome to prepare such a large sample and to dispose of it after test if it is hazardous. This paper will present a new numerical simulation technique used to quantify the boundary effects on electrical resistivity measurements made by a cone penetrometer. Relationships between the shape factor as a function of chamber diameter, cone diameter, and electrode spacing are presented in a design chart. The shape factor determined in a small chamber can thus be adjusted properly for use in field measurements.

1. INTRODUCTION
Since its introduction in early 1930s, the cone penetration test (CPT) has been becoming a standard tool for geotechnical subsurface investigation (Robertson and Campanella 1984; Briaud and Miran 1992). The early designs could measure the penetration resistance at the tip of the cone and frictional resistance along the sleeve to provide information on the shear strength of the soil and to characterize subsurface stratification (Douglas and Olsen 1981). Recent advances in electronics and sensor technology have allowed installation of different sensors on the cone to measure different properties of the subsurface and to collect and interpret geotechnical and geo-environmental engineering data in real time. The measurements may include pore water pressure, pH, electrical resistivity, chemical concentration, temperature, redox potential, etc. (Hornell 1988; Yong and Hoppe 1989; Campanella and Weemers 1990; Cooper et al. 1991; Woeller et al. 1991; Bowders and Daniel 1994; Akhtar 1995; Bratton et al. 1995).

CPT offers these advantages in a geo-environmental subsurface investigation: (1) the technique can provide a rapid and inexpensive means to study the subsurface physical and chemical characteristics of a hazardous waste site; (2) continuous subsurface profile information can be obtained; (3) the data collected can be processed in real time on site so that field decisions can be made; (4) in-situ measurements are made and there is no time lag between the collection and chemical analyses of samples; (5) there will be no direct contact between workers and contaminated materials; (6) the results obtained are almost operator independent; (7) no contaminated cuttings have to be handled; and (8) impact to the surrounding environment is kept at a minimum.

Electrical resistivity measurements made by a cone penetrometer may be used to detect and delineate subsurface contamination in-situ if appropriate correlations can be developed (Yeung 1994). This paper presents the results of numeri-
2. USES OF ELECTRICAL RESISTIVITY

The electrical resistivity (inverse of conductivity) of a soil is a function of the soil matrix that is affected by soil mineralogy, particle size, shape and orientation, temperature, and the type and viscosity of the pore fluid if present (Okoeye et al. 1995). Several researchers have made use of the electrical properties of soils to determine their phyaicochemical properties. Anuranandan (1991) developed a relationship between the relative permittivity of a saturated soil and its porosity. Rhosad et al. (1989, 1990) applied different models to determine the salinity of a soil from its electrical conductivity. Asch and Morrison (1989), Takahashi and Kawase (1990), and Seedhar and Arora (1992) used resistivity measurements to determine the properties of multi-layer earth systems. Thus, electrical resistivity measurements can be used to detect and delineate subsurface contamination rapidly and reliably.

3. THE SHAPE FACTOR

When a cone penetrometer is used to measure the electrical resistivity of a soil, a two-electrode or four-electrode array can be employed. Electrical excitations and measurements can be made by using different combinations of the electrodes. Ratio of the measured voltage to the measured current gives the bulk electrical resistance of the soil that is a function of the resistivity of the soil, diameter of the cone, and geometry of the electrode array. A shape factor is conventionally used to relate the measured electrical resistance of a soil to its resistivity by (Campanella and Weemeees 1990)

\[ \rho = G \frac{\Delta V}{I} \]  
(1)

where \( \rho \) = electrical resistivity of the soil (\( \Omega \cdot m \)); \( G \) = shape factor (m); \( \Delta V \) = voltage applied across the electrodes (V); and \( I \) = current passing through the electrodes (A). For one-dimensional electrical current flow through a rectangular section with parallel electrodes at the ends, \( G \) is given by Ohm's law to be \( A/L \) where \( A = \) cross-sectional flow area, and \( L = \) distance between the electrodes. For three-dimensional current flow around a cone penetrometer, the problem becomes much more complex. \( G \) is thus determined experimentally in the laboratory. The electrode array is set up and immersed in an electrolytic solution of known resistivity. The electrical resistance of the solution is then measured. The resistivity of the solution is varied systematically and the corresponding resistance is measured by the cone. The slope of the graph of resistivity versus resistance gives the shape factor of the cone for the particular electrode geometry. Although the approach is simple and straightforward, it suffers from these drawbacks: (1) many electrolytic solutions of different resistivities have to be used for each calibration; (2) every cone and every electrode array of it has to be calibrated individually; (3) the dimensions of the cone and electrode array are not expressed explicitly in the shape factor; (4) the boundary effects of the calibration chamber are not quantified and the required size of the chamber has to be determined by trial and error; and (5) effects of many other parameters, such as locations of electrodes and thickness of the electrodes, on the shape factor cannot be studied economically. Thus, there is a need to better quantify the shape factor.

4. PARAMETRIC FORMULATION

By analogy to one-dimensional electrical current flow between two parallel electrodes, the length of the flow path for the current flow around a cone penetrometer should be a function of the distance between electrodes and the flow area should be a function of the cross-sectional area of the cone. Thus, a new dimensionless shape factor \( G' \) is defined to be

\[ \rho = G' \left( \frac{A}{d_s} \right) \left( \frac{\Delta V}{I} \right) \]  
(2)

where \( d_s \) = distance between electrodes, and \( A = \) cross-sectional area of the cone.

As the size of a non-conductive calibration chamber reduces the area for electrical current flow, the shape factor measured in a calibration chamber should also be a function of \( (d_c/d_s) \) where \( d_c \) is the diameter of the chamber and \( d_s \) is
the diameter of the cone. Moreover, \( G' \) is also a function of \( (d/d_0) \) as the ratio affects the flow pattern of electricity around a cone penetrometer. Numerical analyses are performed to evaluate these relationships.

5. NUMERICAL SIMULATION
A new technique has been formulated to simulate the three-dimensional electrical current flow around a cone penetrometer through a conducting medium. The medium is discretized into elemental volumes and each volume is represented by a node at its geometric center. These nodes are inter-connected electrically by resistors. The values of these resistors are computed from the resistivity of the medium and dimensions of the elemental volumes. The electrical behavior of the medium can thus be simulated effectively by an equivalent three-dimensional electrical circuit. The variables in the circuit can be solved by the program SPICE.

SPICE is the de facto world standard for analog electrical and electronic circuit simulation developed at the University of California at Berkeley. Circuits consisting of passive elements (resistors, capacitors, and inductors), independent and dependent voltage and current sources (DC and AC), and semi-conductors can be analyzed. Types of analyses that can be performed are time-dependent analysis, frequency-dependent analysis, source-value-dependent analysis, DC analysis, noise analysis, and distortion and spectral analysis (Tinnesa 1988).

Therefore, different scenarios can be studied economically by varying the input parameters of concern and performing numerical simulations. The various parameters that have been studied include medium resistivity, cone diameter, electrode spacing, and chamber size.

6. VALIDATION OF THE TECHNIQUE
The validity of the approach was evaluated by comparing simulated results to analytical solutions derived for surface geophysical resistivity techniques (Telford et al. 1990; Kaufman 1992). As shown in Fig. 1, two current electrodes C and D, 0.6 m apart, are installed on the surface of homogeneous isotropic ground of electrical resistivity \( 100 \, \Omega\cdot\text{m} \). An electrical current of 0.1 A enters the ground at electrode C and leaves at electrode D. The plot of potential variation of the surface along a straight line passing through C and D is shown in Fig. 1. It can be observed that the simulated results are in good agreement with the theoretical values. The infinite values at the singular points C and D are unrealistic in practice and cannot be modelled by finite mathematics. The discrepancies that occur at locations near the boundaries of simulation are due to boundary effects as the theoretical values are derived on the assumption of an infinite medium.

7. DESIGN CHART
Numerical simulations of the electrical current flow around a cone penetrometer were performed to determine the values of \( G' \) in Eq. (2) as a function of \( (d/d_0) \) and \( (d_0/d) \). As both the calibration chamber and the cone were cylindrical, the analyses took advantage of the axiymmetric nature of the problem. The chamber and the cone were assumed to be concentric on the horizontal plane. Therefore, only a slice of the chamber and the cone in plan was used in the analyses as current could only flow in radial directions on vertical planes. As a result, the three-dimensional problem was reduced to a two-dimensional problem on a vertical plane. The conducting medium was discretized into elemental volumes and each volume was represented by a node at its geometric center. These nodes were inter-connected by resistors. The resistances of these resistors were calculated on the basis of the resistivity of the medium, the distance between the nodes, and the equivalent current flow area perpendicular to the fictitious line joining the nodes.

The nodes were numbered. The node numbers and resistances between them were tabulated in an input file following the format required by SPICE. A DC voltage was applied across the electrodes and the electrical current flowing through the system was obtained from the program SPICE through simulation. As only a slice of the chamber and the cone was simulated, the total current flowing through the system was obtained by multiplying the simulation results by a factor of \( 2\pi \) divided by the angle subtended by the slice in radians. The ratio of the applied voltage to the current gave the bulk resistance of the medium measured by the cone penetrometer.
As the resistivity of the medium, diameter of the cone, and geometry of the electrodes were known in the simulation, the shape factor could be calculated by Eq. (2) from the simulated bulk resistance. The variation of \( G' \) as a function of \( d_{b}/d_{c} \) and \( d_{b}/d_{c} \) is depicted in Fig. 2.

It can be observed in Fig. 2 that the shape factor increases with chamber diameter when other parameters are kept constant. The results are intuitive from Eq. (2). When the electrical resistivity of the medium is unchanged, the bulk resistance measured by a cone penetrometer decreases with increase in chamber diameter as a larger area is available for the electrical current flow. As a result, the shape factor in Eq. (2) increases with increase in chamber diameter. Moreover, the shape factor reaches an asymptotic value as the ratio of \( d_{b}/d_{c} \) increases.

The proper dimensionless shape factor \( G' \) can be selected from the design chart on the basis of the chosen operating parameters. The electrical resistivity can then be determined from the measured electrical resistance by Eq. (2). Moreover, it is not necessary to use a large calibration chamber to eliminate adverse boundary effects on the determination of the shape factor as the design chart has taken the boundary effects into account properly. Thus, the quantities of contaminated soil to be prepared and handled can be kept at a minimum in geo-environmental engineering research projects.

8. CONCLUSIONS
A new numerical technique has been developed to simulate the electrical current flow around a cone penetrometer. The conducting medium is idealized as an electrical circuit. The resistances of the resistors in the circuit can be computed by the resistivity of the medium, diameter of the cone penetrometer, and geometry of the electrode array. The validity of the approach has been established by comparing simulated results to analytical solutions derived for surface geophysical-resistivity techniques. The technique has been used to investigate the boundary effects of a calibration chamber on electrical resistivity measurements by a cone penetrometer. A design chart relating the shape factor of the cone, diameter of the chamber, and spacing between electrodes installed on the cone has been developed.

The technique is being further developed to include other electrical elements such as capacitors to better model the electrical behavior of soil. Effects of other details of a cone penetrometer, such as locations of electrodes and dimensions of electrode on electrical resistivity measurements are also being studied.

9. ACKNOWLEDGEMENT
The research is being supported by the United States National Science Foundation under grant no. CMS-9211336. The program directors are Drs. Mehmet T. Tumay and Priscilla P. Nelson. The support is gratefully acknowledged.

10. REFERENCES
FIG. 1 Comparison between theoretical and simulated voltage distributions

FIG. 2 Variation of $G^*$ as a function of $d_i/d_o$ and $d_i/d_o$
No. 5, 557-567.


Session 2 - Interpretation of Test Results
CONE PENETRATION TESTS (CPT) AND RADAR SCANNING (GPR)

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SYNOPSIS: This paper looks at the use of static CPT, Seismic (shear wave) cone penetration tests (CPTS), and various ground probing radar (GPR) systems for the geotechnical characterization of sites in the London basin. From the CPTS (seismic cone tests) it is possible to estimate values for the soil stiffness at very small strain $G_s$ (shear modulus at very small strain, i.e. strains of the order $1 \times 10^{-4}$ %) if values of soil density are assumed. The $G_s$ values obtained from CPTS tests are point specific and it is hoped to make these estimates more holistic using radar and other geophysical tests.

INTRODUCTION

South Bank University has been involved in the Limehouse Link Highway Project by way of a Transport Research Laboratory (TRL) funded contract to assess the performance of cantilever in-situ cast diaphragm walls in stiff clay. The diaphragm walls form the walls of the cut and cover tunnel. The contract site investigation made available conventional cable percussion drilled boreholes, electric cone penetration test (CPT) and self boring pressure meter (SBM) data. Laboratory tests on samples from the contract boreholes generated soil index and other soil properties. From the above and published geological information the main lithological boundaries at the site were fairly well known.

On a small site adjacent to the tunnel West portal an additional borehole was made as part of the TRL work. On this same site the author has conducted additional CPT, Seismic (shear wave) cone penetration tests (CPTS), and various ground penetrating radar surveys using the Pulse EKKO IV 1006, UTSI Electronics, Georadar 1 (OYO) and SHH systems with various antenna. On samples from the borehole it is intended to carry out laboratory triaxial compression tests with local strain measurements to determine stress strain parameters. From the CPTS (seismic cone tests) it is possible to estimate values for the soil stiffness at very small strain $G_s$ (shear modulus at very small strain, i.e. strains of the order $1 \times 10^{-4}$ %) if values of soil density are assumed. The $G_s$ values obtained from CPTS tests are point specific and it is hoped to make these estimates more site specific using the radar and other geophysical tests.

It is then hoped to draw some comparisons between the various in-situ and geophysical test methods and the laboratory measurements. This paper reports on the work to date.

GROUND CONDITIONS

The site is in the old docklands area of the London East End on the west portal of the Limehouse link tunnel that gives ready access from the City of London to the Isle of Dogs and Canary Wharf. The ground conditions at the site, fill and a mantle of Pleistocene alluvium and Terrace Gravel overlying Eocene London clay and Woolwich and Reading beds typify much of the inner London area. The results of the South Bank borehole are shown in figure 1 with a static CPT over plotted. A water content profile in the clay soils from the same borehole is shown in figure 2 which is again typical of the area.
LIMEHOUSE STATIC CPT TESTS

On the Limehouse test site a total of three static tests have been executed using 15 cm² non reference (The 1977 ISRM SE standard for penetration testing incorporated into ISRM 1981 refers to a 10 cm² cone tip) friction sleeve cones. Different operators were used but the in the hole equipment for each test was essentially the same.

The cones used were so called subtraction cones and have two sets of electrical resistance strain gauges on the one load column. The lower set generate point resistance and the upper set generate point resistance (q), plus sleeve resistance (f). Thus to obtain the friction sleeve resistance the point transducer output must be deducted sleeve transducer output. The so-called compression cone has two load columns giving entirely separate output signals for point and friction sleeve resistances (q, f). The advantage of the former type of system being greater robustness because the very low friction loads require a very low capacity load column in the separate system which can easily be damaged by accidental overload. This together with the use of the larger geometry cone has resulted in the potential to satisfactorily penetrate the dense gravels that generally mantle the London basin geology. From the writer's experience these subtraction cones give excellent linearity of output over the full range.

For both systems used the transducer output being measured and digitized at the surface is in all cases a signal voltage one system appears to convert these to engineering units, i.e. MPa whilst others have raw transducer output for later conversion and processing.

Notwithstanding the above comments one of the tests in fact have penetration problems in the terrace gravels which it is felt was due operator inexperience and thus only two of the test were used. Both point resistance and friction ratio where smoothed using a 9 point rolling average and then the arithmetic mean of the two test was
made and is plotted in figure 3

![Figure 3 Static CPT average of tests 1 & 2](image)

**SEISMIC CONE PENETRATION TESTING (CPTS)**

The general arrangement for the CPTS test is as shown in figure 4. The seismic module is mounted just above the 15 tonnes 15 cm³ cone element. In the single element system a single triaxial geophone array is located just above the friction sleeve cone. In the dual element equipment another array is located one metre above. In the single element system total travel time for the shear wave must be measured from the surface at each shot and thus triggering of the system is very important. There is also less certainty about depth measurements with the single element arrangement. Results for both systems which show reasonable agreement can be seen in figure 5. Reasonable agreement is also shown with the static CPT tests

![Figure 4 Set-up for seismic CPTS test](image)

![Figure 5 Seismic CPTS results](image)
GROUND PENETRATING RADAR

The basic principle in ground penetrating radar sounding is fairly simple. The radar antenna transmits a short electromagnetic pulse of radio frequency (RF energy) into the medium. When the pulse reaches an electric interface or target in the medium, some of the energy will be reflected back (backscattered) while the rest will proceed forwards. The radar system will then measure the time elapsed between wave transmission and reflection (the two way travel time). This is repeated at short intervals while the antenna is in motion, and the output signals, i.e. scans, are drawn consecutively by means of a video intensity recorder which thus produces a continuous profile of the electric interfaces in the medium. The basic system is shown in Figure 6.

Figure 6 Ground Probing Radar (GPR) Systems - Basic principles

The technique utilizes downward-looking radar transmitting electromagnetic pulses about 10 ns long at repetition rates of 50 to 400 kHz. The impulses are partially reflected by subsurface interfaces. The reflected radar echoes are picked up by the receiver at scan rates of between 8 and 64 scans per second. Each scan or so-called wiggle diagram consist of between 256 and 1024 samples per scan and when plotted as a continuous two-way travel time video image produces a pseudo-geological section. The frequency content of the transmitted electromagnetic energy (EM) can vary between 100 and 1000 MHz depending on depth and resolution required. For greater penetration the lower frequency antenna of a nominal 100 MHz are needed with a frequency band width between 50 and 250 MHz but with corresponding loss in resolution.

The vertical depth scale can be derived from the measured two-way travel times of reflected events and by assuming inter-layer velocity values for the radar waves through each layer, or alternatively, by correlating the time-section with borehole logs or other profiling devices such as cone penetration tests (CPT).

The equipment so far discussed can be used in two basic modes:

1) In the basic reflection profiling mode were the (EM) source (TX) and receiver (RX) are collocated.

2) In the wide angle reflection and refraction mode (WARRS) enabling the inter-layer velocities to be measured. To this end, some radar equipment enables the source (TX) and receiver (RX) to be used at different spacings for the determination of these layer velocities.

It can be shown relatively easily from Maxwell's electromagnetic theory that

\[ \nu = \frac{c}{\sqrt{\varepsilon_r}} \]

\( c \) = velocity (0.3 m/ns) when EM waves are travelling in free space.

\( \nu \) = velocity (m/ns) when EM waves are travelling in media other than free space.

\( \varepsilon_r \) = relative electric permittivity (dimensionless) or dielectric constant. A dielectric is an insulating medium through which electricity can pass but not by conduction typical maximum
and minimum values for which are.

\[ \varepsilon_{\text{air}} = 1.0 \]
\[ \varepsilon_{\text{water}} = 81 \]

Certain aspects concerning the efficacy of radar systems need to be borne in mind.

Interface or target types

In general, any dielectric discontinuity is detected. In particular, targets can be classified according to their geometry: planar interfaces, long, thin objects, localised spherical or cuboidal objects. The radar system can be designed to detect a given target type preferentially and may potentially produce an image of the target in three dimensions.

Lithology

The signal attenuation at the desired operating frequency is the main factor to be considered when assessing the usefulness of radar probing in a given material. **Dry materials will have a lower signal attenuation than wet ones.** As a rule, material, which has a high value of low-frequency conductivity will have a large signal attenuation. Thus gravel, sand, dry rock and fresh water are relatively easy to probe using radar methods, while salt water, clay soils and conductive ores or minerals are less so. But a reduction in the transmitted frequency means that even these materials can be adequately investigated, though at the expense of a reduced resolution between targets.

Radar signal enhancement and processing

The main aim of signal processing and enhancement is to generate a signal that has constant amplitude with depth, such that electrical interfaces may easily be defined. To achieve the above objective use is made of the application of signal amplification or gain variable with depth and high and low pass filters. Thus signal or image processing done at a later stage is applied to an already modified signal than received by the \( R_n \) antenna.

Estimating two way travel times

Referring to figure 1 to estimate the anticipated inter layer travel times we need to guess at a figure for the dielectric constant \( \varepsilon_r \). From table 1 it can be seen that for all soil types \( \varepsilon_r \) varies considerably, however, it can also be seen from the same table that \( \varepsilon_r \) is mainly a function of soil water content. For clay soils this can be established from simple tests on disturbed samples. For the granular soils we must deduce a theoretical void ratio and hence water content from the relative densities obtained from the CPT profiles. This can be seen in figure 1. From this data table 2 was produced.

<table>
<thead>
<tr>
<th>Material</th>
<th>Dielectric Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Air</td>
<td>1</td>
</tr>
<tr>
<td>Metal</td>
<td>1</td>
</tr>
<tr>
<td>Ice</td>
<td>3</td>
</tr>
<tr>
<td>Frozen soil</td>
<td>3</td>
</tr>
<tr>
<td>Concrete</td>
<td>4</td>
</tr>
<tr>
<td>Rock</td>
<td>4</td>
</tr>
<tr>
<td>Road structures</td>
<td>5</td>
</tr>
<tr>
<td>Clay</td>
<td>4</td>
</tr>
<tr>
<td>Sand</td>
<td>4</td>
</tr>
<tr>
<td>Silt</td>
<td>9</td>
</tr>
<tr>
<td>Till</td>
<td>9</td>
</tr>
<tr>
<td>Peat</td>
<td>50</td>
</tr>
<tr>
<td>Water</td>
<td>80</td>
</tr>
</tbody>
</table>

After the Finnish Geotechnical Society

Table 1 Dielectric value

Figure 7 is radar video image from an 110 MHz antenna in profiling mode where the range time has been set to 250 ns, from the above table this equates to a depth of about 11 m. The range is the time the sampling window is left open awaiting for a return signal. The borehole can be seen clearly and there is evidence of the surface layering range. This image was produced on site and the only processing was the variable gain and signal filtering set by the radar system in automatic mode.
Table 3 below shows results from wide angle refraction and reflection radar (WARR) done at the beginning of the study suggesting water content in the basal horizons of the London clay of 38%.

<table>
<thead>
<tr>
<th>Soil Profile</th>
<th>Depth (m)</th>
<th>Borehole</th>
<th>Water content</th>
<th>Inter-layer</th>
<th>Inter-layer</th>
<th>Suggested water content (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FE2</td>
<td>0.9</td>
<td>1.0</td>
<td>0.34</td>
<td>2.0</td>
<td>1.0</td>
<td>0.28</td>
</tr>
<tr>
<td>Dark CLAY</td>
<td>1.5</td>
<td>1.5</td>
<td>0.34</td>
<td>2.0</td>
<td>1.0</td>
<td>0.28</td>
</tr>
<tr>
<td>GRAVEL, etc.</td>
<td>2.0</td>
<td>2.0</td>
<td>0.34</td>
<td>2.0</td>
<td>1.0</td>
<td>0.28</td>
</tr>
<tr>
<td>Fine CLAY</td>
<td>2.5</td>
<td>2.5</td>
<td>0.34</td>
<td>2.0</td>
<td>1.0</td>
<td>0.28</td>
</tr>
<tr>
<td>Dark CLAY</td>
<td>3.0</td>
<td>3.0</td>
<td>0.34</td>
<td>2.0</td>
<td>1.0</td>
<td>0.28</td>
</tr>
<tr>
<td>FE2</td>
<td>3.5</td>
<td>3.5</td>
<td>0.34</td>
<td>2.0</td>
<td>1.0</td>
<td>0.28</td>
</tr>
<tr>
<td>GRAVEL, etc.</td>
<td>4.0</td>
<td>4.0</td>
<td>0.34</td>
<td>2.0</td>
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</table>

Table 2 Estimated two way travel times

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<th>Inter-layer distance (m)</th>
<th>Suggested water content (m)</th>
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<tr>
<td>26.247</td>
<td>0.024</td>
<td>41.786</td>
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</tbody>
</table>

Table 3 Results from wide angle refraction and reflection Radar

CONCLUSIONS

As might be expected for CPT and seismic CPT show good agreement. At the moment this can not be said for radar until geotechnical engineers get into the minutia of image processing this will remain the case.

REFERENCES


Figure 7 Radar Profile scan
Coefficient of Consolidation ($c_h$) from Type 2 Piezocone Dissipation in Overconsolidated Clay

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SYNOPSIS: In dissipation tests using piezocones with type 1 (face) filters, the magnitude of excess pore water pressure always decreases with time. However, with type 2 (shoulder) filters in heavily overconsolidated clays, the pore pressure values can show a temporary increase followed by a subsequent decrease in magnitude. The pore pressures recorded during a dissipation test are a combination of an octahedral component due to plastic failure and a shear component. While the octahedral component affects a relatively large radius surrounding the cone body, the shear component is limited to a thin zone adjacent to the piezocone. This paper seeks to decouple the octahedrally-induced component of excess pore pressure, which is always positive, from the shear-induced component of excess pore pressure, which can be either positive or negative in the type 2 position, in order to model the decay of excess pore pressures and explain the non-standard dissipation curves which are obtained in overconsolidated clays.

1. INTRODUCTION

Much effort has been devoted to determining the coefficient of consolidation, $c_h$, from piezocone dissipation test results. Current methods to model the consolidation that occurs when piezocone penetration is stopped are based on cavity expansion theory, for example Torstensson (1977), or strain path methods, for example Levadoux and Badish (1986) and Houlsby and Teh (1988). The penetration pore pressures that are measured by the piezocone and their subsequent dissipation, are strongly dependent on filter location. In practice, two types of filters are common (Figure 1): a face element (type 1) and a shoulder element (type 2).

Many authors have successfully modeled the decay of pore pressures surrounding the piezocone as long as the tests display standard dissipation patterns (pore pressures always decreasing with time), as is the case for a type 1 piezocone filter regardless of clay consistency. Similarly, when performing dissipation tests in soft to stiff clays with a type 2 filter, standard dissipation curves result. However, type 2 filter dissipation tests in heavily overconsolidated and fissured clays often show a temporary increase in pressures followed by a subsequent decrease in magnitude (Davidson, 1988; Sully, 1991; Chen and Mayne, 1994). Assuming that the pore pressure element was properly saturated, the increase in pore pressures results in a non-standard dissipation curve which does not conform to existing theories.

![Figure 1. Piezocone schematic showing different pore pressure filter locations.](image)

For the non-standard dissipation curves obtained during type 2 dissipation, Sully and Campanella (1994) recommend taking the
maximum pore pressure value which occurs during the dissipation test, and using it as the peak value. The zero time is then empirically shifted to correspond to the time at which the maximum pore pressure occurs, and the curve is then evaluated by standard methods. Few other attempts have been made to deal with nonstandard dissipation curves.

Using an analytical model based on spherical cavity expansion (Vesic, 1972) coupled with an anisotropic constitutive model (Olita, Nishihara, and Morita, 1985), this paper seeks to examine the decay of both the octahedrally-induced pore pressures and of the shear-induced pore pressures separately in order to explain the nonstandard dissipation curves that are seen in overconsolidated clays.

2. DISSIPATION CURVES

It has been demonstrated clearly that the excess pore pressures measured during piezocene penetration depend on filter location on the cone body. Additionally, filter location also affects the shape of the dissipation curve. Dissipation curves recorded for pore pressure filters located on the cone tip (type 1) always show decreasing values of excess pore pressure, regardless of the consistency of the clay. In soft to stiff clay clays, type 2 dissipation records also show pore pressure decreases; however, in heavily overconsolidated clays, type 2 dissipation records often show a temporary increase in pore pressures followed by a subsequent decrease (Davidson, 1988; Sulu, 1991; Chen and Mayne, 1994) (Figure 2).

3. EXCESS PORE PRESSURES

During piezocene penetration in clay soils, significant pore pressures in excess of the hydrostatic pore pressure are generated. The excess pore pressure recorded by the piezocene penetrometer is actually a combination of two different stresses: \( \Delta u = \Delta u_{\text{oct}} + \Delta u_{\text{shear}} \). The octahedral component, \( \Delta u_{\text{oct}} \), is generated due to plastic failure which occurs as the probe penetrates the soil. The zone of soil subjected to plastic failure is a function of the rigidity index, \( I = G/s \), of the soil and is relatively large compared to the cone body. Using cavity expansion theory, the radius of the affected plastic zone can be calculated by the following equations: \( t_{\text{plate/}}t_{\text{cone}} = (L)^{1/3} \) for cylindrical cavity expansion and \( t_{\text{plate/}}t_{\text{cone}} = (L)^{1/2} \) for spherical cavity expansion. Additionally, there is a shear-induced component of excess pore pressure, \( \Delta u_{\text{shear}} \), which affects a smaller thin zone adjacent to the penetrometer.

The octahedral component of the excess pore pressure is always positive, regardless of the filter location or the soil type. However for type 2 cones, the shear induced component may be positive for normally to slightly consolidated soft clays and negative for overconsolidated clays. Since shear-induced pore pressures affect a much smaller zone surrounding the cone, they dissipate more rapidly, thus explaining the initial increase in \( \Delta u \) followed by a subsequent decrease with time in overconsolidated clays. Combining negative shear pore pressures with the octahedral pore pressures leads to the characteristic type 2 dissipation curve in overconsolidated clays. However, in dissipation tests performed with a type 1 filter, both the octahedral and shear component will be positive, leading to a standard dissipation curve in all instances.

![Figure 2. Example dissipation curves in overconsolidated clay with two different filter locations, Baton Rouge, Louisiana (Chen and Mayne, 1994).](image-url)
4. COUPLED PORE PRESSURE MODEL

In this formulation, the component of pore pressure generated by octahedral stresses is decoupled from the component generated by shear stresses and the model handles the dissipations separately. The procedure follows previous work outlined in Chen and Mayne (1994). Additionally, the model uses piezocene penetration data to predict both OCR and rigidity index. Because rigidity index is a parameter that is difficult to estimate reasonably, this is a significant advantage over models which require an input value for the rigidity index.

A standard type 2 dissipation curve from the soft clay at the Bothkennar site in the United Kingdom (Jacobs and Coutts, 1992), and a nonstandard type 2 dissipation curve from the overconsolidated clay at the Strong Pit site in British Columbia (Sully, 1991) were modeled using the following procedure. First, an estimate of OCR (Kulhawy and Mayne, 1990) based on piezocene data was obtained by the following:

\[ OCR = 0.33/(q_0 - \sigma_{so})/\sigma_{uo} \]  

(1)

Shear strength was obtained from an assessment of CK UCS triaxial data (Jamiołkowski et al., 1985):

\[ \sigma_{so}/\sigma_{uo} = 0.33OCR^{1.5} \]  

(2)

The penetration rate during a piezocene test is 20 mm/s, a rate which is much higher than strain rates in laboratory testing. Kulhawy and Mayne (1990) reviewed data from 26 clays tested in triaxial compression and found that the \( s_p \) obtained from the piezocene is 53% higher than that determined from laboratory tests. Due to these differences in strain rate, it is necessary to correct the \( s_p \) predicted in equation (2) by a factor of 1.53.

The rigidity index based on piezocene data was calculated by the following inversion from cavity expansion theory (Chen and Mayne, 1994):

\[ l = \exp [(q_0 - \sigma_{so} - 3.9s_p)/(1.33s_p)] \]  

(3)

Next, the octahedral component of the pore pressure was calculated using spherical cavity expansion (Torstensen, 1977):

\[ \Delta u_{oct} = 4/3s_p\ln(l) \]  

(4)

Finally, the magnitude of shear-induced pore pressures for a type 2 cone was calculated (Chen and Mayne, 1994):

\[ \Delta u_{shear} = \sigma_{so}[(1+2K_o)-2/(2-M)(\sigma_{so}/\sigma_{uo})_{OCR}] \]  

(5)

where:

\[ K_o = (1-\sin\phi')OCR^{1.0} \]  

(6)

\[ M = 6\sin\phi'/(3-\sin\phi') \]  

(7)

The initial distribution for the octahedral component was calculated using spherical cavity expansion while the shear component was assumed to vary linearly in a thin zone, assumed to be a distance of 10 mm, surrounding the cone body. Using the method of finite differences, the components of pore pressure were allowed to decay separately and the results were then summed to provide the measured \( \Delta u \) at the cone/soil interface.

5. RESULTS

The model was applied to two clay sites selected from the literature.

The first site is Bothkennar, a soft clay site located in Scotland which has been well-characterized through previous research. Using equations (1) through (7), the following input values were predicted from the piezocene penetration data: OCR = 1.9, \( s_p = 33 \) kPa, \( l = 81 \), and \( \phi' = 33^\circ \), all of which are in agreement with values reported in the Geotechnique volume “Bothkennar soft clay test site: characterization and lessons learned”.

By trial and error, the value of \( c_0 \) was varied until the predicted dissipation curve gave the minimum standard deviation from the measured dissipation curve (Battaglio et al., 1981). For this location, \( c_0 = 9.5 \times 10^5 \) cm²/s was determined (see Figure 3). The coupled pore pressure model gave an excellent prediction of
the dissipation measured by a type 2 filter in soft clay.

The method was also applied to the Strong Pit site using the following values predicted from the piezocone penetration data: OCR = 4.2, k_s = 135 kPa, L = 22 and ϕ' = 25°. Sully (1991) reported values similar to the values predicted by the piezocone data. Again, the value for the horizontal coefficient was fitted to the dissipation curve by trial and error and was found to equal 8x10⁻³ cm/s, a value within the range measured from laboratory tests given by the author (Sully, 1991). Although the model overpredicts the dissipation pore pressures in the overconsolidated clay, it is able to reasonably predict the shape of the nonstandard type ? dissipation curve at the Strong Pit site (Figure 4).

Similar modeling of pore pressure dissipation for cones with type 1 tips can also be formulated using a triaxial shear mode and stress path analysis beneath the cone tip (Chen and Mayne, 1994).

6. CONCLUSIONS

While methods to establish the horizontal coefficient of consolidation from piezocone dissipation test results are fairly well established, none to date has explained the nonstandard dissipation curves that are seen in heavily overconsolidated clays. By decoupling the octahedral components of excess pore pressure from the shear components, and looking at their decay independently, this preliminary version of an analytical model shows that the negative shear values of pore pressure in overconsolidated clays are of a sufficient magnitude to produce the drawdown that is often seen in type 2 dissipation curves in overconsolidated clays. Work is ongoing to further the development of this approach.
7. REFERENCES


8. LIST OF SYMBOLS

\[ \sigma_h \] = horizontal coefficient of consolidation.

\[ f_s \] = sleeve friction.

\[ G \] = shear modulus.
2.2

\( L \) = rigidity index.
\( K_n \) = lateral stress coefficient
\( = \sigma_{vv}/\sigma_{nn} \).
\( M \) = failure line in Cambridge q-p' space
\( = (6\sin \phi')/(3-\sin \phi') \).
\( q_c \) = measured cone resistance
\( q_t \) = corrected cone tip resistance
\( = q_c + (1-a)u_2 \).
\( s_u \) = undrained shear strength.
\( u_2 \) = penetration pore water pressure measured on the cone face/tip.
\( u_1 \) = penetration pore water pressure measured at cone shoulder.
\( \Delta u \) = excess pore water pressure.
\( \Delta u_{oc} \) = octahedral induced pore water pressure.
\( \Delta u_{taw} \) = shear induced pore water pressure.
\( \sigma_{so} \) = total vertical stress.
\( \sigma_{sv} \) = effective vertical stress.

9. ACKNOWLEDGMENTS

The authors appreciate the continued support of NSF Grant MSS-02-57642 under the direction of Dr. Priscilla P. Nelson, Program Director for Geomechanics.
Type 1 and 2 piezocene evaluations of overconsolidation ratio in clays.

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SYNOPSIS: The overconsolidation ratio (OCR) of natural clay deposits can be evaluated using an analytical model for piezocones where pore water pressure measurements are either midface \( u_d \) or at the shoulder \( u_s \). The formulation is based on spherical cavity expansion and critical state theory and incorporates the effects of initial stress state \( K_0 \), strength anisotropy, strain rate, soil rigidity, and friction angle \( \phi' \). Approximate forms of the model relate OCR directly to \( (q-u)/\sigma' \) for routine use in soft to stiff to hard intact and fissured clays. Statistical analyses of a piezocene database derived from 205 sites verify the approach.

1. INTRODUCTION
The practical problem of determining the in situ stress history of soils requires an approach based upon physical reasoning, theoretical soil mechanics principles, and normalized engineering parameters (Wroth 1988). The method should be calibrated for a variety of known conditions to examine its merits and shortcomings, and if possible, the procedures for data reduction should be stated simply to encourage its implementation into practice.

The determination of yield stress \( \sigma_y \) and OCR in clays by piezocene penetration tests (PCPT) has been of great interest in recent research. Towards this purpose, there have been theoretical developments (e.g., Senneset et al. 1982; Konrad & Law 1987) as well as empirical methods (e.g., Robertson et al. 1986). Generally, the methods address only one type of piezocene: either a type 1 cone, where pore pressures are measured on the cone face \( u_d \); or a type 2 cone, where the filter element is located at the shoulder \( u_s \). Larsson & Mladenović (1990) derived empirical correlations for \( \sigma_y \) from either type 1 and 2 cones, while Sully et al. (1988) required data from both.

Herein, a piezocene formulation based on soil behavioral aspects is briefly described to address both type 1 and 2 cones. Statistical analyses of a large database give relationships of very similar magnitude.

2. MODEL DEVELOPMENT
The formulation is an extended version of an earlier isotropic version by Mayne (1991). To address soil behavioral influences on an advancing cone, a consistent framework using spherical cavity expansion theory and constitutive soil modeling was employed, with due consideration given to: (1) initial stress state, (2) strength anisotropy, (3) stress path analysis, and (4) strain rate effects. A desired aspect for making the method attractive for practical use was to examine these effects within the realm of simple analytical solutions, without the need for complex numerical schemes.

If the full set of input parameters are utilized, an iterative solution is required. However, extensive parametric studies indicate that approximate closed-form expressions can...
be obtained and used adequately for routine explorations. Due to space limitations, this paper only briefly outlines the primary features. Detailed information is given elsewhere (Chen 1994; Chen & Mayne 1994).

The model utilizes spherical cavity expansion theory (Vesić 1977) to express the net cone tip resistance in terms of undrained shear strength and rigidity index. Measured excess pore pressures ($\Delta u_{ex}$) induced by the advancing probe are due to a combination of changes in: (a) octahedral normal stresses associated with cavity expansion, (b) normal stresses from elastic total stress paths, and (c) shear-induced stresses:

$$\Delta u_{ex} = \Delta u_{oct} + \Delta u_{esp} + \Delta u_{shear}$$  

While it is impossible to decouple the measured pore pressures in reality, the components may be evaluated in terms of the soil behavioral aspects. Spherical cavity expansion theory (Vesić 1972) provides the magnitude of $\Delta u_{oct}$. The other two components rely on the specific filter location on the cone and can be evaluated from a stress path analysis.

Initially, soil elements are assumed to be $K_r$-consolidated. During penetration, the magnitude of $\Delta u_{shear}$ depends upon the governing shearing mode to failure, while $\Delta u_{esp}$ varies with the total stress path. The slope of the total stress path is determined by the change of loading conditions surrounding the soil element as the penetrometer approaches. A wedge is formed in the vicinity of the tip within which the soil exhibits elastic behavior. Loading of soil elements located immediately adjacent to the cone tip can therefore be estimated via elasticity theory and failed plastic zones beyond the cone are represented by cavity expansion.

Many type 1 piezocones have the filter element located mid-face, and therefore, an axisymmetric elastic theory solution gives $\Delta \sigma / \Delta \sigma_s = 1/3$ as representative for soil elements under the cone tip. This corresponds to a total stress path of $3V : 4H$ in a Cambridge

Fig. 1. Stress paths for type 1 piezocones.

$$q-p' \text{ space (Chen 1994), in which } q = (\sigma_1-\sigma_3)$$
$$p' = \frac{1}{k}(\sigma_1+2\sigma_3) \text{. A consequence is that } \Delta u_{esp} > 0 \text{ for type 1 piezocones. Figure 1 illustrates the governing stress path for soil elements deformed adjacent to a type 1 cone.}$$

Keaveny & Mitchell (1986) found that the CK,UC triaxial test provides the appropriate strength mobilized immediately beneath the cone tip. The principal loading direction rotates as the soil element moves from the cone face to the shaft. For type 1 cones, both $\Delta u_{esp}$ and $\Delta u_{shear}$ are represented by the CK,UC mode because of the high compression zone beneath the tip. In fact, positive pore pressures are always observed for Type 1 measurements in clays at all OCR ranges (Mayne et al. 1990).

Fig. 2. Stress paths for type 2 piezocones.

For type 2 piezocones, Fig. 2 shows the stress paths corresponding to soil elements adjacent to shoulder filter position. The porous element is located outside of the elastic compression region. Therefore, a pure shearing action occurs, corresponding to a direct simple
shear (DSS) mode, such that pore pressures are all shear-induced and \( \Delta u_{g} = 0 \). Positive \( \Delta u_{g} \) are characteristic of type 2 cones in soft to stiff intact clays, but zero to negative pore pressures have been observed in heavily overconsolidated and fissured clays (Lunne et al. 1986).

Constitutive soil models have been adopted for representing strength anisotropy and differences in stress paths. For example, an anisotropic version of Cam-clay is obtained if the yield surface is rotated about the line: \( K_c = 1 - \sin^2 \theta \) (Ohta et al. 1985). Fig. 3 shows the predicted \( s'_{c}/\sigma_{m}' \) ratios for NC states. For overconsolidated states, \( K_c \) and \( s'_{c}/\sigma_{m}' \) are expressed as power functions of OCR (Kulhawy & Mayne 1990). Considerations are also given in correcting \( s'_{c} \) for strain rate effects resulting from a much faster time-to-failure for piezocones than for standard laboratory tests.

![Diagram showing strength predictions for NC clay using anisotropic constitutive model.](image)

Fig. 3. Strength predictions for NC clay using anisotropic constitutive model.

The resulting equations for type 1 and 2 are quite lengthy and involve the following input soil parameters: effective cohesion intercept \( c' \), effective stress friction angle \( \phi' \), lateral stress coefficient \( K_c \), ratio of undrained shear strength in DSS mode to that in CK,UC mode \( K_{ss} = s_{ud}/s_{uc} \), plastic volumetric strain ratio \( \Lambda = 1 - C_{p}/C_{o} \), and strain rate \( (\partial s/\partial t) \). An extensive series of parametric studies indicated that the predictive equations could be simplified substantially for practical usage. Therefore, the following simplified equations are suggested:

**Type 1 (midface element or \( u_\theta \):**

\[
OCR = 0.81(q_1 - u_\theta)/\sigma_{m}'
\]  

**Type 2 (shoulder element or \( u_\phi \):**

\[
OCR = 0.46(q_1 - u_\phi)/\sigma_{m}'
\]  

An interesting outcome is the similarity of this approach and that of others. The independent analyses of Sennestet et. al. (1982) using plasticity theory and Konrad & Law (1987) using an effective stress approach both suggest the following expression for type 2 cones (Robertson et al. 1988).

\[
OCR = 0.49(q_1 - u_\phi)/\sigma_{m}'
\]  

Moreover, Battaglia et al. (1986) and Larsson & Mulabdić (1990) also related OCR to the parameter \( (q_1 - u_\phi)/\sigma_{m}' \).

### 3. DATA FILTERING TECHNIQUE

A method of filtering and smoothing the data was developed so that the predictions could be systematically compared with reference values of OCR obtained from standard laboratory oedometer results. An example of the filtering process is illustrated using piezocone data from an offshore clay in the Kringalik Plateau of the Beaufort Sea (Jefferyes et al. 1987; Hughes et al. 1984).

Fig. 4 shows a set of raw piezocone data from the Kringalik site and indicates measured variations in cone tip resistance \( q_1 \) and penetration pore pressures \( u_\phi \). The irregularities in the \( q_1 \) and \( u_\phi \) profiles might be due in part to natural soil fabric, the inclusion of sand particles or sandy seams within the clay matrix, as well as possible electronic noise. These variations (squiggly lines) are not indicative of the stress history of the clay, however, and must be factored out for a proper analysis.
The piezocone data were initially evaluated via the type 2 formulation to provide estimated OCRs (see Fig. 5). The OCR values were then processed using a "moving average" technique over a vertical autocorrelation distance of 0.30 meters. A mathematical function was then used to curve-fit the data and generate a smoothly-transitional profile of OCR with depth, as indicated in Fig. 6. Typically, a power function format was sufficient for many sites with simple stress histories, although this proved inadequate for layered deposits and sites with complex stress histories.

Figure 7 shows the final product with good agreement evident between the smoothed piezocone interpretations at the Kringalik site and the reference OCRs obtained from laboratory oedometer tests.

4. APPLICATIONS
The formulations have been applied to a number of case studies where type 1 or 2 or both types of piezocone data have been made available. The authors also conducted field tests with different cones at new test sites.

One new test site is located in Baton Rouge, Louisiana at the I-10 interchange with state highway 42. The thick deposit (> 40 m) of desiccated deltaic clay is of Pleistocene age. Average index properties of the clay include: water content = 34 ± 12; liquid limit = 60 ±
20; plasticity index = 33 ± 13, indicating CH material. Thin-walled tube samples were retrieved and consolidation tests determined a rather constant profile of yield stress with depth $\sigma' = 1 \text{ MPa}$ (Chen 1994). In Fig. 8, results from type 1 cone predictions are seen to be comparable with the reference OCRs.

![Graph](image)

Fig. 8. Filtered OCR predictions and reference oedometer values for Baton Rouge site.

5. STATISTICAL TRENDS
A large piezocene database was compiled and evaluated using regression techniques. Files from over 205 clay sites were reviewed with the summary results presented in Figures 9 and 10 (Chen & Mayne 1994). The relationships from log-log regressions may be expressed:

Type 1 (face: $n = 611$; $r^2 = 0.800$)

$$\frac{\sigma'_v}{\rho_a} = 0.744[(q_1-u_1)/\rho_a]^{0.06} \quad (4)$$

Type 2 (shoulder: $n = 884$; $r^2 = 0.804$)

$$\frac{\sigma'_v}{\rho_a} = 0.490[(q_1-u_1)/\rho_a]^{1.05} \quad (5)$$

where $\rho_a = 1 \text{ atm} = 100 \text{ kPa} = 1 \text{ tsf}$. These may be further simplified to give:

Type 1: $\text{OCR} = 0.75(q_1-u_1)/\sigma'_v \quad (6)$

Type 2: $\text{OCR} = 0.50(q_1-u_1)/\sigma'_v \quad (7)$

The empirical statistical equations given by (6) and (7) are remarkably similar to the model predictions given by (2) and (3). Effects of soil plasticity may also be considered, as suggested by Larsson & Mulabdić (1990). Figures 9 and 10 show results from multiple (log) regression analyses that include the plasticity index ($I_p$). Perhaps additional factors, such as sensitivity, structure, age of the

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5. CONCLUSIONS
A cavity expansion/anisotropic Cam-clay formulation relates OCR to the normalized effective cone resistance, \(q_{c}/a'_c\), and addresses pore pressures measured either midface or at the shoulder. Statistical database analyses support the analytical expressions.

6. ACKNOWLEDGMENTS
Appreciation is given to NSF for grant MSS 91-08234 under Dr. Mehmet T. Tumay and continuing grant MSS 92-37647 under Dr. Priscilla P. Nelson.

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Robertson, P.K., et al. (1986). Use of piezometer cone data. Use of In-Situ Tests in Geotechnical Engineering (GSP 6), ASCE, New York, 1263-1280.
Determination of friction angles in sand based on CPT results

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Hans Denyer
Danish Geotechnical Institute, Lyngby, Denmark

SYNOPSIS:
This paper suggests a simple method to determine the strength of sands in terms of the friction angle. The background for the proposed method is based on an intensive literature review of several existing procedures mainly valid for normally consolidated sand. Based on these methods the paper presents a simple expression valid for normally consolidated sand. Since the natural sand deposits in many cases is preconsolidated the effect from this loading on the in situ stresses must be taken into account. The influence of the preconsolidation is evaluated and a method valid for both normally consolidated and preconsolidated sand is proposed. A simple expression is offered to calculate the angle of friction dependent on the tip resistance, vertical effective stress and the overconsolidation ratio (OCR).

1. INTRODUCTION
Through the last 25 years various methods to determine the strength of soils based upon CPT results have been proposed. In this paper a simple method is proposed, qualitatively and quantitatively based on these methods.

2. LITERATURE REVIEW
In the literature a wide range of different methods is proposed to determine the friction angle in sand based on CPT results. Some of the more known and employed correlations have been included in this study. These correlations are summarized in Table 1. The different approaches in the calculation process and the explicit equations can be found in the references. However, the parameters used as input in the different correlations are listed in Table 1. It should be noted that in general the equations are only valid for normally consolidated sand (NC-sand) with the assumption that the stress at rest can be determined as $K_o = \sigma' / \sigma_{sl} = 1 - \sin(q') / 1.1$

The following notation is utilized:
- $\sigma'$, and $\sigma_{sl}$: Effective in situ stresses (vertical and horizontal)
- $q'$: Tip resistance
- $D_i$: Relative density
- Index OC: Preconsolidated sand
- Index NC: Normally consolidated sand
- $a$: Attraction, $a = c \cot(q')$
- $\alpha, \beta$: Constants
- $K_o$: Earth pressure coefficient
- $\psi'$: Plane friction angle (tangential)

As illustrated in Table 1, the strength correlations presented are simple and involve relatively few parameters. In most cases the determination of the friction angle is only based...
Table 1. Summary of existing correlations

<table>
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<th>$q_c$</th>
<th>$\sigma'_v$</th>
<th>$\sigma'_h$</th>
<th>$K_o$</th>
<th>$D_r$</th>
<th>$a$</th>
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<td>4. Schmertmann (1978)</td>
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<td>5. Meyerhof (1974)</td>
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<td>x</td>
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$^1$ Angle of plasticity

upon the tip resistance $q_c$ and the effective vertical stress $\sigma'_v$.

For the correlations where the horizontal effective stress $\sigma'_h$ is introduced (or $K_o$), it is in principle also possible to estimate the friction angle in a preconsolidated sand. However, a major limitation is introduced since the horizontal stress is difficult to estimate.

Method no 9 (in Table 1) is the equation based upon Bolton’s Stress-Dilatation theory. It determines in fact the friction angle $\phi'$ on the basis of $D_r$, $\sigma'_v$, and $\sigma'_h$, but $D_r$ can be estimated based on $q_c$ and $\sigma'_v$ (see relation 2a in Table 1).

Figure 1 shows six of the correlations given in Table 1 (as $q_c$ vs $\sigma'_v$ plots). Each of the correlations is illustrated for three different friction angles ($\phi' = 30^\circ$, $35^\circ$, $40^\circ$).

The figure shows the variation of the friction angle estimated by the different equations. It can also be seen that the span of variation increases with increasing friction angle.

Methods by Bolton (1986) and Houlshy & Worth (1989) predict the greatest friction angles whereas methods by Lunne and Christoffersen (1983) and Durgunolu & Mitchell (1975) estimate the smallest values. It should be added that for several procedures it is not clearly stated whether or not the estimated friction angle is a tangent or a secant angle. However, the difference between these values cannot explain the substantial deviations observed.

![Figure 1. Friction angles for normally consolidated sand.](image)

Table 1 also includes Schmertmann’s method to convert the measured tip resistance from a preconsolidated sand to the value that would have been measured, if the same sand was normally consolidated. Unfortunately, the conversion factor is a function of the overconsolidation ratio (OCR), which again cannot be determined on the basis of CPT results alone.

The principal problems in the interpretation process of $\phi'$ based upon CPT results can be summarized as:

- An increase of the horizontal stress produces a greater tip resistance for a specific sand type with a given density (this is known from tests in calibration chambers).
• The stress conditions in situ are only known partly, since only the vertical stress \( \sigma_v \) can be reasonably estimated. The horizontal stress \( \sigma_h \) is influenced by the overconsolidation (the OCR value) which again is difficult to estimate.

3. NORMALLY CONSOLIDATED SAND

The correlations presented to determine the friction angle in sand in section 2 show a deviation in the interpretation of CPT results.

One of the reasons is the fact that the correlations are based on different types of sand. However, the variation of the correlations is still within a limit which makes it reasonable to try to evaluate a general equation based upon these expressions (from Table 1).

The general equation is aimed to represent an "average solution" compared to the presented correlations. Since the evaluation of the friction angle is very dependent on the vertical as well as the horizontal stress, the method must incorporate both. Bolton's (1986) equation is adopted as the basis for this "average solution".

For sand Bolton's general equation can be written as (\( p'_p = 1 \) kN/m²):

\[
\psi' - \psi_{\text{opt}} = A(D_p (10 - \ln(p'_p/p_s)) - R)
\]

(1)

\[
p' = (\sigma'_v + 2\sigma'_h)^{1/3}
\]

(2)

By adjusting the coefficients and constants (\( \psi_{\text{opt}} \), \( A \), \( R \)) in equation (1) and comparing the result with the correlations shown in Figure 1, a relatively good average solution is obtained for \( \psi_{\text{opt}} = 30^\circ \), \( A = 4^\circ \) and \( R = 1 \). Inserting in (1):

\[
\psi_{\text{nc}} = 30^\circ + 4(D_p (10 - \ln(p'_p/p_s)) - 1)
\]

(3)

where \( D_p \) and \( p'_p \) are determined as (Lunne & Christoffersen, 1983):

\[
D_p = (1/2.91) \ln (q_s/(61 \sigma'_v/p_s)^{0.71}))
\]

(4)

\[
p'_p = \sigma'_v (1 + 2K_c)/3
\]

(5)

where the triaxial friction angle is \( \psi' / 1.1 \).

Equation (3) is not an explicit expression, and hence, not very easy to apply. However, it can be approximated with the relatively simple equation (6) given below.

\[
\psi_{\text{nc}} = 17.2^\circ (q_s / \sigma'_v)^{0.185}
\]

(6)

Figure 2 shows this equation together with existing methods (from Figure 1). Figure 2 illustrates that equation (6) predicts an "average" value reasonably well in comparison with the existing procedures.

4. PRECONSOLIDATED SAND

In a preconsolidated sand the horizontal stresses differ from the stresses in a normally consolidated state. In the normally consolidated state the earth pressure coefficient at rest, \( K_c \), is expected to be constant with depth:

\[
K_c = \sigma_{\text{Isn}} / \sigma'_v
\]

(7)

In the preconsolidated case it is still possible to express the ratio between horizontal and vertical stresses:

\[
K = \sigma_{hs} / \sigma'_v
\]

(8)

where \( \sigma_{hs} \), \( \sigma_{Isn} \) are the effective stresses in the normally consolidated and overconsolidated states, respectively. The definition of the stresses is illustrated in Figure 3.

Although the stresses still are in situ stresses, \( K \) will not be constant with depth.

Assuming the maximum preconsolidation load acting previously is denoted \( \Delta \sigma_{\text{asc}} \). The vertical stress is then equal to:

\[
\sigma_{Isn} = \sigma'_v + \Delta \sigma_{\text{asc}}
\]

(9)
As the state at this time is normal consolidation, the earth pressure coefficient at rest $K_0$ is

$$K_0 = \frac{\sigma_{\text{oc}}}{\sigma_{\text{oc}}'}$$  \hspace{1cm} (10)

Consequently, if $K_0$ and $\sigma_{\text{oc}}'$ are known $\sigma_{\text{oc}}'$ can be calculated.

As the preconsolidation load is reduced, $\sigma_{\text{n}}'$ will also decline. However, the horizontal stress will be partly preserved. Due to this the measured tip resistance ($q_t$) in a preconsolidated sand will be higher compared to (the same sand) in a normally consolidated state. From tests performed in calibration chambers it has namely been observed that the increase in tip resistance is proportional with the horizontal stress in the power $\beta$.

Consequently, the ratio between the measured tip resistance in a preconsolidated sand ($q_{t,\text{max}}$) and an imaginary measurement ($q_t$) at same depth (i.e., same $\sigma_{\text{n}}'$) but with the soil assumed to be normally consolidated, can be written as:

$$q_{t,\text{max}} / q_t = (\frac{\sigma_{\text{tt}}}{\sigma_{\text{tt}}'})^\beta$$  \hspace{1cm} (11)

or by inserting application of equations (7) and (8):

$$q_{t,\text{max}} / q_t = (\frac{K}{K_0})^\beta$$  \hspace{1cm} (12)

A widely used expression to describe the unloading curve (approximately) is

$$\frac{K}{K_0} = \text{OCR}^{-\alpha}$$  \hspace{1cm} (13)

where $\alpha$ is a constant. It can be shown that equation (13) is not valid for high values of OCR, since this will introduce a stress condition exceeding the ratio given by failure criterion. The maximum value of $K$ can be approximated to.
\[ K = \sigma_{R0} / \sigma' = (1 - \sin(\phi'/1.1))^{-1} \] 

or

\[ K / K_0 = (1 - \sin(\phi'/1.1))^{-2} \] (14)

The constants \( \alpha \) and \( \beta \) have been evaluated in the literature. Parkin (1988) has on basis of CPT results from calibration chamber tests proposed a value of \( \beta = 0.5 \). Lunne & Christoffersen (1983) proposes \( \alpha = 0.45 \) whereas Schmertmann (1978) suggests \( \alpha = 0.42 \) and \( \beta = 0.75 \).

For the correlation presented in this paper it was found that \( \alpha = 0.42 \) and \( \beta = 0.5 \) resulted in the best fit.

The reduction of the measured tip resistance in a preconsolidated soil can thus be introduced by a reduction factor denoted \( F \):

\[ q_t = F \cdot q_{\text{meas}} \] (15)

where \( F \) is defined in equations (13) and (14) as:

\[ F = K / K_0 = \min(\text{OCR}^{0.2}, (1 - \sin(\phi'/1.1))^{-2}) \] (16)

The evaluation of a general equation estimating the friction angle and taking the overconsolidation effect into account can now be accomplished by introducing the reduction factor, \( F \), in the equations (4) and (5).

An approximate expression similar to the one valid for normally consolidated sand was evaluated. The friction angle is thus calculated in terms of tip resistance, vertical stress and the overconsolidation ratio. A suitable simple expression is determined as:

\[ \phi'_s = (17.2^\circ)(q_t / \sigma'_v)^{0.125}\text{OCR}^{0.06} \] (17)

assuming \( \text{OCR}^{0.2} < (1 - \sin(\phi'/1.1))^{-2} \) which normally is the case.

The expression (17) has been tested against other in situ test results from one site near Sallerod (north of Copenhagen). The in situ tests performed to determine the friction angle was pressuremeter tests, screwplate tests and plate loading tests. The results are shown in Table 2.

<table>
<thead>
<tr>
<th>Pit</th>
<th>( \sigma'_v ) [kN/m²]</th>
<th>q_t [MN/m²]</th>
<th>OCR</th>
<th>( \phi'_{s1} ) [°]</th>
<th>( \phi'_{s2} ) [°]</th>
<th>( \phi'_{s3} ) [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>54</td>
<td>4.0</td>
<td>10</td>
<td>33.8</td>
<td>32.9</td>
<td>33.2</td>
</tr>
<tr>
<td>B</td>
<td>36</td>
<td>5.9</td>
<td>15</td>
<td>37.9</td>
<td>37.0</td>
<td>36.4</td>
</tr>
<tr>
<td>C</td>
<td>36</td>
<td>6.5</td>
<td>15</td>
<td>39.2</td>
<td>39.3</td>
<td>38.2</td>
</tr>
</tbody>
</table>

1) measured by in situ tests
2) expression (3) with F-factor
3) expression (17)

The results from Sallerod show reasonably good agreement between the proposed method expression (17) and the results based on other test types.

4. CONCLUSIONS

Based on a literature study the paper presents a calculation method to estimate the plane friction angle \( \phi' \) in sand based upon CPT results.

The problems concerning the influence of the results by preconsolidated sand have been considered. Hence the proposed method incorporates the OCR in the calculation algorithm.

The method has been tested on one test site in Denmark with preconsolidated sand with well-known properties determined from pressuremeter tests, plate load tests and screw plate tests. The measured values compared to the estimated were in excellent agreement with the corresponding calculated values.

However, further work must be done to verify CPT data from sites where the friction angle is known, based on other in situ tests or laboratory tests (e.g. triaxial test). Furthermore, the expression to determine \( D_3 \) should be overhauled as poor fitting was noted for the Sallerod site.
5. REFERENCES


Correlations between the CPT and the SPT for some Brazilian soils

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SYNOPSIS. Correlations between 120 CPTs and 114 SPT boreholes up to 40 m depth have been carried out from 22 foundation jobs situated in Rio de Janeiro city. Different types of correlations were tentatively tested, the simple linear correlation $q = KN$ providing as good correlation coefficients as the other functions. A general trend of $K$ values increasing with increasing grain size was obtained, as also found by other authors. Values of $K_{40} = q/N_{40}$ have been suggested in the range 0.21–0.50 MPa/blow/0.3 m, respectively for clay and silty clay to pure sand.

1. INTRODUCTION
Many foundation jobs are designed and constructed in Brazil with the sole information obtained from the SPT. Experience has shown that notwithstanding the roughness character of the SPT, its importance and usefulness can be attested by the behaviour of many jobs for which important design decisions and execution control procedures have been based on SPT profiles alone.

The CPT is common in Brazil for offshore sites and some special jobs. On the latter case the CPT is employed as an in situ complementary test to the SPT borings.

As long as the CPT can be directly related to the bearing capacity of piles, the establishment of correlations between both tests are of utmost importance for design purposes in Brazil for current foundation jobs.

In fact, many authors (e.g. Meyerhof, 1956, Costa Nunes and Fonseca, 1959; Velloso, 1959, Schortemeyer, 1970, 1978, Alonso, 1980, Robertson et al, 1983) have suggested correlations between the blow count $N$ from the SPT and the cone resistance $q$, from the CPT.

Moreover, some authors (e.g. Sanglerat, 1972, Robertson et al, 1983) have collected existing correlations from different countries.

Results of correlations between 120 CPTs and 114 SPT boreholes up to 40 m depth are presented in the paper. The available data are related to 22 foundation jobs situated in Rio de Janeiro city and correspond to 2022 ($q$, N) data pairs. The data were collected by the authors when both worked for Franki Piles Ltd., Brazil.

A mechanical penetrometer cone was used in all CPTs.

2. CRITERIA FOR SELECTING THE DATA
The values of ($N$, $q$) representing a very well characterized stratum needed to be selected for the statistical analysis. For this task it would be recommended to draw a soil profile for each CPT and SPT boring, in order to eliminate the data related to change in layer depth and local discontinuities in soil profile. Due to the great number of CPTs and SPTs available, the establishment of many soil profiles turned out to be impractical. It was therefore decided to select...
all the data for an initial analysis and to choose a criterion for selecting the data in another step of the analysis.

For the cone resistance, the average value from the readings (4 or 5) for each meter depth was considered. The blow count value from the SPT borings concerned to each meter depth was joined to the average q<sub>c</sub> at the same depth, related to the closest CPT profile.

After selecting the data pairs (N, q<sub>c</sub>), those related to the same soil type were plotted in a graph with N values on the horizontal scale and q<sub>c</sub> values on the vertical one. The observation of such graph has shown some data points substantially far from the typical trend of most of the data. It was verified that these distant points were really related to depths of changing layer or local discontinuities that could distort the statistical correlations if they were not eliminated from the analysis.

Two statistical analyses were performed:

i) An analysis in which all available data related to each soil type were included.

ii) A partial data analysis in which those data situated very far from the general trend of most data in a N versus q<sub>c</sub> graph were disregarded. In order to perform the second analysis a statistical criterion had to be established. Such procedure aimed at eliminating some data as long as it was not possible to verify the origin of each set of (N, q<sub>c</sub>) value situated far from the general data trend. It was not possible to reconstitute occurrences such as sample imperfections, human faults in counting the blows, reading errors an so on.

A simple procedure for selecting the data was used, i.e. the values of K=q<sub>c</sub>/N away from one standard deviation of the mean K value were disregarded. After the elimination of those data the same trend of most data was confirmed to be maintained in the N versus q<sub>c</sub> plot, as expected.

3. TYPES OF CORRELATIONS

The next step in the analysis consisted in choosing the type of function most suited to the data for each soil type. Three functions were tested:

i) Linear correlation, by the method of least squares obtaining a line, q<sub>c</sub>=a+bN with an intercept at the origin.

ii) Linear correlation by the method of least squares, with an adaptation of the method to obtain a line q<sub>c</sub>=KN with a zero intercept.

iii) Power correlation, q<sub>c</sub>=KN<sup>d</sup> by the method of least squares.

For each function both analyses described above were carried out, i.e. an analysis in which all data were included and a partial data analysis in which the data were selected according to the previous section.

This procedure was carried out for each soil type and depth range. However, due to the lack of space in the present paper, for each soil type just the data for all depths are presented. The main results are summarised in the following section.

4. MAIN RESULTS

From a general point of view the following conclusions were reached:

i) For the three functions established, the correlation coefficients found were significantly greater for the partial data analysis, as expected. Partial data analysis indicated correlation coefficients in the range 0.53-0.87 and all data analysis in the range 0.40-0.69. These values are in the same range obtained by Schmertmann (1970) for a similar analysis.

ii) The power function, q<sub>c</sub>=KN<sup>d</sup>, did not indicate greater correlation coefficients when compared to the linear function, q<sub>c</sub>=KN.

iii) For the partial data analyses, the three functions - linear, linear with zero intercept and power- were very close to each other, the origin intercept a approaching zero and the exponent d approaching unity.

iv) Very close values of K were found for the linear function, q<sub>c</sub>=KN, for a sandy clay and for a clayey sand, indicating that the same soil type can be grouped in both classifications depending on the operator criterion. This aspect was also observed in silty sands classified as sandy silts and so on. All these mixed soils presented correlation coefficients on the lower range. It must be pointed out that
the samples from the SPT sampler are not submitted to sieve analysis, the soil classification is done from visual inspection.

Figure 1 summarises the results found for the linear correlation, \( q_c = KN \). The application of this type of function is suggested by the following reasons:

i) Simplicity.

ii) The other functions tried did not provide better correlation coefficients. Moreover, the other functions, \( q_c = a + bN \) and \( q_c = cN^d \) provided coefficients close to zero and \( d \) values close to unity.

iii) The linear function \( q_c = KN \) is better suited for practical applications to Brazilian semi-empirical methods to predict pile bearing capacity, Auksi and Yellowou (1975) and Yellowou (1981).

Soil classification is indicated on the horizontal axis of Figure 1 according to decreasing grain size. The horizontal axis has no scale as the representation of decreasing grain size is just qualitative. As mentioned before, the samples from the SPT sampler are submitted just to visual inspection for classification purposes. The \( K \) values are indicated on the vertical axis.

The thin line on Figure 1 represents the correlation for the all data analysis and the thick line that for the partial data analysis. It can be observed that there is a trend of \( K \) values to decrease with decreasing grain size. It can also be verified that for silts the all data and partial data analyses present the most distinct behaviour on \( K \) values.

Figure 1 also shows high values of \( K \) for materials classified just as clays. This is attributed to the fact that these soils (or sometimes also silty clays) correspond in the present data to very soft soils, for which the \( N \) values are very close or even equal to zero. In that case the \( K \) values tend upward. Schmertmann (1978) found the same trend for sensitive clays which presented a \( q_c/N \) ratio very high because of very low \( N \) values. It is quite clear that in soft soils the SPT is not able to properly sense "how soft the soil is" and therefore is not suited to be used in this kind of material.

From the information given on Figure 1, and taking into account the partial data analysis in the cases of much scatter (mainly for silty soils), Table 1 was prepared.

Table 1 provides \( K \) values suggested to be applied to estimate cone resistance in cases where only the SPT profile is available.

---

**Fig. 1 - \( K = q_c/N \) as a function of soil type.**
Table 1. Suggested values of K

<table>
<thead>
<tr>
<th>Soil nature</th>
<th>Suggested value of K (q&lt;sub&gt;s&lt;/sub&gt;=KN, K in MPa/blow/0.3m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>0.60</td>
</tr>
<tr>
<td>Silty sand, clayey sand, sand with clay and silt</td>
<td>0.53</td>
</tr>
<tr>
<td>Silt, sandy silt, sandy clay</td>
<td>0.48</td>
</tr>
<tr>
<td>Silt with sand and clay, clay with silt and sand</td>
<td>0.38</td>
</tr>
<tr>
<td>Clayey silt</td>
<td>0.30</td>
</tr>
<tr>
<td>Clay, silty clay</td>
<td>0.25</td>
</tr>
</tbody>
</table>

It should be pointed out that the values suggested in Table 1 were obtained in correlations established in soils from Rio de Janeiro city. The suggested K values do not reproduce intrinsic particularities from a given site.

5. IMPORTANCE OF STATISTICAL PROCEDURE AND NORMALIZED STANDARD ENERGY

Before comparing existing correlations, two main points should be emphasized.

The first is related to the procedure in which the correlations were established by several authors. Unfortunately most of the authors do not specify the statistical procedures employed in the suggested correlations. In fact the K value can be obtained by two distinct procedures:

i) Establishing a linear correlation by the method of least squares. This was the procedure employed in the correlations presented in Table 1, also presented by e.g. Schmertmann (1970).

ii) Calculating the average value of K=q/N from a set of individual values as in a normal distribution. The authors believe that this is the most common procedure.

While in the first procedure a statistical correlation is obtained between two variables, the second represents a normal distribution of a single variable K=q/N in which K just denotes the most frequent value of this single variable.

A criticism that can be pointed out from many correlations published in the literature is the lack of information concerning the statistical procedures applied to obtain most of the correlations.

The second point is related to the importance of the average energy level on which local SPT design correlations are based. Actually, among the several factors that can influence SPT results (see e.g. Seed et al, 1985, Skempton, 1986, Décourt, 1989), the most important one is certainly the energy delivered to the sampler rods, named Enthru energy by Schmertmann and Palacios (1979), and denoted E<sub>e</sub>. Schmertmann and Palacios (1979) have shown that N varies inversely with E<sub>e</sub>. Schmertmann and Palacios (1979) also mentioned that "any standardization of the SPT aimed at reducing variability in N values must include standardizing E<sub>e</sub> or correcting N values to a standard E<sub>e</sub>". According to Robertson et al (1983), Schmertmann (1976) has suggested that based on limited data, an efficiency or energy ratio (ratio between Enthru energy and theoretical maximum energy, 475 J) of 55% may be the norm for which it can be assumed that many North American correlations were developed. Kovacs and Salomone (1982) have suggested the concept of the National Average Energy. Seed et al (1985) and Skempton (1986) have proposed a standard value of 60%. Therefore, in order to obtain the value of the number of blows corresponding to a standard energy ratio of 60%, N<sub>60</sub>, expression (1), from Seed et al (1985) must be used:

\[ N_{60} = \frac{N \times ER}{60} \]  

where

- N = number of blows for the method used
- ER = energy ratio or efficiency

The measurement of the Enthru energy for Brazilian SPT has been carried out by Belincanta (1985) and Décourt et al (1989). It was concluded that the average efficiency of a typical, correctly performed, Brazilian SPT was about 72%. This value was used to correct the suggested K values in Table 1 to obtain K<sub>60</sub> values, which are presented in Table 2.
in Table 1 to obtain $K_0$ values, which are presented in Table 2.

Table 2. Suggested values of $K_0$.

<table>
<thead>
<tr>
<th>Soil nature</th>
<th>Suggested value of $K_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>0.50</td>
</tr>
<tr>
<td>Silty sand, clayey sand, sand with clay and silt</td>
<td>0.44</td>
</tr>
<tr>
<td>Silt, sandy silt, sandy clay</td>
<td>0.40</td>
</tr>
<tr>
<td>Silt with sand and clay, clay with silt and sand</td>
<td>0.32</td>
</tr>
<tr>
<td>Clayey silt</td>
<td>0.25</td>
</tr>
<tr>
<td>Clay, silty clay</td>
<td>0.21</td>
</tr>
</tbody>
</table>

Values of $K_{soil}$ and $K_{silt}$ for some types of soils included in the present analysis are also plotted in Figure 2. It must be pointed out that $D_30$ values were just estimated, as sieve analysis is not carried out in connection with SPTs in Brazil. It can be seen that for the case of coarser materials the suggested $K$ values are in the same range as the ones collected by Robertson et al (1983). In the case of finer materials, there is a trend of slightly higher $K$ values in respect to those of Robertson et al. (1983).

Fig. 2 - Variation of $q_s/N$ ratio with mean grain size (Robertson et al, 1983).

6. COMPARISON WITH EXISTING CORRELATIONS

The correlations presented herein are an updating of the data previously presented by Costa Nunes and Fonseca (1959) and Velloso (1959). Other Brazilian data were presented by Alonso (1980) for soils from the city of São Paulo. Except for exceptionally high values of $K=0.94$ for a clayey sand and $K=0.72$ for a silty clay, Alonso (1980) suggested $K$ values in the range $0.21-0.64$ (i.e. $K_{soil}$ in the range 0.18-0.53) for soils ranging from silty clay to clayey sand, respectively.

Robertson et al (1983) tried to rationalize the variations of the $q_s/N$ ratio obtained in a considerable number of publications by plotting the derived $q_s/N$ as a function of mean grain size, $D_{50}$. Figure 2. Robertson et al (1983) mentioned that they had some difficulty in defining $D_{50}$ from some of the references. They also pointed out that most of the data shown in Figure 2 were obtained using the standard donut type hammer with a rope and earhead system for which the typical energy delivered is about 50-60% of the theoretical maximum.

Other data from Robertson et al (1983) indicated that the general trend shown in Figure 2 corresponded to an average energy ratio of 55%.

A recent and promising proposition by Jeffries and Davies (1993) could not be tested with the present data since the proposition is to be used just with results from piezocones tests.

7. CONCLUSIONS

Correlations between 120 CPTs and 114 SPT boreholes up to 40 m depth have been carried out from 22 foundation jobs situated in Rio de Janeiro city. Different types of correlations were tentatively tested, including a linear function with an intercept at the origin, a linear correlation passing through the origin and a power correlation. The simple linear correlation $q_s=K_N$ provided as good correlation coefficients as the other functions.

A general trend of $K$ values increasing with increasing grain size was obtained, as also found by other authors.
The suggested values have been compared to existing correlations collected by Robertson et al. (1983). It was found that for coarser materials the K values suggested in the present paper are in the same range as the ones collected by Robertson et al. (1983). In the case of finer materials, there is a trend of slightly higher K values in respect to those from Robertson et al. (1983).

8. REFERENCES


INTERPRETATION OF CPT FOR ALLUVIAL SUBSOIL AT HAJIRA

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SYNOPSIS: The investigation for a project site in alluvium near Surat is taken as case study. The conventional exploration programme, pre-specified numbers of bores and conducting SPT - vane - UDS at specified interval provides a jungle of data e.g. vane in silt and sand and SPT in clays. The zoning profile evolved requires considerable running. This enables revision of CPT/DPH correlation for given geological formation. Still for each strata range of values are more confusing than resolving the foundation problem.

The authors have presented an approach to obtain DPH data at 100 to 200 m grid to evolve zoning, profile, properties for stratum (critical), GWT and provisional foundation depth, system and ground treatment. All the design parameters are then cross checked by CPT. Any doubts left in particular layer could be explored by open pit, pressurometer, gamma radiations etc.

1.0 REGIONAL FORMATION
Surat city is situated on west coast of India and on banks of river Tapi. This area has rapid industrial growth with steel, cement, fertilizer, chemicals, petrochemicals and power projects in last decade. The easy and economical land available is flood plain-alluvium at confluence of Arabian sea and river. The flood and tidal deposits have led to fresh and saline water sedimentation. The deposits are stratified with cohesive soils on left bank and layered silts, sands with clay layers/lenses or clods on right bank. The alluvium is deeper than 90 m. The stratifications are variably thin.

The sampling may or may not cover one layer and hence samples collected and tested for classification gives large range. A thin layer of massive clay in silt and fine sand will be classified as CH-SC. Thus designers predicted behaviour, on basis of report, will not be reflected by structure on such deposits. These soil samples leached by rain water seepage or tidal water ingress gives different properties at different times. Clay clods in fine sand will behave as fine sand but profile will classify it as SC - cohesive.

Thus experience and judgement based insitu tests in such soils for each formation plays major role. The commonly used tests are SPT, DPH, CPT, vane and occasionally pressurometer.

The average ground level is 3.5 to 4.5 m above sea level and this area of Hajira belt is covered by the tidal variations on 3 sides. (River, Sea, Teena creek). The highest tide is 4.8 m RL and the lowest level is 0.2 m RL.

2.0 SOIL EXPLORATION PRACTICE
The usual standard tender specifications for site investigations (irrespective of local conditions) based on IS code or International standards have been evolved by many consultants. They require drilling sampling and SPT alternately at 1.5 m interval. The generalized depth of exploration is 30 m or so. Stages of exploration except in case...
of dams, are merged into one for want of time and money.

The number of bores, samples, SPT and vane data for stratified deposit in area provides data in bulk like jungle. There are many cases in which 4 to 5 samples are tested to log 30 m mostly based on the drillers visual description.

To break this chain and get out of time, cost, confusing data and contradictions in properties by different tests in present practice, authors have attempted different modified approach of exploration for alluvial deposits.

3.0 MODIFIED EXPLORATION
The site plan is marked with a grid of 100 m x 100 m. Initially exploration by DPH (IS - DCPT) is carried out on 200 m to 300 m grid. The data of sounding test Nₜ blows/30 cm penetration is plotted against depth. The tests results are superimposed to make out zoning (having similar trends) and hence similar behaviour. Additional tests are then planned if required to delineate boundary between zones more precisely.

DCPT (ISSMFE : Reference test DPH) is quick, (4 persons can execute 3 tests/day), low capital based (250 $ to 270 $) and can be performed by unskilled helpers. The subsoil disturbances due to release of stress, internal piping and lubrication by ground water flow in boring - SPT are eliminated.

For a zone, average change in Nₜ blows/30 cm/meter depth is obtained to evolve the stratification of subsoil broadly classifying cohesive soil (φ = 0), cohesive soil (C - φ), non cohesive soils etc. with appropriate consistency or denseness.

In addition, based on approximate correlations, engineering properties are forecasted. Depending on the type of structure and G.W. table, depth, type of foundation and proportioning is worked out. In this process, critical design parameters for site and project are established. The need or otherwise of ground treatment and suitable technology for ground improvement is analysed to anticipate soil exploration programmes.

Now these approximate critical parameters, properties are to be confirmed or improved to represent field conditions. Atleast 2 - CPT in each zone is conducted to independently evaluate critical factors. The contradictions are listed for checking by atleast 2 bores with specific programme of UDS, vane, SPT etc. for specified depth indicated in evolved profile. The observations of joints/ fissures are taken note of. Thus error of SPT in clay or vane in sandy silt by tender specified intervals for test is eliminated, reducing contradictions.

Final zoning, profile, properties of soil in each stratum that could be safely adopted, variation of ground water, is presented in report.

In case of specific contradictions, open pits for shallow foundation and model or prototype tests - footing/pile load test are recommended. This model establishes improved interpretation of CPT, SPT, DPH each time for given region.

4.0 PENETROMETER TESTS
The IS 4968 (Part-I) code specified a dynamic uncased cone widely used in India. Int. Soc. of Soil Mech. and Foundation Engineering TC-16 has specified reference test for Dynamic Penetrometer Heavy (DPH).

![Fig.1](image)

Fig.1 Comparison of Nₜ vs depth by DPH and DCP (IS).
The parallel tests by both specifications Fig. 1 are comparable within reasonable range (Desai M. D. et al '82).

Specifications of tests of IS code and recommended procedures by TC-16 for static cone (CPT) have been discussed. The SPT test of IS 2131 and recommended by INSSMFE (TC-16) have been deliberated by Pratima Mehta et al 1991. The drilling is mostly shell boring on Auguring upto 5 m to 10 m and mud drilling beyond.

DPH type test has gained wide acceptance for low cost of equipment, little skill required, cheap, very quick and more reliable for subsoils near and below water table.

5.0 CASE STUDY
A project investigation in this region is used to illustrate advantages of revised exploration approach.

For known formation, DPH survey was conducted. A typical overlap of results is shown in Fig. 2. Similar plot of static cone resistance q_c versus depth for same area is shown in Fig. 3. The log of bores showing SPT values along typical profile is shown in Table 2.

6.0 ANALYSIS OF DATA

6.1 Zoning
The area has, by and large, homogeneous profile excluding DPH No. 5. Latter CPT 4 and BH 12 also confirmed exception. Thus plot exclusion area of CPT 4, DPH-5 and BH-12 is grouped as different zone from rest of area.

6.2 Stratification
The results of DPH, Fig. 2 have been used to indicate stratification thickness. The type of subsoil is anticipated on basis of local empirical index R (change in N, per meter depth). R varies from 0 for saturated soil cohesive soil to 4 for normality consolidation clays. For soft over consolidated clays it could be almost 8. The soil is stratified, but behaving as C-q, soil, gives range of R varies from 10-20 and for noncohesive sand it varies from 18 to 35. Local geology and water table gives more precise sub-group. (Pratima et al 89). The stratification, based on local experience was classified as shown in Fig. 2.

The data of CPT is interpreted by \( q_c / q_s \times 100\% \) and use of the chart by

![Fig. 2: Soil profile by DPH.](image)

![Fig. 3: Soil profile by CPT.](image)
Robertson and Campanella. The logs of bore holes and laboratory classification of samples is used for comparison. The result is obvious from Table-1.

Table-1 : Comparison of Stratification.

<table>
<thead>
<tr>
<th>Test Method</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>DPH (IS)</td>
<td>0-1.8</td>
<td>1.8-3*</td>
<td>6.4-12</td>
<td>12-15</td>
</tr>
<tr>
<td>CPT (T, -16)</td>
<td>0-1.8</td>
<td>1.8-5.4</td>
<td>5.4-9.6</td>
<td>9.6-12.6</td>
</tr>
<tr>
<td>Bore - SPT</td>
<td>0-2.0</td>
<td>2.0-4.7</td>
<td>4.7-9.3</td>
<td>9.3-13.5</td>
</tr>
<tr>
<td>Adopt</td>
<td>0.2-6**</td>
<td>2.6-5.2</td>
<td>5.2-9.6</td>
<td>9.6-14</td>
</tr>
</tbody>
</table>

* Open pit 0-2.4 m ** above WT

6.3 Soil type
The type of soil was forecasted by DPH and then compared with predictions by CPT and samples from bore holes. The data is given in Table-2.

The comparison shows poor reliability of predicting stiffness of soil by bores around water table. The difference in stiffness of layer A and B was reconciled by an open pit and moisture profile. Field vane shear and plate load test was used to cross check critical shear parameter (Cv). The prediction of layer by BH for C is incomplete without DPH or CPT.

6.4 Ground water table
The predicted water table by DPH (Pratima et al 1989) is 3.0 m below GL. The CPT do not show depth to water table. In drilling water table was reported to be at 4.5 m when striked and it stabilized at 3.0 m below ground. This hydraulic pressure explains low SPT in layer ‘B’, compared to CPT or DPH.

6.5 Design profile
The design profile adopted is as under after cross checking data.

(A) 0-2.6 m Top weathered clay subject to seasonal moisture changes (Expansive)
(B) 2.6-5.2 m Silty clay saturated (p<10) (N, = 10 to 20, q, = 10) Probably stiff
(C) 5.2-9.6 m Stratified deposit, layers and silty fine sand and thin clay deposits, clay in form of clods or lenses. (C: soil by log but likely to be non cohesive medium to dense state).
(D) 9.6-14 m Very dense sand and gravelly sand. (C - soil).

The layer ‘B’ has critical parameter cohesion and layer ‘C’ being stratified soil, has classification of mixed sample for 300 mm or more which is not truly reflecting field behaviour.

6.6 Engineering properties of each stratum
Designers require the engineering properties of each stratum. The predicted values by use of different techniques makes decision very difficult.

Table-2 : Predicted soil type by penetrometers and bore.

<table>
<thead>
<tr>
<th>Test</th>
<th>DPH</th>
<th>CPT</th>
<th>Bore SPT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer A</td>
<td>Medium moist (April) silty clay MH-CH, weathered (C-p)</td>
<td>Silty clay</td>
<td>Cl-CH Blackish stiff-Sticky sides.</td>
</tr>
<tr>
<td>Layer B</td>
<td>MH-CH cohesive soft silty clay saturated (p&lt;0)</td>
<td>Silty clay (soft)</td>
<td>Cl-CH Brownish very soft silty clay with fine sand.</td>
</tr>
<tr>
<td>Layer C</td>
<td>Stratified layers silts - sands - silts below WT (C-p)</td>
<td>SM-SC-ML (Dense)</td>
<td>Sand fine to med. with some silt &amp; clay (Stratified layers Med. to Dense).</td>
</tr>
<tr>
<td>Layer D</td>
<td>SW-cemented (p) Dense</td>
<td>Sand SW-SM (Very dense)</td>
<td>Cemented sand and silt-compact.</td>
</tr>
</tbody>
</table>
Table 3: Shear resistance of Layer A and B.

<table>
<thead>
<tr>
<th>Layer</th>
<th>C_s kg/cm² by test</th>
<th>DPH*</th>
<th>SPT</th>
<th>CPT**</th>
<th>UCC on U.D.S.</th>
<th>Plate Load test</th>
<th>Consistency</th>
<th>Vane</th>
<th>Sensitivity</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.8</td>
<td>0.7</td>
<td>0.9 to 1.5</td>
<td>0.6 to 0.9</td>
<td>-</td>
<td>0.6</td>
<td>1.6</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>0.6</td>
<td>0.2</td>
<td>0.55</td>
<td>0.3 to 0.5</td>
<td>0.4</td>
<td>0.2</td>
<td>1.5 to 2.4</td>
<td>3</td>
<td></td>
</tr>
</tbody>
</table>

* C_s = (N_s/10), ** For CPT C_s = (q_s - p_o)/17 kg/cm², p_o is overburden pressure.

6.6.1 Layer A The top layer subject to change of moisture seasonally, behaves as C-φ to ϕ = 0 soils. In this cohesive soil, cohesion is critical for design. Assuming near saturation the value of C_s, predicted by different tests, is shown in Table 3.

The layer A due to lower moisture in January records higher N_s or N_p. The C_s assuming ϕ_s = 0 by DPH, SPT, UCC on UDS gave identical values of C_s = 0.8 kg/cm². The less disturbed CPT predicts a higher range, highest value being obtained by in-situ vane shear test in open pit. The C_s being function of moisture content (wide variations is explained by Fig. 4 (B). There is wide variations of natural moisture content evaluated by SPT, UDS and the samples, obtained from core of Shelby (dry drilling). The analysis of disturbance, sensitivity when considered, C_s by vane could be considered as reliable.

6.6.2 Layer B This layer is cohesive but nearly saturated, could be treated as ϕ_s = 0 soil. The wide range of moisture content variation, Fig. 4 (A) is used to understand soil shear resistance. The CPT and DPH gives C_s = 0.55 kg/cm² and SPT is unreliable due to seepage, swelling and sensitivity to boring. The vane shear records C_s = 2.0 kg/cm². This subsoil is critical as the profile gives foundation depth for structures as 2.5 m. The C_s by plate load test is under estimated, as the failure by two tangents is not subsoil failure. The sensitivity analysis of foundation design is based on C_s = 0.6 kg/cm² (F_c = 2) to C_s = 1.5 kg/cm² (F_c = 3). The subsoil 2.4 to 3.0 m is clay and silt clods in fine sand matrix. The soil has higher permeability. The shear parameters for consolidated undrained state shows C_s = 0, q_u = 24.

The variation in moisture at 2.3 m below ground is 46.7% for shell sample, 44% for SPT and 34% for UDS sample.

Fig. 4 Moisture variation with depth (A) Layer B, (B) Layer A.

As cohesive soil having medium consistency, compressibility is still critical factor. The E_s for C_s = 0.7 kg/cm² is 200 kg/cm². By plate load, SPT, DPH and CPT, estimated values are 41, 10,
35, 200 kg/cm². The block UDS sample in oedometer test for stress range 10 to 20 T/m² gave mₚ = 1.28 to 3.5 x 10⁻² cm³/kg. For settlement analysis CPT value needs prototype performance data. Eₜ = 75 kg/cm² is recommended for evaluating settlement. Even cohesion of 1 kg/cm² would give Eₜ = 200 kg/cm² empirically.

Cohesion of 1 kg/cm² and Eₜ = 150 kg/cm² are recommended for design for prototype test footings.

6.6.3 Layer C The layered subsoil is expected to behave as noncohesive fine sand with silt. The CPT, qₜₜ = 800 Kₚₑ, (P₀ = 65 Kₐₜ) predicts relative density of 45% to 60% with φ = 33° to 36°. Both DPH (Nₖ = 20) and SFT at 7 m depth gives Rₙ as 50%. This thus strata could be considered as medium dense sand with Rₙ = 50%, φ = 32°. From CPT, value of Eₜ at σ = 1.0 kg/cm² is 400 kg/cm². The prediction using overburden corrected Nₖ or N₅ₐ gives Eₜ = 200 to 300 kg/cm². The value of Eₜ = 250 kg/cm² is recommended.

6.6.4 Layer D The SW-SM layer has Rₙ = 50% to 60%, φ = 38° by SPT (30-40 blows/30 cm) and qₜₜ = 120 kg/cm². The low water content 12 to 20%, Cₛ = 0.8 kg/cm², qₜₜ = 22° on UDS are very conservative compared to parameters from penetrometers.

6.7 Preliminary foundation design
For economy, time to construct and seasonal field conditions may require alternate foundation systems. Based on DPH test minimum depth for foundation of structures is 1.9 m. The subsoil 1.9 to 3.0 will be low expansive as the moisture content is 85% of equilibrium moisture.

Structural foundations for buildings and plants are planned at depth of 2.7 m below G.L. The subsoil is practically non swelling. The design can be worked out for safe bearing capacity of 22 T/m². The settlement of footings are computed to control total settlement to 65 mm.

Treatment to ground, by sand columns at 100 m c/c 6 m deep, was proposed to release the pressure on GWT. For each foundation, for settlement computation, recommended mₚ = 1.3 x 10⁻² cm³/kg, H = 3.3 m.

For deep foundation, piles driven (cast in situ) could be considered at 6 m to 8 m below G.L. The pile will be a bearing pile.

The SBC for pile and footing at 2.7 m below ground level was to be reaffirmed by prototype tests on piles and footings. For guidance 450 mm φ driven (cast in situ) pile at 13 m depth was expected to give 100 T safe load capacity. 250 mm φ pile, bored, cast in situ resting at 7 m below G.L was to carry a safe load of 24 T.

7.0 CONCLUSION
The case study explains importance of using an approach by which preliminary report is evolved on the basis of DPH carried out extensively. Zoning, profiling, properties of layers and W.T. can be evolved which provides hint of critical problems and types of foundations.

This parameters are rechecked by few CPT tests in each zone and the variable parameters are resolved more specifically. Bores, with SPT in noncohesive soils and vane in cohesive soils under control conditions are then planned to remove remaining ambiguities. The trial design is modified accordingly. The ground treatment and its suitability could be explored simultaneously.

Above exploration narrows down wide range provided in tender specified, routine investigation of boring SPT/vane/UDS at 1.5 m interval. DPH and CPT interpretations confirmed by cross checks will ultimately provide better regional codes for interpretations. Classification of bore hole soil samples in stratified alluvium do not indicate engineering behaviour.

REFERENCES
Bamun of Indian Standards, New Delhi.
IS 4968(II), Sounding Dyn. Cone test 50 mm.
IS 4968 (III), Static cone penetration test.
Soil mechanical properties and in-situ temperature from arctic offshore CPT data

Aleksei Dlugach
Arctic Marine Engineering Geological Expeditions, Murmansk, Russia

Olli Okko
Technical Research Centre of Finland, Espoo, Finland

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Arctic Marine Engineering Geological Expeditions, Murmansk, Russia

Sergei Rokos
Arctic Marine Engineering Geological Expeditions, Murmansk, Russia

SYNOPSIS: The continuous CPT data consisting of peak resistance, skin friction, and temperature readings is acquired with a Fugro-McClelland CPT-system. The "Wixon" CPT system is operated within 3 m long drill string. After each of the individual tests the cone is kept at least 15 min at the final depth to monitor the temperature changes for the evaluation of the formation temperatures with depth in the arctic sea bottom. The soil classification and soil mechanical properties, including modulus of deformation and strength in cohesion and friction angle, are determined according to empirical relationships obtained during the application of the method. As well the undrained shear strength, consistency and liquidity index of clayey soils are estimated.

1. INTRODUCTION

Engineering-geological investigations in the Russian Barents sea are carried out since 1981 by SE AMIGE (Arctic Marine Engineering Geological Expeditions). The offshore investigations have included shallow seismo-acoustic profiling, shallow drilling with soil sampling for laboratory analyses, and penetration testing. The main purpose of these investigations is to obtain engineering-geological data of the quaternary-pliocene stratigraphy in the section down to a few hundred meters below the sea bottom.

In this arctic regime there are several observations on subaquous permafrost as ice crystals and frozen soil sections. However, these samples have been partly melted during the drilling process. In order to improve the possibilities to make laboratory tests of frozen soils, a Finnish-made container, the temperature of which can be controlled, was installed on the drill ship "Barents" during summer 1993. In order to find out the correct testing temperature new CPT-probes with temperature sensors were designed and built at the special design bureau of AMIGE in Riga, Latvia with some instrumental aid from VTT (Technical Research Centre), Finland. In this paper the temperature records will be described as well as the Russian interpretation methods of CPT data.

2. OFFSHORE CPT SYSTEM

The drilling unit of the vessel can be applied with push-in type sampler or with rotary barrel sampler. As well the Fugro-McClelland CPT-system operates in-side of the drilling tubes by pressurizing the Wixon control unit against the tubes and pushing the CPT-cone downwards in 3 m increments. The speed of penetration is 1 m/min and the recording increment in measuring of peak resistance and skin friction is 1 s. The capacity of the system is 50 kN. The CPT-tests were controlled on the vessel and the data obtained was registered and stored with the aid of an HP-85 microcomputer.
The Fugro-McCelland geotechnical system (see Fig. 1) mounted on the "Bavenit" involves:
- downhole system for CPT (Wilson);
- downhole system for push-in sampling;
- seabed frame with hydraulic clamp
- control and measuring desk for recording
- additional equipment (winches, hydraulics, umbilical cable etc.).

The CPT-unit is operated in two phases: after each of the 3 m test intervals the probe is kept at the final sounding depth over 15-20 minutes to allow the temperature at the sensors to cool down to the temperature of the formation. After each of these long period temperature measurements the CPT-tested depth interval was penetrated by the coring system. The temperature sensors were calibrated in the sea water at the sea bottom to find out possible shifts.

3. INTERPRETATION OF CPT-DATA

3.1 Soil properties

The interpretation of CPT-data consists of computing soil mechanical characteristics from the cone resistance and skin friction at the CPT-cone through empirical relationships. The procedure begins by the averaging of all CPT-data over 20 cm intervals before computing the soil parameters:

1. Soil classification is carried out from the ratio between cone resistance and skin friction,
   Table 1:

<table>
<thead>
<tr>
<th>Friction ratio</th>
<th>Soil type</th>
</tr>
</thead>
<tbody>
<tr>
<td>more than 80</td>
<td>sand</td>
</tr>
<tr>
<td>50-80</td>
<td>sandy sil (supes)</td>
</tr>
<tr>
<td>10-50</td>
<td>lean clay (suglinok)</td>
</tr>
<tr>
<td>less than 10</td>
<td>clay</td>
</tr>
</tbody>
</table>

2. If the soil is classified as sand, the compaction of packing is determined from the cone resistance Table 2:

<table>
<thead>
<tr>
<th>Cone resistance q, MPA</th>
<th>Packing compactness</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-3.0</td>
<td>loose</td>
</tr>
<tr>
<td>3.0-14.0</td>
<td>medium</td>
</tr>
<tr>
<td>over 14.0</td>
<td>compact</td>
</tr>
</tbody>
</table>

Figure 1. CPT system of drill ship "Bavenit". Continuous coring and CPT testing are performed in 3 m lengths by a wireline system.

The standard CPT-cones (ISSFME 1977, GOST 1981, SNIP 1987) with a diameter of 36 mm an area of 10 cm² were manufactured either in Holland by Fugro or in Riga, Latvia by Geomaster Ltd., according to ASTM standard D3441. Two cones were provided with temperature sensors AD 590 to give continuous reading on the temperature as well.
3. Modulus of deformation (E) and strength as the angle of internal friction ($\phi$) and cohesion (C) are computed from cone resistance values with relationships determined on a basis of the application of CPT in Baltic States over a long period of time. Okunthov & Fedorov (1988) have published several nomograms on this subject. The computing algorithm (Tables 3-5, equations 1-3) based on this experience is the following:

**Table 3. Derivation of internal friction angle and cohesion for sands.**

<table>
<thead>
<tr>
<th>$q$, MPa</th>
<th>$\phi$, deg</th>
<th>C, kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q &lt; 3$</td>
<td>21.0 + 2q</td>
<td>3.0 $\times 10^{-3}$ q</td>
</tr>
<tr>
<td>3 &lt; $q \leq 11$</td>
<td>25.4 + 0.89q</td>
<td>3.0 $\times 10^{-3}$ q</td>
</tr>
<tr>
<td>11 &lt; $q \leq 18$</td>
<td>28.2 + 0.39q</td>
<td>(1 + 0.16q) $\times 10^{-3}$</td>
</tr>
<tr>
<td>18 &lt; $q \leq 40$</td>
<td>31.2 + 0.24q</td>
<td>(1.5 + 0.16q) $\times 10^{-3}$</td>
</tr>
<tr>
<td>$q &gt; 40$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4. If the soil is classified as clayey (clay, silty clay, sandy silt), undrained shear-strength $S_u = q/20$ as well as the consistency and the liquidity index are also estimated (Tables 6 and 7).

**Table 6. Estimation of consistency.**

<table>
<thead>
<tr>
<th>Cone resistance $q$, MPa</th>
<th>Consistency</th>
<th>Liquidity index, I</th>
</tr>
</thead>
<tbody>
<tr>
<td>less than 0.1</td>
<td>very soft</td>
<td>larger 1</td>
</tr>
<tr>
<td>0.1-0.6</td>
<td>soft</td>
<td>1.0 - 0.75</td>
</tr>
<tr>
<td>0.6-1.2</td>
<td>firm</td>
<td>0.75 - 0.50</td>
</tr>
<tr>
<td>1.2-4.0</td>
<td>stiff</td>
<td>0.50 - 0.25</td>
</tr>
<tr>
<td>4.0-10.0</td>
<td>hard</td>
<td>0.25 - 0.10</td>
</tr>
<tr>
<td>more than 10.0</td>
<td>very hard</td>
<td>less than 0</td>
</tr>
</tbody>
</table>

**Table 7. Estimation of liquidity index.**

<table>
<thead>
<tr>
<th>$q$, MPa</th>
<th>$I_l$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1 &lt; $q \leq 1.2$</td>
<td>1.06 - 0.45q</td>
</tr>
<tr>
<td>1.2 &lt; $q \leq 4$</td>
<td>0.62 - 0.09q</td>
</tr>
<tr>
<td>4 &lt; $q \leq 20$</td>
<td>0.42 - 0.042q</td>
</tr>
</tbody>
</table>

3.2 Interpretation of in-situ temperature measurements

Soil temperature is estimated from the recorded temperature curves. The penetration process warms up the metallic cone body as well as the surrounding soil. The temperature establishment after the tests can be approximated by the exponential law:

$$ T = T_0 + \Delta T e^{-\alpha t} $$

where $T_0$ - undisturbed soil temperature,

$\Delta T$ - effect of heating at the time $t=0$,

$\alpha$ - parameter, determining rate of the stabilization,

$T$ - measured temperature at time $t$.

The problem in estimating the $T_0$-value deals with the selection of $\Delta T$-value, that most closely correspond to the experimental data.

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The procedure in the analysis of the obtained temperature monitoring records consists of:
- differentiation of initial time series for the determination of "constant component" T₀;
- logarithmisation of obtained series;
- determination of ΔT and α by least mean square method;
- the selection of the best fit for T₀ by shifting derived exponent along temperature axis and by estimating with least mean square criteria, this quantity is taken as prognosed soil temperature.

Depending on the penetration friction (soil type), the recording of representative series requires observation over 10-20 min after the end of the penetration.

According to comparative tests carried out in the temperature range -1.2...+4.9 °C with precision mercury thermometer, the error in the analysis of the absolute accuracy in the estimation of natural soil temperature is 0.23°C, while the error in the single readings is 0.05 °C.

4. EXAMPLES FROM IN-SITU TESTS

The interpretation charts for CPT-test are derived from several comparisons between several in-situ and laboratory methods. The Russian soil classification system GOST 25100-82 is based on the Atterberg’s limit, the plasticity indices, thus there are these laboratory tests available for comparison. As well there are and triaxial laboratory tests and micropenetrometer measurements along the core samples. The laboratory tests and micropenetration tests can be compared (Table 8) through Russian and American standards.

<table>
<thead>
<tr>
<th>N</th>
<th>Russian GOST (laboratory)</th>
<th>ASTM (micropenetration)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>turgoplastic-chinay 0.25&lt;m&lt;0.05</td>
<td>very stiff (hard) t=200±220 kPa</td>
</tr>
<tr>
<td>2</td>
<td>polutverday 0&lt;m&lt;0.25</td>
<td>hard (very stiff) t=160(200),400(500) kPa</td>
</tr>
<tr>
<td>3</td>
<td>0&lt;m&lt;0</td>
<td>very hard t=420 kPa</td>
</tr>
</tbody>
</table>

As an example the curves of cone resistance and sleeve friction during one of 3 m long penetration increment obtained close to Matveev Island in Pechora Sea are in Fig. 2. The calculation of the corresponding geotechnical parameters is in the Table 9.

There are also micropenetration measurement of t > 300 kPa at the depth of 33.5 m, which characterize the soil to be hard. The laboratory analyses for grain size and Atterberg’s limits classify the soil as suglinok (lean clay or loam), like the friction ratio. The plasticity index derived from laboratory test is very small, less than 0.05, even negative (tverday) while the CPT indicates some plasticity (0.10-0.12). The oedometer and triaxial tests on the suglinok (loam) unit show stiff character of this soil unit in the sequence.

After each of the penetration increments the temperature was recorded separately. Typically the minimum temperatures are obtained at the depth interval of 30 - 75 m. As an example of the estimation of the in-situ temperature the record with the fitted temperature stabilization curve at the depth of 34 m below the sea bottom is shown in Fig. 3. In general the root-mean square deviation of experimental points from approximating curve and transition time to temperature differs from prognosed soil temperature less than 0.2 °C.
Table 9. Interpretation of mechanical soil properties from CPT data.

<table>
<thead>
<tr>
<th>Depth, m</th>
<th>Lithology</th>
<th>q, MPa</th>
<th>f, MPa</th>
<th>q/T</th>
<th>E, MPa</th>
<th>q, deg</th>
<th>C, kPa</th>
<th>S, kPa</th>
<th>I_p/L_d</th>
</tr>
</thead>
<tbody>
<tr>
<td>32.0</td>
<td>clay</td>
<td>1.04</td>
<td>2.093</td>
<td>0.25</td>
<td>6.06</td>
<td>11</td>
<td>19</td>
<td>52.3</td>
<td>0.58</td>
</tr>
<tr>
<td>32.2</td>
<td>sand</td>
<td>7.67</td>
<td>0.086</td>
<td>89</td>
<td>16</td>
<td>20</td>
<td>2</td>
<td>32.7</td>
<td>0.58</td>
</tr>
<tr>
<td>32.4</td>
<td>clay</td>
<td>8.6</td>
<td>0.243</td>
<td>35</td>
<td>49.9</td>
<td>25</td>
<td>76</td>
<td>410.2</td>
<td>0.05</td>
</tr>
<tr>
<td>32.6</td>
<td></td>
<td>1.0</td>
<td>0.216</td>
<td>46</td>
<td>58</td>
<td>29</td>
<td>87</td>
<td>500</td>
<td>0</td>
</tr>
<tr>
<td>32.8</td>
<td></td>
<td>8.02</td>
<td>0.233</td>
<td>34</td>
<td>46.5</td>
<td>25</td>
<td>72</td>
<td>401.1</td>
<td>0.08</td>
</tr>
<tr>
<td>33.0</td>
<td></td>
<td>7.61</td>
<td>0.277</td>
<td>27</td>
<td>44.1</td>
<td>25</td>
<td>69</td>
<td>380.8</td>
<td>0.1</td>
</tr>
<tr>
<td>33.2</td>
<td></td>
<td>7.03</td>
<td>0.294</td>
<td>23</td>
<td>40.8</td>
<td>25</td>
<td>64</td>
<td>351.7</td>
<td>0.12</td>
</tr>
<tr>
<td>33.4</td>
<td></td>
<td>7.61</td>
<td>0.198</td>
<td>38</td>
<td>44.1</td>
<td>25</td>
<td>69</td>
<td>380.8</td>
<td>0.1</td>
</tr>
<tr>
<td>33.6</td>
<td></td>
<td>7.44</td>
<td>0.343</td>
<td>21</td>
<td>43.6</td>
<td>27</td>
<td>68</td>
<td>372.1</td>
<td>0.1</td>
</tr>
<tr>
<td>33.8</td>
<td></td>
<td>7.2</td>
<td>0.344</td>
<td>20</td>
<td>41.8</td>
<td>26</td>
<td>66</td>
<td>360.4</td>
<td>0.13</td>
</tr>
<tr>
<td>34.0</td>
<td></td>
<td>6.91</td>
<td>0.379</td>
<td>17</td>
<td>40.2</td>
<td>25</td>
<td>64</td>
<td>345.9</td>
<td>0.12</td>
</tr>
<tr>
<td>34.2</td>
<td></td>
<td>6.74</td>
<td>0.467</td>
<td>14</td>
<td>39.1</td>
<td>25</td>
<td>62</td>
<td>337.2</td>
<td>0.13</td>
</tr>
<tr>
<td>34.4</td>
<td>clay</td>
<td>6.97</td>
<td>0.82</td>
<td>8</td>
<td>40.4</td>
<td>25</td>
<td>64</td>
<td>348.8</td>
<td>0.12</td>
</tr>
<tr>
<td>34.6</td>
<td></td>
<td>6.91</td>
<td>2.52</td>
<td>2</td>
<td>40.2</td>
<td>25</td>
<td>64</td>
<td>345.9</td>
<td>0.12</td>
</tr>
</tbody>
</table>

SUMMARY

The offshore CPT system used in the characterization of the overlying soil sequence of the Barents Sea can be considered a reliable tool when acquiring in-situ mechanical data on the sediments. The comparisons between different tests indicate that the calculation models may be improved when more statistical data is obtained.

The method of thermologging after each of the CPT increments gives the profile of in-situ temperatures. However, the effect of heating due to penetration has not yet been analyzed and included in the analyses of the mechanical soil properties.

ACKNOWLEDGEMENT

The Finnish-Russian Offshore Technology Working Group is highly acknowledged for the permission to present this paper.
REFERENCES


Statistical correlations between $V_s$ and cone penetration data for different soil types

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Paul W. Mayne
Georgia Institute of Technology, Atlanta, GA, USA

SYNOPSIS: Cone penetration and shear wave velocity ($V_s$) data are collected from 61 sites worldwide representing different types of clays, sands, intermediate soils and mine tailings. Previous correlations relating $V_s$ to cone tip resistance ($q_c$) in clean quartz sands and $V_s$ - $q_c$ relationships for natural clays are reviewed. New correlations based on multiple regression analyses are proposed for specific soil types (i.e., sands, clays), as well as a more global relationship where $V_s$ is expressed as a function of both $q_c$ and sleeve friction ($f_s$) that is applicable to all soil types.

1. INTRODUCTION
Estimation of the shear wave velocity and the low-strain tangent shear modulus ($G_{tan}$) as fundamental soil properties has held the interest of many researchers for the last two decades. Based on the results of resonant column tests on sands and clays, Hardin (1978) and Vucetic and Doby (1991) suggested that $G_{tan}$ is dependent on void ratio ($e_v$), mean effective stress ($\sigma_{eff}$), over-consolidation ratio (OCR), and plasticity index (PI). Direct measurement of $V_s$ is always preferred, however not always possible.

Different empirical expressions have been developed to correlate cone penetration data with low-strain elastic soil properties. Baldi et al. (1989) estimated $V_s$ in Italian sands as a function of the cone tip resistance ($q_c$) and the effective overburden stress ($\sigma_{eff}$). This study was modified by Rix and Stokoe (1991) who suggested a wider normalized range of $G_{tan}/q_c$ and $q_c/\sigma_{eff}$. Additional studies were performed to correlate $V_s$ with $q_c$ in specific clay soils. Most recently, Mayne and Rix (1993, 1995) developed a correlation for different clays worldwide and suggested that $q_c$ can reflect the effect of $\sigma_{tan}$ and OCR on $G_{tan}$ or $V_s$ and obtained an expression in terms of $q_c$ and $e_v$ as independent parameters.

The purpose of this study is to improve the estimation of $V_s$ in sand and clay soils and to develop a correlation to estimate $V_s$ independent of soil type.

2. DATA BASE
Cone penetration data and $V_s$ data are collected from 61 sites with different soil types. A listing of 24 sands, 5 clays, and one mine tailing sites is given in Table 1. The other 31 clay sites are listed in Mayne and Rix (1993). The shear wave velocity data were measured using different in situ techniques including either seismic cone (SCPT), crosshole (CHT), downhole (DHT), or spectral analysis of surface wave (SASW) tests.

3. REGRESSIONS FOR CLAY SOILS
The effects of four independent parameters including $q_c$, sleeve friction ($f_s$), $e_v$, and $\sigma_{out}$ collected from 36 different clay sites are studied to explain the variability of the dependent variable $V_s$. Simple regression analyses were performed for each independent parameter with $V_s$. The most significant parameters are $q_c$ and $e_v$ as suggested by Mayne and Rix (1993). Multiple regression analyses were performed with all possible combinations of the independent parameters indicating ($n = 406; r^2 = 0.885$):

Clays: $V_s = 14.13 q_c^{0.339} e_v^{-0.473}$ (1)
Table 1. Database listing of 30 sites with different soil types.

<table>
<thead>
<tr>
<th>Site/Location</th>
<th>Soil Type</th>
<th>In-Situ Tests</th>
<th>References/Sources</th>
</tr>
</thead>
<tbody>
<tr>
<td>200 St, BC</td>
<td>Soft clayey silt</td>
<td>CHT, DHT, PCPT</td>
<td>Sully and Campanella (1995)</td>
</tr>
<tr>
<td>Lower 232 St, BC</td>
<td>Soft clay</td>
<td>CHT, DHT, PCPT</td>
<td>Sully and Campanella (1995)</td>
</tr>
<tr>
<td>Anacoxie Site, BC</td>
<td>Clean sand</td>
<td>SCPT</td>
<td>Robertson et al. (1986)</td>
</tr>
<tr>
<td>Bagdad, AZ</td>
<td>Mine tailings</td>
<td>SCPT</td>
<td>Mayne et al. (1994)</td>
</tr>
<tr>
<td>Broadway Site, CA</td>
<td>Clean sand</td>
<td>SCPT</td>
<td>Chameau et al. (1991)</td>
</tr>
<tr>
<td>Douglas Lake, Michigan</td>
<td>Clean sand</td>
<td>SCPT</td>
<td>Thomann and Hryciw (1992)</td>
</tr>
<tr>
<td>Georgia Tech, Atlanta</td>
<td>Silty sand</td>
<td>SASW, CPT</td>
<td>Mayne and Harris (1993)</td>
</tr>
<tr>
<td>Haber Road Site, CA</td>
<td>Clean sand</td>
<td>CHT, PCPT</td>
<td>Rix (1984)</td>
</tr>
<tr>
<td>Hollywood Canal, SC</td>
<td>Clean sand</td>
<td>SASW, PCPT</td>
<td>Martin and Clough (1990)</td>
</tr>
<tr>
<td>Holmen Site, Norway</td>
<td>Clean sand</td>
<td>SCPT</td>
<td>Lunne et al. (1986)</td>
</tr>
<tr>
<td>Hunter's Point, CA</td>
<td>Clean sand</td>
<td>CHT, PCPT</td>
<td>Chameau et al. (1991)</td>
</tr>
<tr>
<td>Kettner Plaza, CA</td>
<td>Silty sand</td>
<td>SCPT</td>
<td>Spang (1988)</td>
</tr>
<tr>
<td>Laing Bridge South, BC</td>
<td>Clean sand</td>
<td>SCPT</td>
<td>Sully (1991)</td>
</tr>
<tr>
<td>NE Atlanta, GA</td>
<td>Silty sand</td>
<td>SCPT</td>
<td>Burns et al. (1995)</td>
</tr>
<tr>
<td>Major Power Plant, Utah</td>
<td>Sand to silty sand</td>
<td>DIT, CITT, PCPT</td>
<td>Konstantinovski et al. (1986)</td>
</tr>
<tr>
<td>McDonald's Farm, BC</td>
<td>Clean sand</td>
<td>SCPT</td>
<td>Robertson at al. (1986)</td>
</tr>
<tr>
<td>Montague Avenue, SC</td>
<td>Clean sand</td>
<td>SASW, PCPT</td>
<td>Martin and Clough (1990)</td>
</tr>
<tr>
<td>Mt. Pleasant Pits, SC</td>
<td>Clean sand</td>
<td>SASW, PCPT</td>
<td>Martin and Clough (1990)</td>
</tr>
<tr>
<td>Oakland Plantation, SC</td>
<td>Clean sand</td>
<td>SASW, PCPT</td>
<td>Martin and Clough (1990)</td>
</tr>
<tr>
<td>Po River, Italy</td>
<td>Clean sand</td>
<td>SCPT</td>
<td>Baldi et al. (1988)</td>
</tr>
<tr>
<td>Savannah, GA</td>
<td>Soft silty clay</td>
<td>SCPT</td>
<td>Mayne and Burns (1995)</td>
</tr>
<tr>
<td>Sod Farm, SC</td>
<td>Clean sand</td>
<td>SASW, PCPT</td>
<td>Martin and Clough (1990)</td>
</tr>
<tr>
<td>St. Alhan, Quebec</td>
<td>Soft clay</td>
<td>SASW, PCPT</td>
<td>Lefebvre et al. (1994)</td>
</tr>
<tr>
<td>Strong Pk, Canada</td>
<td>Clayey silt</td>
<td>CHT, DHT</td>
<td>Sully and Campanella (1995)</td>
</tr>
<tr>
<td>Telegram Hill, CA</td>
<td>Clean sand</td>
<td>SCPT</td>
<td>Chameau et al. (1991)</td>
</tr>
<tr>
<td>Ten Mile Hill, SC</td>
<td>Clean sand</td>
<td>SASW, PCPT</td>
<td>Martin and Clough (1990)</td>
</tr>
<tr>
<td>Texas A&amp;M, TX</td>
<td>Sand to silty sand</td>
<td>CHT, PCPT</td>
<td>Briaud and Gibbens (1994)</td>
</tr>
<tr>
<td>Treasure Island, CA</td>
<td>Sand to silty sand</td>
<td>CHT, PCPT</td>
<td>Pass (1994)</td>
</tr>
<tr>
<td>Wildlife Site, CA</td>
<td>Silty sand</td>
<td>SCPT</td>
<td>Robertson et al. (1986)</td>
</tr>
<tr>
<td>Yerba Buena Cove, CA</td>
<td>Clean sand</td>
<td>SCPT</td>
<td>Chameau et al. (1991)</td>
</tr>
</tbody>
</table>

Notes:

- SCPT = seismic cone penetration test
- PCPT = piezocone test
- DHT = downhole test
- CHT = crosshole test
- SASW = spectral analysis of surface wave

where \( V_s \) is in m/s and \( q_u \) is in kPa. The gain in the coefficient of determination (\( r^2 \)) is not significant by adding \( k_f \) and/or \( \sigma_{vc} \) as additional independent parameters in the multiple regression analysis. In addition, another proposed equation which is independent on \( k_f \) and dependent only on cone data including \( q_u \) and \( k_f \) is considered as follows (\( n = 229; r^2 = 0.778 \)):

\[
V_s = 3.18q_u^{0.29}k_f^{0.023}
\]  

(2)

where \( V_s \) is in m/s and \( q_u \) and \( k_f \) are in kPa. The advantage of (2) over (1) is that \( k_f \) is normally not known beforehand. The measured and predicted values of \( V_s \) using equation 1 and 2 are compared for the clay at Lower 232 St site. Figure 1 illustrates the results and also the predictions by Mayne and Rix (1993). Their estimates and the predictions using equations 1 and 2 are in a good agreement with the measured values except at depths less than 3 m and greater than 23 m. The
predictions by equation 2 are, in general, lower than the other predictions and have approximately the same trend of the measured values with depth. Equation 2 is still preliminary and needs more verification by being applied at different clay sites.

Fig. 1. Comparison between measured and predicted $V_s$ for the clay at Lower 232 St. site. (data from Sully 1995)

4. REGRESSIONS FOR SAND SOILS

Simple and multiple regression analyses were performed with $V_s$ as the dependent parameter and three independent parameters including $q_u$, $\phi$, and $\sigma_{v0}$ collected from 24 different sand sites. Both regression types indicate that $q_u$, $\phi$, and $\sigma_{v0}$ are the most significant regressors and the following correlation is proposed ($n = 133; r^2 = 0.684$):

Sands: $V_s = 13.18q_u^{0.392}\sigma_{v0}^{-0.579}$ (3)

where $V_s$ is in m/s and $q_u$ and $\sigma_{v0}$ are in kPa. Another suggested formulation to estimate $V_s$ depending on $q_u$ and $\phi$, is as follows ($n = 92; r^2 = 0.574$):

Sands: $V_s = 12.02q_u^{0.513}\phi^{-0.8465}$ (4)

where $V_s$ is in m/s and $q_u$ and $\phi$ are in kPa. Equation 4 does not depend on the index $e$, which is difficult to obtain in natural clean sands. Figure 2 illustrates the collected data plotted with $G_m$ versus $q_u\sqrt{\sigma_{v0}}$ and validates that about 80% of the collected data are within the ranges obtained by Baldi et al. (1989) and Rix and Stokoe (1991), about 20% of them are lower than those ranges and the average trends obtained are also shown. The Holmen sand site was chosen to verify equations 3 and 4, as shown in Fig. 3, indicating that the predicted values of $V_s$ compare well with the measured values except at the shallowest depths.

5. ANALYSES FOR ALL SOIL TYPES

Data are compiled together from the 64 sites including clays, sands, intermediate soil types and mine tailings. Three independent parameters are considered for all soils including $q_u$, $\phi$, and $\sigma_{v0}$ since $e$ is not directly available for clean sands. A statistical formulation in a linear form was suggested by Mayne and Burns (1995) as follows ($n = 144; r^2 = 0.776$):

All: $V_s = 117+1.33q_u$ (5)

where $V_s$ is in m/s and $q_u$ is in kPa. The simple and multiple-fitted expressions for $V_s$ and the three
regressors indicated that equation 5 did not hold for a larger number of data set and $q_s$ is the most significant parameter. However, $f$, $c$, and $m_w$ are almost equally important. The following is the suggested statistical expression from multiple regression analyses ($n = 323$, $r^2 = 0.695$):

$$V_s = (10.1 \log_{10}q_s - 11.4)^{1.07}(Lq_s + 100)^{0.83}$$  (6)

where $V_s$ is in m/s and $q_s$ and $f$ are in kPa. Figure 4 shows the trend between the measured and the predicted values of $V_s$ using equation 6. A mine tailings site (not in the data base) at San Manuel, AZ was chosen to cross-check equation 6. Figure 5 indicates a good agreement between the measured in-situ and estimated values of $V_s$ except at the shallowest depths.

6. CONCLUSIONS

In this research study, possible correlations between $V_s$ and cone penetration data are studied for different soil types. The proposed statistical expressions indicating the variability of $V_s$ in correlative relationships are shown in Table 2. The most reliable method to obtain the shear wave velocity is the direct in-situ measurement (e.g., seismic cone), as the statistical regression formulations cannot explain all the variability associated with natural soil deposits.
Table 2. Proposed expressions between $V_s$ and cone penetration data for different types of soil.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Proposed Correlation</th>
<th>n</th>
<th>$r^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clays</td>
<td>$V_s = 14.12q_u^{0.259} \cdot 0.443$</td>
<td>406</td>
<td>0.885</td>
</tr>
<tr>
<td></td>
<td>$V_s = 3.18q_u^{0.546} \cdot 0.623$</td>
<td>229</td>
<td>0.778</td>
</tr>
<tr>
<td>Sands</td>
<td>$V_s = 13.18q_u^{0.502}\cdot 0.179$</td>
<td>133</td>
<td>0.684</td>
</tr>
<tr>
<td></td>
<td>$V_s = 12.02q_u^{0.393} \cdot 0.0466$</td>
<td>92</td>
<td>0.574</td>
</tr>
<tr>
<td>All</td>
<td>$V_s = (10.1 \log_6 - 11.4) r_{c}^{0.46}(\zeta/500)^{0.53}$</td>
<td>323</td>
<td>0.695</td>
</tr>
</tbody>
</table>

Notes:
- $V_s$ given in m/s, $q_u$, $f_c$, and $\sigma_{cr}$ are in kPa.

7. REFERENCES


The progress of the method of static sounding in the investigation of geotechnical properties of frozen soils

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ABSTRACT: The results of experimental and theoretical studies of static sounding to estimate geotechnical properties of frozen and frost soils are given in the paper. The main results were received resulted from in-situ experimental studies by using the equipment of static sounding S-632M that makes it possible to control automatically soil resistance on recorders under the penetrometer cone and along the coupling of friction. Different constructions of sounds were designed; they were equipped with thermal sensors located in the cone and along lateral surface of sound in order to study thermophysical interaction of sound and frozen soil. The tests were conducted according to the technology proposed by authors with "stabilization" of sound together with freezing it into the frozen and frost soil. The sounding was made on 10 experimental sites under different engineering and geological conditions. Parallel to static sounding temperature of the soil was controlled in thermal hole and numerous tests of frozen soil were carried out by standard method used in the USSR (static tests of piles, compression tests and tests by a sphere plate). The comparison analysis of sounding data and data of geotechnical properties of frozen soils was made by the methods of mathematical statistics. As a result of these investigations the peculiarities and regularities of mechanical and thermophysical interaction of sound and frozen soil depending on the main technological, engineering and geological conditions (rate of sinking time of it's "stabilization", temperature, general humidity and number of soil plasticity) were ascertained. The technique of soil state estimation (thawed or frozen) was developed. It was based on using "stabilized" and "high-speed" resistances of soil under penetrometer cone. The technique of soil temperature control while sounding was developed. Empirical dependences enabling to estimate equivalent cohesion and deformation modulus of frozen soils were proposed as well as the technique of determining bearing capacity of driven and bored and driven piles using the sounding data.

The increasing rates of the construction in the northern parts of the country require every kind of reduction of the expensive cycle "investigations - design - construction".设计 and investigation works are the essential part of it. It is difficult to fulfill this task using only in-situ and laboratory methods of engineering and geocryological investigations. The in-situ methods of determination of geotechnical properties of soils in permafrost are labour-consuming expensive and prolonged. In laboratory investigations there are some additional difficulties: the necessity of refrigerators, the complexity of choice and safety of monoliths during their transportation at large distances and so on. That's why the designers usually use calculation values of...
characteristics of frozen soils given in the tables SNIP 2.02.04-88. However these characteristics are given for the whole territory of the USSR and they don’t include the regional peculiarities.

Thus, the present most important problem of foundation engineering in the permafrost is the development of quick and reliable enough in-situ express-methods of soils investigations.

Static sounding widely used in usual unfrozen soils is the most progressive one. However it’s use in frozen soils requires the solving of a whole number of scientific and technical problems in working out of optimum technology of static sounding and methods of using the data which would be able to include the peculiarities of frozen soils.

The first investigations carried out in our country [1,2,3,4,5] and abroad [6,7] showed firstly the principle technical opportunity of static sounding of permafrost soils by the existing technical means. Secondly, they revealed the prospect and direction of using static sounding for estimating of geotechnical properties of frozen soils. The complex of experimental and theoretical investigations was done to carry out the practical methods of using sounding in engineering and geocryological investigations. The main results of the work have been obtained in in-situ experimental investigations carried out with the help of highly production self-moving plant for static sounding. The maximum depth of the soundings was 20 m. As a rule the average sounding was made to depths of 6-10 m. Soil resistance was measured by the recordings of instrumentation measuring apparatus, to ensure the possibility of measuring very small displacement of the deflectometer. Bench marks were used in addition. The improved construction of the thermometrical penetrometer equipped with thermal sensors was developed in order to control the temperature of the penetrometer on the base of the apparatus.

In addition, the thermometrical penetrometer that would be able to control temperature not only in the cone but also along the lateral surface of penetrometer shaft was developed. The sounding was carried out in more than one hundred places on 10 experimental sites under different engineering and geological conditions including the towns Vorkuta and Labytnangi. Parallel to sounding, temperature of the soils was controlled in thermometrical hole. The statistic processing of the results of investigations was made by the methods of statistic analysis on a personal computer using the programme REGAN. The comparison analysis of sounding data and data of physical and mechanical properties and characteristics of ultimate long-term resistances of frozen soils to pile foundation was made using materials. Other design-investigation organizations conducted tests including more than 200 tests by a sphere plate method by N.A. Tsyryutich [8], more than 80 unconfined compression tests and 34 static loading tests of piles. Experimental investigations were made in Vorkuta, Labytnangi which are situated in plastic frozen soils on the most typical sites from the point of view of geology. The soils of the sites consisted of frozen silty and clayey soils or the whole but included some gravel (5-20%). Temperatures in general were from 0 to -1 °C, but sometimes as low as -2 °C. The static sounding of permafrost soils was made by using the equipment 8-82M with “stabilization” of the penetrometer [8].

During investigations three types of soil resistances under the penetrometer cone “q1” and along the coupling of friction “f” [fig.1.] were controlled and measured. These included “high speed” “q1,f” (uniform press) “stabilized” “q1, f” (in stable condition of system sounding) and “peak” “q1,f” (in the initial moment of sounding pressing after it’s freezing). During the investigations the influences of technological and geocryological factors on frozen soil resistance under the penetrometer cone and along the coupling during sinking and “stabilization” of the
penetrometer were determined. The speed of penetrometer sinking $s$ and time of "stabilization" $t_s$ were technological factors. The plasticity index of the soil $I_p$, temperature of the soil $T_b$, summary soil water content $W_c$ were geophysical ones.

The investigation of the change of frozen soils resistances under the cone $q_s$ with the change of the speed of the penetrometer $s$ showed that in the interval $10^{-6} \ldots 1$ m/min the relationship between parameters $q_s$ and $s$ can be described by a continuous power curve (without points of bends). The index of power practically doesn't depend on soil temperature changes in narrow range, it is 0.1 for plastic frozen silty and clayey soils. At the same time this relationship is very weak for the soil resistance along the coupling of friction $f$. However we can point out the following regularities with the increase of speed, the magnitude of the resistance exceeds the maximum caused by the drop of soil temperature. After stopping the penetrometer in the process of "stabilization", the process of intensive penetrometer freezing into soil starts.

![Diagram](image)

Fig.1. Typical diagrams of permafrost soils sounding
a) soil resistance $q_s$ under the penetrometer cone
b) soil resistance $f$ along the coupling of friction

Within about 10 minutes the "peak" resistances $f-s$ of frozen soil along the friction coupling reach 90-95% of the total value for the maximum corresponding to total freezing. Multifactors statistical analysis done by personal computer helped to determine that soil temperature $T_b$ had more potent influence on the indices of frozen soils resistances to sounding. Especially it concerns
the "peak" soil resistances \( f' \). Plasticity number \( Ip \) and summary soil water content \( Wc \) have a weak influence (table 1). The sounding near thermometrical holes showed that ratios of "high-speed" frontal \( q^* \) to "stabilized" \( q_\text{f} \) for thawed and frozen soils differ greatly. In the thawed soils the average ratio is 1.4 but in the frozen soils it is 3.0. More exactly, soils can be divided by using two parameters: "high-speed" resistance and it's ratio to "stabilized" \( q^* \) / \( q_\text{f} \) (fig. 2).

where \( Bc \) and \( n_c \) are empirical coefficients determined experimentally by comparing test results of soils from static sounding with those of sphere plate tests. For silty and clayey soils of glacial complex of Vorotna district, it was determined that \( Bc=0.0031 \) and \( n_c=1.48 \). More than 200 tests by a sphere plate were used to obtain a selective correlation ratio \( R^2 = 0.87 \).

2) Compressive modulus of deformation

\[
E = Bc q^*(q^*_1)
\]

where \( Bc \) and \( n_c \) are empirical coefficients determined experimentally as a result of comparison tests for soils by static sounding and in laboratory unconfining compressive tests. For silty and clayey soils of glacial complex of Vorotna district, values were determined to be \( Bc=7.36 \), \( n_c=0.49 \). More than 80 compressive tests were used to obtain a selective correlation ratio for \( R^2 = 0.75 \).

To investigate thermophysical interactions of the penetrometer with frozen soil temphothermometrical and thermometrical soundings were performed. Static soundings of

<table>
<thead>
<tr>
<th>Geo-cryological factors:</th>
<th>0.50* (81)</th>
<th>0.46* (81)</th>
<th>0.47* (37)</th>
<th>0.53* (33)</th>
<th>0.60 (35)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f' )</td>
<td>-0.49*</td>
<td>-0.16</td>
<td>-0.20*</td>
<td>-0.21</td>
<td>-0.10</td>
</tr>
</tbody>
</table>

Comment: In the numerator there are values of correlation ratios, in brackets there are volume of extract and linear connection with \( \rho = 0.95 ** \)

On the base of comparison experimental investigations close correlation dependences between "high-speed" resistances \( f' \) of frozen soil under penetrometer cone \( q^* \) and characteristics of strength (\( C\)) and deformation \( (E) \) were ascertained. As a result of regressive analysis the following mathematical models were developed, and it's possible to estimate:

1) Ultimate long-term value of equivalent cohesion

\[
C\text{el} = Bc (q^*_1)
\]

2) in the denominator there are values of correlation coefficients and significant linear correlation connection with \( \rho = 0.95 ** \).
the soils were done with penetrometer "stabilization" near thermostatical holes.

To describe thermal interaction of the system "sound-frozen soil" - a thermophysical model based on two mutually opposed thermal flows that occur during the sinking of the penetrometer into "sound-frozen soil", was suggested. The first flow is the heat absorbed by the penetrometer as a result of friction. The second is the returned heat by the penetrometer to the soil-ice as a result of drop of the temperature of ice melting in high pressure. Depending on correlation of these heat flows the penetrometer temperature can rise or fall. To estimate approximately thickness of thawed zone around the penetrometer cone and friction coupling during it's sinking, the theoretical dependences taking into account thermophysical properties of penetrometer and frozen soil, geometrical dimensions of penetrometer, speed of it's sinking, soil resistance to sounding change of penetrometer temperature were done.

The results of experimental investigations confirmed the possibility of using our model. It was mentioned that in plastic frozen soil during the sinking of the penetrometer it cooled down and in strong frozen soil it warmed up. It was pointed out that in the last case the degree of warming up of the penetrometer was in proportion to fall of the penetrometer temperature.

The comparison of temperature curves and data from thermostatical
of the glacial complex of Vorcuta district the following values of the coefficients were found: 
\[ B_o = 0.87, \quad \eta = 0.54, \quad R = 0.93 \]

It is recommended to determine ultimate long-term frozen soil resistances required to overcome friction \( f_p \) along the surface of pile in frozen ground, that the "peak" soil resistances \( f'_p \) corresponding to total penetrometer frozen into the soil be used. The value \( f'_p \) takes into account forces of freezing to the greatest degree. As a result of statistic analysis of comparable data the empirical dependences for two cases of pile loading were established:

- one, without a predrilled hole

\[ f_p = B_0 x (f'_p) \quad (1) \]

- second, a predrilled hole with the area of cross-section equal to the area of cross-section of pile

\[ f_p = B_1 x (f'_p) \quad (2) \]

where \( B_0, B_1, \eta \) are empirical coefficients determined experimentally as a result of comparing tests of frozen soils by static sounding and tests of pile by static loading. For silty and clayey soils of the glacial complex of the Vorcuta district, the following values of the coefficients were determined:

\[ B_0 = 0.95, \quad \eta_1 = 1.56, \quad B_1 = 0.45, \quad \eta_2 = 1.38 \]

Selective correlational ratios were \( R = 0.94 \) and 0.98 residual dispersions were

\[ 5 \quad 4 \]

To determine resistances of frozen soils to the displacement along the surface of a frozen pile with different correlations of geometrical dimensions of pile and predrilled hole based on the known theoretical P.K. Lapshin's formula (about correlation of radial earth pressures on lateral surface of driven and bored piles) the following dependence is suggested:

\[ f_p = \left( 1 - \frac{1}{2} \right) x (f_p - f_p) + f_p \quad (4) \]

\[ f_p = \left( 1 - \frac{1}{3} \right) x (f_p - f_p) + f_p \quad (5) \]

\[ f_p = \left( 1 - \frac{1}{4} \right) x (f_p - f_p) + f_p \quad (6) \]

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where \( A_p \) and \( A_b \) are areas of cross-section of pile and predrilled hole, \( \tan (45 - \phi / 2) \) is the coefficient of lateral pressure of frozen soil, \( \phi \) is angle of internal friction of frozen soil.

\[
(1)
\]

\( f_p \) and \( f_e \) are the same as in formulas (4) and (5). Taking into consideration that in frozen silt and clayey soils the long angle of internal friction is approximately \( 6^\circ \), then the coefficient of lateral pressure can be \( 0.77 \) for practical use in formula (6).

The comparison of standard ultimate long-term resistances of piles, obtained by using the results of static pile tests with calculated resistances based on the data from soundings and the tables SNiP 2.02.04-86, preferences for using the static sounding method was established (table 2).

Table 2. The comparison of standard ultimate long-term resistances (F) of piles with those calculated using the data from static sounding (Fs) and tables SNiP 2.02.04-88 (Ft)

<table>
<thead>
<tr>
<th>Statistic indices</th>
<th>Value of indices</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suggested method</td>
<td>Value from SNiP 2.02.04-88</td>
</tr>
<tr>
<td>Volume of extract</td>
<td>25</td>
</tr>
<tr>
<td>Selective coefficient of correlation</td>
<td>0.94</td>
</tr>
<tr>
<td>Middle value of ratios ( F_p/Ft )</td>
<td>1.03</td>
</tr>
<tr>
<td>Middle quadratic deflection of ratios ( F_p/Ft )</td>
<td>0.24</td>
</tr>
<tr>
<td>Maximum relative error in determination ( F ) % to overestimating</td>
<td>55</td>
</tr>
<tr>
<td>% to underestimating</td>
<td>32</td>
</tr>
</tbody>
</table>

The investigations which have been carried out made it possible to work out practical recommendations in using static sounding mainly in frozen soils. They include 4 sections: general clauses, preparation and carrying out of tests, estimation of condition and property of soils determination of bearing capacity of pile. Static sounding is recommended to carry out in conjunction with traditional methods of engineering and geocryological investigations using the equipment S-832M, BP-72... equipped with tennothermometrical and tensothermometrical sound with coupling of friction to project foundations and bases of buildings and constructions.

As static sounding is expressed on the technological basis it helps to reduce cost, labour-consuming character and duration of investigations at the expense of reduction of corresponded condition of standard field and laboratory test of frozen soils.

The degree of such reduction depends on concrete engineering geological conditions and particularities of designing building or construction. Technical-economic effect received from changing traditional methods of investigations of permafrost soils by static sounding is the reduction of cost of experimental work average 30%, the reduction of periods of receiving information about results of field and laboratory tests of soils in 2 - 4 times, cutting labour - consuming character of soils tests in 2 - 3 times.

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REFERENCES


CONES FACTORS IN SAND

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SYNOPSIS: The mapping of cone penetration resistance to a state index for a sand, such as relative density, is an important aspect of the CPT. A sound theoretical basis for such a mapping is provided by cavity expansion theory. Collins et al. (1992) have presented a cavity expansion solution based on the state parameter and critical state parameters for several sands, for which there are also large calibration chamber test data. A comparison of this cavity expansion solution with the experimental data suggests a systematic mismatch for sands in a denser (more dilatant) state. The absence of plastic hardening in the cavity expansion model is postulated to be the most significant factor contributing to the mismatch between theory and experiment.

1. INTRODUCTION
Sands are difficult materials to sample in anything like an undisturbed condition and accordingly the engineering of sands has come to depend on in situ testing, and penetration tests in particular. All in situ tests, however, present an inverse boundary value problem as in situ tests measure a material response to a loading rather than a material property. Soil behaviour cannot be simply read from the data but must be interpreted from the measurements using theory and/or calibration tests.

Although a few early theoretical studies (eg Vesci, 1972) provided some basis for understanding the CPT in sand, the very idealized mohr-colemb constitutive model is a poor choice for analysing the CPT and leads to arbitrary influences such as “compressibility” being invoked to explain departures between data and theory. Most interpretation today is actually based on large calibration chamber (CC) testing in which CPT response is measured under controlled conditions to develop a mapping between response and some combination of sand density and stress. But, there is no unique mapping applicable to all sands, so that one either has to test the particular sand in a calibration chamber or develop a proper understanding of the factors involved in the mapping of the CPT. The importance of developing this understanding cannot be understated and is easiest understood by example. Hilton Mines sand at 60% relative density produces the same CPT resistance as Monterey sand at 40% relative density all other factors being equal (see Figure 4 of Robertson & Campanella, 1983). In many circumstances a 40% relative density will be regarded as inadequate while 60% could well be acceptable. The engineering decision about suitability of a sand for a specific purpose then hinges on nuances attached to unquantified factors, not the CPT data itself.

Because few projects can support calibration chamber testing, it becomes essential to understand the nature of the CPT response and how sand properties can be recovered from a CPT for any chosen sand.
One possibility is to develop advanced finite element models with the actual CPT geometry (e.g., Willson, 1985), but to date these approaches have produced limited guidance or understanding. An attraction of finite elements is their ability to represent real CPT geometry and sand/steel interface slip but this attraction diverts attention away from the constitutive model which is the real limitation of finite element methods applied to date.

A CPT in sand provides just two signals, the tip resistance and the sleeve friction. Thus, only two independent parameters (at most) can be deduced from CPT data in sand. For this reason most CPT data are interpreted in terms of one parameter alone, commonly relative density but sometimes peak friction angle. There is no point using an advanced 27 parameter (or the like) constitutive model in a finite element analysis because the properties can never be understood from CPT testing. What is required is a good representation of the constitutive behaviour of sand in terms of some in situ state index (which is to be measured by the CPT) and a few material properties that can readily be established by testing disturbed samples.

Relative density is an almost universally used state index for sand. However, it is easy to show with a modest laboratory test program that relative density is misleading. When dealing with sands with a few per cent silt, one sand/silt mixture at 40% relative density can dilate while another mixture at 60% relative density can be contractive. In addition there is the deficiency that dilatancy can be suppressed by mean stress (Been & Jefferies, 1985 & 1986). There are two alternatives which avoid these errors: the relative dilatancy index (Bolton, 1986) and the state parameter approach (Been & Jefferies, 1985). The state parameter approach is preferred as it provides material independence over a wider range of sands and more importantly it is a fundamental variable in a full constitutive framework (Jefferies, 1993). The state parameter $\psi$, is defined as the void ratio difference between the current value and the value at the critical state for the same mean stress, in essence going back to the fundamental idea behind critical state soil mechanics.

Initial work with the state parameter and the CPT comprised processing the then available database of CC tests for all sands and developing simple dimensionless relationships (Been et al., 1986 & 1987a) which could be expressed in the form:

$$Q = k \exp(-m\psi)$$  \[1\]

There remains a difference from one sand to another with the coefficients $k$, $m$ in eqn [1] dependent on sand type. It was suggested that both coefficients were functions of the slope of the critical state locus (CSL), as the state parameter was a generalisation of critical state theory. In that theory the slope of the CSL controls the plastic hardening (i.e., a plastic equivalent to the role of shear modulus, $G$, in the elastic theory of cavity expansion).

Equation [1] is not established on either numerics or closed form theory and as such carries no assurance of sufficiency. The weakest point of [1] is actually the suggested dependence of the two parameters $k, m$ on the slope of the CSL alone. However it is the basic form of [1] that has attracted most attention with Sladen (1989a, 1989b) suggesting that there remained an effect of stress level and that this aspect was of first order importance.

An important contribution to the subject was provided by Collins et al (1992) who noted the importance of cavity expansion theory to understanding the CPT and developed a state parameter based theoretical analysis. Their analysis is important firstly because it uses material properties in the cavity solution, so possibly providing an explanation of the observed differences between sands, and secondly because it addresses the issue of stress level. The Collins et al. work is the starting point for this paper.
2. CAVITY EXPANSION CONTROLLED BY STATE PARAMETER

The cavity expansion theory presented by Collins et al is a spiritual descendent of the well known Vesic (1972) approach with these key features:

- peak friction varies throughout the domain as a function of state
- a stress-dilatancy rule is used
- the elastic shear modulus depends on mean stress

The friction angle - state relationship uses the data from Been & Jefferyes (1985) but identifies individual trends through a single constant rather than simply using an average relationship. The stress dilatancy model follows Bolton (1986) which is essentially the same relationship found independently on a different database by Been & Jefferyes. Both the friction-state and stress-dilatancy relationships are established from several hundred tests and so can be regarded as reliable. The shear modulus relationship is less certain, the analysis adopting a relationship proposed by Richart et al (1970) which has the familiar square root of the mean stress form but with a coefficient which has an uncertainty of about a factor of three.

Because the dilatancy and friction angle are allowed to vary throughout the plastic zone, there is no simple closed form solution for cavity expansion and the solution technique is to cast the problem as a set of simultaneous differential equations and use a standard numerical solver. The computed cavity pressure is approximated by a relationship of the form:

\[
Q = c_p p^\gamma \gamma/\gamma \exp(-c_v v)
\]  \[2\]

which is similar in form to [1] but carries an additional term giving stress level dependence. The coefficients in [2] can be calculated for any sand provided that the critical state parameters are known.

Collins et al. provide a set of constants \(c_1\) to \(c_4\) in [2] for a several sands for which there are also large calibration chamber test data. These are reproduced on Table 1, taken directly from Collins et al (who in turn obtained the critical state parameters \(\lambda\) and \(\Gamma\) from Been et al, 1987a). Table 1 also details the database of large calibration data for these sands. It is therefore possible to assess the cavity expansion theory against the laboratory measurements.

Table 1 does not include all CC studies, and the authors are aware of several CC data sets for which Collins et al do not quote parameters. Perhaps most important of the missing sands is Hilton Mines Tailings (Harman, 1976) for which \(\lambda = 0.17\) (Been et al, 1987a). The effects of the high value of \(\lambda\) and angularity of Hilton Mines Tailings means that its behaviour is very different from the sands in Table 1 and many models have difficulty matching the Hilton Mines data set.

<table>
<thead>
<tr>
<th>Sand</th>
<th>(\lambda)</th>
<th>(\Gamma)</th>
<th>(c_1)</th>
<th>(c_2)</th>
<th>(c_3)</th>
<th>(c_4)</th>
<th>No of CC Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monterey</td>
<td>0.029</td>
<td>1.878</td>
<td>9.382x10^7</td>
<td>0.938</td>
<td>0.374</td>
<td>7.426</td>
<td>31</td>
</tr>
<tr>
<td>Hokkaido</td>
<td>0.054</td>
<td>1.934</td>
<td>4.279x10^7</td>
<td>0.918</td>
<td>0.549</td>
<td>6.915</td>
<td>80</td>
</tr>
<tr>
<td>Ottawa</td>
<td>0.028</td>
<td>1.754</td>
<td>2.663x10^7</td>
<td>1.012</td>
<td>0.450</td>
<td>8.658</td>
<td>30</td>
</tr>
<tr>
<td>Reid</td>
<td>0.065</td>
<td>2.014</td>
<td>1.599x10^7</td>
<td>0.871</td>
<td>0.314</td>
<td>6.207</td>
<td>17</td>
</tr>
<tr>
<td>Tiziano</td>
<td>0.056</td>
<td>1.986</td>
<td>2.102x10^7</td>
<td>0.875</td>
<td>0.326</td>
<td>6.481</td>
<td>212</td>
</tr>
</tbody>
</table>

Reference:
- Triptale (1983), Huntman (1985)
- Harman (1976)
- Lhoier (1976)
- Baldini et al (1986)
In addition, silty sands have been tested in calibration chambers in recent years (e.g., Brandon et al., 1990) which will also be difficult to match using models based on quartz sands. Finally, it is noted that Collins et al. include Kogyuk Sand in their parameter listing, although there are no chamber tests for Kogyuk sand. While Kogyuk Sand is also a Canadian Beaufort Sea sand, it is sufficiently different from Erksak sand (the Beaufort Sea sand that has been tested in a CC, Been et al., 1987b) that we have not attempted to use the Kogyuk parameters for the Erksak sand data base.

Figure 1 compares the CC data for normally consolidated Ticino sand with the spherical cavity expansion calculation. Each CC test consists of the following information:

- specific volume of the sample
- applied boundary stresses (σ and q)
- cone penetration resistance q

From this information and the values of λ and Γ, in Table 1, the state parameter (ψ) and the normalised penetration resistance Q can be calculated for each test. These have been plotted on Figure 1. (It is noted that a correction factor to qₙ is needed to account for calibration chamber size effects, and was applied in this case, Been et al., 1987a). Similarly, a normalised resistance can be computed from the right hand side of equation [1] for each test and plotted as shown on Figure 1.

Based on Figure 1, the following are observed:

- The spherical cavity expansion points are not a single line when plotted in this form. The "scatter" represents the stress level effect noted by Collins et al. (1992) and Sladen (1989)
- The actual CC data show somewhat more scatter than the cavity expansion points, presumably the consequence of experimental variation. The stress level effect is therefore not particularly significant in terms evaluation of the real data.
- For loose sand (i.e. state parameter in the range of -0.1 to 0) there appears to be a close correspondence between the CC data and the cavity expansion theory, but as the sand becomes more dilatant (i.e. state parameter -0.3 to -0.2) the two sets of points progressively diverge.

Figure 2 shows similar data for overconsolidated Ticino sand. However, for the overconsolidated sand the difference between data and theory is somewhat more than for normally consolidated sand, and the difference appears to be independent of stress/density.

Ottawa sand data are presented on Figure 3, and suggest much more strongly that it is dilatation effects that cause the experiments and cavity expansion solutions to differ. The trends for Ottawa sand (Figure 3) and Ticino sand (Figures 1 and 2) represent the range that arises from the database for sands listed in Table 1.

3. DISCUSSION & CONCLUSION
Spherical cavity theory may be expected to overpredict CPT resistance, based on kinematics. This expectation is further heightened because the approximation of sand behaviour used by Collins et al does not invoke yield until the stress ratio reaches the Mohr-Colomb envelop, even though real sands will show yield at smaller stress ratios.

Comparing the two normally consolidated data sets, Figures 1 and 3, we see that the Collins et al solution is quite good for sand near the critical state but that the solution progressively degrades as the sand becomes more dilatant (negative state parameter).

In order to examine the comparison and reasons for divergence between the data and cavity expansion theory further, Figure 4 shows the data plotted as a "predicted" vs "measured" plot for the normalised resistance. The plotted points on Figure 4 are remarkably well clustered, but the best fit line diverges...
Figure 1: Comparison of CC test data with cavity theory for NC Ticino 4 Sand

Figure 2: Comparison of CC test data with cavity theory for OC Ticino 4 Sand

Figure 3: Comparison of CC test data with cavity theory for NC Ottawa Sand
from the line of equality as the penetration resistance increases. It is therefore concluded that the differences between the CC data and cavity expansion theory are related to dilatancy effects rather than geometry. The difference between the trends for normally consolidated and overconsolidated Ticino sand are likely to be related to how overconsolidation affects the parameters in the constitutive model. In particular, overconsolidation has a major impact on modulus in CC test samples.

Many different constitutive models can be developed around the state parameter and that used by Collins et al is in many ways too simple. In particular, the Collins et al model makes no allowance for plastic hardening and by analogy with the fact that their solution essentially scales with G (see their Figure 4) we might expect this absence of hardening to be the cause of the misprediction at dense states. One state based model which reasonably replicates sand is the NorSand generalization of critical state theory (Jefferyes, 1993) and within this model we find that there is typically about a threefold increase in the plastic modulus as we move from loose sand to dense sand (Jefferyes & Been, 1992). Given the scaling of CPT resistance by G, it is a reasonable guess that introduction of plastic hardening within an elastic-plastic spherical cavity expansion analysis should provide an excellent basis for interpreting the CPT in any sand.

**Notation**

- \( q_e \) = cone tip resistance
- \( \sigma' \) = vertical effective stress
- \( Q \) = \( q_e / \sigma' \)
- \( e \) = void ratio
- \( e_c \) = critical state void ratio
- \( \Psi \) = \( e - e_c \), \( e \), measured at same mean stress as \( e \)
- \( v \) = 1 - e
- \( c, m, k \) = coefficients defined in text
- \( \lambda \) = slope of critical state locus in e - log (p) space

*Figure 4. Cross-plot of predicted versus CC measured Q for sands from Table 1.*
4. REFERENCES


University of Southampton. (1984). Results obtained at Southampton and some preliminary interpretation. Seminar on Cone Penetration Testing in the Laboratory, University of Southampton.


Proceedings CPT'95
PIEZOCONE DISSIPATION CURVES WITH INITIAL PORE PRESSURE VARIATION

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SYNOPSIS: This paper illustrates the initial excess pore pressure drop observed during piezocone penetration and dissipation tests in homogeneous cohesive soil specimens in a calibration chamber. The penetration rate changes abruptly from 2 cm/s to 0 cm/s when a piezocone is stopped for a dissipation test, resulting in a steep decrease in cone resistance accompanied by a sudden drop in the excess pore pressures at the tip. Hence interpretation of dissipation at the tip to evaluate radial coefficient of consolidation, should be based on the initial dissipation value of excess pore pressure, and not the penetration excess pore pressure. Various factors contributing to the initial variation in excess pore pressure are surmised. The dissipation that occurs even during piezocone advance should be taken into consideration for a proper interpretation of pore pressure dissipation, recorded at locations higher above the cone base.

1. INTRODUCTION

Determination of consolidation and flow characteristics of fine grained soils for seepage and settlement analysis depends on proper interpretation of dissipation test results. The piezocone penetrometer has become widely popular as an in situ investigation tool of choice for determining the consolidation and flow characteristics of fine grained soils. However a number of pressing needs still exist in design, testing practice and in the proper interpretation of piezocone dissipation results. Non-standard dissipation curves arising from initial excess pore pressure variation due to normal stress reduction, redistribution and stress history effects needs to be further investigated. The excess pore pressure drop due to normal stress release when a cone penetrometer is stopped has not been paid proper attention in the past especially since this feature is not easily identified in field tests because of soil inhomogeneities. The initial drop in excess pore pressure if ignored can significantly affect interpretation of dissipation results. This paper illustrates the initial excess pore pressure drop observed during piezocone penetration test (PCPT) in cohesive soil specimens in a calibration chamber. Various factors contributing to the initial variations in excess pore pressure are surmised. The dissipation that occurs even during piezocone advance is examined.

2. EXPERIMENTAL STUDY

Piezocone penetration and dissipation profiles obtained from clay calibration chamber studies
Kurup et al. 1993, Voyiadis et al. 1993, Voyiadis et al. 1994), on four cohesive soil specimens 525 mm in diameter and 812 mm high are shown in figure 1. The tests were performed using a 1 cm$^3$ miniature piezocene penetrometer (fabricated by Fugro McClelland Engineers B.V., The Netherlands) with the filter located either in the lowest 1/4 of the cone at the very tip or starting 0.5 mm above the cone base and 2 mm in vertical height. A sudden drop in the penetration pore pressure, $\Delta p_e$ (indicated by $\Delta u/\Delta u > 1$, figure 1) is observed as soon as the penetration ceases. This initial drop in excess pore pressure especially at the tip (6%, 15.7%, 26.1% and 12.5% for specimens 1, 2, 3 and 4 respectively) if ignored can significantly affect the interpretation of dissipation results to evaluate the radial coefficient of consolidation, $c_r$. Hence it was recommended (Kurup et al. 1993, Kurup et al. 1994, Kurup et al. 1995), that the interpretation of the dissipation results to evaluate the radial coefficient of consolidation, $c_r$, should be based on the initial dissipation values of the excess pore pressure, $\Delta u_e$ and not the penetration excess pore pressure, $\Delta p_e$.

3. INITIAL EXCESS PORE PRESSURE VARIATION
The initial excess pore pressure variation giving rise to non-standard dissipation curves are due to:
- pressure drop due to normal stress release
- pore pressure redistribution due to gradient
- dissipation (a) during advance (b) after stoppage of cone tip advance

3.1 Factors influencing initial excess pore pressure variation
The initial excess pore pressure variation may be influenced by a variety of factors relating to soil characteristics, piezocene design and testing practice.

**Soil Characteristics**
- stress-strain behavior at very high strain rates
- over consolidation ratio
- lateral stress coefficient
- plasticity index
- hydraulic conductivity and compressibility

**Piezocene Design**
- cone size (scale effects)
- filter expansion (flexible filters)
- filter location

**Testing Practice**
- continuous or intermittent push
- rate of penetration
- Improper clamping of the penetrometer rod

Pore pressure redistribution around the tip which might conceivably be faster for a miniature piezocene can also influence the pressure drop observed at the tip.

3.2 Excess pore pressure drop due to normal stress release
There are essentially two effects that influence the cone resistance and excess pore pressure during varying rate of penetration: (1) viscous and dynamic effects and (2) drainage effect. At higher rates of penetration, the cone resistance and pore pressures (especially at the tip) may increase due to the viscous and dynamic effects. The penetration rate changes abruptly from 2 cm/s to 0 cm/s when the cone is stopped for a dissipation test, resulting in a steep decrease in cone resistance accompanied by a sudden drop in the excess pore pressures at the tip. This normal stress reduction and pore pressure drop is not indicated by existing interpretation methods, since penetration rate effects have not been incorporated in constitutive models and numerical simulations to analyze PCPT.
Figure 1. PCPT Profiles
Differences and uncertainty still seem to exist on the interpretation of recorded pore pressure dissipation. Teh (1995) attributes the initial pore pressure drop entirely to the high gradient around the tip, ignoring the drop due to normal stress release. Pore pressure gradient around the tip is primarily influenced by the stress history. It is unclear how the analysis of Teh and Houlsby (1991) that does not properly account for different stress histories (Teh 1995) is able to attribute the sudden cone-tip pore pressure drop entirely to redistribution due to gradient around the cone.

Sully and Camppanella 1994 have suggested the use of peak value of excess pore pressure once the reduction in tip bearing stress has occurred. It took 5 s for 25% reduction in their test. It is the authors' opinion that most of the normal stress release occurs almost instantaneously. Significant dissipation and redistribution could occur in 5 s that should be taken into account by a coupled consolidation analysis. Monitoring tip resistance during dissipation tests can provide further information regarding this normal stress reduction. In stiff heavily overconsolidated clays the excess pore pressures above the base of the cone (sometimes even negative) could increase initially due to a redistribution of Δu around the tip (i.e. Type II and III, in figure 2). Approximate procedures (Sully and Camppanella 1994) have been outlined to correct such non-standard dissipation curves so that they may be interpreted using existing interpretation methods. These methods are approximate and require further refinement.

The excess pore pressure drop due to normal stress release when penetration is stopped has not been paid proper attention in the past especially since this feature is not easily identified in field tests because of soil inhomogeneities and sometimes coarse data sampling frequency. The effect of rate of penetration on cone resistance and penetration pore pressures, and drop in their values when stopped for a dissipation analysis needs to be further investigated.

3.3 Rate of penetration and drainage response
Depending on the drainage characteristics and penetration rate, the pore pressure dissipation during piezocene advance can cause significant pore pressure gradients along the shaft. This is in addition to those caused by normal stress release and dilation effects due to shear. In coarse grained material, penetration takes place under drained conditions. In fine grained soils (clays and clayey silts), penetration (at the standard rate of 2 cm/s) takes place under predominantly undrained conditions. In fine sands and silty sands, partially drained conditions may exist during penetration at 2 cm/s. However, the rate of penetration may be increased or decreased to produce undrained or drained conditions. There is however a critical point that needs to be mentioned. Undrained conditions are with reference to, at or
immediately around tip of the cone. In other words the minimum rate of penetration that gives undrained soil response at the tip might only give a partially drained response for a location on the shaft higher up the cone base (say above the friction sleeve). This is essentially due to pore pressure dissipation that occurs even during piezocone advance. Excess pore pressure dissipation that occurs during piezocone advance becomes important especially when interpreting the pore pressure data recorded by filter elements located on the shaft. The higher the location of the filter elements above the cone base and slower the rate of penetration, the greater is the influence due to this effect. To illustrate this, analysis was performed (Kurup 1993) in specimen 1 using a method (Gupta and Davidson 1986) that simulates cone advance by a series of successive spherical cavity expansions. Penetration rates of 2 cm/s and 0.2 cm/s were investigated and excess pore pressure dissipation was allowed during piezocone advance. Penetration was carried to a depth of 570 mm (level 1) in a single stroke. The results of the analysis are shown in Figure 3. Time t = 0 seconds corresponds to the instant the tip of the piezocone reaches level 1 (i.e. penetration ceases). Figure 3a shows the radial pore pressure distribution at level 2, 180 mm above the cone base. The time taken for the tip to travel from level 2 to level 1 (180 mm) is 9 seconds at a penetration of 2 cm/s, and 90 seconds at a penetration rate of 0.2 cm/s. The initial radial pore pressure distribution at level 2, at time t = 0 seconds after penetration ceases, (with the tip of the cone at level 1) is shown in Figure 3a (by solid lines). The initial excess pore pressure distribution for the two different penetration rates is significantly different, especially at smaller radial distance from the cone.

The significance of the above discussion is now apparent. If the piezocone had a filter element located on the shaft at a distance of 180 mm above the cone base, based on this analysis, it would have recorded normalized excess pore pressure \(\Delta u/\sigma'\) 48% higher at a penetration rate of 2 cm/s than at 0.2 cm/s. This would be due to the effect of dissipation alone. It can be seen that for a filter element located just above the cone base, this effect would be very small (Figure 3b). At higher dissipation times, during the dissipation phase after penetration has ceased (shown for t = 300 seconds in Figures 3a and 3b), the spatial pore pressure distribution for the two different penetration rates is seen to converge. Precautions should be taken while interpreting...
pore pressure results along the shaft of piles jacked into soil at slow rates (sometimes 120 times slower than a standard PCPT). Significant dissipation and spatial pore pressure redistribution can take place during the time lag that needs to be taken into account during interpretation.

4. CONCLUSIONS
The penetration rate changes abruptly from 2 cm/s to 0 cm/s when a piezoecone is stopped for a dissipation test, resulting in a steep decrease in cone resistance accompanied by a sudden drop in the excess pore pressures at the tip. Hence interpretation of dissipation at the tip to evaluate radial coefficient of consolidation, \( c_r \), should be based on the initial dissipation value of the excess pore pressure, \( u'_e \), and not the penetration excess pore pressure, \( u'_p \). The dissipation that occurs even during piezoecone advance should be taken into consideration for a proper interpretation of pore pressure dissipation, recorded at locations higher above the cone base.

5. REFERENCES
Processing of data from CPT tests

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SYNOPSIS: The CPT test yields a large quantity of data which have to be corrected and processed before they are presented and evaluated. This is facilitated with the aid of computer programmes. Such programmes should be used with caution and the process should preferably be interactive and controlled by the user. The operation of one such programme, CONRAD, which has been produced at SGI with special reference to Swedish experience from mainly soft soils and international experiences from other soils, is outlined in the paper.

1. PROCESSING DATA FOR REPORTING TEST RESULTS
1.1 Parameters for interpretation
The basic parameters obtained from a CPT test are:
- Total cone resistance, $q_T$
- Total sleeve friction, $f_T$
- Total pore pressure, $u$

In order to obtain the basic parameters $q_T$ and $f_T$, the pore pressure has to be measured at the base of the conical tip between the tip and the friction sleeve and the measured values of cone resistance and sleeve friction corrected for this. In the literature, a number of empirical correlations have been suggested to enable the use also of pore pressures measured on the conical face of the tip for correction of cone resistance and sleeve friction. However, these can be grossly misleading and should not be used.

The following basic parameters are also required for interpretation of the results:
- Initial in situ pore pressure, $u_0$
- Initial vertical stress in situ, $\sigma_{v0}$ (calculated from the density of the soil)

The initial pore pressure is estimated from observations of the free ground water level and equalised pore pressures measured in more permeable layers at temporary stops in the penetration test. If the latter values are missing, supplementary pore pressure measurements will have to be made at a number of levels.

The initial vertical stress in situ is estimated by using the density of the soil. This estimation can often be made with an interactive procedure using the classification of soil type and stiffness obtained from the test results to estimate an approximate density. In clay and organic soil, an accurate evaluation of the test results requires samples to be taken for determination of the liquid limit, $w_L$. Among the basic parameters required for interpretation of CPT tests in fine-grained soil, Liquid limit, $w_L$, can therefore indirectly be included.

1.2 Secondary parameters for interpretation
Different relations between the basic parameters are used for interpretation of the test results. For a preliminary interpretation, the following parameters are used:
- $u = u_0 - u_T$
- Friction ratio $R_f = f_T/f_{v0} \cdot 100$, %
- Differential pore pressure ratio $DPPR = \Delta u/q_T$
1.3 Data processing
The values of the parameters read off in the field are stored in a memory for further processing, presentation and interpretation in the office. In order to reduce the storage space required, a certain selection and reduction can be made in the field. For cone resistance and sleeve friction, it is sufficient if values are stored at about every 50 mm of depth. In contrast, all measured values of pore pressure are to be stored. Preferably, averages of the measured values over the depth interval should be stored.

The recorded depth of penetration must be checked against the manual measurement at termination of the penetration. In the event of significant differences, the operator's notes and the field plot of the data are studied in order to find out whether the error has occurred at a certain level or gradually, if this is possible. In the first case, the recorded depths below the level of error are adjusted with a constant value and in the second case, all the recorded values of depth are multiplied by a correction factor to make the recorded depth match the manually measured depth.

The zero values read off before starting the test must be corrected for possible water pressures acting on the probe when the zero reading was taken. If the probe is submerged in water, for example in a pre-drilled water filled hole, it will be affected by a certain water pressure at the zero reading, which also affects the zero values read off. This applies foremost to the pore pressure, and also to the cone resistance and the sleeve friction in various degrees depending on the design of the probe. These corrected zero values must be compared to the zero readings read off after retraction of the probe after the test.

Should significant differences have occurred, these must be analysed. If the zero shift can be traced to the end of the test, the measured values are used without correction. If the zero shift can be traced to a certain level where very high loads have occurred, for example in a very stiff dry crust or when passing a very stiff layer or a stone, the first zero values are used down to this level and the latter zero values are used onwards.

In the case of large zero shifts which cannot be deduced with certainty, the test must be repeated. Also in other cases where the corrections have been of such a size that they have significantly affected the interpretation, the results should be treated with caution and the tests supplemented.

In the following presentation, all measured values of pore pressure are plotted versus depth in order to obtain the highest resolution possible. The measured values of cone resistance and sleeve friction are corrected according to calibration data and the pore pressures that were read off simultaneously with these data respectively. If the values of cone resistance and sleeve friction have been averaged for certain depth intervals, a corresponding averaging has to be made for the pore pressures measured simultaneously in the particular depth intervals before the correction is made.

At insertion of pore pressure, sleeve friction and cone resistance in the formulas for calculation of the parameters $R_c$ and $D_{PFR}$, averages are also used for the corresponding depth intervals. However, in this averaging and also in correlation of the different parameters to penetration depth, it has to be observed that the parameters are measured at different locations along the probe. In relation to the very apex of a new tip on the probe, the cone resistance is measured about 21 mm higher, the pore pressure about 38 mm higher and the sleeve friction about 110 mm higher (and the pore pressure at mid-height of the conical face about 16 mm higher). The penetration depths which are recorded simultaneously with the readings of the respective parameters therefore have to be recalculated with regard to the location where the parameter was measured. This is done after the correction of cone resistance and sleeve friction for related pore pressures.

2. GRAPHICAL PRESENTATION
The test results are presented as curves for the basic parameters $q_r$, $f_s$ and $u$ versus depth.

Uncorrected values of $q_r$ and $f_s$ must not be presented without clear information that they represent uncorrected values which cannot be used for interpretation, except for those special cases in which the influence of the pore pressures is not significant, (i.e. mainly in sands)
As support for a preliminary manual assessment and interpretation, curves for $u_r$, $A_r$, $R_s$ and $DPPR$ (and possible measurements of $\Delta W_{FACE}$) versus depth are presented. The recommended standards for CPT tests specify that the presentation shall be made with the following relations between the scales:

Depth - $q_T$  
1 m - 2 MPa

Depth - $f_T$  
1 m - 50 kPa

Depth - $u$  
1 m - 20 kPa

An additional, more detailed presentation of all parameters is made in scales selected with consideration to the measured values in the particular test. In this latter presentation, the parameters $u_r$, $A_r$, $R_s$ and $DPPR$ (and possible measurements of $\Delta W_{FACE}$) are also plotted versus depth.

3. FURTHER PROCESSING OF DATA AND EVALUATION OF STRATIGRAPHY

In the further processing of the data, these normally have to be filtered and averaged. The filtering is performed in such a way that measured values which are not relevant are screened off. Examples of such data are relatively low values of cone resistance and pore pressure, and high values of sleeve friction respectively, which are measured directly at restart of the penetration after an interruption for adding new sounding rods and re-clutching the pushing equipment.

Before this is done, the presented curves for $q_T$, $f_T$, $u$, $A_r$, $R_s$ and $DPPR$ must be studied. Depths of boundaries between significant seams and layers are identified and marked.

For very thin layers, a classification and interpretation of the soil and its properties has to be performed manually. In order to obtain an interpretation with the aid of a computer programme, relevant measured values of cone resistance, pore pressure and sleeve friction are required. However, in order to obtain relevant measured values of cone resistance and sleeve friction, the layer has to have a certain minimum thickness (0.2 - 0.7 m). Much thinner seams and layers can be detected and classified by studying the generated pore pressures in the profile and by relating possible variations to tendencies for changes in cone resistance and sleeve friction at the corresponding levels, and also by relating them to results from samplings and corresponding generated pore pressures at adjacent test points. This type of interpretation can only be made manually and should take place before a subsequent computer aided interpretation, if any.

In filtering and averaging, the soil profile is divided into depth intervals, e.g. 0.2 m. An interval should not be allowed to pass over a pre-marked boundary between layers, but in this case the pre-marked level should constitute the lower limit of this interval and the upper limit of the next. For each interval, the corrected parameters within its limits are collected. If a boundary of the interval consists of a pre-marked boundary between layers, the values on this boundary are excluded. Otherwise, the values on the boundaries of the intervals could be included in the data for both the overlying and the underlying intervals.

In a simple filtering process, the averages of the parameters within the intervals are calculated first, followed by the standard deviations. All values that differ more from the averages than the standard deviations are filtered out before new averages are calculated. These averages are then used for classification of the soil within the respective intervals and for interpretation of its various properties.

4. SOIL CLASSIFICATION

In the tests, only cone resistance, sleeve friction and pore pressure are normally measured. Since a large number of factors affect the results, no unambiguous soil classification can be made on the basis of these parameters alone and supplementary data are required. These mainly consist of a measurement of the natural pore pressure profile in situ, based on sampling, followed by further measurements of parameters or parallel tests with other methods.

A preliminary classification of the soil is often made from the results of the CPT test. In addition to the parameters measured in the test, the foremost supplement required is then a measurement of the initial pore pressure profile. Usually one (or several) of the classification charts is used, which have been
produced empirically on the basis, for example, of how the relation between cone resistance and friction ratio normally varies with the type of soil. The evaluation of soil type made in this way can then be checked by comparison with a corresponding evaluation based on the normal variation between pore pressure ratio and cone resistance with soil type. Often, especially in even-grained soils and pure clays, the classifications are in agreement but in other cases conflicting classifications may be obtained. The soil classification must then be made more elaborate and, if possible, further bases for forming an estimate must be utilised. A preliminary classification can be made as follows:

All known values of the density of the different soil layers are used to calculate the variation of the total overburden pressure with depth. The effective vertical stress is then calculated using the measured in situ pore pressures. On levels or in entire profiles where determinations of the density are missing, an empirical estimation of the density has to be made parallel to the classification of the soil.

In the uppermost layers of dry crust or fill, an estimated density is applied. The total and effective vertical stresses are then calculated in the underlying depth intervals, which are normally given thicknesses of 0.2 m. The density in the particular interval is given as the determined density or, when this is missing, as the density in the adjacent overlying soil.

The parameters \( \varepsilon_1 \) and \( \varepsilon_2 \) are calculated for the interval and the principal character of the soil is estimated with the aid of the diagram in Figure 1. The division into sand, silt and more fine-grained soil is thus mainly made on the basis of the relation between the cone resistance and the normal in situ stress condition and the developed sleeve friction in relation to this cone resistance. In soft clay, the measured sleeve friction is very small and relatively unreliable but in overconsolidated clay, where the cone resistance may be of the same size as for soft coarser soil, the measured values normally become larger and more reliable. Possible uncertainties in the measurements of sleeve friction normally have a relatively small influence on this division. The main exceptions are highly sensitive clays and/or silty clays. In these soils, the sleeve friction may be very low, at the same time as the measured stiffness in relation to the overburden pressure places the soil in the region for silt in the diagram in Figure 1. However, in these soils very high pore pressures are often developed in the tests and a check on whether the factor \( B_1 (\frac{\Delta u}{\varepsilon_1}) \) is higher or lower than 0.6 can be used to judge whether the soil should be classified as silt or clay.

In cases where the soil in the interval is classified as sand or silt, its stiffness is also classified and, if values of this are missing, its density by using the same diagram. Previously the classification of stiffness in coarser soil was normally made on the basis of the cone resistance \( \varepsilon_1 \) or alternatively the net cone resistance \( \varepsilon_1 - \varepsilon_2 \). This has been shown to result in a gradual change in the classification with depth as the vertical stress and, to an even
higher degree, the cone resistance increases. The classification according to the normalised net cone resistance \((q_{\nu} - \sigma_0)/\omega\) is more correct from this point of view, but empirical experience is somewhat more limited. The limits for the various sub-designations with respect mainly to the density of sand and silt are therefore preliminary. However, the borderline between silt and “clay and organic soil” can be considered to be well established.

The normalisation by division by the effective vertical stress may introduce a source of error in superficial layers where the vertical stresses are low. A check must therefore be made so that the net cone resistance is \(\geq 1.5\) MPa if the soil is to be classified as a sand and \(\geq 500\) kPa to be classified as silt. A check must also be made so that the net cone resistance is greater than 20 MPa if a sand is to be designated as very dense, greater than 10 MPa to be designated as dense, greater than 8 MPa to be designated as medium dense and greater than 2.5 MPa to be designated as loose. The corresponding limits for silt are 10, 5, 2.5 and 1.0 MPa.

The limits between the different groups of soil are furthermore not distinct and apply in principle only to even-grained soils. Soils whose parameter plot near the border between sand and silt may thus, for example, consist of silt, sandy silt, silty sand or sand. Correspondingly, materials whose parameters fall within the silt region but are close to the border with cohesive soils may often be clayey. More multi-graded materials, such as clayey sand or fine-grained till, will probably fall within the regions for silt or clay depending on their clay content. The fact that the certainty and resolution in the soil classification based on CPT tests is limited has led to a preference to assign this classification to “type of soil behaviour from a geotechnical point of view” rather than to classification with respect to grain size distribution. Because of the complex influence of different factors on the measured parameters, this simple division based only on cone resistance and sleeve friction should not be made without a parallel check of the generated pore pressures. Otherwise, there is considerable risk of error, for example, a layer where relatively high pore pressures are developed being classified as pure sand only because the soil is also stiff and the friction ratio is low.

In those cases where the soil has been classified as “clay and organic soil” in the first diagram or after further checks or where this has been specified at an earlier stage, the classification process passes over to the special classification charts for this type of soil. These charts are based on the parameters net cone resistance \((q_{\nu} - \sigma_0)\) and \(B_2 = \Delta u/(q_{\nu} - \sigma_0)\). The parameter \(B_2\) is calculated first.

In the diagram in Figure 2, the soil is divided into five main groups:
- Heavily overconsolidated clay
- Overconsolidated or very silty clay
- Normally consolidated clay or slightly overconsolidated silty clay
- Normally consolidated silty clay and/or highly sensitive clay
- Normally consolidated gyttja or organic clay

The designation “normally consolidated” refers to an overconsolidation ratio of OCR 1 - 1.5, overconsolidated to OCR 1.5 - 10 and heavily overconsolidated to OCR > 10 in accordance with the Swedish classification system.

The soil in the groups is also summarily classified with respect to the undrained shear strength as very soft, soft, medium stiff, stiff or very stiff. This division is preliminary because the relation between the net cone resistance and the shear strength depends on the consistency limits of the soil, among other things. In cases where density values are missing in the interval, a chart for density estimation is used.

After completion of the classification process and possible estimation of density in the first interval, the process is repeated in the next interval with insertion of known or estimated density in the now overlying interval. In extreme cases, the effective stresses in superficial layers calculated with assumed and evaluated density in the interval respectively may differ so much that the evaluation is affected. This should be checked and if this is the case the process is repeated with insertion of the new estimated density for the particular interval.

The main purpose of the CPT test is not to classify the soil but to clarify the stratification
and the boundaries between different layers and to give an overview of the properties of the soil. Presentation and evaluation are normally made with computer support in order to rationalise the heavy work of calculation and plotting. The computer programmes then often perform a preliminary classification for those depth intervals in which the type of soil has not been given manually as an input. Because of the difficulties and ambiguities in the interpretation process described above, this preliminary classification must be checked manually and judged by using all available penetration test data, parallel field tests, existing soil samples and further information on soil conditions. The classification should most appropriately be performed as an interactive process.

Even after a manual check and assessment, the classification is not unambiguous but simply an indication of the type of soil for which the results of the test are typical. The classification can be made more reliable if results from parallel tests with alternative filter location or excess pore pressure dissipation tests are available. The CPT test ought always to be supplemented by at least disturbed sampling for a correct classification of the soil and determination of, for example, liquid limit for a more careful evaluation of the properties in clay and silt. The preliminary classification and interpretation are then a great aid in determination of the required sampling and can later be modified with regard to the results from this.

5. EVALUATION OF PROPERTIES
A number of properties in the different intervals in the soil profile can be estimated from the CPT parameters by using mainly empirical correlations. These properties are primarily:

a) For cohesive soils
- undrained shear strength
- preconsolidation pressure (and overconsolidation ratio)

b) For friction soils
- relative density
- friction angle
- deformation moduli

6. CONRAD
The computer programme CONRAD, which has been produced at SGI, operates according to the procedure outlined above. The empirical relations used for evaluation of properties in cohesive soils are mainly based on Swedish experience and the relations used for friction soils are mainly based on reported international experience. The programme CONRAD operates in a standard PC. An output from the programme showing measured and basic parameters is in Figure 3 and a plot with evaluated parameters is in Figure 4. Description in detail of the correction and evaluation process in the programme is given in SGI information No.15.
Fig 3. Measured and basic parameters output from the programme CONRAD.

Fig 4. Evaluated parameters output from the programme CONRAD.

7. REFERENCES
Linköping
The use of $c_u$ from Danish triaxial tests to calculate the cone factor.

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SYNOPSIS: Anisotropic consolidated triaxial tests using height, $H$, equal diameter, $D$, and smooth pressure heads have been used to estimate the undrained shear strength in six Danish soils. The cone factors calculated from these undrained shear strengths are all very close to 10. The result is in good agreement with the cone factors estimated analytically by the cavity expansion theory in connection with a failure pattern (Vesic (1975)) and the strain path method (Housby and Teh (1991)). The explanation of cone factors estimated in other countries in the range of 15-20 are explained either by the existence of fissures in the clay or by the use of specimen height $H = 2D$ in the triaxial test.

1. INTRODUCTION
In 1991 to 1994 an investigation concerning CPT in Danish soils were carried out. The results from the investigation are published in Luke (1994). The main concern of the investigation was estimation of the undrained shear strength $c_u$ of cohesive soils applying the results from CPTs. Six test areas of $10 \times 20$ $m^2$ with different types of soil were closely examined at shallow depths.

The undrained shear strength was estimated from 1 or 2 Consolidated Anisotropic (oedometric) Volume constant Compression triaxial tests (CAoeVC) carried out in the laboratory on intact soil specimens according to the Danish practice. In these triaxial tests the specimen height equals the diameter, and smooth pressure heads are used. The undrained test was performed at a constant rate of strain (3% of the total sample height per hour) and $c_u$ was defined at a deformation of 10% of the total sample height. A more detailed description of the Danish triaxial test is described in Luke (1994).

The CPT cone used was the van den Berg piezo-cone. The geometry of the cone penetrometer is within the standards specified in the international guidelines, ISSMFE No 7 (1989). The cross sectional area of the cone is 10 cm$^2$, the cone apex angle is 60° and a penetration velocity of 20 mm/sec was used.

1.1. Description of the six soils
The six investigated soils have very different strength, deformation and classification characteristics, and can be regarded as representative for a wide spectrum of Danish soils.

The soils are:
- a rather fat ice sea Yoldia clay (Aalborg),
- a soft (NC) clay till (Purhus)(clay till 1),
- a stiff (OC) clay till (0lst)(clay till 2),
- a very plastic Tertiary clay (0lst),
- a Holocene silty clay (Aalborg) and
- an organic mud (Kaaas).

All six sites are located in the central or northern part of Jutland, Denmark. Table 1 shows a summary of the obtained classification parameters.
Table 1: Classification parameters obtained for the Danish soils at the 6 test areas.

<table>
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<tr>
<th>Soil</th>
<th>level m</th>
<th>w</th>
<th>u_1</th>
<th>w_p</th>
<th>I_p</th>
<th>clay cont.</th>
<th>c_u</th>
<th>σ_u</th>
<th>σ^2_u</th>
<th>OCR</th>
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<td>Yold. clay</td>
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<td>65</td>
<td>26</td>
<td>39</td>
<td>62</td>
<td>29</td>
<td>225</td>
<td>8-11</td>
<td></td>
</tr>
<tr>
<td>Clay till 1</td>
<td>1.50</td>
<td>17</td>
<td>22</td>
<td>15</td>
<td>7</td>
<td>15</td>
<td>32</td>
<td>110</td>
<td>3-4</td>
<td></td>
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<tr>
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<td>174</td>
<td>37</td>
<td>137</td>
<td>83</td>
<td>24</td>
<td>850</td>
<td>2-3</td>
<td></td>
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<tr>
<td>Holoc. clay</td>
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<td>37</td>
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<td></td>
<td></td>
</tr>
<tr>
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<td>23</td>
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<td>32</td>
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</table>

2. TEST RESULTS

The test results from the six soils are shown in Table 2. The Table also includes the cone factors defined as

\[ N_{ck} = \frac{q_c - \sigma}{c_u} \]  \hspace{1cm} (1)

where \( \sigma = \sigma_{ul} \) and the rigidity indices:

\[ I_{c,E50} = \frac{E_{50}}{3 \cdot c_u} \]

The undrained modulo \( E_{50} \) is defined as the modulo going through the initial point and the point corresponding to 50% of the failure load. A Poisson ratio of \( \nu = 0.5 \) is assumed.

Table 2: Average \( q_c \), undrained shear strength \( c_u \) (C<AovC), \( E_{50} \), \( I_{c,E50} \) and \( N_{ck} \).

<table>
<thead>
<tr>
<th>Soil</th>
<th>level m</th>
<th>( q_c ) kPa</th>
<th>( c_u ) kPa</th>
<th>( E_{50} ) MPa</th>
<th>( I_{c,E50} )</th>
<th>( N_{ck} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yold. clay</td>
<td>1.50</td>
<td>1292</td>
<td>133</td>
<td>19.6</td>
<td>49</td>
<td>9.9</td>
</tr>
<tr>
<td>Clay till 1</td>
<td>1.50</td>
<td>681</td>
<td>52</td>
<td>29.0</td>
<td>188</td>
<td>12.2</td>
</tr>
<tr>
<td>Tert. clay</td>
<td>1.25</td>
<td>3104</td>
<td>65</td>
<td>15.0</td>
<td>77</td>
<td>10.5</td>
</tr>
<tr>
<td>Holoc. clay</td>
<td>1.25</td>
<td>238</td>
<td>21</td>
<td>2.2</td>
<td>35</td>
<td>10.6</td>
</tr>
<tr>
<td>Clay till 2</td>
<td>1.25</td>
<td>3210</td>
<td>350</td>
<td>24.0</td>
<td>23</td>
<td>8.5</td>
</tr>
<tr>
<td>Org. mud</td>
<td>2.40</td>
<td>243</td>
<td>23</td>
<td>1.1</td>
<td>15</td>
<td>8.4</td>
</tr>
</tbody>
</table>

In Fig. 1 the undrained shear strength \( c_u(C<AovC) \) obtained from the triaxial tests is plotted against the average value of \( (q_c - \sigma_{ul}) \) for each of the six soils.

The data are all close to the line \( N_{ck} = 10 \). The average is \( N_{ck} = 10.01 \) with a standard deviation of 1.17.

3. EVALUATION OF OBTAINED RESULTS

In the following the results from the measurements will be compared with existing analytical solutions and previously found empirical cone factors estimated from consolidated triaxial compression tests in other countries.

3.1. Analytical estimation of \( c_u \)

Analytical models describing the cone resistance of a penetrating cone are either based on the classical bearing capacity concept, the cavity expansion theory or the strain path method.

It is generally agreed that the bearing capacity method does not adequately represent the steady cone penetration problem, primarily because it assumes the soil to be ideally plastic, while in deep penetration the displaced material is accommodated by elastic deformations of the soil.
Using the measured tip resistances, $c_u$ has for comparison reasons been back-calculated from four analytical models based on either the cavity expansion theory or the strain path method. The models are briefly outlined in the following.

The simple spherical cavity expansion solution have been used by Vesic (1972) to describe the penetration of a cone. The cone factor was determined as

$$N_c = \frac{4}{3} (1 + \ln(L))$$

Baligh (1975) used a spline solution combined with cavity expansion. The cone factor was found to be

$$N_c = (1 + \ln(L)) + 11$$

Vesic (1975) suggested that the cone resistance can be estimated by determining the ultimate pressure needed to expand a spherical cavity associated with a failure pattern. He then obtained the following expression for the cone factor

$$N_c = \frac{4}{3} (1 + \ln(L)) + 2.57$$

Housby and Teh (1991) used the strain path method with an additional equilibrium correction provided by large strain finite element analysis. Also they incorporated effects of the rigidity index $I_r$, the horizontal in situ stress and the roughness of the cone and shaft.

$$N_c = N_r \left(1.25 + \frac{I_r}{2000}\right) + 2.4\alpha_f - 0.2\alpha_s - 1.8\Delta$$

where

$$\sigma = \sigma_{inh} \text{ in eq. (1)}$$

$$N_r = \frac{4}{3} (1 + \ln(L))$$

$\alpha_f$ and $\alpha_s$: roughness factors.

$$\Delta = \frac{\sigma_{inh} - \sigma_{inh}}{2\sigma_s} \text{, in situ stress factor}$$

As none of the theories prescribe which modulus of elasticity to use when calculating the rigidity index, $E_{so}$ has been used. The roughness factors in the strain path solution by Housby and Teh (1991) has been estimated using the sleeve friction resistance $f_s$.

Fig. 2 shows the measured versus the analytically calculated undrained shear strengths.

![Fig. 2: Measured versus analytically calculated undrained shear strengths.](image)

From the figure it is seen that $c_u$ is generally overestimated using the solution suggested by Vesic (1972) and underestimated using the solution suggested by Baligh (1975). The solutions suggested by Vesic (1975) and Housby and Teh (1991) corresponds to the measured results to quite a high degree.

Although the theories are approximations, not including all aspects describing the problem of a steady penetrating cone (e.g. the cavity expansion theory does not consider the form of the cone tip and none of the theories take into account the effect of high strain rates) at least two of the models, based on different approaches (Vesic (1975) and Housby and Teh (1991)), are in very good accordance with the measured values of $c_u$.

3.2. Empirical cone factors based on triaxial test results

The $N_{12}$ values found for the 6 Danish soils are in the following compared with results from 11 well documented clay deposits, scattered throughout the world (Lunne and Rad (1986)), at which anisotropic consolidated undrained compression triaxial tests (CAUC) have been used to estimate $c_u$. As shown in
Fig. 3. The cone factors, \( N_{\text{cl}} \), from the 11 sites were found to be in the range of 8 to 27.

Fig. 4: \( N_{\text{cl}} \) plotted against \( I_{E50} \) for the 3 Danish soils and reported data from Lunne et al. (1985).

4.2. Rigidity index.
Another possible explanation for the deviant behaviour of the cone factors for the soils from international sites is a dependency on the rigidity index \( I_r \). Unfortunately very few reported data sets include information about the modulus of elasticity of the soil (e.g., \( E_{50} \)) and the few that do have \( I_{E50} \) nearly in the same range as the Danish soils (10 < \( I_{E50} < 300 \)) and cone factors as shown in Fig. 4.

4. DISCUSSION ON DEVIATING CONE FACTORS
In the following a discussion concerning possible explanations of the large deviations of the \( N_{\text{cl}} \) values reported by Lunne and Rad (1986) will be given.

4.1. Fissured soils
Many of the very high cone factors exceeding \( N_{\text{cl}} = 20 \) have been observed in hard OC fissured or weathered clays (see Fig. 3). A possible explanation for these high cone factors is that the failure in the triaxial test is liable to occur in the fissures. The undrained shear strength can therefore be somewhat smaller than that measured by CPT, which to a higher degree tends to measure the intact strength in between the fissures.

But even when the results from the fissured clays are disregarded there is still a large number of tests which have cone factors in the range of 15 - 20 see Fig. 3.

Fig. 5: \( N_{\text{cl}} \) plotted against \( I_{E50} \) for 6 Danish soils and reported data from Lunne et al. (1985).

One data set is marked out on Fig. 4 as \( N_{\text{cl}} = 15 \). This large cone factor must be explained by other factors than the rigidity index since the corresponding \( I_r \) is quite low.

The other data might support the dependency of \( N_{\text{cl}} \) on \( I_r \), as suggested by Vesic (1975) in which \( N_{\text{cl}} \) rises from 7 to 11.3 by an increase in \( I_r \) from 10 to 250. But the variation might as well be due to small deviations of \( N_{\text{cl}} \) around an average value of \( N_{\text{cl}} = 10 \).

4.3. Different procedures for triaxial tests
In a triaxial test stresses, strains and pore pressures are measured on the surface of the soil sample why homogeneous stress and strain conditions are important during the test. In order to obtain homogeneous stress and strain conditions, Jacobsen (1970) pre-
scribed the use of smooth pressure heads and specimens with height equal the diameter $H = D = 70$ mm. This procedure is used in the Danish triaxial test. In other countries $H$ equals at least twice $D$ and usually rough pressure heads are used.

According to the theory by Jacobsen (1970) the consequence of using rough pressure heads and $H = 2D$ is that the shear stresses at the ends of the specimen will result in the creation of stiff bodies as shown on Fig. 5b which will yield in a slight over-estimation of $c_u$. If the soil expands under rupture conditions a narrow rupture zone will be formed with an inclination of $45^\circ - \frac{\phi}{2}$ to the vertical whether rough or smooth pressure heads are used ($\phi$ is the angle of internal friction). In these tests $c_u$ will be underestimated (mean 40% for clay till, Jacobsen (1970)). The three types of failure modes are shown in Fig. 5.

Tests in which a narrow rupture zone is formed (Fig. 5c and 5d) show maximum deviator stress as a peak point. Fig. 6 shows the test results of undrained triaxial tests performed on specimens with $H = D$ and $H = 2D$ on a dilating soil.

It is possible to explain the deviating behaviour of the cone factors estimated at the 11 sites applying the theory concerning triaxial specimen size by Jacobsen (1970). Soft soils ($q_v < 500$ kPa), in which no dilation is to be expected, have cone factors very close to 10. Some of the cone factors are slightly lower than 10, which could reflect the use of rough pressure heads in connection with non-dilating soils (slightly overestimated $c_u$). A possible explanation of the existence of cone factors in the range of 15-20 for ($q_v > 500$ kPa) can be either the use of practically smooth pressure heads or that dilating soils have been tested.

5. CONCLUSION
The cone factors estimated from the six Danish soils are all in very close alignment with $N_{18} = 10$.

The solutions that best describes $c_u$, corresponding to the undrained shear strength estimated by triaxial tests in the six investigated soils, is the one by Vesic (1975) in which the cavity expansion method is used in connection with a failure pattern and the solution suggested by Houlsby and Teh (1991) using the strain path method.

The cone factors from international sites using (CAUC) triaxial test results are in the range of 8 to 27. This wide range can possibly be explained by:

- the existence of fissures in the clays, as failure in triaxial tests is liable to happen in the fissures, whereas CPT tends to measure the intact strength in between the fissures.
- the cone factor dependency on the rigidity index $L$, which can change the value of $N_{18}$ from 7 to 11 by an increase in $L$, from 10 to 250 according to the solution proposed by Vesic (1975).
- the use of specimen height twice the diameter in the triaxial tests, as the use of triaxial specimens $H = 2D$ in non-fissured dilating soils may cause cone factors in the range of 15 due to non-uniform deformation of the triaxial specimen which can result in smaller $c_u$ values than those measured by CPT.

6. FUTURE WORK
It should be verified whether the cone factor estimated from Danish triaxial tests can be taken as $N_{18} = 10$ even in soils with very high or very low rigidity indices.

The importance of strain rates in the CPT on the measurement of $c_u$ should be examined.

The significance of specimens with $H = 2D$ compared to specimens with $H = D$ when estimating the undrained shear strength in triaxial tests should be verified, especially in dilating soils.
Fig. 5: Outline of a triaxial test performed with a) Homogeneous stress and strain conditions, zone failure occurs. b) Inhomogeneous stress conditions as a result of rough pressure heads. c) Small rupture zone can occur under non-uniform strain conditions using smooth pressure heads. d) Small rupture zone can occur under non-uniform strain conditions for dilating soils using rough pressure heads.

Fig. 6: The consequence of an undrained test performed on a specimen with $H = 2D$ on dilating soil. a) Development of the locally weakened zone in which the shear plane is subsequently formed. b) The drained zone dominates the performance curve. c) Two practically solid bodies sliding past each other.

REFERENCES


CPT determination of overconsolidation ratio and lateral stresses in clean quartz sands

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SYNOPSIS: An assessment of CPT calibration chamber data compiled from 26 different test series on uncemented clean quartz sands provides a basis for estimating the approximate magnitude of in-situ horizontal stress state and degree of overconsolidation. An interrelationship between Kc and OCR must be assumed a priori. The methodology is applied to six field sites for comparison where either the determination of lateral stress by pressuremeter or the profile of preconsolidation stress via geologic inference has been made independently.

1. INTRODUCTION
The interpretation of CPT data to indicate the geostatic stress state in sands is a desired facet of site characterization. Methods of assessing Kc in sands from tip resistance (qT) readings have been proposed independently by Schnaid & Houlasby (1994) and Manassero (1994), but both require the limit pressure (pL) from pressuremeter tests. An approach by Massod & Mitchell (1993) uses sleeve resistance (qL) data, but necessitates an evaluation of OCR beforehand. In this paper, an empirical expression is derived from CPT calibration chamber test data on uncemented clean quartz sands that relates the applied lateral stress \( \sigma_{L} \) to \( q_{T} \), OCR, and Kc. For one-dimensional loading and unloading, an assumed Kc-OCR relationship can be utilized to find \( \sigma_{L} \) by iteration.

The methodology is based on the widely-known observation that \( q_{T} \) depends strongly on \( \sigma_{L} \). For example, Fig. 1 shows combined CPT calibration chamber data for Hoksund sand (Parkin 1988) and Ticino sand (Bellotti et al. 1993) for a relative density \( D_r = 90 \) percent. A unique trend appears between \( \sigma_{L} \) and \( q_{T} \), independent of OCR and applied boundary conditions. In addition, the \( \sigma_{L} \)-\( q_{T} \) relationships have been shown to be independent of initial lateral stress ratio, \( K_c = \sigma_{L0}/\sigma_{L} \) (Houlasby & Hitchman 1988). For this, Parkin (1988) suggested the empirical expression:

\[
q_{T} = A (\sigma_{L0})^B
\]
where $A$ varies with relative density and $B = 0.5$ to 0.6 for quartzitic sands. Here, however, the calibration chamber data were not corrected for boundary effects.

2. CHAMBER DATABASE TRENDS

The generality of the $q_s' - q_s$ relationship for clean quartz sands is shown in Fig. 2 using a compiled calibration chamber database from 24 different test series (Kulhawy & Mayne 1990) plus two additional sets of recent data (Fioravante et al. 1991; Rix & Stokoe 1991). The $q_s$ data were corrected for boundary effects due to flexible chamber walls of limited extent (Mayne & Kulhawy 1991). Regression statistics ($n = 703; \, r^2 = 0.740; \, S.E. = 0.276$) give the expression:

$$q_{s'} = (q_s)^{1.6} \left(1.144 - 0.0186D_r\right)$$  \hspace{0.5cm} \text{(2)}$$

in which $q_{s'}$ is in kPa, $q_s$ in MPa, and $D_r$ reported in percent.

A more traditional use of CPT data in sands is the evaluation of $D_r$ (e.g., Robertson & Campanella 1983). Analysis of the chamber test database indicated the following relationship (Kulhawy & Mayne 1990):

$$D_r (\%) = 100\left(q_{s'}/(305 \, OCR^{-0.2})ight)^{0.5}$$  \hspace{0.5cm} \text{(3)}$$

where $q_{s'} = (q_s)/(\sigma_{u'}/\rho_s)^{0.3} = \text{normalized cone tip resistance corrected for overburden stress and } \rho_s = \text{reference stress } = 1 \, \text{atm } = 100 \, \text{kPa. This normalization format has been used previously (Parkin 1988; Bellotti et al. 1989), although recent work by Olsen (1994) suggests that the exponent for } \sigma_{u'} \text{ actually varies inversely with } D_r.$

A difficulty with the above is that a profile of unit weights ($\gamma$) consistent with $D_r$ must be adopted prior to calculating $q_{s'}$ and $D_r$. To circumvent this, a direct relationship between void ratio ($e_r$) and $q_{s'}$ was sought, as suggested by Ghionna & Jamieson (1991). Thus, the database was reevaluated to give the trends shown in Fig. 3 and summarized by:

NC: ($n = 494; \, r^2 = 0.668; \, S.E. = 0.053$)

$$e_r = 1.159 - 0.230\log(q_{s'})$$  \hspace{0.5cm} \text{(4)}$$

OC: ($n = 149; \, r^2 = 0.820; \, S.E. = 0.035$)

$$e_r = 1.232 - 0.245\log(q_{s'})$$  \hspace{0.5cm} \text{(5)}$$

ALL: ($n = 643; \, r^2 = 0.691; \, S.E. = 0.049$)

$$e_r = 1.152 - 0.233\log(q_{s'}) + 0.043\log(OCR)$$  \hspace{0.5cm} \text{(6)}$$

For these and subsequent statistics, only data where internally consistent sets of $\gamma_r$-OCR values were considered. That is, calibration test results for NC sands with arbitrary stress states (e.g., $K_o = 2$) were not considered.

Use of (4) or (5) is easily accomplished via manual iteration mode on a spreadsheet. For clean sands with no capillarity, a dry unit weight $\gamma_d = G_{s'}/(1+e_r)$ is assumed for depths above the water table, and saturated unit weight $\gamma_{s'} = (G_s + e_r)\gamma_s/(1+e_r)$ taken below the water table. By the conclusion of this paper, the utilization of (6) will become evident. Using the appropriate unit weight, the total vertical overburden is calculated cumula-
tance, $q/[(\sigma_{w}^{'})^{3/2} \text{OCR}^{3/2}]$ for NC and OC sands. A trend is quite evident which can be represented (Mayne 1991) by the following cumbersome expression:

$$n = 590; \ r^2 = 0.871; \ S.E. = 0.214$$  \hspace{1cm} (7)

$$\frac{\sigma_{w}^{'}}{p_{a}} = \frac{(q/p_{a})^{1.6}}{145 \exp \left[ \frac{(q/p_{a})^{0.9}/(\sigma_{w}^{'}/p_{a})^{0.65}}{12.2 \ \text{OCR}^{0.5}} \right]}$$

An a priori relationship between $K_{w}$ and OCR must be assumed in order to solve (7). For calibration chamber data, the average trend has been determined (Mayne & Kulhawy 1994).

$$K_{w} = 0.428 \ \text{OCR}^{-0.414}$$  \hspace{1cm} (8)

However, it is unclear whether accurate $K_{w}$ values are obtained in such tests because only the average lateral strains are maintained near zero during consolidation. Consequently, the expression from Mayne & Kulhawy (1982) has been adopted henceforth:

$$K_{w} = (1 - \sin \phi) \text{OCR}^{\cos \phi}$$  \hspace{1cm} (9)

Furthermore, an evaluation of lab (Fig. 4) and field data suggests the alternative format:

$$\sigma_{w}^{'}/q = 1.33(q/\sigma_{w}^{'})^{0.22}(\sigma_{w}^{'})^{0.60} \text{OCR}^{-0.27}$$  \hspace{1cm} (10)

with $\sigma_{w}^{'}, \sigma_{w}^{''},$ and $q,$ in kPa and $\sigma_{w}^{''}$ in MPa. The effective stress friction angle ($\phi^{'}$) may be determined at each incremental depth using the method of Robertson & Campanella (1983):

$$\phi^{'} = \arctan[0.1 + 0.38 \log(q/\sigma_{w}^{'})]$$  \hspace{1cm} (11)

with $\phi^{'}$ (degrees) and $q,$ & $\sigma_{w}^{'},$ in same units. Thus, (9)-(11) provide an internally consistent set of equations for assessing both the in-situ horizontal stress ($K_{w}$) and preconsolidation ratio (OCR) for uncremented clean quartzitic sands. The effects of aging, mineralogy, and bonding are unfortunately not quantified, however.
3. APPLICATION TO FIELD DATA

The empirical procedure has been applied to six field cases involving sands where either: (a) the stress history has been independently assessed using geologic information or the results of one-dimensional consolidation tests on samples of interbedded clay layers that were retrieved from the site, or (b) lateral stresses have been evaluated via pressuremeter tests. Each of the sand sites is described briefly in following paragraphs. Where available, corrected $q_v$ resistances are used.

At McDonald Farm, British Columbia, the horizontal geostatic stress state has been assessed via self-boring pressuremeter tests (Sully 1991). In Fig. 5, the estimated profile of $\sigma_{ho}$ from $q_v$ data is seen to be comparable with lift-off stresses from SBPMT (given in total stresses, as reported).

Holmen, Norway has served as a test site for extensive cone calibration studies in normally consolidated sand (Lunne et al. 1986). Fig. 6 shows that profiles of $\sigma_{ho}$ are underpredicted by the $q_v$ data when compared to SBPMT lift-off values. The predicted OCRs were also low with deeper values $\leq 1$.

At Evanston, Illinois, sand fill was placed circa 1966 to form reclaimed land along Lake Michigan for Northwestern University (Finno 1989). Lift-off values from Menard-type pressuremeter tests (PMT) in prebored holes gave interpreted total horizontal stresses ($\sigma_{ho}$) that indicate moderately high $K_s$ values. Corresponding $K_s$ interpretations using CPT data from the site gave comparable values, as indicated by Fig. 7.

The Stockholm, Sweden site initially consisted of a glacial deposit of dry normally consolidated natural sand 24 m thick over bedrock. Excavation operations removed 16 m of overburden and the remaining 8 m of sand were subjected to a variety of in-situ tests (Dahlgren 1974; Mitchell & Lunne 1978). Using the aforementioned correlations, profiles of $\phi$, $K_s$, and OCR have been developed from the CPT data. In Fig. 8, results from two soundings are shown to be comparable with
been made by SBPMT (Ghienna et al. 1994).

Offshore soundings at North Sea Site B indicated very dense sands (Mitchell & Lunne 1978), results reportedly being very typical of the area. Inference of the magnitude of preconsolidation stress $\sigma_{pc}' = 1$ MPa was made from standard oedometer tests conducted on associated OC clay units from the region. Fig. 11 shows that the approximate interpretative procedure (eqns 9 to 11) gives a reasonable profile of effective yield stresses.

The proposed procedure remains empirical and preliminary at this time and should be verified by additional field case studies. Perhaps, improved assessments of the interrelationships between $K_s$, OCR, aging, cyclic loading history, and cementation will better formulate a means of accurately predicting geostatic stress states in sands.

4. CONCLUSIONS
Cone tip resistances in sands depend significantly on the effective horizontal stress regime. An assessment of CPT calibration chamber on uncemented quartz sands provides an empirical basis for relating $\sigma_{pc}'$ in terms of $q_u$, $\sigma_{mc}'$, and OCR, which may be solved if $K_s$ and OCR are assumed interdependent. Despite the neglect of aging and bonding effects, application of the method to several natural sand sites show credible results.
REFERENCES


First order estimate of yield stresses in clays by cone and piezocone.

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SYNOPSIS: The results of cone, piezoprobe, and piezocone tests can be used to provide a quick first-order estimate of the effective yield or preconsolidation stress ($\sigma'_y$) in uncemented and unstructured inorganic clays of low to medium sensitivity. The empirical methodology is based on statistical analyses of penetration data on intact clays and relates $\sigma'_y$ to the corrected tip resistance ($q_t$) and penetration pore water pressures measured either at the midface ($u_t$) or at the shoulder ($u_z$). Piezocone data from six clay sites in North America are used to illustrate this method.

1. INTRODUCTION

The yield or preconsolidation stress ($\sigma'_y$) in natural clays is an important indicator of stress history effects caused by erosion, glaciation, desiccation, groundwater fluctuations, aging, and geochemical alterations. Conventionally, the profile of $\sigma'_y$ is evaluated through a laborious effort involving high-quality sampling and one-dimensional consolidation testing. In some commercial laboratories, loads using dead-weight type devices are increased only daily, requiring up to two weeks for a single test specimen. An initial quick and approximate estimate of the magnitude of $\sigma'_y$ is therefore desirable and can be made directly from the results of in-situ tests.

Tavenas & Leroueil (1979) suggested the direct use of cone tip resistance ($q_t$) for profiling yield stresses in natural clays, whereby the correlation $q'_t = q_t/3$ was proposed for several sensitive clays of Eastern Canada. Also, a number of theoretical formulations have been derived to relate $q'_t$ to cone penetration test results. For example, Schmertmann (1978) related $q_t$ to overconsolidation ratio (OCR = $q'_t/\sigma'_o$) using a SHANSEP approach, and Wroth (1984) related OCR to the piezocone pore pressure parameter, $B_p = \Delta u_t/(q_t - \sigma'_o)$, by analogy with the triaxial pore pressure parameter, $A_p$.

Koerner & Law (1987) derived an effective stress interpretation of cone penetration that related $q'_t$ to effective tip resistance ($q_t$), soil friction angle ($\phi'$), interface friction ($\delta$), and penetration pore pressure ratio ($u_t/u_z$). An analytical method based on cavity expansion theory and Cam-clay (Mayne & Bachus, 1988) showed OCR to be a function of $(\Delta u/\sigma'_o)$, $\phi'$, rigidity index ($I = G/K_s$), and volumetric strain ratio ($\lambda = 1-\varepsilon/\lambda$), in which $G = \text{shear modulus}$, $K_s = \text{undrained shear strength}$, and $\varepsilon$ and $\lambda$ = isotropic swelling and compression indices, respectively.

Sandven (1990) used a limit plasticity theory to relate $q'_t$ with effective tip resistance and $\phi'$. Using a strain path simulation, Whittle & Aubeny (1993) showed that OCR was related to the normalized cone parameters ($q_t - \sigma'_o)/\sigma'_o$ and $\Delta u/\sigma'_o$.

The difficulties with these approaches for profiling $\sigma'_o$ by cone tests include: (a) need for soil information a priori; (b) uncertainty in
determination of appropriate values of input parameters (e.g., I, or G, etc.) for analysis; and (c) lack of adequate validation over a wide variety of soils. Consequently, an approximate empirical methodology is needed that can provide quick first-order assessments of $\sigma'_y$ from CPT results.

2. **PIEZOCONE STATISTICS**

A piezocene database in clays was initially compiled from a variety of sources where reference values of oedometer preconsolidation stresses were available. The data were obtained primarily from soft to firm to stiff clays of marine, lacustrine, glacial, fluvial, and alluvial origins, having wide ranges in plasticity, water content, and mineralogy. For these data, statistical regression analyses indicated the following mean relationships for intact clays (Kulhawy & Mayne, 1990):

**Electric Cones:**

$$\sigma'_y \approx 0.33(q_{sat} - \sigma_u) \quad (1)$$

($n = 74, r^2 = 0.904, \text{S.E.} = 1.02\sigma_p$)

**Type 1 Piezocones:**

$$\sigma'_y = 0.47(\Delta u_{cl}) \quad (2)$$

($n = 77, r^2 = 0.838, \text{S.E.} = 1.48\sigma_p$)

**Type 2 Piezocones:**

$$\sigma'_y = 0.54(\Delta u_{cl}) \quad (3)$$

($n = 68, r^2 = 0.827, \text{S.E.} = 1.12\sigma_p$)

in which $n$ = number of data sets, $r^2$ = coefficient of determination, S.E. = standard error, and $\sigma_p$ = atmospheric pressure.

Although the database included a few sensitive clay sites from Eastern Canada and Norway, it did not contain results from the highly organic clays and gyttja of Sweden (Larson & Mulabdić 1991) or the highly plastic Mexico City clays (Jaime & Romo 1988) because these data sets were unavailable at the time of compilation. For those soils, water contents ($w_u$) > 200% and plasticity indices ($I_p$) > 100 are common. Therefore, equations (1)-(3) should be considered applicable only to uncedented, unstructured, and relatively inorganic clays of low to medium plasticity ($I_p \leq 80$) with water contents ($w_u$) $\leq 100%$. Furthermore, a macrofabric of cracks and fissures in heavily overconsolidated clays will result in underestimates of $\sigma'_y$ by the above equations.

Multiple regression analyses of piezocene data have since been conducted with similar trends reported (Larson & Mulabdić 1991; Chen 1994), yet indicating a secondary trend inversely proportional to plasticity index ($I_p$) and/or liquid limit ($w_L$). Notably, the difficulty with these later approaches is the need to know $I_p$ or $w_L$ beforehand.

![Fig. 1. Piezocene sites examined.](image)

3. **APPLICATION TO CASE STUDIES**

To evaluate the general validity of this approach, the first-order estimation procedures were applied to data from six sites in North America, as listed in Table 1 and shown on Fig. 1. The piezocene results for all but one site (#2) were obtained after the statistical database had been compiled, and therefore they represent unbiased data sets. Three of the sites were tested using type 1 piezocones and three by type 2 piezocones. Only with type 2 piezocones are proper corrections possible (Mayne et al. 1990).

Figures 2-7 show the evaluations for each of the six sites and include $\sigma'_y$ estimated from equation (1), $\sigma'_y$ estimated from either (2) or (3), and $\sigma'_y$ evaluated from oedometer or other tests. Brief descriptions of each site are given below for reference. Comparisons of $\sigma'_y$ are discussed in the next section.
Table 1. Clay sites, average index properties, and sources of data.

<table>
<thead>
<tr>
<th>Site Number &amp; Location (Cone)</th>
<th>w_i</th>
<th>w_L</th>
<th>I_p</th>
<th>S_i</th>
<th>OCR</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Amherst, Massachusetts (2)</td>
<td>65</td>
<td>54</td>
<td>19</td>
<td>5-19</td>
<td>1-12</td>
<td>DeGroot &amp; Lutenegger (1994)</td>
</tr>
<tr>
<td>2. Washington, DC (2)</td>
<td>68</td>
<td>83</td>
<td>37</td>
<td>4-7</td>
<td>1-3</td>
<td>Mayne (1987)</td>
</tr>
<tr>
<td>4. Langley, British Columbia (2)</td>
<td>45</td>
<td>40</td>
<td>19</td>
<td>11</td>
<td>1-6</td>
<td>Campanella et al. (1988)</td>
</tr>
<tr>
<td>5. San Francisco, California (1)</td>
<td>55</td>
<td>65</td>
<td>30</td>
<td>2-7</td>
<td>1-1.5</td>
<td>Koutsofias (1989)</td>
</tr>
<tr>
<td>6. Baton Rouge, Louisiana (1)</td>
<td>34</td>
<td>60</td>
<td>33</td>
<td>N.A.</td>
<td>4-11</td>
<td>Chen (1994)</td>
</tr>
</tbody>
</table>

Note: Cone types for measuring pore water pressures: 1 = midface position; 2 = shoulder element.

Site 1 is one of three secondary level U.S. National Geotechnical Experimentation Sites (NGES) for conducting and comparing in-situ and lab test results. The site is underlain by a desiccated clay crust overlying soft normally consolidated varved clays of the Connecticut River Valley (DeGroot & Lutenegger 1994).

Site 2 is located at the confluence of the Potomac and Anacostia Rivers, and the alluvial clays have been the source of difficulty in excavation projects (Swanson & Larson 1990). The clays have been preconsolidated by slight erosion and aging effects (Mayne 1987).

Site 3 is another NGES property and is located along the eastern shore of Lake Michigan (Finno 1989). The site was formed in 1966 when a sand fill was placed above natural deposits of glacial lacustrine clays, thus forcing the materials into a normally consolidated state.

Site 4 consists of a Quaternary glacio-marine sequence of silts and clays in British Columbia (Campanella et al. 1988). The sediments are lightly overconsolidated with a desiccated crust.

Site 5 involves a subway excavation project near San Francisco Bay and is underlain by Young Bay Mud (Koutsofias 1989). The clays are relatively soft to firm.

Site 6 was selected as a test site for an NSF-funded research project on piezocones (Chen 1994). The property is located at the interchange of interstate I-10 and state route 42 in Baton Rouge, LA. Beneath a 4-meter thick sandy clay fill lies a thick (> 40 m) deposit of Pleistocene desiccated clay of deltaic origin. The uppermost 12 meters of clay are highly fissured and slickensided.

![Fig. 2. Site 1 comparisons.](image-url)
4. DISCUSSION

For Sites 1, 2, and 4 that were tested by type 2 piezometers, reasonable estimates of $\sigma'_{vr}$ were obtained by the independent $q_d$ and $v_d$ readings. At Site 2, the large correction for net area ratio ($a = 0.60$) results in a small overprediction based on the $q_d$ readings and a small underprediction for the $v_d$ interpretation. This could result from use of a larger 15 cm² tip cone.

For the clay sites tested by type 1 cones, somewhat conservative predictions were obtained from the tip resistance readings. In the soft clays (Sites 3 and 5), this is likely because of the uncertainty involved in correcting $q_v \rightarrow q_d$ since an assumption regarding the conversion ($k_c = v_d / u_d$) must be made for type 1 cone data (Lunne et al. 1986). For the stiff clay (Site 6), the underprediction in the desiccated zone results from the presence of fissures in the upper 12 m. Predictions based on the $v_d$ measurements were reasonable for Sites 5 and 6, while they somewhat overestimated at Site 3.
As shown by Figures 2-7, reasonable first-order estimates of yield stress are obtained from either type 1 or type 2 piezocene data. The case studies presented include natural clays with magnitudes of \( \sigma'_y \), spanning a large range from 50 to 1200 kPa. Equations 1-3 also have apparently proved useful in other deposits as well (Leroueil & Jamilokowski 1991).

Moreover, improved reliability can be obtained if site-specific correlations are developed for localized geologic formations and clay deposits. For example, in lieu of the average statistical relationship between \( \sigma'_y \) and \( \Delta u_i \) in equation (2), localized trends have been reported for plastic Swedish clays (Larsson & Mulabdic 1991):

Swedish clays: \( \sigma'_y = 0.29(\Delta u_i) \) (4)

Organic gyttja: \( \sigma'_y = 0.21(\Delta u_i) \) (5)

Similarly, for the sensitive marine clays of Eastern Canada (e.g., Champlain Sea), the general relationship between \( \sigma'_y \) and \( \Delta u_i \) given by equation (3) should be replaced by the following (Mayne 1988):

Champlain clays: \( \sigma'_y = 0.48(\Delta u_i) \) (6)

Better estimates might also be obtained using multiple regression statistics that include additional factors such as \( w_i \) (Larsson & Mulabdić 1991) or \( l_i \) (Chen 1994). However, this type of information is not normally known a priori when testing a given site.

5. CONCLUSIONS

The direct results of cone and piezocene tests can provide approximate profiles of \( \sigma'_y \) in intact, unconfined, unstructured clays. General statistical trends are applied to six case studies in North America. Improved estimates are obtained with localized correlations for site-specific geologies, and caution should be exercised in the use of empirical trends for special soils, such as highly organic or plastic clays and structured or cemented soils.
6. REFERENCES


Soil classification with cone penetration test in Tertiary Clays

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Wojciech Tsuchacke
Agricultural University, Poznań, Poland

SYNOPSIS: The engineering properties of a clay is very dependent on the geological processes that have taken place. It is a significant task to identify this from soil investigations. Such a subsoil is glacioregionally disturbed Tertiary clay. In case of strength tests carried out by means of penetration technique, interpretation of their results requires analysis of the subsoil glacioregionics. Physical parameters and grain size distribution of glacioregionically disturbed clays can be determined from CPTU measurements in a similar way as of those where no such disturbances were found.

1. INTRODUCTION
In the analysis of subsoil stability for construction purposes it is necessary to determine the range of layers with the lowest load capacity and to establish shear strength parameters of the soil forming these layers. This problem appears in case of strongly elevated Tertiary clay roof covered with sediments from water - glacial accumulation, particularly when such clays were disturbed glacioregionically. On one hand heavy overconsolidation of the clay and related high undrained shear strength guarantee appropriately high load capacity of these soils. On the other, however, sicken sides resulting from glacioregionical disurbances cause lowering of shear strength. Traditional test techniques for this kind of clay require performing standard boreholes, taking appropriate quality samples and carrying out laboratory strength tests. Recent development of modern in-situ tests resulted in an attempt at introducing one of them - cone penetration test - to problems concerning glacioregionically disturbed soils. The results of piezocone tests were used to establish stratigraphy and classification, to determine state of consistency and to assess shear strength parameters of these clays.

2. TEST SITE AND EQUIPMENT
The test site involved northern fragment of the Silesian Lowland and central part of the Wielkopolska Lowland, both regions included in Poznań - Wrocław basin of Tertiary Poznań series. The basin was formed by the end of Upper Miocene and the beginning of Pliocene in north - southern part of Poland. In-situ tests consisted of 120 piezocone penetration tests (CPTU) located in rectangular grid and taking undisturbed soil samples from depths selected based on the CPTU results. The cone penetration tests were carried out in the depth range from 13 to 37 m with a Hyson 12 Tp penetrometer (van den Berg). A standard piezocone with cross - section area of 10 cm$^2$ was used with a metal filter placed directly behind cone tip. During penetration three measurements were recorded continuously: cone resistance - $q_c$, sleeve friction - $f_s$, and pore water pressure - $u$ (Fig 1). Samples for laboratory tests were taken with Mostap 65 sampler (van den Berg). The laboratory examinations determined grain size distribution,
basic physical parameters of the analysed soils, Atterberg’s limits and shear strength parameters of selected clay samples. The liquid and the plastic limits according to Casagrande’s procedure were determined to assess the state of consistency of the cohesive soils described by the liquidity index - \( I_l = w_p - w_l \). Evaluation of this parameter was verified in parallel tests using the “fall - cone” test according to (Polish standard, 1988). Shear strength parameters of the soils was determined from the consolidated undrained compression triaxial tests using the samples of 65 mm diameter.

3. CHARACTERISTICS OF THE TERTIARY CLAYS

The clay of the Poznań series, as mentioned earlier, were formed in a wide, shallow water reservoir covering almost all Polish Lowland. Southern range of the reservoir of the Poznań clay reaches the verge of the Sudety Mountains and Mid Poland Highland. Due to erosion processes, particularly those related to Riss glaciation the formations of the Poznań series were exposed and modification of the parameters of these soils started (e.g. due to glaciotectonics). A characteristic of Tertiary formations appearing on the studied area is a morphologically diversified roof. In places the roof of the Poznań clays is found close to surface but in the neighboring sites it can fall vertically downwards forming troughs and chutes. These erosive forms are often filled with Quaternary sediments water content. With respect to grain size the Tertiary clays are mainly represented by clays and silty clays. It is important to notice considerable variability of volumetric density and moisture content of the clays, especially in this part of the subsoil which was glaciotectonically disturbed. Such a variability of these parameters reflects their geological history, and particularly glaciotectonic disturbances, postglacial erosion and later erosion processes in the surface zone of the Tertiary formation. Also the change in clay grain size with depth is very distinct, especially when it is expressed by the clay fraction content. The state of consistency of the clays indicated lower differentiation with depth from soft to very stiff. However, the highest differentiation was observed in the assessment of the clay shear strength parameters, particularly concerning the samples which

### Table 1. Statistical data of physical properties, shear strength parameters and CPTU measurements for Tertiary clay

<table>
<thead>
<tr>
<th>Statistical parameters</th>
<th>Clay content ( f_c ) [%]</th>
<th>Moisture content ( w_c ) [%]</th>
<th>Liquid limit ( w_l ) [%]</th>
<th>Plastic limit ( w_p ) [%]</th>
<th>Liquidity index ( I_l ) [-]</th>
<th>Effective friction angle ( \phi^* ) [%]</th>
<th>Cohesion ( C ) [kPa]</th>
<th>Net cone resistance ( q_c ) [MPa]</th>
<th>Pore pressure parameter ( B_s ) [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>N = 28 Tertiary clay without glaciotectonic slicken sides</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \bar{x} )</td>
<td>45.8</td>
<td>24.9</td>
<td>68.7</td>
<td>22.9</td>
<td>0.039</td>
<td>11.2</td>
<td>51.1</td>
<td>2.188</td>
<td>0.018</td>
</tr>
<tr>
<td>( x_{max} )</td>
<td>69.0</td>
<td>48.0</td>
<td>104.2</td>
<td>31.6</td>
<td>0.381</td>
<td>24.3</td>
<td>98.0</td>
<td>6.550</td>
<td>0.143</td>
</tr>
<tr>
<td>( x_{min} )</td>
<td>25.5</td>
<td>15.8</td>
<td>46.1</td>
<td>18.5</td>
<td>-0.143</td>
<td>6.0</td>
<td>12.0</td>
<td>0.389</td>
<td>-0.018</td>
</tr>
<tr>
<td>S</td>
<td>12.32</td>
<td>7.64</td>
<td>14.15</td>
<td>3.12</td>
<td>0.119</td>
<td>3.73</td>
<td>24.7</td>
<td>1.410</td>
<td>0.032</td>
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<tr>
<td>N = 38 Tertiary clay with glaciotectonic slicken sides</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \bar{x} )</td>
<td>54.1</td>
<td>23.9</td>
<td>88.7</td>
<td>27.0</td>
<td>0.029</td>
<td>9.4</td>
<td>37.3</td>
<td>1.722</td>
<td>0.012</td>
</tr>
<tr>
<td>( x_{max} )</td>
<td>93.0</td>
<td>56.3</td>
<td>136.0</td>
<td>38.0</td>
<td>0.187</td>
<td>13.5</td>
<td>72.0</td>
<td>3.206</td>
<td>0.067</td>
</tr>
<tr>
<td>( x_{min} )</td>
<td>20.0</td>
<td>19.6</td>
<td>60.2</td>
<td>18.3</td>
<td>-0.080</td>
<td>6.5</td>
<td>6.0</td>
<td>0.755</td>
<td>-0.028</td>
</tr>
<tr>
<td>S</td>
<td>19.81</td>
<td>6.91</td>
<td>19.24</td>
<td>4.52</td>
<td>0.066</td>
<td>1.75</td>
<td>19.4</td>
<td>0.682</td>
<td>0.021</td>
</tr>
<tr>
<td>t</td>
<td>1.94</td>
<td>2.21</td>
<td>4.61</td>
<td>4.06</td>
<td>0.42</td>
<td>2.60</td>
<td>2.52</td>
<td>1.77</td>
<td>0.78</td>
</tr>
<tr>
<td>( I_{L,F,T} )</td>
<td>1.98</td>
<td>1.98</td>
<td>1.98</td>
<td>1.98</td>
<td>1.98</td>
<td>1.98</td>
<td>1.98</td>
<td>1.98</td>
<td>1.98</td>
</tr>
</tbody>
</table>

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were taken from the subsoil zone with glacioclimatic slicken sides. Table 1 presents statistical characteristics of the analysed geotechnical parameters of the clays distinguishing those where glacioclimatic slicken sides were observed and the clays without such disturbances.

![Diagram](image)

Figure 1. Typical piezcone record in glacioclimatically disturbed clay formation.

4. STRATIGRAPHY AND SOIL CLASSIFICATION

In order to determine in the subsoil zones of Tertiary clay occurrence from the CPTU results several soil classification systems established for natural soils were used (Senneset et al. 1989, Robertson 1988, Larsson and Mulabdic. 1991). The diagramme analysis indicated that the Larsson - Mulabdic’s system is in good agreement and high discrimination capacity between laboratory results of grain size distribution and CPTU measurements (Fig. 2). A necessary condition for application of this system for heavily overconsolidated Tertiary clays, i.e. the soils with over 30% clay content, is fulfillment of the condition related to the third penetration characteristics, i.e. friction ratio \( R_c \) higher than 5.7. However, appearing on the diagramme areas for glacioclimatically disturbed clays and those without observable glacioclimatic slicken sides, indicate that in the subsoil strongly differentiated with respect to grain size and consistency, the results of cone penetration test do not differentiate clays with respect to disturbances. A confirmation of this hypothesis can be the result of statistical analysis. In order to establish variability between grain size and moisture content and CPTU measurements of the two analysed clay groups a zero hypothesis was made about lack of difference between mean values of the parameters. Confirmation of the zero hypothesis clearly proves that the glacioclimatic disturbances in the clays or lack of them do not cause changes in grain size and consistency of the soil and do not affect significantly CPTU measurements. Hence, practically, no measurable factor determining discrimination of glacioclimatically disturbed zones in the clay complex was found (Table 1).

To determine the effect of clay consistency
on CPTU measurements, empirical relationships were established in which clay state of consistency described by liquidity index - \( I_L \) is a function of net cone resistance (\( q_c - \sigma_{vo} \)) and pore pressure parameter (\( B_h \)). From the analysis of the issue results that a similar equation of simple multiple correlation describes this relationship in two considered clay groups (Fig. 3).

\[
I_L = 0.045 - 0.022(q_c - \sigma_{vo}) + 2.428 B_h \\
\text{where:} \quad R = 0.84
\]

\[
I_L = 0.116 - 0.056(q_c - \sigma_{vo}) + 0.722 B_h \\
\text{where:} \quad R = 0.74
\]

In the glaciotectonically disturbed clays (equation 2) as compared to those without observable glaciotectonic slicken sides (equation 1), the changes of the cone resistance better describe the changes of soil state of consistency than changes of \( B_h \) parameter.

Checking the significance of multiple correlation on basis of estimated regression coefficient - \( R \) indicated that at the level of \( \alpha = 0.05 \), the values of determined statistics are higher than the critical ones which proves a significant correlation between the analysed variables (Table 1).

5. SHEAR STRENGTH PARAMETERS OF THE CLAYS

Shear strength parameters determined from cone penetration test are related to the state of stress, stress history and mineralogy of soil. During the penetration the deformation of soil below the cone does not prefer some surfaces along which shear could take place but rather it is presented as a mean strength of a limited soil space around the cone. In a classical triaxial compression test shear usually takes place along favoured surfaces which, in case of glaciotectonically disturbed clays, are slicken sides. Hence, a question arises if the shear strength parameters determined from CPTU take into account local structural changes of the subsoil or, in other words, if the cone is sensitive to glaciotectonic disturbances. To answer this question appropriate correlation relationships were constructed and statistical analysis was carried out. As it has been mentioned earlier, it was indicated that for the analysed soils there are no statistically significant differences between mean values of the CPTU measurements from the glaciotectonically disturbed clays and those without observable glaciotectonic slicken sides. A similar zero hypothesis was made concerning the shear strength parameters from the triaxial compression test. This zero hypothesis was rejected for shear strength parameters what proves that the differences in evaluation of these parameters for the two analysed clay groups are statistically significant (Table 1). From the statistical analysis results the following practical conclusion arises: from CPTU results it is impossible to predict degree of subsoil weakening due to glaciotectonic disturbances. As in case of analysis of clay consistency,
empirical relationships were established with the simple multiple correlation method, between cone resistance and shear strength parameters from the triaxial compression test.

\[ q_t - \sigma_u = -1.42 + 10.41 \tan \phi' + 30.42 - C \]  
where: \( R = 0.79 \)  

\[ q_t - \sigma_u = 0.50 + 1.52 \tan \phi' + 24.60 - C \]  
where: \( R = 0.66 \)  

The solutions obtained confirm an earlier thesis that a relationship of other kind expressed in the function of cone resistance describes shear strength of the clays subjected in the part to glaciotectonical processes (equation 4) than those which were not (equation 3), (Fig. 4). Also the results of analysis of multiple correlation significance carried out on basis of estimated regression coefficients. For the clays classified in the group „without glaciotectonic slicken sides - \( R = 0.79 \)" the value of determined statistics, at the level \( \alpha = 0.05 \), is higher than the critical one indicating that the correlation of this kind is statistically significant. In the glaciotectonically disturbed clays \( R = 0.66 \) the results of statistical analysis was different. The reason for poorer correlation between the variables is a random character of glaciotectonic slicken sides both concerning their spacing and orientation, determining destruction mechanism and, consequently, about the angle of internal friction and cohesion values.

6. CONCLUSIONS

The results of in-situ tests carried out on the area with strongly elevated roof of Tertiary clays indicated great usefulness of cone penetration test for subsoil stratigraphy. Three penetration parameters registered simultaneously excellently differentiate subsoil with respect to soils forming it both concerning their grain size distribution and physical parameters. Hence, the following classification systems can be used: Senneset's, Robertson's, or Larson - Mulabdic's, particularly the last one while introducing a limitation for the third penetration characteristics - friction ratio.

Another important aim of the paper was to investigate the effect of glaciotectonic disturbances in clays on physical and shear strength parameters determined from CPTU measurements. Analysis of this problem indicated that:
- the state of consistency of clays expressed by liquidity index does not depend on the degree of glaciotectonic disturbances and can be estimated on basis of CPTU measurements from equations 1 and 2;
- it is impossible to clearly determine clay shear strength parameters from CPTU not knowing subsoil glaciotectonics. Random character of this phenomenon causes that at the same cone resistance strength of glaciotectonically disturbed clays can be lower by 50% as
compared to clays without the disturbances. From the analysis carried out it was found that for Tertiary clay layers without glacioteconic slicken sides the shear strength parameters can be estimated on basis of cone resistance according to equation 3, while for glacioteconically disturbed clays according to equation 4.

7. REFERENCES


Conductivity piezocone penetration test for evaluation of soil contamination

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SYNOPSIS: Evaluation of soil contamination around the Żelazny Most tailings dam which accumulates post-flotation waste from copper mine, is of particular importance for environment surrounding the reservoir. Application of conductivity cone enabled continuous registration of the soil conductivity with depth. Calibration of the cone was carried out in the Department of Geotechnics of Poznań Agricultural University. The study project was carried out in the vicinity of the Żelazny Most Reservoir in Lubiń, Poland. Results of the testing program enabled determination of accuracy of conductivity cone tests and precision of groundwater contamination assessment. The paper presents analysis of the factors affecting registered values of the subsoil conductivity and possibilities of application of piezocone test to identify these factors in the subsoil surrounding the tailings dam.

1. INTRODUCTION

Among many techniques for assessment of the degree of subsoil contamination the most favoured are those which yield continuous picture of contamination change with depth. This can be obtained from cone penetration test if conductivity cone or envirocone are used in tests. For interpretation results is important if the cone can simultaneously register standard CPT data: changes in cone resistance, sleeve friction and pore pressure with depth. If these values are known then some of the factors determination about the value measured by conductivity cone - bulk resistivity of the soil, can be identified.

Identification of the factors affecting registered values of electrical resistivity of the soil is discussed in several reports, e.g. Campamella and Weemnes (1990), De Graaf and Zuidberg (1985), Horne (1988).

To describe electrical conductivity of subsoil in the neighbourhood of the tailings dam relationships were used which constitute the basis of interpretation of the degree of contamination in natural soils. The electrical resistance is determined in the following way:

\[ R = \frac{U}{I} \]  

(1)

where: \( U \) - voltage

\( I \) - current

However, the measured resistance is not a unique material property but is a function of the cross-sectional area (A) and length of the electrical conducting material being measured (L). Resistivity - \( \rho \) can be defined as:

\[ \rho = \frac{A \cdot R}{L} = \frac{1}{C} \]  

(2)

The formula exposes the factors which affect the measured value of electrical resistivity of soil, namely the electrode geometry. Other factors which influence the measured bulk resistivity of the soil are: proportions between
bulk resistivity ($\rho_b$) and fluid resistivity ($\rho_f$), soil structure ($\theta$), viscosity of pore fluid ($\eta$) and pore fluid temperature ($T$) (Campanella and Weemans, 1990). Resistivity of the soil particles depends on clay mineral content ($f_c$), clay type ($f_c$), sand content ($f_s$) and silt content ($f_l$). From statistical point of view the measured bulk resistivity is a common function (equation 3) of several independent random variables.

$$\rho = f_1 (\rho_b, \rho_f, \theta, \eta, T)$$  (3)
$$\rho_f = f_2 (f_c, f_s, f_l)$$  (4)

The relationship which describes dependence of bulk resistivity on the above mentioned variables is most often assumed in its simplified version which is known as Archie's formula (Telford et al., 1976):

$$F = \left( \frac{\rho_b}{\rho_f} \right) = \alpha \cdot \mu^{\beta}$$  (5)

where: $F$ - formation factor.
$\mu$ - porosity

Hence this relationship is a particular solution of the equation 3. The value of the $\alpha$ coefficient depends on the degree of subsoil overconsolidation. Jackson et al. (1978) and Tonks et al. (1993) determined effect of particle size distribution and porosity on $\alpha$ coefficient. Tonks et al. (1993) presents equation 3 in the following form:

$$C_{\mu} = F \cdot C_c$$  (6)

where: $C_{\mu}$ - conductivity of pore fluid,
$C_c$ - cone conductivity (bulk conductivity of soil/water as measured by the cone).

In this notation coefficient $F$ accumulates a group of factors which affect both parameters $\alpha$ and $\mu$. Determination of the value of $F$ coefficient for genetically differentiated subsoil within the tailings dam was one of the aims of this project.

2. CONE CALIBRATION, TESTING PROCEDURES

In-situ tests were carried out with static penetrometer Hyson 12 TF (A.P. van den Berg) with conductivity cone (Fig. 1). The base area of the conductivity cone was 15 cm$^2$. The measuring system of the penetrometer enables registration of electrical conductivity within the range from 0 to 400 ms. The measurement is carried out at the frequency level of 2000 Hz. The cone is pushed into the subsoil at the rate of 2 cm/s, and electric conductivity $C_c$ is registered every 2 cm. The tests were made in the vicinity of the Żelazny Most tailings dam. On the tailings dam area the subsoil is differentiated. In the northern and eastern parts quaternary non-cohesive soils predominate and in the southern and central parts there are boulder clays on the foreland of front moraine. Deeper layers consist of the disturbed glaciectonically planose clays. The tailings dam constructed with hydroagglomeration method (Werno et al., 1993, Tschuschke et al.1995) accumulates post-flotation sediments and highly salinated water containing considerable amounts of sulfates and chlorides. Migration of water is possible through the bottom and embankments of the reservoir. Therefore the tests were carried out in the foreland of the embankments to determine the range of possible subsoil degradation.

![Figure 1. Van den Berg conductivity cone.](image)

Calibration conductivity cone tests are carried out in the laboratory of the Department of Geotechnics of the Poznań Agricultural University. Groundwater from the reservoir foreland was used in the test. Chemical analysis of the water was limited mainly to those elements and compounds which are included in...
the ionic balance. The calibration test was carried out in a chamber using different electrolyte concentrations. The electrical conductivity was recorded with a portable conductivity meter. The determined cone factor was in accordance with the value given by the manufacturer. In the replication test measurement accuracy was determined which amounted to 2% of the absolute value of the measurement (Fig. 2).

Figure 2. Results of the conductivity cone calibration tests.

3. TEST RESULTS, INTERPRETATION AND DISCUSSION

Figures 3 and 4 present examples of results from the conductivity cone test - CCPT, standard piezocone test - CPTU and distribution of conductivity in the subsoil in the foreland of the northern embankments of the Zelany Most Reservoir. To indicate differentiation of the degree of the subsoil contamination depending on the distance of the test point from the embankment there are results of tests made directly at the embankment (Fig. 3) and about 400 m away (Fig. 4).

It was stated in chapter 2 that pore fluid resistivity is a function combining several variables including porosity and, in case of cohesive soils, the effects of mineral surface conduction (clay mineral content and type).

The effect of porosity is taken into account in the Archie's formula (5) or in the notation (6). Current check of the effect of porosity or density on conductivity can be carried out using a standard CPT measurements - friction ratio $R_f$.

Figure 3. Results from conductivity and piezocone tests in the region of tailings dam area.

Figure 4. Results from conductivity and piezocone tests at the tests site 400 m from tailings dam.

A justification for assuming such an interpretation is the fact that the coefficient $R_f$ is
also a function of the following variables:

\[ R_f = f_n (n, f_n, f_c, f_l) \]  \hspace{1cm} (7)

Accepting the \( R_f \) coefficient for interpretation considerably simplifies analysis since it eliminates several independent variables. This kind of interpretation is given in Fig. 5.

![Figure 5. Relationship between friction ratio - \( R_f \) and formation factor - \( F \).](image)

From the relationship presented in figure results interesting observations: the pore water conductivity can be determined in two ways - by a priori assumption of formation coefficient, or determining it by means of defining common function which describes relationship between pore fluid conductivity and porosity, clay content, water temperature, etc. Moreover, from Fig. 5 results that the relation obtained formation factor is very similar to relationship determined from data for soils of other genetic origin (Tonks et al., 1993). The value of the formation factor changed within the range from 2 to 9. Campanella et al., (1990) determined formation factor for clayey silt from 4 to 5. This confirms the opinion of Campanella and the authors that pore fluid resistivity in clay is a more complex problem. This is well illustrated by geometrically presented function between formation factor and friction ratio - \( R_f \). Relationship of this kind describes well the effects of changes in soil grain size on the value of formation factor, however, it does not take into account the state of the tested soil. A good identifier of the soil state, i.e. its porosity, is cone resistance used for this purpose in classification systems for CPTU. The concept of presenting formation factor as a function of friction ratio - the variable indicating grain size distribution, and cone resistance - the variable related to the soil state is given in Fig. 6. The course of isolines on the diagramme was established experimentally for, limited at this stage of studies, number observations, hence this relationship has a character of tentative diagramme.

![Figure 6. Tentative chart to estimation of formation factor on the basis of penetration characteristics (the concept).](image)

4. CONCLUSIONS
Conductivity cone tests indicated that it is providing sufficiently accurate identification of the degree of groundwater contamination. The results from the conductivity cone test were very similar to laboratory groundwater analysis. There is great potential for the construction of conductivity cone which would enable simultaneous measurement of cone resistance, sleeve friction and pore pressure. Knowledge about these parameters will facilitate accurate determination of qualitative effect of factors on formation factor or better description of functions which define relationships between this coefficient and parameters of soil.
5. REFERENCES
Evaluation of the Wissa Piezocone in a Marine Clay Deposit

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SYNOPSIS: A Wissa piezocone was used to explore a soft sensitive marine silty clay at a research site at Pease Air Force Base along highway I-95 in Portsmouth, New Hampshire, USA. Testing included both continuous soundings and soundings with regular dissipation testing and was performed at two locations, an area with a Kp stress condition and an area at the toe of a 10.7 m high embankment (non-Kp). This paper presents results in terms of stratigraphy, soil classification, strength, stress history and consolidation characteristics which clearly demonstrate that the Wissa piezocone can be used overall for first approximations of several geotechnical parameters and for development of accurate local correlations.

1. INTRODUCTION
As part of an ongoing research program by the University of New Hampshire (UNH) involving several in situ testing tools, a site in Portsmouth, New Hampshire, USA was explored using a Wissa piezocone (Murray, 1995). The sensitive silty illitic marine clay had been previously studied for construction of a major interstate highway (Ladd, 1972). That work included construction of a 6.4 m high, 69 m long test embankment to failure of the clay stratum and extensive instrumentation to monitor settlements. More recent work involved self-boring pressuremeter, field vane and dilatometer testing (Atwood, 1991; Findlay, 1991; Neame, 1991). Exploration by piezocone presented the opportunity both to examine the capabilities of the piezocone in this well-characterized site, as well as to increase the body of information on the site.

The Wissa piezocone used for this testing program had standard dimensions of tip and sleeve, each with a load capacity of 44.8 kN (5 ton). The filter, located at the tip, was made of sintered stainless steel with a cylindrical shape and 7 micron pore size. The saturation fluid was deaired water, with the filters having been saturated after a 24 hour vacuum process. The data acquisition system, consisting of a 486 portable computer with an internal, low noise 16-bit A-to-D converter, was capable of sampling sensors of qsec, T, u, temperature and depth every 0.3 seconds. The piezocone was advanced at the rate of 2 cm/sec using a portable hydraulic drive system consisting of a hydraulic ram mounted atop a thin steel frame and anchored by four helical soil anchors.

The research site is located on the former Pease Air Force Base approximately 6 km from the ocean coastline and 80 km north of Boston, Massachusetts. The site is a swamp, about 80 m across, and bordered on the east by Interstate Highway 95 (I-95) and on the west by a bedrock outcrop.

The research at Pease was conducted at two adjacent areas, approximately 100 m apart, having slightly different characteristics. The first area is in the center of the swamp and designated as the Kp area (at-rest) due to its location outside the stress influence of the I-95...
embankment. The second area, designated as the embankment toe area or non-$K_s$ area, is at the toe of a 10.7 m high embankment for I-95 and is subject to the associated loading influence from the embankment.

2. SITE CHARACTERISTICS
The stratigraphy of the $K_s$ area consists of approximately 30 to 60 cm of surface water over a surficial layer of organic muck approximately 30 cm thick. Below this is a gray marine illitic silty clay extending to a depth of up to 8 m, below which is dense glacial till over bedrock. The top 1.5 to 2.5 m of the clay is weathered and stiff, while the lower portion is soft and highly sensitive. Interbedded within the soft clay are several thin lenses of silty sand. The stratigraphy at the non-$K_s$ area is similar to the $K_s$ area except that the water table is just below ground level, there are fewer lenses of coarse-grained soil, and the soft clay is approximately 1 m thicker.

At the $K_s$ area, the liquid limit of the soft clay ranges between 33.4 and 39.4% with a plasticity index between 12 and 16. The natural moisture content varies between 36 and 53%. The total unit weight ranges between 16.5 and 18.1 kN/m$^3$. Similar index properties are found at the toe area.

At the $K_s$ area, the overconsolidation ratio (OCR) decreases from approximately 10 in the stiff upper crust to close to normally consolidated below 4 m. At the toe area, the OCR starts again at about 10 in the upper crust and then decreases to normally consolidated at about 5 m.

Field data from piezometer observations during embankment construction (Ladd et al., 1972) indicated a vertical coefficient of consolidation, $c_v = 5.4 \pm 1.4$ m$^2$ per year with a horizontal value, $c_h$, approximately 1.5 to 2 times greater, while settlement data alone indicated a $c_v$ varying from 9.2 to 13.6 m$^2$ per year, the variation being a function of a possible drainage layer.

3. STRATIGRAPHY
Six piezocone soundings were conducted at the $K_s$ area and two at the non-$K_s$. Of those at the $K_s$ site, the first was mainly for equipment calibration, the second and third were continuous soundings, and the last three were performed over an extended period to carry out dissipation testing at each rod break. The two profiles at the non-$K_s$ site consisted of one continuous sounding and one sounding for dissipation measurements.

Figure 1 presents profiles for the $K_s$ and non-$K_s$ areas in the form of corrected tip resistance, $q_t$, penetration pore pressure, $u$, and corrected sleeve friction, $f_s$, versus depth. The $K_s$ area sounding begins at the predrilled depth of 0.30 m (0.82 m below the surface water level) and continues to refusal at 7.35 m. Four general strata are evident. Beneath the surficial layer of organic muck is a thin sand layer at 0.5 m indicated by a peak in $q_t$.

![Figure 1. Piezocone profiles.](image)

Below this layer, the resistance drops and then both $q_t$ and $u$ begin a rapid climb, indicating a dense, low permeability material, reaching maximum density between 1.2 and 1.4 m and extending to 1.5 m. This is a hard desiccated clay crust which has been described by prior researchers (e.g., Ladd, 1972; Findlay, 1991). Below 1.5 m, the point resistance sharply drops, signifying transition into the third stratum, the soft silty clay, which extends to a
depth of about 6.8 m. Average values in the soft clay are approximately $q_s = 0.250 \text{ MPa}$, $f_d = 0.02 \text{ MPa}$, and $u = 0.225 \text{ MPa}$. Within this clay stratum are lenses of coarser and more freely draining material at 2.3, 5.5, and 6.1 m. Several thinner lenses are also evident as indicated by smaller spikes in the graphs of $q_s$ and $u$. Below approximately 6.8 m, the clay appears to be increasingly mixed with coarser material in a gradual transition to glacial till.

In the non-$K_s$ area profile, the desiccated crust is apparent from just below the surface to approximately 2.1 m. Below this, the soft clay extends almost without interruption to refusal, except for one well-defined lens at 2.4 m.

While some differences are apparent between the two profiles, such as in the region of the upper crust and the frequent lenses in the $K_s$ area, the values in the soft clay are quite similar. Below the upper crust, each area shows a lens at 2.3 to 2.4 m, just after the transition to the soft clay. From 2.8 to 3.6 m, $q_s$ and $u$ for the non-$K_s$ area are slightly higher than those for the $K_s$ area, while $f_d$ appears to be slightly lower. However, between 3.8 and 5.0 m, no clear difference is evident. Although $f_d$ appears to be greater in the $K_s$ area, when comparing CPTU_M2 with CPTU_M7, the difference could be due to zero-load calibration, since the difference was not evident with the second sounding. Below 5 m, direct comparison is difficult because of the many lenses in the $K_s$ sounding, although the non-$K_s$ area values, especially $q_s$, are closely aligned with the baseline values of the $K_s$ area plot, i.e. between the sand lenses.

4. CLASSIFICATION

Figures 2 and 3 show classification plots for the $K_s$ area profile according to methods by Robertson (1990) and Larsson and Mulabdić (1991). Eighteen points were selected representing average measurements over the extent of the sounding. Of the 18 points, one corresponds to the desiccated crust at 1.18 m, 14 correspond to portions within the soft clay, and 3 correspond to readings within hard lenses near the bottom of the profile. Robertson (1990) employs a dual chart system, with a normalized $q_s$ versus normalized $R_u$ on the left chart and normalized $q_s$ versus $B_u$ on the right. All of the points corresponding to the sensitive clay plot in Zone 3 (clays - clay to silty clay) with the exception of one point which plots in Zone 1 (sensitive, fine grained) on the right chart. The three points corresponding to lower lenses plot in Zone 5 (sand mixtures) and Zone 6 (sands). The point corresponding to the upper crust at 1.18 m plots in Zone 6 (sands - clean sand to silty sand) on the left chart and outside all zones, but adjacent to Zone 6 on the right chart.

In general, this chart provides an excellent classification of the tested soil. Although the left chart gives a somewhat misleading indication of the soft clay being higher OCR and low sensitivity, this is probably due to the low friction resistance values, which could be significantly affected by small calibration errors. Any inferred parameters based on such low friction resistances should be regarded with caution. Therefore, for implied parameters such as OCR and sensitivity, the right chart should be given greater weight in a very soft clay and, in this case, did provide the correct indication.

Since the chart of Larsson and Mulabdić (1991) was prepared specifically for classification of clay soils, it is not surprising that the method appears to be quite accurate for the clays. The point corresponding to the upper clay crust (1.18 m) correctly classifies as stiff, overconsolidated or very silty clay. Otherwise, the points corresponding to the soft clay stratum plot within the zone for soft, low plastic and/or highly sensitive clays (8 points) or within the zone for (soft) normally consolidated or slightly overconsolidated clays (5 points). The three points corresponding to the hard lower sand lenses are not appropriate to this chart and therefore not shown. When plotted, they appear in the zones for stiff heavily overconsolidated, overconsolidated or very silty clay. Therefore, this method seems...
Figure 2. Classification at $K_v$ area according to the method of Robertson (1990).

Figure 3. Classification at $K_v$ area according to the method of Larsson and Mulabdić (1991).

to provide a reliable classification of the clay soil, indicating both the soft, low OCR, high sensitivity of the soft-clay and the stiff, overconsolidated, silty nature of the clay in the upper crust. Similar results were obtained for the non-$K_v$ area profile. It would seem, therefore, that these two classification methods work well together, the first providing a general identification of the clay soil and the second providing a clearer definition of the clay.

5. STRENGTH AND STRESS HISTORY

Figure 4 shows undrained shear strength, $s_u$, profiles for the two test areas using a method from Larsson and Mulabdić (1991). The method estimates the cone factor $N_u$ based on the liquid limit, $w_l$, such that $N_u = 13.4 + 6.65 w_l$, for use in the equation $s_u = (q_s - \sigma_{vo})/N_u$. The results compare favorably with the reference Geonor uncorrected field vane data.

Figure 4. Undrained shear strength profiles.
Maximum past pressures, $\sigma_0'$, were estimated by two published methods, namely by Larsson and Mulabidi (1991), where $\sigma_0' = \Delta u(2.05 + 2.62 \phi')$, and by Kulhawy and Mayne (1990), with $\sigma_0' = 0.33(\phi' - \phi_0)$. The influence of the many lenses in the $K_0$ area profile resulted in erratic estimates of $\sigma_0'$. The non-$K_0$ area profile was more consistent, though still somewhat irregular. It was found, however, that fitted curves from the data averaged out these inconsistencies and generally provided curves in fair agreement with laboratory oedometer data. Best fit curves of the non-$K_0$ area results are shown in Figure 5 and are compared with a best fit curve from oedometer testing. The method of Kulhawy and Mayne (1990), using $q_s$, provides close agreement with oedometer data in the soft clay, while the method of Larsson and Mulabidi (1991), using $\Delta u$, is in fair agreement in both the overconsolidated clay and the soft clay strata. Thus it would appear that this method can provide a reliable first approximation of $\sigma_0'$.

6. CONSOLIDATION PROPERTIES
Twenty-five full dissipation tests were conducted. The results were interpreted using the method of Teh and Houlsby (1991) and yielded a range of values of horizontal coefficient of consolidation, $c_h$, spanning two orders of magnitude. When the data for tests in the soft clay was sorted to remove tests influenced by local sand lenses, the range narrowed to $17 \leq c_h \leq 67 \text{ m}^2/\text{yr}$. This data was then found to separate into two distinct groups determined by the operational procedure used to arrest piezocone advance and initiate dissipation. When dissipation began without unloading the drill rod string, results ranged from 65 to 67 m$^2$/yr. When the rod string was unloaded by a quick reverse action allowing the pushing frame to settle back on the ground, the results ranged from 17 to 26 m$^2$/yr, comparing very well with field data described earlier. These results are shown in Figure 6. It would appear therefore that by careful examination of the profile, probable estimates of $c_h$ can be obtained.

![Figure 5. Preconsolidation stress profile.](image1)

![Figure 6. Horizontal coefficient of consolidation profile.](image2)
7. CONCLUSIONS
This research demonstrates the value of the Wissa piezocone test. The detailed stratigraphy provided insight for results of other in situ tests and identified drainage layers. Classification systems were accurate. Undrained shear strength, maximum past pressure, and horizontal coefficient of consolidation compared well with laboratory, in situ and field performance results.

8. REFERENCES
Prediction of Clay Strength using the combination of Cone and Sleeve Resistances

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SYNOPSIS. Historically, CPT based prediction of clay undrained strength was performed with a simple equation derived from bearing capacity theory. However, this formulation requires an bearing factor, $N_c$, which is dependent on clay type, silt content, overconsolidation level, and other factors. The only way to use this method was to back calculate $N_c$, based on uncompacted undrained (UU) strength tests used soil samples from nearby borings. A new technique was developed using CPT measurements to develop contours of normalized undrained strength ($c/p$) on the CPT soil classification chart. This technique does not require soil samples to estimate $N_c$, although confirmation tests are always a good practice. Contour plots of UU strength and field vane shear strength were established. Contours of the Bjerrum (1972) $\mu$ parameter were also established on the CPT soil classification chart, based on these strength contours which show that $\mu$ decreases with increasing silt content and overconsolidation level.

1. INTRODUCTION
Prediction of clay undrained strength ($S_u$) using the Cone Penetrometer Test (CPT) is one of the original CPT applications. Prediction of $S_u$ can be achieved with a simple relationship, based on CPT cone resistance and derived from simplified bearing capacity equations. However, this relationship has a bearing factor which must be estimated or predetermined using measured strengths. A new technique will be described which uses both CPT measurements (i.e., cone and sleeve friction resistances) to determine the triaxial unconsolidated undrained strength and field vane shear strength of clays and clayey silts for normally consolidated and over consolidated conditions.

2. DEVELOPMENT OF THE BEARING FORMULATION
The basic limit equilibrium bearing capacity equation (Terzaghi & Peck, 1967) (Equation 1) can be used to derive the historical equation for CPT prediction of clay undrained strength, $S_u$

$$ q_u = c N_c \zeta + \sigma N_N \zeta + \frac{B_T}{2} N_y \zeta $$

$$ S_u = \frac{q_u - q_{sv}}{N_c} $$

where

- $q_u$ = Bearing stress
- $N_c, N_N$ = Bearing factors
- $q_{sv}$ = Measured cone resistance
- $B_T$ = Total vertical stress
- $\zeta$ = Clay undrained strength
- $N_c$ = Cohesion bearing factor the CPT

The $N_c$ bearing factor to predict the triaxial unconsolidated undrained (UU) strength typically ranges from 9 to 13 for soft normally consolidated clays and 14 to 17 for stiff and/or slightly over consolidated clays (Aas, Lacasse, Lonne, and Hoeg (1984); Olsen, 1995). These $N_c$ ranges are only generalizations. The heat means of determining $N_c$ is with nearby soil
samples which have been tested for UU strength in the laboratory. However, it defeats the purpose of predicting strength from CPT when \( N_k \) must be determined by laboratory testing.

The first step towards a new formulation for CPT prediction of clay strength is normalization of the bearing formulation. When both sides of Equation 2 are divided by vertical effective stress, the resulting equation is:

\[
\frac{S_e}{\sigma_v} = \left( \frac{q_e - \sigma_{atm}}{\sigma_v} \right) \frac{1}{N_k} \quad (3)
\]

where

- \( \sigma_v \) = Vertical effective stress
- \( q_e \) = Measured cone resistance

The normalized undrained strength on the left side of Equation 3 is also the classical clay c/p ratio which can also be expressed as \( S_{ud} \) using stress normalization concepts (Olsen, 1994):

\[
\frac{S_{ud}}{\sigma_v} = \frac{N_k}{p} = S_{ud} \quad (4)
\]

For reference, the normalized cone resistance, \( q_{c,n} \) (Olsen, 1994; Olsen and Mitchell, 1995) (and normalized sleeve friction resistance, \( f_{s,n} \), and friction ratio) for all soil types and overburden stress conditions are defined as:

\[
q_{c,n} = \frac{q_c - \sigma_{atm}}{\sigma_v} \quad (5)
\]

\[
f_{s,n} = \frac{f_s}{\sigma_v} \quad (6)
\]

\[
FR = \left( \frac{f_{s,n}}{q_{c,n}} \right) 100 = \frac{100}{q_{c,n}} \quad (7)
\]

where

- \( q_c \) = Measured cone resistance (atm)
- \( \sigma_v \) = Vertical effective stress (atm)
- \( c, s \) = stress exponents
- \( f_s \) = Measured sleeve resistance (atm)
- \( FR \) = Normalized friction ratio

The 1 subscript signifies that normalization is to one atm (atmospheric pressure) (i.e., 100 kPa) vertical effective stress and the "e" indicates that a variable stress exponent was used for normalization calculations (Olsen, 1994). The stress exponent, \( c \), is approximately equal to 1.0 (one) for most normally consolidated clays, drops to 0.8 for slightly over consolidated clay, and can be as high as 1.2 for unstable silt clays. The sleeve friction resistance stress exponent, \( s \) (in Equation 6) is approximately equal to the cone resistance stress exponent, \( c \).

Adding a stress exponent, \( c \), to the vertical effective stress term on the right side of Equation 3 would make part of this equation equal to the normalized cone resistance (Equation 5). The inclusion of the stress exponent does not change the formulation because the stress exponent for normally consolidated clay can be assumed equal to 1.0 (one). Equations 3, 4 and 5 can now be combined to yield a new equation for determining the normalized undrained strength using the normalized cone resistance:

\[
q_{c,ne} = N_k S_{ud} \quad (8)
\]

3. DETERMINING \( S_{ud} \), USING THE CPT SOIL CHARACTERIZATION CHART

Equation 8 can now be combined with the normalized CPT sleeve friction resistance to determine clay strength without assuming an \( N_k \) value. Equation 8 can be represented in graphical form on the CPT soil characterization chart as illustrated in Figure 1. The \( q_{c,ne} \) (Equation 8) is equal to combined effect of \( S_{ud} \) contours and \( N_k \) contours. Over consolidation or higher silt content increases the \( S_{ud} \) contour level as shown by arrows and annotations in Figure 1. Therefore, any point on the CPT soil characterization chart (for an example point “A”) has a \( N_k \) contour value, a \( S_{ud} \) contour value, and a \( q_{c,n} \) value from the vertical axis. Contours of \( S_{ud} \) can be established directly on the CPT soil characterization chart using measured \( S_e \) data (i.e., laboratory Unconsolidated Undrained (UU) triaxial strength data \((S_e)_{triax}\) or field vane shear strength data \((S_e)_{vane}\) data) together with normalized CPT data. There is no need to determine \( N_k \) contours because \( S_{ud} \) is obtained directly.

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4. REMOVING BIAS DATA DURING DEVELOPMENT OF $S_u$ CONTOURS

Natural deposits are rarely uniform. Consequently, CPT measurements do not always correlate to a laboratory determined strength based on nearby soil samples. For example, a CPT sounding can probe a clay layer but a nearby soil sample may contain silt. A bias correlation will occur if silt samples are included when evaluating the strength of soft clay samples. The higher silt strength samples will bias the soft clay strength average too high. The objective is therefore to exclude as much bias data as possible.

The first step toward removing biased data is to subjectively rank all data based on stratigraphic uncertainty and CPT/laboratory data quality using the Academic Quality Index (AQI) (Olsen, 1994). The AQI is based on the academic college grading scale, i.e., excellent is 90 to 100 percent, good is 80 to 90 percent, and average is 75 percent. The stratigraphic AQI is a little more arbitrary, at a standard distance of 6 meters between a CPT sounding and boring, if 85 percent of the soil layers at a site/location are continuous than the stratigraphic AQI is also equal to 85 percent. This CPT-to-boring stratigraphic-based AQI will decrease as the CPT to boring distance increases. The total AQI is a combination of the data quality AQI and stratigraphic AQI. The AQI will be used in the next section to establish strength contours.

5. ESTABLISHING $S_u$ CONTOURS ON THE CPT SOIL CHARACTERIZATION CHART

Normalized undrained strength ($S_u$) contours were established on the CPT soil characterization chart using the procedure shown in Figure 1 together with the AQI technique (to account for bias effects). A very large database of CPT and laboratory data (consisting of 670 CPT soundings (15000 metres), 580 borings, and 1200 soil samples from 90 projects) were used to develop to final $S_u$ correlations. Two sets of $S_u$ contours were established using measured undrained strength data from the database: 1) unconsolidated undrained triaxial tests ($S_u, h_{vane}$, and 2) field vane shear test ($S_u, h_{max}$). An example contour for ($S_u, h_{vane}$) equal to 0.25 (range of 0.2 to 0.28) is shown in Figure 2 and an example contour ($S_u, h_{max}$) equal to 0.31 (range 28 to 34) is shown in Figure 3. These contours would have plotted at a higher $q_{cap}$ level if all of the data were used to establish the correlations rather than only the higher AQI level data (i.e., higher confidence data). All of the CPT predicted normalized unconsolidated undrained triaxial-based strength ($S_u, h_{vane}$) contours are shown in Figure 4 and all of the CPT predicted field vane shear strength, ($S_u, h_{max}$) contours are shown in Figure 5.

Several examples of CPT-predicted normalized strength versus measured strengths (converted to normalized values) are shown in Figures 6 and 7. To the right of the depth-based strength chart is the CPT predicted soil type (Olsen, 1984; 1986; 1988; 1994).
The Bjerrum $\mu$ parameter of Equation 9 is used to reduce the field vane shear strength to an equivalent UU triaxial strength level, $(S_\text{UU})_{\text{UU}}$ using Equation 10:

$$\mu = \frac{(S_\text{UU})_{\text{UU}}}{(S_\text{UU})_{\text{flow}}} \quad \frac{(S_\text{UU})_{\text{UU}}}{(S_\text{UU})_{\text{flow}}} \quad (9)$$

$$\frac{(S_\text{UU})_{\text{UU}}}{(S_\text{UU})_{\text{flow}}} \mu \quad (10)$$
For low rotational vane strains, the vane is trying to cut a larger cylinder than the physical vane size (Olsen, 1994). As a result, the field vane strength is too high, which is why $\mu$ parameter is always less than unity (1.0). This $\mu$ parameter is even lower for silty clays and over consolidated clays (i.e., 0.6 to 0.7). Historically, $\mu$ was estimated using Figure 8.

Confirmation of the $S_u$ contours were partially achieved by calculating $\mu$ contours using both CPT strength contours. Contours of $\mu$ shown in Figure 9 are equal to contours of $(S_u)_{\text{true}}$, divided by the contours of $(S_u)_{\text{true}}$. The $\mu$ contours in Figure 9 demonstrate that $\mu$ is equal to 0.9 to 1.0 for normally consolidated clay and decreases as the silt content increases or with over consolidation.

6. CONCLUSIONS
Clay undrained strengths were historically estimated using the cone resistance and bearing $N_q$ factor. A new technique is introduced which uses CPT cone and sleeve friction resistances (by means of the CPT soil characterization chart) to estimate normalized undrained strengths (i.e., unconsolidated undrained triaxial strength or field vane shear strengths). These correlations are partially confirmed by calculating trends of $\mu$. This technique does not require soil samples to estimate $N_q$, however, confirmation laboratory/field strength tests are always good practice.
Figure 8. Correlation of \( \frac{S_{u}}{S_{p}} \) to \( p' \) to establish the overconsolidation character and estimate the Bjerrum \( \mu \) correction factor (Aas, Lacasse, Lunne, and Hoeg, 1985)

7. REFERENCES


Figure 9. Calculated contours of \( \mu \) (i.e., \( \frac{S_{u}}{S_{p}} \) on CPT soil characterization chart by calculating the ratio of strength contours.


Prediction of Liquefaction Resistance using the CPT

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SYNOPSIS: Prediction of liquefaction resistance using in situ test data has progressed, over the last 10 years, from techniques based on the Standard Penetration Test (SPT) to the Cone Penetrometer Test (CPT). Current CPT-based techniques for prediction of liquefaction resistance are only based on correlations to the cone tip (or end-bearing) resistance. This paper presents an improved technique for prediction of liquefaction resistance which uses both of the primary CPT measurements (cone and sleeve friction resistances).

1. INTRODUCTION
This paper will describe an updated technique for estimating liquefaction resistance that uses both Cone Penetration Test (CPT) measurements (i.e., cone and sleeve friction resistances). This technique was originally developed in 1983 (Olsen, 1984) and improved in 1988 (Olsen, 1988). All other CPT-based techniques use only the cone resistance to estimate liquefaction resistance (Seed & De Alba, 1985; Robertson & Campanella, 1985; Ishihara, 1986).

CPT sleeve friction resistance is employed in the current approach because it is an index of high strain behavior. If a soil has an unstable structure, the passage of soil around the cone tip will disrupt the initial soil structure; the sleeve will measure the effect of high strain. As a result, the cone resistance is an index of soil structure strength and the sleeve friction resistance is an index of high strain strength.

2. LIQUEFACTION DEFINED
The classic definition of liquefaction involves breakage of the sand-to-sand contacts in a loose, saturated honeycomb sand (due to the earthquake shearing action) and the temporary suspension of sand grains in water. The consequence of liquefaction was originally defined as “flow failure,” but over the last 15 years a dynamic strain level of 5% (e.g., 10% double amplitude cyclic strain) has been used as a failure criterion. Loose to medium dense sands can experience liquefaction, however, loose sands have more strain potential compared to medium dense sands. Loose sands which liquefy can cause large slope movements whereas slopes of medium dense sands might deform only slightly.

In the 1960s, liquefaction resistance was typically determined using expensive cyclic triaxial laboratory tests. However, undisturbed sampling of sand is very difficult (e.g., loose sands densify during sampling). Also, natural in situ variability is more complex than can be generalized with a few tests on undisturbed soil samples. For these reasons, the late Professor H. Bolton Seed began to emphasize the SPT for liquefaction resistance determination in the early 1970s.

3. NORMALIZATION OF PARAMETERS
Normalization of parameters is important for CPT- and SPT-based liquefaction resistance determination techniques. This section will describe stress normalization for determining equivalent CPT and SPT measurements at a vertical effective stress of one atm (= 100 kPa).
3.1 CPT normalization

Recent research has significantly improved our understanding of the exponential relationship of cone resistance with vertical effective stress (Olsen, 1994). Normalized cone and sleeve friction resistances are described below:

\[ q_{c,ex} = \frac{q_c}{(\sigma_v)^{n}} \]  
\[ f_{s,ex} = \frac{f_s}{(\sigma_v)^{n}} \]  
\[ FR = \left( \frac{f_{s,ex}}{q_{c,ex}} \right) 100 = \left( \frac{f_s}{q_c} \right) 100 \]  

where

- \( q_c \) = Measured cone resistance (atm) (=tons/foot² (tsf) = bars = kgf/cm² or = 100 KPa)
- \( q_{c,ex} \) = Normalized cone resistance (\( q_c \) at \( \sigma_v = 1 \) atm using an variable exponent)
- \( q_e \) = Measured cone resistance (atm)
- \( c \) = Cone resistance stress exponent
- \( f_s \) = Measured sleeve friction
- \( f_{s,ex} \) = Normalized sleeve friction
- \( s \) = Sleeve stress exponent (approximately equal to c)
- FR = \( R_f \) = Friction Ratio

The field-measured SPT blow count (N) must be converted to an equivalent \( N_{ex} \) either based on an understanding of the hammer type or field SPT energy measurements. The stress exponent, \( n \), in Equation 4 was originally established (Seed, et al., 1983) based on laboratory SPT chamber tests at varying chamber confining stresses (data from Marcisz & Bieganowski, 1977). These tests were performed using a short drill rod, and a constant 1 metre depth of drilling mud fluid (i.e., constant mud pressure at the SPT sampler for all chamber confining stresses). This constant borehole mud pressure reduced the confining stress around the SPT sampler at high chamber confining stresses. Consequently, the calculated stress exponents based on this data are too low (Olsen, 1994). The true SPT stress exponent, \( n \), appears to be equal to the CPT determined stress exponent value, \( c \).

4. HISTORICAL FIELD-BASED TECHNIQUES FOR PREDICTION OF LIQUEFACTION RESISTANCE

The industry standard for field-based liquefaction resistance prediction for the last 25+ years has been the Seed SPT technique (Seed & Idriss, 1971; Seed & De Alba, 1986). This section will describe current SPT and CPT techniques for estimating liquefaction resistance Cyclic Stress Ratio (CSR).

4.1 SPT Liquefaction technique

Professor H. B. Seed developed the classic relationship between the normalized SPT blow count \( (N_{s,ex}) \) and the CSR causing liquefaction in the field as shown in Figure 1.

Increasing fines content (i.e., silts in sand) will generally increase liquefaction resistance (in all but very loose sands). One means of accounting for fines content is to determine an equivalent clean sand SPT blow count \( (N_{s,ex}) \) using the percent passing the #200 sieve as shown in Figure 2 and using Equation 5.
\[ (N_{s})_{20} = (N_{t})_{60} + \Delta N_{20} \] (5)

where

- \( (N_{s})_{20} \) = Silt-corrected clean sand normalized SPT blow count
- \( \Delta N_{20} \) = Silt correction for blow count
- \( (N_{t})_{60} \) = Normalized SPT blow count

4.2 CPT Liquefaction techniques

Most historical CPT techniques for estimating liquefaction resistance only rely on the cone resistance measurement. The best known cone resistance-based CPT liquefaction technique is summarized in Figure 3 (Seed & De Alba, 1986).

The only CPT-based liquefaction resistance prediction technique using both CPT measurements was developed by Olsen (1984, 1988). Contours of liquefaction resistance CSR for this technique are shown in Figure 4 (on the CPT soil characterization chart) and were established based on cyclic laboratory tests and trends of CPT-predicted SPT equivalent clean sand blow counts. What makes this approach unique is that soil index tests are not required to predict liquefaction resistance. The other CPT-based liquefaction prediction techniques require soil gradation tests to determine percent passing the #200 sieve or \( D_{10} \).

5. UPDATED CPT TECHNIQUE FOR PREDICTION OF LIQUEFACTION RESISTANCE

The proposed CPT-based technique for prediction of liquefaction resistance is an improvement on the method depicted in Figure 4. Three principle steps toward the development of this improved technique are described on the next page.
5.1 Correlating cyclic laboratory results to normalized CPT data

Cyclic laboratory triaxial and simple shear strengths from nine projects were correlated to normalized CPT parameters from nearby CPT soundings. The cyclic laboratory strengths (obtained using a failure criterion of 5% strain) were converted to normalized CSR at 15 equivalent uniform load cycles (appropriate for a magnitude 7.5 earthquake) and adjusted to an equivalent vertical effective stress of 1 ksf (approximately 1 atm or 100 kPa). Normalized CPT parameters (q<sub>n</sub> and FR) were determined using CPT data from nearby CPT soundings for each corresponding laboratory data value. These q<sub>n</sub> and FR values were located on the CPT soil characterization chart and the corresponding normalized laboratory CSR values were reported at each point as shown in Figure 5. Circles bound each CSR value in recognition of the fact that no soil layer is uniform; the laboratory liquefaction CSR should fall somewhere within the circles. The trend of these laboratory values indicate that normalized liquefaction resistance increases with friction ratio, at least within the soil mixture and clay classification zones.

5.2 Equivalent strain-based cyclic strength of clay

Clays have the potential for straining during earthquakes. Figure 5 contains results of several laboratory cyclic tests on clay (three test results located in the clay zone are marked with “A”). These tests show that normally consolidated clays may develop 5% cyclic strain if cyclic shear stress approaches the static strength. A static undrained strength σ<sub>c</sub>/p (clay undrained strength divided by vertical effective stress) of 0.3 and an earthquake-induced CSR of 0.25 represents an induced shear stress equal to 80% of the static undrained strength. Normally consolidated soft clays should therefore be expected to have equivalent liquefaction resistance CSR between 0.25 and 0.30.

5.3 Establishing CPT-predicted equivalent clean sand SPT blow count

Figure 6 shows the concept used to construct equivalent clean SPT blow count contours on...
the CPT soil characterization chart. Trends of liquefaction resistance can be estimated if trends of equivalent clean sand SPT blow counts are known. The first step required is to plot contours of CPT-predicted normalized SPT blow count ($N_{eq}$) (illustrated with solid lines in Figure 6). The silt correction for blow count ($\Delta N_{eq}$) becomes larger with increased fines content (and/or decreasing $D_{s}$). CPT-based soil classification can be used to estimate the fine content for the purpose of estimating $\Delta N_{eq}$ (note the annotation in Figure 6 concerning soil classification change). The ($N_{eq}$) contours (illustrated with dashed lines) will bend away from the ($N_{eq}$) contours within the silty sand portion of the chart and have larger values than underlying ($N_{eq}$) contours.

The concepts illustrated in Figure 6 to determine the equivalent clean sand SPT blow counts ($N_{eq}$) were implemented in Figure 5. The ($N_{eq}$) contours (dashed lines) in Figure 5 bend into the region of the chart where normalized laboratory CSR's are available.

5.4 Final CPT correlation for prediction of liquefaction resistance. The final CPT chart for estimation of liquefaction resistance cyclic stress ratio (CSR) is shown in Figure 7, based on the discussion in sections 5.1 to 5.3. The SPT-based CSR and laboratory cyclic test-based CSR trends match within the “Sand mixtures” zone of the CPT soil characterization chart. These contours improve the 1988 version (Olson, 1988) as a result of current understanding of stress effects, availability of more laboratory cyclic data, and better CPT-based soil classification.

6. COMPARISON OF CPT-BASED APPROACHES
This section will compare some of the differences between the cone resistance based liquefaction approach and the updated liquefaction technique. These two techniques will also be compared using an unique plotting procedure.

6.1 CPT liquefaction resistance prediction technique comparison
The updated CPT liquefaction technique shows that as friction ratio increases the predicted liquefaction resistance also increases (for a given $q_{ca}$ value). However, cone resistance-based techniques infer a constant liquefaction resistance for all friction ratio levels. These cone resistance-based techniques
use indexes (percent passing #200 sieve or D_20) to approximate the soil type effects. However, liquefaction resistance and the CPT sleeve resistance are both strength parameters. As stated earlier, sleeve friction resistance is an index of the soil structure sensitivity and as such can be an important contribution for prediction of liquefaction resistance.

6.2 Plotting the cone resistance-based liquefaction relationships onto the updated chart.

Each Ω, to CSR line (e.g. 10% fines and D_25=0.25mm) for the Seed cone resistance-based liquefaction relationship (Figure 3) can be plotted as a separate “soil type”-based contour as shown in Figure 8. “Soil type”-based contours for Ishihara (1985) and Robertson and Campanella (1985) are also shown. These “soil type”-based contours roughly follow the CPT-based soil classification boundaries. For example, the Seed & De Alba cone resistance-technique “soil type” contour having a fines content of 10% is located near the CPT soil boundary of SCN=1. A CPT soil classification boundary of SCN=1 (in Figure 8) was originally designed to correspond to a sand with 10 to 12% fines content (Olsen, 1988, 1984). Therefore, both procedures are indicating the same trends, except that the proposed technique does not require soil index testing.

7. CONCLUSIONS

An improved CPT-based technique for estimating liquefaction resistance is presented. This technique uses both cone and sleeve measurements and was developed using CPT predicted SPT N, trends and correlations to cyclic laboratory test results. It represents a more refined prediction of liquefaction resistance compared to the cone resistance-based techniques requiring soil gradation tests.

8. ACKNOWLEDGMENT

Permission was granted by the Chief of Engineers to publish this information.

9. REFERENCES


Figure 8. Superposition of techniques for prediction of liquefaction resistance which use only the cone tip resistance on the new chart.

SGF Report 3:95
CPT Stress Normalization and Prediction of Soil Classification

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SYNOPSIS: An updated CPT soil classification chart is presented based on a new cone penetration resistance normalization technique. This chart was developed using a very large database of CPT/boring data together with an improved understanding of how to predict soil strength. The new normalization technique is based on the Stress Focus concept. The Stress Focus is an in situ point at a vertical effective stress in the range of approximately 100 to 300 atm (10 to 30 MPa) where sand densities and strength have values independent of the initial relative density at lower stress states. As a result, the trends of cone resistance (for different relative densities) with vertical effective stress can be described using an variable stress exponent.

1. INTRODUCTION
The CPT soil characterization chart shown in Figure 1 was developed in 1987 using a limited CPT/boring database (Olsen and Malone, 1988). The soil type boundaries, word descriptions, and Soil Classification Number (SCN) concept from this chart have been used by numerous writers over the last seven years (Robertson, 1990, ARA, 1992, etc.). The Stress Focus concept, to be presented in this paper, describes how cone resistance is exponentially related to by vertical effective stress. The Stress Focus concept was deduced from the results of high pressure tests, CPT/SPT chamber tests, and field CPT tests in uniform in situ soil layers. It provides the basis for a new comprehensive stress normalization technique using CPT data. A new CPT soil characterization chart is also presented based on the new stress normalization technique, a much larger database of CPT/boring data, and finally a better ability to predict soil strength.

2. THE STRESS FOCUS
The basis of the new stress normalization concept is the observation that sand has the same strength at very high pressures (the Stress Focus) irrespective of the initial relative density. The Stress Focus is a unique point which occurs at a vertical effective stress between 100 atm (10 MPa or 100 tf) and 300 atm (30 MPa), as shown in Figure 2. It is because a given sand compresses to the same density and strength (at the Stress Focus) from all initial states.

![Figure 1. The 1987 CPT Soil Characterization Chart (Olsen, 1988)](image-url)
Failure envelope curvature reduces available strength with increased confining stress until the Stress Focus is achieved. At the Stress Focus, all initial relative densities have an available friction angle equal to that of initially loose sand at high confining stress.

![Diagram of Stress Focus](image)

Figure 2. Strength decrease to the Stress Focus for Sacramento sand in terms of friction angle and overburden stress (Olsen, 1994)
(data from Daligh, 1976)

The CPT calibration chamber trends shown in Figure 3 were established by Baldi, Bellotti, Ghionna, Jamiołkowski, and Pasqualini (1981) based on many CPT chamber test results. When these curves were replotted, using log scales as shown in Figure 4, they pointed to a Stress Focus. These same trends were also demonstrated using individual relative density chamber tests at various overburden stresses (Olsen, 1994). The cone resistance versus stress curves trending to a Stress Focus is caused by failure envelope curvature and cavity expansion effects.

The normalized cone resistance, $q_{c,ref}$, (Olsen, 1994) is defined by Equation 1 and can be used to describe the non-linear trend of cone resistance with vertical effective stress to the Stress Focus.

$$q_{c,ref} = \frac{q_c - q_{c,ref}}{\sigma_v^{ref}} = \frac{q_{c,ref}}{\sigma_v^{ref}}$$  \(1\)

where

- $q_c$ = Vertical effective stress (atm units)
- $q_{c,ref}$ = Measured cone Resistance (atm units)
- $q_{c,ref}$ = Normalized cone resistance
- $\sigma_v^{ref}$ = Total vertical stress (atm units)
- $c$ = Cone resistance stress exponent

The normalized cone resistance, $q_{c,ref}$, has a 1 subscript to represent normalization to our atmospheric (100 KPa) vertical effective pressure; the e subscript signifies that a variable stress exponent (based on the Stress Focus concept) was used for normalization.

The CPT cone resistance Stress Focus can now be generalized as shown in Figure 5 (Olsen, 1994). CPT soundings start penetration by failing the surficial soil using shallow bearing capacity failure. At the critical depth boundary, penetration is controlled by compression and cavity expansion. Surface expression failure can be defined by limit equilibrium theories (Terzaghi 1943, Durgunoglu & Mitchell, 1975) and have a linear relationship between bearing stress and vertical effective stress. Cavity expansion can be expressed as an exponential
relationship between bearing stress and vertical effective stress. The cone resistance stress exponent, $c$, decreases as the sand relative density increases (Olsen, 1994) and can be estimated as shown below using the relative density ($D_r$):

$$ c = 1 - (D_r - 10\%) \times 0.007 $$

(2)

The cone resistance at the Stress Focus is $q_{sf}$, and the vertical effective stress at the Stress Focus is $\sigma_{ve}$. The Stress Focus location ($q_{sf}$ and $\sigma_{ve}$) is soil type dependent; both parameters decrease as the soil type changes from sand to clay (Olsen, 1994).

3. **PREDICTION OF THE STRESS EXponent USING THE CPT**

Data from uniform in situ layers allowed establishment of cone resistance stress exponent contours on the CPT soil characterization chart (Olsen, 1994). An example of one such uniform layer is shown in Figure 4 together with the computer displayed $q_{min}$ (equivalent $q_c$ at $\sigma_{ve}=1$ atm), $c$ (log-log slope), and soil layer limits from the database. The stress exponent, $c$, is equal to the linear chart measured slope over one log vertical effective stress cycle. A stress exponent of 0 corresponds to a vertical line, while a exponent of 1 (linear scale) represents a line with a slope of one horizontal log scale to one vertical log scale. Normalized sleeve friction resistance parameters ($q_{sf}$ and $s$) for each uniform soil layer were also required to assign values to the CPT soil characterization chart. However, the sleeve is the more difficult of the two measurements. Data from several uniform soil layers could not be used because normalized sleeve friction parameters could not be established with adequate confidence.

Nonetheless, approximately 65 excellent uniform layers (from 240 potential uniform soil layers) were available to establish contours of cone resistance stress exponent, $c$, on the CPT soil characterization chart (Figure 7). In situ data points having the highest level of confidence were given most weight during establishment of the contours. Chamber test exponent trends were also used as a guide during establishment of exponent contours.

![Figure 4](image1.png)  
**Figure 4.** Replotting of Baldi, et al., 1981 data curves in terms of $\log q_c$ net cone resistance versus $\log \sigma_{ve}$ vertical effective stress

![Figure 5](image2.png)  
**Figure 5.** Annotated description of the cone resistance Stress Focus together with bearing stress partitioned into surface bearing capacity failure and cavity expansion failure (Olsen, 1994)
The cone resistance stress exponent contours in Figure 7 exhibit several predictable trends: 1) high values for loose sands, 2) very low values for over consolidated sands, 3) values of approximately 1.0 (and slightly lower) for normally consolidated clays, 4) values slightly below unity (i.e., 0.75 to 0.9) for slightly over consolidated clays, and 5) values as high as 1.2 for unstable silty clay mixtures. These stress exponent contours are used to determine the stress exponent for normalization of the cone resistance (Equation 1). However, an iterative solution is required. Initially, a stress exponent is assumed for Equation 1, the resulting \( q_{c,0} \) and calculated friction ratio are used to determine the chart-based contour stress exponent from Figure 7. If the chart-based stress exponent does not equal the assumed stress exponent, a new assumed value must be tried. Approximately 3 to 9 iterations are usually required until the chart-based value is sufficiently close to the assumed value (Olsen, 1986, 1988, 1994).

A vertical effective stress of one atmosphere typically occurs at a depth of 7 to 10 metres (23 to 33 feet) where the measured values of \( q_{c,0} \), \( f_c \), \( V_n \), SPT are approximately equal to their normalized values, \( q_{c,0}^{*}, f_c^{*}, V_n^{*}, N_s \). However, near the ground surface (i.e., 1 metre), and at great depth (i.e., 40 metres), use of an improper stress normalization technique will yield incorrect normalized values by a factor greater than two. This procedure (Olsen, 1994) represents the most advanced available normalization technique. However, the use of a constant stress exponent (i.e., 0.61) for normalization can be justified for general investigations within the depth range of 4 to 12 metres.

4. THE NEW CPT SOIL CHARACTERIZATION CHART

The new CPT soil characterization chart in Figure 8 is a significant improvement over the 1988 version (Figure 1) for three reasons: 1) an improved stress normalization has been developed, 2) a larger CPT/horing database (by a factor of 5) is available, and 3) an improved understanding of how to predict strength has been developed (Olsen, 1994).
a new CPT cone resistance stress normalization technique. This technique takes into account the fact that the cone resistance is exponentially affected by vertical effective stress (as observed in CPT chamber tests). Evaluation of data from uniform in situ soil layers provided the comprehensive correlations required for prediction of the stress exponent when using field CPT data. An updated CPT soil characterization chart was developed based on the improved stress normalization concept, a larger database of CPT/boring data, and better tools for prediction of strength.

6. REFERENCES

6. CONCLUSIONS
The Stress Focus concept provides the basis for

Approximately perpendicular to normalized
strength contours,

The CPT Soil Classification Number (SCN)
(Olsen, 1988) scaling on the CPT soil
characterization chart (see Figure 1) has been
changed to improve SCN understanding.
SCN=0 now represents a pure silt, SCN=1
represents a fine sand or low silt content silty
sand, and finally SCN= -1 represents the
boundary between silty clay and clayey silt. As
a result, SCN’s greater than 1 represent sand
and SCN’s less than -1 represent clay. The
boundary between normally consolidated and
over consolidated are also shown in Figure 8
together with trends for increasing over
consolidation. Soil classification descriptions
are also shown near SCN boundaries. Cone
resistance stress exponent contours (from
Figure 7) are also included for use with
Equation 1.

Figure 7. Contours of cone resistance stress
exponent based on data from in situ uniform soil
layers with constant relative strength
(i.e. constant sand friction angle or constant clay
c/c ratio) (Olsen, 1994)
Figure 8. Updated CPT soil characterization chart developed using a better understanding of stress effects, a larger CPT/boring database and based on improved techniques for prediction of soil strength.
ENGINEERING GEOLOGICAL MAPPING
OF SOFT CLAY USING THE PIEZOCONC

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SYNOPSIS: The paper addresses the question of the use of piezocone data to detect and map subtle variations in soft clay deposits. The paper shows that the subtle variations in the facies that have been identified in the detailed studies on the samples, can be clearly identified from the data derived from high quality piezocone profiles adjacent to the sampling borehole. It is also shown that, with a little care, excellent repeatability can be achieved in the field in close grouped piezocone tests. As a result of this repeatability it is then shown how the variations in the facies could be mapped, with some degree of confidence, over the rest of the site using the piezocone.

1. INTRODUCTION
In major investigations of soft clay sites it is usually important to map ground property variations over large areas, especially when consolidation is a major factor. At first sight the alluvium may appear to be homogeneous, unless there are obvious changes, say, peat or sand; however, differences may be more subtle, reflecting a changing depositional environment.

The piezocone test has the potential for rapid investigation over a large area and, when coupled with sampling and laboratory testing information, can provide interpolation of data over a large area. However, it has been found too often that in commercial testing the scatter of results is too great to allow this interpolation with any degree of confidence.

This paper presents data from piezocone testing undertaken by the Building Research Establishment (BRE) at the Engineering and Physical Sciences Research Council (EPSRC) national soft clay test site at Bothkennar, Scotland. Excellent geological profiles and engineering geology data from a number of sources are available for the site and the subtle variations in the facies that have been identified in detailed studies of samples, are compared with data from high quality piezocone profiles adjacent to the sampling borehole.

The question of repeatability of cone test results in soft clays will be addressed with a view to assessing the confidence with which piezocone data may be used to map the site.

2. THE SITE
Bothkennar is located on the River Forth, approximately midway between Edinburgh and Glasgow. The site is a low lying field bounded on 3 sides by flood embankments. A site plan is shown in Figure 1. The site was chosen as a test site as it was believed to have relatively uniform deposits as a result of the post-glacial history of the area (see Nash et al, 1992). The lithology of the site comprises a buried gravel (the Bothkennar gravel), above which lies a sequence of micaceous silty clays. These mainly comprise the Claret Beds which form the soft clay sequence and extend up to within 1.5-2m of the ground surface (Paul et al 1992). The sequence is in part overlain by the clayey silts of the Grangemouth Beds and, at the margins of the estuary, is completed by modern intertidal deposits.

Paul et al 1992 suggest that the Claret beds
are shallow water (subtidal) marine to intertidal estuarine deposits, laid down under a reducing water depth of less than 20 m between 5000 and 3000 years before present (BP). Figure 2 shows the detailed facies profile established by Paul et al as part of their engineering geological study of the site. They identified 3 principal facies types within the sequence: a bedded facies, in which the primary sedimentary layering remains visible, a mottled facies, in which the bedding has been partially or totally destroyed by bioturbation, and a laminated facies in which numerous silt laminae are present at spacing of a few centimetres or less. The facies were established both from visual appearance of the sediment and from their high resolution bulk density signatures (Paul et al 1992). In this way the profile of Figure 2 was established for b/h HW3 (Figure 1). They also established lithological units within the profile based on sedimentological and water content variations and these are also shown in Figure 2 (based on their latest assessments, Paul 1995 personal communication). The upper, middle and lower divisions of the Claret beds also related to the dominant types of facies; in the upper and lower the bedded facies and in the middle the mottled facies dominate. They proposed that the laminated bed was a local variation within the middle division. Reference to these detailed sequences will be made later in the paper.

3. PIEZOCONE REPEATABILITY

In order to have confidence in using piezocene tests in extrapolating data it is necessary to establish the repeatability of the test. Within the BRE area (Figure 1), an extensive programme of research on in situ testing has been undertaken. This has included some 50 cone and piezocene tests to investigate such topics as the rate of penetration, cone type, filter element location, load cell capacity and repeatability. In this paper only the work on repeatability is presented so that the reader will have confidence in the subsequent extrapolations.

In Figure 3a the results from a number of piezocene tests (5), undertaken over a number of years within the BRE test area, are presented.
in terms of family plots of the cone resistance, $q_c$, and the measured pore water pressure, $u$, (shoulder position). Excellent repeatability is seen. Care was always taken to calibrate load cells accurately and over the appropriate load ranges for the ground conditions (ie typically up to 1 MPa), to take zero readings before and after every test at a known temperature (preferably at or close to ground temperature), to make quick check calibrations throughout the testing programme and to record data faster (10 Hz rather than the more typical 0.5 Hz). In contrast, Figure 3b shows $q_c$ results from an independent testing programme in the same area using cones of higher capacity. The results appear to be far less repeatable. The exact explanation for this is not fully known but load cell capacity and sensitivity, i.e. the load cell output for the range of interest and the electrical noise on the signal, as well as calibration and zero errors are the most likely explanations. This is more a reflection on the necessity of specifying the accuracies required in the testing rather than the quality.

The problems of load cell calibration and sensitivity and frequency of reading are not only important for repeatability but also for obtaining reasonable detail in the profiles of the measured parameters. Figure 3c compares data from a BRE piezocone (with sensitive load cells) with that obtained with a cone of higher capacity (typical of those that might be used commercially but not showing the electrical noise of Figure 3b). The figure shows the higher capacity cone data plotted both at a scale typical of routine reporting practice (2 MPa to 1 unit on depth scale) as well as to the expanded scale used in this BRE work. It can be seen that for the higher capacity cone the standard plot gives very little definition whilst at the expanded scale the plot lacks the definition available with the lighter more sensitive load cells and more frequent readings. This highlights the need to specify correctly the sensitivity or capacity of the cone and the frequency of readings for the required purpose.
With the level of detail available in these piezocene profiles even with little experience in their interpretation, but with confidence in their quality, it would have been possible to have realised that they were potentially variations in the deposit down the profile.

It can be seen in Figure 4 that for a facies a piezocene signature might be established, not in terms of actual values of q or u but in terms of the shape of the profiles of each of these parameters in Figure 4. Pore water pressure profiles from filters on the cone face are very similar to the q profile but those from shoulder filters give a slightly different shape. Therefore in Figure 4 the pore water pressure profile is that from a shoulder, thereby giving two differing shapes as part of the facies signature. It is proposed that these signatures can now be used to map the location of the various facies across the site.

It can also be seen in Figure 4 that more major changes in the piezocene profiles can be observed in the overall changes of slope of the q, and u profiles and indicated by the dotted lines in figure 4. There is the obvious surface crust down to about 1.5 m and breaks in the profiles at about 5 and 12 m for example. This latter feature was identified by Hight et al (1992) as a possible old drying surface and marked by a subdivision in their main lower division at this depth. It also corresponds to the point in the geotechnical profile where the water content begins to reduce.

It is suggested that these breaks may well relate to the lithological units of the lower, middle and upper Claret beds proposed by Paul et al (1992). This is discussed further below.
5. MAPPING WITH THE PIEZOCONE

Several series of piezocone tests have been undertaken at various times in order to study the soil variability around the site. The initial cone and piezocone investigations reported by Nash et al. (1992) were unable to do much more than to identify the underlying gravel layer and much silty zone in one corner of the site, to confirm that there were no peat or other pronounced layers and to suggest that there could be variations in the soft clay profile across the site. This last conclusion was arrived at because of the significant scatter in the profiles of q, (a similar magnitude to Figure 3b, see Nash et al., 1992).

A series of piezocone tests around the site was undertaken by BRE observing the test procedures mentioned above. It was immediately obvious that there was far less scatter in the profiles than had been found in the previous investigations. From a very superficial inspection of the BRE data, all the profiles tended to show the marked kick in the q and u profiles at between 10 - 13 meters (See Figure 4), thus indicating the potential for a common feature across the site in addition to the very obvious gravel layer.

By following the procedure suggested earlier of looking for changes in the overall q and u profiles then other breaks could be observed in the profiles. In this way cross-sections across the site were established as shown by the example in Figure 5 (Figure 1 shows locations). Also shown in Figure 5 are the variations in the lithological units suggested by Paul et al. (1992) based on b/h information. It can be seen that the section based on piezocone interpretation implies a much more uniform layering across the site than that from b/h. It is felt that the piezocone interpretation is a potentially more reliable one as that by Paul et al relies primarily on visual evidence and the assessment of the percentage of each facies type.

Having established confidence in the quality

Figure 6. Comparison of piezocone signatures
and detail of the piezocone data, a more detailed inspection of the profiles was then undertaken. By beginning at the engineering geology and BRE test areas with the piezocone signatures of Figure 4 and working away it was possible to establish an approach to be adopted for more detailed mapping. Figure 6 shows part of the profile from Figure 4 to an expanded scale along with part of a profile from a location A1F1 (see Figure 1), the second profile has been shifted about 1 metre in depth to obtain this match. It is immediately apparent that there is a good match in shape of both $q_u$ and $u$ over much of the section. By using both $q_u$ and $u$ the match was established with reasonable certainty; either parameter on its own would have been insufficient to ensure agreement. Local variations could be detected in the same way. These matches or not are in the profile shapes not in their absolute values; these would only agree if they were from the exactly the same depth and had experienced identical stress histories.

Figure 7 shows an example of how one 2m deep feature has been mapped by its piezocone signature (here just $q_u$) over some 250m on section line C-C.

Detailed matches obtained in this way also helped to confirm the lithological divisions proposed in Figure 5 based on gross features. The procedure of matching features in this way will hopefully allow the detailed geotechnical data gathered from the original study area to be reliably utilised elsewhere on the site.

6. CONCLUSIONS
In this paper the potential for the use of high quality piezocone tests to study the variation of a known engineering geology profile across a site has been presented and the following conclusions may be drawn:

- It has been shown that by taking care, adopting appropriate test procedures and using a piezocone of the correct capacity then excellent repeatability and definition can be achieved in very soft deposits; even when the data have been gathered over a number of years.
- Having piezocone profiles of sufficient detail and plotted to a satisfactory scale has allowed potential lithological units to be mapped with confidence.
- With this increased definition it has been possible to clearly identify with the piezocone subtle changes in facies down a soil profile.
- By assigning a signature to the facies or sequence thereof it has been possible to identify common features as well as variations in the soil profile around the site.
- The use of piezocone testing will allow detailed geotechnical and geological engineering data gathered at one location on a site to be transferred to other locations with confidence thus reducing the costs otherwise incurred in sampling and testing.

7. REFERENCES
CPT in Very Weakly Cemented Sand: A Calibration Chamber Study

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**SYNOPSIS:** The cone penetration testing (CPT) is not calibrated for classifying cemented sands and interpreting their properties. An attempt is made to investigate the influence of cementation on cone penetration test results. Difficulties in sampling large natural cemented specimens and unavailability of natural cemented deposits at various densities and confining pressures led authors to use artificial cemented sand specimens. Cemented specimens are prepared using pluviation technique and then cone penetration tests are carried out in a calibration chamber under controlled boundary conditions. This paper discusses different aspects of artificial cemented specimen preparation and the cone penetration testing procedures. The cement bonding, carbon dioxide saturation effects in the specimen preparation, the influence of drained conditions, relative density and cement contents on cone test results are discussed. A summary of the results is provided.

1. **INTRODUCTION**

CPT parameters like tip resistance, $q_t$, sleeve friction, $f_s$, and if the piezocone is used, the total pore pressure $u$, along the tip and/or along the shaft of the penetrometer are used to classify the soil strata and to determine the soil properties such as relative density, overconsolidation ratio, strength, deformation properties and hydraulic conductivity values. Estimates of relative densities of uncemented granular deposits are often made using charts developed by tests conducted in calibration chambers. Internal friction angles are then determined either by using correlations of the tip resistance with relative density or through the correlations of tip resistance with the state parameter or the relative state parameter (Baldi et al. 1981; Puppala et al. 1995).

These analysis techniques can be further improved by including the influence of various variables like size and shape of the aggregates, fabric, creep and cementation on the cone test results. Among these variables, cementation is considered to be one of the most important environmental variables to be included in the interpretation methods. The recent Loma Prieta earthquake in San Francisco where several infrastructures built on weakly cemented deposits failed shows the importance of including cementation of granular deposits in the existing interpretation charts (Puppala et al. 1995). Ghionna and Jamiołkowski (1991) also explained the necessity to carry out calibration chamber
2. TESTING PROGRAM

A total of thirty seven calibration chamber tests were carried out on both uncedmented and cemented specimens. Cementation levels of one and two percent were investigated since these cementation levels represent very weakly cemented sands (Rad and Tumay, 1986). Specimens were prepared at three different ranges of relative densities (45-55, 65-75 and above 85 %) and consolidated under vertical stresses of 100, 200 and 300 kPa.

Monterey No. 0/30 sand was used in this study. It is a clean beach sand with subangular to subrounded particles. Monterey No. 0/30 sand is considered as a medium compressible sand. The maximum and minimum dry densities of this sand are 16.65 kN/m$^3$ and 14.04 kN/m$^3$, respectively.

The calibration chamber was the basic testing equipment used in the experimental testing with cone penetrometer. A pluviation setup was associated with the specimen preparation procedure. In this study, air was used as the pluviation medium since it provides an easy and less cumbersome preparation technique and more importantly produces reasonably homogeneous specimens (Rad and Tumay, 1986, Puppala et al. 1995).

The details of the pluviation set-up are described by Puppala et al. (1995). This setup consists of three cylindrical chambers placed one above the other. All chambers are made of PVC sections. The top chamber stores the sand that needs to be pluviated while the
middle chamber gives sufficient height of fall for the sand leaving the top chamber. The bottom chamber is the one in which the sand is deposited and formed.

Since the study involved cemented sands, the bottom chamber needed to be transferred first to the humidity room for curing and then to the calibration chamber for testing. This transferring required that the top two chambers be permanent fixtures and the specimen chamber be the only removable member of the setup. During pluviation, the sand leaves the top chamber and pours into the specimen chamber. The height of fall is one of the variables that influences the relative density of the specimen (Rad and Tumay, 1986). If a constant height of fall is maintained, it is expected to achieve a relatively homogeneous specimen. The opening of diffuser sieves through which sand is pluviated prior to final deposition in the bottom chamber is the other important variable that controls the relative density. Further details of these variables can be found in Puppala et al. (1995).

The statistical variation in relative density among specimens prepared at 45% to 55% relative density and for those prepared at 55% - 65% relative density was found to be 3.0 to 3.3% and 1.6 to 1.9%, respectively. The variation for higher densities (greater than 80%) was around 2.0 to 2.7.

The calibration chamber system at Louisiana State University was described by Puppala et al. (1995). The chamber is 1.78 m in height and 0.64 m in diameter. The cell can house a sample of 0.33 m in diameter and 0.79 m in height. Horizontal stress is applied to the specimen by pressurizing the inner or outer annulus made of steel sleeves. Vertical stress is applied by pressurizing the desired water in the piston assembly. A control panel with pressure regulators is used for stress applications. Other details on these operations are given by Puppala (1993).

It is essential to allow water into the cemented specimen both to achieve a reasonable saturation and to activate the pozzolanic reaction. Water and carbon dioxide (CO₂) are used in saturating the specimen. The CO₂ is allowed into the chamber at a low pressure of 7 kPa as a result of hydrostatic pressure obtained due to the differences in the static head levels between the water tank and the specimen chamber. The carbon dioxide replaces the air in the specimen since it has a higher molecular weight than air. Then, water is allowed into the specimen for saturation. Carbon dioxide dissolution in water is higher than air in water; its use promotes better saturation.

A diameter ratio of at least 40 may be necessary in dense and overconsolidated sands if the constraints imposed by the boundary conditions are to be avoided (Parkin and Lunne 1981). If a standard size cone is used, the required minimum chamber dimension would be at least 1.6 m, making the specimen
preparation and handling quite time-consuming and cumbersome. Therefore, a miniature quasi-static cone penetrometer (MQSC) with a push rod of 9.53 mm diameter, 1.82 m in length incorporating a 6.3 cm long friction sleeve was used in this study. MQSC is a Fugro-McLelland type cone and has an apex angle of 60°. This cone when used in the LSU chamber, yields a diameter ratio of 42.

The miniature friction cone was used in all tests except for one in which a reference type piezocene was used. This one test was conducted on a saturated two percent cemented sand in order to assess the influence of cementation on the pore pressure measurements.

2.1. Specimen Preparation

Cone penetration testing of sands in calibration chambers provided the conclusion that testing dry or fully saturated specimens did not show significant variation in cone tip resistance (Schmertmann 1976; Rad and Tumay 1986; Baldi et al. 1981a). Hence, un cemented specimens are tested in dry conditions and cemented specimens are tested in full saturation. Saturated conditions promote curing and facilitate hydration in forming the crystallization products.

2.2. Testing

The chamber testing consists of three operations: specimen transfer and placement, consolidation and cone penetration testing. The specimens were transferred into the chamber and consolidated under K0 conditions by maintaining a zero lateral volumetric strain around the specimen. Cone penetration testing was conducted under zero strain in the lateral direction and a constant vertical stress. The cone was pushed into the specimen at a rate of 2 cm/s. The readings were recorded at 2 cm intervals by a data acquisition system (Puppala 1993).

3. EVALUATION OF TEST RESULTS

3.1. Cement Bonding

One concern in specimen preparation procedure was segregation of sand and cement particles during pluviation resulting in non-homogeneous specimens with partly cemented and partly un cemented zones. It was decided to examine cementation at the micro-fabric level. Samples were collected randomly at different depths and were scanned through an electron microscope. Micrographs consistently display cementation bonds homogeneously scattered around the grains and no segregation was observed. Cement particles around the sand aggregates could be seen in all micrographs at all depths. A typical scanning electron micrograph of one of the samples (2% cementation) is presented in Figure 1. Also, the unconfined compression strength (UCS) of the specimens retrieved from a CC specimen (2% cement content, 81% relative density) is
51 kPa and that of a similar cemented specimen for triaxial tests is 49 kPa. This indicates that the strength and deformation behavior of CC cemented specimens do not differ from artificial specimens prepared for triaxial testing. The qualitative (micrographs) and quantitative (UCS results) assessments provided ample evidence of homogenous cementation between the grains.

Figure 1: Typical scanning micrograph

3.2. CO$_2$ Saturation

The use of carbon dioxide saturation process needs to be assessed to determine the influence of compounds formed by CO$_2$ reactions on the strength of the cemented specimens. This is specifically necessary since the triaxial specimens were not prepared using CO$_2$ saturation. Existence of CO$_2$ in the pores may result in an acidic environment in the CC sample which in turn may affect the cementation process. This may lead to a cemented chamber specimen which may not be comparable to a cemented specimen prepared for triaxial conditions.

In an attempt to understand the variations in cementation processes due to the CO$_2$ and air saturation techniques, unconfined compression tests were carried out on 71.1 mm (2.8 in.) diameter cemented specimens with and without CO$_2$ saturation. Tests were conducted on 2% cemented specimens with relative densities around 80 - 88% and 65 - 70%.

Test results presented in Table 1 indicate that there is no significant change in strength due to CO$_2$ saturation. The pH readings taken in the soil and on the water drained from the specimen indicated that only a basic environment prevailed in the specimens. This study alleviated the concerns on possible negative effects of CO$_2$ saturation to induce acidic environment and reduce the strength of cemented specimens.

3.3. Drainage Conditions

Cementation decreases the hydraulic conductivity of sands. Mitchell (1980) reported a hydraulic conductivity of 1 to 6 X $10^{-5}$ cm/s for standard proctor compacted specimens of 4% cemented sand. Hydraulic conductivity tests with 1% cemented Monterey No. 0/30 sand specimens at relative densities of greater than 80 percent rendered hydraulic conductivities of 1 to 4 X $10^{-3}$ cm/s (Arslan, 1993). The hydraulic conductivity for uncedent sand was around 6 to 8 X $10^{-3}$
cm/s which is not significantly different (Arslan, 1993).

In an attempt to assess whether the levels of cementation used in this study would lead to undrained loading conditions during penetration, one calibration chamber specimen also was prepared at a relative density of 65 \% and 2 \% cementation level. Cone penetration test was conducted using a standard, piezocene (3.56 cm diameter, 60 degrees apex angle) at a penetration rate of 2 cm/s. Figure 2 presents the total pore pressure distribution measured at the tip level versus the depth of the specimen. Only hydrostatic pore pressures were recorded in this test. The reduction in hydraulic conductivity at higher cementation levels is not significant enough to result in undrained conditions at a penetration rate of 2 cm/s.

3.4. Results

The following empirical correlations are obtained through non-linear regression analysis using the CPT results in cemented specimens and the triaxial test data. It was determined that predictions obtained by individual correlations at each cementation level rendered slightly better predictions of relative density than that obtained by a regression performed using all cementation data (Puppala et al. 1995). The regression analysis renders the following expression for tip resistance.

\[
\left( \frac{q_t}{\sigma_v} \right) = 10^{(0.15 + 1.11 D_i \phi)} \left( \frac{(\sigma'_0 + a'_0)}{\sigma_v} \right)^{0.764}
\]

where \(D_i\) is given in percent and \(a'_0\) is peak friction (ratio of peak cohesion intercept to the tangent of friction angle). The \(R^2\) value of this correlation was 0.82. The ratio of cemented tip resistance to uncemented one with normalized vertical confining stress is given in Figure 3. It is interesting to note that the tip resistance in very weakly cemented sand (unconfined compression strength less than 60 kPa) is nearly four times higher than the uncemented values, specifically when the confining stress is less than 50 kPa. The effect of confinement gradually overshadows the effect of very weak cementation with depth. However, very weak cementation may give tip resistance values of up to 50 \% higher than the tip resistance of clean sands even when the vertical confining stress is around 300 kPa.
As cementation increases, the friction ratio decreases slightly, in particular at low vertical confining stresses. At each cementation level and confining stresses employed in this study, the above semi-empirical relationship demonstrated a very weak dependence on the normalized vertical effective stress. The friction ratio values at both cementation levels was approximately around 0.57%. The friction ratio, however, expected to decrease at higher cementation levels (> 2%) and/or at lower confining stresses (σ_3/σ_o < 1.0).

4. SUMMARY AND CONCLUSIONS

This paper presented miniature cone penetration testing results on artificial cemented calibration chamber specimens of very weakly cemented sand. The CO₂ saturation does not have any influence on the cementation process at cementation levels used in this study. The cementation bonds in sand induce a cohesion intercept, increasing the tip and friction resistances. However, the friction ratio, defined as the ratio between the sleeve friction to the tip resistance, does not change significantly with very weak cementation. The results suggest that the effect of very weak cementation will be predominant with in the first 100 kPa confinement. The increase in confinement will gradually overshadow the effect of very weak cementation. However, it is still possible to get tip resistance values of very weakly
5. ACKNOWLEDGEMENTS

This study was supported by the National Science Foundation under the grant no. MSS-9020368 awarded to the second author. The LSU calibration chamber was designed, assembled, and installed under previous support to the third author by the National Science Foundation, Louisiana Transportation Research Center and Louisiana State University. The authors would like to acknowledge these supports.

6. REFERENCES


Table 1: Influence of Saturation on UCS

<table>
<thead>
<tr>
<th>With CO₂</th>
<th>Without CO₂</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dₚ (%)</td>
<td>UCS (kPa)</td>
</tr>
<tr>
<td>82.0</td>
<td>45.6</td>
</tr>
<tr>
<td>88.4</td>
<td>55.0</td>
</tr>
<tr>
<td>67.1</td>
<td>39.3</td>
</tr>
<tr>
<td>68.8</td>
<td>41.2</td>
</tr>
</tbody>
</table>

Note: Dr - Relative Density; UCS - Unconfined Compression Strength in kPa
CPT Evaluation of Soil-Gases for Los Angeles Metro Rail Tunnel

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John Sepich
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SYNOPSIS: Initial soil exploration borings for the Mid-City tunnel segment of the Los Angeles Metro Red Line identified localized high concentrations of hydrogen sulfide (H₂S) and methane (CH₄) gases. These appear to be related to groundwater conditions. H₂S is highly toxic at relatively low concentrations, and both gases are flammable. Therefore, mud was used in the drilling process to suppress gas emissions from the boreholes. Unfortunately, this process precluded accurate evaluations of groundwater and soil-gases, raising significant questions regarding subway constructability and operability.

The second-phase exploration included 40 CPT soundings to a maximum depth of 28 meters. Information logged included continuous physical soil characteristics, pore-water pressure, soil-gas pressure, and H₂S and CH₄ concentrations for selected strata. The combined soil, water, and gas testing capabilities of the modified CPT equipment proved safe and efficient for evaluating the desired subsurface conditions. However, because of the high concentrations of toxic gases encountered, the future of the proposed alignment is in question.

1. INTRODUCTION
Most segments of the Los Angeles Metro Red Line are designated as gassy or potentially gassy because of their proximity to known active and inactive oil fields. Both these deep underground oil fields and the surrounding ground surface areas are well known for the potential presence of explosive gases, primarily methane. Indeed, methane explosions have affected residential and commercial developments in the area. However, the area has not previously been associated with any appreciable amounts of hydrogen sulfide.

So far, construction of the Metro Red Line has only required mitigation of methane. These mitigation measures include ventilation to reduce methane concentrations to well below the lower explosive limit (LEL) and the use of a high-density polyethylene (HDPE) membrane between initial and final tunnel liners. The HDPE membrane doubles as a methane gas barrier and a waterproofing material.

Beginning in July 1993, a geotechnical investigation by Law/Crandall (1994) and an environmental investigation by Enviro-Rail (1994) for the Mid-City Segment of the Metro Red Line revealed high concentrations of hydrogen sulfide gas (H₂S), in addition to methane (CH₄). H₂S concentrations greater than 10,000 parts per million (ppm) were found along the proposed alignment.

Basic literature research has established the explosive potential of methane (5 to 15 percent by volume) and H₂S (4 to 46 percent by volume) as well as the toxicity of H₂S. H₂S is twice as soluble in water as carbon dioxide and...
is heavier than air, whereas methane is lighter than air.

Besides being explosive, \( \text{H}_2\text{S} \) is also highly corrosive to concrete and ferrous metals, and extremely toxic to humans. Prolonged exposure to moderate concentrations can cause injury or death (Table 1). The gas may be called a “one-event” hazard because, unlike methane, which requires both a source and ignition to be hazardous, \( \text{H}_2\text{S} \) only requires a source. \( \text{H}_2\text{S} \) also has a “rotten egg” odor, which some people can detect at concentrations as low as about 0.003 ppm. Thus, the presence of \( \text{H}_2\text{S} \), even in extremely small concentrations, can be a public nuisance during construction and operation of the subway. This could result in reduced ridership and public ill will.

Table 1. Human Physiologic Responses to \( \text{H}_2\text{S} \) Exposure (After Reiffenstein, et al., 1992)

<table>
<thead>
<tr>
<th>Concentrations of ( \text{H}_2\text{S} ) ppm</th>
<th>Physiological Responses</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.003</td>
<td>0.004</td>
</tr>
<tr>
<td>3</td>
<td>4</td>
</tr>
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<td>20</td>
<td>28</td>
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<td>50</td>
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<td>350</td>
</tr>
<tr>
<td>500</td>
<td>700</td>
</tr>
<tr>
<td>&gt;500</td>
<td>&gt;700</td>
</tr>
</tbody>
</table>

3. LOCAL GEOLOGIC AND HYDROGEOLOGIC CONDITIONS

The sequence of geologic formations encountered by Law/Crandall (1994) over the area of the Mid-City Segment (Figure 1) are generally described (in inverse order of deposition) as artificial fill, recent alluvium (younger and older), Lakewood Formation, San Pedro Formation, and Fernando Formation. The general characteristics of each stratum are described below. For more detail on geology, see Lamar (1970) and Yeeke, et al., (1965).

Artificial Fill Commonly found over the entire alignment, this includes structural fill, utility trench backfill, and road base.

Recent Alluvium These unconsolidated Holocene deposits consist of layers of sands, silts, and clays with scattered gravel. Perched groundwater occurs at varying depths within the sand intervals.

Lakewood Formation The Lakewood Formation is composed of late Pleistocene marine and non-marine silt and silty sand with minor amounts of clay. Generally, the Lakewood Formation acts as a groundwater and gas barrier.

San Pedro Formation These early Pleistocene age deposits typically consist of very dense, fine- to medium-grained sand, with scattered lightly cemented and silty zones. Locally, layers of tar sands occur within this formation. A zone of groundwater lies within the San Pedro Formation, perched upon the underlying, relatively impermeable Fernando Formation (siltstone).

Fernando Formation The Fernando Formation consists primarily of massive siltstone with sandstone interbeds. This formation occurs below the depths of most exploratory borings for the Mid-City Segment.

2. PROJECT DESCRIPTION

The proposed twin/bored subway tunnels will each have an outside diameter of about 6.4 m. The tunnel crowns are expected to vary from approximately 6 m to 25 m below the existing ground surface. The stations will be constructed by cut-and-cover methods and will require excavations up to 23 m below the existing ground surface.
4. SOIL-GAS SURVEY

Various subsurface investigations have identified high concentrations of H₂S and methane gas at the proposed tunnel depth along the Mid-City alignment. These gases were detected particularly in the San Pedro Formation sands, and were reported only locally in the overlying Lakewood Formation and older alluvium. To better delineate the extent and concentration of H₂S along the alignment, a soil-gas survey was performed at various depths along the alignment.

The survey consisted primarily of cone-penetration testing (CPT)/gas-probe soundings to evaluate soil, groundwater, and soil-gas characteristics. Figure 2 is a schematic of the soil-gas testing apparatus used. The field exploration program consisted of the following specific procedures:

- Logging subsurface soil conditions using the CPT
- Measuring pore pressure dissipation in each sand layer encountered
- Measuring subsurface gas pressures
- Identifying the presence and concentrations of H₂S using electrochemical meter detection for H₂S concentrations up to 2000 ppm and sensidyne-chemical-packed tubes for concentrations above 2000 ppm
- Identifying the presence and concentrations of methane using catalytic-type electron detection equipment
- Evaluating the soil-gas recharge rate by attempting to vacuum pump these gases to the surface
- Obtaining 1-liter Tedlar bag samples of the gases for laboratory analysis and confirmation of field measurements
- Obtaining sulfur samples by bubbling gas through a cadmium solution for later laboratory isotopic analyses
Figure 2. Schematic of Soil-Gas Testing Apparatus

Initially, soil-gas samples were obtained through a modified CPT probe with an external sintered steel filter and an internal hydrophobic filter (allowing gas flow and inhibiting water flow). These were connected to 6 mm plastic tubing threaded through the CPT rod that allowed soil-gas samples to be drawn out at the surface, while pore-pressure-dissipation readings were obtained at selected depth intervals. The CPT measurements allowed for continuous soil and groundwater information and accurate selection of soil-gas testing depths. The soil-gas sampling filter was located approximately 15 cm behind the tip of the piezoecone. This allowed for soil-gas readings to be obtained at the top of a sandy stratum while pore pressure dissipation readings were obtained 15 cm below.

Later, because of smearing of the sintered steel filter, the procedure was modified. With a new procedure, soil and groundwater readings were obtained with standard CPT soundings, and then soil-gas samples were obtained at selected depths via a BAT probe advanced by the CPT rig in the same hole. The BAT probe was fitted with 6 mm plastic tubing running to the surface soil-gas testing apparatus. The procedure worked satisfactorily, allowing soil-gas and water-level readings to be obtained as deep as 28 m below ground surface. The depth was typically limited by the very dense San Pedro Formation sands, and occasionally by cemented zones in the Lakewood Formation.
5. STUDY RESULTS
This investigation confirmed the presence of saturated soil along the northeastern reach of the alignment and unsaturated San Pedro Formation sands from about Olympic Boulevard southwesterly along the alignment. The testing confirmed high levels of H$_2$S and methane in those unsaturated San Pedro Formation sand zones along the alignment. Measurements in the Lakewood Formation below the water table typically indicated lower concentrations of H$_2$S, which generally dissipated rapidly during our study.

Above the water table, no H$_2$S was measured in the Lakewood Formation. This may result from lack of confinement (and resulting H$_2$S dispersion and co-transport with methane), or from contact with aerobic bacteria (which convert the H$_2$S) above the water table.

The highest levels of H$_2$S recorded during these CPT/gas-probe explorations were found in the unsaturated San Pedro Formation in two areas:
- near Olympic Boulevard, with a high reading of 5,600 ppm; and
- along Pico Boulevard near the southwestern terminus, with a high reading of 8,000 ppm.

Methane was measured at about 2 to 93 percent by volume (20,000 ppm to 93,000 ppm) in the San Pedro Formation; this compares to the LEL of about 5 percent (50,000 ppm) when exposed to temperatures above 53°C. In structures, regulatory agencies typically consider methane levels above 25% of the LEL, or above 12,500 ppm, to be unsafe.

This study confirmed low concentrations of methane within the Lakewood Formation. The saturated Lakewood Formation overlies and appears to cap upward-movement of gases from the San Pedro Formation. Because both methane and H$_2$S exist at depth in the San Pedro Formation, and because methane is lighter than air and H$_2$S is heavier than air, it appears that H$_2$S is co-transported with the lighter methane. Thus, it is possible that the saturated Lakewood Formation would strip the highly water-soluble H$_2$S during any off-gassing through the formation. In fact, readings of up to 220 ppm H$_2$S were measured below the water table in sandy lenses within the Lakewood Formation with no detection of methane.

Laboratory isotopic analyses of the carbon and hydrogen from the methane indicated two sources: (1) thermogenic, coming from deep seeps up through the fractured Fernando Formation, and (2) bacterial, possibly coming from tar sands in the San Pedro Formation. Isotopic analyses of the hydrogen sulfide indicated it was bacterial in origin, possibly produced in the anaerobic, unsaturated zone within the San Pedro Formation.

6. FINAL OBSERVATIONS
The CPT probe with gas testing capabilities allowed controlled, safe, in-situ sampling of highly toxic and combustible gases. Testing depths of up to 28 m below ground surface with the CPT rig were far greater than the 3 m to 6 m depths of typical soil-gas surveys. In addition, the simultaneous soil, pore-water pressure, and soil-gas measurements helped confirm:
- low concentrations of H$_2$S within saturated sand layers of the Lakewood Formation in the northeastern reach of the alignment,
- a confined aquifer in the northeastern reach of the alignment,
- unsaturated conditions in the upper San Pedro Formation sands southwest of Olympic Boulevard, and
- high soil-gas concentrations (and volumes) in the unsaturated San Pedro Formation sands.

The exposed sintered steel filter did not hold up well, due to the high friction forces developed during probe advancement. In the future, the alternative procedure used with a modified BAT probe that allows for an external sleeve to slide up exposing a protected filter could easily be combined into the CPT
equipment to avoid the need for a second probe advancement.

The internal hydrophobic filter had difficulty maintaining a tight water seal at the edges. However, testing for soil-gases below the groundwater table is not a common need or practice. Therefore, this would not be necessary for normal applications.

Currently the tunnel design is being reevaluated. Engineering Management Consultant (1994) has performed a reassessment study to evaluate the possibility of elevating sections of the alignment. In addition, Law/Crandall (1994) has performed the first phase of a hazard evaluation to identify possible hazardous concerns related to tunnel and station construction and operation. Because of the high concentrations of H2S encountered, and its toxic and odorous characteristics, the future of the proposed alignments is in question.

7. REFERENCES
Enviro-Rail (1994). Alignment Reassessment for Metro Red Line Segment 2/3 Mid-City and Western Extension.
Comparison between Cone Factor, Activity Index and OCR for Clay

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SYNOPSIS: For several years attempts have been made to correlate the results of the cone resistance \( q_c \) to the undrained shear strength of clay. The empirical cone factor \( N_k \) has been applied using the formula suggested by Terzaghi (1943) rewritten as

\[
N_k = \frac{q_c - \gamma z}{c_u}
\]

where \( q_c \) = cone resistance
\( \gamma \) = total unit weight of soil
\( z \) = depth of penetration
\( c_u \) = undrained shear strength

In this study the activity of the clay and OCR is included in the evaluation at the cone factor and the undrained shear strength. The activity of a clay is defined by Shempton (1953) as the plasticity index of the clay divided by the clay content.

This paper presents an evaluation of the relation between the cone factor \( N_k \), OCR and the activity of a clay. The evaluation is primarily based upon Danish Great Belt clay sill, but other investigations are also included.

On the basis of the presented results, a relationship between the cone factor, OCR and the activity of the clay is established.

1. INTRODUCTION
The Storebalt is an approximately 18 km wide channel between the islands of Fyn to the West and Sjælland to the East in Denmark. Storebalt is separated into an Eastern and a Western Channel by the tiny island of Sprogø lying at the centre. The Storebalt Project includes:
- An approximately 8 km long railway tunnel under the Eastern Channel
- An approximately 6 km long railway and road bridge across the Western Channel (the West Bridge)
- An approximately 7 km long road suspension bridge across the Eastern Channel (the East Bridge).

Various firms involved in the geotechnical investigations for the Fixed Link across Storebalt have evaluated the cone resistance, \( q_c \) versus the vane shear strengths, \( c_v \).

This case study includes the presently available information from site investigations in the Storebalt area, eastern part.

In Storebalt, site investigations have been carried out with CPT soundings as well as traditional geotechnical borings with field vane tests in cohesive material.

In this investigation \( q_c \) values are compared with \( c_v \), values from field vane tests, whereas the comparison for the other areas is based on the relationship between \( q_c \) values and \( c_v \) values derived from plate load tests and laboratory tests.
The authors are aware that the undrained shear strength cannot be found directly by either CPT or the field vane tests. Bjerrum (1973) indicated that the nature of the undrained shear strength is depending on direction and rate of shearing.

2. STOREBÆLT CLAY TILL
The general description of the glacial clay till formation given in Geomodel ref. Porvig et al (1989):
Clay till, sandy with some gravel and a few lime grains, dark grey. Occasionally the content of clay is reduced and the soil type changes into sand till, clayey. Within these soil types boulders and/or blocks are to be expected.

Classification Properties
The typical range of classification properties for the penetrated till are summarized in Table 1 using mean values:
- Moisture content, $w$ (\%)
- Bulk density (kN/m$^3$)
- Void ratio, $e$
- Specific gravity, $d_i$
- Liquid limit, $w_L$ (\%)
- Plastic limit, $w_p$
- Plasticity index, $I_p$ (\%)
- Clay content (clay % weight with $\mu < 0.002$ mm)
- Calcium carbonate CaCO$_3$ (\%)
- Activity ($I_q$/clay content)

<table>
<thead>
<tr>
<th>Mean Value</th>
<th>East Bridge</th>
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<tbody>
<tr>
<td>$w$ (%)</td>
<td>11.6</td>
</tr>
<tr>
<td>$\gamma$ (kN/m$^3$)</td>
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<tr>
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<td>$I_p$ (%)</td>
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<tr>
<td>CaCO$_3$ (%)</td>
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<tr>
<td>Activity $I_q$</td>
<td>0.29</td>
</tr>
</tbody>
</table>

Table 1. Mean Value of Classification Properties, Glacial Till

3. PROCEDURES TO CALCULATE $N_q$
The analyses are mainly based on the following procedures:
The comparisons have been performed in glacial till (mostly clay till) with relevant activity, field vane tests and nearby CPTs within a radius of 5 m generally available in the Storebælt "GEOMODEL" data base.
The cone factors $N_q$ have been calculated using the following formula:

$$N_q = \frac{q_u - \sigma}{c_r}$$

where $q_u = \text{cone resistance}$
$\sigma = \text{total overburden pressure}$
$c_r = \text{field vane shear strength}$

All the derived $N_q$-factors have been reviewed. Where extreme values of $N_q$ were recognized these were checked against the soil type, activity index, and other parameters which could affect them, before acceptance.

Vane tests influenced by stones have been disregarded. Vane tests with refusal readings have also been omitted.

The registered $q_u$-values from each individual CPT have been smoothed and averaged every 0.2 m of depth.
Smoothed $q_u$-values within a horizontal radius $R = 5$ m from the boring have been interpolated to the boring location by weighting with the horizontal distance to the boring.

Within a limited vertical band with height $H = 0.42$ m ± 0.21 m above and below the considered vane test elevation the interpolated $q_u$-values have likewise been smoothed.
Average interpolated $q_u$-values are calculated at the individual vane test elevation in the borings for comparison with individual vane tests or every 0.2 m of depth for comparison with the interpreted vane test. Interpolated $q_u$-values with friction ratio $R_t = f_u/q_u$ in \% outside the interval 1.0% $\leq R_t \leq 6.0\%$ have been disregarded.
4. PROCEDURE TO CALCULATE THE ACTIVITY
The activity of the clay is calculated as the plasticity index of the clay as determined by the Atterberg limits, divided by the clay content (μ ≤ 0.002 mm).

5. PROCEDURE TO CALCULATE THE OCR
The overconsolidation ratio OCR is calculated in accordance with the SHANSEP Method. The SHANSEP method was developed at MIT in the USA and is described by Ladd et al. (1977).

The OCR is calculated as:

\[ OCR = \frac{\sigma_v}{0.4\sigma'} \]

where σ is the vertical effective in situ strength.

The constants 0.4 and 0.85 are taken from Steenfelt et al., 1992.

6. DATA PROCESSING
The vane shear strengths used in the data set for the correlation analysis are obtained from Danish Deep Vane Test System. The equipment is described by Mortensen et al. (1991) in detail.

A total number of 98 data sets of Nc, Ia, OCR have been found suitable for direct comparison with a nearby CPT. The CPT qc results are applied in the correlation analysis and are taken as a smooth cone resistance.

In Table 2 the results are given with the geometric mean values of activity index, vane shear strength, cone resistance and OCR. S is the standard deviation.

<table>
<thead>
<tr>
<th>No</th>
<th>Ia</th>
<th>S</th>
<th>c'</th>
<th>S</th>
<th>qc</th>
<th>S</th>
<th>OCR</th>
<th>S</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>98</td>
<td>0.36</td>
<td>0.11</td>
<td>230</td>
<td>93</td>
<td>2.21</td>
<td>0.7</td>
<td>9.1</td>
</tr>
</tbody>
</table>

Table 2. Data Set Values

7. CORRELATION BETWEEN CONE FACTOR Nc, THE ACTIVITY INDEX AND OCR
The distribution of the considered data sets (Nc, Ia, OCR) are illustrated in Figure 1.

![Distribution of Considered Data Sets (Nc, Ia, OCR), Eastern Storebælt](image)

In Fig. 1 all the data sets are plotted and straight lines with the OCR-values are plotted. The OCR-lines are determined by linear regression analysis through (0,0). The cone factor increases with activity and OCR.

It appears that the values of Nc are grouped around Nc = 10, which was derived at the Storebælt project (Mortensen et al).

8. COMPARISON WITH OTHER INVESTIGATIONS
Rad (1988) summarized information concerning the cone resistance, total overburden, undrained shear strength, OCR and
classification parameters for 11 different clayey soils.

The undrained shear strengths are based upon consolidated undrained compression triaxial tests. Based upon the available information the cone factor $N_k$ and the activities of the clays are calculated by the authors. Luke (1994) summarized information concerning 7 different clayey soils in Denmark. These data sets are also illustrated in Figure 2.

The distribution of the considered data sets ($N_k$, $I_A$, OCR) is illustrated in Figure 2.

![Figure 2: $N_k$, $I_A$, OCR Plot Different Sites](image)


In Fig. 3 mean values of cone factor and activity at different locations are presented.

Marsland (1988) presents results of plate load tests, cone penetration tests, and classification parameters for Cowden, Yorks glacial till and Boom clay in Belgium.

Cancelli et al. (1982) reported a cone factor of 25 for overconsolidated Modena clay in the southern Po Valley in Italy. The undrained shear strength is estimated based upon laboratory tests.

The activity index is reported as generally close to 0.75. The clay is described as silty clays to clayey silts.

The Niva site, Denmark, consists of mixed clay profile with glacial and late glacial meltwater clay deposits. The clay is overconsolidated. Ref. Denver 1988.

![Figure 3: $N_k$, $I_A$ Plot Mean Values for Different Sites](image)

Denver (1988) reported the following classification parameters, plasticity, and vane shear strength for the Niva clay.

$I_A = 10-15\%$
$c_v = 200-400 \text{ kPa}$

The cone factor $N_k$ is in average determined to 8 for the clay, based upon field
vane shear test. From other investigations the clay content is approx. 45%.
The activity of the clay is calculated to approx. 0.28.

9. CONCLUSIONS

A comparison between different classification parameters has revealed a relation between cone factor \( N_c \), the Activity \( I_a \) as well as the OCR.

The results show two trends:
- An increase in \( N_c \) with increasing \( I_a \).
- An increase in \( N_c \) with increasing OCR.

A better estimate of \( N_c \) is obtained by correcting for the Activity Index than by assuming \( N_c \) to be a constant values.

A better estimate of \( N_c \) is obtained by correcting for \( I_a \) and OCR than by only correcting for \( I_a \).

10. REFERENCES


Seepage analysis from piezocone dissipation tests

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SYNOPSIS: Piezocone testing (CPTU) has proven to be an excellent method to delineate stratification in highly layered soils such as mine tailings dams. Dissipation tests carried out during piezocone testing give a very good indication of the permeability of the soil as well as the distribution of piezometric pressures. This paper gives a new perspective on the interpretation of CPTU dissipation tests.

It is customary to carry out complete dissipation tests at intervals of 3 to 4m in tailings dams. However, there are many opportunities to record the initial portions of dissipation curves at rod changes or other stoppages, producing incomplete dissipation results. A procedure is presented to determine the ambient pore pressure from such incomplete dissipation results. Furthermore, a method is presented to determine the anisotropy in permeability from the pore pressure regime determined from the dissipation tests.

The above, together with the standard interpretation of dissipation rates, define the entire flow regime in a tailings dam. This is illustrated by means of a case study recently carried out on a mine tailings dam in South Africa.

1. INTRODUCTION
In February 1994 a major failure of a gold mine tailings dam occurred at the Merriespruit Gold Mine on the Free State Gold Fields, South Africa. A section of the suburb of Merriespruit was swept away and engulfed by slimes. The number of fatalities totalled 17. The cost of the disaster, including production loss and civil claims, which have not been finalised, will be enormous. This graphically illustrates the risks of tailings dam failures and sparked a campaign of investigations into the safety of tailings dams in South Africa.

One of the routine aspects of such an investigation is the determination of the seepage state and flow regime in a tailings dam. It is now almost mandatory in South Africa to carry out piezocone tests to assess the conditions of tailings dams. These tests give an excellent delineation of the profile (Vermeulen and Rust, 1995) with estimates of all the geotechnical parameters needed to carry out a complete safety analysis of the dam. Dissipation tests are carried out during piezocone testing in order to obtain the ambient pore pressure and an estimate of the permeability at the test position. This paper deals with the use of dissipation test results in determining the flow regime in tailings dams.

2. CPTU DISSIPATION TESTS
It is customary to carry out full dissipation tests at three or four metre intervals, or where a large excess pore pressure is generated during a CPTU sounding. The time for 50% of the induced excess pore pressure to dissipate (1/2) provides an indication of the permeability of the soil (Leduc and Baligh, 1980 and Torstensson, 1977). The asymptote of the decaying pore pressure is the ambient pore pressure at the position of the test. These full dissipation tests, although

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time consuming, are vital in the interpretation of the seepage regime. A typical profile taken from a tailings dam is shown in Figure 1 and consists of numerous layers.

\[ \frac{\partial u}{\partial t} = c \left[ \frac{1}{r} \frac{\partial}{\partial r} (r \frac{\partial u}{\partial r}) \right] \]  \hspace{1cm} (1) 

where \( u \) represents pore pressure, \( t \) time, \( c \) the coefficient of consolidation and \( r \) any radius (measured from the probe’s axis of symmetry).

The equation can be solved to produce an expression for the pore pressure in terms of \( r \) and \( t \) (equation 2). (The solution of the problem is discussed in detail by Randolph and Wroth (1979)).

\[ u = \sum_{n=1}^{\infty} B_n e^{-\alpha_n^2 t} C_n(\alpha_n r) \]  \hspace{1cm} (2)

\( \alpha_n \) is a separation constant and 

\[ B_n = \frac{4c_0}{\lambda_0^2} \frac{[C_d(\alpha_n r_0) - C_d(\alpha_n R)]}{[\alpha_n^2 C(\alpha_n r_0) - \lambda_0^2 C_d(\alpha_n r_0)]} \]  \hspace{1cm} (3)

where \( c_0 \) is the undrained shear strength, \( \lambda_0 \) a constant determined from the boundary conditions, \( r_0 \) the probe radius, \( R \) the radius where pore pressure increase due to penetration is zero immediately after penetration and \( r^* \) the minimum radius where pore pressure increase is negligible during and after dissipation. \( C_d \) and \( C_f \) are cylinder functions in terms of Bessel functions.

From the theory of expanding cavities \( R \) can be expressed in terms of \( r_0 \):

\[ R = r_0 \left[ \frac{G}{c_0} \right]^{1/2} \]  \hspace{1cm} (4)

where \( G \) is the shear modulus of the material under consideration.

Terzaghi’s consolidation equation is solved subjected to the relevant boundary conditions for a range of \( G/c_0 \) values. A radial logarithmic initial excess pore pressure distribution was assumed immediately after penetration arrest. The resulting expressions obtained for pore pressure (at the cavity wall) (equation 2) may be normalised in the usual way. Time can be normalised as follows.
where $T$ is normalised time and $r$ the radius of the cavity.

The family of normalised curves is shown in Figure 2.

![Figure 2. Normalised dissipation curves and the mine tailings envelope](image)

4. AMBIENT PORE PRESSURE FROM INCOMPLETE DISSIPATION TESTS

An observed dissipation curve may be normalised, given the correct choice of ambient pore pressure and $G/c_w$ ratio, as follows.

$$ U = \frac{u - u_0}{u_T - u_0} $$

where $U$ is the normalised pore pressure, $u_T$ the ambient pore pressure and $u_0$ the total initial pore pressure before dissipation.

Time may be normalised as follows:

$$ T = \frac{ct}{r^2} = \frac{T_{50}}{r_{50}} $$

where $T_{50}$ is the time for 50% dissipation from the theoretical curve and $r_{50}$ the time for 50% dissipation from the observed curve.

Because $T_{50}$ depends on the choice of $G/c_w$, the first step in finding the ambient pore pressure entails an estimate of the material’s $G/c_w$ ratio. A trial ambient pore pressure is chosen to normalise the observed curve. The normalised curve is then compared to the relevant theoretical curve. If the observed curve follows the theoretical curve very closely, the correct ambient pore pressure has been chosen initially. If the end of the observed curve falls below the theoretical curve, the ambient pore pressure chosen was too high and vice versa. If the maximum curvature of the observed curve is more than that of the theoretical curve, the $G/c_w$ ratio chosen was too high.

This method was found to be valid for clayey material, but for more silty material such as mine tailings, the normalised dissipation curves tended to undercut the theoretical curves. Many complete dissipation curves from gold mine tailings were normalised on the $G/c_w = 25$ curve and were all found to fall within the envelope shown in Figure 2. By following the routine described above and using the envelope as theoretical curve, it is possible to estimate the ambient pore pressure from incomplete dissipation tests in mine tailings.

5. ANISOTROPY IN MINE TAILINGS

Mine tailings are highly layered due to the segregation during the deposition process. This results in a macro anisotropy in permeability. Coarse layers typically consist of a fine to medium sand and the fine layers of fine silt to silty clay. The thickness of these layers varies greatly but is typically between 0.1 and 1 m. These layers are not continuous due to the nature of the deposition process, as well as cracking and infilling that takes place after deposition when the tailings dry on the beach. As mentioned earlier, dissipation tests are carried out in only a limited number of these layers. With the additional information from incomplete dissipation tests a more complete picture of the permeabilities of the various layers can now be obtained. A complete profile of all layers can be drawn up from the Soils Identification Chart (Jones and Rust, 1982) adopting the approach by Vermeulen and Rust (1995). Using the permeabilities obtained from dissipation tests as benchmark, the permeability of all other layers can be estimated from the profile. It is therefore possible to calculate the
total horizontal, as well as vertical permeabilities from first principles, assuming that all layers are continuous. The anisotropy in permeability calculated in this way will be an absolute upper bound. The vertical permeability will be more affected than the horizontal permeability by the fact that the layers are not continuous. In fact, the calculated overall horizontal permeability will be close to the actual in situ value.

6. **ANISOPTROPY FROM PIEZOCONE TESTS**

It is possible to determine the actual in situ macro anisotropy in permeability at any point along the phreatic surface. The method takes into account the effects of non-continuity and cracking of layers. It is derived from the slope of the phreatic surface and the ambient pore pressure profile. This profile is a plot of the ambient pore pressure with depth derived from the CPTU dissipation tests. There is a unique relationship between the slope of the phreatic surface, the rate of ambient pore pressure increase with depth and the anisotropy at any point along the phreatic surface.

Consider a section through a tailings dam (see Figure 4). For this two dimensional situation, the total head $h(x,y)$, which is the solution to the continuity equation

$$k_x \frac{\partial h}{\partial x} + k_y \frac{\partial h}{\partial y} = 0 \quad (8)$$

represents a surface over the $xy$ plane (Figure 3). The CPTU test results provides information describing some of the characteristics of this surface. This information includes the following:

Firstly the CPTU test is conducted in the vertical or $y$ direction. In the $x$ direction the ambient pore pressure at various depths can be calculated to obtain the ambient pore pressure profile. By choosing an arbitrary datum the total head $h$ can be calculated by using the simple relationship

$$h = \frac{p}{y} + z \quad (9)$$

By fitting a curve through the ambient pore pressure profile, the slope of the curve

$$\frac{\partial h}{\partial y} = \tan \beta \quad (10)$$

can be calculated, where $\beta$ is illustrated in Figure 3.

![Figure 3. Definition of the geometry of the surface $h(x,y)$](image)

Secondly by interpolating between CPTU tests conducted at the section, the position of the phreatic surface in the $xy$ plane can be determined. The slope of the phreatic surface

$$\frac{dy}{dx} = \tan \alpha \quad (11)$$

can be calculated, where $\alpha$ is illustrated in Figure 3. Furthermore it is known that along a flow line, the phreatic surface in this case, the relationship

$$\frac{y}{x} = \tan \alpha \quad (12)$$

is applicable.

It is also known that on the phreatic surface $p = 0$ and therefore equation 9 reduces to

$$h = z = y(x) \quad (13)$$

It is therefore evident that
Figure 4. A section through a gold tailings dam showing the CPTU test results and information determined from the test result used to calculate the anisotropy at each test position.

\[
\frac{\partial h}{\partial x} = \frac{dz}{dx} \frac{dy}{dx} = \tan \alpha 
\]

(14)

on the phreatic surface.

Applying Darcy's law in the \( x \) and \( y \) directions gives

\[
v_x = k_x \frac{\partial h}{\partial x}
\]

(15)

and

\[
v_y = k_y \frac{\partial h}{\partial y}
\]

(16)

Equation 15 divided by equation 16 results in

\[
v_x = k_x \frac{\partial h}{\partial y} \frac{\partial y}{\partial x} 
\]

\[
v_y = k_y \frac{\partial h}{\partial x} \frac{\partial x}{\partial y}
\]

(17)

and substituting equations 10, 12 and 14

\[
1 = k_x \tan \alpha \frac{\tan \alpha}{k_y \tan \beta}
\]

(18)

or

\[
k_x = \frac{\tan \beta}{\tan^2 \alpha} k_y
\]

(19)

Since \( \alpha \) and \( \beta \) can be determined from the CPTU test results, equation 19 can be used to calculate the ratio of \( k_x \) to \( k_y \), the anisotropy, on the phreatic surface at the section.

To illustrate the calculation of the ratio between vertical and horizontal permeabilities, consider the section through a tailings dam shown in Figure 4. Four CPTU tests were conducted at the positions indicated.

Using \( \alpha \) and \( \beta \) values as indicated in Figure 4 the anisotropy at each of the test positions can be calculated using equation 19. The results are summarised in Table 1.

<table>
<thead>
<tr>
<th>Test position</th>
<th>Slope of phreatic surface (( \alpha ))</th>
<th>Slope of pressure distribution (( \beta ))</th>
<th>Ratio ( k_x/k_y )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9°</td>
<td>29°</td>
<td>0.28</td>
</tr>
<tr>
<td>2</td>
<td>10°</td>
<td>17°</td>
<td>0.57</td>
</tr>
<tr>
<td>3</td>
<td>11.5°</td>
<td>23°</td>
<td>0.52</td>
</tr>
<tr>
<td>4</td>
<td>11°</td>
<td>17°</td>
<td>0.68</td>
</tr>
</tbody>
</table>

These values are significantly higher than the ratios quoted by Blight and Steffen (1979) of between 1.5 and 3. It should however be noted that the ratios quoted by Blight and Steffen were based on permeability measurements which were made under isotropic confining stresses in triaxial cells on undisturbed tailings specimens taken from five different dams. The permeabilities determined in the laboratory do not take into
account all the factors which may influence the anisotropy. These factors include:
- the development of cracks and the infilling of these cracks,
- the complexity of the layering in the dam, and
- the extent and continuity of the various layers.

7. COMPLETE FLOW REGIME
Once the anisotropies at points along the seepage path are calculated and an estimate of the overall horizontal permeability has been made, it is possible to solve the complete flow regime. It is also possible to compare the calculations of seepage with measurements taken from the drainage system as well as estimates from the rate of deposition. Readings of piezometric pressures recorded from properly installed piezometers at the time of the investigation should correlate with the seepage analysis. Changes in piezometer readings should prompt a reassessment of the phreatic regime to establish its influence on the stability of the dam.

8. DISCUSSION AND CONCLUSIONS
Due to the extensive stratification present in most tailings dams, conventional methods of piezocone dissipation test interpretation do not supply enough information to reliably assess the condition of a tailings dam. By interpreting previously unused data according to the methods described in this paper, the flow regime in a tailings dam can be determined in much greater detail than before.

When an observed incomplete dissipation curve is available where, as a rule of thumb, 50% of the dissipation process has been recorded, it is possible to estimate the ambient pore pressure remarkably accurately. By interpreting a number of dissipation tests the entire ambient pore pressure profile can be determined which can be further interpreted to yield the anisotropy in permeability, thereby defining the entire flow regime in a tailings dam.

Due to the large flow gradients in tailings dams, piezometers do not indicate the phreatic surface and do not provide sufficient data to carry out such an analysis. However, it is possible to use the information from the piezometers to detect any radical change in the seepage regime and send alarm signals for further action. It is essential to record the flow regime in a tailings dam in fairly great detail in order to assess its implications on the slope stability and therefore also on the safety of the dam. The techniques described in this paper provide the means to accurately determine the condition of a tailings dam and thus minimise the risk of dam failure.

9. REFERENCES
The Approach to Application of Static CPT Together With Other Methods of Soil Investigation.

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SYNOPSIS: The method of joint usage of static Cone Penetration Test (CPT) together with other methods of soil investigation is worked out. The method provides the reciprocal correction of results obtained and allows to choose the design parameters that take into account all available information. It is supposed that the CPT number significantly exceeds the number of other soil investigation methods. The tests, results of which can be taken as pattern, are used for less precise investigations correction including the CPT itself. The choice of design parameter is made with the account of possible errors of each test method. For this, data of previous practice - tables and histograms of possible errors distribution - is used in computation. The theoretical basis of computations is Bayes theorem.

The suggested complex evaluation of results provides more reliable and economical decisions.

1. INTRODUCTION
The simplicity and quickness of static CPT compensate in a significant degree its inaccuracy in evaluation of geotechnical parameters such as pile bearing capacity, the deformation modulus, soil strength, etc.

The large number of measurements allows to define the dimensional variation of these parameters and to decrease the accidental errors at the expense of averaged values. Nevertheless, the inevitable errors of systematic character are not decreased at averaging. In order to take account of these errors the additive measurements are necessary that are carried out with other methods, more precise. Such methods are pile tests, in situ plate load tests, sometimes boring and laboratory soil analysis, etc. However, the parallel usage of methods with different degree of reliability is a complex, not fully decided problem. In practice, in such cases an engineer must rely upon its own experience and intuition. Below the possible way of given problem decision is shown that is used in practical work of the Institute BashNIIstroi.

2. THE THEORETICAL PRECONDITIONS AND MAIN POINT OF THE SUGGESTED APPROACH
The investigations of two recent decades for example J.Kay (1976), A.Cividini, et.al(1983), I.B.Ryzhkov (1989) showed that in the sphere of complex evaluation of the geotechnical parameters it is long-term enough to use the Bayes theorem. Its essence is the following. Let's take 'n' incompatible hypotheses $H_1, H_2, ..., H_n$ with probabilities $P(H_1), P(H_2), ..., P(H_n)$. According to any of these hypotheses we may expect elementary events $\alpha_1, \alpha_2, ..., \alpha_m$ with their probabilities $P(\alpha_i/H_1), P(\alpha_i/H_2), ..., P(\alpha_i/H_n), P(\alpha_i/H_1), P(\alpha_i/H_2), ..., P(\alpha_i/H_n), P(\alpha_i/H_1), P(\alpha_i/H_2), ..., P(\alpha_i/H_n)$. After the realization of one of the events $\alpha_k$ the probabilities of all hypotheses are changed and become equal to:

$$P(H_i | \alpha_k) = \frac{P(H_i) \cdot P(\alpha_k / H_i)}{\sum_{i=1}^{n} P(H_i) \cdot P(\alpha_k / H_i)} \quad (1)$$

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Fig. 1. Histogram of errors distribution $\Delta$ at pile bearing capacity evaluation by CPT data. $n'/N$ - errors relative frequencies $\hat{\Delta}$; quantization increment of errors is 0.1.

where $P(H_i)$ and $P(H_i/\alpha_k)$ are probabilities of $i$-th hypothesis before and after precise definition, named "a priori" and "a posteriori", accordingly:

$P(\alpha_k/H_i)$ is the probability of the event $\alpha_k$ provided the hypothesis $H_i$ is true;

$\sum_{i=1}^{n'} P(H_i) \cdot P(\alpha_k/H_i)$ is the full probability of the event $\alpha_k$.

In given geotechnical problems it is advisable to consider as hypotheses $H_1$, $H_2$, ... $H_n$ the suppositions of possible mean values of the parameter to be determined: soil deformation modulus, pile bearing capacity, etc. At the beginning of computation these hypotheses can be considered as equiprobable in the whole range of possible values of this parameter. In dependence on the desired accuracy of the computation the number of such hypotheses can be various.

For example, having taken the precision of deformation modulus calculation to be 1 MPa and the range of modulus possible values 1...50 MPa, one can consider 50 hypotheses $E=1$, 2, 3, ... 49, 50 MPa. Each hypothesis probability will be 1/50. At precision 0.1 MPa the hypotheses number will increase up to 50 (E=0.1, 0.2, 0.3, ... 50 MPa). The probabilities will be equal to 1/500.

The event $\alpha_k$ is the obtaining of any approximated value of the parameter to be determined directly in situ. The probability of errors arising will depend on the reliability of the test method and can be stated according to results of static analysis of the previous test data. The results must be presented as the distribution of possible errors frequencies. Such frequencies must be evaluated both for CPT and all other test methods. After the corresponding aligning and making corrections for soil heterogeneity the corrected relative errors frequencies can be approximately considered as probabilities.

Figure 1 shows an example of possible errors distribution of pile bearing capacity determination by CPT data (with a unit S-832 that has a probe "SE" $A_F=310$). It is evident that the character of such distribution will depend not only on the test method, but on applied formulas as well.

The probabilities of the specific test result (i.e. $P(\alpha_k/H_i)$) obtaining are stated as applied to each hypothesis $H_i$. For this, errors relative to supposed true parameter values by each hypothesis $H_i$ are determined. Then with the help of errors distribution (fig.1) the probabilities of the result obtained for each hypothesis are stated. Thus, the values found allow to make calculation by formula (1).

Results of all site tests are put in turn into this formula. A posteriori hypotheses probabilities of each former calculation
cycle are taken as a priori in the next cycle. As new data is inputting, the range of possible values of the parameter to be determined, their dispersion gradually decrease. The final distribution of hypotheses probabilities usually has small dispersion. The calculation parameter is chosen proceeding from the given confidence probability.

Thus, in above approach the procedure of taking account of the data obtained becomes the same with any information sources. Different by reliability the methods are characterized only with the different errors distribution and their conditional probabilities \( P(\alpha_1 | H_1) \), accordingly. That's why CPT data can be supplemented with any information, the influence of which upon the final result will depend on its reliability.

It is evident that in practice such calculations should be computerized.

If at site except the approximate tests the exact tests are carried out results of which can be taken as pattern, the possibility of additive correction of approximate data occurs. So, at pile bearing capacity determination the exact tests are pile static tests, at the modulus of deformation evaluation - in situ plate load tests.

For soil correction the "key zones" at the site are necessary where places of exact and approximate tests coincide and their results can be compared. The correction consists in specifying the conditional probabilities distribution \( P(\alpha_1 | H_1) \) in conformity with the specific site.

The principle possibility of above correction is conditioned by relative stability of existing errors within small sites. As a rule, the range of their possible variations decreases compared with a general case. The mean error values are biased at each site differently. The Institute BashNIIsstroj investigations that include the analysis of large number of experimental data, showed that when practical problems solving it is possible for small sites (3000...4000 m²) to take approximately the standard deviation of particular error values to be 0.8σ, the standard deviation of mean error to be 0.6σ (σ - the standard deviation of errors in a general case, i.e. for all possible conditions).

The correction process is based on Bayes formula. In this case possible errors values are considered as hypotheses, and the actual error at the "key zone" - as an event. A priori probabilities of such hypotheses \( P(H_1) \) and conditional probabilities of the corresponding events \( \alpha_1 \), i.e. \( (\alpha_1 | H_1) \) are determined by means of the reorganization of common distribution (of the type shown in figure 1). As all former distributions, such a reorganization can be made without approximation itself with the analytical law. The standard deviation decrease can be reached at the expense of intervals width decrease (multiplication by 0.8 and 0.6) and their following regrouping by the necessary width. In the presence of several "key zones" the error distribution correction is made as much as there are "key zones". The a posteriori hypotheses probabilities are transformed in the following computations into the new conditional probabilities of obtaining one or another certain result \( P(\alpha_1 | H_1) \).

The correction significantly increases the approximate tests efficiency. The dispersion of possible values of the parameter to be determined decreases with regard for every new result significantly faster than without correction. This leads to more definite final distribution and allows to take more economical decisions without prejudice to foundation reliability.

The above approach to data processing differs from already known approaches of J.Kay (1976), A.Cividini (1983) in the first place with the discrete distributions usage without their analytical approximation. This provides the necessary stability of the design parameter choice. In particular the distributions approximation with the normal law makes the calculation parameter "travel" through the whole range from \(-\infty\) up to \(+\infty\) without limits. In spite of any confidence probability of the parameter, new data can move it into the zone of values the possibility of which was up to now neglected. The use of analytically not approximated discrete distributions eliminates this defect. These distributions can be presented as truncated distributions.
3. THE EXPERIMENTAL EXAMINATION

The experimental examination of the above approach has been carried out as applied to pile bearing capacity determination. The examinations were carried out at sites where more than 6 piles were tested with the static load. The bearing capacity determined according to so large number of static pile tests could be taken as standard for comparison with the result of the examination with less number of tests.

In most full scale the tests were carried out in Ufa, at the site of 12 x 12 m size where 11 piles were driven and tested with static load. Two piles were tested with dynamic load. The tested piles were 0.3 x 0.3 m section and 6 m long. Points of driving were evenly distributed along all the site, so the distance between piles was 3.5...5 m. At the point of each pile driving CPT up to the depth of 11 m was preliminary carried out with the unit S-832 that has a probe with a friction coupling "SF" (A=310 mm). Five boreholes were bored between piles up to 15 m depth, monoliths were sampled and laboratory tested.

Soils at given site were alluvial-deluvial medium clays without any foreign layers and lenses.

Results of pile bearing capacity determination with all methods applied at given site are shown in the table.

<table>
<thead>
<tr>
<th>Points of pile driving</th>
<th>CPT 241</th>
<th>laboratory soil analysis 304</th>
<th>dynamic tests 200</th>
<th>static tests 250</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>234</td>
<td>234</td>
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<td>2</td>
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<tr>
<td>11</td>
<td>247</td>
<td>247</td>
<td>282</td>
<td>250</td>
</tr>
</tbody>
</table>

Calculations were made by the programme SYNT, the algorithm of which corresponds to the content of section 2.

Figure 2 shows the results of such calculations as dependencies of calculated bearing capacity upon the number of static pile tests N. As each variant of works gave its own calculated pile bearing capacity, figure 2 shows the generalized results of its evaluation. It is mean F̄ and maximum Fmax of its value for each group of uniform works (they should not be mixed with the mean and maximum result of tests at site).

Figure 2 shows also the results of pile bearing capacity calculation by data of different number of pile tests according to Russian Code SNiP 2.02.03-85. According to this Code with pile static test number form 1 up to 5 the minimum result of these tests is taken as bearing capacity, the result then is divided by reliability coefficient γk=1.2. With more tests number the minimum value is estimated by statistical calculation with confidence probability 0.95. Figure 2 shows that the influence of geotechnical works structure is the greater the less number of static pile tests is carried out.

When using only static CPT at 11 points of pile driving the pile bearing capacity was 12% less than in case when except CPT,
Fig. 2. Diagrams of design pile bearing capacities variations depending on static tests number N: a) the mean values for each group of uniform works \( (F_{\text{mean}}) \); b) the maximum values \( (F_{\text{max}}) \): 1 - CPT and static pile tests; 2 - the whole complex of investigations; 3 - static tests only.

calculations with the help of laboratory soil analysis data and dynamic pile tests were used. If static test of one pile was added, the bearing capacity \( F_{\text{mean}} \) increased on the average by 17% with testing of 2 piles the bearing capacity \( F_{\text{mean}} \) increased by 25% and so on. The increase of static pile tests number gradually brings the calculated bearing capacity nearer to its true value of 235 kN.

The result of evaluation by Code SNIP 2.02.03-85, i.e. by static tests data, is on the average about 20% lower than by complex evaluation.

The maximum evaluation results shown in figure 2b characterize the reliability of taken decisions. They show that the complex estimation doesn't lead anywhere to increase of the true value 235 kN. But if we use only one static pile test, the bearing capacity can be taken as 275 kN, it is 17% more than the true value 235 kN. Thus, the complex evaluation provides much more reliability.

The same results have been obtained at other, much larger sites, with greater distance between places of investigation.

4. PRACTICAL REALIZATION

The technology of soil conditions investigations based on above approach can be presented as follows. Points of CPT are evenly distributed over the site, the distance between them depending on complexity of soil conditions is 10...30 m. Other methods of soil evaluation are used in much less number than CPT number. The exact tests, the results of which are used as pattern, are carried out at the most typical site parts. Static CPT must be made at the same site parts and if it is possible, tests with other methods must be carried out just here. It
will allow to correct the errors of all data obtained. Because of calculations difficulty the information must be computer processed.

5. SUMMARY
It is suggested to consider all the results obtained not separately but as a single whole of data for design soil parameters determination while using static CPT together with other methods of soil investigation. The CPT results are shown as some relative characteristic of parameter to be determined. This characteristic is corrected and supplemented with other data. The above approach based on Bayes formula allows to take into account the effect of each particular result and soil heterogeneity. This approach allows to increase the reliability and economy of design soil parameters choice.

In particular, when making investigations for pile foundations the approach allows to take the values of pile bearing capacity by 18...20% higher than with the traditional approach and provides the reliability of decisions when carrying out only one pile static CPT.

6. REFERENCES
The Swedish Geotechnical Society (SGF) was formed in 1950 and has currently 650 members with at least two years experience in geotechnics. In addition, there are some 30 corporate members comprising institutions, universities, official bodies, consultants, contracting companies and manufacturers with activities in the area of geotechnics.

The objective of the SGF is to promote development in geotechnics and foundation engineering through lectures, discussions and committee work, and to cooperate with Swedish, Nordic and other international bodies having a similar orientation.

The SGF is the Swedish representative of the International Society for Soil Mechanics and Foundation Engineering (ISSMFE). Every member of the SGF is also a member of the international society.

The series of Reports published by the SGF contains recommendations for geotechnical standards, in addition to monographs and documentation from conferences, seminars and other events.

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