International Symposium on Cone Penetration Testing

Volume 2 • Technical Papers 2B

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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-1 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 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 Rydell
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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Proceedings CPT’96
CPT Performed In Sand Models On A Centrifuge

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SYNOPSIS: Cone penetration tests were performed to check the consistency and uniformity of soil samples used in centrifuge model tests and estimate the shear strength of sand being tested. This paper presents some test data which are compared with theoretical or experimental methods. In addition, some factors which affect cone resistance are also discussed.

1. INTRODUCTION

Since model tests are generally performed on different samples, it is necessary to quantify the soil resistance and its variation between different samples in order to interpret test results. Cone penetration tests can give a continuous variation of cone resistance (q) with soil depth (D), thus it is often taken as ‘site investigation’ in the centrifuge model tests.

In practice, the cone penetration test is widely used for the in situ determination of soil parameters such as internal friction angle (φ°) or relative density (D_r). There are numerous theoretical and empirical correlations between measured cone resistances and soil engineering properties in recent geotechnical literature, and it may be difficult to confirm which method would be more suitable to interpret the test results. Thus a comparison between some available methods for evaluating the shear strength of soil from cone resistance may be appropriate.

Five cone penetration tests have been undertaken at different g levels and summarised in Table 1, together with key values of cone resistance. In this paper some test data are presented briefly followed by a description of two methods for deducing shear strength from the cone resistance.

<table>
<thead>
<tr>
<th>Test</th>
<th>g (m/s²)</th>
<th>ρ_s (kN/m³)</th>
<th>e</th>
<th>D/B (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SQ18</td>
<td>28.6</td>
<td>15.9</td>
<td>0.67</td>
<td>20.37</td>
</tr>
<tr>
<td>SQ19</td>
<td>28.6</td>
<td>15.8</td>
<td>0.67</td>
<td>16.55</td>
</tr>
<tr>
<td>SQ20</td>
<td>20.0</td>
<td>16.0</td>
<td>0.65</td>
<td>14.26</td>
</tr>
<tr>
<td>SQ22</td>
<td>40.0</td>
<td>15.9</td>
<td>0.66</td>
<td>21.65</td>
</tr>
<tr>
<td>SQ23</td>
<td>40.0</td>
<td>15.9</td>
<td>0.66</td>
<td>21.96</td>
</tr>
</tbody>
</table>

Note: \( \gamma_s = 0.920 \), \( c_{max} = 0.548 \)

2. SAMPLE PREPARATION

The soil tested was Fontainebleau sand, with particle size in the range of 0.13-0.18mm. The properties of the sand have been investigated extensively (Luong, 1980).

Three samples of saturated sand were prepared in centrifuge tubs with an internal diameter of 830mm and a depth of 400mm. The sand was poured dry from a hopper suspended from a crane at a given pouring rate of about 1kg/min, and a constant free fall height of about 1m, because the void ratio (e) of a sample is...
dependent on both pouring rate and free fall height. The sand was generally placed in 5 layers, the bottom layer was a layer of coarse sand about 40-50mm thick, acting as a drainage to allow uniform water flow during saturation. Care should be taken to ensure that the sand surface was raised uniformly across the whole area of the tub. The final sand surface was levelled using a vacuum cleaner, giving a sand sample of about 280-310mm thick.

A steel lid with rubber ring was fixed by bolts on the top of the tube to form a closed container. A suction pressure of 26 inches mercury was applied in the tub, carbon dioxide was then gently introduced and flushed through the sample until the vacuum dropped to zero. The process was repeated to ensure the spaces between sand grains were occupied by carbon dioxide. Finally water flowed very slowly into the sample from two inlets fixed on the bottom of the back wall of the tub.

The saturated unit weights of samples were 19.6-19.7 kN/m³, and the void ratios between 0.65-0.67.

3. CONE PENETROMETER

Basically the penetrometer can be divided into two parts, i.e. the hydraulic system and the electrical system, as indicated in Figure 1.

In the hydraulic system there is a double acting hydraulic cylinder. A 30mm diameter rod fitted with a 60 deg. conical tip is connected to the piston of the cylinder. Pressurised water is fed to the upper side of the piston by high pressure nitrogen stored in two accumulators situated near the axis of the centrifuge. The lower side of the piston is fed with a regulated supply of nitrogen to balance the weight of the probe and the piston, and also the head of water in external piping along the centrifuge arm, then in flight the penetration of the probe into the sand is provided by this pressure.

During penetration water is fed from high pressure accumulators, while nitrogen is forced out from the bottom as the probe enters the sand, and a pressure relief valve prevents a build up nitrogen pressure. To ensure constant penetrating rate the pressurised water is fed through a flow control valve.

The electrical system consists of load and pressure transducers. A load cell directly behind the top measures the cone resistance. The depth of penetration is monitored by a rotary potentiometer mounted on the bottom cap. A recoiling spring is wound round the potentiometer drum and attached to the position at the top of the probe. The maximum travel of the probe is limited by the end stop mounted on the lower position of the cylinder. In the penetration tests the travel was about 365mm, and penetrating rates were in the range of 1.6-3.5mm/s.

Figure 1. The cone penetrometer (a) hydraulic system (b) electrical system

The other two transducers are fitted at the water and nitrogen inlets to measure the water and
nitrogen pressure. Thus, the measurement of load needed to force the probe into the sand is obtained based on a difference in pressures, apart from the differential pressure transducer fitted into the piston.

4. TEST PROCEDURE

All transducers were calibrated and checked in laboratory floor. The general arrangement of the centrifuge test package is shown Figure 2.

The centrifuge speed was increased in steps of 10g until a required acceleration. In each step it was important to check the change of water surface according to outputs of a pore pressure transducer inserted in a standing pipe which was mounted on the tub base. When the speed has reached the required value, the centrifuge was allowed to run for half an hour to enable the pore pressure to approach equilibrium before cone penetration tests commence.

![Figure 2. The centrifuge test package](image)

Signals from transducers were amplified through amplifiers and recorded on a 14 analogue tape recorder, data stored in the magnetic tape were digitised on a minicomputer using a processing programme and plotted out later.

5. TEST RESULTS

The profile of soil resistance from tests is shown in Figure 3 in the model scale. Two features could be found from this diagram. Firstly, an approximate linear relationship between cone resistance and soil depth was held to some depth (about 200mm) except for an initial zone of about 100mm deep where there were some surface effects. Secondly, below this depth the cone resistance tended to reduce and then eventually increased once more. In tests SQ18 and SQ19 the zone in which the reduction of cone resistance was very obvious, and occurred almost at the same depth. However, in the other tests this reduction was smaller. This may be due to a result of entraining a relatively loose sand profile. Alternatively, this phenomenon might be explained in terms of the change of the frictional resistance of the soil with the stress level, since as the effective stress increases with depth, the peak friction angle reduces, thus giving a lower cone resistance. Other possible explanations for the reduction of cone resistance could be due to release of side friction on the shaft or the generation of large positive pore water pressure, i.e. at the high pressure at the tip the sand suffers significant volume reduction. The subsequent increase in cone resistance is considered to be caused by proximity of the bottom of the tub.

Cone penetration tests SQ22 and SQ23 performed at 40g were carried out at different positions on the same sand specimen. The cone resistance depth profiles are almost identical indicating that for the specimen the void ratio was relatively uniform. Cone penetration tests SQ18 and SQ19 performed at 28.6g were also carried out at different positions, again on the same sand sample and the significantly different cone resistance depth relationships in this case indicate significantly different void ratio in the two locations.

Since tests were conducted at different g levels, the test data can not be used directly for comparing with each other. Consequently, in order to enable a quantitative comparison to be made, it is appropriate to replot the cone resistance against effective overburden pressures.
Figure 4 shows the variation of cone resistance with effective overburden pressure and indicates that the cone resistance, over the upper region of the specimen, assuming this to be within $q_c = 0-30kPa$ approximately, was in general similar for the tests. The cone tests at 40g seem to indicate that this sample was less dense than the others. In fact at $q_c = 30kPa$, the cone resistance for these tests was about a factor of two less than the other test results. The reason for the looser state for the sample in tests SQ22 and SQ23, and for it to occur especially at the uppermost layers are not known. A lot of factors could have caused a variation of void ratio, for example, the sample may have been disturbed by the upward flow of water during the vacuum-saturation technique, or even by levelling procedure when levelling sand surface. Other possible factors may be due to shocks and vibrations when placing the specimens on the arm of the centrifuge, so that some sections of the specimens may have become denser, i.e. their void ratios where less than original void ratios.

Based on the cone penetration tests some soil parameters such as friction angle or relative density may be determined. There are numerous theoretical and empirical correlations between measured cone resistance and soil engineering properties, and it may be difficult to confirm which method would be more suitable to interpret test results. Thus, some available methods for evaluating the shear strength of soil from cone penetration resistance measured are chosen for comparison.

As early as 1945 De Beer suggested an empirical correlation between cone resistance and bearing capacity factor $N_q$, i.e.

$$q_c/\gamma D = 1.3 N_q$$

(1)

Then the friction angle of soil can be determined from a theoretical relationship of $N_q$ and $\phi$, as indicated in equation (2) as long as the cone resistance is known. This method is still widely used, although it is thought that it will give conservative value of $\phi$ (Sangerat, 1972).

However, the results obtained from the centrifuge tests indicate higher $\phi$:

$$N_q = \tan^2 (\pi/4 + \phi/2) e^{\pi/2 - \phi}$$

(2)

Another method proposed by Meyerhof (1976) is based on the in situ tests, and a semi-empirical
6. SOME FACTORS AFFECTING THE TEST RESULTS

(a) Scale effect

As the same size penetrometer probe was used in the centrifuge test at different g levels, scale effect may be significant. Tests SQ19 and 20 and 22 were conducted at a nominal acceleration of 28.6, 20 and 40g respectively, corresponding to 280mm, 200mm and 400mm in the prototype scale. In all tests undertaken it was found that the smaller the prototype size of the probe the higher the cone resistance. Such scale effect was also observed in relation to the bearing capacity problem.

(b) Stopping of the centrifuge effect

Two cone penetration tests were performed at different locations on the same sample apart from test SQ20, hence the centrifuge needed to be stopped to reposition the cone penetrometer. This procedure led to unloading - reloading stress cycle on the sand although other test conditions were not changed. Cone penetration test SQ23 was performed after test SQ22. The quite close agreement between these two tests as shown in Figure 3, indicating that the effect of stopping of the centrifuge may be insignificant for the
2.28

Fontainbleau sand being tested (relative density about 0.7). Thus, it may be argued that the effect of stress cycling due to stoppage of the centrifuge is negligible for 'dense' soil samples.

(c) Boundary effects

The tub container used in the centrifuge test is rigid-walled. Phillips et al. (1987) carried out a series of centrifuge tests to investigate the side effect of the tub container and found no influence of the side boundary when penetration tests on dense sand were performed at a distance greater than 5 times the diameter of the probe from the side wall.

The cone penetrometer used was located at a distance of 220mm from the side wall of the tub container, i.e. 22 times the diameter of the probe. Thus, it may be considered that there was no significant effect of the side boundary on the cone resistance.

However, the bottom of the tub had some effect on the cone resistance. When the cone approached the bottom of the tub, the deformation of the soil near the cone was restrained and the cone resistance then increased rapidly. This phenomenon was observed for all the tests. The region of influence of the bottom boundary was within a distance of 15 times the diameter of the probe from the bottom of the tub.

7. CONCLUSIONS

Cone penetration tests provide not only a continuous profile of soil resistance but also a determination of shear strength of soil, hence it is important for interpreting test results obtained from different samples. Some empirical correlation was used for deducing the shear strength according to cone resistance. It was found that the friction angles of soil determined from Meyerhof's method were in reasonable agreement with those from triaxial tests.

Some factors which affect cone resistance were discussed. However, from the present tests these factors have not been determined quantitatively, since there is a lack of sufficient test data. Therefore it appears that further studies on cone penetration tests conducted in the centrifuge, are necessary.

ACKNOWLEDGEMENTS

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REFERENCES


CPT - Contraption for Probing in Tills?

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SYNOPSIS: The very complex till deposits encountered in connection with the establishment of the Storebælt Link have triggered an extensive application of CPT testing. The clay tills are inherently heterogeneous with a high content of very coarse particles up to boulder size. Due to varying degrees of overconsolidation they, furthermore, exhibit highly varying undrained strength in the range from 50 kPa to well over 1000 kPa. Despite these seemingly insurmountable difficulties CPT testing proved feasible. It allowed a much more detailed and varied picture of the stratigraphy and the geotechnical parameters compared with traditional field vane testing.

The vast number of CPT tests and adjacent borings with field vane tests allowed a comprehensive calibration of the cone factor, Nc, and evaluation of the influences of overburden, pore pressure and carbonate content.

In the paper the advantages and shortcomings of using offshore CPT testing to provide the geotechnical design basis for the foundation of the structural elements of the Storebælt Link bridges are described.

1. INTRODUCTION
The Storebælt Link project in Denmark has paved the way for several innovations and introduction of practices hitherto neglected or kept out by the force of tradition. As an example the acceptance of CPT testing in Denmark is closely linked to the project.

Due to the geological setting of Denmark, there is an abundance of stiff and hard soil types in marked contrast to the peats and soft clays of the Netherlands and other countries associated with extensive CPT testing. This certainly applies to the Storebælt area, where glacial tills, Selandian marl and Danian limestone (from the top down) are the main soil types in question. In a way both the owner (A/S Storebæltssforbindelsen) and the contractors were brave to instigate the more than 4000 m of CPT testing in soils with undrained shear strength typically from 250 kPa to well over 1000 kPa. In the early days a number of cones were lost due to teething problems but the overall experience and results have been very promising.

The paper will focus on the lessons learned from CPT testing particularly in clay tills for the Storebælt Link illustrated by some case histories.


2. STOREBAELT TILLS IN SHORT
In the Storebælt region the basement strata consist of Maastrichtian chalk (Cretaceous deposits at depths exceeding some 85-100 m below the
sea bed) and a Paleocene sequence consisting of Danian limestone and Selandian marl. These layers are covered by Quaternary deposits comprising glacial, late glacial and Holocene deposits.

The predominant part of the Quaternary is the glacial sequence which lithologically is subdivided into an upper and lower glacial unit of tills and melt water deposits.

The upper unit is dominated by a single clay till deposit, the Sprogø till, with a maximum thickness of 25 m. For geotechnical reasons it was subdivided in Clay Tills 0, 0/1 and 1 according to limits of undrained shear strength of 100, 200 and 400 kPa, respectively.

Three till types (Till 2, 3 and 4) with associated melt water deposits may be distinguished in the lower unit. The strength of the lower tills is high, typically $c_u > 400$ kPa measured by field vane or CPT.

Thus, the glacial sequence is dominated by clay tills of highly varying strength with significant seams, lenses and layers of sand and gravel or sand till.

From the envelope of grain size curves for the clay tills shown in Figure 1 it is apparent that these deposits are very heterogeneous with a high content of coarse particles. Boulders occur frequently, more than 1 in 100 m³, but it was established that the frequency of boulders decreases with a factor ten for an increase in boulder diameter by 0.5 m (Dittevæn, 1989). This is of course not reflected in Figure 1 which is based on borehole samples.

According to the Danish soil classification system (Larsen et al, 1995) clay tills have a clay content not less than 12% and a plasticity index, $I_p > 4%$. For the Storebælt region average values of clay content = 15-17%, $I_p = 6-7$% and natural water content $w = 10-12$% are found. In general the carbonate content ($CaCO_3$) is 17-22% but in the lower till unit deposits characterised as Till, very calcareous ($CaCO_3$ content in the range 35-70%) were identified. By triaxial testing it was established, however, that the undrained behaviour is similar to that of clay tills.

![Figure 1: Grain size of clay tills at Storebælt](image)

**Fig.1 Grain size of clay tills at Storebælt**

### 3. STRENGTH TESTING OF THE TILLS

Soil investigation campaigns of 1962-63, 1977-78 and 1983 - preceding the final campaign for the Storebælt Link presently being finalised, starting in 1986 - were based on geotechnical borings, laboratory testing and seismic profiling.

The strength of the clay tills was inferred from field vane tests (FVT) - based on “well-windowed experience” of several decades in Denmark using the Danish deep vane system and it was tacitly assumed that the standard interpretation was valid for the tills of Storebælt:

$$c_u = c_u \text{, nom}$$

(1)

However, due to the high cost of drilling offshore geotechnical boreholes the client decided to add CPT testing when the detailed investigations for the West bridge were initiated at an average ratio of 8 CPTs to 2 boreholes for each of the 62 offshore piers.

The CPT tests were performed by Fugro-McClelland with additional tests carried out by the Danish Geotechnical Institute. At an early stage (DGI POS 68, 1989) a rough calibration of CPT and VST, based on 170 m of penetrated Clay Till 1, indicated a cone factor of 10.

Taking more than 500 data points of neighboring CPT and FVT (mathematically smoothed CPT profiles with a 0.21 m bandwidth average value at the level of VST value without refusal or stones) cone factors without and with correction for overburden stress $\sigma_v$,

$$N_c = \frac{c_u}{c_u \text{, nom}} = 10$$

(2)
were confirmed to represent equally best fits for the West Bridge data (Mortensen et al, 1991).

Both the $c_{u, \text{vane}}$ and $q_c$ values were log normally distributed, but with the coefficient of variations of 0.36 and 0.20, respectively, the CPT test is the better predictor.

Subsequent, extensive CPT testing (some 400 CPTs to app. 20 m depth) by the Danish Geotechnical Institute for the East Bridge, including CPT tests close to some of the borings from the 1962-1963 campaign confirmed the general validity of (2) as a very robust cone factor. However, for Clay Tilt 0 ($c_{u, \text{vane}} < 100$ kPa) Eq (3) with $q_c$ substituted by $q_T$, i.e. with correction for pore pressure, should be applied to obtain relevant strength values.

4. CPT versus VST

The field vane profiles very clearly indicate the problems associated with in situ strength testing of the tills. On numerous occasions refusal or influence from "stones" are reported in the field journals. This results in jagged undrained strength profiles where any naturally occurring trending with depth may vanish.

It is therefore little wonder that the "foreigners" on the Storebælt project were dubious about the ability by the Danes to assess the undrained strength. This is moreover amplified by a general feeling of inadequacy of VST data for overconsolidated clays as reported by Young et al (1988).

In response to a questionnaire concerning VST testing for design purposes only 6% of the respondents indicated that VST would be useful if OCR>4. Indeed only a minority felt that VST could be performed in soils stronger than 225 kPa and 60% believed the practical limit of the test to be 150 to 200 kPa!

Thus considering the typical strength distribution found for the "weaker" Upper Till in the Storebælt region (Figure 2) conditions would indeed appear dire. Note, that the relatively high number of values $325 \text{ kPa} < c_{u, \text{vane}} < 350 \text{ kPa}$ and $700 \text{ kPa} < c_{u, \text{vane}} < 725 \text{ kPa}$ corresponds to refusal for the two types of vanes applied (at 365 and 714 kPa, respectively).

![Image](image_url)

**Fig. 2 Strength distribution in Upper Till**

The results in Figure 2 are drawn directly from the Storebælt GEOMODEL data base (for a description see Porsvig and Christensen, 1994).

The occurrence of the stones/boulders and the highly varying strength naturally affects the CPT profile as well. However, due to the continuous registration it is possible to reduce the spiky appearance of the profiles by mathematical smoothing. Thus, if the coarse particles due not break the cone it is still possible to provide meaningful data. The stones in the clay till has by far the biggest impact on the sleeve friction and the pore pressure measurements. Omission (by mistake) of a stop criterion for the sleeve friction in the Project Quality Plan led to a relatively high loss of cones until proper stop criteria were established.

As a consequence and drawing on the experience gained at Storebælt in general the Danish Geotechnical Institute has introduced the following stop criteria for its offshore dedicated CPT rig SCOPE (used for more than 2000 m of CPT testing on Storebælt) in order to accept the risk of losing cones due to soil conditions:

- maximum thrust: 200 kN
- maximum tip resistance: 80 MPa
- maximum sleeve friction: 0.53 MPa
- maximum sudden increase in inclination with high tip resistance: 3°
- maximum inclination with vertical: 20°
These stop criteria have very significantly reduced the loss of cones, and importantly have ensured that data are not invalidated due to damaged but not malfunctioning probes. This is particularly important for offshore applications where cost-benefit considerations require that the rig stays submerged during location changes, f. inst. within a pier position.

During penetration in the tills the high content of coarse materials has the consequence that pore pressure measurements are dubious. Upon contact with a stone the pore pressure drops almost instantaneously to zero or even negative values. Only in the softest tills $c_s < 100$ kPa or notably at the transition between Upper and Lower till formations were high pore pressures recorded.

The latter highlights one of the shortcomings of alone-standing CPTs. The geologically interpreted boundary between Upper and Lower Till Units is almost unequivocally identified by the CPT tests as a sudden significant increase in tip resistance, $q_t$, and sleeve friction, $f_s$. However, for a considerable number of CPTs a decrease in $q_t$, accompanied by significant pore pressure increases were seen at the approach to the Lower Till Unit. An interpretation in terms of undrained shear strength may therefore be misleading and the lower tip resistance may be a bogus measurement. The reduction in resistance may instead be a tell tale of presence of silt, lime or sand and the “dynamic” effect of the testing method in these materials better represented by drained strength parameters.

5. CASE HISTORIES

5.1 Strength model, East Bridge

Being the last of the three main components to be constructed, the East Bridge designers could draw upon the experience and data gained from the West Bridge and the East Tunnel.

Hence the dedicated CPT test, vane tests and laboratory tests, notably triaxial compression tests, were cross calibrated very carefully to furnish a strength model for the intact clay till (Upper Till) to be used in the design basis.

To limit the number of independent variables it was decided to accept a cone factor of $N_\text{ct} = 10$. The calibration resulted in (Sørensen et al, 1995)

$$ c_{v,\text{new}} = 0.42\sigma_{v,\text{new}} \left(\sigma_{v,\text{new}} / \sigma_v\right)^{0.85} $$

$$ c_{s,\text{new}} = 0.88c_{v,\text{new}} $$

$$ t_{s,\text{new}} = q_s / 10 $$

In (4a) the overall factor 0.4 has been replaced by the specific value of 0.42.

It is apparent that (4b) differs from (1). This is a result of the choice of cone factor $N_\text{ct} = 10$ in (4c).

However, if the proposal by Luke (1994),

$$ c_{v,\text{new}} = \frac{q_t - \sigma_v}{10} $$

$$ c_{s,\text{new}} = \frac{q_t - \sigma_v}{10 \left[ f_s \% \right]^{0.3}} $$

$$ t_{s,\text{new}} = 0.67 \left[ f_s \% \right]^{0.9} $$

where $f_s$ is the friction ratio, is adopted this may explain the deviation. Inserting the typical value of $f_s = 2 \%$ (5c) gives a ratio of 0.88, i.e. as in (4b), whereas $f_s = 3 \%$ gives a ratio of 1.03 corresponding to (1). Hence a cone factor of $N_\text{ct} = 11.4$ ($f_s = 2 \%$) in (4c) would change (4b) into (1).

This clearly demonstrates that correlations are no better than the data base and that they should be used with very open eyes particularly as most correlation systems work as a Pandora’s box.

During the calibration, attempts were made to correlate the cone factor with the activity $A$ (ratio of % clay content and $f_s$). However, since the activity is mainly an indicator of clay mineralogy, with no apparent physical link to the cone factor, and values of $A = 0.2$ - 0.6 are found rather irregularly for the clay till samples, this should be done with caution.

5.2 Strength anomaly - lime content

During foundation inspection for the West Bridge piers supplementary soil investigations revealed areas with low values of CPT and vane resistances in a layer of the Lower Till Unit. Borings revealed low activity ($A = 0.2$ com-
pared with average of \( \approx 0.4 \) due to high CaCO\(_3\) content. The soil was therefore termed Till, very calcareous but the grain size curve corresponds to clay till.

The same type of soil, albeit with lower CaCO\(_3\) content and higher activity, had been found about 25 years before at an onshore site some 50 km from Storebelt. CPT testing close to the original borings were carried out to provide a comparison and update of the cone factor \( N_k \) for this soil type (DGI POS 162, 1992). The trends of CPT and vane strength values from the offshore piers and the onshore site were similar and unequivocally indicated the use of a cone factor \( N_k = 8 \) instead of the “universal” value of 10, \( F_{QU}(1) \).

The vane tests at the onshore site were strongly affected by stones, whereas the CPT profiles, when aggregated, showed a very clear trend with spikes (stones?) at exactly the same depths. Note, that the friction ratios \( f_p \) were 2-5% as opposed to the values of 1.5-2% typical of the “normal” Storebelt clay till. According to (5) this might indicate a reduction in cone factor.

The effect of the high CaCO\(_3\) content seems to be added structure - and accompanying strength - of the till with a ratio of \( c_u/\sigma' = 0.48-0.60 \) instead of the value 0.40 generally accepted for the West Bridge tests.

![Diagram](image)

Fig. 3 ACU\(_{0-0}\) triaxial test on Till, very calcareous (deviator stress \( q \) versus mean stress \( p' \) and axial strain \( \varepsilon_1 \))

The anisotropically consolidated, undrained triaxial test of Figure 3 shows a pronounced peak strength followed by strain softening and subsequent dilational behaviour at increasing strain level.

5.3 Strength anomaly - soft clay till

In general the designers gained high confidence in the use of undrained shear strength inferred from the CPT tests in the Upper Clay Till.

However, for some of the East Bridge piers, notably Pier EB04, low strength areas were studied by laboratory testing too as values of vane strength down to 30 kPa (single value) were recorded. The lowest strengths were found to correspond to water contents higher than average and using full correction for pore pressure the CPT induced strengths were only slightly higher than the vane strengths.

Using (4a), OCR values close to unity could not be ruled out for some of the specimens. This was in contrast to previous findings for Clay Till \( 0/1-1 \), where all specimens were found to be strongly overconsolidated (OCR from 5 to 20).

The conclusions from the results of four triaxial tests of low-strength clay till were

- presence of strong stress path dependant strength anisotropy
- transition to behaviour as remoulded clay till when the pre-consolidation pressure was exceeded
- a reduced strength ratio \( c_u/\sigma' = 0.32 \) (compared with the 0.42 of (4a)) was recommended for design purposes.

The CPT testing was not invalidated, but again caution is recommended in application of the promising "ubiquitous" extrapolation tools.

6. LESSONS LEARNED

The extensive CPT-testing (more than 4000 m penetration) applied for the Storebelt Link Project has facilitated a very thorough calibration with the Danish “well-winnowed experience” of vane shear strength measurements in clay till. An overall robust cone factor of \( N_k = 10 \) proved applicable, but adjustments were called for in low strength areas (where use of
N_{20}, i.e. with correction for both overburden and pore pressure, is essential or in areas with lithology differing from the average (f. inst. high CaCO₃ content).

Despite the very heterogeneous nature of the tills with a high content of coarse particles and highly varying undrained shear strength (from below 50 to well over 1000 kPa) penetration with standard cones (1000 mm²) proved very feasible. In general they were found to be better than enlarged cones.

Due to the reduced cost of CPT, compared with offshore borings, CPT testing was found to be a valuable tool in verification of seismically determined soil boundaries and delineation of areas with anomalous soil properties.

To gain full use of the CPT results it is essential that the CPT-profiles are mathematically smoothed to reduce the inherent spiky appearance dictated by the coarse particles. Moreover aggregated profiles of neighbouring CPTs, f. inst. within a pier, was a valuable tool in assessing anomalous strength behaviour and delineation of boundaries horizontally.

However, the overriding message from the CPT experience from Storebælt is the need for integration of testing tools (seismic profiling, borings, in situ tests and laboratory tests). Application of one tool alone may seriously limit the possibility of rendering "a complete picture" for the purpose of facilitating a safe and economical design.

7. ACKNOWLEDGEMENTS
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8. REFERENCES
Relations between cone resistance and mechanical properties of the loess soils

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A dozen of cone penetration tests have been made on location Gornji Grad in Zemun, Serbia, using the 200kN of pushing force. Investigation soil borings have been made on the very same location. Undisturbed soil samples have been recovered from these borings. These samples have been taken from several depths. Investigation pits have been made on the ground surface. Undisturbed samples have been taken from these pits, too. Laboratory tests have been made on these samples. Laboratory testing included: identification tests, oedometer's tests, tests of uniaxial compression, direct shear tests and triaxial tests. The obtained results are presented in systematically form as well as cone resistance. It can be observed that there exists a correlation between cone resistance and some of loess's mechanical properties such as unit dry weight, uniaxial strength of loess and degree of saturation. The unit dry weight of loess soil can be used as indicator of loess' collapsibility. The observed correlation led us to conclusion that it is possible to determine the potential of collapsibility of loess and bearing capacity of foundation on this soil, based on the cone penetration tests alone.

1. INTRODUCTION

Loess deposits cover large areas of Eastern Europe, Asia (China), Russia, and central parts of Northern America. It is generally accepted that the origin of these deposits dates back to postglacial periods of Earth history. At these times strong winds blew the particles off the areas that were covered with glaciers; when these winds got weaker the deposition of these particles occurred, the result of this process is the formation of loess deposits. Granular composition of the loess may vary from place to place but they often fall into the group of low plasticity clays (CL), with no more than 15% of clay particles. One of the structural characteristic that can frequently be encountered in loess soil is the vertical macroporosity- the pores in this soil, extending mainly in vertical direction, may be as large as few centimeters in diameter. This feature often causes low unit dry weights of this soil, 11-12 kN/m³. The structure of this soil is stable as long as the water content of this soil is unchanged. However, when the water content of the loess soil with low unit dry weight is increased, sudden change of soil structure happens, resulting in large volume changes.

This feature is known as collapse, the loess that can experience this feature is known as collapsible loess. This behavior is usually disastrous for engineering objects built on these soils: the 15-stories high building's settlement larger than 90cm is experienced in Belgrade in 1975 after the breakage of water supplying pipe, Milovic, (1987); few dozens of private houses have literally sunk into the soil in the area of Zemun after the breakage of water conduits and heavy rainfalls. Collapsibility of loess soils is encountered in other parts of the world, too.

2. PREVIOUS WORK

The collapsibility of the loess is usually determined on the basis of laboratory investigation. The most practiced routine for determination of collapsibility is done in oedometer; in this procedure a saturation of a sample is induced at a prescribed vertical stress. The settlement of the loess sample due to wetting only is measured. The final result of the measurements is expressed as the coefficient of specific settlement, iₜₒ, that is given as
\[ i_n = \frac{\Delta e}{1 + e_0} \]  

(1)

where \( \Delta e \) is the change of coefficient of porosity due to wetting and subsequent saturation, \( e_0 \) is the coefficient of the porosity corresponding to the vertical stress acting before and during the event of the saturation. These measurements can also be obtained on two different samples of the same loess soil. one test is done on a sample with the natural water content while the other is done on the sample that is saturated. This version of the test is known as double consolidation test.

The mentioned procedure has several disadvantages: it requires high quality of soil samples (which is not an easy task in sensitive structure of loess soil); it fails to give the measure of the collapsibility in case of increased water content of soil without saturation.

Several investigators have addressed these problems, Holtz & Gibbs (1951), Milovic (1987). Their results clearly show that the mechanical properties of loess soil as measured in laboratory are functions of unit dry weight and water content. However, these results show that the basic disadvantage regarding the high quality of loess sample has not been overcome.

Therefore, some other kind of loess soil investigation, based on in situ measurements, is required. It is proposed in this article that cone resistance, measured during the cone penetration testing in loess soil, can be used as a sound base for the determination of the most important properties of loess soil (such as unit dry weight and water content).

3. CONE RESISTANCE AND LOESS SOIL PROPERTIES

Using the basic concepts from soil mechanics theory (e.g. Lambe & Whitman, 1969) it can be shown that the coefficient of loess soil porosity is given by the following expression:

\[ e = \frac{\gamma_s - 1}{\gamma_d - 1} \]  

(2)

where \( e \) is the coefficient of porosity, \( \gamma_s \) (kN/m²) is the specific unit weight of loess soil and \( \gamma_d \) (kN/m³) is the unit dry weight. Although it is reported that \( \gamma_s \) for loess soil ranges from 26.6 to 27.5 kN/m³ it is assumed that its value is constant and is equal to 27 kN/m³.

Maximal moisture of the loess soil with a given unit dry weight (i.e. maximal water content of the loess soil assuming it had the same unit dry weight during the process of saturation) is given by the expression:

\[ \max w = \gamma_s \times \left( \frac{1 - e}{\gamma_d - \gamma_s} \right) \]  

(3)

However, during the process of saturation, dry loess soil changes its structure (i.e. it collapses and changes its unit dry weight as the same amount of the solid particles is contained in decreased volume of the soil) and the water content at saturation point does not correspond to the one obtained from expression (3). Unit dry weight at the point of saturation can be determined only from laboratory measurements (however, these measurements do not require as intact samples as those from double consolidation test).

For these reasons it is better to establish a degree of saturation as one of the governing state parameters in loess soil. Knowing the water content of the soil, and the unit dry weight degree of saturation, \( S_n \) is defined as:

\[ S_n = \frac{w}{\gamma_s \times \left( \frac{1 - e}{\gamma_d - \gamma_s} \right)} \]  

(4)

Cone resistance, measured during the cone penetration testing, seems to be the function of the degree of saturation. This feature has been observed by the other investigators (e.g. Milovic, 1987), however they have failed to quantify it.

This can be observed from the cone resistance measured during CPT on two locations in Gornji Zemun, Belgrade. Results of the cone resistance measurements are given...
in tabular form. Cone resistances in the first column correspond to the CPT done on the loess soil with natural water content of about 20% in both cases. Typical unit dry weight of this soil is about 12.5 kN/m$^3$. Thus, the natural degree of saturation of the loess soil determined from the samples taken from investigation pits is about 0.47 (0.5). Cone resistances in the second column correspond to the CPT done on the loess soil after the event of wetting.

Table 1. Cone resistances, loc 1

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<th>qc/pa after wet</th>
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Table 2. Cone resistances, loc 2

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This layer of clay is usually due to sedimentation break which has occurred sometime in geological past. This layer is found at a depth of about 7.00m below the ground for this location.

The investigated loess soil was the subsoil for the two multi-stories building. The collapse of the loess soil has happened under both of these buildings. Settlements, due to wetting of the loess soil, larger than 70 cm have been measured on these locations.

4. ANALYSIS OF THE RESULTS AND PROPOSED CORRELATION

From the results presented in the previous chapter it can be concluded that for the same loess soil the cone resistance is the decreasing function of the degree of saturation, $S$, (this is in accordance with Dudley, 1970). In this article it is assumed that the cone resistance is the linear decreasing function of the $S$, i.e.

$$ q_c = a + b \times (1 - S) $$

As the degree of saturation, $S$, ranges from 0 to 1 (in loess soil it typically ranges from 0.3 to 1) it can be concluded from expression (5) that factor $b$ gives the increase of cone resistance, qc (kPa), due to decrease of $S$, only. Factor $a$ is the normalized cone resistance that corresponds to the fully saturated loess soil (it can be considered as "residual" cone resistance). Atmospheric pressure $p_a$ (100 kPa) is the normalizing quantity.

The expression (5) is applied to the results given in the tables 1 and 2. The average ratio of factors $a$ and $b$ is very close to 1 (values 1.22 and 0.94 have been obtained). For the practical purposes it is assumed that $a$ equals $b$. Therefore, the full saturation of the loess soil has occurred in the places where the cone resistance before wetting is more than 1.5 larger than cone resistance after wetting.

The water content of the loess soil at saturation point, on the location Gornji Zemun,
rarely exceeds 30%. Using the expression (4), unit dry weight corresponding to the saturation point can be determined. It can be deduced that the value of this quantity is 14.8 kN/m³.

Next, it is assumed in this procedure that soil settles in the conditions of confined compression. The vertical deformation caused by the soil settlement due to saturation can be determined then on the basis of the known volume of the loess soil before wetting and after the event of wetting, i.e.:

\[ \epsilon_s = \frac{V_0 - V_f}{V_0} \]

This quantity is analogous to the quantity \( \epsilon_{\sigma_1} \) from equation (1). For this case its value is equal to 0.16. It is interesting to note that, according to Milovic, (1987), values of \( \epsilon_{\sigma_1} \) ranging 0.03-0.035 at \( \sigma_1=100 \) kPa have been measured in laboratory conditions.

The zone of full saturation of loess soil as determined from the CPT, is about 6m high in the first case and about 5m high in the second one, using (6), this results in settlements equal to 95cm in the first case (measured settlement due to wetting only equals 80cm), and 79.3cm in the second one (measured settlement due to wetting only equals 70cm).

Discrepancies between the measured settlements and those that were predicted on the basis of CPT may be caused by several reasons:
- initial unit dry weight is not homogeneously distributed over the layer of loess soil;
- final unit dry weight is not homogeneously distributed over the layer of loess soil;
- during the occurrence of the settlement, loess soil is not completely in the conditions of confined compression.

5 CONCLUSION

In this article, use of cone penetration testing in determination of loess soil properties is proposed. The main advantage of this procedure is that high quality samples of loess soil are not required. A correlation between the degree of saturation of soil and cone resistance in collapsible loess is proposed. Based on this correlation degree of saturation of the loess soil is determined. Knowing the unit dry weight of the collapsible loess soil at the point of saturation, it is possible to estimate the settlement due to wetting only.

The procedure is based on the correlation given in the expression (5). Further investigations are to be made about the improved formulation of this correlation. However, the basic statement in this article is that the degree of saturation is the parameter of state which governs the mechanical behaviour of loess soil during the cone penetration testing.

It is the opinion of the authors that one more variable, regarding the unit dry weight of the, has to be introduced if the procedure is to be generalized for the loess soil.

6 REFERENCES

CPT-investigations in young sediments in the northern part of Jutland.

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SYNOPSIS: This paper presents results of analyses of CPTs carried out at the sea shore on the west coast of Northern Jutland, Norre Lyngby, and in the town of Skagen. The results of the CPTs are analysed on the basis of values for the cone factors for Danish soils as recommended by Kirsten Luke 1994. At Norre Lyngby the results of the CPTs are correlated with the results of vane tests. The results from Norre Lyngby together with the results from vane tests in a boring at the very top of Denmark are used to estimate the kind of sediments from 40 to 80 m below the surface in the town of Skagen.

The CPTs have been carried out in connection with investigations in progress to explain some geophysical problems at Norre Lyngby, and to explain the geological structure and the land movements in the area around Skagen.

1. INTRODUCTION
The use of CPT to classify Danish soils has been investigated with special emphasis on measuring the undrained shear strength (Luke, 1994). This investigation has been concentrated on examination of soil at shallow depths e.g. less than 20 m below surface.

In 1993 Kampias Geodon carried out a series of CPT’s in Late Glacial clay at the sea shore on the west coast of Northern Jutland, Norre Lyngby (fig. 1). Vane tests were carried out in borings at a depth of 2 m below surface, and the variation of the cone factors with the friction ratio was determined. (Luke, 1994).

Close to two existing borings taken to a depth of 40 m and 70 m respectively, CPTs were taken to 11 m and 24 m respectively.

In January 1995 Kampias Geodon carried out a CPT from 40 m to 80 m below surface in the town of Skagen (fig. 1). A boring was taken to a depth of 40 m, and the CPT was carried out from the bottom of the boring. The purpose was to investigate if the sediments below the town of Skagen were similar to the sediments found in a boring at the very top of Denmark, the Grenen Point.

Fig. 1. The position of Norre Lyngby and Skagen.

2. CPT-EQUIPMENT
The CPT-equipment used in the actual investigations is the van den Berg system developed in Holland. The cone used is a piezo-cone. The penetration resistance on the tip (cone resistance), the side friction resistance on a friction sleeve (sleeve friction) and in addition also the pore water pressure, present
just above the cone shoulder are measured during the penetration. The friction ratio, e. g. the ratio of sleeve friction to cone resistance is calculated.

Fig. 2. The CPT-rig at the sea shore on the west coast of Northern Jutland, Norre Lyngby.

3. SYMBOLS
List of symbols used in this paper.

- \( c_u \) = undrained shear strength
- \( c_v \) = vane shear strength
- \( c_m \) = remoulded vane shear strength
- \( C_s \) = index of compressibility
- \( e_0 \) = initial void ratio
- \( f_s \) = friction ratio \( f_s/c_v \)
- \( f_r \) = measured sleeve friction
- \( I_p \) = plasticity index
- \( n_k \) = cone factor = \( q_k/c_v \)
- \( n_h \) = cone factor = \( q_h/c_v \)
- \( N_k \) = cone factor = \( (q_k - \sigma_{v0})/c_v \)
- \( N_h \) = cone factor = \( (q_h - \sigma_{v0})/c_v \)
- \( n^{+} \) = measured cone resistance
- \( n^{\ast} \) = cone resistance, corrected for pore pressure effects. For the actual piezo-cone \( n = q_{\ast} \pm 0.3 u \)
- \( Q \) = \( C_s/(1+e_0) \)
- \( S_s \) = sensitivity = \( c_v/c_m \)
- \( u \) = hydrostatic pore water pressure
- \( u_r \) = measured total pore water pressure
- \( w \) = water content
- \( y \) = total unit weight
- \( \sigma_{v0} \) = total initial vertical stress
- \( \sigma^\prime \) = effective initial vertical stress

4. NORRE LYNGBY
The area has been the subject of several geological and geophysical investigations through the years (Jessen, 1931; Knudsen, 1978; Lykke-Andersen, 1992). Seismic investigations have shown some remarkable spatial variations in the frequency spectra of reflected signals.

The actual investigation is a part of an EU geological research project supported by the research program MAST 2. The purpose is to find a correlation between parameters for soil determined in the geotechnical laboratory, results from geotechnical field investigations and seismic results from the field.

The work are carried out as a collaboration between Aarhus University and Aalborg University.

4.1 Existing results from CPTs
Along a seismic investigation line 10 CPTs were carried out, and the results were correlated to results from vane tests from 2 m's depth along the same line (Luke, 1994). The results of this correlation are shown in fig. 3.

![Fig. 3. N_k and n_k plotted as a function of f_s for the clay in Norre Lyngby (Luke, 1994).](image)

The results from Norre Lyngby shows, that the cone factor decreases with increasing friction ratio. This is supported of results from other Danish areas. Average values of \( n_k \) and \( N_k \) for the investigated Danish soils to depth's of about 20 m are listed in fig. 4 (Luke, 1994).
4.2 Correlation between CPTs and results from the deep borings.

In connection with the geological research project two borings were carried out in areas with very different acoustic properties. Boring A was taken to 70 m and boring B to 40 m below the surface.

The borings were carried out as rotary borings with continuous coring. Undisturbed 70 mm diameter samples were extracted and vane tests carried out at 6 meters interval in clay deposits.

The samples from the borings were used for geotechnical laboratory testing and in fig. 5 some of the results from the upper 25 m of the borings are shown.

CPTs were carried out to 24 m's depth close to boring A and to 11 m's depth close to boring B. Boring A has shown Late Glacial silty clay to more than 24 m's depth. In boring B silty clay was found to a depth of 9.5 m, underlain by sandy and stony deposits, all supposed to be of Middle Weichselian age.

The results of the vane tests together with the results of the CPT tests, $f_v$, $u_r$, and 0.1$q_r$ (corresponding to $n_v=10$), are shown in fig. 6 and fig. 7.

$N_u = 15(f_v \alpha)$
On the basis of the results from the CPTs the cone factors have been calculated. The results are shown in fig. 8. The two values above 20 for some of the cone factors are from the very silty top soil in boring A.

$$c_v = \frac{(q - \sigma_z)}{10}, \quad (1)$$

and the estimated values for

$$c_v = \frac{(q - \sigma_z)}{15(f_B^{44})}, \quad (2)$$

are shown together with the results of the vane tests in boring A. Except for the silty top soil, the estimated values for the undrained shear strength and the vane strength are very close to or below the measured values.

5. SKAGEN

At the very top of Denmark, the Skaw Spit, detailed levellings during the period from 1942 to 1991 have shown that the most northern part of the Skaw Spit has not followed the general isostatic uplift in the northern part of Denmark (Hauersbach, 1992).

The area has been the subject of several geological investigations through the years. A study of the foraminiferal faunas in samples from an older boring in the area (Knudsen, 1985) has indicated an unusual thickness of very young sediments at the Skaw Spit.

Seismic investigations have been carried out around Skaw Spit in an integrated geological research project, the GeoKat project, run at the Department of Earth Sciences, University of Aarhus. The results of these investigations confirmed the thickness of very young sediments (Lykke-Andersen & Knudsen, 1992, Lykke-Andersen et. al. 1993a & b).

5.1 Boring Skagen 3

In connection with the GeoKat project a boring at the outermost part of Skaw Spit, the Grenen Point, was taken to a depth of 220 m below sea level.

The Boring showed at the top 30 m sand, which has been deposited within the latest 1000 years. (Consadres & Nielsen, 1994). Below the sand was found about 85 m of Holocene clayey silt and clay underlain by clay and sand deposits from Weichselian, Eemian, Saalian and Lower Cretaceous (Knudsen, 1994).

The Skagen 3 boring was carried out as a rotary boring with continuous coring from the depth of 30 m. In the Holocene silt and clay layers, undisturbed 70 mm diameter samples were extracted and vane tests carried out at 3 to 6 meters interval. In the clay deposits below level -80 the undrained shear strength measured by the vane represents values of about 0.2 - 0.25 \(\sigma'_w\). The results of the vane tests are shown in fig. 10.

The samples from the borings have been used for laboratory testing (Thorsen, 1995a & b). Some of the results are shown in fig. 11.
5.2 Boring Skagen 5

To investigate if the sediments below the town of Skagen are similar to the sediments at the Grenen Point as indicated by Fredericia, (1988), a boring, the Skagen 5 boring, was taken to a depth of 40 m, and a CPT was carried out from the bottom of the boring and to 80 m below surface.

The boring showed at the top 25 m sand underlain by Holocene silty clay. A few 70 mm diameter samples were extracted from the silty clay and vane tests were carried out in the clay sediments. Some of the results from the laboratory testing are shown in fig. 13. The results of the vane tests together with estimated values for the undrained shear strength from (1) and (2), based on results from the CPT are shown in fig. 12. The results of $f_u$, $u$, and $0.1\lambda$ are shown in fig. 14.

The measured pore water pressure in the CPT shows an excess pore water pressure of about 0.8 MPa, corresponding to the excess pore water pressure measured in the CPTs in the clay sediments in Nørre Lyngby.
The age of a sample from level -35 m in boring Skagen 5 is radiocarbon dated by the Accelerator Mass Spectrometry (AMS) to about 400 BP, an age corresponding to about level -60 m in boring Skagen 3 (Conradsen & Nielsen, 1994).

Comparing the results from the Skagen 5 boring, soil description, the vane strength above level -35 and results of the CPT, $f_v$, $u_r$, with the results from the Skagen 3 boring, it seems reasonable to assume, that the sediments below the town of Skagen are similar to the sediments found at the Grenen Point, and that estimations on the basis of (1) and (2) give a good estimation for the values of the shear strength and a minimum value for the vane strength, respectively.

6. CONCLUSION

The results found by Luke (1994) for estimation of the undrained shear strength from CPT for Danish soils have been used to estimate the kind of sediments from 40 - 80 m's depth in the town of Skagen.

Results from two CPTs to 10 m and 24 m below surface respectively, carried out close to two borings in Nørre Lyngby, have been analysed. The estimated values for the undrained shear strength on the basis of $N_u = 10$, and $N_u = 15(f_v^{0.45})$ as suggested by Luke (1994) are very close to the measured vane strength.

Comparing the results from the boring and CPT, Skagen 5, with the results from the CPT's at Nørre Lyngby and the results from the boring, Skagen 3, at the Grenen Point, it seems reasonable to assume, that the sediments below the town of Skagen are similar to the sediments at the Grenen Point, but also that the estimations on the basis of $N_u = 10$, and $N_u = 15(f_v^{0.45})$ may give good estimation for the values of the shear strength in the actual case.

7. REFERENCES


Correlation of CPT Data with Static and Dynamic Properties of In Situ Frozen Samples

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SYNOPSIS: Correlations between cone penetration resistance and static and dynamic properties of sandy soils are examined, based on high quality undisturbed samples. For this purpose, the cone penetration tests are conducted at eight sites where sands with fines contents up to 30% were sampled by in situ freezing method and their static and dynamic properties have been determined in the laboratory. The comparison of the CPT resistance with the static and dynamic soil properties of the in situ frozen samples shows that: (1) the modified cone penetration resistance corrected for a vertical effective stress of 98 kPa, $q_u$, shows good correlation with the static properties of the in situ frozen samples at large strains such as the modified internal friction angle, $\phi_m$, and the modified dilatancy angle corrected for a confining effective stress of 98 kPa, $\psi$; and (2) the modified cone penetration resistance is well correlated with the dynamic undrained strength such as the liquefaction resistance particularly at small number of cycles.

1. INTRODUCTION
The standard penetration test (SPT) has been widely used to evaluate the liquefaction potential of a sandy soil deposit subjected to earthquake loading. However, because of providing a more consistent evaluation of soil strength and stiffness, the cone penetration test (CPT) has received considerable interest in recent years.

One of the concerns for using the CPT lies in its drainage condition. Namely, the CPT is essentially a drained test for sands in situ, while the liquefaction occurs under undrained conditions. This contrasts well with the SPT test, considered as an undrained test. Thus, legitimate questions have been raised whether the liquefaction resistance is really correlated with the CPT resistance as well as with the SPT resistance.

Recently, CPT tests were performed at several sites where high-quality undisturbed samples had been obtained by the in situ freezing method and their static and dynamic soil properties had been determined. This enables one to examine how the CPT resistance correlates with those soil properties. The object of this paper is to study the correlations between the CPT data and the static and dynamic soil properties of the high-quality undisturbed samples.

2. IN SITU TEST
2.1 CPT Test Sites and Results
The CPT tests were conducted at eight sites; Niigata Station, Showa Bridge and Meike Elementary School, Niigata Prefecture; a man-made fill; Higashi-Ogishima (2 sites), Kanagawa Prefecture; and Urayasu, Kemigawa, Chiba Prefecture. The liquefaction resistance of the in situ frozen samples for the first seven sites has been published elsewhere (Tokinatsu et al., 1995). To increase data base, in situ freezing sampling was made at Kemigawa site. The sands at three sites in Niigata Prefecture and at Urayasu site are fluvial deposits, while the sand at Kemigawa site is diluvial deposits. Two Higashi-Ogishima sites are reclaimed lands and the sand at one of them had been improved by a vibratory sand compaction pile.
The electric CPT tests were conducted at each site with a penetration rate of 2 cm/s, which resulted in continuous records with depth of three components, i.e., cone penetration resistance \( q_c \), sleeve friction \( f_s \), and pore water pressure \( P_w \).

The CPT result for Kewigawa site is shown in Fig. 1, along with the SPT result. The CPT result is presented in the form of distributions of the above three components in addition to the friction ratio \( R = \frac{f_s}{q_c} \).

### 2.2 CPT–SPT Correlation

The variation of SPT \( N \)-value with depth shows a tendency very similar to that of the cone penetration resistance, as shown in Fig. 1. Thus, the correlation between CPT and SPT resistances \( q_c/N_c \) have been computed for all sites, and are shown in Fig. 2. Both resistances, i.e., modified SPT \( N \)-value \( N_1 \) and modified cone penetration resistance \( q_{N_1} \), are corrected to an effective overburden pressure of 98 kPa from the following relationships.

\[
N_1 = N \left( \frac{\sigma'_v}{\sigma'_c} \right)^{0.5}
\]

(1)

\[
q_{N_1} = q_c \left( \frac{\sigma'_v}{\sigma'_c} \right)^{0.5}
\]

(2)

where \( N \) is SPT \( N \)-value measured with an energy efficiency of 78%, \( \sigma'_v \) is an effective overburden pressure at depth of sample in kPa, \( \sigma'_c \) is an effective overburden pressure of 98 kPa, and \( q_c \) is the average of the CPT resistances over a length of 30 cm where the corresponding \( N \)-value was measured.

The \( q_{N_1}/N_1 \) ratio appears to decrease with increasing fines content, which is consistent with the previous studies (e.g., Muramachi et al., 1982). The solid lines shown in the figure are the best fit curves to represent these trends. Unlike previous studies in which the \( q_{N_1}/N_1 \) ratio is only a function of fines content, there is a definite tendency in which the \( q_{N_1}/N_1 \) ratio increases with decreasing \( N_1 \) value for sands with the same fines content. The reason for this may lie in the difference in drainage conditions between the two tests, i.e., the CPT is essentially a drained test for sand, while the SPT is an undrained cyclic test. It is known
that the drained strength vs. undrained strength ratio increases with decreasing soil density and vice versa (Tatsuoka et al., 1982), due to the positive and negative dilatancy characteristics of sands. The trend of $q_u/N_1$ ratio shown in Fig. 2 appears to follow these characteristics, and suggests that, in establishing a CPT-based liquefaction based on the from the SPT-based chart, the use of a constant CPT-SPT resistance ratio may result in overestimation at small $q_u$ and in underestimation at large $q_u$.

2.3 $R_1$ – FC Relationship
Fig. 3 shows the relationship between friction ratio $R_1 (=f_f/q_f)$ and fines content FC of the samples obtained from the SPT (Tokimatsu et al., 1995). There exists a fairly good correlation in which the friction ratio $R_1$ increases with increasing fines content FC. This suggests a possibility that the fines content FC can be approximately estimated from the friction ratio $R_1$. The friction ratios of 0.5% and 1.0% roughly correspond to fines contents of 5%, and 10%, respectively.

3. LABORATORY TEST
3.1 Liquefaction Test
Undrained cyclic shear tests were conducted on the samples taken from Kemigawa at eight different depths. The samples had relative densities of 63.6 – 72.3%, mean grain sizes of 0.15 – 0.18 mm, and fines contents of 3.3 – 12.4%. A cylindrical specimen 50 mm in diameter and 100 mm high was used in the test. After the specimen had thawed completely and had been consolidated isotropically under the vertical effective stress of the sample in situ, it was subjected to sinusoidal axial loading of a constant amplitude $\sigma_1$ at a frequency of 0.1 Hz. The cyclic stress ratio $\sigma_3/2\sigma_1$ required to cause a double amplitude axial strain DA of 5% at $N_v=5, 15$, and 25 cycles are hereby called $R_v$, $R_v^{15}$, and $R_v^{25}$, respectively.

3.2 Consolidated Drained Test
Consolidated drained (CD) tests were conducted on the samples taken from seven sites except for Showa Bridge site. The specimen preparation and consolidation methods are the same as those of the liquefaction test. The test was performed by increasing axial strain at a strain rate of 0.1%/min.

Assuming that the sand has no cohesion, the modified internal friction angle of the test sand is defined as

$$\phi_m = \sin^{-1}\left(\frac{\sigma_{max}}{\sigma_1'}\right)$$

in which $\sigma_{max}$ is the maximum deviator stress during the test and $\sigma_1'$ is the confining pressure. The modified dilatancy angle is defined, by correcting the dilatancy angle $\nu$ in terms of the confining pressure, as

$$\nu = \tan^{-1}\left(\frac{\Delta e_v/\Delta y}{\sigma_1'/\sigma_0}\right)$$

in which $\sigma_0'$ is a confining pressure of 98 kPa, $e_v$ and $y$ are volumetric strain and shear strain at an axial strain of about 5%, and the dilatancy angle is defined as $\nu = \tan^{-1}(\Delta e_v/\Delta y)$.

4. CORRELATION BETWEEN IN SITU AND CD TEST RESULTS
4.1 Internal Friction Angle
Fig. 4 shows the correlation between modified internal friction angle $\phi^m$ and the modified cone penetration resistance $q_c$ in terms of mean grain sizes $D_M$. Both of these values are obtained by static loading tests in the field and laboratory. As expected, there is a fairly well-defined trend in which the modified internal friction angle increases as the modified cone penetration resistance increases. The correla-
tion appears to be independent of mean grain sizes used. Shown in a solid line in the figure is the best fit correlation defined as

\[
\phi_m = 35 + 0.4q_0
\]

4.2 Dilatancy Angle

Fig. 5 shows the correlation between modified dilatancy angle \(\gamma_d\) and modified cone penetration resistance \(q_0\) in terms of mean grain sizes \(D_{90}\). In contrast to \(\phi_m\) that varies within a limited range from 35° - 45°, \(\gamma_d\) ranges over a broader band from almost 0° - 45°. However, there also exists a well-defined trend in which the modified dilatancy angle increases with increasing modified cone penetration resistance, regardless of mean grain sizes \(D_{90}\).

The internal friction angle and dilatancy angle are considered to be the indices representing positive dilatancy characteristics of sands. The good correlations shown in Figs. 4 and 5, therefore, suggest that the cone penetration resistance does reflect the positive dilatancy characteristics.

5. CORRELATION BETWEEN LIQUEFACTION AND CD TEST RESULTS

Figs. 6 to 9 show the correlations between either modified internal friction angle \(\phi_m\) or modified dilatancy angle \(\gamma_d\) and the liquefac-

![Fig. 4](image1)

Fig. 4. Correlation between modified internal friction angle and modified cone penetration resistance

![Fig. 5](image2)

Fig. 5. Correlation between modified dilatancy angle and modified cone penetration resistance

![Fig. 6](image3)

Fig. 6. Correlation between liquefaction resistance \(R_s\) and modified internal friction angle

![Fig. 7](image4)

Fig. 7. Correlation between liquefaction resistance \(R_s\) and modified internal friction angle
Fig. 8. Correlation between liquefaction resistance $R_3$ and modified dilatancy angle

Fig. 9. Correlation between liquefaction resistance $R_{15}$ and modified dilatancy angle

Fig. 10. Correlation between liquefaction resistance $R_3$ and modified cone penetration resistance

Fig. 11. Correlation between liquefaction resistance $R_{15}$ and modified cone penetration resistance

Fig. 12. Correlation between liquefaction resistance $R_{15}$ and modified cone penetration resistance

The liquefaction resistance at 5 or 25 cycles, $R_3$ or $R_{15}$, in terms of fines content FC. Each figure shows that the liquefaction resistance increases with increasing $\phi_0$ or $\nu$, and that the higher the fines content, the higher the liquefaction resistance for the same $\phi_0$ or $\nu$. In these correlations, $\phi_0$ and $\nu$ are more closely correlated with $R_3$ than with $R_{15}$. This indicates that the positive dilatancy characteristics have a considerable effect on the liquefaction resistance particularly at small number of cycles.

6. CORRELATION BETWEEN IN SITU AND LIQUEFACTION TEST RESULTS

Figs. 10 to 12 show the correlations between the modified cone penetration resistances and
the liquefaction resistances at 5, 15, and 25 cycles, \( R_p \), \( R_{15} \), and \( R_{25} \), respectively, in terms of fines content FC. Each figure shows that the liquefaction resistance tends to increase as the \( q_s \) increases. In Figs. 11 and 12, however, the liquefaction resistances, \( R_p \) and \( R_{15} \), appear slightly insensitive to an increase in \( q_s \) if \( q_s \) is less than about 15 MPa, and increase abruptly when \( q_s \) exceeds a critical value of about 17 MPa. In Fig. 10, in contrast, the liquefaction resistance \( R_p \) is sensitive to an increase in \( q_s \) for any \( q_s \) value, and the data points appear to fall within a narrow band compared with those shown in Figs. 13 and 14. The good correlation between \( q_s \) and liquefaction resistance at small number of cycles is attributed to the preceding findings that both of these values are controlled by the positive dilatancy characteristics.

7. CONCLUSIONS

The cone penetration tests were conducted at eight sites where the dynamic and static properties of sandy soils with fines contents up to 30% had already been determined from in situ frozen samples. The comparison of the CPT result with the liquefaction resistance and the CD test results (internal friction angle, dilatancy angle) of the samples leads to the following conclusions:

1. The \( q_w/N_1 \) value of sandy soil with the same fines content increases with decreasing soil density, probably reflecting the difference in drainage conditions between the CPT and SPT tests.

2. There are good correlations between modified cone penetration resistance \( q_m \) and the positive dilatancy characteristics (modified internal friction angle \( \phi_m \), modified dilatancy angle \( \nu_m \)). This is probably because the cone penetration resistance reflects the positive dilatancy characteristics.

3. The positive dilatancy characteristics (modified internal friction angle \( \phi_m \), modified dilatancy angle \( \nu_m \)) are more closely correlated with the liquefaction resistance at small number of cycles \( R_p \) than with that at large number of cycles \( R_{15} \). This is probably because the liquefaction resistance at small number of cycles reflects the positive dilatancy characteristics.

4. The modified cone penetration resistance \( q_m \) is well correlated with the liquefaction resistance at small number of cycles \( R_p \). This is because both of these values are controlled by the positive dilatancy characteristics.

8. REFERENCES


Application of cone penetration test for evaluation of geotechnical parameters of post-flotation sediments.

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SYNOPSIS: The paper presents of methodology for examining geotechnical parameters, and evaluating state of consolidation of post-flotation sediments accumulated in the Zelazny Most tailings dam by means of cone penetration test. The empirical relationships presented facilitate estimation of geotechnical parameters of the sediments necessary for determining embankment stability from CPTU measurements. The correlations presented in this paper take into account the effects of the stress state of the tailings, the degree of consolidation and also drainage conditions. It is also shown that the grain size distribution of the tailings deposit can be determined from CPTU results in a similar way as for natural soils.

1. INTRODUCTION

Annual output of copper ore in Poland amounts to 25 x 10^6 Mg. After mechanical processing 93% of the product is post-flotation waste which since 1977 has been stored exclusively within the single tailings dam - the Zelazny Most Reservoir. Accumulation of the wastes in Zelazny Most by the end of 1994 was 378 x 10^6 Mg which is equivalent to 242 x 10^6 m³ and makes Zelazny Most the greatest tailings dam of this kind in Europe. The scale of the tailings dam can also be illustrated by the following data: tailings dam area 12.3 km²; embankment length 9.4 km; maximum embankment height 40 m; final design capacity 350 x 10^6 m³ with the possibility of further development to the height of 100 m. The applied technology of depositing tailings with hydrotransportation causes separation of sedimentation material silted up. In the tailings area and on beaches kept for safety reasons the sediments corresponding to sandy soils predominate. Further from the embankment the proportion of finer fraction of sediments increases as well as the thickness of laminations built of sediments with characteristics similar to cohesive soils. In the central part of the reservoir the finest particles are sedimented. Such a considerable differentiation of tailings with respect to grain size makes the testing locations very important. In the neighbourhood of the embankments the conditions correspond to full drainage and sediments are slightly overconsolidated while in the pond not full drainage are assumed and their characteristics of sediments corresponds to that of underconsolidated soils. The conditions on the reservoir beaches are more complex and depend on many local factors.

In such complex conditions a good identifier turn out to be penetration characteristics obtained from piezocone test - CPTU. These tests are of particular value when pore pressure dissipation test is carried out during a test. Use of CPT/CPTU for checking geotechnical parameters and grain size distribution of tailings dams have also been reported by others in the geotechnical literature.
2. METHODS FOR IN - SITU TESTING 
OF POST - FLOTATION SEDIMENTS

Testing of sediments with CPTU was carried out in three zones of the reservoir: near embankments, on beaches and in the pond (Fig. 1). Cone penetration tests were made with Hyson penetrometer manufactured by a Dutch - van den Berg company. During penetration three measurements were recorded: cone resistance - qε, sleeve friction - fε, and pore pressure - uε. At selected depths pore pressure dissipation test are carried out. Pore water pressure was measured with a standard piezometer with metal filter placed directly behind cone tip. Special care was exercised in saturation of the pore water system. Maximum penetration depth within the reservoir bowl was 40 m. To identify sediment grain size and to work out a system of classifying sediments on basis of CPTU measurements, undisturbed samples were taken with Mostap-65 sampler (van der Berg). Undrained shear strength of sediments classified as cohesive was determined by field vane tests. PS-1 type vane probe with 80 x 160 mm vane-cross was used.

3. CLASSIFICATION OF SEDIMENTS ON BASIS OF CPTU MEASUREMENTS

Traditional classification systems for CPT developed for natural soils have only limited application for artificially formed soils. Campanella et al. (1983) used Douglas - Olsen diagramme for identifying grain size of wastes from copper ore processing. Larson and Mitchell (1986) for wastes from uranium mine and Jones et al. (1981) for wastes from gold and platinum mine, worked out original classification systems based on: cone resistance and friction ratio (Larson and Mitchell 1986), and cone resistance corrected for value of vertical stress and net pore pressure (Jones et al. 1981).

A similar classification system of post - flotation sediments deposited in the Żelazny Most tailings dam were developed with five groups with respect to their grain size (Table 1). Concerning their physical parameters and grain size the three first groups correspond to non-cohesive soils while the remaining ones correspond to cohesive soils. The method worked out for identification and determination of layers of sediments with similar grain size in the profile uses two classification systems:

![Figure 1. Location plan of the tailing disposal Żelazny Most.](image)

![Figure 2. CPTU location and interpretation procedure for penetration characteristics.](image)
Table 1. The classification of post-flotation sediments into a characteristic soil types

<table>
<thead>
<tr>
<th>Tailing group</th>
<th>Soil type</th>
<th>$\rho_r$</th>
<th>$\gamma_r$</th>
<th>$\gamma_c$</th>
<th>sand content [%]</th>
<th>silt content [%]</th>
<th>clay content [%]</th>
<th>$t_{max}$</th>
<th>$t_l$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>fine sand</td>
<td>0.17</td>
<td>-</td>
<td>-</td>
<td>&gt;90</td>
<td>&lt;10</td>
<td>&lt;10</td>
<td>4/6/1,13</td>
<td>-</td>
</tr>
<tr>
<td>II</td>
<td>silty sand</td>
<td>0.42</td>
<td>-</td>
<td>-</td>
<td>70-90</td>
<td>10-30</td>
<td>&lt;2</td>
<td>0.50/1.29</td>
<td>-</td>
</tr>
<tr>
<td>III</td>
<td>sandy silt</td>
<td>0.71</td>
<td>-</td>
<td>-</td>
<td>50-70</td>
<td>30-50</td>
<td>&lt;2</td>
<td>0.52/1.34</td>
<td>-</td>
</tr>
<tr>
<td>IV</td>
<td>silt</td>
<td>-</td>
<td>0.90</td>
<td>-</td>
<td>&gt;70</td>
<td>&gt;30</td>
<td>2-10</td>
<td>-</td>
<td>33/326/9</td>
</tr>
<tr>
<td>V</td>
<td>silty clay</td>
<td>-</td>
<td>0.45</td>
<td>-</td>
<td>&gt;40</td>
<td>&gt;50</td>
<td>10</td>
<td>36.5/28.2</td>
<td>-</td>
</tr>
</tbody>
</table>

Legend: $\varepsilon_{max}$ = maximum void ratio
$\varepsilon_{min}$ = minimum void ratio
$t_r$ = relative plasticity index
$t_l$ = consistency index from fall-cone test

Basic and ancillary, Harder - Bloh's procedure (Harder and Bloh, 1988) and elements of statistical analysis. The basic classification system enables determination of sediment type from cone resistance and sleeve friction (Fig. 2 and 3). A limitation of this system is unclear division of sediments within two separated cohesive groups due to their low load capacity and slight thickness of layers. Different to non-cohesive sediments, cohesive tailings have high values of pore pressure recorded during piezocone test (Mlynarek et al. 1993). This is expressed in terms of the pore pressure parameter - $B_h = \Delta u/\gamma_t - \gamma_z$. Parameter $B_h$ has been used in the ancillary system for accurately distinguish sediments of groups IV and V (Fig. 3). An additional benefit from using the ancillary system is information about liquid limit of sediment (Mlynarek et al. 1993). Harder - Bloh's procedure is used for statistical computation of penetration curves while sequential test is a criterion for dividing subsoil into homogeneous layers (Tsuchische et al. 1993).

4. GEOTECHNICAL PARAMETERS OF TAILINGS ESTIMATED FROM CPTU

The chosen interpretation procedure of CPTU data in artificially formed sediments must take into account grain size distribution as in case of natural soils. Further strength and compressibility parameters, drainage conditions and degree of overconsolidation should be considered.

Figure 3. Diagramm of classification system of post-flotation sediments (Fig. 3a - basic system; Fig 3b - ancillary system).
To estimate relative density of postmining sandy sediments Kohn (1984) used the diagram for uncedmented fine sands. Much lower estimates of the degree of consolidation are obtained from diagramme worked out by Matyas et al. (1984) for wastes from uranium ore. Jones et al. (1981), to estimate relative density of sediments from gold and platinum mines, use a relationship applied for natural soils according to which relative density is a function of cone resistance and total vertical stress.

According to East and Ulrich's concept (East and Ulrich, 1989) shear strength parameters of postmining sediments belonging to the non-cohesive groups are described by an effective angle of internal friction while the cohesive ones by undrained shear strength. These parameters are functions of cone resistance and vertical stress. According to other concepts, undrained shear strength of sediments is a function of depth at which they are deposited (Campanella et al., 1983, Kotzias et al. 1984) or determinant diameter and porosity of sediments (Abadjiev 1988). The angle of internal friction can be estimated from void ratio which is determined based on cone resistance (Matyas et al. 1984) or directly from $q_u - \gamma_z$ (Jones et al. 1981). Due to different conditions in which tailings are deposited within the Zelany Most reservoir, interpretation procedure requires application of different group of correlation relationships for sediments in the pond, those sediments on beaches and sediments used for construction of embankments (Fig. 2).

4.1 Geotechnical parameters of sediments forming beaches

Index parameters used to describe the state of non-cohesive sediments is relative density determined from cone resistance and vertical stress (equation 1).

$$D_r = a_1 + b_1 \log q_u + c_1 \log \sqrt[3]{\sigma_s}$$  \hspace{1cm} (1)

where $a_1$, $b_1$, and $c_1$ are constants

Regressions coefficients of equation 1 are also a function of grain size and porosity of sediments hence they can be determined on basis of mean value of friction ratio - $R_f$ one of the penetration characteristics (Fig. 4). A very significant issue of reliable evaluation of sediment density is the way of determining maximum and minimum void ratios. Depending on the method used, differences in evaluations of relative density can amount up to 30% (NGI report, 1994). For evaluation of shear strength parameters of non-cohesive sediments a procedure was worked out in which angle of internal friction can be determined from relative density being a function of cone resistance according to equation 2 (Mlynarcz et al., 1991). The angle of internal friction determined in this way corresponds to the angle obtained from classic consolidated drained compression triaxial test.

$$\phi' = 10(2a_2 + b_2D_r)$$  \hspace{1cm} (2)

where $a_2$, $b_2$ are constants

To evaluate the state of consistency of tailings the value of relative plasticity index was

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used which was determined on basis of fall - cone test according to Polish standard. Compared to the classic Casagrande method this method has higher accuracy due to high content of silt fraction and calcium carbonate in the tailings, and the obtained values of relative plasticity index correctly reflect changes in the state of consistence of the tailings with change in their moisture content. In this approach relative plasticity index is determined by equation 3.

$$I_r = a_3 + b_3 \sqrt{q_t} + c_3 q_t$$  \hspace{1cm} (3)

where $a_3$, $b_3$, $c_3$ are constants

Shear strength parameters of cohesive tailings were determined with field vane test due to their high sensitivity and very soft consistence. Construction of empirical relationships between undrained shear strength and cone resistance was carried out indirectly by means of the undrained shear strength ratio and relative plasticity index (Mlynarcz et al., 1991):

$$\frac{S_u}{\sigma_{uc}} = a_4 + b_4 I_r$$  \hspace{1cm} (4)

where $a_4$, $b_4$ are constants

4.2 Geotechnical parameters of tailings deposited in the pond

Evaluation of undrained shear strength of tailings deposited within the reservoir bowl where they are slowly consolidating process, besides technical problems, it was difficult to interpreting the test results. In these sediments are observed very low (on the threshold of equipment sensitivity) values of cone resistance and sleeve friction and relatively high values of pore pressure excess. Due to test conditions which correspond to those without drainage, shear parameters of the tailings can be expressed by undrained shear strength determined with vane test. The best estimation of shear strength from cone penetration test is obtained when measured cone resistance is corrected for pore pressure effects by means of, so called, effective area ratio of cone resistance (Mlynarcz et al. 1994).

$$S_u = (q_t - \sigma_{uc})/N_{ct}$$  \hspace{1cm} (5)

For the tailings stored in the pond mean value of $N_{ct}$ is $15.5 \pm 4.5$. The empirical relationships used in CPTU using excess of pore pressure $\Delta U$ to determine undrained shear strength had much higher measurement uncertainties. Analysis of this problem (Mlynarcz et al., 1994) indicated that the cause of low correlation is assuming pore pressure distribution as hydrostatic. Depending on the place of tailings deposition actual values of pore pressure are lower on beaches (Werno et al., 1993) or higher in the pond (Mlynarcz et al., 1994) than the values determined theoretically, assuming hydrostatic conditions.

Deformation characteristics of the sediments according to Senneset concept (Senneset et al.1989), can be determined on basis of cone resistance from the equation 6:

$$M = m(q_t - \sigma_{uc})$$  \hspace{1cm} (6)

Oedometer compression modulus determined for the tailings samples taken from the pond at the depths 5.6 - 17.2 m, for the stress range from zero to preconsolidation stress, $M$ changed from 0.6 to 3.5 MPa. The mean value of $\sigma^\prime$ coefficient determined for this group of tailings (equation 6) is $18.4 \pm 12.3$.

4.3 Geotechnical parameters of tailings used for embankments construction

For building up the reservoir embankments are used only non-cohesive sediments from group I sedimenting near the construction site and composed of the particles with the greatest grain size. Relative density of these sediments can be determined on basis of equation 7 where dry density of the sediment - $\rho_d$ is function of cone resistance and vertical stress (Tschuschke et al. 1992):

$$\rho_d = 1,040 - 0,115 \log 0,400 + 0,025 \log \frac{\sigma_{uc}}{P}$$  \hspace{1cm} (7)

where $\rho_d$ - reference unit weight (water)
assuming reference pressure - P equal to atmospheric one. This relationship is better than other relations of this kind that in evaluation of sediment density it eliminates the effect of measurement uncertainties related to evaluation of maximum and minimum void ratios (NGI report, 1994). Shear parameters deciding about stability of embankments and these fragments of beaches on which they were founded, due to grain size enabling conditions of free drainage, can be determined from equation 8:

\[ \theta = -1,547 + 1,574 \rho_{w} - 0,012w(1-0,083w) \]  \( (8) \)

The value of angle of internal friction from equation 8 corresponds to the angle obtained from the test in direct shear apparatus. The \( \rho_{w} \) parameter appearing in the equation is determined on basis of cone resistance (equation 7) while moisture content \( w \) from testing undisturbed soils samples. A certain estimate of sediment moisture content at known density can also be obtained from the plot of characteristics of pore pressure vs depth.

5. CONCLUSIONS

Studies carried out in the Department of Geotechnics of Poznan Agricultural University and HEBO Ltd. Poznan for the last few years related to introduction of the cone penetration test into current check of the state of post - flotation sediments stored in the Zelazny Most tailings dam indicate that this method is at present the most effective way of determining tailings parameters. One of the many benefits of this method is registering three different penetration characteristics with depth which can be supplemented with the test of pore water pressure dissipation. Interpretation of results yields continuous picture of changes in geotechnical parameters and grain size distribution with depth, which is not provided by traditional methods based on randomly taken samples. Among other virtues of the method are testing speed, possibility to repeat a test and the possibility of testing in extreme different soil conditions. Particularly important is the fact that this method enables identification of cohesive sediment parameters with respect to which traditional ways of describing their consistence state are of little effectiveness. A present, worked out procedure for interpreting penetration characteristics of post - flotation sediments makes it possible to:
- identify tailings with respect to grain size and separate in the profile sediment layers statistically homogenous with respect to granulation,
- determine basic physical and strength parameters in three zones of the reservoir different with respect to their parameters: pond, beaches and embankments,
- estimate deformation characteristics of the tailings deposited within the pond.

More effective application of the method requires solving some problems such as determination of actual maximal and minimal values of void ratio for non-cohesive sediments and determination of distribution of hydrostatic pressure in tailing profile.

6. REFERENCES


PROFILING OCR AND $K_0$ FROM PIEZOCONE PENETRATION TESTS

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SYNOPSIS: Successful profiling of the stress history of cohesive soil deposits from piezocone penetration test (PCPT) data depends on the accuracy of the interpretation method used. The method proposed in this study incorporates the influence of lateral stress coefficient ($K_0$) in predicting the overconsolidation ratio (OCR) of clays from piezocone data. The proposed method is compared with some of the existing methods to predict OCR, and is validated by PCPT results from calibration chamber studies and well documented test sites. The method proposed by Sully and Campanella 1991 for $K_0$ profiling of field deposits is seen to be very effective for anisotropically consolidated specimens. The pore pressure difference approach proposed by Sully et al. 1988 and the method proposed by Mayne 1991 are found to give acceptable and fairly good predictions of OCR respectively. The method proposed in this study is seen to give more accurate OCR predictions.

1. INTRODUCTION

The overconsolidation ratio (OCR) defined as the ratio of the preconsolidation pressure, $\sigma'_{pc}$ and the existing effective overburden pressure, $\sigma'_e$, is an important factor governing the strength, stress-strain behavior, and the compressibility characteristics of soils. Knowledge of the OCR is hence essential in selecting relevant soil parameters for proper design of geotechnical systems. The conventional method for determining OCR from laboratory oedometer tests on undisturbed samples obtained from the field is influenced by the type and procedure of testing and also by unavoidable sample disturbance. If a continuous profile of OCR with depth is required, the conventional laboratory method becomes time consuming labor intensive and expensive. In recent years, the estimation of OCR from piezocone tests has gained a lot of attention. The pore pressure gradient around the cone tip is presumably the most pronounced feature in predicting OCR of fine grained soils. Previous research by the authors have shown that $K_0$ is also an important factor influencing the pore pressure gradient around the tip. This paper incorporates the influence of $K_0$ in OCR prediction of clays from piezocone data. The proposed method is compared with some of the existing methods to predict OCR, and is validated by PCPT results from calibration chamber studies and well documented sites.

2. CALIBRATION CHAMBER STUDY

Four cohesive soil specimens 525 mm in diameter and 812 mm high were prepared and
tested under a back pressure \( u_0 \), of 138 kPa. PCPT’s were conducted using a 1 cm\(^2\) miniature piezocone penetrometer (fabricated by Fugro McClelland Engineers B.V., The Netherlands). Details of specimen preparation and PCPT results are given by Kurup 1993; Voyiadis et al. 1993; Kurup et al. 1994 and Voyiadis et al. 1994. A summary of the specimen stress histories and PCPT results are given in Table 1.

### Table 1. Specimen properties and PCPT results

<table>
<thead>
<tr>
<th>Specimen no.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil sample</td>
<td>K50</td>
<td>K33</td>
<td>K50</td>
<td>K50</td>
</tr>
<tr>
<td>Plasticity index, ( I_p (%) )</td>
<td>14</td>
<td>6</td>
<td>14</td>
<td>14</td>
</tr>
<tr>
<td>( K_r )</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>0.52</td>
</tr>
<tr>
<td>( \sigma''_u ) (kPa)</td>
<td>207</td>
<td>207</td>
<td>41.4</td>
<td>207</td>
</tr>
<tr>
<td>OCR</td>
<td>1</td>
<td>1</td>
<td>5</td>
<td>1</td>
</tr>
<tr>
<td>Undrained shear strength, ( s_u ) (kPa)</td>
<td>60</td>
<td>80</td>
<td>40</td>
<td>65</td>
</tr>
<tr>
<td>Rigidity index, ( I_r = G_u )</td>
<td>267</td>
<td>100</td>
<td>150</td>
<td>567</td>
</tr>
<tr>
<td>Cone resistance, ( q_r ) (kPa)</td>
<td>1183</td>
<td>1249</td>
<td>661</td>
<td>644</td>
</tr>
<tr>
<td>( \Delta u ) at tip (kPa)</td>
<td>562</td>
<td>632</td>
<td>528</td>
<td>490</td>
</tr>
<tr>
<td>( \Delta u ) above cone base (kPa)</td>
<td>624</td>
<td>591</td>
<td>406</td>
<td>368</td>
</tr>
</tbody>
</table>

### 3. LATERAL STRESS COEFFICIENT (\( K_r \))

Sully and Campanella 1991 proposed a linear relationship (Equation 1) to estimate \( K_r \).

\[
K_r = a + b \cdot (PPSV)
\]  

(1)

where \( a = 0.5 \) and \( b = 0.11 \). The normalized pore pressure parameter, PPSV, is defined as:

\[
PPSV = \frac{(u_1 - u_2)}{\sigma''_u}
\]

(2)

where \( u_1 \) and \( u_2 \) are pore pressures at the tip and just above the cone base respectively.

Comparison of PPSV vs. \( K_r \) for the four chamber specimens are shown in Figure 1. The results of specimen 1 and 2 is seen to fall away from the relationship proposed by Sully and Campanella 1991. This is because their relationship is based on results from a number of research sites around the world. In the field, one rarely comes across soils that are normally consolidated having a \( K_r \) value equal to unity (i.e., isotropic, normally consolidated specimen). A \( K_r \) value of unity in the field would usually mean overconsolidated soils for which the PPSV will be high due to the high pore pressure gradient around the tip for such soils. The PPSV value for the overconsolidated specimen 3 is close to the proposed value. The slight discrepancy is due to the difference in the stress paths in the laboratory specimen \( (K_r = 1) \); isotropic loading and unloading) and field deposits \( (K_r < 1) \) during NC stage and increases with increasing OCR. Specimen 4 was normally consolidated, anisotropically, under conditions of zero lateral strain (similar to in-situ soils). The method proposed by Sully and Campanella, to determine \( K_r \) is seen to be very effective for this specimen.
4. OVERCONSOLIDATION RATIO (OCR)

4.1 Interpretation methods to predict OCR

Some of the interpretation methods (suggested by Schmertmann 1978; Mayne 1987; Mayne and Bachus 1988; Sully et al. 1988; Mayne 1991, 1992) to estimate OCR are evaluated using the chamber PCPT data in Table 2. Schmertmann’s method (1978) is seen to overestimate OCR for all the specimens. The only CPT data that has been used is the corrected cone resistance $q_c$. $N_{cr}$ is the empirical cone factor.

From Table 2, (Specimen 3) it appears that the $B_n$ method is not sensitive to OCR. No universal correlation is known to exist between $B_n$ and OCR. $B_n$ method is thought to be influenced by variations in soil plasticity and sensitivity and could be more of a site specific parameter. Moreover, the unreliability of the tip resistance in soft clays (Tumay and Acar 1985) can add to the errors in estimating OCR using the $B_n$ method. Significant scattering has been observed in the past in $B_n$ vs. OCR.

The pore pressure difference (PPD) method (Sully, et al. 1988) based on the pressure gradient existing around the tip and its dependence on OCR, seems to predict OCR reasonably well for specimens 3 and 4. It is however seen that for specimen 2, the OCR is slightly underestimated, and for specimen 1, the OCR is severely underestimated. The reason for this is obvious. Specimens 1 and 2 are isotropically consolidated and the difference in the pore pressures at the tip and above the cone base (i.e., $u_t - u_b$) are close to zero. In fact for specimen 1, this value is negative because the pore pressure measured at the tip is lower than that above the cone base. The low pore pressure gradient around the cone tip in isotropically consolidated specimens results in low values of predicted OCR.

The predicted OCR’s using the methods suggested by Mayne 1987; Mayne and Bachus 1988 and Mayne 1991, 1992 are shown in Table 2. These prediction methods have been formulated from the theories of cavity expansion and critical-state soil mechanics. $M$ is the slope of the critical-state line. The methods proposed by Mayne 1987, and Mayne and Bachus 1988 derived from spherical cavity expansion theory and excess pore pressures measured above the cone base give reasonable predictions of OCR for specimens 1, 2 and 3. However for specimen 4, the OCR is underestimated. Interpretation using the expression derived from cylindrical cavity expansion and tip pore pressure overestimates the OCR’s. The method suggested by Mayne 1991, 1992 using pore pressures measured above the cone base give fairly good predictions of OCR. Interpretation using pore pressures measured at the cone tip overestimates the OCR’s for specimens 1, 2 and 4.

The methods proposed by Mayne 1991, 1992 was developed based on the spherical cavity expansion theory of Vesic 1975 which has been formulated for the octahedral normal stress ($\sigma_n = \sigma_{om}$). However in the expression proposed by Mayne 1991, $\sigma_n$ has been taken equal to $\sigma_{om}$ since the in-situ lateral stress is difficult to determine. The method proposed in this study (Table 2), combines the technique of $K_u$ profiling suggested by Sully and Campanella 1991 (i.e., equation 1) with the equation proposed by Mayne 1991 (substituting $\sigma_n = \sigma_{om}$ instead of $\sigma_n = \sigma_{om}$). If $K_u$ is known (as in laboratory specimens of this study) the first expression proposed in this study (Table 2) may be used. For field deposits where $K_u$ is unknown, OCR may be estimated from the second expression proposed in this study (Table 2).
Table 2. Interpretation of OCR from PCPT data

<table>
<thead>
<tr>
<th>Methods</th>
<th>Equations</th>
<th>Over Consolidation Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Schmert-mann</td>
<td>$s_m = (\sigma_{0} - \sigma_{uu})/(\sigma_{0} - \sigma_{uu})$</td>
<td>2.3 3.9 12.3 2.1</td>
</tr>
<tr>
<td>(1978)</td>
<td>$s_u = (\sigma_{0} - \sigma_{uu})/(\sigma_{0} - \sigma_{uu}) \leq 0.11 + 0.0037\Delta\sigma$</td>
<td></td>
</tr>
<tr>
<td>$B_n$ method</td>
<td>$B_n = (\sigma_{0} - \sigma_{uu})/(\sigma_{0} - \sigma_{uu})$</td>
<td>1.1 1.2 1.1 0.9</td>
</tr>
<tr>
<td>Sully, et al. 1988</td>
<td>$OCR = 0.49 + 1.50 PPD$</td>
<td>0.1 0.8 4.9 1.4</td>
</tr>
<tr>
<td>PPD = $u_i - u_o$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mayne 1987</td>
<td>$OCR = (0.317 \Delta\sigma/\sigma_{uu})^{1.18}$</td>
<td>0.9 0.8 7.6 0.4</td>
</tr>
<tr>
<td>Mayne and Bachus 1988</td>
<td>$2[(\Delta\sigma/\sigma_{uu})^2]/((M/2)ln(G/\sigma_{uu}) - 1)^{1.32}$</td>
<td>1.7 1.4 1.0 0.4</td>
</tr>
<tr>
<td></td>
<td>spherical $2[(\Delta\sigma/\sigma_{uu})^2]/((2M/3)ln(G/\sigma_{uu}) - 1)^{1.35}$</td>
<td>1.0 0.8 1.1 0.3</td>
</tr>
<tr>
<td>Mayne 1991, 1992</td>
<td>$OCR = 2[(\sigma_{ff} - \sigma_{uu})/(1.95M + 1)]^{1.3}$</td>
<td>1.6 4.3 1.8 0.6</td>
</tr>
<tr>
<td>Proposed in this study</td>
<td>$OCR = 2[(\sigma_{ff} - \sigma_{uu})/(1.95M + 1)]^{1.3}$</td>
<td>1.6 4.7 0.11</td>
</tr>
<tr>
<td>Specimen OCR</td>
<td>EXPERIMENTAL (From calibration chamber tests of this study)</td>
<td>1.0 1.0 5.0 1.0</td>
</tr>
</tbody>
</table>

and values of ‘a’ and ‘b’ suggested by Sully and Campanella may be used. The OCR prediction technique proposed in this paper is seen to be more accurate and promising since it includes the influence of $K_n$ and the pore pressure gradient that develops around the tip in the method based on the spherical cavity expansion theory and critical state soil mechanics.

4.2 Applications to field data

The validity of the expression proposed in this study for predicting OCR is verified by application to PCPT data from four sites (Bakklandet, Troll East area 2, Glava, and Valoya) reported by Sandven 1990. Comparisons between predicted OCRs and their reference values (from one-dimensional consolidation tests) for the four sites are shown in Figures 2a through 2d. The method proposed in this study give better OCR predictions than those proposed by Mayne 1991 and Sully, et al. 1988, in Bakklandet, Troll East, and Glava clays. For Valoya clay, both the proposed method and that proposed by Mayne 1991 give fairly good OCR predictions. The method proposed by Sully, et al. 1988 underestimates the OCR values in Bakklandet, Glava and Valoya clays and overestimates the OCRs in Troll East.
5. CONCLUSIONS
The method proposed in this study incorporates the influence of $K_s$ in predicting OCR of clays from piezocene data. The proposed method is compared with some of the existing methods to predict OCR, and is validated by PCPT results from calibration chamber studies and well documented test sites.
The method proposed by Sully and Campanella 1991 for $K_s$ profiling of field deposits is seen to be very effective for anisotropically consolidated specimens. The pore pressure difference approach proposed by Sully et al. 1988 and the method proposed by Mayne 1991 are found to give acceptable and fairly good predictions of OCR respectively. The method proposed in this study is seen to give more accurate OCR predictions.

6. ACKNOWLEDGMENTS
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7. REFERENCES


CPTU profiling: A numerical method

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SYNOPSIS: Several soil identification charts exist for evaluating soil type from piezocone test (CPTU) data. These charts, together with published information on the sensing of soil interfaces by cone resistance and pore pressure, enable the engineer to profile piezocone soundings. A numerical method of profiling CPTU soundings is proposed based on trigonometric polynomial approximation and the Jones and Rust (1982) soil identification chart. The method requires field data to be corrected for pore pressure effects, overburden pressure, equilibrium pore pressures and the pore pressure offset. Trigonometric polynomial approximation leads to a rough indication of the principal layering which is then refined and classified using the soil identification chart. The method shows good agreement when compared with field examples.

1. INTRODUCTION
The piezocone test has proven itself a popular in-situ test for site investigation and geotechnical design. The usual progression of site investigation with piezocone testing is to perform the soundings, develop detailed site profiles with soil classification charts and then selectively sample and test to provide additional information regarding ambiguous classifications. Considerable experience exists concerning stratification and the classification of soil types. Soil identification charts have been developed by Jones and Rust (1982, 1983), Senneset and Janbu (1985; Senneset et al., 1989), Robertson et al. (1986), Robertson (1990, 1991), Larsson and Muladbic (1991) and Jeffries and Davies (1991). Together with information on the sensing of soil interfaces by Treadwell (1975), Schmertmann (1978), Torstensson (1982), Sutcliffe and Waterton (1983), Campanella et al. (1983) and Campanella and Robertson (1988) these charts provide a means of profiling piezocone soundings by hand. This, however, is a time consuming task relying heavily on the experience and judgment of the engineer.

The alternative method proposed here, provides a numerical basis of profiling CPTU data thus saving time and effort without compromising the accuracy and flexibility of hand methods. The method, suitable to computer implementation, is based on trigonometric polynomial approximation to indicate the principal layering and on a digitised Jones and Rust soil identification chart for detailed stratification and soil classification.

2. DATA PREPARATION
Raw field cone penetration data, especially piezocone data, need to be corrected for certain effects before analysis in terms of stratification and soil behaviour type is possible. Apart from possible calibration, data need to be corrected for differential pore pressure effects (Biligh et al., 1981; Campanella et al., 1982, 1983), overburden pressures (Robertson & Campanella, 1985; Douglas et al., 1985; Olsen & Farr, 1986), ambient or equilibrium pore pressures (Jones & Rust, 1982, 1983) and the pore pres-
2.35

Volume 2

sure offset (Vermeulen, 1994).

In addition to these corrections, a further reduction in the number of data sets is generally necessary as a consequence of the shear number of readings taken by electric penetrometers, i.e., up to 200 readings per meter. It is recommended that a depth interval of approximately 25 mm (40 readings per meter) be adopted, resulting in a minimum interpreted layer thickness of 75 mm.

Figure 1 shows the effect of data preparation on an example piezocone sounding in a deltaic deposit (Rust, 1993).

2.1 Adjusting the cone resistance values

The measured cone resistance values need to be corrected for differential pore pressure effects,

\[ q_e = q_i + \lambda \cdot u_r \]  

where \( q_e \) is the corrected cone resistance, \( q_i \) the measured cone resistance, \( u_r \) the measured pore pressure and \( \lambda \) the end area ratio.

The net cone resistance is the effective remaining cone resistance after eliminating the overburden pressure. Therefore,

\[ q_n = (q_i - \sigma_w) \]  

where \( q_n \) is the net cone resistance and \( \sigma_w \) the overburden pressure at the depth of \( q_i \).

2.2 Adjusting the pore pressure values

The measured pore pressures need to be converted to excess pore pressures, characteristic of material type,

\[ \Delta u = u_r - u_e \]  

where \( \Delta u \) is the excess pore pressure, \( u_r \) the measured pore pressure and \( u_e \) the equilibrium pore pressure.

2.3 The pore pressure offset

A small depth offset generally exists between corresponding values of cone resistance and pore pressure measured at the same time. This offset is equivalent to that found between cone resistance and friction sleeve values measured with friction cones and is believed to be the result of both the physical separation between the cone tip and the filter element and the fact that cone resistance is affected by soil stiffness ahead and behind the tip whereas pore pressure is in turn governed by the pore pressure distribution around the cone tip. However, these variations with depth are generally not large and the phenomenon can best be resolved by lining up the pore pressure plot with the cone resistance plot to match, over the average, peaks and troughs in the profile.

3. TRIGONOMETRIC POLYNOMIAL STRATIFICATION

This section describes the use of trigonometric polynomials as a first approximation to the principal stratification of a piezocone sounding.

3.1 Approximation theory

Trigonometric polynomial approximation provides a method for the least-squares approximation and interpolation of large amounts of data when the data is given at equally spaced intervals. This proves to be an effective way of studying piezocone data, that is cone resistance versus depth, where it is not uncommon to be faced with 8000 or more data sets.

Trigonometric polynomials of degree \( n \) can approximate \( 2m \) paired data sets \((x_j, y_j)\), \( j = 0 \) to \( 2m-1 \), with the first elements in the pairs equally partitioning the closed interval \([-\pi, \pi]\), by,

\[ S_n(x) = \sum_{k=0}^{n} a_k \cos kx + b_k \sin kx \]  

where

\[ a_k = \frac{1}{2\pi} \sum_{j=0}^{2m-1} y_j \cos kx_j \quad \text{for each} \quad k = 0, 1, \ldots, n \]  

\[ b_k = \frac{1}{2\pi} \sum_{j=0}^{2m-1} y_j \sin kx_j \quad \text{for each} \quad k = 1, 2, \ldots, n-1 \]
3.2 Fitting trigonometric polynomials to piezocone data

Piezocone test data generally tend to be spaced at equal intervals. Trigonometric polynomial approximation can, therefore, be applied in the following manner,

- 2m paired data sets imply an even number of data sets; if there is an uneven number of data sets in the prepared file, the last set is duplicated without any significant influence on the final result.
- (x,y) is represented by depth and net cone resistance values respectively; it is also possible to use depth and excess pore pressure values, but this relationship has not been investigated.
- As for the [-π,π] interval, a simple transformation of the form
  \[ z_j = \pi \left( \frac{2\pi}{2m - 1} \right) j \text{ for } j = 0,1, \ldots, 2m - 1 \]

results in transformed data of the form \((Z_j,Y_j)\) for \(j = 0\) to \(2m - 1\) over the interval \([-\pi,\pi]\).

These assumptions together with equations 4, 5, and 6 can now be used to stratify a piezocone sounding. Layer interfaces are taken at the inflection points of the polynomial. Increasing the degree of fit, \(n\), increases the accuracy with which the polynomial approximates the data.

Figure 2 illustrates the effect of fitting trigonometric polynomials of degrees 5, 10, 15 and 20 to the net cone resistance values of another sounding close to the previous example.

3.3 Reducing the number of approximated layers

It is possible to reduce the total number of approximated layers without losing "accuracy", thus enhancing the effectiveness of the method. This is accomplished by comparing the average net cone resistance values of consecutive layers and combining those layers with similar values. A limiting difference of 0.1 MPa was adopted for the case in question.

3.4 Selecting the optimum degree of fit

There exists, after layer reduction, an optimum degree of fit that represents the principal layering adequately, with the least number of terms. To quantify "goodness of fit" it was decided to calculate a least squares error between values of \(q_s\) and \(\Delta u\) from the test and average values of \(q_s\) and \(\Delta u\) from the interpolated layers. The fit that results in the smallest error qualifies as the opt-
Figure 2. Fitting trigonometric polynomials to the net cone resistance values of an example piezocone sounding with (a) n = 5, (b) n = 10, (c) n = 15 and (d) n = 20.

Figure 3a shows the end result of fitting trigonometric polynomials to the example sounding in Figure 1 after reducing the number of interpolated layers with a limiting difference of 0.1 MPa and selecting the optimum degree of fit as described above. For a detailed discussion of the technique refer to Vermeulen (1994). Figure 3b shows the profile generated from undisturbed sampling in a nearby borehole, superimposed on the CPTU results.

From Figure 3 it is clear that trigonometric polynomials show tremendous potential as a first approximation to the greater stratification...
4. DETAILED STRATIFICATION AND CLASSIFICATION

The next step after polynomial approximation, is to identify details in the stratigraphy and to classify the interpolated layers using one of the existing classification charts. The soil behaviour type classification chart developed by Jones and Rust (1982; 1983) was chosen for the following reasons: It identifies a wide range of soil behaviour types and consistencies, provision is made for both positive and negative excess pore pressures, it allows for the effect of overburden pressure, it was developed among others for South African soils and it is applicable to mine tailings. Figure 4a shows the classification chart with data from the example sounding, note the crowding in the very soft clay zone. Figure 4b shows the same chart but with net cone resistance on a log scale. This plot gives a much better representation of the spread of data in the very soft clay zones.

4.1 Stratification and classification

Trigonometric polynomials were used as a first approximation to the principal stratification, after which the identification chart can be used to examine each layer and to interpret details in the stratigraphy (the refinement process, see section 4.2). Trigonometric polynomial stratification is only concerned with the net cone resistance values, but by using the identification chart, excess pore pressure also starts to play a role. This is important because certain features in a piezocone sounding are best brought out by the pore pressure response. The pore pressure response also allows for a finer delineation than is possible with cone resistance alone.

The average net cone resistance and excess pore pressure values of the resulting layers can then be plotted on the digitised identification chart and classified in terms of soil behaviour type and consistency.

4.2 The refinement process

Three classes of refinement are recommended, i.e. coarse, medium and fine. The coarse option results in a stratification similar to that after polynomial approximation with a minimum
layer thickness of approximately 750 mm. With medium refinement the minimum layer thickness reduces to approximately 250 mm and with fine refinement to approximately 75 mm. A refinement constant is accordingly defined. The engineer needs to decide which of these are relevant for the purpose of his analysis, for example choosing the fine option when analysing mine tailings dams and the coarse option for a preliminary estimate of settlement of a typical estuarine clay deposit.

The refinement process scans through the data sets in a layer as defined by the polynomial approximation, counting the number of sets occurring in every zone on the soil identification chart. Three zones containing the maximum number of points are identified. These zones are considered those most likely to represent the layer and are hereafter labelled defining zones. The layer is scanned again, but this time counting the number of consecutive data sets occurring in any one of the defining zones. If the number of sets occurring in a defining zone exceeds the refinement constant, an additional layer gets added. Added layers are merged if two consecutive layers occurring in the same defining zone are separated by a number of data sets smaller than the refinement constant. The gaps remaining between added layers are defined as additional layers.

The numerically derived soil profiles compare well with undisturbed sampling, especially in stratification, see Table 1 (from the example in Figure 1). Discrepancies in this table may be caused by the fact that the some layers are so thin that their full cone penetration resistance cannot be mobilised and therefore plot in the very soft or loose zones rather than in the stiffer or more dense zones. Dissipation test results may prove to be the answer in clarifying such discrepancies when identifying thin layers of material with the identification chart.

Classification is strongly influenced by factors such as changes in stress history, in-situ stresses, sensitivity, stiffness, macrofabric and voids ratio (Robertson, 1990) which can not be readily predicted by soil classification charts. Keep in mind also that the identification chart refers to soil behaviour type rather than a particle size distribution classification, which explains some of the differences in descriptive terms for the layers.
Table 1. Soil profiles from the proposed method and from undisturbed sampling.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth range</th>
<th>Description</th>
<th>Layer</th>
<th>Depth range</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.04 - 1.42m</td>
<td>Loose sand</td>
<td>1</td>
<td>0 - 1.3m</td>
<td>Fill (loose sand)</td>
</tr>
<tr>
<td>2</td>
<td>1.42 - 4.45m</td>
<td>Clay</td>
<td>2</td>
<td>1.3 - 4.5m</td>
<td>Firm silty clay</td>
</tr>
<tr>
<td>3</td>
<td>4.45 - 5.58m</td>
<td>Sand</td>
<td>3</td>
<td>4.5 - 5.3m</td>
<td>Medium dense sand</td>
</tr>
<tr>
<td>4</td>
<td>5.58 - 8.89m</td>
<td>Clay</td>
<td>4</td>
<td>5.3 - 10.2m</td>
<td>Medium dense silty sand</td>
</tr>
<tr>
<td>5</td>
<td>8.89 - 25.98m</td>
<td>Clay</td>
<td>5</td>
<td>10.2 - 23.8m</td>
<td>Soft clay</td>
</tr>
<tr>
<td>6</td>
<td>25.98 - 28.23m</td>
<td>Clay</td>
<td>6</td>
<td>23.8 - 26.3m</td>
<td>Medium dense silty sand</td>
</tr>
<tr>
<td>7</td>
<td>28.23 - 29.41m</td>
<td>Clay</td>
<td>7</td>
<td>26.3 - 28m</td>
<td>Stiff clayey silt</td>
</tr>
</tbody>
</table>

5. SUMMARY OF THE PROPOSED NEW METHOD

- First the principal layering is found by trigonometric polynomial approximation, which includes layer reduction and optimisation.
- A refinement constant is defined in accordance with the minimum desired layer thickness.
- Each principal layer is then examined and further stratified by using the identification chart.
- Consecutive layers in the sounding are merged if the difference between the average net cone resistance and excess pore pressure values, calculated over the mid two thirds of the layer, are less than 0.1 MPa and 100 kPa respectively and if they fall within the same soil behaviour type and consistency zone as defined by the soil identification chart.
- The resultant layers are classified in terms of soil behaviour type and consistency with the identification chart.

6. CONCLUSIONS

The alternative numerical piezocene profiling method described in this report lends itself to computer implementation thus saving time and effort without sacrificing the accuracy or flexibility of hand methods.

Trigonometric polynomial approximation shows great potential as a first approximation to the greater stratification of cone penetration data.

Implementation of the Jones and Rust (1982) soil identification chart allows further identification of thin drainage layers and other details in the profile. This is extremely useful in highly stratified profiles such as mine tailings where the profile may consist of over 100 layers. The identification chart is also used to classify resulting layers.

The envelope over which the cone senses soil variation leads to some imprecisions in locating soil interfaces and it has some effect on the evaluation of engineering properties. This sensing envelope can result in the cone resistance not developing its full potential in relatively thin layers which can lead to incorrect classification on the soil identification chart. However, the proposed method is flexible in its ability to accommodate any soil identification chart and future modifications to the charts.

It remains the responsibility of the engineer to verify profiles derived by the numerical method and to compare results with sampling and other tests.

7. REFERENCES


CPT, shear wave propagation and freeze probing to estimate the void ratio in loose sands

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SYNOPSIS: The applicability of CPT and of horizontally polarized shear wave tests to estimate the void ratio in open mine fills is investigated. To measure the void ratio directly, two different freeze probing procedures, with and without coring, are carried out. A correlation between shear wave velocity and void ratio is proposed, based on shear wave velocities measured in the laboratory and in situ. The choice of the earth pressure coefficient plays an important role in evaluating the normalized shear wave velocities in situ. A correlation between CPT-cone resistance and void ratio proposed by Laubag company was successfully verified. Under the water table the void ratio could be sufficiently well estimated using the method proposed by Been et al. In such kind of soil an accurate estimation of the void ratio can be achieved only if the thin-layered stratigraphy is identified in detail.

1. INTRODUCTION

Refilling of open pit mines in the Lausitz region in the eastern part of Germany leads to the formation of very loose sandy deposits. Accurate description of the undisturbed state of soil plays a decisive role in predicting spontaneous liquefaction.

Generally the undisturbed in-situ state of granular soils can be defined by the following state variables: relative density, degree of saturation and principal stresses. Although these variables could be measured directly, usually, in engineering practice, they are correlated with the results of field tests through theoretical or empirical relationships.

Freeze probing is the most accurate method in order to determine the in-situ void ratio of granular materials. Unfortunately this procedure is expensive, time consuming and its practical execution is complicated. Indirect methods like wave propagation measurements are fraught with similar disadvantages. On the other hand, CPT can be an appropriate alternative if an adequate correlation between the measured soil response and the void ratio can be used.

2. SOIL PROFILE

Two situations were considered in this work:

Site A): The deposit consists of a 27 meter thick refilled open mine near Hindenberg. It is composed of many very loose sand layers with a fine content due to lignite varying between 0% and 15%.

Soil compaction in the lower 20 meters was achieved by blasting and in the upper 3 meters by vibratory rollers. The original density between -3m and -7m could not be changed by the compaction methods used (Gudehus et al., 1995).

Site B): The deposit consists of a 45 meter thick refilled open mine near Senftenberg divided in 3
layers separated by working planes. Alike site A, the fill is composed of loosely deposited fine and medium sands with a small fine content (Gudehus et al., 1993 and Raju et al., 1994).

3. FREEZE PROBING

Frozen soil samples in-situ were obtained using two methods.

In the case of site A, a casing was carefully pressed to a depth of 18 m, a freeze pipe was put inside the casing and a cylinder of soil was frozen along the total length of the casing. A sample of an external diameter of up to 0.39 m could be pulled out (Fig. 1). Accurate soil profiles showing the strong inhomogeneity of the fill could be identified, see Fig. 1. The void ratios at point 2 were determined by freeze probing and the corresponding limit void ratios were obtained by laboratory tests as shown in the same figure.

Fig. 1: Soil profile, void ratios from freeze probing GS2 and limit void ratios, site A, point 2

The void ratio and the degree of saturation depend on the distance from the casing as shown in Fig. 2. With increasing distance from the casing the void ratio rises and the degree of saturation decreases. The undisturbed values could have been precisely estimated only if a large diameter of the frozen volume had been sampled. The volume of the samples was determined by submerging it in anti-freeze using a rubber membrane around the sample (Gudehus et al., 1995).

Fig. 2: Influence of compaction due to the installation of the casing, site A

At site B a borehole was drilled down to a depth of 11 m and filled with bentonite suspension to prevent failure of the borehole. Later a freeze pipe was lowered by pressing it down to a depth of 13 m. A soil cylinder with a radius of 0.80 m and a height of 2.0 m was frozen and samples with a diameter of 0.1 m were taken by core-drilling in the frozen cylinder. In this case samples were recovered at a distance of 0.59 m from the center of the freeze pipe. This distance can be regarded as large enough to obtain accurate values of void ratio and degree of saturation. The determination of the volume of the samples was carried out by submerging it in water using a cover of paraffin around the sample.

4. SHEAR WAVE VELOCITY MEASUREMENT

Shear wave velocities were measured in the field using crosstube tests and in the laboratory using a free-free Resonant-Column-device (Prange 1983).

In situ horizontally polarized shear waves were generated at depths of 4 m (site A) and 11 m
(site B) by a vane source. Monitoring the arriving waves, geophones at different distances from the source up to 110 m were installed.

Soil specimens for the RC-tests were prepared using thawed material taken on the axis and at the depths where the crosshole measurements were executed.

![Diagram showing void ratio vs normalized shear wave velocity, site A and B.](image)

**Fig. 3:** Void ratio vs normalized shear wave velocity, site A and B

The normalized shear wave velocities of both sites are given by

\[ v_s = \frac{v_s_{\text{meas.}}}{\sqrt{\frac{\sigma'_0}{P_0}}} \]  

where \( \sigma'_0 = \frac{1}{2} (\sigma_t + 2 \cdot K \cdot \sigma_t) \), average granular pressure; \( P_0 = 100\,\text{kPa} \), atmospheric pressure; \( K \), earth pressure coefficient, see Fig. 3.

In case of loose sand layers, field and laboratory results indicated that the earth pressure coefficient \( K = 0.5 \) assumed in the RC-tests is slightly larger than the field value, (Fig. 3, GS2). The actual value of \( K \) in the uncompacted fill lies between the active earth pressure coefficient and the one at rest (Kudella 1995). On the other hand, in compacted layers the assumed \( K \)-value of 0.5 underestimates the real one (Fig. 3, GS3).

A numerical analysis with the hypoplastic constitutive law of Gudehus (Gudehus 1994) allowed to estimate the \( K \)-value after blasting depending on the in-situ void ratio, (Fig. 4) (Gudehus et al., 1995). For \( e = 0.65 \) a shift of the shear wave velocity towards the other experimental points was achieved, (Fig. 3).

![Graph showing earth pressure coefficient vs void ratio after compaction, theoretical calculation.](image)

**Fig. 4:** Earth pressure coefficient vs void ratio after compaction, theoretical calculation

A correlation between the normalized shear wave velocity and the void ratio for the soil investigated can be expressed by the following relationship:

\[ e = -0.0117 \cdot \left( \frac{v_s}{v'_s} \right)^{0.25} + 2.85 \]  

(2)

5. CONE PENETRATION TESTING

At site A cone penetration tests were carried out by Laubag company (Turski et al. 1995) following the procedure recommended by the German standard DIN 4094. In order to control the inhomogeneity of the fill, two tests, CP1 and CP2, were carried out in each position close to each other. As it can be seen in Fig. 5 a shallow layer was densified due to vibratory rollers on top and it is underlain by a loose fill.

Although the friction ratios enable qualitative identification of some layers with a larger fine content, for instance at 6 - 7 m depth, the complicated thin-layer stratigraphy of the soil did not permit a more detailed interpretation (Fig. 6).
The German standard DIN 4094 gives the following formula for uniform graded sands above water table:

\[ I_d = -0.33 + 0.73 \cdot \log(q_v) \]  

(3)

Although no expression is proposed for sandy soils below the water table, it is suggested that (3) could be still applied. However, in our case, reasonable estimations of the void ratio below the water table could not be obtained using (3).

A better correlation was found for the Niederlausitz sands below the water table by Laubag company (Turski et al. 1995)

\[ I_d = 0.0245 + 0.5539 \cdot \log(q_v) \]  

(4)

using laboratory tests with a large calibration chamber. It was verified in the field using freeze probing and conventional methods to determine the relative density.

In order to determine the void ratio at different depths it is necessary to consider the limit void ratios of remoulded samples obtained by freeze probing in the corresponding depths. The limit void ratios for the soil profile considered were plotted in Fig. 1. A summary of these values is presented in Tab. 1.

<table>
<thead>
<tr>
<th>depth [m]</th>
<th>sand</th>
<th>( \varepsilon_{m,\text{min}} )</th>
<th>( \varepsilon_{m,\text{max}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 2.0</td>
<td>mixed</td>
<td>0.463</td>
<td>0.930</td>
</tr>
<tr>
<td>2.0 - 3.9</td>
<td>clean</td>
<td>0.423</td>
<td>0.847</td>
</tr>
<tr>
<td>3.9 - 6.2</td>
<td>mixed</td>
<td>0.463</td>
<td>0.930</td>
</tr>
<tr>
<td>6.2 - 7.2</td>
<td>mixed</td>
<td>0.466</td>
<td>0.966</td>
</tr>
<tr>
<td>7.2 - 15.0</td>
<td>clean</td>
<td>0.417</td>
<td>0.841</td>
</tr>
</tbody>
</table>

Tab. 1: Limit void ratios

Additionally, the method to estimate the void ratio proposed by Been et al. 1986/1987 was used. The procedure applies to sandy soils with fines. The authors defined a state parameter \( \psi \) as

\[ \psi = \varepsilon - \varepsilon_{ss} \]  

(5)

where \( \varepsilon \) is the void ratio on the normal consolidation line and \( \varepsilon_{ss} \) is the void ratio on the steady state line, both corresponding to the same mean normal stress. The relationship between the state parameter \( \psi \) and the cone resistance is
\[ \psi = -m \cdot \ln \left( \frac{\sigma_v - \sigma_0}{\sigma_0} \cdot \frac{1}{k} \right), \]  

(6)

where \( \frac{\sigma_v - \sigma_0}{\sigma_0} \) is the stress normalized cone resistance and \( m, k \) are constant values depending on the inclination of the steady state line \( \lambda_{ss} \):

\[ m = 8.1 - \ln \lambda_{ss} \]  

(7)

\[ k = 8 + 0.5 \]  

(8)

Values of \( \lambda_{ss} \) were 0.0303 for clean sands and 0.0350 for mixed sands. Determination of the laboratory from CIU triaxial tests on remoulded samples (Gudehus et al., 1995). The factors \( m = 11.6 / 11.5 \) and \( k = 35.1 / 30.0 \) resulted from equations (7) and (8) for clean and mixed sand respectively. These values are in accordance with those found by Becu et al., 1987. To calculate \( \psi \) an earth pressure coefficient of \( K = 0.5 \) and a perfectly dry soil above the water table were assumed.

Below the water table the \( e \)-values estimated by equations (4,5) are in good agreement. Above the water table the void ratios estimated by expression (5) are lower than the ones calculated by (3), because the influence of the negative pore pressure was disregarded. Additionally, in Fig. 7, the measured void ratios from freeze probing are shown. These values are mostly lower than the correlated ones, because of the soil disturbance around the casing as observed in Fig. 2.

At a depth of about -7 m a high lignite content in the soil samples results in a large scatter of the measured values. The large variations of the mass density of lignite in this area (between 0.3 and 1.3 g/cm³) made an accurate evaluation of the void ratio at this depth difficult (Fig. 7).

6. CONCLUSION

A continuous freeze probing profile at site A allowed a detailed identification of the thin-layered stratigraphy. We recommend a larger frozen volume, however, to be sampled in order to obtain undisturbed values of the void ratio and degree of saturation in-situ. Undisturbed frozen specimens could alternatively be obtained by coring (site B), but this method appears to be more expensive.

A correlation was established between void ratios and shear wave velocities from RC-tests. This relationship holds for the field conditions if the mean effective stress \( \sigma'_e \) is known. The latter can be estimated from a numerical analysis, e.g. using a hypoplastic constitutive law as demonstrated in section 4.

Predictions of the void ratio in-situ made using two correlations proposed by Laubag company and by Been et al. appear realistic. Below the water table the two correlations coincide fairly well. The directly measured void ratios are mostly lower than the correlated ones because of the soil disturbance around the casing.

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Estimation of strength parameters of decomposed granite soil using portable cone penetration test

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SYNOPSIS: In Japan many slope failures due to heavy rainfall have occurred at the weathered granite soil region. The strength parameters are needed to consider the slope stability. The preparation of undisturbed specimen is very difficult because the weathered granite soil is a sandy soil. A portable cone penetrometer is suitable to investigate the strength of soil in a slope. It was confirmed that the cone resistance $q_c$ at a saturated condition corresponded to the strength of a saturated-drained condition. In-situ static penetration tests were conducted at eight sites. The relationships between the inclination of $q_c$ to a depth, $\theta_q$, and the angle of shear resistance $\phi_s$ were investigated. The linear correlation was observed between $\theta_q$ and $\phi_s$. It can be obtained $\phi_s$ of a weathered granite soil slope from a portable static penetration test.

1. INTRODUCTION

Annual incidence of slope failures, landslides and debris flows in Japan are shown in Fig. 1. These disasters happen at a heavy rain, such as Baiu front and Typhoon. Most of the slope failures have occurred at the slope composed of a weathered granite soil, called Masado, and a volcanic soil, called Shirasu. The failures of Masado slopes approximately happen along surfaces of base rocks or different boundaries of weathering degree as shown in Fig. 2. Fig. 3 shows the thickness and angle of actual failed slopes. This figure shows that the depth of slip surface of failed slope is comparatively shallow, about 0.5 ~ 1.0 m. Strength parameters of a weathered granite soil were needed to analyze the stability of slope. As a weathered granite soil is a sandy soil, the preparation of a undisturbed specimen is very difficult. So it is practical to use the in-situ test. A portable cone penetrometer is suitable to investigate the strength of soil in a slope. It is
made compact for the carrying convenience and is possible to interpenetrate by the human power. Because the depth of slip surfaces is very shallow. The relation between the strength of a weathered granite soil and the cone bearing capacity was investigated.

2. STRENGTH CHARACTERISTICS OF WEATHERED GRANITE SOILS

Weathered granite soils, so called Masado, are widely distributed in Japan. At these regions, many slope failures have occurred due to rainfall and have given much damage.

2.1 Grain size distribution and coefficient of permeability

In Fig. 4, an examples of case of grain size distribution of weathered granite soil is shown. In Fig 5, the coefficient of permeability of weathered granite soils is shown. It is clear from these figures that the weathered granite soil is a sandy soil and the coefficient of permeability is comparatively good, about $10^{-1}$ to $10^{-3}$ cm/sec.

2.2 Failure line

Failure lines obtained from the simple shear test on weathered granite soils in the saturated and unsaturated conditions are shown in Fig 6, respectively. It can be seen in the figure that the
angle of shearing resistance $\phi_s$ is not change with the saturation, while the cohesion $c_s$ drops to about zero. These characteristics of weathered granite soils are one of the causes that many slope failures happen by a heavy rain. Especially, the drops of the cohesion due to a seepage of rainwater is important. As the slope failure occurs at the rainfall the soil of a slope is in a saturated condition. In a saturated condition, $c_s$ is about zero, so $\phi_s$ must be obtained accurately for slope stability analysis. It is very difficult to prepare an undisturbed specimen of sandy soils such as Masado. To obtaining of strength parameters of Masado slope will require a great deal of labor. If the strength parameters of Masado slope can be obtained by a portable cone, it is very useful.

3. INFLUENCE OF PENETRATION RATE ON STATIC PENETRATION RESISTANCE

The strength parameters of Masado must be obtained on saturated conditions because the slope failure occur usually due to rainfall. It is thought that the penetration resistance is affected significantly by the penetration rate. Because, the effective stress changes by the pore pressure generated during a penetration. It was investigated the influence of the pore pressure during a penetration on static penetration resistance.

3.1 Model test on influence of pore pressure

Fig.7 shows the relationships of static penetration resistance $P_u$ and effective overburden pressure $\sigma_{ov}$ on the dense ground. of Toyoura sand, and penetration rate was 0.33 cm/min. The penetration resistance on saturated ground is obviously larger than one on dried ground when the ground is dense. But the relation is contrary when the ground is loose. These difference between dried ground and saturated ground is caused by the pore pressure generated around the cone tip during the penetration.

3.2 In-situ tests on influence of degree of saturation and penetration rate

The results of in-situ penetration tests on Masado ground are shown in Fig.8, 9. The water contents are shown in Fig.10. The water content is uniform with depth. Fig.8 and 9 show respectively the influence of a degree of saturation and a penetration rate on the penetration resistance $q_s$. The $q_s$ increase linearly with depth. This is mainly due to the increase of overburden pressure. From Fig.8, it is clear that the $q_s$ on natural and saturated conditions is shown with two parallel line. The decrease of $q_s$ on a saturated condition is considered to be due to the disappearance of c-component of strength of Masado. It is capable to obtain the relation between the $q_s$ on a saturated condition and $\phi$ of Masado. Because the $q_s$ on a saturated condition is mainly due to the $\phi$ component of Masado, as shown in Fig.8. Fig.9 shows that the influence of penetration rate is relatively small. It is the reason for this result that the pore pressure didn’t generate around the cone tip. The saturation of test field conducts by the flooding to slope surface. As a soil of a slope don’t completely saturate, about Sr=97%, the pore pressure during a penetration don’t generate.
4. **IN-SITU PENETRATION TESTS AT MASADO SLOPE**

4.1 Apparatus and test conditions
A cone tip with sectional area of 6.45 cm\(^2\) and a tip angle of 30°, as shown in Fig.11. A pore pressure during a penetration can be measured through the porous stone. The speed of cone penetration test is 1 cm/sec.

The static cone penetration tests were conducted at eight sites of Hiroshima prof. and Ehime pref. where many slope failures due to a heavy rainfall had happened every year.

4.2 Results of tests
An example of test results is shown in Fig.12. The cone resistance, \(q_c\), increase linearly with depth. The pore pressures measured during penetration tests were about zero. As the pore pressure don’t affect to the test results, \(q_c\) corresponds to the saturated-drained strength of Masado.

As the \(q_c\) increase linearly with depth, the relation between the inclination of \(q_c\) to a depth, \(\theta_q\) as shown in Fig.12, and the angle of the shear resistance of a weathered granite soil, \(\phi_\alpha\), was investigated.

The strength parameters of undisturbed weathered granite soils were obtained by triaxial and box shear tests. An example of failure lines obtained by the triaxial tests is shown in Fig.13. The cohesion of Masado was about zero at a saturated condition and the failure line is straight.

The relation between the inclination of \(q_c\) to a depth, \(\theta_q\), and the angle of the shear resistance of a weathered granite soil, \(\phi_\alpha\), is shown in Fig.14. It was cleared that the relation between \(\theta_q\) and \(\phi_\alpha\) was approximately linear.

The portable cone penetration test is useful to estimate \(\phi_\alpha\) of masado slope.
5. CONCLUSIONS
The following results are obtained.
1) The thickness of a sliding soil mass at Masado region is comparatively shallow, about 0.5~1.0 m. It is possible to interpenetrate the
portable cone by human power.
2) Cohesion c of a weathered granite soil is nearly zero at saturated condition. q_s at
saturated condition correspond to the strength
of saturated-drained condition.
3) The relation between the inclination of q_s
to a depth and the angle of the shear resistance
of a weathered granite soil is approximately
linear. Therefore, it can be obtained \( \phi_d \) of a
weathered granite soil slope from a portable
cone test.

Acknowledgments
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Boundary conditions and chamber size effects on CPT

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SYNOPSIS: This paper investigates the influence of chamber size as well as the boundary conditions on CPT in dense sand in the modified NGI calibration chamber. It quantifies the correction factors needed for cone resistance as well as sleeve friction values obtained from the tests performed under BC1 and BC3 for both NC and OC samples. This investigation gave the opportunity to examine the sleeve friction results in depth and quantify its importance in conjunction with cone resistance to determine the state of in-situ stresses in the soil (i.e. $K_0$ or OCR) from a single CPT. The results suggest that in other interpretations relative density has been under-predicted for both very dense sand and medium dense sands. This work is being continued further with the prospect of design of a smaller chamber which could encompass different testing conditions through replaceable compressible boundaries to simulate infinite conditions.

1. INTRODUCTION

Probably the most important assumption inherent in CPT testing is that the cone resistance value in the field is in the range BC1 to BC3 as measured in the CC, provided that the material is identical and at the same state of stress and overconsolidation. Calibration chamber tests are generally carried out in 1.2 m diameter chambers and, for the standard cone, the diameter ratio is about 34. From a series of penetration tests performed at Southampton using different size penetrometers (Last et al., 1987) as well as those by Parkin and Lunne (1982) using different chamber sizes, it was shown that for very dense Hokkaido sand, $q_c$ was still a function of chamber diameter. Penetration resistance in the field was believed to lie between the results for a rigid-wall chamber and that under a constant lateral stress boundary (BC1). The rigid-wall boundary overestimates $q_c$ as higher stresses exist at the radius of the chamber boundary than those found in the field. The results for both boundary conditions converge when the diameter ratio is greater than 50. Thus, the measured $q_c$ values for a given diameter ratio have so far been adjusted to reflect this ratio.

A great deal of testing have been performed under the condition of constant volume in the lateral boundary (BC1). One can argue that BC3, although controlling zero average boundary deformation where local expansion is accompanied by contraction elsewhere, is not truly a zero deformation test. It is now generally believed that BC3 should be avoided as its compatibility with actual in-situ penetration is not well defined.

Based on the work of Parkin and Lunne (1982) and Ghionna (1984), the boundary effects can be quantified and in most cases a modest (less than 20%) adjustment to the measured $q_c$ can satisfactorily reflect the field conditions. On the basis of a limited number of experimental data, Lunne and Christophersen (1983) suggest increases of 25% and 6% to the measured $q_c$ respectively, for BC1 and BC3 of NC sands and a 24% increase for OC samples under both boundary configurations. Baldi et
al. (1982) have suggested increases of 8% and 18% to the measured $q_c$ [presumably under BC1], respectively, for dense and very dense NC tests with as much as 39% and 67%, respectively, for dense and very dense OC tests. The variation seems to be around 20% for very dense normally consolidated BC1 test but it is scattered for OC tests.

2. CHAMBER MODIFICATIONS
To overcome the boundary condition effects discussed in the preceding section, the NGI chamber was modified to simulate an ideal infinite boundary (Zohrab, 1993). In this way, the imposed stresses from the soil would produce strains at chamber radius which would correspond to those that the same soil to infinity would undergo with the same stress at this radius. Since the strains required were greater than could be accommodated by a simple metal jacket it was decided to use a compressible layer rather than a stretched hoop. By running a series of expansion tests with very dense Hokkaido sand under a range of stress histories it was possible to pick out one NC and another OC for each OCR which required the same boundary stiffness. This stiffness was achieved by strips of rubber placed at pre-designed intervals on the lateral wall as well as the base and top boundaries for each of a test condition. The cone tip resistance and its sleeve friction profiles were then compared with those under BC1 and BC3.

3. ANALYSIS OF TEST RESULTS
The chosen stiffness corresponded to a NC test at $\sigma'_{vd} = 2.0$ kg/cm$^2$ and two OC tests: one at $\sigma'_{vd} = 1.0$ kg/cm$^2$ with OCR = 4 and another at $\sigma'_{vd} = 1.5$ kg/cm$^2$ with OCR = 3 for very-dense Hokkaido sand. For comparison purposes, some additional tests were performed under different stress levels and stress histories for both medium-dense and very-dense states. The test results are discussed in the following sections with reference to different soil parameters.

3.1 $K_o$ Conditions
Continuous measurements of both the horizontal and vertical stresses during consolidation gave reliable $K_o$ values. The results indicated that this can be estimated by a linear relationship for NC samples as

$$K_o^{NC} = 0.589 - 0.0271 \sqrt{D_r} \quad (R^2 = 0.94)$$  \hspace{1cm} (1)

which yields a $K_o$ value of 0.31 for a very dense sand (D_r = 100 %). The medium dense to very dense range gives the following for an OC sand

$$K_o^{OC} = 0.757K_o^{NC} + (0.13 + 0.00011D_r)OCR 0.733$$  \hspace{1cm} (R^2 = 0.96) \hspace{1cm} (2)

Another form of this relationship can also be obtained directly as

$$K_o^{OC} = 0.489OCR^{1.044}D_r^{-0.07} \quad (R^2 = 0.94)$$  \hspace{1cm} (3)

These deductions are, however, based on large scale calibration chamber testing. In practice, it is not convenient to obtain such parameters as OCR and D_r in advance. Therefore, it is desirable to correlate $K_o$ to the measured stresses, $q_c$ and $f_{es}$ during a CPT. This has been possible in this study since the obtained cone stresses are for simulated in-situ conditions. The following relationships describe the dependence of $K_o^{OC}$ to $q_c$ and $f_{es}$ only through $\sigma'_{vd}$, which can be estimated to a reasonable accuracy for an assumed soil density.

$$K_o^{OC} = 5.1655\sigma'_{vd}^{0.444}q_c^{-0.425}f_{es}^{-0.331}e^{0.263f_{es}/\sigma'_{vd}}$$  \hspace{1cm} (R^2 = 0.98) \hspace{1cm} (4.1)

[all stresses are in kgf/cm$^2$] This relationship can also be expressed in the following forms in which sleeve friction is normalised against vertical stress:

$$K_o^{OC} = 9.83\left(\frac{f_{es}}{\sigma'_{vd}}\right)^{-0.051}q_c^{-0.633}e^{0.406f_{es}/\sigma'_{vd}}$$  \hspace{1cm} (R^2 = 0.98) \hspace{1cm} (4.2)

$$K_o^{OC} = 13.452\left(\frac{f_{es}}{\sigma'_{vd}}\right)^{0.453}q_c^{-0.601}$$  \hspace{1cm} (R^2 = 0.96) \hspace{1cm} (4.3)

[all stresses are in kgf/cm$^2$] It may be noted that the accuracy of these equations is much higher than any of those obtained earlier. It can

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be used readily in the field to obtain the $K_0$ and, hence, the horizontal stress from a single cone penetration test.

Eqns (4.2) has been plotted in Fig. 1 for in-situ $K_0$ as a function of normalised sleeve friction for various $q_c$ values.

Fig. 1 In-situ $K_0$ relationship with measured cone stresses for Hokksund sand

In order to generalise the form of Eqns (4.1) to (4.3) to other sands, an analysis was made on the results of parallel CPT and DMT on Po river sand (reported by Marchetti, 1985) and the following non-dimensional relationship was obtained

$$K_0 = 0.31 \left( \frac{f_s}{\sigma'_{v}} \right)^{0.28} \left( \frac{q_c}{\sigma'_{v}} \right)^{0.23} e^{(0.23 f_s / \sigma'_{v})}$$

$$R^2 = 0.91$$

(5)

Note that this equation can determine $K_0$ within 3% of that estimated by Eqn. (4.1) for Hokksund sand, which may even be due to differences in the mechanical properties between the two sands.

3.2 Relative Density

The significance of the controlled stiffness boundaries can best be shown in the cone resistance and sleeve friction results. The new infinite boundary results of cone resistance appear to be over-estimated by the $q_c - \sigma'_{v}$ plots of the Schmertmann (1978) curves, which in turn are further over-estimated by correction procedures of Lunne and Christophersen (1983) and Lunne (1991). They suggested a 25% increase to these curves for field conditions [derived only for unaged, clean, fine to medium quartz sands with rather high lateral stress ($K_0 = 0.45$) as compared to maximum $K_0$ of 0.32 for Hokksund sand]. Hence, their method under-predicts relative density by 12 - 17% for the very dense sand and by 5% or less for medium dense sands. Using the charts given by Baldi et al. (1986), $D_r$ is under-predicted by 2 to 11% for very dense sand with no difference for the medium dense ones.

Based on limited test results on very dense samples, the effects of the boundary conditions on cone resistance and sleeve friction can be isolated through the following relationships:

$$q_c = 387.3 \sigma'_{h}^{0.657}$$

$$R^2 = 0.97$$

(6)

$$f_s = 0.821 \sigma'_{h}^{1.006}$$

$$R^2 = 0.99$$

(7)

$$f_t = (0.866 \sigma'_{h} - 0.053 \sigma'_{v}) e^{0.012(D_r - 60)}$$

$$R^2 = 0.99$$

(8)

These relationships can be combined in such a way that relative density can relate to only measured cone stresses. The following form is obtained from a regression performed on the stiffness boundary test results:

$$q_c = 121.43 \sigma'_{h}^{0.66} e^{0.006 D_r}$$

$$R^2 = 0.99$$

(9)

This equation has been plotted in Fig. 2 to show relative density as a function of in-situ cone resistance and sleeve friction values. The use of this plot for results of tests under the conventional boundary conditions [i.e., BC1 and BC3], will, however, require correction factors for both measured cone resistance and sleeve friction.

Eqns (6) and (9) show a rise of 10-20% to $q_c^{nc}$ under BC1 for $\sigma'_{v}$ ranging from 0.5 to 1.25 kg/cm², respectively, with virtually no correction for BC3. The $q_c^{nc}$ results, however, indicate a rise of 9% for a $\sigma'_{v}$ of 0.5 kg/cm² decaying down to zero for a lateral stress of 1.25 kg/cm².

The sleeve friction results also indicate increases to the measured value under conventional boundary conditions. For a typical 2.0 bar NC dense sand under BC1, the $f_s$ value should be increased by about 26%. The OC.
samples show increased values by at least 14%.

\[ \frac{M}{q_{c \text{in-situ}}} = 5.676 \sigma_v^{0.486} \text{ (R}^2 = 0.92) \] (12)

where sleeve friction is in kg/cm². In this way, soil deformation modulus can be readily obtained from a CPT result alone. This can be seen graphically in Fig. 4 where constrained modulus is a function of sleeve friction for selected \( q_c \) values of Hokkaido sand (HS).

\[ \ln M = 4.06 + 0.88 \sigma_v^{0.5} (2.1 - 0.6 \sigma_v) D_s \] (10)

where \( \sigma_v \) and \( M \) are in kg/cm² and \( D_s \) in %. This equation has been plotted in Fig. 3 for NC sands.

Since the penetration stresses are functions of both the vertical stress and relative density under a given OCR, it may be anticipated that a relationship exists between \( M \) and \( q_c \). The penetration results give the following relationship for NC sands:

\[ M = 90.685 q_c^{0.413} \sigma_v^{0.205} \] (11)

[all terms in kg/cm²]. The use of this equation in the field requires some knowledge of the stress history of the sand. This can be obtained by combining \( M = f(\sigma_m, \text{OCR}, D_s) \), as suggested by several researchers, with that of Eqn. (4) and performing regression to eliminate

3.3 Constrained Modulus

It is important to obtain soil deformation parameters from in-situ tests because of the sensitivity of deformation moduli to small disturbance. Hence, the constrained tangent modulus, \( M \), values obtained under simulated infinite boundary configurations should ideally resemble field conditions. The results indicate that this modulus is a function of both the applied stress and the overconsolidation ratio, OCR, as well as the relative density of the sand. The following relationship was obtained from the consolidation phase for NC sands:

3.4 Overconsolidation Ratio

Numerous studies have shown that cone resistance is only influenced by the current level of the in-situ horizontal stress; it remains insensitive to the effect of the accumulated plastic strain [being the case with the constrained modulus] which appears to be linked with preconsolidation mechanisms, i.e. ageing, cementation, and stress history induced
by earthquake, etc. However, overconsolidated conditions result in an increase in \( \sigma_v' \) for a given value of \( \sigma_v \).

The results of tests in the new stiffness boundary chamber show that OCR is only a function of the cone stresses through the vertical stress. It is believed that the effect on lateral stress is inherent in sleeve friction, as seen from Eqn. (7). Analysis of the data gives the following relationship for all densities:

\[
\text{OCR} = \left( \frac{q_c}{\sigma_v'} \right)^{0.19} \left( \frac{f_s}{f_5} \right) \quad (R^2 = 0.995)
\]  

(13)

This relationship is shown graphically in Fig. 5 for selected vertical stresses.

Fig. 5. OCR vs. \( \sigma_v' \) for in-situ \( q_c \) and \( f_5 \)

3.5 Shear Strength

The shear strength of cohesionless soils is normally related to the mobilised angle of internal friction, \( \phi' \). Due to the curvilinearity of the strength envelope in sands, the friction angle depends on the magnitude of the normal stress at failure. Hence, any inference of \( \phi' \) value from penetration results should correspond to the secant angle of friction controlled by average \( \sigma_v' \) acting around the cone tip.

Recently, Been et al. (1986) have used the state parameter concept to incorporate relative density and stress level in a single parameter, i.e. state parameter (\( \Psi \)). In this way, failure parameters including \( \phi' \) have been normalised against \( \Psi \) for several sands (Been and Jeffries, 1985). The \( \phi' \) values from drained triaxial tests of Kédalen et al. (1982) on Hokksund sand were plotted by Been et al. (1986) against \( \Psi \) and are seen to fit the following exponential relationship:

\[
\phi'' = 35.89 \exp(-0.932\Psi) \quad R^2 = 0.90
\]  

(14)

Based on the data supplied by Been and Jeffries (1985) on seven predominantly quartz sands, including Hokksund sand, a similar form of relationship, on the average, was derived as

\[
\phi'' = 32.06 \exp(-1.274\Psi) \quad R^2 = 0.998
\]  

(15)

The state parameter has also been related to normalised cone resistance for Hokksund sand:

\[
\frac{q_c - \sigma_m'}{\sigma_m'} = 26 \exp(-1.117\Psi)
\]  

(16)

The same for the seven sands is given by Been et al. (1987) as:

\[
\frac{q_c - \sigma_m'}{\sigma_m'} = 17 \exp(-10.5\Psi)
\]  

(17)

Eqns. (14) and (16) can be combined with the cone resistance - mean stress relationship developed for Hokksund sand from the stiffness boundary chamber test results as

\[
q_c = 17365 \sigma_m'^{0.467} f_s^{0.802} \sigma_v'^{0.312} \quad (R^2 = 0.998)
\]  

(18)

to give the following relationship for determining \( \phi' \) for Hokksund sand

\[
\phi'' = 45.81 \left( \frac{f_s}{\sigma_v'} \right)^{0.07} / q_c^{0.008}
\]  

(19)

If Eqn. (18) can be generalised for the predominantly quartz sands analysed by Been et al., then the shear strength for such sands can be approximated by using Eqns. (15) and (17) to give

\[
\phi'' = 48.25 \left( \frac{f_s}{\sigma_v'} \right)^{0.11} / q_c^{0.012}
\]  

(20)

Eqn. (20) appears to give results that are, on
the average, 2.5° higher than those obtained by Eqn. (19) for very dense sands with same values predicted for medium dense samples.

Eqn. (19) has been plotted in Fig. 6 which can be used in the field to determine \( \phi' \) for the measured values of cone resistance and sleeve friction and an estimated vertical stress.

![Fig. 6. \( \phi' \) vs. in-situ \( q_u \) and \( f_s \)](image)

4. CONCLUSIONS
This paper attempted to quantify the effects of boundary conditions and chamber size on both the cone resistance and the sleeve friction values obtained from conventional calibration chamber tests. On this basis, the following recommendations are made:

1. More importance should be given to sleeve friction results which can then be used in conjunction with cone resistance to determine soil parameters from a single test without due reference to any other in-situ tests.

2. Further tests are required in other sands to verify the validity of the relationships obtained for the tested sand.

3. Smaller scale chambers can be built which could encompass a greater range of stress levels and stress histories through easily replaceable flexible curtains.

4. This can give the opportunity to consider a group of sands and differentiate their main characteristics by the use of the CC based correlations in determining the soil parameters.

5. REFERENCES


Statistical Treatment of CPT Data

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SYNOPSIS: The paper reviews briefly how statistical approaches can be applied to interpret and present the results of cone penetration tests (CPT). The usefulness of the statistics is illustrated with an geostatistics application example from an actual case study. The experience with the methods suggests that when enough data are available, one should use a site description strategy including identification of the correlation structure and autocorrelation function of the soil property within geologic units, and stochastic interpolation technique (called kriging) to estimate the soil property at the location of interest. The added knowledge on the uncertainties in the soil parameters should lead to safer designs. Modern graphical representation also make the visualisation of the variability in the soil characteristics much easier.

1. INTRODUCTION  
Statistical approaches can be usefully applied to interpret and present the results of cone penetration tests (CPT). The paper briefly reviews existing approaches and summarises an example of geostatistics analysis that helped interpret CPT results and reduce the uncertainties in soil layering and input parameters required for stability analysis.

Reliability-based techniques are useful in geotechnical engineering as a complement to the more conventional deterministic analyses. Reliability approaches aim at accounting for the effects of uncertainties in soil properties and calculation models on the results of geotechnical analyses.

An important part of a reliability analysis is the statistical treatment of the data used to evaluate soil parameters. The statistical estimates give a mean value and an estimate of the uncertainty in the data. The statistical estimates should be combined with engineering judgement to select the parameters for design, and should always consider both the quality of data and the geologic evidence. Statistical methods are applicable within 'homogeneous' layers, and it is important to identify the main soil or geologic layers before performing statistics. However, statistics can be used to identify layer boundaries.

Statistical approaches are especially relevant for applications to in situ test results that generate a lot of data in parallel soundings, such as the cone penetration test.

Statistical methods belong to either traditional statistics or geostatistical approaches. Table 1 summarises the different types of statistical approaches that can be applied to CPT data. The list is not an exhaustive list of existing approaches, but a survey of techniques that are known to have had successful geotechnical applications. (If a soil parameter is obtained from a complex calculation with many random parameters, one should consider using simulation methods, with Monte-Carlo or Latin Hypercube sampling methods).

2. UNCERTAINTIES IN SOIL PROPERTIES  
Two types of uncertainties affect a soil property within a geologic layer: Aleatory (or inherent) uncertainty, which represents the natural randomness of a property. The variation in the ocean wave height and the variation in a soil property in the horizontal direction are aleatory uncertainties. This type of uncertainty cannot be reduced.
### Table 1: Statistical tools and their application to geotechnical analysis

<table>
<thead>
<tr>
<th>Method</th>
<th>Application</th>
<th>Recommendation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Short-cut estimates (Bacher, '85, Krumbein &amp; Grayhill, '65, Snedecor &amp; Cochran, '64)</td>
<td>-When little data are available -Obtains mean and variance, gives bound for standard deviation (multiplies range of values by factor &lt; 1 to account for lack of data points)</td>
<td>-Use to check variance when in doubt whether value is too low «Quick» method</td>
</tr>
<tr>
<td>Mean, variance, histograms and probability density (Ang &amp; Tang, '75)</td>
<td>-Best method when enough data available -Distribution function obtained from plotting on probability paper or goodness-of-fit tests</td>
<td>-Use whenever possible</td>
</tr>
<tr>
<td>Cluster/grouping methods (Mynarski &amp; Lunne, '87)</td>
<td>-Study of local variations from distance between data points</td>
<td>-Use rather stochastic interpolation</td>
</tr>
<tr>
<td>Geostatistics with stochastic interpolation (Matheron, '63, Nadim, '88)</td>
<td>-To do statistical site description -Software adapted to geotechnical parameters exists -Important advantage in data presentation</td>
<td>-Apply if enough data -Applicable to all properties (also depth, layer thickness)</td>
</tr>
</tbody>
</table>

### Table 2: Definitions of terms used in statistical analysis

<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Autocorrelation distance</td>
<td>Distance beyond which there is no correlation between two values of a random variable</td>
</tr>
<tr>
<td>Autocorrelation function</td>
<td>Function describing the correlation of the residuals about a trend</td>
</tr>
<tr>
<td>Coefficient of variation</td>
<td>Ratio of standard deviation to mean</td>
</tr>
<tr>
<td>Correlation</td>
<td>Mutual dependency between two or more variables</td>
</tr>
<tr>
<td>Histogram</td>
<td>Graphical representation of a range of measured or observed values and of how frequently these values occur (also called frequency diagram)</td>
</tr>
<tr>
<td>Linear regression</td>
<td>Linear relation between two random variables expressed in terms of mean and variance of one random variable as a function of the other variable</td>
</tr>
<tr>
<td>Mean</td>
<td>Measure of the most likely value of a random variable (also called average)</td>
</tr>
<tr>
<td>Median</td>
<td>Value of a variable at which values above and below it are equally probable</td>
</tr>
<tr>
<td>Population</td>
<td>Set of data points considered</td>
</tr>
<tr>
<td>Probability distribution</td>
<td>Law for describing the probability associated with each of the values of a random variable</td>
</tr>
<tr>
<td>Random process</td>
<td>Process associated with the numerical outcome of random variable(s)</td>
</tr>
<tr>
<td>Random variable</td>
<td>Variable which exhibits scatter or dispersion and which value cannot be predicted with certainty. For each outcome of a random variable is associated a numerical value</td>
</tr>
<tr>
<td>Residuals</td>
<td>Algebraic measure of distance between a data point and the trend</td>
</tr>
<tr>
<td>Scale of fluctuation</td>
<td>Distance within which a soil property shows relatively strong correlation between two values of a random variable</td>
</tr>
<tr>
<td>Standard deviation</td>
<td>Measure of dispersion or variability of a random variable and of the closeness of the values; the standard deviation is the square root of the variance</td>
</tr>
<tr>
<td>Trend</td>
<td>Direction or tendency of a pair of variables (often the slope of a function)</td>
</tr>
<tr>
<td>Variance</td>
<td>Measure of dispersion or variability of a random variable, and of the closeness of the values; dispersion is taken with respect to the mean value</td>
</tr>
</tbody>
</table>
Epistemic uncertainty, which represents the uncertainty due to lack of knowledge for a given property. Measurement uncertainty and model uncertainty are epistemic uncertainties. This type of uncertainty can be reduced, by for example increasing the number of tests, improving the measurement method or evaluating a calculation procedure with model tests.

For any extensive volume of natural soil layer, the characteristics fluctuate spatially. The soil properties are then considered to be controlled by a random process, with a random pattern of variation and at times an overall trend. The variability may be small or large. There is a greater tendency for the properties to be similar in value at closely neighbouring points than at widely spaced points. Soil properties are expected to show dependence both laterally and with depth.

The spatial variability of soil properties in situ within a geologic layer is often modelled by trend surfaces and residual variations about the trend. With the advent of rapid computers, these aspects can now be routinely accounted for.

3. TERMS FROM STATISTICAL ANALYSIS
Because some of the terminology may be new to the reader, a few terms used in statistical analysis are defined in Table 2.

4. STATISTICAL TREATMENT
4.1 Short-cut estimates
The mean is obtained by averaging all available data. For an approximate estimate of the variance with only a limited number of data, one can use short-cut estimates, as suggested by Snedecor and Cochran (1964). The standard deviation is obtained by multiplying the range of the available values by weighting factors shown in Table 3. The range is defined as the difference between the largest and smallest values in the data population. For example, for five data points, the standard deviation would be the range multiplied by 0.43. The approach is a good estimator for symmetric data populations.

<table>
<thead>
<tr>
<th># points</th>
<th>Weighting factor</th>
<th># points</th>
<th>Weighting factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>11</td>
<td>0.315</td>
</tr>
<tr>
<td>2</td>
<td>0.886</td>
<td>12</td>
<td>0.307</td>
</tr>
<tr>
<td>3</td>
<td>0.591</td>
<td>13</td>
<td>0.300</td>
</tr>
<tr>
<td>4</td>
<td>0.486</td>
<td>14</td>
<td>0.294</td>
</tr>
<tr>
<td>5</td>
<td>0.430</td>
<td>15</td>
<td>0.288</td>
</tr>
<tr>
<td>6</td>
<td>0.395</td>
<td>16</td>
<td>0.283</td>
</tr>
<tr>
<td>7</td>
<td>0.370</td>
<td>17</td>
<td>0.279</td>
</tr>
<tr>
<td>8</td>
<td>0.351</td>
<td>18</td>
<td>0.275</td>
</tr>
<tr>
<td>9</td>
<td>0.337</td>
<td>19</td>
<td>0.271</td>
</tr>
<tr>
<td>10</td>
<td>0.325</td>
<td>20</td>
<td>0.268</td>
</tr>
</tbody>
</table>

4.2 Mean, variance and histogram
Conventional statistics include developing a histogram and obtaining mean and coefficient of variation. When enough data are available, the probability distribution function should be established. In geotechnics the normal and lognormal distributions are the most commonly used.

It is often the relation between two parameters which is of relevance for a calculation, for example correlation between corrected cone penetration resistance and undrained shear strength from triaxial compression tests via a cone factor, or variation of undrained shear strength with depth. Ang and Tang (1975) described in detail the mechanics of linear regression analysis. The goodness of fit is given by how close the correlation coefficient is to unity.

Estimates of cone penetration parameters have also been obtained with filtering and smoothing techniques (Vivatrath, 1978; Harder and von Bloh, 1988) to obtain an unbiased average representation.

Mortensen et al (1991), in a careful study of the correlation of cone penetration and field vane test for clay tills, also used a smoothing of the in situ cone resistance curves and obtained a frequency histogram for cone resistance and for the cone factor correlating cone resistance and vane shear strength. The quality of the correlation can (and should be) corrected by setting 'quality criteria', for example by rejecting anomalous data points or data that do
not fit in the specified criteria. (For example, Mortensen et al. (1991) set the criterion of friction ratio between 1.0 and 6.0 % otherwise the data were disregarded.)

4.3 Autocorrelation

The variation of a soil property in space is illustrated in Fig. 1, as a function of a trend, \( T(x) \), and residuals, \( \varepsilon(x) \) (Vanmarcke, 1977, 1984, De Groot and Baecher, 1993). The residuals over a large volume of soil are assumed to have zero mean. The trend function is obtained by regression analysis, the residuals are correlated, unless the data are very widely spaced.

The spatial variation of a soil property can be modelled as the sum of a trend component and a residual term:

\[
Y(x) = T(x) + \varepsilon(x)
\]

where \( Y(x) \) = measurement at location \( X \)
\( T(x) \) = trend component
\( \varepsilon(x) \) = residual (deviation about trend)

The properties of an extensive volume of natural soil layer inevitably fluctuate. Soil properties tend to exhibit a strong spatial correlation structure, which appears as a waviness about the trend. The properties tend to be more similar in value at closely spaced points than at widely spaced points. The larger the width, length or depth over which a parameter is averaged, the more the fluctuations tend to cancel each other in the process of “spatial averaging.” This correlation structure can be important both for improving estimates at unsampled locations and for assessing the reliability of a soil parameter.

The degree of spatial correlation can be expressed through an autocovariance function, \( C(r) \), where \( r \) is the vector of separation distance between two points. The normalised form of the autocovariance function \( \{C(r)/C(0)\} \) is known as the autocorrelation function, where \( C(0) \) is the autocovariance function at a distance \( r = 0 \).

The three autocovariance functions best suited for soil properties are shown in Fig. 2. In the exponential models, the distance at which the autocovariance function \( C(r) \) decays to a value of 1/e (where \( e \) is the base of the natural logarithm) is called the autocovariance distance, \( \tau_0 \). This characteristic length describes the extent of the spatial correlation. Figure 3 illustrates how the autocovariance distance and the variance influence the fluctuation of a soil property (De Groot and Baecher, 1993).

The uncertainties associated with soil characteristics are generally attributed to two primary sources: (1) inherent (natural soil) variability, and (2) sampling and testing errors (identified as “noise”). The soil variability component and noise are separated with the use of an autocorrelation function. Nadim (1988) and Keaveny et al. (1989) derived autocorrelation functions in the vertical and horizontal directions for various sets of in situ and laboratory data.

The autocorrelation is the basic reason for a reduction in the scatter in the data (standard deviation) as the averaging dimension decreases. Figure 4 gives an example of the autocorrelation structure of cone penetration data at a depth of 9 m in a dense sand. The scale of fluctuation, which is a function of the shape of the autocorrelation function, is directly related to the autocorrelation distance, \( \tau_0 \), and represents the distance over which the soil property shows strong correlation.

In the example in Fig. 4, the autocorrelation function had the form:

\[
C(r) = 0.986 e^{-0.375 r^2}
\]

where \( r \) and 37.5 are distances in meters. The closeness of the factor 0.986 to unity indicates that there was little measurement noise in the recorded data.

4.4 Geostatistics

Properly accounting for the variability in soil properties when predicting geotechnical performance may reduce substantially the uncertainties in the soil parameters and therefore the uncertainties associated with a design. Unfortunately, one is never able to gather enough subsurface data to get an exact picture of the variation of a soil property for an engineering structure. One must therefore
Fig. 1  Spatial variability of soil property

Fig. 2  Autocovariance functions
(a) exponential
(b) Squared exponential
(c) Spherical

Fig. 3  Influence of variance and autocovariance distance on variation of soil property

Fig. 4  Autocorrelation structure of 17 CPT data points at depth 9 m in dense sand (Keaveny et al., 1989)

Fig. 5  Location of cone penetration tests at location of example calculation (Coordinates are in metres)
interpolate the soil properties within a large volume, and sometimes one needs to extrapolate.

Traditional methods of interpolation used in geotechnical engineering give little regard to the uncertainties associated with soil properties and the fact that soil properties tend to exhibit a spatial correlation structure. A stochastic interpolation technique that is well suited for this is the ‘kriging’ approach (Matheron, 1963). Kriging is a stochastic interpolation method that can account for the uncertainties associated with the soil properties and minimise the variance in the observed data. To do the kriging interpolation, one needs to first identify the spatial structure of the soil characteristics or the autocorrelation function. The analysis method is described in Nadim (1988).

4.5 Example

Geostatistical analysis is illustrated with an example. In the neighbourhood of a shallow foundation, several cone penetration tests were run. Figure 5 illustrates the locations of available soundings from the soil investigation, and Fig. 6 gives example of some of the cone penetration test results (cone resistance q versus depth.)

The soil profile consists of a top sand (7-10 m) over a relatively weaker clay, partly laminated, with varying shear strength. In this case, the location of the weaker clay layer, and the undrained shear strength for the weaker material were the main variables that conditioned the feasibility of the foundation. Using minimum values would result in large added costs.

Figure 7 presents the contours of cone penetration resistance at a depth of 9 m, for the soundings closest to the foundation. Although the scatter in the data is large, which is to be expected in a layer where both sand and clay can be intermixed, the mean determined by kriging is significantly higher (46 MPa) than what had originally been assumed in design at that depth (about 1.5 to 2 MPa) based on the cone penetration profiles similar to those in Fig. 6. The 3-D plot also shows that there does not seem to be a continuous layer with weaker shear strength directly beneath the structure.

At this depth, the best fit autocorrelation function had the following exponential form:

\[ C(r) = 1.0 e^{-r/9.6} \]

where \( r \) and 9.6 are distances in meters. The factor of unity indicates no noise in the measurements. However the coefficient of variation for the data was about 50%.

Figure 8 gives a graphical representation at each meter between 7 and 12 m. It is naturally possible to make statistical analyses at as many depths as required for a problem. Calculation and plotting are done rapidly on PC with computer programs such as developed at NGI by Nadim (1988). By illustrating the spatial distribution in three dimensions of a soil property, one obtains much better insight into the possible variation in the cone resistance and the most likely value beneath the foundation.

In this particular case study, the geostatistical analysis enabled the designers to adjust the assumed position of the clay layer below the depth that had originally been assumed and to use slightly higher cone penetration resistance values, and therefore higher shear strength in design.

4.6 Autocorrelation distances

Several workers studied the spatial autocorrelation structure of cone resistances. A brief review suggests the autocorrelation instances in Table 4.

4.7 Reduction of variance to account for spatial variability

Figure 9 illustrates how one obtains the reduction factor to account for spatial averaging. The reduction factor is obtained from the square root of the autocovariance function. Within the scale of fluctuation, the reduction in variance can by a factor as much as 0.4 to 0.8. Vanmarcke (1984) suggested that the autovariance for most autocorrelations functions used in geotechnical engineering could be approximated by a unique curve, which results in a simple relation between reduction factor and distance over which the soil parameter is averaged.
Table 3: Autocorrelation distance for cone penetration resistance

<table>
<thead>
<tr>
<th>Soil</th>
<th>Direction</th>
<th>Autocorrelation distance (m)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Offshore soils</td>
<td>Horizontal</td>
<td>30</td>
<td>Hoeg and Tang (1976); Tang (1979)</td>
</tr>
<tr>
<td>Silty clay</td>
<td>Horizontal</td>
<td>5-12</td>
<td>Present study</td>
</tr>
<tr>
<td>Clean sand</td>
<td>Vertical</td>
<td>3</td>
<td>Alonzo and Krizek (1975)</td>
</tr>
<tr>
<td>Mexico clay</td>
<td>Vertical</td>
<td>1</td>
<td>Alonzo and Krizek (1975)</td>
</tr>
<tr>
<td>Clay</td>
<td>Vertical</td>
<td>1</td>
<td>Vanmarcke (1977)</td>
</tr>
<tr>
<td>Silty clay</td>
<td>Vertical</td>
<td>1</td>
<td>Present study</td>
</tr>
</tbody>
</table>

5. SUMMARY
Basic statistics represent a useful means to establish mean and variance. In geotechnical problems that involve large soil volumes, the "spatially averaged" properties govern foundation analysis, as local fluctuations average over a large soil volume.

One should use a site description strategy including (1) identification of the correlation structure and autocorrelation function of the soil property within geologic units, and (2) use of the stochastic interpolation technique to estimate the soil property at the location of interest. The added knowledge on the uncertainties in the soil parameters should lead to safer designs. In addition, modern graphical representation make the visualisation of the variability much easier.

The advantage of the kriging interpolation technique is that the geotechnical parameters
Fig. 7. Stochastic interpolation of cone resistance at depth 9 m in 2- and 3-D (Coordinates are in metres)

Fig. 8. Variability of cone resistance between depths 7 and 12 m (Coordinates are in metres)

Fig. 9. Reduction of standard deviation and variance by spatial averaging
for analysis are more clearly defined with respect to their mean and variance.

Geostatistics provide useful complementary information to help guide on the interpretation of soil profiles, stratigraphy and horizontal and vertical variability in soil characteristics. The analysis will also give an estimate of the noise in the measurements and help discern actual in situ trends from anomalies. The approach is well suited for the interpretation of CPT data since a lot of data points are generated, and hopefully it will become more usual to run a large quantity of profilings.

The application of geostatistics requires that a large quantity of data be available for generating the autocorrelation function and obtaining reliable mean and standard deviation or variance values. Statistical methods can also be used to plan site investigation programmes and for optimising the location and number of additional tests.

6. REFERENCES


Session 3 - Solution of Practical Problems
Use of CPT instead of traditional Swedish sounding methods

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SYNOPSIS: The CPT test is becoming increasingly acknowledged as a more rational and reliable site investigation method compared to many traditional Swedish methods. Its usefulness has been shown at a number of different sites, not only for evaluation of the boundaries and character of different soil layers but also in providing data for estimating settlements. In the paper, two cases are presented where both traditional weight sounding tests and Swedish dynamic probing tests (HFA) as well as CPT tests have been performed in soil profiles consisting mainly of sands but also with layers of silt and clay. These cases show the usefulness of the CPT test as a method for detection and characterisation of layers of weaker soils embedded in stiffer soils and for estimating overconsolidation ratios and compression characteristics in the assessment of settlements.

1. INTRODUCTION
The introduction of new methods of site investigation is often a slow process. Well established traditional methods have the advantage of enabling the use of accumulated experience in the interpretation of the results. The Swedish weight sounding test is such a traditional testing method which is used for investigating the thickness and relative stiffness of different layers in soil profiles. It has been extensively used since about 1916 in applications such as investigations for designing new roads. Also the Swedish version of the dynamic probing test (HFA) is very commonly used.

However, once employed in site investigations, the CPT test normally proves itself more useful in several respects, leading to increased use in preference to traditional investigation methods.

2. HYBO
Along the railway between Bollnäs and Ljusdal, south-west of Sundsvall, the bridge at Hybo log chute was to be rebuilt. The three span bridge founded on wooden piles should be replaced by a railway embankment and a bridge with a smaller span. For this design to be feasible, the size of the settlements was of crucial importance.

To enable an estimation of the settlements, it was decided to use CPT tests and dilatometer tests in the investigations for the new bridge, in addition to earlier weight sounding tests and Swedish dynamic probing tests (HFA) (Andersson et al., 1990, Andersson & Moller, 1991).

2.1 Geotechnical conditions
The soil consists of loose sand or silty sand to about 8 m depth. The soil changes to a silt at about 8 - 12 m depth. Beneath is a layer of clayey gyttja and clay, 4 - 5 m thick. The undrained shear strength of the clay has been es-
timed at 30 - 40 kPa. The soundings stopped at about 18 m depth.

2.2 Field investigations
The field investigations were carried out in three stages. In the early design stages, four weight sounding tests, dynamic probing tests (HfA) and disturbed sampling were performed. Later on, for estimation of the settlements, as well as to obtain a better picture of the stratification of the soil, it was decided to complement the investigations with CPT tests at eight points and dilatometer tests at two points. Due to the results of these tests, undisturbed sampling was also performed.

2.3 Results from field tests
The weight sounding tests and the dynamic probing tests indicated loose, cohesionless soil whose stiffness increases with depth. They showed no indication of a layer of clay. Figure 1 shows results from weight sounding tests, dynamic probing tests and sampling.

An interpretation of the soil layers from the CPT tests and the dilatometer tests, on the other hand, showed very similar results. Both methods indicated sand at the top, which was confirmed by the disturbed sampling. Also the change to silt fraction at 8 - 12 m depth was registered by the two methods, as well as the layer of clay and gyttja. The undisturbed sampling later confirmed the results. Figure 2 shows results from CPT tests, dilatometer tests and sampling.

2.4 Settlements
To ensure that no settlements should occur after the completion of the railway embankment, it was decided to use pre-loading in combination with expanded polystyrene.

Based on the CPT tests, calculation of settlements was carried out according to Schnurmann (1970) and DeBeer (1965). Two methods commonly used for footings on sand at normal foundation conditions. The results indicated settlements of 15 - 20 cm. Calculations based on dilatometer results indicated settlements of about 15 cm. However, the settlements measured during the six month pre-loading period were not more than 5 cm. A factor that may have influenced the size of the settlements is the existing wooden piles for the bridge.
3. SUNDSVALL

In the construction of the new stretch of the E4 north from Sundsvall on the Swedish east coast, comprehensive field investigations were made in connection with building the foundation of one of the bridges over the Indalsälven river delta. The bridge was built on eight piers and soil improvement was carried out by vibro compaction to about 12 metres depth around each pier.

The results from the field investigations and the analyses of compaction effects have been reported by Möller & Åhros (1991).

3.1 Geotechnical conditions

The soil profile consists of delta sediments to about 35 m depth below the ground surface. The uppermost 10 - 15 m of these sediments consists of mainly coarse delta sand. Beneath this layer, the grain size decreases with increasing depth in layers of mainly silt and clay. Lenses of silt and clay, with occasional inclusions of organic matter, are also found embedded in the upper delta sand. The sediments lie on a firm bottom of non-cohesive soil i.e. till.

Figure 3 shows a section of the soil profile determined by sampling carried out in the design stage.

3.2 Field investigations

In the design stage, a dynamic probing test, a weight sounding test and sampling to 20 or 40 m depth were performed in each pier area. At one of the abutments, further sampling and weight sounding tests were performed.

Control of the compaction effects was in
tended to comprise two weight sounding tests and two CPT tests in each compacted area. These CPT tests were to be carried out without measurements of pore pressure. However, since this was one of the first projects in Sweden where vibrocompaction was to be used for soil improvement, the testing and control program was enlarged to comprise piezocone tests, dilatometer tests and further sampling for investigation of different effects of vibro compaction and evaluation of the control methods. In this context, it was decided that also the CPT control tests should be performed as piezocone tests.

3.3 Results from CPT tests
In control of the first compacted areas, the CPT tests proved to be faster and gave better information on the compaction effects than the weight sounding tests. The remaining weight sounding tests were therefore replaced by an equal number of CPT tests.

The CPT tests showed that rather thick lenses of fine-grained soil occur to a varying extent within the compacted areas. Figure 4 shows the measured cone resistances before and after compaction at pier No. 4. Considerable increase in cone resistances because of the compaction, could be observed in the sand layers but not in the silt layers. The influence of a wedge-shaped silt lens can be clearly seen. In this layer, the effect of compaction was small and noticeable only in the pore pressure measurements, where lower and sometimes negative values of generated pore pressures indicate a densification of the silt and a tendency to dilate.

3.4 Settlements
The requirements for compaction were initially set as demands for certain minimum resistances.
to be registered in CPT tests and weight sounding tests. The use of piezocone tests, enabling an estimate of settlements, changed the criteria to requirements for calculated settlements based on the test results in the compaction areas.

Calculations of settlements were carried out according to Schmertmann (1970). Figure 5 shows results from the calculations carried out in connection to the compaction control compared to settlements measured one year after construction of the pier. The results show reasonable agreement for most of the piers. However, considerable differential settlements between the upstream part of the pier and the downstream part can be observed for three of the piers, among them pier No. 4. Figure 6. The differential settlements can be related to the varying thickness of the wedge shaped silt lens in the sand which can be observed in Figure 4. At pier No. 5, considerably larger settlements occurred than those predicted from the CPT.
In the compacted areas, the CPT tests were performed faster than corresponding traditional tests, thus also reducing the cost of the compaction control.

5. REFERENCES

4. CONCLUSIONS
The presented case histories showed that the CPT test offers more certain detection of layers of weaker soil embedded in stiffer soil, compared to the traditional Swedish methods of weight sounding and dynamic probing. In addition, the CPT tests render valuable information on stiffness parameters, often enabling a fairly good estimate of settlements.
Environmental Site Characterization in the United States Using the Cone Penetrometer

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SYNOPSIS: Remediation of environmentally contaminated soils is a high priority in the United States. Before any remediation can occur, the extent of this subsurface contamination must first be determined. This is called Environmental Site Characterization. Environmental site characterization includes determining subsurface information such as soil stratigraphy, permeability, the type of contamination present, and the concentration of the contaminants. The Cone Penetrometer, and other tools pushed by cone penetrometer hydraulic systems, are rapidly becoming the preferred tools for site characterization in the U.S.

1. INTRODUCTION
Environmental Site Characterization can be accomplished by drilling and establishing monitoring wells, or by pushing various tools into the ground with heavy weight cone penetrometer hydraulic systems. Drilling is time consuming, leaves a substantial hole in the ground, generates large volumes of contaminated cuttings and ground water, and requires extensive procedures to ensure that the hole is properly closed after the test is completed. Conversely, pushing tools such as the cone penetrometer, is quick, leaves a very small hole in the ground, generates no contaminated cuttings or ground water, and is usually easy to ensure that the hole is properly closed. Also, performing some of these tests in situ, instead of in the laboratory eliminates handling of the contaminants and provides data that may be more representative of actual conditions.

2. SOIL STRATIGRAPHY
The first step in determining the extent and motion of contaminants is to determine the subsurface stratigraphy. Since the contaminants will flow through the ground water, it is impossible to characterize an environmental site without a valid stratigraphic profile. The cone penetrometer, using tip, friction, and pore pressure measurements, has been used as a stratigraphic tool for many years. The pore pressure channel of the cone can also be used to approximate the distance to the water table or to locate perched water zones.

When attempting to retrieve a soil gas or water sample, it is advantageous to know where the bearing zones (permeable zones) are located. Although soil gas and water can be retrieved from non-bearing zones such as clays, it would be quicker to retrieve these samples from bearing zones, such as sands. The cone penetrometer test can be used to accurately identify and locate these bearing zones.

3. SOIL PERMEABILITY
The cone penetrometer also can be used to determine the soil permeability (usually called “hydraulic conductivity” in the environmental industry.) This can be accomplished by measuring the time required for excess pore pressure to dissipate. Knowing the permeability of the soil is vital in modeling contaminant flow.

In general, since the ground water flows primarily through sands and not clays, modeling the flow through the sands is most critical.
A pore pressure decay in a sand is almost instantaneous and therefore, very difficult to determine with a cone penetrometer. As a result, the cone penetrometer is seldom used for determining permeability in environmental applications.

A thorough study of ground water flow also includes determining where the water cannot flow. Cone penetrometer pore pressure dissipation tests can be used to study the permeability of aquitards (non-bearing soils such as clays). This is most often used where a soil is semi-permeable.

The permeability of sands can be determined by utilizing cone penetrometer hydraulic systems. For example, a miniature monitoring well (Figure 1) can be installed by pushing a PVC slotted well screen into the ground. These mini-wells can be installed very quickly to substantial depths. The PVC pipe and well screen is protected on the way down by an outer steel casing. Once the appropriate depth is obtained, the outer steel casing is withdrawn and the PVC casing is left in the ground.

![Figure 1](image1.png)

**Figure 1**
Installation of PVC Mini-well

A common in situ pump test or falling head test can now be easily run to determine permeability. Since the PVC has been left in place, subsequent tests or water samples can be taken later. This type of subsequent testing is also useful during remediation since the type and concentration of contaminant may significantly alter the permeability and ground water flow.

4. **CONTAMINANT TYPE AND CONCENTRATION**

Identifying the contaminant type and concentration is also a very important aspect of environmental site characterization. Even though the United States appears to be extremely interested in developing sensors that quickly and accurately identify contaminants and concentration levels, in practice, very few of these sensors are in actual use. Soil and water sampling methods are almost always used for determination of contaminant types and concentrations.

4.1 **Sensors**

The most common additional sensor used with a cone penetrometer is the conductivity (or electrical resistivity) measurement. (Figure 2)

![Figure 2](image2.png)

**Figure 2**
Cone Penetrometer with conductivity module

Although conductivity alone cannot identify a contaminant or quantify contaminant concentrations, it can be very useful in mapping a plume of a known contaminant such as salts, acids, or bases. Since conductivity is a function of concentration, estimates of concentrations can be determined.

The second most common sensor used with the cone penetrometer is the Laser Induced Fluorescent (LIF) system. Since various hydrocarbons fluoresce at different wavelengths, the LIF system has enjoyed significant excitement in the environmental field. A laser is used to generate a pulse of a known wave-
length down a fiber, through a window, and into the soil. The fluorescence of a different wavelength is received through the same window, transmitted up another fiber and is detected by a photo multiplier tube or optical multi-channel analyzer. The intensity of the fluorescence is a function of concentration. The behavior of the fluorescence, analyzed over time and wavelengths is a function of the particular type of contaminants. The results of this test can be graphically illustrated as shown in Figure 3. This method of identifying and quantifying hydrocarbons in the ground water is being utilized extensively in the U.S.

4.2 Sampling

Since few sensors are commercially practical in the cone penetrometer, sampling the soil gas, ground water, or soil is still the most common method of identifying and quantifying contaminants. Cone penetrometer hydraulic systems are excellent delivery vehicles to deploy these samplers. The cone penetrometer hydraulic system is so widely used for environmental investigations that the acronym, CPT, for Cone Penetrometer Testing, is often replaced by DPT, Direct Push Technologies.

4.2.1 Soil Gas Samples

Even though most contaminants eventually flow down through the soil and into the ground water, a soil gas sample can still be useful since many contaminants are volatile. If the ground water is deep, or if the tool delivery vehicle is light and cannot push very deep, a soil gas sample will still be representative of a volatile contaminant in the ground water. Soil gas sampling is very simple, requires no sophisticated equipment, and is therefore, very inexpensive.

Soil gas sampling is a rapid way to screen a site and can be performed in real time with a cone penetrometer test with a gas sampling friction reducer. (Figure 4) Since the sample lines may have “memory” the test can be stopped on demand, the lines purged, and a representative sample retrieved. The cone penetrometer, being ahead of the gas sampling ports, gives the operator a prior indication when a gas bearing zone will occur.

![Figure 3](image)

**Figure 3**

LIF data of Intensify vs. time and wavelength

![Figure 4](image)

**Figure 4**

Soil Gas Sampling Friction Reducer

4.2.2 Ground Water Samples

Most quantitative environmental analysis is performed by analyzing ground water samples. The most common “Direct Push” method of retrieving ground water samples is by using batch samplers (Figure 5) and a bailer. A batch sampler is a protected, sealed screen that is pushed into place with a cone penetrometer hydraulic system. Once the sampler is at the appropriate depth, the push rods are retracted enough to expose the screen. The water is then
retrieved with a conventional bailer. After the sample is recovered, the entire tool can be retrieved, decontaminated, and reused.

Figure 5
Batch Water Sampler

The open bailer does have limitations, though. The open bailer vents the sample and contaminant to the atmosphere. This is particularly true with volatile samples retrieved from deep depths. In this case, a closed system that retrieves a pressurized sample, such as the Swedish BAT® system, provides a more representative sample.

4.2.3 Soil Samples
Contaminants can exist in thirteen different states in the ground including vapor, various dissolved states, aqueous phases, non-aqueous phases, suspensions, absorption, adsorption, etc. For this reason, water samples alone may not be adequate to fully characterize the soil conditions. However, soil samples (Figure 6) also can be taken with cone penetrometer hydraulic systems.

Soil samplers are deployed similarly to the water samplers described in section 4.2.2. They are fully sealed until opened at the desired depth. They are advanced to fill the sample chamber and then withdrawn from the ground to retrieve the sample. This operation can be repeated several times in the same test hole with only decontamination of the tool performed between samples.

5. FIELD LABORATORIES
When an environmental site investigation begins, engineers can only guess where and how deep to obtain the first samples. With conventional drill methods, these samples are sent to the laboratories and it is typically weeks before the results are obtained and returned to the engineers. By this time, the drill crew has normally left the job site. Typically there are time delays before the drill crew can return to the job site to attempt a second round of sampling. As a result, the process is time consuming and many drilled monitoring wells are totally unnecessary.

Since many cone penetrometer systems are deployed from enclosed, air conditioned, and heated trucks, they also can be used as a mobile laboratory. This unique capability provides instant on-site analysis. The cone penetrometer data eliminates most guess work in determining where to obtain the first samples. The on-site laboratory analysis can provide important information such as where to obtain the next samples, and avoids most of the unnecessary sampling. On-site laboratory instruments range from simple portable devices to sophisticated gas chromatographs and mass spectrometers, GC-MS. This is often referred to as Expedited Site Characterization.
6. DECONTAMINATION
Generally tools must be cleaned after each use to prevent cross-contamination from one hole to the next and reduce exposure to the operators. Cleaning drill tools, such as large augers, is very time consuming and requires a large decontaminating area. Cleaning cone penetrometer related tools and push rods can be performed quickly with little extra space required. Some CPT systems have automatic “decon” chambers (Figure 7) under the truck that steam cleans the push rods while they are retracted from the ground and before they enter the vehicle.

![Figure 7](image)

**Figure 7**
**Decon Assembly**

7. TEST HOLE CLOSURE
Holes placed in the ground for environmental investigation purposes will usually require proper closing. This is to prevent compounding the problem by allowing contaminants of one aquifer to flow to an uncontaminated aquifer. Grouting a 50 foot (15M) drilled monitoring well could require up to 300 gallons (1140L) of grout. Conversely, grouting a 50 foot (15M) cone penetrometer hole may only require 5 gallons (19L) of grout.

8. WASTE
Drilling environmental monitoring wells generates waste. If the waste is contaminated, it must be properly disposed of by packing it in 55 gallon (208L) containers and paying a disposal company to remove it. Since the laboratory analysis of the samples lags by several weeks, most waste is automatically considered to be contaminated. The monitoring wells also hold large quantities of water. To retrieve a representative water sample, the well must be purged several times before taking the sample. This purge water is also a contaminated waste and also must be disposed properly. Depending on the contaminant present, the waste disposal may cost more than the drilling of the well.

However, since cone penetrometers and related tools are pushed instead of drilled, no waste is generated. Since a water sampling tool would be empty before retrieving the first sample, no purging water is required. Thus, the cost of waste disposal is significantly reduced.

9. REGULATIONS
The United States Environmental Protection Agency (EPA) recognizes the value of direct push technologies and strongly encourages its use. Unfortunately, many years ago, EPA established exhaustive guidelines for environmental site characterization that are based entirely around the deployment of drilled monitoring wells. They are funded to "regulate;" not rewrite these guidelines. As a result, almost all environmental sites are “technically investigated” by expensive monitoring well techniques. Direct push technologies are used as “screening” techniques. Engineering firms often use direct push technologies to evaluate the site, and then establish a few strategically located drilled monitoring wells to meet regulatory guidelines.

10. COST COMPARISON
Evaluating the cost difference between drilled monitoring wells versus direct push methods of site characterization is very complex. However, the larger the project and/or the deeper the testing; the more cost effective direct push will be. The following example was a small project with relatively deep tests.

**Scope:** Develop a stratigraphic map to bedrock (approximately 100 feet below ground surface), and collect samples from up to three sampling zones.

**Drilling:** To perform the desired sampling, drilling would mandate “nested” wells; mul-
multiple wells at each location with each well screened at a different level. Due to cost limitations imposed by the client, it was decided to limit the drilling to:
- Four stratigraphic borings to bedrock and grout all holes.
- Install two wells at each of 10 locations (20 wells) with one well screened at the water table and the other screened at bedrock.
- Containerize all waste products.

The drill contractor was not required to include disposal costs in his bid.

Direct Push: The scope of work for direct push techniques were significantly increased:
- Ten stratigraphic borings with the piezometer to bedrock.
- Collect water samples at three different depths at each of the ten piezometer locations.
- Grout each hole to closure using bentonite and distilled water.
- Install five 1” PVC wells to 30 feet deep.

Both proposals required steam cleaning all down hole equipment and contain the decontamination fluids for disposal by the client. The drilled proposal was $58,000 (without waste disposal). The DPT proposal (a larger scope of work) was $23,665.

11. CONCLUSION
The use of cone penetrometers and related direct push tools can significantly reduce the time and cost required to evaluate an environmentally contaminated site. Direct push technologies offer the following distinct advantages:

a) Penetration to the desired depth is quicker.
b) Penetration generates no waste.
c) The resultant hole is small and requires very little, if any, grout to close the hole.
d) Stratigraphic information is obtained in real time.
e) Perched water zones can be identified rapidly.
f) The depth to the water table can be determined rapidly.
g) Soil gas samples can be obtained inexpensively.

h) Ground water samples can be obtained very inexpensively without generating waste water.
i) Soil samples can be obtained rapidly.
j) Other sensors can be added to the cone penetrometer such as water level indicators, gas sampling ports, conductivity, and laser induced fluorescence.

12. REFERENCES
The use of CPT to evaluate the settlements of shallow foundations on residual soils

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SYNOPSIS. The evaluation of settlements of shallow foundations on sands from cone penetration test (CPT) results has been used for a long time. Nowadays there are several methods available, and most of them make use of concepts from the Theory of Elasticity, where a compressibility modulus is obtained from CPT results.

Based on the Theory of Elasticity and correlations between plate load tests and CPTs, a method to estimate the settlements of shallow foundations not only for sandy soils, but also for other soil types not subjected to classical consolidation processes, has been developed since the 60's by the Senior author. The method is based on data obtained in partially saturated residual soils. Most CPTs and plate load tests have been performed in the natural soil, but some tests in compacted material are also available.

This paper summarises the existing experience about the method.

1. INTRODUCTION
The CPT was used for the first time in Brazil in the middle of the 50's (Velloso, 1989) by Franki Piles Ltd., a Belgium company that started its activities in Brazil in the middle of the 30's. The equipment with a capacity of 20 kN was described by Velloso (1959). The use of the CPT by Franki Piles Ltd. was connected to the design of piles. It must be remembered that De Beer in 1948 mentioned that the CPT was very popular in Holland and Belgium, and had commenced to be used in other countries. Therefore it seems that Holland - Belgium - Brazil was the route by which the CPT arrived in Brazil.

Few years later a Brazilian company, Geotecnia S.A. started to use the CPT, also in connection with the design of piles. This company was responsible for the design of foundations (both deep and shallow) for a large refinery close to Rio de Janeiro. The design of the shallow foundations was performed using a method based on the Theory of Elasticity and plate tests according to the Houlsel's (1929) method (Barata, 1962). However, as the execution of a large number of plate tests was very expensive, the Senior author started at that time (1959-60) to develop correlations between the results of plate tests and CPTs to obtain the deformation modulus to use in the method. Since then new correlations have been established (e.g. Barata, 1967, 1983), all of them for residual soils (generally non saturated), mainly in the natural soil and in a few cases also in compacted material. The next section summarises the existing experience with the method.

2. THE BARATA'S METHOD
The settlement \( s \) of a footing with a width (or diameter) \( B \) and length \( L \) can be obtained as

\[
s = \lambda c_d \frac{A_p}{E_s} B (1 - \mu^2)
\]  

(1)

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where

\[ \lambda = \text{correction factor to take into account the embedment depth} \, h, \text{and can be obtained from the Fox's (1948) chart, included in Fig. 1.} \]

\[ c_s = \text{coefficient depending on the shape and rigidity of the foundation, obtained from Table 1} \]

\[ \Delta P = \text{stress (net) on the foundation} \]

\[ \mu = \text{Poisson’s ratio. A mean value of} \, \mu = 0.3 \text{ is generally accepted} \]

\[ E_c = \text{compressibility modulus,} \]

\[ E_c = a \cdot q_c \] \hspace{1cm} (2)

\[ q_c = \text{cone resistance} \]

The coefficient \( a \) was named Buisman’s coefficient by Barata (1962), in honor to K. Buisman, as Buisman (1940) suggested a similar expression to obtain the constrained modulus from \( q_c \). The Buisman’s coefficient can be obtained from Table 2.

### Table 1 - Values of \( c_s \) to use in expression (1), after Barata (1983).

<table>
<thead>
<tr>
<th>Shape/rigidity</th>
<th>Length/width (L/B) ratio</th>
<th>( c_s ) mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>Circular, rigid</td>
<td></td>
<td>0.79</td>
</tr>
<tr>
<td>Circular, flexible</td>
<td></td>
<td>0.85</td>
</tr>
<tr>
<td>Square, rigid</td>
<td>1</td>
<td>0.85</td>
</tr>
<tr>
<td>Square, flexible</td>
<td>1</td>
<td>0.95</td>
</tr>
</tbody>
</table>

| Rectangle, flexible  | 1.5                      | 1.15           |
|                      | 2.0                      | 1.30           |
|                      | 2.5                      | 1.41           |
|                      | 3.0                      | 1.53           |
|                      | 4.0                      | 1.68           |
|                      | 5.0                      | 1.83           |
|                      | 6.0                      | 1.93           |
|                      | 10.0                     | 2.25           |
|                      | 20.0                     | 2.68           |

Values of \( E_c \) must be evaluated in the region limited by the 0.1 \( \Delta P \) isobar below the footing, or in other words by the region 0 - \( nB \) below the footing, where \( n \) depends on the ratio L/B and can be obtained from Table 3.

### Table 2 - Values of the Buisman coefficient (after Barata, 1983).

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Buisman coefficient</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandy silt, clayey, residual soil from gneiss</td>
<td>1.15</td>
<td>Barata (1962)</td>
</tr>
<tr>
<td>Silty sand, residual soil from gneiss</td>
<td>1.20</td>
<td>Barata (1962)</td>
</tr>
<tr>
<td>Clayey silt, residual soil from gneiss</td>
<td>2.40</td>
<td>Barata (1962)</td>
</tr>
<tr>
<td>Sandy clay, residual soil from gneiss</td>
<td>2.85</td>
<td>Jardim (1980)</td>
</tr>
<tr>
<td>Clayey silt, compacted material</td>
<td>3.00</td>
<td>De Mello and Cepolini (1979)</td>
</tr>
<tr>
<td>Sandy clay, compacted material</td>
<td>3.40</td>
<td>Barata (1962)</td>
</tr>
<tr>
<td>Sandy clay, residual soil from gneiss</td>
<td>3.60</td>
<td>Jardim (1980)</td>
</tr>
<tr>
<td>Residual clay, compacted material</td>
<td>4.40</td>
<td>Barata (1962)</td>
</tr>
<tr>
<td>Sandy silty clay, residual soil from gneiss</td>
<td>5.20</td>
<td>Jardim (1980)</td>
</tr>
<tr>
<td>Sandy silty clay, residual soil from basalt, &quot;porous clay&quot;</td>
<td>5.20</td>
<td>Barata et al (1970)</td>
</tr>
</tbody>
</table>

### Table 3 - Values of the maximum depth (below the footing), \( nB \), of the 0.1 \( \Delta P \) isobar (after Barata, 1983).

<table>
<thead>
<tr>
<th>Shape</th>
<th>L/B</th>
<th>( n^* )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Circular or square</td>
<td>1</td>
<td>( \approx 2 )</td>
</tr>
<tr>
<td>1.5</td>
<td>( \approx 2.5 )</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>( \approx 3 )</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>( \approx 3.5 )</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>( \approx 4 )</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>( \approx 4.25 )</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>( \approx 5 )</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>( \approx 5.5 )</td>
<td></td>
</tr>
<tr>
<td>Strip load</td>
<td>( \infty )</td>
<td>( \approx 6.5 )</td>
</tr>
</tbody>
</table>

* Based on the Steinbrener chart, for clayey soils. In case of sandy soils, increase \( n \) by 20%.
The $E_a$ values must then be plotted against depth. From the trend found for the $E_a$ behaviour with depth, the value of $E_a$ to be used in expression (1) must be obtained from the mid-level (see Burmister, 1948) of the region previously mentioned (i.e., B below the footing in the case of square footings).

3. DISCUSSION

i) In the method proposed in the present paper a simple approach is suggested regarding the evaluation of settlements of footings, i.e., expression (1), derived from the Theory of Elasticity is used in connection with the between mechanical and electric penetrometers. Schmertmann (1978) concluded that additional data were necessary in order to obtain a more accurate trend.

Nowadays it is generally recognized the importance of correcting the measured cone resistance to take into account the pore pressure at unequal end areas (e.g., Campanella et al., 1982, Lunne et al., 1986), particularly important in the case of soft clays. Therefore, in this case very different values of cone resistance could be obtained from mechanical and electric penetrometers, and even from different electric penetrometers (e.g., Lunne et al., 1986).

However, the soils analysed in the present paper are generally non-saturated, and even when saturated they are not soft clays.

Therefore, and in the lack of other data, it is suggested that the correlations established herein between plate test and CPT results can also be used for electric penetrometers.

ii) In all the correlations established between plate tests and CPTs a mechanical penetrometer cone was used. The Authors do not know any correlation between the results of mechanical and electric penetrometers as far as residual soils are concerned.

De Ruiter (1971) mentioned that differences on the measurement of cone resistance and sleeve friction from mechanical and electric penetrometers are expected due to 2 reasons: influence of the shape of the penetrometer and difference in the method of advancing the cone. However, as far as cone resistance is concerned, De Ruiter (1971) has not found any systematic difference between the values measured from the 2 types of penetrometers.

Schmertmann (1978) summarised some data, primarily for sands, concerning a comparison
compressibility modulus taken at the mid-level of the region limited by the 0.1 $\Delta p$ isobar below the footing. This is a different approach from that proposed by Schmertmann (1970, 1978) where a simplified vertical strain distribution below the center of the loaded area is used. Although the Schmertmann’s (1970, 1978) approach is more rigorous than the one suggested herein, Denver (1981) pointed out that there are some drawbacks connected with the application of the strain influence factor calculated by the Schmertmann’s (1970) method. Denver (1981) argued that it is possible to use the same concept not only on the axis of the footing, but also on all vertical lines within the edge of a rigid footing to calculate the total displacement. Denver (1981) also mentioned that it is difficult to know whether the Schmertmann’s (1970) approximation is on the safe or on the unsafe side of the correct solution.

Moreover, the method proposed herein was developed based on plate tests for which no significant heterogeneity was found below the plates. Therefore, due to the reasons above, it does seem justifiable to use such a simple procedure. In the case of very large footings on heterogeneous soils, however, it may be important to consider the influence of the different layers using a more rigorous approach.

ii) It has been generally recognized (e.g. De Beer, 1965, Schmertmann, 1978, Bellotti et al., 1986, Leonards and Frost, 1987, Decourt, 1989) that penetration tests including SPT and CPT are not able of sensing the effect of prestressing of granular soils. Thus, methods of calculating the settlements of footings based on correlations between these tests and soil modulus will overestimate the settlements if the deposit has been prestressed (Leonards and Frost, 1987). De Beer (1965) and Schmertmann (1978) both mentioned that their methods are valid for the case of normally consolidated sands.

The concept of overconsolidation in its classical sense is not applicable to residual soils, since the behaviour of these soils is predominantly influenced by physico-chemical weathering during the soil formation. However, Vargas (1951) has established the concept of "virtual overconsolidation stress", to allow treating the residual soils in the same framework as for the sedimentary soils.

The influence of overconsolidation was not considered during the development of the method proposed herein. It is possible that different values obtained for the Buisman coefficient (Table 2) for the same type of soil is due to different values of "virtual overconsolidation.

4. A MORE SIMPLIFIED EXPRESSION

Although the method proposed in the present paper is quite simple to use in engineering practice, it is still possible to turn it even simpler for footings with a base shape close to a square. In fact, most footings are generally constructed at relative depths, $h/B$, around 0.5-1.0. Thus, it is possible to consider a mean value of $\lambda$ around 0.8. Most footings can be considered rigid, thus $c_n$ is taken as 0.85. A mean value of $\mu$=0.3 is generally accepted for the Poisson’s ratio, thus $(1-\mu)$ is about 0.9. Combining these 3 factors, expression (1) can be simplified to

$$s = 0.6 \frac{\Delta p}{E_s} B$$

(3)

If there is just one soil layer in the region limited by the 0.1 $\Delta p$ isobar below the footing, it is not necessary to plot $E_s$ versus depth, just $q_s$ versus depth, taking $q_s$ from the trend found at the mid-level of the region described above, i.e. B below the footing for square footings.

Moreover, according to dimensional analysis concepts, it is more convenient to express the settlement $s$ normalized by the width $B$, then expression (4) is suggested to be used in the case of square footings on homogeneous soils.

$$\frac{s}{B} = a \frac{\Delta p}{q_s}$$

(4)
5. CONCLUSIONS
A method to estimate the settlements of footings on residual soils from the results of CPTs has been presented.

The method has been developed and used by the Senior author since the 60’s, and is based on correlations between plate load tests and CPTs.

The data have been obtained from residual soils, generally non saturated. Most tests have been performed in the natural soil, but some tests in compacted material are also available.

6. REFERENCES


Comparative assessment of the bearing capacity out of CPT-results.

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SYNOPSIS: The paper proposes a comparison of the assessment of the bearing capacity of a single pile out of CPT-results according to three methods: the method of prof. E. De Beer amended by prof. Van Impe, the French method of Bustamante (DTU 13.2) and the Dutch geotechnical code NEN 6743. The theoretical results are compared with those of full-scale load tests on Omega-piles in Belgium and Sweden.

1. INTRODUCTION

The CPT-test has been extensively used over the last decades in Belgium in order to assess the bearing capacity of a single pile. The most widely used method is the method of Prof. E. De Beer [4], amended by Prof. W. Van Impe [7]. The same kind of method was proposed in the Netherlands by Beegemann [1,2] and a modified version is now used and described in the Dutch Code NEN 6743 [6]. Recently, some comments were formulated in Belgium with respect to the principles of the method and the results obtained and some engineers recommended to use the Bustamante method described in the French DTU 13.2 Code [5].

We try here to explain why the differences are not only a question of details but in some cases a real difference in the basic philosophy. We compare the results obtained by the methods chosen for this demonstration with those of instrumented full-scale load tests.

2. METHODS' PHILOSOPHY


The first problem occurs when trying to find back which definition of the "ultimate" bearing capacity is used in the different methods. A clear distinction should be made between:

1) The ultimate bearing capacity of the soil surrounding the pile, accompanied by large settlement of the pile base.

2) The conventional ultimate bearing capacity with a relative pile base settlement of 0.1 x \( \delta_b \).

3) The limit load, causing a relative pile base settlement of 0.025 x \( \delta_b \).

We will assume in the rest of the paper that the ultimate bearing capacity of the pile we refer to is the conventional one, as it is clearly the case in the De Beer and Van Impe method. However, when comparing the results by the different methods, it should be reminded that the so-called ultimate load is not clearly defined neither in the French method nor in the Dutch code.

2.2. End and shaft bearing capacity.

The ultimate bearing capacity is the sum of the end bearing capacity and the shaft resistance. Each one is deduced separately out of the CPT results.

Most of the methods propose to calculate the unit ultimate end bearing capacity of a single displacement pile out of an average value of the cone resistance \( q_c \) above and below the pile base. The Dutch code chooses three trajectories along which the average has to be calculated, limiting the gradient of increase. The methods of De Beer and Van Impe introduce the concept of critical density and scale effect between the cone and the pile. The French method of Bustamante more simply applies a reduction factor to the \( q_c \)-value at the depth of the pile base.

The shaft resistance is usually deduced out of the \( q_c \)-value multiplied by coefficients taking into account the soil type and the influence of the pile installation process. It is interesting to compare the coefficients proposed by the different methods.

It is also possible to evaluate the total pile shaft resistance directly out of the measurement of the total skin friction, if this value is available from the CPT.
3. ULTIMATE END BEARING CAPACITY

The ultimate bearing capacity of a pile is given by:

\[ q_{u,b} = \Omega_0 \cdot q_{u,b} \]

where \( q_{u,b} \) is the cross-section of the pile base

\( \Omega_0 \) is the ultimate unit bearing capacity of the pile.

The ultimate bearing capacity is given by:

\[ q_{u,b} = \Omega_0 \cdot \epsilon_b \cdot q^{*}_{u,b} \]

with

\[ q^{*}_{u,b} \] is the ultimate bearing capacity directly derived from the CPT results by one of the methods we develop below;

\( \epsilon_0 \) is the installation factor taking into account the influence of the pile installation in the given soil condition;

\( \epsilon_b \) is the scale factor for soil discontinuities such as fissuring. This factor is not taken into account in all the calculation methods and will not be studied here. It will be considered equal to 1.

3.1. Evaluation of \( q_{u,b} \)

3.1.1. De Beer/Van Impe Method

In the early seventies, Prof. Dr. E. De Beer developed a prediction method based on the CPT results obtained with the M4 type of cone [De Beer, 1971, 4].

- critical penetration density

Between the loose and dense states, there exists an intermediate state, which De Beer calls critical state or critical penetration density for which the cone resistance continues to increase linearly with depth. Above the critical depth \( h_{crit} \), the cone penetration is mostly governed by expulsion, underneath the critical depth it is mostly governed by densification, (fig. 1).

![Schematic soil profile.](image)

For the pile, the critical depth \( h_{crit} \) will be given with a very good approximation by

\[ h_{crit} = \frac{\Omega_0 \cdot \epsilon_b}{\epsilon_0} \]

The simple shape of figure 1 is not often met in the reality. Mostly the soil profile consists in a succession of layers of different nature and densities, resulting in a complicated shape of the \( q_{u,b} \)-diagram. The assumption is that every step of lecture (usually equal to 0.20 m) there is a passage of a soft layer to a strong layer, or vice-versa and that the critical depth for the cone \( h_{crit} \) equals the lecture interval of 0.2 m.

- Downward values \( q_{u,b,j} \)

If we consider the passage from a soft layer to a resistant layer, the fig. 2 gives the diagram of the \( q_{c} \)-values and of the unit ultimate load \( q_{u,b} \) at the base of the pile.

The gradient of the \( q_{u,b} \)-values will be lower than the gradient of the \( q_{c} \)-values in the ratio \( \frac{\Omega_0}{\epsilon_0} \) (\( \epsilon_0 \), diameter of the cone; \( \frac{\Omega_0}{\epsilon_0} \), diameter of the pile).

![Diagram of \( q_{c} \)-values and unit ultimate load \( q_{u,b} \).](image)

Expressions (1, 2, 3, 4) are valid for \( h_{j-1} < h_{j} \), i.e. for \( \frac{\Omega_0}{\epsilon_0} \) which is expected with high accuracy. The \( \frac{\Omega_0}{\epsilon_0} \) is the ratio of the critical penetration density to the penetration density at the base of the pile. If we consider that, for each layer, the cone resistance increases linearly with the penetration, it follows that the \( \frac{\Omega_0}{\epsilon_0} \) is equal to the ratio of the ultimate load at the base of the pile to the ultimate load at the level of the layer.

- \( q_{u,b,j} + 1 \)

The expressions (1, 2, 3, 4) are valid for \( h_{j-1} < h_{j} \), i.e. for \( \frac{\Omega_0}{\epsilon_0} \) which is expected with high accuracy. The \( \frac{\Omega_0}{\epsilon_0} \) is the ratio of the critical penetration density to the penetration density at the base of the pile. If we consider that, for each layer, the cone resistance increases linearly with the penetration, it follows that the \( \frac{\Omega_0}{\epsilon_0} \) is equal to the ratio of the ultimate load at the base of the pile to the ultimate load at the level of the layer.

400

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where $\beta_c$ and $\beta_b$ are the angles of Meyerhoff related to the diameter of the sounding cone and of the pile.

The complete method allows for the determination of $q_u, b, j + 1$ starting from the soil surface and going downward. When a depth is reached where the calculation gives $q_u, b, j + 1 > q_u, h, j + 1$ one puts $q_u, b, j + 1 = q_u, h, b, j + 1$. The gradient of increase is in that way limited.

- **Upward values** $q_u, b, j (\text{De Beer})$

  We introduce the same concept of scale effect for the passage from an upper stronger layer to a softer lower layer. The pile will feel the presence of the soft layer at an earlier stage than the cone:

  \[
  h_{\text{crit}} = h_{\text{crit}} \cdot \frac{q_u}{q_c}.
  \]

  \[
  q_u, b, j + 1 = q_u, b, j + \frac{2q_c}{q_u} (q_u, b, j + 1 - q_u, b, j).
  \]

  If the obtained $q_u, b, j + 1$ is larger than $q_u, b, j + 1$ at $j + 1$, we put $q_u, b, j + 1 = (q_u, b, j + 1) + 1$.

- **Improved upward values** (Van Impe)

  Prof. Van Impe adapted the calculation of the upward values of $q_u, b$ [2]. Out of numerous test results on screwed piles, it appeared that the $h_{\text{crit}}$ value was overestimated for the transition from resistant to softer layers by a factor of more than two. He proposed to modify $h_{\text{crit}}$ in

  \[
  h_{\text{crit}} = \frac{b_{\text{crit}}}{2q_c}.
  \]

  so that the modified upward values of $q_u, b$ become:

  \[
  q_u, b, j + 1 = q_u, b, j + \frac{2q_c}{q_u} (q_u, b, j + 1 - q_u, b, j).
  \]

- **Blending or mixing**

  The obtained values are already very close to the reality. However, in order to take into account the influence of the dimension of the pile base relatively to the calculation step, the values $q_u, b, j + 1$ are blended by calculating the average value of $q_u, b, j + 1$ over a thickness equal to the pile diameter, under the considered level.

This gives the $q^*_u,b$ values which are the final result of the calculations of the ultimate end bearing capacity following the method of De Beer/Van Impe.

### 3.1.2. Method of DTU13.2

The French method of Bustamante evaluates the end bearing capacity by:

\[
q^*_u,b = k_c \cdot q_c
\]

where $k_c$ is a reduction factor depending on the soil type and the pile type, for an anchoring into the resistant layer which is greater than 3. $q_c$ is the cone resistance at the pile base level. For a displacement pile (driven piles), $k_c$ is given in the table below:

<table>
<thead>
<tr>
<th>Soil type</th>
<th>$q_c$ (kPa)</th>
<th>$k_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>&gt;5000</td>
<td>0.4</td>
</tr>
<tr>
<td>2000 to 5000</td>
<td>0.35</td>
<td></td>
</tr>
<tr>
<td>&gt;5000</td>
<td>0.45</td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>0 to 2500</td>
<td>0.4</td>
</tr>
<tr>
<td>&gt;10000</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>Chalk</td>
<td>&gt;5000</td>
<td>0.2</td>
</tr>
</tbody>
</table>

The values of $k_c$ for the other pile types are discussed in point 3.2. A major problem of the method is the choice of the $q_c$-value to use for the calculations of the bearing capacity, especially in the case of a discontinuous or intermediate layer, as it will be shown in point 6.

### 3.1.3. Dutch method (NEN 6743)

A calculation method was introduced in the Netherlands by Begemann in 1963 [Begemann-1963, 1969-67]. The Dutch code now presents an evaluated method based on the same principles. The unit ultimate end bearing capacity of the pile is given by

\[
q^*_u,b = \frac{L}{\beta \cdot s (q_{c, I, \text{av}} + q_{c, II, \text{av}} + q_{c, III, \text{av}})}
\]

with

- $\beta = \text{factor taking into account the shape of the pile base}$;
- $s = \text{factor taking into account the shape of the cross-section of the pile base}$.

$q_{c, I, \text{av}} = \text{average value of } q_c\text{-values along trajectory I, from the level of the pile base to a level >0.7 } \cdot \phi_{eq} \text{ and < 4 } \cdot \phi_{eq} \text{ so that } q_{c, I, \text{av}} \text{ is minimal}$.
The ultimate shaft bearing capacity of a pile can also be evaluated as follows:

\[ Q_{us} = \sum \zeta_{pi} \cdot q_{us,i} \]

where \( \zeta_{pi} \) is the lateral surface of the pile for the considered layer.

Most of the methods are now evaluating the \( q_{us,i} \) out of the qc-value:

\[ q_{us,i} = \zeta_{i} \cdot \eta_{p} \cdot q_{c} \]

with \( \zeta_{i} \) = installation factor taking into account the influence of the pile installation.

\( \eta_{p} \) = factor depending on the soil.

The values of \( (\zeta_{i}, \eta_{p}) \) are given in table 3 according to the different methods and for different soil conditions. The values are also given for displacement piles \( (\zeta_{i} = 1) \).

As for table 2, one should use table 3 carefully.

The values of \( 1/\eta_{p} \) are simplified for purpose of presentation. Each value is in reality directly related to specific conditions of particular test sites. For the precise value, one should refer to the original publications.

For Van Impe [7, 8], for example, the values of \( \eta_{p} \) for sands are valid for N.C. quartz sands. He also introduces a distinction with regard to the consistency of clay:

- Consistent clay \( 0.3 < IL < 0.6, \) 2MPa < qc < 8MPa \( 1/\eta_{p} \)
- Driven piles and screw piles \( 60 \)
- Precast piles and bored piles \( 85 \)
- Stiff clay \( IL > 0.8 ; \) qc > 2MPa \( 1/\eta_{p} = 80 \)

**Table 2. END BEARING CAPACITY - INSTALLATION COEFFICIENT α_b**

<table>
<thead>
<tr>
<th>Type</th>
<th>De Beer/Van Impe</th>
<th>DTU 13.2</th>
<th>NEN 6743</th>
</tr>
</thead>
<tbody>
<tr>
<td>Driven pile</td>
<td>1 to 1.15 (*)</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Screwed pile</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Bored pile</td>
<td>Stiff O.C. Clay</td>
<td>0.8</td>
<td>0.82</td>
</tr>
<tr>
<td></td>
<td>Soft cohesive soil</td>
<td>0.33 to 0.8 (**)</td>
<td>0.75 to 0.8</td>
</tr>
<tr>
<td>Precast pile</td>
<td>0.85 (Stiff clay)</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

(*) 1 for cast in situ driven concrete pile
1.15 for driven concrete piles with low water-cement factor concrete (type FRANKI)

(**) 0.75 to 0.8 for bored piles, the relative displacement of \( 0.1 \times \alpha_b \) is usually not sufficient to mobilize the end bearing capacity. An estimation of \( \alpha_b \) should therefore be linked with relative smaller settlements \( \alpha_{b,z} = 0.8 \cdot (\alpha_b \cdot \alpha_b) \) with \( K = q_u \cdot b \cdot q_u \cdot b \cdot \alpha_{b,max} - q_u \cdot b \cdot \alpha_{b,min} \)
Table 3. SHAFT RESISTANCE-INSTALLATION COEFFICIENT

<table>
<thead>
<tr>
<th>DRIVEN PILES</th>
<th></th>
<th></th>
<th>DTU.13.2</th>
<th>NEN 6743</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil type</td>
<td>qc (kPa)</td>
<td>Van Impe</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand, gravel</td>
<td>0 to 2500</td>
<td>0 to 100</td>
<td>80</td>
<td>83 (71)</td>
</tr>
<tr>
<td></td>
<td>2500 to 10000</td>
<td>30 to 100</td>
<td>100</td>
<td>83 (71)</td>
</tr>
<tr>
<td></td>
<td>10000 to 15000</td>
<td>100</td>
<td>150</td>
<td>83 (71)</td>
</tr>
<tr>
<td></td>
<td>15000 to 20000</td>
<td>100 to 200</td>
<td>150</td>
<td>83 (71)</td>
</tr>
<tr>
<td>Clay</td>
<td>0 to 2000</td>
<td>50</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>2000 to 5000</td>
<td>85</td>
<td>40</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>&gt;5000</td>
<td>85 to 80</td>
<td>60</td>
<td>30</td>
</tr>
</tbody>
</table>

PRECAST PILES

<table>
<thead>
<tr>
<th>Soil type</th>
<th>qc (kPa)</th>
<th>Van Impe</th>
<th>DTU.13.2</th>
<th>NEN 6743</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand, gravel</td>
<td>0 to 2500</td>
<td>0 to 100</td>
<td>80</td>
<td>83 (71)</td>
</tr>
<tr>
<td></td>
<td>2500 to 10000</td>
<td>30 to 100</td>
<td>100</td>
<td>83 (71)</td>
</tr>
<tr>
<td></td>
<td>10000 to 15000</td>
<td>100</td>
<td>150</td>
<td>83 (71)</td>
</tr>
<tr>
<td></td>
<td>15000 to 20000</td>
<td>100 to 200</td>
<td>150</td>
<td>83 (71)</td>
</tr>
<tr>
<td>Clay</td>
<td>0 to 2000</td>
<td>50</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>2000 to 5000</td>
<td>85</td>
<td>40</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>&gt;5000</td>
<td>85 to 80</td>
<td>60</td>
<td>30</td>
</tr>
</tbody>
</table>

SCREW PILES OF DISPLACEMENT TYPE

<table>
<thead>
<tr>
<th>Soil type</th>
<th>qc (kPa)</th>
<th>Van Impe</th>
<th>DTU.13.2</th>
<th>NEN 6743</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand, gravel</td>
<td>2500 to 10000</td>
<td>30 to 100</td>
<td>110</td>
<td>110 to 165</td>
</tr>
<tr>
<td></td>
<td>10000 to 15000</td>
<td>100</td>
<td>110</td>
<td>110 to 165</td>
</tr>
<tr>
<td></td>
<td>15000 to 20000</td>
<td>100 to 200</td>
<td>110</td>
<td>110 to 165</td>
</tr>
<tr>
<td>Clay</td>
<td>0 to 2000</td>
<td>50</td>
<td>20/40</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>2000 to 5000</td>
<td>85</td>
<td>60</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>&gt;5000</td>
<td>85 to 80</td>
<td>60</td>
<td>30</td>
</tr>
</tbody>
</table>

BORED PILES

<table>
<thead>
<tr>
<th>Soil type</th>
<th>qc (kPa)</th>
<th>Van Impe</th>
<th>DTU.13.2</th>
<th>NEN 6743</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand, gravel</td>
<td>0 to 2500</td>
<td>0 to 100</td>
<td>60 to 120</td>
<td>165</td>
</tr>
<tr>
<td></td>
<td>2500 to 10000</td>
<td>30 to 200</td>
<td>100 to 180</td>
<td>165</td>
</tr>
<tr>
<td></td>
<td>10000 to 15000</td>
<td>200</td>
<td>150</td>
<td>165</td>
</tr>
<tr>
<td></td>
<td>15000 to 20000</td>
<td>300</td>
<td>150</td>
<td>165</td>
</tr>
<tr>
<td>Clay</td>
<td>0 to 2000</td>
<td>50</td>
<td>20/40</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>2000 to 5000</td>
<td>85</td>
<td>40</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>&gt;5000</td>
<td>85 to 80</td>
<td>60</td>
<td>30</td>
</tr>
</tbody>
</table>

(*) for z<0<z, 1/np = 20
for z>0>z, 1/np = 40

5. ALLOWABLE BEARING CAPACITY - SAFETY FACTOR
Here again, a great difference is found in the way to apply the safety factor. The Belgian methods apply a higher safety factor on the shaft resistance than on the end bearing capacity (the usual factors are : 2 on the end bearing capacity and 3 on the shaft).
The French method applies a higher safety factor on the end bearing capacity (3) than on the shaft resistance (2). This leads to a ratio shaft/end bearing which is quite different than with the other methods.
The Dutch code adopt a more sophisticated statistical approach which is closer to the principles of the Eurocode 7.
In this interesting approach, the ultimate limit state (ULS) has to be tested according to two conditions:
ULS 1A : failure in the soil where the design value of the load is compared to the design value of the bearing capacity.
ULS 1B : failure of the construction as a result of differential settlement of the foundation.
The safety level, in terms of the overall safety factor for ULS1A, failure in the soil lies between 1.8 and 2.2 [10].

6. NUMERICAL EXAMPLE AND LOAD TESTS.
6.1. Introduction
We give hereunder two numerical examples and compare the ultimate bearing capacity obtained.
with the different methods for three single piles in different soil conditions. The results are also compared with those of full-scale load tests. In one case, the load tests were instrumented so that it was possible to measure separately the end and the shaft resistance of the piles. The piles installed on both sites were Omega screw piles. This new type of screw pile is a soil displacement auger pile based on a screwing-in - screwing out procedure. The displacement auger head (fig. 3) consists of a discontinuously diameter-increasing steel tip, on top of which a continuous screw flange with variable tip is mounted. The soil displacement is automatically ensured by downwardly oriented slots mounted on the auger head at different well-chosen locations on the flanges. Above the full diameter body, the soil displacement, with severely reduced transfer energy, is due to the downwardly inclined overlapping of various steel shields oriented in an opposite inclination as the flanges below the diameter body [3].

Steel tell-tale rods were used for determination of the settlement in the different cross sections. The load-settlement curve for test pile no 5 is represented in fig.5.

In the analysis of the various methods, we used the installation coefficients recommended for the displacement piles type, as the purpose of the paper is the comparison of the methods (load distribution, ratio shaft/end bearing capacity,...) and not the detailed analysis of the specific bearing capacity of the Omega pile which is treated in other papers.

6.2. Pile load tests - Vilvoorde-Belgium.
6.2.1. Geotechnical conditions
The piles were installed at Vilvoorde (Belgium).
A typical CPT-log is shown in fig. 4.
We analyse the results of two piles : one stopped in layer 3 (pile no 5) and one stopped in layer 5 (pile no 3). The piles had a diameter of 41 cm.

Fig. 4 Typical CPT-Vilvoorde.

6.2.2. Discussion of the results
Table 4.1 above compares the results obtained by the different methods for the ultimate end bearing capacity and the measured one for piles 3 and 5. Table 4.2 shows the same comparison for the ultimate shaft resistance. The measured values in table 4.1 and 4.2 correspond to a base settlement of 0.1 x ∆p. The load test on pile no 3 didn’t reach that settlement. The conventional ultimate load was estimated with the help of a specific developed computer code called PALOS [Gwiżdala et al, 9].
Finally, table 5 summarizes the results for the allowable bearing capacity and gives the ratio shaft/end bearing capacity. It also gives the measured load for a settlement of 0.0075x∆p,
### Table 4.1 Ultimate unit end bearing capacity

<table>
<thead>
<tr>
<th>Pile number</th>
<th>De Beer</th>
<th>Van Impe</th>
<th>Bustamante</th>
<th>Dutch Code</th>
<th>Measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>5.93</td>
<td>7.52</td>
<td>16</td>
<td>9.7</td>
<td>10.1</td>
</tr>
<tr>
<td>5</td>
<td>5.14</td>
<td>6.33</td>
<td>4</td>
<td>7.8</td>
<td>9.9</td>
</tr>
</tbody>
</table>

### Table 4.2 Ultimate shaft bearing capacity

<table>
<thead>
<tr>
<th>Pile number</th>
<th>De Beer (total shaft)</th>
<th>Van Impe</th>
<th>Bustamante</th>
<th>Dutch Code</th>
<th>Measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>1119</td>
<td>1009</td>
<td>1498</td>
<td>1110</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>602</td>
<td>565</td>
<td>594</td>
<td>600</td>
<td></td>
</tr>
</tbody>
</table>

### Table 5 Allowable Bearing Capacity - Vilvoorde

<table>
<thead>
<tr>
<th>Method</th>
<th>Pile no. 3 Allowable load</th>
<th>Pile no. 5 Allowable load</th>
<th>Ratio shaft/end</th>
<th>Ratio shaft/end</th>
</tr>
</thead>
<tbody>
<tr>
<td>Debeer</td>
<td></td>
<td></td>
<td>Ratio shaft/end</td>
<td>Ratio shaft/end</td>
</tr>
<tr>
<td>Van Impe</td>
<td></td>
<td></td>
<td>0.75</td>
<td>618</td>
</tr>
<tr>
<td>DTU 13.2</td>
<td></td>
<td></td>
<td>0.72</td>
<td>459</td>
</tr>
<tr>
<td>NEN 6743 (*)</td>
<td></td>
<td></td>
<td>1.01</td>
<td>1104</td>
</tr>
<tr>
<td>Measured</td>
<td></td>
<td></td>
<td>0.79</td>
<td></td>
</tr>
<tr>
<td>(0.00755x10b)</td>
<td></td>
<td></td>
<td>1.00</td>
<td>(Palos)</td>
</tr>
<tr>
<td>(1.1x0b)</td>
<td></td>
<td></td>
<td>0.64</td>
<td>1788</td>
</tr>
</tbody>
</table>
| (*) The values of the allowable load are given for purpose of comparison with a safety factor of 2.

inclusive the elastic shortening, which is the limit stated by the Belgian national code (STS).

It is important to note that we took the installation coefficients recommended for displacement piles. According to the results, the Omega pile can surely be considered as a displacement pile.

The values of the ultimate shaft bearing capacity following the different methods fit quite well. The value of the ultimate unit end bearing capacity gives similar results with the exception of the DTU-method. This is due to the principle of the method which simply applies a reduction factor to the cone resistance. This leads, for layers with a local high qc-value to much higher values and for constant qc to much lower values than with the other methods.

6.3. Pile load tests - Mellansel - Sweden

6.3.1. Geotechnical conditions

Mellansel is a little town situated in the North of Sweden. Omega-piles were installed for the foundation of a new bridge for the highspeed railway line.

The piles had a diameter of 0.43 m. and were anchored at a depth varying between 17.50 m. and 20.50 m. The design load was 700 kN.

The soil profile is illustrated on the typical CPT shown in fig. 6.

The installation of the piles was controlled by an extensive program of load-tests. We analyse the results of pile no 31, installed at a depth of 19.50 m.

6.3.2. Discussion of the results

The load-settlement curve is given in fig. 7.

---

Fig. 6 Typical CPT-Mellansel

![Typical CPT-Mellansel](image)

Fig. 7 Load settlement curve-pile n° 31

![Load settlement curve](image)
Table 6. Ultimate Bearing Capacity - Mellansel

<table>
<thead>
<tr>
<th>Method</th>
<th>Unit End Bearing Capacity (MPa)</th>
<th>Shaft bearing capacity (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>De Beer</td>
<td>Van Impe</td>
</tr>
<tr>
<td>1.76</td>
<td>9.60</td>
<td>4.8</td>
</tr>
</tbody>
</table>

Table 7. Allowable Bearing Capacity - Mellansel

<table>
<thead>
<tr>
<th>Method</th>
<th>Allowable load (kN)</th>
<th>Ratio shaft/end</th>
</tr>
</thead>
<tbody>
<tr>
<td>Van Impe</td>
<td>1134</td>
<td>0.63</td>
</tr>
<tr>
<td>DTU 13.2</td>
<td>908</td>
<td>2.91</td>
</tr>
<tr>
<td>NEN 6743</td>
<td>1394</td>
<td>0.93</td>
</tr>
<tr>
<td>Measured at 0.0075 x 0</td>
<td>950</td>
<td>/</td>
</tr>
</tbody>
</table>

(*) The values of the allowable load are given for purpose of comparison with a safety factor of 2.

Table 6 compares the results obtained by the different methods for the ultimate end and shaft bearing capacity.

Table 7 summarizes the allowable bearing capacity and compares the results with the measured load at a settlement equal to 0.0075 x 0, inclusive the elastic shortening. The latter is of the order of 1.5 mm taking into account that 2/3 of the load is taken by the shaft. This is a very drastic limit, surety for long piles as it is the case here. The obtained results are thus most satisfactory.

7. CONCLUSION

Tables 5 and 7 clearly show the discrepancy between the different methods, especially with regard to the ratio shaft/end bearing capacity for the allowable load, which is, of course, the final result which interests the designer. If we deduce the ratio shaft/end bearing capacity from the load-settlement curve of pile n° 3 in Vilvoorde, it appears (table 5) that this ratio is equal to 2 for a settlement of 0.0075 x 0 and equal to 0.84 for a settlement of 0.1 x 0, corresponding to the conventional ultimate load. This means that in the common range of settlements authorized for the pile foundations, the load is mostly ensured by the shaft resistance. It seems that the French method tries to take this into account by giving more weight to the shaft bearing capacity (see table 5 and 7). This problem is in fact directly related to the principle of the determination of the allowable bearing capacity by calculating a conventional ultimate load -which corresponds with small differences between the methods- to a settlement of 0.1 x 0 and expecting that this allowable load corresponds to a much smaller settlement (0.0075 x 0 in the Belgian Code).

The logical approach should be to consider both the ultimate load and the settlement problem (such as in the Dutch Code) or to assess - if we are able to do so - a bearing capacity corresponding to smaller settlements and to apply a safety factor just taking into account the uncertainties of the model. In these limits however, it appears that the calculation methods give a good approximation of the measured bearing capacity, considering the Omega pile as a displacement pile.

8. REFERENCES

Analysis of horizontally loaded pile behaviour from cone penetration tests in centrifuge

Ali Bouafia
Dept of civil engineering, Blida University, Algeria
Zein-Eddine Merouani
Dept of civil engineering, Blida University, Algeria

SYNOPSIS: Tests from a miniature CPT equipment taken on a centrifuge allowed the analysis of the behaviour of small scale centrifuged single piles horizontally loaded in sand.

After a description of the experimental set up, it shows that a standard correlation between the pile response parameters and the cone resistance gives a good prediction of the pile behaviour.

Thereafter, the effect of the proximity of a slope on the behaviour of the pile is considered. This analysis is based on data from cone penetration testing.

1. INTRODUCTION
Initially, piles were designed in order to transmit vertical loads from the structures to the soil. When these foundations were besides loaded horizontally, inclined piles, often difficult to achieve, were to be added.

Due to the progress done in the knowledge of the behaviour of deep foundations, it is now admitted that vertical piles can sustain horizontal loads.

The earth pressure on a bridge pile, the lateral displacement of a layer of soft clay under an embankment for access to a motorway, the effects of the wind on slender structures built on piles are usual examples of horizontal loading of foundations.

While tests on full-scale foundations are limited by the cost, the time and the quasi impossibility to carry out parametric studies, they can easily be carried out on small scale models under perfectly controlled experimental conditions.

Mass forces have an essential role in the behaviour of soil, which is a non-linear material. According to similarity conditions, in order to keep stresses and strains between the model and the prototype when the dimensions are reduced to scale 1/N, it is necessary to increase the mass forces inversely, that is N. This condition can be accomplished by placing the model in a centrifuge. The geotechnical modelling in centrifuge has known these last decades a great success, and has allowed engineers to explore domains difficult to access by other approaches such as full scale tests or numerical analysis.

The usual similarity scales are shown in table 1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Scale</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strain</td>
<td>kN/m²</td>
<td>1</td>
</tr>
<tr>
<td>Rotation</td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Stress</td>
<td>kN/m²</td>
<td>1/N²</td>
</tr>
<tr>
<td>Displacement</td>
<td>m</td>
<td>1/N</td>
</tr>
<tr>
<td>Dimension</td>
<td>m</td>
<td>1/N</td>
</tr>
<tr>
<td>Load</td>
<td>N</td>
<td>1/N²</td>
</tr>
<tr>
<td>Moment</td>
<td>N.m</td>
<td>1/N²</td>
</tr>
<tr>
<td>Stiffness (E, tₚ)</td>
<td>N.m²</td>
<td>1/N⁴</td>
</tr>
</tbody>
</table>

Unlike the laboratory tests where the extraction of intact soil samples is a problem,
the cone penetration testing is short and easy to carry out, and gives the mechanical properties of the soil in situ. However, since it is a destructive test, it does not give the stress-strain relationship for the soil.

The modelling of the cone penetrometer in centrifuge has given interesting results on the effect of certain parameters on the cone resistance, $q_c$. The tests carried out by Phillips & Valsangkar (1987) and Mokri (1988) on centrifuged CPT models in dry sand showed that the rate of penetration has a little influence on $q_c$. However, when the cone diameter increases, $q_c$ decreases (particularly for diameters less than 40 mm).

This work is concerned with the analysis of tests on small scale piles, carried out on the centrifuge of the Laboratoire Central des Ponts & Chaussées (LCP), centre of Nantes, France. The main objective is to show the reliability of cone penetration testing as a design tool for the response of a horizontally loaded pile.

2. EXPERIMENTAL SET UP

2.1 The centrifuge
The centrifuge of LCP, Nantes is a performing device for experimental research in geotechnical engineering. It is capable of taking a model weighing 2000 kg at a centrifugal acceleration of 100 times that of terrestrial gravity (Garnier, 1990).

2.2 The pile model and instrumentation
The pile model is a smooth surface aluminium pipe, simulating at the scale 1/17.8 a circular pile of diameter $D=0.50$ m, a fluxural stiffness $E_s I_p=57$ MN.m$^2$ and embedment depth $D=5$ m. The load is applied 1 m above the ground surface. Twelve pairs of strain gauges were fixed on the external surface of the pile, along two diametrically opposite axes (Bouafia, 1990). The measured strains give the bending moment along the pile.

2.3 The soil
The soil was taken from the experimental site Le Rheu, France. After the USCS classification, the grade of the soil is SP. This is therefore a poorly graded sand with an efficient diameter $d_5=0.3$ mm.

The maximum and minimum dry unit weight determined after the ASTM standards, are 16.8 and 13.4 KN/m$^3$ respectively. The sand is pluviated in a container using an automatic hopper. This technique gives a good homogeneity and repetitive characteristics for various tests. The pile model is placed in a container which is 1200 mm long, 800 mm wide and 360 mm high. The sand bank is built up by pluviation around the pile.

2.4 The CPT model
A miniature CPT of 6mm cone diameter is taken on the centrifuge basket. At scale 1/17.8 this model simulates a static CPT with a cone diameter of 107 mm (figure 1). A load cell and LVDT transducer are fixed on the miniature CPT in order to measure continuously the cone resistance at various depths.

![Figure 1 The φ6 CPT](image)

3. EXPERIMENTAL RESULTS
In the following, the presented results correspond to the simulated prototype. The cone penetration testing is carried out at various positions in the sand mass in order to check homogeneity.

Figure 2 shows the penetration profile of a medium density sand ($D=63\%$) which is characterised by constant value beyond a depth of 25 $D$ approximately. This might be the critical depth beyond which $q_c$ remains constant. For dense sand ($D=95\%$), $q_c$ increases considerably without reaching an asymptote of constant value.

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4. ANALYSIS OF HORIZONTALLY LOADED PILES
The horizontal loading of a deep foundation is a relatively complex soil-foundation interaction problem. The three-dimensional behaviour and the number of geometric and geotechnical parameters involved make it difficult to use a rigorous theoretical analysis of the problem. Usually, two categories of design methods are used for analysis of this problem:
- Methods for continuum medium: the soil is considered as an elastic isotropic continuum characterised by the Young’s modulus E and Poisson’s ratio.
- Methods for discrete medium (subgrade reaction modulus): the soil consists of an infinite number of springs characterised by a load transfer curve, called the P-Y reaction curve.

The first category of methods can be applied on the basis of the usual correlation between the cone resistance q_c and the elasticity modulus E. Although very often used, the correlation E/q_c is rather delicate since it relates a parameter for small deformation to another one dealing with failure. The large number of factors involved in such a correlation makes it nowadays a scientific debate.

Table 2 summarises some usual correlations for sand. For certain references, no precision is given about the type of CPT, the characteristics of the soil and the interval of q_c values.

<table>
<thead>
<tr>
<th>Table 2. Values of E/q_c ratio for sand after various authors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference</td>
</tr>
<tr>
<td>Schmertmann (1978)</td>
</tr>
<tr>
<td>Milovitch &amp; Stefanovitch (1982)</td>
</tr>
<tr>
<td>Poulos (1988)</td>
</tr>
<tr>
<td>Verbruggen (1981)</td>
</tr>
<tr>
<td>Elson (1984)</td>
</tr>
<tr>
<td>Van Impe (1986)</td>
</tr>
<tr>
<td>Cassan (1978) and Van Wambeke (1982)</td>
</tr>
</tbody>
</table>

After table 2, the ratio E/q_c for a bored pile varies between 2 and 4.5. The first category of design methods was used to evaluate the pile surface deflection \( Y_s \) under a horizontal load of 50kN. The sand is very dense. The penetration profile from figure 2 is used to evaluate the soil elastic parameters. The measured deflection \( Y_s \) is 12.20mm. It has to be compared to predictions from various authors summarised in table 3.

<table>
<thead>
<tr>
<th>Table 3. Predicted deflection ( Y_s ) from various authors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference</td>
</tr>
<tr>
<td>Randolph (1981)</td>
</tr>
<tr>
<td>Davies &amp; Banerjee (1978)</td>
</tr>
<tr>
<td>Reese &amp; Matlock (1960)</td>
</tr>
</tbody>
</table>
This table shows that it is possible to accurately predict the pile deflection by means of relations considering the cone resistance of the soil. It is illusory to look for a high accuracy for the results since the design parameters are difficult to obtain.

Figure 4 shows a good agreement between the reaction modulus $E_u$ evaluated from $q_u$ and the one deduced from empirical P-Y curves. The reaction $P$ is the second derivative of the measured bending moment $M$ with opposite sign while the deflection $Y$ is the double integral of the latter (Bouafia and Garnier, 1991).

Figure 2  Cone resistance in a horizontal soil

Predictions can be made from a non-linear design method such as the subgrade reaction modulus theory based on the P-Y curves. For sands there are relations between the cone resistance and the pressuremeter parameters. They are as follows (Cassia 1978, Van Wambeke 1982):

$$\frac{E_u}{q_u} = 1 \text{ to } 1.5 \quad (1)$$

$$\frac{q_u}{p_c} = 7 \quad (2)$$

where $E_u$ is the pressuremeter modulus and $p_c$ is the limit pressure.

The P-Y curve linking the soil reaction $P$ to the horizontal pile displacement $Y$ at a given depth can be drawn after Menard recommendations. This curve (figure 3) is three-linear, consisting of two straight lines with slopes $E_u$ and $E_u/2$ successively and a horizontal line at $p_c-B$. $B$ is the pile diameter. $E_u$ is the subgrade reaction modulus which can be calculated from $E_u$. From equations (1) and (2), $E_u$ can be calculated at various depths and the P-Y curves are built up along the pile.

Figure 3  Menard P-Y curve

The P-Y curves are set up in accordance with Menard recommendations and based on the penetration profile. These curves are
thereafter input in the programme PILATE of non-linear design of piles from P-Y curves.

Figures 5 and 6 show that the P-Y curves deduced from \( q_e \) give close values to measured deflection and fluxural moment.

5. EFFECT OF SLOPE ON CONE RESISTANCE
The CPT used has a 12 mm cone diameter and can be automatically positioned during the rotation of the centrifuge.

Figure 7 shows penetration profiles corresponding to various distances from the top of the slope. Therefore when \( t \) decreases, \( q_e \) decreases: this shows the sensitivity of the cone resistance to the proximity of a slope.

After figure 8, at a depth of 1 m, \( q_e \) can be reduced to half of the value corresponding to a horizontal soil.

![Figure 5 Measured and predicted \( V_H \)]

![Figure 6 Measured and predicted bending moment]

![Figure 7 \( q_e \) profiles near a slope]

![Figure 8 Influence factor of slope on \( q_e \)]

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6. PREDICTION OF THE BEHAVIOUR NEAR A SLOPE
Nowadays, the construction needs urge engineers to design audacious works implemented near slopes, such as bridges and marine structures.
Tests on centrifuged small scale models showed a big sensitivity of the pile deflection located in the proximity of a slope (Bouafia & Bouguerra 1995, Teraishi et al 1991). The predicted design assumes that relations described by equations (1) and (2) are yet valid at the proximity of a slope.
The Menard P-Y curves are built up for a pile in a horizontal soil and the same pile at the top of the slope (t/B=0), based on the corresponding penetration profiles shown in figure 7.
After figure 9, the the design of pile from PILATE programme shows a good accordance between predicted and measured displacements, for loads below 100kN. This allows to conclude that it is possible to analyse the effect of the proximity of a slope on the the response of a pile horizontally loaded using the same methods as a horizontal soil and based on the penetration tests in the proximity of the slope.

7. CONCLUSIONS
Tests on small scale models of horizontally loaded piles were carried out on the LCPC centrifuge. The miniature CPT allowed to analyse satisfactorily the pile displacements.
The analysis of cone penetration tests in the proximity of a slope showed that the cone resistance decreases when approaching the top of the slope.
It is possible to use the same method of P-Y curve construction of a horizontal soil for the analysis of the pile behaviour near a sloping soil based on the penetration tests in the proximity of the slope.

8. REFERENCES


CPT IN BRIDGE FOUNDATION AND EMBANKMENT DESIGN

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Frode Oset
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SYNOPSIS: At the Norwegian Road Research Laboratory (NRRRL) we have more than ten years of experience with the use of CPT in ground investigations for road and bridge projects. The CPT equipment has proved to be a valuable tool in obtaining design parameters especially in silty and sandy soils, and even in clay. It can in many cases provide sufficient data to reduce the amount of more expensive piston sampling.

The article focuses on the interpretation of strength and deformation parameters with case histories including embankments and pile foundation design. CPT interpretations are compared with laboratory test results, and design calculations based on CPTs are compared with field observations and other design methods.

1. INTRODUCTION
The first CPT soundings for the Norwegian Public Roads Administration were performed with the site investigations for the bridge over the lake Mjosa in 1973 and in 1980 - 82 (Rygg, 1983). The positive results from this project led to the purchase of our first CPT equipment in 1981, and later on to a more practically applicable equipment purchased in 1987.

Our present CPT equipment is an electronic 3-channel ENVI-cone with cone resistance, pore pressure and friction measurement. The annual amount of CPT soundings is about 1000 metres.

Traditionally, rotary-pressure sounding has been our main tool obtaining information about the overall ground conditions, including the location of quick clay deposits (Rygg and Andresen, 1988). This method is cost-effective, and with its development into the so-called total sounding method during the last years it provides most of the basic information needed (Fredriksen et al., 1992).

For more detailed information on soil layering, and for the assessment of strength and deformation parameters, CPT is a valuable supplement. In cases where it is difficult or very expensive to obtain good results from 54 mm piston sampling, CPTs can reduce the number of samples necessary. This has been the case both for on-land and sub-sea investigations, especially for projects in silty and sandy soils, but even in soft clay.

Penetration capability is a limitation for the CPT equipment due to rod friction, especially when compared to the sounding methods mentioned above. Penetration can be improved to some extent by rather simple means (Oset, 1989), and soundings have been performed to depths of more than 60 metres in clay.

In several projects CPT data have formed a major part of the design basis for:
* bearing capacity of piles
* stability and settlements of embankments
* detailed mapping of soil layering for wellpoints.

Most of the existing interpretation methods are reviewed in the report by NGI (1992). The interpretation methods used by The Norwegian Road Research Laboratory are summarized by

The interpretation of data from CPT soundings and their use in a number of cases is outlined in the following sections.

2. INTERPRETATION OF SOIL PARAMETERS
In the following section we will present a comparison between strength- and deformation parameters obtained by CPT soundings and by other methods based on laboratory testing of soil samples like triaxial and oedometer tests. The same methods for interpretation are used in both cases (Sandven et al. 1988 and Senneset et al. 1988).

2.1 Verdalssøra bridge
Verdalssøra bridge is situated in Nord Trøndelag county, Norway, and is planned as a 178 m long steel beam bridge founded on steel pipe friction piles.

The ground mainly consists of silty sand / sandy silt to depths exceeding 70 m. In connection with the ground investigations CPT soundings were performed, as well as soil sampling with 54 mm piston sampler. The soil conditions and design of bridge foundations are reported by Vastedstad (1995).

As shown on figure 2.1 there were good accordance, on this site, between parameters obtained by CPT and by triaxial / oedometer tests.

![Figure 2.1 Results from Verdalssøra bridge.](image)

2.2 Jungerveien bridge
Jungerveien bridge is situated in Buskerud county, Norway, and is a 69 m long bridge founded on end bearing piles.

The ground consists of soft clay. Depths to bedrock are from 25 m to 57 m along the bridge. Among other soil investigation methods there were performed CPT soundings, triaxial tests and oedometer tests. The ground conditions are reported by Solberg (1994).

As shown on figure 2.2 there is a slightly poorer accordance between parameters obtained by CPT soundings and by triaxial / oedometer tests for the clay material. The interpretation of the CPT soundings gives higher values for the
shear strength parameter, $\tan \varphi$, and lower values for the deformation modulus, $M$.

3. BEARING CAPACITY OF PILES

Large diameter driven steel pipe piles is a common method for the design of highway bridge foundations in Norway.

The most common method for predicting the bearing capacity of piles in bridges in Norway is the Janbu-method (Janbu, 1976). The method of Flaste and Selnes (1977) are also used.

Both of these methods are theoretically based, and there is a need for a method based on in-situ investigations.

Bustamente and Gianeselli (1982) introduced a method based on results of in-situ investigations, namely CPT. The method is based on 197 full scale static loading tests with different types of piles and ground conditions. Most of the static loading tests were carried out by the laboratory network of the French Highway Department (LCPC), and is called the LCPC-method.

The method is based on cone resistance from CPT directly (direct method) and there is no need for evaluating any intermediate values like effective shear strength parameters used in the Janbu-method (indirect method).
The LCPC-method does not directly require the CPT sleeve friction value. This is a desirable feature since the cone resistance is generally obtained with more accuracy and confidence than the sleeve friction.

Robertson et al. (1988) reported predictions from thirteen axial pile capacity methods compared with the results from eight full scale pile load tests on six different steel pipe piles. The direct methods, which incorporate CPT-pile scaling factors, provided the best predictions for the piles and methods evaluated. For the piles tested the LCPC-method was shown to be the best method.

The LCPC-method was used to predict the bearing capacity of steel pipe piles in the foundation of two bridges in Norway.

The first bridge was the Minnesund bridge, reported by Rygg (1991). The ground conditions consists of mainly overconsolidated silty clay with sand layers. CPTU testing were performed.

The steel pile pipes were open ended, 24 m long with diameter 916 mm and steel thickness 16 mm. Static load testing were performed on one of the piles.

The following characteristic bearing capacity Qk were predicted:

- The Janbu method: $Q_k = 4200 \text{ kN}$
- The LCPC-method: $Q_k = 3100 \text{ kN}$

Result from static load testing:

$Q = 1900 \text{ kN}$

The reasons for the low bearing capacity obtained on the open ended steel pipe piles will be investigated further.

The second bridge is Fin涅lstromen bridge, reported by Sleipner (1993). The soil consists of silty clay with undrained shear strength between 40 and 80 kPa predicted from CPTU-testing down to 65 m depth.

Steel pipe piles with length 40 m, diameter 1000 mm had the following predicted bearing capacity:

- The Janbu method: $Q_k = 8000 \text{ kN}$
- The LCPC-method: $Q_k = 6500 \text{ kN}$

The predictions based on the LCPC-method were used in the final design.

4. SETTLEMENT OF EMBANKMENTS

In 1990 - 1991 there were performed ground investigations in connection with planning and building of the new main road E6, from Hommelvik to Varnes in Nord Trondelag county, Norway. The new road consists of embankments, partly along the shore line, as well as a 330 m long bridge crossing the river Sjordalselva and two smaller cross-over bridges. Settlement analysis was performed based on deformation parameters obtained from the CPT soundings.

The ground in the area consists of loose sands, silty sands and silt. A clay layer at 25 - 30 m depth were also registered. The results for one of the boresholes are shown on figure 4.1.
Figure 4.1  Results from Hellstranda.

Using deformation parameters from the CPT, the settlements for a 3 m high embankment was calculated to be 10 - 12 cm. The settlements were expected to be completed in short time due to the loose sand / silt. After the building of the embankment the settlements were measured with settlement plates at regular intervals. The measurements showed that the total settlements were only about 6 cm at this particular site. Settlements measured on other sites on this project (abutment fills) were also smaller than predicted by using the deformation modulus, M, interpreted from the CPT - soundings. As an average the measured settlements were about 50 % of the calculated settlements.

5. CONCLUSIONS
With its limitations on penetration, CPT is not likely to replace the rougher sounding methods like Rotary-pressure sounding and Total sounding, but it is a useful supplement where more detailed mapping of soil layers is needed. In addition, CPT can replace or reduce the amount of piston sampling in many cases.

CPT has proved to be an efficient tool providing soil data for design of bridge foundations, embankments etc. Interpretations of shear strength parameters on effective stress basis have proved to correspond well to laboratory results from triaxial testing.

Deriving deformation parameters from CPTs for use in settlement calculations seem to be a more uncertain task when the calculations are compared to actual observations.
6. REFERENCES


Dynamic Cone Penetrometer Application For Embankment/Subgrade Inspection

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SYNOPSIS: In situ soil moisture content and density are most often measured today with nuclear gauges to assure that adequate compaction requirements are met during highway construction. Direct measurement of soil stability to minimize soil deformations by construction traffic has not been done in a quality control/quality assurance setting.

Frequently, soil moisture - density parameters are attained in the field, but adequate stability under construction traffic loading is not achieved. The Dynamic Cone Penetrometer (DCP) is frequently used by the Illinois Department of Transportation (IDOT) to ascertain subgrade stability during highway construction. The penetration test is conducted by driving a prescribed cone, attached to a drive shaft, with a slide hammer. The Penetration Rate (PR) is then converted to an equivalent California Bearing Ratio (CBR) as a measure of subgrade stability.

Numerous (67) moisture - density/PR field tests were conducted in this study correlating DCP measured stability versus nuclear gauge moisture - density values. Correlations of laboratory measured versus field measured soil properties are also presented. Based on this data evaluation, it is concluded that DCP measured stability parameters can successfully be substituted for moisture - density parameters when monitoring embankment/subgrade construction for final product acceptance.

1. INTRODUCTION

The in situ soil moisture and density are often measured with nuclear gauges for acceptance of embankment materials. The initial cost and periodic maintenance of these nuclear gauges quite often makes it difficult to achieve timely and frequent testing. Also, quite often, silty soils meet the moisture - density requirements, but do not exhibit adequate stability under construction traffic loads.

The dynamic cone penetrometer (DCP) is used by some state and local transportation agencies, including IDOT, to check the subgrade stability during construction. The DCP test is conducted with a U.S. Army Corps of Engineers penetrometer (60-degree cone and 3.23 cm² base area), driven into the soil by dropping an 8-kg weight from a 57.4 cm height. The DCP was successfully used for rapid shear strength evaluation of in situ granular materials (3). Penetration of the DCP cone into the soil is similar to the mechanism of bearing capacity failure which mobilizes the ultimate shear strength of soil. This demonstrates that the DCP can be used as an excellent tool for evaluating soil stability, which is a function of the soil bearing capacity. Because of the similarity in the failure mechanism of both, the DCP and CBR test results could be correlated with each other. The CBR test, in which a circular plunger penetrates into the soil, is another measure of the soil bearing capacity. Because of the similarity in the failure mechanism of both, the DCP and CBR test results could be correlated with each other. Using available empirical relations, the cone PR can be converted into an equivalent value of CBR.
This study presents results from field tests at 67 locations, using the DCP for evaluating the equivalent CBR and the nuclear gauge for measuring the moisture-density values. Samples from 50 of these locations were tested in the laboratory for grain size analysis, Atterberg limits and compaction characteristics. One-point Proctor tests were conducted at 17 locations. This study also presents correlations between the cone index (or CBR) and different soil properties.

Based on this extensive study, it is concluded that: 1) compared to the nuclear gauges, the DCP has the advantages of being inexpensive, requiring minimum maintenance with no health and safety monitoring, and being easy to use, thereby allowing more frequent field testing, 2) when the subgrade soil was considered stable based on the DCP data, the soil almost always met the moisture/unit weight requirements, while the opposite was not observed, and 3) because of the inherent DCP advantages, the DCP test could be used as a valuable tool for providing quick, consistent and reliable results in varying soil types, and for inspection of embankments and subgrades.

2. FIELD TESTING AND SAMPLING
Field testing was typically done in a subgrade/embankment construction setting in order to guarantee that compactive effort had been applied where testing was conducted. The varying site locations tested at different seasons dictated random sampling and testing, insuring that a broad range of soil types were evaluated.

The soil types studied included A-1 through A-7 soil groups as defined by the AASHTO M 145 soil classification system. A-1 soils exhibit the best load carrying capacity, while A-7 soils exhibit the poorest. A-1 soils are granular - nonplastic while A-7 soils are fine grained - highly plastic. Figure 1 categorizes typical soil types historically compiled for a variety of projects located in this study’s geographical location. The Plasticity Index is an indicator of soil type in this correlation.

![Figure 1. AASHTO M 145 CLASSIFICATION vs. PLASTICITY INDEX](image)

The DCP utilized in this study is shown in Figure 2, illustrating the U.S. Army Corps of Engineer's specifications for this instrument (3).

![Figure 2. DYNAMIC CONE PENETROMETER](image)
Nuclear moisture - density tests were first conducted to a 203 mm (8 inch) depth. In situ Standard Dry Density (SDD) and Optimum Moisture Content (OMC) were determined in accordance with AASHTO T 238-86 test method - direct transmission procedure and AASHTO T 239-91, respectively. Test sites were carefully prepared, making sure that the soil surface was flat with minimal voids. After the moisture - density test was completed, the DCP cone was set and driven within the visible imprint left by the nuclear gauge, away from the source rod hole. The cone of the DCP was then driven to a 203 mm depth by dropping an 8 kg. (17.6 lb.) hammer a distance of 575 mm (22.6 in.) onto a drive anvil, and recording the blow counts. Where non-homogeneous soils were encountered, two or more DCP tests were done in the same imprint. When penetration rates of adjacent tests were radically different, a third or fourth test was completed to minimize the observed anomalies. After DCP testing was completed, the volume of soil beneath the nuclear gauge imprint was removed to a 203 mm depth and placed in a cloth bag for later laboratory testing.

3. LABORATORY TESTING
Laboratory testing consisted of conducting the following AASHTO standard tests on the field samples described above:

<table>
<thead>
<tr>
<th>Test Description</th>
<th>AASHTO Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td>Particle Size Analysis</td>
<td>T 88-93</td>
</tr>
<tr>
<td>Atterberg Limits Analysis</td>
<td>T 90-92 &amp; T 89-93</td>
</tr>
<tr>
<td>Moisture - Density Relations</td>
<td>T 99-93</td>
</tr>
</tbody>
</table>

Fifty of the 67 field tested soil samples were subjected to the referenced analyses, while the 17 remaining soil samples were prepared during field testing for a one-point moisture - density test using the “Family of Curves” AASHTO T 272-86 method. When laboratory time was limited and where field tested soils were found to be of reasonably uniform grain size, the one point method was used. The resultant moisture - density point was plotted on the family of moisture - density curves for those soil types indigenous to the geographical location of the related construction project. Subsequently, a moisture - density curve was selected for comparative analysis. If more than one curve satisfied this criteria, then previous field soil classifications/descriptions were also used in selecting a representative moisture - density curve. Equivalent SDD and OMC were determined based on this analysis.

4. RESULTS AND DISCUSSION
The nuclear method (AASHTO T 238-86) for the in situ soil moisture - density testing is an acceptable tool for measuring the desired results for compaction and moisture control during construction inspection. Acceptable soil stability under construction traffic is generally assumed to occur after compacting the soil, when the moisture - density criteria is satisfied. However, when these parameters are satisfied in the field, observed stability of some soils is often insufficient for supporting construction traffic without exhibiting substantial soil rutting deformation.

The IDOT Subgrade Stability Manual (2), developed to establish acceptable levels of subgrade stability, demonstrates that in most fine grained soils a measurable, minimum CBR of 6 to 8 percent is required in order to hold rutting deformations to 13 mm (0.5 inch) or less. In terms of required soil bearing pressure, this is equivalent to 1655 to 2206 kPa of pressure. The same criteria were utilized for defining acceptable stability with the DCP/PR data, which was then correlated with the
nuclear density gauge findings, the procedure of which was defined earlier. The correlations (4) in Figure 3 were implemented for quantifying stability based on the generated DCP PR vs. CBR values.

Figure 3. CBR - DCP ALGORITHM

Figure 4 depicts percent SDD values plotted versus equivalent CBR/PR values. It is apparent from the field data presented that stability is significantly enhanced by increased compaction. The relatively flat trending of data points illustrates that increasing soil density directly influences CBR but the range of influence is comparatively small. This small %SDD range encompasses nearly every CBR value shown, suggesting that stability as a function of increasing SDD is not readily predictable.

Figure 4. CBR vs. % STANDARD DRY DENSITY

Most transportation agencies worldwide, including IDOT, require that soils be compacted to at least 95 percent of SDD. The correlations presented in Figure 4 demonstrate that instability can exist in some soils even when adequate compaction is measured during construction. With the exception of two, all data points plotted within the acceptable 6 to 8 CBR range of stability met the 95% SDD IDOT compaction requirement. The need for any remedial measure to ensure adequate subgrade stability can only be determined by the DCP test data.

One important remedial measure is moisture control in achieving adequate soil stability during construction. Current IDOT specifications do not allow for moisture contents to exceed 120 percent of OMC, except when otherwise specified to be further restricted due to special conditions. Once again, the DCP/PR - CBR correlations in Figure 3 were used for quantifying stability versus percent OMC in Figure 5, which shows the effect of %OMC on the CBR/PR values for the subgrade soils tested.
Figure 5. CBR vs. % OPTIMUM MOISTURE

The correlations presented show that increasing stability is loosely controlled by decreasing moisture content, as illustrated by the pronounced sloping trend of the data points. The range of moisture change that influences stability change is comparatively large. This indicates that predicting acceptable stability (CBR 6 - 8) by %OMC is more reliable than predicting by %SDD because a smaller number of CBR values are intercepted within the required %OMC value range. These correlations also demonstrate that unstable conditions can exist even when measurable moisture contents are within the specified limits. In granular soils, of low plasticities (P.I. <5%) and low fines content (<10% passing U.S. #200 sieve), adequate stability was achieved even when moisture contents exceeded the specified limits. However, for high plasticity soils, stability cannot be achieved unless the compaction moisture content is close to or even less than OMC.

Figure 6. PLASTICITY INDEX vs. %OPTIMUM MOISTURE @ INCREASING CBR RANGES

Figure 6 shows the relation between %OMC and the PI for the range of soil types tested at different CBR/PR values. The trend of data indicates that the allowable %OMC to achieve adequate stability decreases with the increase in a soil’s plasticity. For some low plasticity silty soils, PI <15, stability was achieved only at 100% - 105% OMC, as indicated by the square and diamond data points. For soils with PI >15, stability was achieved at lower moisture contents. Figure 6 also demonstrates by wide point scatter, the ability of the DCP to predictably measure stability as a penetration rate that inversely varies relative to changes in moisture content for most soil types defined by PI, given that compactive effort has been applied. The horizontal banding of individual CBR ranges demonstrates the predictable nature of PR vs. %OMC.

The corresponding %SDD for the same soils is plotted versus the PI in Figure 7, which shows a slight increase in %SDD with PI. This indicates that %SDD necessary to achieve adequate stability should be slightly increased for soils with high plasticity. With a better control of compaction moisture, the desired
%SDD can be achieved. Figure 7 also demonstrates, by narrow data point scatter, that the CBR/PR values are not as predictable as a function of soil densification (%SDD).

Arguably, the data point trends plotted in Figures 4, 5, 6 and 7 suggest that tighter moisture - density acceptance standards should be implemented to insure that adequate subgrade stability is achieved during construction. However, the DCP PR, as influenced in this study by soil moisture - density variations, is easily predictable and reliably duplicatable under field testing conditions.

![Graph showing Plasticity Index vs. %Standard Dry Density at Increasing CBR Ranges]

Figure 7. PLASTICITY INDEX vs. %STANDARD DRY DENSITY @ INCREASING CBR RANGES

5. CONCLUSIONS
The advantages of using the DCP over the nuclear - density gauge for earthwork inspection can be categorized in two areas: stability assurance benefits and practical benefits.

5.1 Stability Assurance Benefits
Based on the correlations developed in this study, moisture - density control does not always assure stability, especially with moisture sensitive soils. The inverse relationship between CBR and OMC (Figure 5) is more significant than the direct relationship between CBR and SDD (Figure 3). Stability, as measured by DCP penetration rates, is more predictable by moisture content than by soil density, and that control of moisture content is therefore more critical for obtaining stability. The DCP PR necessary for achieving adequate stability (minimum 6 - 8 CBR) also indirectly indicates that moisture - density levels are acceptable.

The correlations established in this study support using the DCP for directly measuring soil stability, possibly as a substitute for the established methodology of indirectly measuring stability based on a soil's compaction characteristics. Targeting subgrade/embankment stability as a measured parameter for product acceptance in the future appears to be a very reliable method of testing; however, an expanded data point base may be beneficial in establishing the necessary specifications.

5.2 Practical Benefits
The obvious practical benefits of the DCP over the nuclear - density gauge include:

- Lower cost - The DCP can be purchased for one-tenth to one-twentieth the cost of a nuclear density gauge.
- Low Maintenance - There is very little maintenance required for the DCP while there is periodic maintenance and regulations required for the nuclear density gauge.
- Quick and Easy to Operate - A technician can be trained to operate the DCP in a matter of minutes and can complete five (5) tests for every one (1) completed by nuclear - density gauge.
- Less Subject to Interpretation - The operator of the DCP does not have to always rely on his/her judgment to select the appropriate proctor curve.
More Versatile - The nuclear - density gauge can be used to evaluate the top 203 mm (8 inches) of material, while the DCP can measure stability to a depth of 1 meter. This versatility allows the operator to investigate and determine the limits or source of surface instability.

The DCP has proven to be a valuable tool for evaluating subgrade conditions and monitoring embankment construction and could be used for quality assurance on QC/QA projects in the future.

6. ACKNOWLEDGMENTS
This study was supported and conducted at IDOT’s District 2 located in Dixon, Illinois. Assisting in field testing were Robert Dennis, Brad Dyer, James Westervelt, Dave Biddix and Jason Ayars. Laboratory testing was conducted by Jeff Shipman and Leal McDonald. Special thanks are due to Jennifer Farrell and Shari Wechter for word processing/figure preparation.

7. REFERENCES


The contents of this paper reflect the views of the authors, who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of IDOT. This paper does not constitute a standard, specification or regulation at IDOT.
ENVIRONMENTAL SITE APPLICATIONS OF THE CPT

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Albuquerque, New Mexico, USA

SYNOPSIS: Clean up of the environmentally contaminated sites created over the last one hundred years promises to be one of the most significant engineering challenges of the 1990's. The end of the cold war has allowed nations to place a high priority on developing solutions to these substantial problems. The application of new technology utilizing the cone penetrometer can lead to a dramatic reduction in the cost of the site characterization and remediation process. This new CPT application is gaining wide acceptance in the United States where the U.S. Department of Energy and Department of Defense have sponsored research and demonstration projects to extend, evaluate and promote the technology. This paper reviews the major advantages of the CPT for environmental site characterization, discusses the new sensors and techniques involved and presents a number of comparison with current approaches. These comparisons illustrate that the CPT approach is less expensive, less hazardous for the workers, brings minimal hazardous material to the surface and is significantly faster than other methods.

1. INTRODUCTION
The standard approach to environmental site characterization in the U.S. is process of drilling and sampling followed by laboratory testing. After analysis of the data the process is repeated multiple times until a good description of the site and the contaminant plume is obtained. Monitoring wells are then placed in areas thought to be both contaminated and uncontaminated. These are observed over a period of time to confirm the nature and location of plume and to monitor the remediation process. Since these wells are quite expensive, the number installed is usually rather limited. This process is very slow due to the waiting time for laboratory analysis of the samples and the repeated mobilizations to the site. In addition, the monitoring wells often are not in the optimum locations due to an incomplete understanding of the site because of the limited data available. This requires the placement of additional monitoring wells. U.S. regulations however, prohibit the abandonment of the non-useful wells therefore, the cost of continuing to monitor these wells increases the inefficiencies of this approach.

Adaptation of the CPT to environmental site characterization has required the development of additional equipment. These include: 44.5 mm rods, decontamination equipment, water and soil gas sampling equipment, grouting fixtures and special chemical sensors. In addition, to penetrate through the contaminated zone it is often necessary to penetrate to quite deep depths. This often requires a CPT unit with 30-40 tons push capacity. The larger rods provide the strength capacity for the heavy loads while also allowing the placement of sampling wells, allow the insertion of down hole pumps and more internal
space for chemical sensors.

2. SAMPLING
The standard CPT measurements, of course, are of critical importance in environmental site characterization. The continuous nature of CPT measurements provide data only available from drilling operations by the expensive process of continuous sampling. This is particularly expensive at contaminated sites because all material taken from the hole must be tested and if contaminated it must be disposed of at an approved hazardous material landfill. Thin layers of permeable material provide pathways for movement of contaminates. Continuous measurement provide the only way to locate these layers.

2.1 "PERMANENT SAMPLERS"
A schematic of a standard drill rig placed monitoring well is shown in Figure 1. Typical costs to place wells to the U.S. Environmental Protection Agency specifications to a depth of about 14 meters run about $1250 U.S. including developing the well and disposing of the cuttings. After placing these drilled wells they must be "developed" by removing several well volumes of water to remove the drilling fluids and insure that the samples are not influenced by any drilling or well materials. Regulations require that these wells be sampled quarterly for an indefinite time period. Figure 2 shows a monitoring well placed with a CPT. Since is is pushed in there is no need for the filter pack and bentonite seal. This well may be placed at a cost of about $200.

When difficult soils must be penetrated the PVC can be placed inside the 44.5mm rods. Sampling is done with a bailer or a down hole pump.

2.2 TEMPORARY SAMPLING
For temporary or one time water samples a device like that shown in Figure 3 is often used with the CPT. Samples are extracted as in the
monitoring wells. After taking the sample the rods are withdrawn and the hole grouted through the open pipe.

A number of soil gas sampling techniques are available. Figure 4 shows one example. This sampler allows continuous sampling as the probe is advanced. The gas samples are drawn to the surface through a standard pore pressure type filter with a vacuum pump. The gas may then be tested with a number of devices inside the CPT track. Figure 5 shows data from a site contaminated with volatile organic compounds. The variation of the concentration of VOC's were confirmed with samples taken with standard procedures at discrete depths.

Figure 4 also shows another device required for environmental site work. In most cases it is necessary to grout the hole on withdrawal of the rods. The sliding ring slips off when the rods are withdrawn and grout is allowed to flow through the tube to seal the hole. Figure 6 shows a unit for decontamination of the rods as they are withdrawn. The rod is drawn into a chamber through a boot. This boot removes any soil adhered to the rods. Steam is injected into the chamber and the contaminated water is drawn off into a storage container.

Soil sampling also plays an important role in environmental site characterization. The equipment used is the same as in geotechnical work.

Figure 3. Water sampler for use with 44.45-mm OD cone rods.

Figure 4. Schematic of soil gas probe.

Figure 5. Example of Downhole VOC measurements.
2.3 IN-SITU SAMPLERS

A third class of samplers which promises to significantly enhance the utility of the CPT approach to environmental site investigations are devices which allow a gas or water sample to be brought into the probe, subjected to a chemical test and then expelled and the chamber decontaminated. For gas samples an inert gas can be forced through the chamber and for water samples either distilled water or a neutralizing agent may be used. Several devices of this type are under development or in the evaluation process. This type of device has the potential of reaching the regulatory limits of sensitivity and will allow testing for a wider variety of chemicals.

3. SENSORS

It is the new chemical sensors developed for or adapted to the CPT which have made the technology ideal for environmental work. Table 1 shows the sensors available today and a number of those under development. A number of these new sensors are the subject of papers in these proceedings, Pluijmgraaff, Hilhorst and Bratton (1995), Shinn and Bratton (1995). Discussion of all these sensors is not possible given the length restrictions of this paper. A few of the most promising which have advanced to the field evaluation stage.

Fluorescence is one of the newest methods of identifying aromatic hydrocarbon contamination. A sapphire window is used to allow the light to illuminate the soil is shown in Figure 6. This type of window is used with most of the optical sensors. Shinn and Bratton (1995) discuss these sensors in more detail.

The gamma probe is one of several special purpose probes which have been adapted to the CPT. Figure 7 shows data from a former nuclear fuel production site obtained with a sodium iodide crystal. The survey of this site with this device resulted in a reduction of about 20% in the volume of material which had to be removed for treatment. The continuous nature of the measurements were very valuable in mapping the extent of the contaminated soil. At another U.S. Department of Energy site a survey involving 90 soundings to an average of about 10 meters with a gamma probe was estimated to have saved over $500,000 compared to doing the work using Standard Penetration Tests samples and measurements in the laboratory, Chernikoff (1995). A significant portion of this cost was associated with the handling and disposal of the radioactive soils.

Soil moisture measurement is useful for geotechnical as well as for environmental projects. A number of different measurement approaches are being implemented. A device using a frequency domain approach is presented in this conference, Pluijmgraaff, Hilhorst and Bratton, (1995). A soil moisture sensor based on the time domain reflection principal has
Table 1. CPT Environmental Sensors

<table>
<thead>
<tr>
<th>SENSOR</th>
<th>MEASURES</th>
<th>STATUS</th>
<th>APPROACH</th>
<th>COMMENTS</th>
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<tbody>
<tr>
<td>pH</td>
<td>Alkalinity of saturated soil environment</td>
<td>Available</td>
<td>Antimony Electrode</td>
<td></td>
</tr>
<tr>
<td>Wave Speed</td>
<td>P and S Wave Velocity (Downhole)</td>
<td>Available</td>
<td>Source at surface, receiver in zone</td>
<td>Geotechnical applications also</td>
</tr>
<tr>
<td>Resistivity</td>
<td>Soil matrix conductivity</td>
<td>Available</td>
<td>Induced Current</td>
<td>Several types available</td>
</tr>
<tr>
<td>Temperature</td>
<td>Soil Temperature</td>
<td>Available</td>
<td>Thermocouple</td>
<td></td>
</tr>
<tr>
<td>Gamma Radiation</td>
<td>Gamma radiation level in soil</td>
<td>Available</td>
<td>Sodium iodide crystal with</td>
<td>Improvement and other concepts under development</td>
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<td></td>
<td></td>
<td></td>
<td>spectral analysis</td>
<td></td>
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<tr>
<td>Fluorescence</td>
<td>Amount of hydrocarbons present in soil matrix</td>
<td>Available</td>
<td>Nitrogen, Neva, or Mercury Bulb</td>
<td>Several different laser systems and optical configuration available, bulb induction system under development</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Source, Isotopex, CCD or</td>
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<td></td>
<td></td>
<td></td>
<td>Photodiode Detectors</td>
<td></td>
</tr>
<tr>
<td>Soil Moisture</td>
<td>Moisture Content</td>
<td>Field</td>
<td>Dielectric, Conductivity, or Time</td>
<td>Different concepts under development</td>
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<td></td>
<td></td>
<td>Evaluation</td>
<td>Domain Reflect in Frequency</td>
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<td>Domain</td>
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<td>ORP</td>
<td>Redox Potential</td>
<td>Field</td>
<td>Evaluation</td>
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<td></td>
<td></td>
<td>Platinum Electrode</td>
<td></td>
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<tr>
<td>Raman</td>
<td>TCE &amp; PCE at high levels</td>
<td>Field</td>
<td>Evaluation</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>514.756 nm laser, CCD Spectrometer</td>
<td>Can also be used for fluorescence</td>
</tr>
<tr>
<td>Ground Penetration</td>
<td>Radar Image of Subsurface</td>
<td>Advanced</td>
<td>Evaluation</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Evaluation</td>
<td>High energy light induced decomposition</td>
<td></td>
</tr>
<tr>
<td>Laser Ablation</td>
<td>General/Chemical</td>
<td>Laboratory</td>
<td>Development</td>
<td></td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>Development in truck</td>
<td></td>
</tr>
<tr>
<td>Vapor-Phase TCE</td>
<td>TCE in soil vapor</td>
<td>Laboratory</td>
<td>Down hole desorption, Fast G-C</td>
<td></td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>development in truck</td>
<td></td>
</tr>
<tr>
<td>FTIR</td>
<td>Chemical (chlorinated hydrocarbons)</td>
<td>Laboratory</td>
<td>Development</td>
<td></td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>Development</td>
<td></td>
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</tbody>
</table>

![Figure 7. Radiation Measurements from a U.S. DOE site.](image)

also been evaluated in the field, Knowlton et al. (1995). This device performed well except in low moisture vadose zone soils in the southwestern U.S.

4. REMEDIATION

Utilization of the CPT in situ remediation is just beginning and has the potential for producing significant savings as well. Potential applications include injection of steam through the CPT rods for steam enhanced recovery from the vadose zone, injection of nutrients for bioremediation, providing electrodes for thermal enhancement, and providing multiple wells or new wells as a plume recedes to keep the extraction wells at the points of highest concentration. There is also the potential of using the CPT for placing cutoff or reactive barriers without generating any wastes.
5. CONCLUSIONS
Cone Penetration Technology is rapidly advancing as an alternative to current methods of environmental site characterization. It can produce more information on the site stratigraphy at lower cost. The new sampling devices make it possible to obtain more samples for laboratory or on-site analysis at lower cost and in a more timely manner. Sensors available now and under development make it possible to do continuous field screening of contaminants in real time. This allows a much more efficient site investigation and can prevent multiple trips to the site. In addition, the field crew’s exposure to hazardous materials is greatly reduced and very little soil or water which must be treated as hazardous material is brought to the surface. The key to realizing this potential is the acceptance of the procedures by the regulatory authorities.

6. REFERENCES


A Case Study on Determination of Pile Capacity Using CPT

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SYNOPSIS: The detailed site study using CPT in design of the foundations for a major housing development with 500,000 square meters floor area in İzmir, Turkey are presented in this paper. CPTs and pile load tests are performed to obtain an optimum pile design. Due to the presence of soft clay deposits to large depths, 65 cm diameter vibrex piles with lengths between 35.0 and 40.0 m in different zones are installed. At early stage, various test piles are constructed and loaded up to failure to estimate the ultimate pile capacity and to have a comparison basis with relevant CPT tests. The pile lengths for various zones of the construction area are optimized with the evaluation of the load test results and final pile lengths in the field are determined based on the driving criteria which was developed as a result of wave propagation analysis. Subsoil conditions at the site are improved by means of preloading. The settlement under the fill is monitored by settlement columns and lateral load capacity of piles are estimated using the shear strength parameters of the improved subsoil conditions using the results of the CPT testing after the realization of settlement under the fill.

1. INTRODUCTION

Geotechnical problems and utilized solutions for the foundations of the second phase of a major housing development with a total three thousand units exceeding 500,000 square meters floor area are investigated. The housing development consists of high rise towers up to 22 stories. The project is divided into five zones each constructed by a different contractor. The building types for each zone are summarized in Table 1.

The site is situated on the shoreline at the estuary of a major river flowing into the Aegean Sea which has recently been diverted to open the area for housing construction. Deep alluvial deposits govern the subsoil conditions. Soil borings and CPT tests have been performed for the determination of initial subsoil conditions. Driven cast-in-situ vibrex piles in 65 cm diameter with lengths between 35 m and 40 m have been selected based on the determined subsoil conditions.

Vibrex piling was very fast compared to other methods, consequently great number of piles were able to be installed in a relatively short period of time.

Optimum pile capacity for different zones of the construction area is achieved with the evaluation of pile load tests performed up to failure on test piles constructed at the design stage of construction. Wave equation analyses have been performed to determine the driving criteria of the vibrex piles. The final lengths of the piles are determined at the site during construction according to the given criteria.
Table 1. Layout of Buildings (ZETAS, 1994a)

<table>
<thead>
<tr>
<th>Contractor / Zone</th>
<th>Site Area (m²)</th>
<th>Number of Stories</th>
<th>Number of Buildings</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>21500</td>
<td>22</td>
<td>5</td>
</tr>
<tr>
<td>B</td>
<td>21500</td>
<td>22</td>
<td>5</td>
</tr>
<tr>
<td>C</td>
<td>45000</td>
<td>8</td>
<td>21</td>
</tr>
<tr>
<td></td>
<td>60000</td>
<td>13</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>60000</td>
<td>22</td>
<td>12</td>
</tr>
<tr>
<td>D</td>
<td>74000</td>
<td>8</td>
<td>29</td>
</tr>
<tr>
<td>E</td>
<td>45000</td>
<td>8</td>
<td>21</td>
</tr>
</tbody>
</table>

A certain number of piles among the constructed ones are randomly selected and these piles are loaded to 1.5 times the design load as part of the quality control process.

Subsoil conditions at the site are improved by means of preloading. The settlement under the fill is monitored by settlement columns and lateral load capacity of piles are estimated using the shear strength parameters of the improved subsoil conditions and the results of the CPT testing after the realization of settlement under the fill.

2. SUBSOIL CONDITIONS

Total of thirty nine borings were performed at the initial stage of investigations. The scale of the project, erratic subsoil conditions and variety of structural loads made it necessary to utilize comprehensive CPT testing to be performed within limited time of the construction schedule.

At the initial stage total of forty CPT’s up to bearing strata have been performed to determine subsoil conditions and pile capacity. Five typical CPT soundings for different zones of the construction area are given in Figure 1. The subsoil stratification present in the site with consequent foundation behavior is outlined below (ZETAS, 1994a).

- A preloading embankment of 3.0 m was constructed at early stages in order to improve the soft subsoil.
- The topmost layer below the fill is soft clay with thickness up to 18.0 m. This clay layer creates the major problems in terms of the pile foundations. The settlements that are expected to occur in this strata under fill create negative skin friction on the pile shaft and reduce the pile capacity considerably.
- Below the clay layer exists a sand layer with varying thickness. The presence of the sand layer helps in the dissipation of the excess pore pressure occurring due to the fill. Negative skin friction depth will be limited with the upper clay layer and will not extend to deeper strata because of such a dissipation.
- Stiff hard clay and dense gravel are present below 30.0 m depths which contribute to most of the pile capacity.

3. PILE DESIGN

Vertical pile capacity is determined utilizing CPT soundings and the soil stratification determined from soil borings. A summary of the results for pile capacity for each zone is given in Table 2. The pile capacities calculated are for the depths of the CPT’s performed, final pile capacities are determined from evaluation of the above results based on pile load tests.

3.1. Pile driving criteria

Vibrex piles with 65 cm diameter are installed as the foundation of the residential buildings. Pile lengths for different zones of the construction area range between 35.0 m to 40.0 m. A minimum pile length is specified for each zone to guarantee that the pile is socketed to the bearing stiff clay or gravel strata.

The pile driving procedure is modeled and analyzed by means of wave equation analysis. The number of blows per 25 cm penetration representing a certain energy is specified as pile driving criteria. It is stated in the criteria that the pile is driven a minimum length specified and it

<table>
<thead>
<tr>
<th>Contractor / Zone</th>
<th>Pile Capacity (kN)</th>
<th>Pile Negative Skin Friction</th>
<th>Downhole Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total</td>
<td>Skin</td>
<td>Total</td>
</tr>
<tr>
<td>A</td>
<td>1200</td>
<td>200</td>
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<td>2800</td>
<td>200</td>
<td>3000</td>
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<td>C</td>
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<td>200</td>
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<td>D</td>
<td>3280</td>
<td>200</td>
<td>3500</td>
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<tr>
<td>E</td>
<td>3520</td>
<td>200</td>
<td>3720</td>
</tr>
</tbody>
</table>
Figure 1. Typical CPT Soundings for the Zones of the Construction Area
performed at various zones of the site are shown in Figure 3.

Two unloading runs are made to see the plastic settlement behavior of the pile and the pile is loaded up to failure to determine the maximum capacity. The load settlement curves are divided into two parts, first a flatter section which is considered to be the range of settlements where skin friction governs. The second section of the curve after the break until failure is a measure of the developed tip resistance (Fellenius, 1980).

The negative skin friction that is likely to develop on the piles is used in the determination of allowable pile load from pile load tests. The depth of the soft clay layer that causes negative skin friction for the piles is estimated from the soil stratification determined from borings. The estimated negative skin friction will result in a reduction in the capacity of the pile and the same amount will act as a load on the pile shaft. The safe capacity of the piles are calculated with a factor of safety against bearing capacity $FS = 2$.

The results of the pile load tests for different zones of the construction area are summarized in Table 3.

### 3.3. Quality control tests

The quality of the piles are monitored at construction stage with several procedures. A
Table 3. Summary of Pile Load Tests

<table>
<thead>
<tr>
<th>CONTRACTION ZONE</th>
<th>FLY</th>
<th></th>
<th>DIP</th>
<th>NPSF</th>
<th>NPSF</th>
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<th>CONTRACTION ZONE</th>
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<th>NPSF</th>
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</table>

NSF/D: Negative Skin friction Depth NSF: Negative Skin Friction

certain number of piles (1 out of 100) are randomly selected among the constructed ones and these are loaded to 1.5 times the design load and the results of the tests are evaluated to check the settlement and vertical load capacity of the constructed piles.

The load settlement curves for three zones of the area under construction are given in Figure 5. The evaluations of the test are given below.

- The tested piles safely carries the applied load which is 1.5 times the design capacity.
- The settlements of the piles under the applied maximum load are in the range of 2.2 mm to 4.2 mm and the plastic settlements of the piles when it is unloaded are in the range 0.6 mm to 1.6 mm. Such settlements are within tolerable limits for the safety of the upper structure.

4. SOIL IMPROVEMENT AND MONITORING
The site is situated at a seismically active zone and this makes it very critical in terms of lateral pile capacity. The initial subsoil conditions resulted in inadequate lateral pile capacity and the construction area is loaded with a preloading embankment of 3.0 m height at the design stage to improve the subsoil conditions.

It has been compulsory to utilize the improved geotechnical parameters in the lateral pile capacity analyses and settlement columns are installed to assess the amount and rate of settlement. Settlements are measured with the vertical movement of magnetic rings within the settling layer.

A typical result of settlement monitoring is shown in Figure 5. Unfortunately the contractor was late in the installation of the instruments and the first readings were obtained 9 months after the construction of the fill. It is observed from the shown data that the settlement in the subsoil is about to stop and some heave is observed at the surface due to piling activity at a nearby location.

The settlement monitoring results have indicated that the settlement under the weight of the fill had been realized. The settlements have been monitored to be realized under the weight of the fill. Therefore, strength parameters of the improved subsoil conditions are used in the lateral pile capacity calculations (ZETAS, 1995).

5. SUMMARY AND CONCLUSIONS
The detailed site study using CPT in design of the foundations for a major housing development with 500,000 square meters floor

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Figure 4. Pile Load Tests for Quality Control
area in Izmir, Turkey are presented in this paper. Deep alluvial deposits govern the subsoil conditions. CPT soundings have been utilized in the determination of initial subsoil conditions and related pile design. At early stages various test piles are produced and loaded up to failure to estimate the ultimate pile capacity.

Vibrex piles in 65 cm diameter with lengths between 35 m and 40 m have been chosen for different zones of the construction area. Wave equation analyses have been performed to determine the driving criteria for the vibrex piles.

Subsoil conditions at the site are improved by means of preloading. The settlement under the fill is monitored by settlement columns and lateral load capacity of piles are estimated using the shear strength parameters of the improved subsoil conditions and the results of the CPT testing after the realization of settlement under the fill.

6. REFERENCES


A Case Study on Determination of Soil Improvement Realization Using CPT

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SYNOPSIS: This paper presents the site reconnaissance survey for the Borçelik Cold Rolling Mill Factory, a joint investment of about 200M US$ by Turkish, French and Italian steel producers in Turkey, related to the identification and geotechnical modeling of erratic subsoil conditions by means of CPT. Soft sensitive clays and loose saturated sands present major problems for the foundations of the factory and the technological units. The seismicity and subsoil conditions indicate a risk of soil liquefaction. Therefore stone columns are utilized to provide soil improvement against liquefaction. They also served the purpose of reducing settlements and increasing bearing capacity under high surcharge loads especially in storage areas. The spacing of stone columns is optimized by utilization of CPT prior to and after implementation of column construction. In addition, large scale zone loading tests by means of a rigid reinforced concrete plate with dimensions of 4m by 4m were performed prior to and after the application of stone columns to observe the settlement and load bearing capacity of the subsoil and to assess the degree of improvement with the installation of stone columns. In this paper critical evaluations of the stone column improvement are based on the interpretation of CPT's and large scale zone loading tests.

1. INTRODUCTION

Borçelik Cold Steel Rolling Mill is constructed by ASKON Construction Corp. The foundation design, design of soil improvement and consulting during foundation construction of factory buildings and technological units are carried out by ZETAŞ Earth Technology Corp. Seismic and geotechnical investigations of the site have shown that the zones with liquefaction potential and subsoil with settlement problem under large surcharge are present in the factory area (Ansal, 1990). The soil layers below the natural ground level down to 12.0 m depths are designed to be improved with stone columns using the results of comprehensive CPT testing. Total of ninety CPT tests covering about 2500 m length are performed for identification of soil conditions and for assessment of the results of the soil improvement. In certain areas where the bearing strata could be reached at reasonable depths stone columns up to bearing strata are constructed to eliminate the bearing capacity and settlement problems in storage areas with large surcharge loads. However, in some areas stone columns were terminated without reaching the bearing strata. The factory buildings and foundations of the technological units are founded on 50-60 cm diameter vibro and 80-120 cm bored piles due to large structural and seismic loads and weak/compressible subsoil conditions. The general layout of the site and areas with stone column application is shown in Figure 1.
2. SUBSOIL CONDITIONS
The subsoil conditions at the site are determined by means of soil borings and CPT testing. Basically five different soil/rock units are identified for realization of geotechnical profile.

Formation A: Sandy silty overconsolidated clay is present to 4.0m depth, the thickness extends up to 6.0m-7.0m in the north sections
Formation B: Loose-medium dense sand, contains occasional gravel lenses, Dr = 40-60%
Formation C: Normally consolidated clays, locally sensitive and having maximum thickness of 10.0m
Formation D: Overconsolidated clays, gravelly in some regions, stiff to very stiff
Formation E: Bedrock, extensively weathered when exposed to surface

Four different zones are differentiated in foundation evaluations and design in the factory area taking the distribution of the soil layers introduced above into consideration. The location of the zones described above and the depth contours of the bedrock from sea level with other pertinent information are also shown in Figure 1.

Zone A: Formation E (bedrock) is very shallow or exposed at ground surface. Generally represents the southern portion of the west side of the factory.
Zone B: Formation E (bedrock) is in 10.0m or larger depth from ground surface, southern and middle portion of the site is within Zone B. In this zone formations A and B take place just above the formation E. Formations C and D are locally observed as intermediate layers.
Zone C: Overconsolidated formation D exits above formation E (bedrock). Formation C is thicker than other regions and generally represents the southwestern portion of the site.
Zone D: Soil stratification is like zone C but bedrock is deeper and in most of the region no bedrock is encountered. It represents the northern portion of the site.

3. SOIL IMPROVEMENT DESIGN
For the purpose of improving the subsoil conditions against liquefaction potential (Ansal, 1990) and to increase the bearing capacity at storage areas, stone columns are implemented. As a trial basis, in Zone C, 0.50m diameter stone columns with 2.0m spacing up to depth of formation D, in Zone D, 0.50m diameter stone columns with 1.5m spacing extending to 12.0m depths from ground level are implemented (ZETAS 1991a).

3.1. Assessment of soil improvement
Cone Penetration Testings and Zone Loading Tests have been performed to predict the degree of the soil improvement and to compare with final design requirements.

The area of stone column application is divided into zones each having 200 to 400 stone columns and six CPT’s in total are conducted in each zone, two of them being before, and four of them after the stone column application with the configuration shown in Figure 2. CPT’s are performed with 200kN capacity electrical piezocone and mechanical cone.

Four zone loading tests have been performed in two different zones in the plant area, two of them in zones without stone columns, and two of them in zones with stone columns. The explanations and examples for these tests are given below.

3.2. CPT tip resistance requirements for soil improvement
The in situ relative density estimated in the range of 40-60% is aimed to be improved to at least 75% with the application of stone columns in zones of saturated sands against the soil liquefaction.

The limiting CPT tip resistance curve with depth to satisfy this requirement is estimated by various procedures (Robertson and Campanella, 1988, ZETAS, 1991b) and represented in Figure 3.
4. CPT TESTS AND EVALUATIONS IN A TEST ZONE

The location of the test zone 5 for the assessment of soil improvement is shown in Figure 1. Two CPT's before and four after the construction of stone columns are performed in this testing zone with the configuration seen in Figure 2 as described earlier for the purpose of assessment. The comparison of the penetration tests CPT5/1 (before improvement) and CPT5/5 (after improvement) are presented in Figure 3.

The typical tip resistance, skin resistance and friction ratio variations with depth for CPT5-5 are also presented in Figure 3. The evaluations related to these graphs are as follows (ZETAS, 1991b):

- Below the 2.0m thick working platform at 2.0m-3.0m and 5.0m-8.0m depths no important difference is observed at clayey layers with friction ratio FR > 3% for CPT tip resistance values before and after improvement. In other words application of stone columns does not improve soil strength in clay layers with FR > 3%. It has been observed in other regions that CPT tip resistance has decreased in sensitive clay layers with stone column application.
- CPT tip resistances measured after stone column application are in average 73% higher than the CPT tip resistance values before improvement in sand-silt layers with FR < 3%, present in 3.0m-5.0m and 8.0-15.0m depths.
- Trial spacings of stone columns were found to be effective to satisfy the design requirement of D, min 75% by means of measured CPT tip resistance curve after the improvement.
- It has been observed that there is an increase in CPT tip resistance in tests performed after a time interval compared to tests performed immediately after stone column application.

The first three observations are very common and are as expected. On the other hand observation four is thought to result from dissipation of pore water pressure in time. In order to determine this, piezoecone is used in some tests in the measurement of pore water pressure. It has been observed that CPT tip resistance increases in time as measured pore pressure decreases to its hydrostatic value. As a result it has been determined that it would be most appropriate to measure the final soil improvement by CPT's performed after a time interval to allow for the dissipation of excess pore water generated with construction of stone columns.
5. ZONE LOADING TESTS

Stock areas with 20, 50, 100 and 200 kN/m² surcharge loads are present within the Cold Steel Rolling Mill Factory area. Large dimension loading tests have been conducted especially in zones with high surcharge loads to investigate the soil bearing capacity and elastic/plastic settlements under such loads. Settlements have been measured in zone loading tests under loads up to 430 tons (~270 kN/m² base pressure) applied with steel rolls symmetrically placed on a reinforced concrete rigid plate with dimensions of 4.0m by 4.0m. Load settlement graphs are given in Figure 4, for two zone loading tests performed at one location before and after stone column application. The evaluations for the two performed zone loading tests are given below (ZETAS, 1992).

- Both zone loading tests show similar load-settlement behavior up to ~300 tons, 190 kN/m² base pressure. But after base pressure of 190 kN/m², settlements show a sudden increase in the zone without improvement while such an increase is not observed at the zone with stone column application.
- The increase in settlement after 190 kN/m² indicates plastic deformations at the subsoil in the zone without settlement columns and finally exhibit a general bearing capacity failure in the subsoil above this stress level.
- In the view of the above evaluations it has been determined that the storage areas up to 100 kN/m² surcharge is not to be improved for settlement and bearing capacity. Factor of safety for bearing capacity in the zone is in the order of 2.0 and it is satisfactory for support of steel rolls. However in zones with surcharge above 100 kN/m², such as surcharge of load 200 kN/m², if no soil improvement is implemented bearing capacity failure will likely to occur. Therefore stone columns are constructed at such zones with high surcharge as part of the soil improvement program.
- The results of zone loading tests have later been utilized in soil modeling and determination
of allowable base pressure for shallow footings of office buildings located at similar subsoil conditions at the site.

6. SUMMARY AND CONCLUSIONS
Application of stone columns for soil improvement in Borçelik Gemlik Cold Steel Rolling Mill are outlined in this paper. The application of CPT tests, piezocore tests and large scale zone loading tests with 4.0m by 4.0m dimensions are realized for the assessment of soil improvement. It has been shown with examples that soils with large settlement and bearing capacity problems can also be treated with stone columns. In addition to improve their strength against soil liquefaction, the procedure for the measurement of soil improvement in different types of soils is explained and critical evaluations are made based on the comparison of CPT and Zone Loading Tests. In this case study CPT testing once more proved to be very beneficial with a site of most complex conditions having very poor and nonhomogeneous subsoil, high groundwater table, high seismicity and a factory building and technological units with high dead and surcharge loads and very strict settlement tolerances.

7. REFERENCES
Evaluation of Soil Properties by CPT.

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SYNOPSIS: The present article analyzes the use of CPT on various soils and the dependence of parameters fixed in the course of CPT and plate load tests and pile tests on q.

The beginning of the extensive use of plate load and pile load tests was in Estonia in 1965 and by the present moment a considerably large data base of those tests on different soils has been formed. Usually those tests have been accompanied by CPT-tests and correct laboratory investigations and at the moment the opportunity to compare the results achieved by different methodologies has been cropped up.

By plate load and pile tests the dependence between the pressure (load) and settlement was fixed, likewise the limit of soil proportionality q_p, the ultimate load q_u and the deformation modulus of the soil under pressure q_s. E_p. At pile tests the proportionality limit N_p was fixed: the side friction N_s and ultimate load N_u of the pile. To evaluate these points the methodologies [1] of temporality was used. Basing on the last-mentioned methodologies the index of non-linear deformation of the soil has been fixed with a part of the tests, the index characterizes the settlement of the plate under pressures over q_u.

In case of pile tests the methodologies of temporality is the basis in evaluating the settlements of pile foundations by the load over N_u and under N_s. The above-mentioned indices have been compared to the results of settlement observations.

The results of a few hundred field tests and a hundred pile tests have been analyzed and compared with CPT tests in the present work.

1. SANDS

Taking into account the matter that sampling and fixing the real shear parameters from sands is complicated, in 1966 [2] in Estonia in evaluating the bearing capacity of sands the use of plate load tests was started. With the help of special investigations it was established that q_s depends relatively little on the plate’s area (the plates with the area of 500...24000 cm² were investigated) and its size may be the basis of evaluation of the pressure permitted to sands. q_s is influenced by the size of the plate in correspondence with the ultimate limit theory.

To evaluate parameters E_p and E_s non linear deformation moduli are fixed. q_s has been the basis of introducing the settlement methodologies based on the tests by L. Bierrum [3].

\[ S = 1.5 \cdot k \cdot \sqrt{B} \]  

where \( k = q / q_s, q \leq q_s \),  
B - the width of the foundation in m,  
S - the settlement in cm,  
q - pressure under foundation.

All the field load tests are usually accompanied with sampling and CPT tests. The samples are taken from survs with metal rings, trying to preserve their structure as well as possible.

From these samples the shear parameters on the creep limit \( \tau_c \) (\( \phi_c \) and \( C_c \)) and the maximum shear strength (\( \phi_s \) and \( C_s \)) have been fixed [4]. Using the formulas of the limit theory and using \( \phi_s \) and \( C_s \) the values of \( q_s \) were fixed and
the values of \( q_r \) fixed by \( q_s \) and \( C_r \), whereas the calculations have shown a better correlation in case of \( q_r \).

The comparison of results of plate load tests and CPT has shown a relatively good correlation between the results of these tests. In various years 1972–86 [5] and analyzing the test numbers the following results have been achieved:

\[
q_s = q_r / k, \quad k = 16...20, \\
E_s = 3...4 \cdot q_s, \\
q_r = 50...60 \cdot E_s, \\
E_{st} = 0.8...1.2 \cdot q_r,
\]

where \( q_r \) - cone’s resistance.

The correlation factors of all of the given dependences are \( \eta = 0.7...0.95 \) and relatively better correlations characterize test plates 2500 and 5000 cm². \( \eta = 0.9 \). In case of screw plates of 600 cm² the correlations are worse \( \eta = 0.7...0.75 \), but here the results are influenced by the deformation of screwplate rods.

The utilization possibilities of these gained dependences are affirmed by the settlement observations. The settlements of plate foundations and oil tanks calculated by the formula (1) coincide with the sizes fixed by observations. Under the oil tanks of Maardu there were the sands containing organic matters \( q_r = 2.0 \) MPa and \( q_s = 0.12 \) MPa (2500 and 5000 cm² - were fixed with the test plate). The medium pressure under the oil tank was \( \eta = 0.11 \) MPa and the calculated settlement was 12.5 cm. The actual settlement was 11.1–13.5 cm. The sands under the ministry building in Tallinn were \( q_r = 1.1...13 \) MPa and \( q_s = 0.7 \) MPa (plates 1000–5000 cm²). The load under the raft foundation is 0.26 MPa. The calculated settlement is 2.8 cm and the measured settlement is 2.2 cm.

On loading the sands with the pressure over \( q_r \) the non-linear deformations occur. To evaluate the deformations the values of non-linear deformation index are used. In case of sands containing the organic material with standard approach it is very simple to surpass \( q_r \) and to hit upon the area of non-linear deformation development. On investigating the sands containing colloidal organic matter in the laboratory their shear parameters are evaluated greater than they actually are. The only opportunity to evaluate their behaviour is to base on field load tests. Under the administrative building in the centre of Tallinn there was much sand containing organic matter \( q_r = 1.5...2.0 \) MPa, \( q_s = 0.1 \) MPa, \( E_s = 5 \) MPa and \( E_{st} = 2 \) MPa.

The pressure \( q \) resulting from the raft foundation on these sands was 0.14 MPa \( > q_s \) and the building settled up to 350 mm that corresponded to the building’s behaviour in the non-linear area and was fixable by the above-mentioned \( E_s \) values. Using \( E_s \) here should have given the settlement 150 mm and using the formula (1) in the linear area \( \cdot 80 \) mm.

It is characteristic to the sands containing the organic matter a relatively good correlation between the vane test and CPT. By the vane test fixed the maximum shear strength and creep limit \( C_{mr} \) correlate \( q_r \) in the following way:

\[
q_r = 40...50 \cdot C_{mr}.
\]

Taking into account the dependence between \( q_r \) and \( q_s \) [4]:

\[
q_r = 6 \sqrt{q_s},
\]

where \( q_s \) and \( q_r \) in kg/cm². The sands containing organic matter are these soils in case of which it is possible to surpass \( q_s \), whereas the difference between \( q_r \) and \( q_s \) is larger. Basing on the results of CPT tests, it is possible to rather exactly evaluate the bearing capacity of the soil and the deformation parameters corresponding to the evaluated size.

2. TILL MORAINES

Taking into account the difficulties in moraine sampling and fixing their properties in the laboratory the plate load tests and CPT are also widely used in case of moraines.

Moraines in Estonia are clayey silty till \( L_p = 3...12 \), that contain 5–40 % of coarse fraction. The containing of coarse fraction is smaller in the moraine covering the North-Estonian Cambrian areas and in the moraine covering the South-Estonian Devon areas. Investigating these areas mainly CPT has been

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used.

Among the Estonian moraines there are plenty of weaker varieties and studying the last-mentioned, CPT has given the results of necessary punctuality evaluating the soil as well as the bearing capacity of piles. Studying the Estonian moraines one has noticed the dependence of the moraine’s properties on the seasons. To evaluate this dependence has been possible only relying upon CPT tests.

The following correlative dependences between the CPT tests and plate load tests have been possible to be fixed (the correlation factor \( \eta = 0.7 \ldots 0.8 \))

\[
q_p = q_i / 12, \\
E_p = 4 \cdot 5 \cdot q, \\
E_w = 0.5 \cdot q_i.
\]

The interval comparison of vane tests and CPT gave the following results:

\[
C_v = q_i / 30, \\
C_v = q_i / 50, \\
q_i = \pi \cdot C_v, \\
q_i = 6 \cdot C_v.
\]

As it could be seen from the above correlations the surpassing \( q_p \) in case of moraines is accompanied with the increase of intensity of settlement. It is especially dangerous if the pressure under some foundations is over \( q_i \) and under some foundations it is under \( q_i \). In case of the Tartu Children’s Hospital founded on weak moraine \( q_i =1.2 \text{ MPa}, q_1 =0.1 \text{ MPa} \) and one hostel the intense development of cracks in these buildings was concurrent with them.

Studying moraines it is necessary to take into account that the properties of the moraine depend on seasons and the changes [5] accompanying with them. Special investigations on special polygons in Tartu and Võnnu showed that soil properties change in the zone with thickness of 6 m near the surface. The maximum values of \( q_i \) were at the end of the winter (February, March) and then the lessing of \( q_i \) began - until May and after that a small rise until August and then the fall up to the minimum values in November - December. The intensity of changes depended on the containing of clay fraction and could be expressed with the following dependence \( (\eta=0.8) \)

\[
q_i = 0.3 \cdot W_{LV}, \\
q_i = 0.3 \cdot q_{LV}, \\
q_i = 0.3 \cdot W_{LV}, \\
q_i = 0.3 \cdot q_{LV}.
\]

\( q_i \) - the change of the cone resistance, MPa, \( W_{LV} \) - the liquid limit by Vassilyev’s cone in %, \( W_{LV} \) - the change in the observed area from 15 to 20.

\( W_{LV} \) - Vassilyev’s cone correlates with the Swedish’ cone \( W_{LS} \) in the following way [6]

\[
W_{LS} = 1.25 \cdot W_{LV} - 2.
\]

In the course of the same investigation it becomes clear that \( q_i \) in the South-Estonian moraines depends on a large scale on the physical properties of the soil:

\[
q_i = 28 \cdot e^{0.2 W} \cdot 350 \text{ tests, } \eta = 0.8, \\
q_i = 0.5 \cdot (W_e / W) \cdot 60 \text{ tests, } \eta = 0.7.
\]

As it could be seen the influence of seasons on the less clayey moraines is very important and not taking it into account could cause the untrue evaluation of soil properties. To evaluate these influences with laboratory investigations has not been successful and in case of softer weak moraines the laboratory tests evaluate the soil strength greater than the actual one.

### 3. SOFT CLAYS

Studying the soft clays with CPT, the problem is the exactness. The methodics is indispenable on studying the clayey silt, because the level of disturbance of the received samples is high. The softer varieties have become denser in the course of sampling and the stronger ones have become loose. At the same time for the soft clayey silt a good dependence between the vane test and CPT [7] has been gained:

For silts \( C_v = q_i / 13 \) and clays \( C_v = q_i / 17 \).

But \( C_v \) is connected with the creep threshold

\[
C_v = 0.65 \cdot C_{vt}
\]
and with the undrained shear strength

$$C_v = 1.4 \cdot C_u.$$  

For soft clays to increase the investigation exactness in Estonia the 50 cm² cone with usual 10 cm³ roads has been used. The investigations carried out have showed a good correlation between the undrained unconsolidated shear strength and $q_{cup}$:

$$C_p = q_c \cdot 50 / k, \quad k = 6...8.$$  

Thus the use of a bigger cone has enabled to enlarge the exactness of CPT tests and to fix quicker than with a vane tests and the soil strength parameters for weak soils.

4. BEARING CAPACITY OF PILES

Special investigations with tensin piles and pile sounds enabled to work out the methodologies for fixing the pile's saft friction and the bearing capacity of the pile's point. The analyze of settlement temporality accompanied with this methodologies enabled to fix usual pile tests the pile's side friction $N_s$ and the pile's proportionality limit $N_p$ and the ultimate load $N_u$ [8].

The comparison of 100 pile tests to the results of CPT tests enabled for Estonia to fix the dependences of the pile's special side friction and for the special bearing capacity of the pile's point the proportionality limit $N_p$. For carrying out the investigations the CPT was used - the old equipment - fixed the point's special resistance and a summary side friction (1-st type) of the pile. The last one divided to the area of the sound gives the special side friction of the sound $q_c$. The sound with the friction muff (2-nd type) was also used.

With the help of the 1-st type of the sound very good correlative dependences between the CPT and pile tests were gained:

$$q_{cup} = 0.2 \cdot q_c, \text{ when } \eta = 0.9,$$
$$q_{cup} = 0.8 \cdot q_c, \text{ when } \eta = 0.85.$$  

Using the sound of the 2-nd type the correlations were very bad ($\eta = 0.3...0.5$) and due to them we do not recommend the penetrometers of this kind.

The bearing capacities of piles prognosticated with CPT tests have been controlled (after the above-mentioned received formulas) by pile tests and in case of all the 16 tests the value over $\pm 10...15$ % of $N_p$ fixed in the course of the test does not differ from $N_p$ - the value calculated on the basis of CPT.

$N_p$ has turned out to be the basis of evaluation of pile's bearing capacity in Estonia. As on the load smaller $N_p$ than it the pile foundation behaves linearity and the settlements of pile foundations remain in the limits of 2...5 cm (the settlement observations of 23 objects). Surpassing $N_p$ more intense settlement of the pile foundation accompanies and this has extended to 10...16 cm (6 objects).

On the grounds of pile tests

$$N_p = 0.5...0.9 \cdot N_u.$$  

5. WEDGED PILES

The extensive use of wedged piles with the length of 2...3 m and the volume of 0.2...0.42 cubic metres is spread in Estonia. The use of them is especially effective in moraine and sands where they give about 60 % of concrete and 40 % of steel economy. To evaluate their bearing capacity the CPT is also suitable and it correlates well with the results of pile tests ($\eta = 0.85...0.9$) [9]:

- 2.0 m pile, $N_p = 0.03 \cdot q_c + 125$,
- 2.5 m pile, $N_p = 0.05 \cdot q_c + 130$,
- 3.0 m pile, $N_p = 0.06 \cdot q_c + 140$,

Where $N_p$ - proportionality limit kN,
$q_c$ - MPa.
6. REFERENCES


Toe Bearing Capacity of Piles from Cone Penetration Test (CPT) Data

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SYNOPSIS A direct CPT method is proposed for determining the toe capacity of a single displacement pile. The method makes use of a simple mathematical rule for determining the average cone resistance, $q_c$, adjusting it to the effective stress and relating it to the unit toe resistance at a pile, $r_t$. Case histories comprising CPT data, soil characteristics, and results from full-scale pile loading tests are referenced to the methods. The case histories involve sites with soft clay and sand, sand interbedded with thin silt and clay layers, and medium to dense sand. The pile embedment lengths range from 9 m through 31 m. The pile capacities range from 300 kN through 5,800 kN with measured toe resistances ranging from 62 kN to 4,050 kN. Pile toe capacities calculated by the proposed method are compared to toe capacities calculated by four other direct methods currently employed in North American practice. The proposed method gives values that are more consistent and closer to the measured than the current methods.

INTRODUCTION

Determining axial capacity of piles is a challenge under the best of circumstances. The engineering practice has developed several methods to overcome the uncertainty in the analysis and design. However, due to simplifying assumptions regarding soil stratigraphy, distribution of shaft resistance along a pile, and soil-pile structure interaction, the methods provide qualitative results rather than truly quantitative values directly useful in the pile design. In recent years, the Cone Penetration Test (CPT) has become the preferred in-situ test for pile design and analysis. This is because the CPT is simple, fast, relatively economical, and provides continuous records with depth that are interpretable on both empirical and analytical bases.

CURRENT METHODS

Two main approaches are used for the application of CPT data to pile design: indirect methods and direct methods. Indirect methods will not be discussed here. Direct CPT methods apply cone bearing for unit toe resistance and sleeve friction for unit shaft resistance by the analogy of the cone penetrometer as a model pile. The following four CPT direct methods for pile capacity estimation are used in current North American practice.

- The Schmertmann and Nottingham method (1975; 1978)
- The DeRuiter and Beringen method (1979)
- The LCPC method (Bustamante and Gianessi, 1982)
- The Eurocode method (1993)
The methods address both shaft and toe resistances. However, this presentation is limited to a study of the pile toe resistance calculated from the CPT cone resistance.

The Schmertmann method is based on a summary of the work on model and full-scale piles presented by Nottingham (1975). The unit toe resistance of a pile, \( r_t \), in sand and clay is taken as equal to the average of the cone resistance, \( q_c \). The average \( q_c \) value is determined from the graphical representation of the CPT measurements in a zone defined by the failure pattern for the pile toe, ranging from 4b above (b is pile diameter) and from 0.7b through 4b below the pile toe (the actual value depends on the trend of \( q_c \) values), as originally proposed by Begemann (1963). An upper limit of 15 MPa is imposed for the toe resistance. The unit toe resistance is further governed by the overconsolidation ratio.

The DeRuiter and Beringen method (also called the European method) is based on experience gained in the North Sea by Fugro Consultants International. This method is very similar to the Schmertmann method. Indeed, for unit toe resistance of a pile in sand, the method is the same as the Schmertmann method. In clay, the unit toe resistance is determined from the undrained shear strength, \( S_u \), as follows:

\[
\begin{align*}
  r_t &= N_c S_u \\
  S_u &= q_c / N_k
\end{align*}
\]

where \( N_c \) is the conventional bearing capacity factor and \( N_k \) is a non-dimensional cone factor ranging from 15 through 20.

The LCPC (Laboratoire Central des Ponts et Chaussées) method (also called the French method) developed by Bustamante and Giuseppelli (1982) is a result of experimental work by the French Highway Department. The experimental database for this method is based on the results of a large number of full-scale pile loading tests. The average \( q_c \) is determined within a zone of 1.5b above and 1.5b below the pile toe and the unit toe resistance of a pile is determined as a percentage of the \( q_c \)-value ranging from 40% through 55%, as governed by cone resistance magnitude, soil, and pile types.

The Eurocode (Frank, 1994) is a combination of the general rules for geotechnical design in the “Eurocode 7-Part 1” and the code of the French Highway Administration. The method is very similar to LCPC method. The difference is that the unit toe resistance is determined as a range of 50% through 55% of average \( q_c \).

When using either of the four methods, difficulties arise in applying some of the recommendations of the methods. For example:

1. All methods include random smoothing of the data, that is, elimination of peaks and troughs, which subjects the results to considerable subjective operator influence.
2. In the Schmertmann and European methods, the overconsolidation ratio, OCR is used to relate \( q_c \) to \( r_t \). However, while the OCR is normally known in clay, it is rarely known for sand.
3. In the European method, considerable uncertainty results when converting cone data to undrained shear strength, \( S_u \), and then, in using \( S_u \) to estimate the pile toe capacity. \( S_u \) is not a unique parameter and depends significantly on the type of test used, strain rate, and the orientation of the failure plane.
4. In the French and Eurocode methods, the extent of the zone above and below the pile toe in which the cone resistance is averaged, appears to be too limited. As considered in the Schmertmann method, particularly if the soil strength decreases below the pile toe, the soil average must include the conditions over a depth larger than 1.5b distance below the pile toe.
5. The upper limit of 15 MPa, which is imposed on the unit toe resistance in the Schmertmann and European methods, is not reasonable in very dense sand where values of \( q_t \) higher than 15 MPa frequently occur.

6. All methods involve a judgment in selecting the coefficient to apply to the average cone resistance to arrive at the unit toe resistance.

7. The measured cone resistances are total stress values whereas effective stress governs the pile capacity.

8. Considerable uncertainty exists due to the effects of installation, strain softening, fissured clay, resistance degradation, sensitivity, dynamic pore pressures, and shallow penetrations of cone and pile into dense sand strata.

**CONE RESISTANCE AVERAGE**

All four current methods employ a graphic approach to relate cone resistance to the unit toe resistance of pile, where the \( q_c \) values are first filtered by excluding peaks and troughs. The mean of the smoothed curve is taken to be the average \( q_c \) to use. Filtering and smoothing the cone data is necessary because a true mean produced from the data, the high and low values would have a disproportionate influence on the average. The filtering approach was developed when the CPT data were obtained in diagrams only and it brings about a considerable judgment leeway in the average cone resistance (eyeballing uncertainty). However, the subjective filtering is now not necessary, because current tests produce quantified results in the form of tables of data, easily accessible for determining the average by direct computer manipulation, making the graphic methods redundant.

The arithmetic average is defined as:

\[
q_d = \frac{1}{n} (q_{d1} + q_{d2} + \ldots + q_{dn})
\]

(3)

Having the CPT data in the computer, the average \( q_c \) according to Eq. 3 can be obtained automatically. However, without first excluding peaks and troughs, this average is only useful in homogeneous soils and soils providing uniform values. Filtering is necessary in most cases and, if done manually, it offsets the advantage of the computer. A filter effect can be achieved directly, however, by instead calculating the geometric average of the \( q_c \) values, defined as:

\[
q_g = (q_{d1} q_{d2} \ldots q_{dn})^{1/n}
\]

(4)

The bias in the arithmetic mean arises from the influence of the absolute magnitude instead of ratios of variations (Kennedy et al., 1986). For example, the arithmetic average of the numbers 0.5 and 2.0 is 1.25, and the geometric average is 1.00. If the numbers are 0.33 and 3.00 instead, the mean becomes 1.65 while the geometric average is still 1.00. If a set of data is made up of the numbers 0.33, 0.50, 2.00, and 3.00, the arithmetic and geometric averages are 1.46 and 1.00, respectively.

Assume that a set of values is as follows: 5, 5, 1, 5, 25, 5, 6, 1, 6, 6, 30, and 6, where the dominant values lie between 5 and 6. The arithmetic and geometric averages are 8.50 and 5.71, respectively. The result shows that the geometric average is closer to the dominant values, as opposed to the arithmetic average which is not representative for the dominant range.

The natural variability of many sand deposits produces \( q_c \) profiles with many sharp peaks and troughs. Therefore, by taking the geometric average of \( q_c \) values in a zone at the vicinity of pile toe, a filtered representative value that is unaffected by operator’s judgment and, therefore, repeatable, is obtained.
PROPOSED METHOD

A direct CPT method is proposed that includes determining the geometric average of all \( q_u \) values at the vicinity of pile toe. For now, the zone at the vicinity of pile toe is taken to be the same as used by the Schmertmann and the European methods.

Pile capacity is governed by effective stresses in the soil, not total. Rather than obtaining the unit pile toe resistance as an arbitrary percentage of the average total cone resistance, the proposed method determines the toe resistance as the cone resistance minus the pore pressure measured by means of the piezocone. The proposed method, therefore, requires the CPT test to be made with the piezocone. However, in sand, normally, the pore pressures can be assumed to be essentially unchanged due to the cone penetration and older types of CPT equipment are still useful.

Thus, the unit toe resistance of a pile is as follows:

\[
\tau_t = (q_u - q_l) \cdot (n)^{1/3}
\]

(5)

where

- \( q_u \) = \( q_u - u \)
- \( q_l \) = total cone resistance = \( q_l + (1-a)u \)
- \( u \) = pore pressure, usually \( u_l \)
- \( a \) = net area ratio of a cone
- \( n \) = number of values in the considered zone

(The most useful location of the piezometer is behind the cone. The pore pressure measured at this location is called \( u_l \)).

CASE RECORDS

Six case histories are included in this study to reference the methods. The cases comprise full-scale pile loading tests where the pile toe capacity are determined, and include CPT soundings performed close to the piles.

UBC Research Site: A 324-mm, 31 m pipe pile was tested at the Lulu Island in Fraser River Delta, British Columbia. The soils consist of about 15 m of organic silty clays underlain by a 15 m thick medium sand deposit followed by 60 m normally consolidated clayey silt containing thin sand layers (Robertson and Campanella, 1987).

Northwestern University: A 450-mm, 15 m pipe pile was tested in conjunction with the 1989 ASCE Foundation Engineering Congress held at the Northwestern University, Evanston. The pile was installed through 7 m dense sand stratum overlying a soft clay layer (Finno, 1989; Fellenius, 1991).

Hunter’s Point, San Francisco: A 273-mm, 9 m pipe pile were tested in conjunction with a pile Prediction Symposium organized by FHWA, 1986. The soil at the site consists of about 2 m miscellaneous fill underlain by 11 m hydraulic sand fill followed by a clay layer (Fellenius, 1986; O’Neill 1988).

Baghdad University: Two 285-mm square concrete piles with embedment lengths of 11 m and 15 m were tested at Baghdad University Complex in 1984. The soil at the site consists of uniform sand (Altace et al., 1992 and 1993).

Port of Los Angeles: A static pile loading test was performed on a 600-mm octagonal concrete piles with embedment length of 26 m at the Port of Los Angeles in 1985. The soil at the site consists of an upper 15 m thick sand layer underlain by a 6 m thick fine-grained soil layer followed by dense sand (CH2M Hill, 1987).

Table 1 summarizes the case information, presenting pile data, soil type at pile toe, and measured pile and toe capacities, as well as the unit toe resistance calculated from the toe capacity and cross sectional area of pile. The \( N_t \)-values are back-calculated from the toe capacity using the following equation:
$$R_i = \tau_i A_i = N_i \sigma^i_{v,1-D} A_i$$  \hspace{1cm} (6)$$

where

- \(R_i\) = pile toe capacity
- \(\tau_i\) = unit toe resistance
- \(A_i\) = cross sectional area of pile
- \(\sigma^i_{v,1-D}\) = vertical effective stress at pile toe
- \(D\) = pile embedment depth
- \(N_i\) = bearing capacity factor

The back-calculated \(N_i\)-values, obtained for the different piles lie within normally observed ranges of 3 through 30 for clay and 30 through 150 for sand (Fellenius, 1993).

Fig. 1 illustrates the CPT results including cone resistance, \(q_c\), sleeve friction, \(f_s\), and measured pore pressure. The groundwater table and the pile toe depths are indicated. At the UBC site, the pore pressures were measured behind the cone \(u_2\), whereas at NWU site, it was measured at the face of cone \(u_0\). For other cases, no pore pressures were measured. However, because the soils at these sites consist of sand, the pore pressures were considered to correspond to the distance to the groundwater table.

**TABLE 1. Pile data, measured capacities, and soil conditions at pile toe**

<table>
<thead>
<tr>
<th>No.</th>
<th>Site</th>
<th>(D)</th>
<th>(b)</th>
<th>(A_t)</th>
<th>(\text{Shape, mrl.})</th>
<th>Soil at pile toe</th>
<th>(R_i) (\text{KN})</th>
<th>(R_b) (\text{KN})</th>
<th>(r_i) (\text{KPa})</th>
<th>(N_i)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Univ. of B.C. (UBC5)</td>
<td>31.0</td>
<td>320</td>
<td>0.082</td>
<td>P, S</td>
<td>Clayey silt</td>
<td>1,100</td>
<td>180</td>
<td>2,195</td>
<td>9</td>
</tr>
<tr>
<td>2</td>
<td>N. W. Univ. (NWU)</td>
<td>15.3</td>
<td>450</td>
<td>0.159</td>
<td>P, S</td>
<td>Soft clay</td>
<td>1,020</td>
<td>90</td>
<td>390</td>
<td>3</td>
</tr>
<tr>
<td>3</td>
<td>Hunter Point (FHWA)</td>
<td>9.1</td>
<td>270</td>
<td>0.059</td>
<td>P, S</td>
<td>Sand</td>
<td>490</td>
<td>335</td>
<td>5,678</td>
<td>70</td>
</tr>
<tr>
<td>4</td>
<td>Baghdad (BGHD1)</td>
<td>11.0</td>
<td>285</td>
<td>0.081</td>
<td>S, C</td>
<td>Sand</td>
<td>1,000</td>
<td>360</td>
<td>4,444</td>
<td>30</td>
</tr>
<tr>
<td>5</td>
<td>Baghdad (BGHD2)</td>
<td>15.0</td>
<td>285</td>
<td>0.081</td>
<td>S, C</td>
<td>Sand</td>
<td>1,600</td>
<td>480</td>
<td>5,926</td>
<td>30</td>
</tr>
<tr>
<td>6</td>
<td>Port of L. A. (POLA)</td>
<td>25.8</td>
<td>610</td>
<td>0.308</td>
<td>S, C, C</td>
<td>Dense sand</td>
<td>5,785</td>
<td>4,050</td>
<td>13,149</td>
<td>60</td>
</tr>
</tbody>
</table>

\(b\)=Pile diameter, \(P\)=Pipe, \(S\)=Square, \(O\)=Octagonal, \(S\)=Steel, \(C\)=Concrete

![CPT soundings from the sites. Notice, all cases use different depth and stress scales](image-url)
RESULTS

Table 2 presents the calculated average total cone resistances, $q_c$, in the zone near the pile toe and the corresponding unit pile toe resistances, $r_t$.

Fig. 2 shows a graphical comparison of results in terms of total pile toe resistance as determined by the four current methods and the proposed method and compared to the measured values. The figure has been separated into two diagrams showing three cases with piles having a measured toe resistance smaller than 400 KN and three cases with piles having a measured toe resistance larger than 400 KN.

Table 3 presents a compilation of the relative error in the calculations of the pile toe capacity by the methods. The relative error is determined as the difference between the calculated and measured values divided by the measured values. A negative value indicates an underestimation of the pile toe capacity.

The relative errors of the estimated pile toe capacity for the Schmertmann and European methods are smaller than those for the French and the Eurocode methods. However, the average error of pile toe capacity estimation for the four current methods is relatively high (20% through 35% with a Standard Deviation of 14% through 27%). In contrast, the proposed method shows a good agreement with the measured values (9% average error with a Standard Deviation of 8%), and more important, the agreement is consistent for all the six cases.

CONCLUSIONS

The proposed direct CPT method for determining the pile toe resistance from the cone resistance has been tested on six piles of different size and lengths installed in different type soils and compared to the results of four current methods. The proposed method is independent of operator judgment in filtering data and in choosing correlation factors, which are affecting all the current methods. The results of the comparison is very favorable to the proposed method.

The study is a part of an ongoing research and it is the intent to develop the method further, including a review of the extent of the zone above and below the pile toe. The method will also include a study of the calculation of pile shaft resistance (early results were excluded from this presentation due to space limitations).

The authors would very much appreciate receiving case history data to add to the data base.
TABLE 2. Average $q_u$ and pile unit toe resistance from CPT methods

<table>
<thead>
<tr>
<th>Method</th>
<th>UBC5</th>
<th>NWU</th>
<th>FHWA</th>
<th>BGHD1</th>
<th>BGHD2</th>
<th>POLA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$q_{avg}$</td>
<td>$q_{avg}$</td>
<td>$r_1$</td>
<td>$q_{avg}$</td>
<td>$r_1$</td>
<td>$q_{avg}$</td>
</tr>
<tr>
<td>Schermer.</td>
<td>1,765</td>
<td>1,765</td>
<td>580</td>
<td>8,450</td>
<td>4,850</td>
<td>3,000</td>
</tr>
<tr>
<td>European</td>
<td>1,260</td>
<td>1,260</td>
<td>360</td>
<td>4,850</td>
<td>4,850</td>
<td>3,000</td>
</tr>
<tr>
<td>French</td>
<td>2,030</td>
<td>1,015</td>
<td>600</td>
<td>7,200</td>
<td>3,600</td>
<td>1,970</td>
</tr>
<tr>
<td>Eurocode</td>
<td>2,030</td>
<td>1,117</td>
<td>600</td>
<td>7,200</td>
<td>3,600</td>
<td>1,970</td>
</tr>
<tr>
<td>Proposed</td>
<td>1,900</td>
<td>1,900</td>
<td>375</td>
<td>6,200</td>
<td>6,200</td>
<td>4,070</td>
</tr>
</tbody>
</table>

TABLE 3. Relative error (%) in capacity as determined by different methods

<table>
<thead>
<tr>
<th>Methods</th>
<th>UBC5</th>
<th>NWU</th>
<th>FHWA</th>
<th>BGHD1</th>
<th>BGHD2</th>
<th>POLA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average Error</td>
<td>Standard Deviation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Schermer.</td>
<td>-21</td>
<td>48</td>
<td>-15</td>
<td>-33</td>
<td>9</td>
<td>14</td>
</tr>
<tr>
<td>European</td>
<td>-43</td>
<td>-8</td>
<td>-15</td>
<td>-33</td>
<td>9</td>
<td>14</td>
</tr>
<tr>
<td>French</td>
<td>-53</td>
<td>-22</td>
<td>-77</td>
<td>-55</td>
<td>-22</td>
<td>-23</td>
</tr>
<tr>
<td>Eurocode</td>
<td>-48</td>
<td>-10</td>
<td>-37</td>
<td>-55</td>
<td>-22</td>
<td>-4</td>
</tr>
<tr>
<td>Proposed</td>
<td>-12</td>
<td>-6</td>
<td>8</td>
<td>-8</td>
<td>9</td>
<td>9</td>
</tr>
</tbody>
</table>

REFERENCES


Correlation of in situ tests for the evaluation of design parameters

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Dar AlHandasah Consultants (Shair and Partners) Ltd, UK

SYNOPSIS: At two sites in the Middle East appropriate in situ tests were performed, and parameters evaluated were correlated with the results of static cone penetration test (CPT) results in order to use more effectively for the whole sites. The first site was some 3.7 km². In situ loose to very loose silty sand was dynamically compacted and the result monitored by CPTs. Considerable variations in the degree of improvement between adjacent parts of the site were noticed reflecting the variations in subsurface conditions. Two full scale loading tests were performed, and measured settlements were compared with the settlements calculated using Schmertmann (1978) in order to evaluate site specific correlation between cone resistance (q_c) and soil deformation modulus (E).

The second site some 32 km long made of over consolidated cohesive deposits overlying a major granular horizon; it was for an urban rapid transportation project. Limited self boring pressuremeter (SBP) tests were carried out through cohesive and granular soil strata, and shear modulus (G) values evaluated were correlated with the cone resistance of CPTs. Some 500 CPTs were performed along the route, and the correlated values of deformation modulus used in the design.

1. INTRODUCTION
For important projects reliable in situ tests are essential in order to evaluate site specific parameters for design and predicting performance of foundations and underground structures. When the sites are large, it may not be possible to perform high quality in situ testing at close proximity in order to cater for variability in subsoil depositions. This paper reports results of two large sites at the Middle East where the results of in situ tests were correlated with cone resistance (q_c) of the CPTs.

In the first site two full scale loading tests were performed using 4 x 4m plate in order to estimate safe bearing capacity and deformation modulus of the in situ material. In the second site, a series of self boring pressuremeter tests through desiccated overconsolidated clay were performed at eleven chosen locations of a 32 km site. The shear modulus (G) evaluated from SBP tests were correlated with q_c. Generalised coefficient of correlation between modulus of elasticity (E) and q_c normally quoted for sand are in the range between 2.5 and 3.5 for square footings and strip footings respectively (Schmertmann 1978). However, Schmertmann has pointed out that the correlation factors quoted should only be used with first loading cases and, if the sand has been prestrained or preloaded the actual settlement will decrease under subsequent loading by a factor greater than the resulting increase in q_c would indicate. There again, not much direct correlation between E or G (shear modulus) and q_c for cohesive material are available especially for desiccated overconsolidated clay.

2. FIRST SITE
The first site was developed for a holiday resort in the middle of Half Moon Bay on the Arabian Gulf in Saudi Arabia, consisting of two hundred fifty chalets surrounding inner bay together with infrastructures, services and...
recreational area. The detail description of the site is given elsewhere (Ghosh and Tabba 1988)

Fig. 1 shows a typical subsurface profile before ground improvement effort was applied on the western part of the site where the load tests were carried out. Following the Sabkha on the surface, fine to medium graded sand with traces of silt and varying amount of shell fragments were found to some 9m depth. Underlying this sand stratum an extensive deposits of silt and clayey sandy silt some 5m thick were found. The SPT blows and the CPT results shown clearly indicated that the sand and the silt below were in a very loose to loose (or soft) state. Underlying the silt stratum sand deposits prevail across the site with a transition zone of silty sand generally some 2m thick, and normally found to be loose to very dense. The above strata overly a hard silty clay generally encountered at 19 to 20m depth.

Datum. Finally, dynamic compaction was applied on the recliared surface using 16 tonne pounder dropping from 25m height, 10 blows per print at 10m centre in order to densify the initial 10m of the subsoil. The ground improvement effort was applied to achieve a safe bearing capacity of 100 kN/m² at the surface and 50 per cent relative density at 10m depth using Schnerrmann (1978). The CPTs were used to verify the improvement and also to estimate relative density at 10m depth. A part of the site, where the in situ conditions prevented any improvement by dynamic compaction, was preloaded (Ghosh and Tabba 1988).

Considerable improvement was noticed in many parts of the site. However, such improvement in many areas was still insufficient to meet the specified relative density of 50 per cent to be achieved at 8m below datum using Schnerrmann’s correlation.

To overcome this problem two full-scale loading tests were performed, and Figs 2 and 3 display the post-compaction CPT profiles for the first and second loading test locations respectively. As a matter of fact a set of five CPT profiles were carried out, one in the centre and one at each corner of the loaded area. The first loading test location was chosen in an area where the dynamic compaction’s response was encouraging, whereas, at the second location no improvement could be achieved below 6m below the surface. The second test area was initially compacted with a single pass of 10 blows from 20m drop per print at 10m centre, but subsequently, the area was recompacted by repeating the same but using 25m drop.

2.1 Full scale loading tests
The full scale load tests were carried out using a 4m x 4m, 0.4m thick reinforced concrete plate loaded to 150 kN/m² pressure using concrete blocks 4x 1x1m as keystone in order to produce an uniform loading on the plate (Fig. 4). For the first loading test of the plate was loaded in increments of 50 and 100 kN/m² on day one and 150 kN/m² on day two. The full load was maintained for eighteen days. For second load test the full pressure of 150
kN/m² was placed on the first day and maintained for 26 days. The total settlement plot for the first and second loading tests are shown in Fig. 5.

The total settlements at the first and second test locations were 5.0 and 13.4 mm respectively.

Fig. 2. Pre-and Post Dynamic Compaction CPT Results at Load Test 1 Location

CONE RESISTANCE : kN/cm²

Fig. 3. CPT Profiles and Strain Influence Diagram at Load Test 2

Fig. 4. Loading Test 1

2.2 Correlation of q₁ and E

The settlements of both plates, which were equivalent to a large footing, under 150 kN/m² pressure were calculated using Schmertmann’s (1978) method and the respective mean CPT profiles. Figs 2 and 3 also show the strain influence diagrams extending twice the width of the plate below the footings. The correlation coefficient α = Eq₁ has been derived for each test location by comparing calculated settlement (ρₑ) and the total measured settlement (ρₑ).

The calculated and measured settlements together with the derived values of α values are listed on Table 1.

TABLE 1: SUMMARY OF SETTLEMENTS - FIRST SITE

<table>
<thead>
<tr>
<th>Load test</th>
<th>(ρₑ) mm</th>
<th>ρₑCAL</th>
<th>(ρₑCAL/ρₑ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.0</td>
<td>16</td>
<td>7.2</td>
</tr>
<tr>
<td>2</td>
<td>23.4</td>
<td>103</td>
<td>7.7</td>
</tr>
</tbody>
</table>
The correlation factors derived from the measured settlements at two test locations are almost identical, in spite of the fact that the total settlement for the second test was 2.68 times larger than that for the first test. The magnitude of the correlation factor obtained essentially reflects the effect of the compacting effort applied on the surface as pointed out by Schmertmann (1978).

Several empirical correlation coefficients of $q_c$ and $E$ for sand deposits are quoted in the literature (Sanglerat 1972, Ruiter 1982, Ghosh 1982). Sanglerat observed large scattering of $\alpha$ values for sand and gave $\alpha = 1.5$ and 2 for $q_c > 10$ MPa. It is important to note the $\alpha$ value evaluated using Schmertmann's method of calculating settlements should only be applied for footing foundations. Ghosh (1982) reported a $\alpha = 1.5$ where $q$ correlated with the $E$ values obtained from plate loading tests and a full scale load test on sand fill. Coincidentally, $\alpha = 1.5$ used to calculate settlements of the silty sand and silt strata under preloading on a very large area in the above site, and the calculated and measured settlements were good agreement. But Schmertmann's method grossly underestimated the settlements (Ghosh and Tabba 1988). Perhaps, the total area under the preloading fill may be considered equivalent to an elastic half space in relation to the compressible strata, and the strain influential pattern for the preloading is different than that for a footing. From the author's experience some practising engineers have encountered similar problems when using Schmertmann's method for calculating settlement of raft foundations; such discrepancies may be associated with the explanation given for preloading case.

3. SECOND SITE
The second site, located in the Middle East, was for a rapid transit transportation project 32 km long. The site consists of generalised succession of cohesive deposits overlying a major granular horizon. The cohesive deposits typically 10 to 20m deep are usually stiff to very stiff for the full depth solely due to desiccation. In addition, the degree of overconsolidation would not necessarily decrease with depth. The full thickness of the granular stratum, generally medium dense to dense silty fine to medium sand, was not known even with 60m deep boreholes. The ground water is generally close to the ground level.

3.1 In situ tests
Some 500 CPTs together with large number boreholes were carried out under the site investigation programme. In addition, eleven locations were chosen to perform self boring pressuremeter tests (SBP). The basic arrangement at each location was two CPTs profiles and a borehole located normally within 1m radius circle which was centred on the SBP position. The sequence of operation was to carry out CPTs followed by SBP testing and borehole.

The SBP instrument used was similar to that described in Windle and Worth (1977). A standard rate of 1% per minute cavity strain was adopted, and generally, at 1 to 3% strain unloading and reloading cycle was carried out in order to allow determination of the shear modulus(G) following Palmer(1972) except for using half the gradient of the best fit straight line for the loading unloading loop. Expansion was then continued up to a maximum value of generally 10% before unloading.
3.2 Correlation of \( q_c \) and \( G \)

From the plots of both \( G \) and \( q_c \) against depth for the eleven test locations it was clear that the mean slope of the respective profiles were almost identical in the cohesive stratum. This has been demonstrated by plotting \( G \) and \( q_c \) values against depth on the same diagram using multipliers of \( 10^5 \) and \( 10^6 \) N/m² for \( q_c \) and \( G \) respectively. Due to lack of space three results are presented in Figs 6, 7 and 8 for demonstration.

Fig.6. CPT-qc and SBP.G Profile 1

Fig.7. CPT-qc and SBP.G Profile 2

This simple plotting technique provide a unique factor \( G/q_c = 10 \). The soil profiles shown on Figs 6,7, and 8 clearly indicate that the derived factor is only suitable for clay and sandy clay. Furthermore, the arrangement adopted for the 32 km long site is extremely reliable.

4. DISCUSSION

Correlation of in situ tests results of static and dynamic probing (CPTs, SPT, DCT) have been reported by various investigators in order to evaluate design parameters such as shear strength and deformation modulus of in situ materials. A number of text books also quote various factors to correlates of \( q_c \) and other probing test results with various design parameters, but the validity of such correlation factors is often doubtful. Empirical correlations are site specific since controlling conditions (such as material distributions, depositional pattern, stress history, climatic and ground water regime) are seldom identical. It is extremely important to understand the deformation characteristics of in situ material in order to design foundations or underground...
structures for the projects with high capital values. Therefore, it is worthwhile to employ reliable testing programme to evaluate such parameters. However, for large sites where a sufficient number of high quality in situ tests are required could the cost and time involved could prevent them to be included in the campaign.

With regard to various probing tools currently available for site investigations CPTs probably provide more reliable data than others. Although SPTs’ popularity remain undaunted especially for its general use with boring machine through granular soils.

For the first site large number of CPTs were carried out in order to monitor the ground improvement effort. Therefore, the results of the full scale loading tests were used most effectively by correlating with $q_c$. There again, had not full scale load tests been performed the calculation of settlements would have been carried out by assuming a value of $\alpha$ which could have differed by a factor of 2.

For the second site, sufficient confidence was already gained, which allowed the selection of $G$ values for the cohesive deposits based on $q_c$ values.

5 CONCLUSIONS

1. CPT results are reliable to correlate with in situ parameters which were evaluated directly from established in situ testing.
2. Correlation factors are generally site specific and related to conditions prevail in a particular site.
3. The settlement of footings on sand can be assess reliably using CPT results with Schmertmann’s method of calculating settlements providing appropriate coefficient of correlation employed.
4. Simple plotting technique of superimposing the results of in situ tests can provide valuable correlation.

6 ACKNOWLEDGEMENT

The author is thankful to his employer Dar Al Handashah Consultants, U.K. Ltd. for the facilities provided to produce this paper.

7 REFERENCES


Tapered Pile Bearing Capacity Calculation by Static Sounding Data

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BashNIstrooi, Ufa, Russia

SYNOPSIS: Results of elaborating an engineering method of tapered piles calculation in clayey soils by static sounding data are given. Static test data of in-situ tapered piles equipped with strain-gauges are reviewed as well as data of static sounding made in place of test pile manufacturing. While tapered pile testing contact and normal stresses were measured along the lateral pile side at different depths as well as soil resistance under the pile tip. By means of mathematical processing of experimental data the empirical dependencies for determination of transfer coefficient from a probe to a pile as function of relative depth of deposited layer are obtained.

Calculation scheme is worked out and formulas for tapered pile bearing capacity determination are obtained, in which calculation soil parameters are defined by static sounding data.

The suggested complex evaluation of results provides more reliable and economical decisions.

1. INTRODUCTION

In construction practice in subsident soils cast-in-place piles, concreted in holes, cut in soil with stamps or compactors, the so-called "cast-in-place piles in the stamped-out hole" find their application. As practice of foundations on such piles application showed, they are also more effective in usual cohesive soils (Ziyazov, Gotman, 1978). In this case the hole is formed with pile hammer driving of standard pyramidal or coned stamp with the following extracting it out of the hole with hydrocylinders, mounted on the special platform and connected up to the pile driver.

Such piles have some definite advantages compared with bored and driven piles of constant section by length. So, as the hole is formed with stamping-out (without soil excavation), the concreted in it pile works in consolidated soil and hence has higher bearing capacity than that of bored pile that brings it nearer to driven piles. The tapered pile form promotes greater soil consolidation around the pile that provides also the pile resistance increase.

At the same time, the stressed - strained state of cast-in-place tapered pile, vertically loaded, as well as regularities of achieving the tapered pile limit state have their own peculiarities. That's why for design scheme obtaining and for solving the problem of such pile bearing capacity determination by static CPT, one should have reliable experimental data of contact stresses along the lateral pile side and under its toe for different load stages up to cracking load and compare it with CPT parameters.

The solving of all these problems is shown in given article.

2. METHODS OF EXPERIMENTS CARRYING OUT

The experiments were carried out at two sites of Ufa, consisting of clayey soils (table 1). Groundwaters were detected at
Table 1.

<table>
<thead>
<tr>
<th>Site No.</th>
<th>Sampling depth, m</th>
<th>Natural humidity, W</th>
<th>Plasti-city number, η</th>
<th>Consistency index, I</th>
<th>Volume mass, t/m³</th>
<th>Porosity coefficient, e</th>
<th>Angle of inner friction, φ</th>
<th>Cohesion, c MPa</th>
<th>Moisture degree, G</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>0.24</td>
<td>0.16</td>
<td>0.16</td>
<td>0.16</td>
<td>0.16</td>
<td>0.16</td>
<td>0.16</td>
<td>0.16</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.28</td>
<td>0.16</td>
<td>0.01</td>
<td>0.18</td>
<td>0.18</td>
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<td>0.18</td>
<td>0.18</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>32</td>
<td>0.19</td>
<td>0.06</td>
<td>1.82</td>
<td>1.28</td>
<td>1.28</td>
<td>1.82</td>
<td>1.28</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>34.4</td>
<td>0.19</td>
<td>0.06</td>
<td>1.81</td>
<td>1.35</td>
<td>1.35</td>
<td>1.81</td>
<td>1.35</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>34.3</td>
<td>0.17</td>
<td>0.06</td>
<td>1.85</td>
<td>1.38</td>
<td>1.38</td>
<td>1.65</td>
<td>1.38</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>26.2</td>
<td>0.13</td>
<td>0.06</td>
<td>1.95</td>
<td>1.54</td>
<td>1.54</td>
<td>1.95</td>
<td>1.54</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>31</td>
<td>0.19</td>
<td>0.06</td>
<td>1.96</td>
<td>1.5</td>
<td>1.5</td>
<td>1.96</td>
<td>1.5</td>
</tr>
</tbody>
</table>

At site No. 2 at the depth of 1.5-2 m. In place of test piles manufacturing static CPT was carried out with the help of the unit C-832, equipped with the standard tapered probe of 10 mm cross-sectional area with friction coupling for measuring the soil friction forces along the pile lateral side.

CPT was carried out by standard method and by the method worked out in BashNIStroi, when probe recordings were taken in probe equilibrium, i.e. in the mode of movements and stresses "stabilization" in soil.

At site No.1 a hole was stamped-out of 60 x 60/20 x 20 sm section and 6 m depth at the bottom of which and by its lateral side strain-gauges were placed in order to measure soil contact stresses, then a pile was concreted.

At site No.2 a cast-in-place pile of 80 x 80/30 x 30 section and 4.2 m length with strain-gauges along its lateral side was manufactured. In order to transfer all the load through the pile lateral side a metal plate was placed 5 sm above the hole bottom up to the place of concreting, i.e. the pile toe had no contact with the soil. At both sites piles were penetration and extraction load tested and recordings were made at all stages of loading. At site No.2 the pile after the extraction test was extracted out of the hole for zero recordings then the pile was driven into the same hole and again vertical penetration load tested.

Figure 1. Dependencies Diagrams.

a - at site No.1; b - at site No.2; 1 - "load-settling" of a driven pile; 2 - idem of cast-in-place pile; 3 - idem at pile extracting; 4 - lateral side resistance because of friction due to pile settling; 5 - idem because of normal pressure due to pile settling; 6 - bottom end resistance due to pile settling.
at this moment the pile toe began to work. In the process of hole stamping-out at both sites a soil uplift zone was measured by levelling.

3. ANALYSIS OF TESTS RESULTS

Pile static tests results are shown in figure 1. Figure 2 a, b shows diagrams of specific friction, normal soil pressure along the pile lateral side and soil resistance under the pile toe according to strain-gauges recordings at different loading stages. Soil CPT data are also shown here for comparison. In figure 1 curves 1, 2, and 3 are obtained by static tests data, curves 4, 5 and 6 - by results of soil contact stress measurement along the pile lateral side and under its toe.

When pile testing at site No.1 at the initial stage of loading friction along the lateral side of the upper pile part is significantly greater than that in its bottom part (figure 2, a). With load increase the friction naturally increases in the bottom pile part and becomes greater in direction to its tip.

At pile load 350 kN the friction of upper soil layers reaches its limit value and is constant up to the limit load. The soil under pile toe begins to work at the load about 200 kN and its resistance reaches the limit value (75 kN) at pile load 600 kN and settlement 15 mm. With load increase from 600 up to 650 kN (near to limit) and corresponding settlement 30 mm one can see sharp increase (by 1.5 times) of soil friction near the pile tip. It is evident that at this settlement the additive compression of the pile bottom part occurs. The share of pile resistance because of its compression with the soil is 5.3% of the total resistance.

At site No.2 (see fig. 2, b) the soil is simultaneously included in the work along the whole pile length from the very first load.
stages. As pile load increases, friction and normal soil pressure increase simultaneously.

The difference in pile behaviour at both sites is due to their different sizes and soil conditions (see CPT data in fig. 2 a, b). The less cross-sectional area and greater pile length as well as more firm soils at site No.1 compared with site No.2 cause significantly greater pile compression under the influence of one and the same load and hence more gradual drawing of soil layers in performance along the pile.

It must be noted that the soil resistance along the tapered pile lateral side due to friction is continuously increased with settlement increase and reaches its maximum at limit load. It is accounted for that the tapered piles are in conditions of constantly increasing normal soil pressure with the load increase along the pile lateral side. This causes friction forces increase even at great pile settlements when its shear relative to soil occurs.

Figure 2 a, b shows that values of friction and normal soil pressure along the pile lateral side at limit load are less according to strain-gauges recordings than that measured with static CPT, this discrepancy decreases with depth and becomes insignificant near the pile tip. The decrease of friction and soil pressure along the pile lateral side is due to soil uplift and loosening when stamp or pile driving. So, at sites No.1 and 2 the height of the uplifting soil near the stamp reached 10 and 8 m accordingly and then decreased up to 6 at the distances 1.8 and 2 m from the stamp. The volumes of uplifting soils at these sites were 0.36 and 0.32 m³ accordingly, that was 35 and 24% of the driven stamp volume. At CPT the reverse effect occurs - the soil around the probe is not loosened, but consolidated, as the probe section is significantly less than that of a pile and hence its relative deepening into the soil is greater. Thus, the uplift and accordingly the upper soil layers loosening when stamp (pile) driving lead to resistance decrease along the pile lateral side. That's why the usage of static CPT data for tapered pile calculation is possible only with the correction coefficients β inputting.

Figure 3 a,b shows these coefficients dependencies on the relative depth of soil layers bedding z/l for both sites as the relations of specific friction $f$ and normal soil pressure $p$ along the pile lateral side at limit load to the corresponding resistances $f_1$ and $p_1$ according to CPT data obtained with the unit C-832 "with stabilization" and "by standard."

At site No.2 the absence of contact with the pile toe changed the conditions of its performance and influenced the state of the soil around the pile, that's why the coefficients $β$ were determined by measurements results for a driven pile. Such an approach is quite rightful, as it was

Fig.3. Coefficient β dependence upon relative depth of layer bedding for specific friction (a) and normal pressure (b): 1 - for a pile with a sharp tip at CPT "with stabilization"; 2 - idem, "by standard"; 3 - for a pile with a blunt tip at CPT "with stabilization"; 4 - idem, "by standard".

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stated earlier that in stiff and medium clayey soils the resistances of cast-in-place and driven piles can be taken equal (Gotman, Ziyazov, 1980). That's why the coefficients \( \beta \) were determined as the mean value between measurements data obtained while cast-in-place pile testing at site No.1 and driven pile testing at site No.2 (Fig.3, curves 3 and 4). The coefficients \( \beta \) obtained by measurements results when cast-in-place pile testing at site No.2 can be approximately applied to tapered piles with the sharp end, as the additive compression of the lateral pile side bottom part lacks at the settling of such piles.

### 4. TAPERED PILE CALCULATION BY CPT DATA

Approximating data of figure 3 with the least square method, we obtain the empirical dependecies for transfer factors \( \beta_1 \) and \( \beta_2 \).

\[
\begin{align*}
\beta_1 &= K_6 \left( \frac{Z}{1} \right)^2 + K_4; \\
\beta_2 &= K_2 \left( \frac{Z}{1} \right) + K_3,
\end{align*}
\]

where \( \beta_1 \) and \( \beta_2 \) - dimensionless transfer factors from a probe to a pile for specific friction and soil resistance under the probe tip, accordingly; \( K_1, K_2, K_3 \) and \( K_4 \) - empirical coefficients, the values of which for given soils are shown in table 2.

The experimental data shown allow to present the design scheme of tapered pile as a vertically loaded stiff rod deepened into the soil with the reactive soil resistance along the lateral side as friction \( f_i \) and normal pressure \( R_i \) and with soil resistance under toe \( R_s \). Let’s divide the soil in the limits of pile length into “n” layers and take soil friction resistance and penetration resistance in the limits of separate soil layers constant and the law of pile section change \( d_s \) take by length as:

\[
d_s = d_i \left( 1 - \frac{Z}{d_s} \right),
\]

where:

\[
\Delta = d_0 - d_i
\]

\( d_s \) and \( d_i \) - the lateral pile size at the soil surface level and at its toe, accordingly; \( l \) - pile length in soil.

### Table 2.

<table>
<thead>
<tr>
<th>Pile type</th>
<th>Mode of soil CPT</th>
<th>Coefficients</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>With the blunt end</td>
<td>With the stabilization</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( K_2 )</td>
</tr>
<tr>
<td>With the blunt end</td>
<td>With the stabilization</td>
<td>0.73</td>
</tr>
<tr>
<td>By standard</td>
<td>0.44</td>
<td>0.13</td>
</tr>
<tr>
<td>With the sharp end</td>
<td>With the stabilization</td>
<td>0.7</td>
</tr>
<tr>
<td>By standard</td>
<td>0.35</td>
<td>0.13</td>
</tr>
</tbody>
</table>

Having projected all the forces onto the vertical axis, we obtain the tapered pile vertical load resistance.

\[
F = U_i A_i + \frac{A_i}{R_s} \left( 2B_i - \frac{Z}{1} B_s \right) - A_i R_s
\]

where \( U_i \) - pile section perimeter at the soil surface level; \( R_s \) - soil resistance under the pile toe; \( A_i \) - the area of the pile toe cross-section;

\[
A_i = \sum_{i=1}^{n} f_i (z_i - z_{i-1});
\]

\[
B_i = \sum_{i=1}^{n} f_i (z_i^2 - z_{i-1}^2);
\]

\[
A_R = \sum_{i=1}^{n} R_i (z_i - z_{i-1});
\]

\[
B_R = \sum_{i=1}^{n} R_i (z_i^2 - z_{i-1}^2);
\]

Using CPT data define soil design characteristics as:

\[
f_i = f_{d_0} \beta_{d_0};
\]

\[
R_i = q_{d_0} \beta_{d_0},
\]

where \( f_{d_0} \) - soil resistance along the probe lateral side; \( q_{d_0} \) - soil resistance under the probe tip.
β₁ and β₂ are found by formula (1) for each layer.

5. COMPARISON OF TEST AND DESIGN DATA
Table 3 shows the results of static tests and calculations of piles according to CPT at 6 sites with cohesive soils. The calculations were carried out by formula (3) using the coefficients β. The calculation by CPT data “with stabilization” using coefficients β by (1) gives the most reliable data (error doesn’t exceed 17%), while the errors obtained “by standard” are more than 19%.

<table>
<thead>
<tr>
<th>Section, m</th>
<th>Length, m</th>
<th>Pile bearing capacity, kN</th>
<th>Error, %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>By static test</td>
<td>By design CPT data</td>
</tr>
<tr>
<td>6x60 / 20x20</td>
<td>6</td>
<td>600 / 590 / 9.3</td>
<td>630 / 5</td>
</tr>
<tr>
<td>6x60 / 20x20</td>
<td>5.5</td>
<td>525 / 580 / 6.5</td>
<td>555 / 5.7</td>
</tr>
<tr>
<td>6x60 / 20x20</td>
<td>5.5</td>
<td>620 / 580 / 6.5</td>
<td>570 / 8.1</td>
</tr>
<tr>
<td>6x60 / 20x20</td>
<td>5.5</td>
<td>620 / 600 / 3.2</td>
<td>555 / 11.2</td>
</tr>
<tr>
<td>6x60 / 20x20</td>
<td>5.2</td>
<td>400 / 468 / 17</td>
<td>485 / 21.5</td>
</tr>
<tr>
<td>6x60 / 20x20</td>
<td>5.5</td>
<td>700 / 620 / 11.4</td>
<td>512 / 26.8</td>
</tr>
<tr>
<td>8x80 / 30x30</td>
<td>3.6</td>
<td>350 / 392 / 12</td>
<td>431 / 23.1</td>
</tr>
<tr>
<td>6x60 / 20x20</td>
<td>6</td>
<td>650 / 706 / 18.6</td>
<td>685 / 5.4</td>
</tr>
<tr>
<td>8x80 / 30x30</td>
<td>4.5</td>
<td>570 / 648 / 13.6</td>
<td>740 / 18.8</td>
</tr>
</tbody>
</table>

6. SUMMARY
1. The experimental investigations of in-situ piles in clayey soils allowed to obtain the regularities of contact stresses change along the lateral surface of vertically loaded tapered pile and to state that these stresses reach their maximum value with the limit pile load.
2. The empirical dependencies are obtained for determination of the coefficients of transfer from static CPT data to soil resistance along the lateral tapered pile side in dependence upon the soil bedding depth.

3. The design scheme of vertically loaded tapered pile is worked out. Formulas for tapered piles bearing capacity calculation are obtained in which design soil parameters are determined according to static CPT data.

7. REFERENCES
Pile settlement analysis using CPT and load-transfer functions $t-z$ and $q-z$

Kazimierz Gwizdala  
Technical University of Gdańsk, Geotechnical Department

Andrzej Tęchman  
Technical University of Gdańsk, Geotechnical Department

SYNOPSIS: The method of determination of total settlement curve (till failure load) using load-transfer functions is presented for large diameter bored piles and Fundex piles. The parameters characterising load-transfer functions, $t-z$ for shaft and $q-z$ for base of pile are determined on the basis of CPT. The proposed calculation model is verified by means of pile load tests in full scale. The calculation results are listed in the form of histograms, checking their approximation by density function. The mean values, standard deviation and coefficients of variation are given and the accuracy of settlement curve for bored piles are evaluated.

1. INTRODUCTION

The calculations of the pile foundations are, in general, of a great difficulty due to, among others, changes in natural subsoil during piles installation. Practical calculation methods are mostly based on the extensive in situ investigations of piles in natural scale and on correlation of the relationships. The problem of calculations for single pile is related to determination of load-settlement dependency. The whole load-settlement curve is a resultant of appropriate relations regarding the resistance of pile base $Q_b$, shaft resistance $Q_s$ and internal deformations pile’s material (Gwizdala, 1995). The reliable settlement curve can be drawn on the basis of loading tests. In practice, such curve has to be quite often predicted analytically using geotechnical parameters characterising given subsoil. The limit state of bearing capacity and exploitation conditions recommended in Eurocode 7 (1994) take into account whole range of settlement curve up to ultimate load and also considers its deviation, Franke 1990, (density functions and standard deviations).

Application of analytical methods and theirs verification on the basis of loading tests enables the determination of mentioned relationships.

In the paper the method of description of settlement curve up to ultimate load in terms of load - transfer functions is presented. The description of transfer functions of $t-z$ type for shaft of pile and $q-z$ for its base utilises the results of CPT tests.

2. METHOD OF LOAD - SETTLEMENT CURVE DETERMINATION

The analysis of load - settlement curves for various types of piles shows that ultimate load is reached at settlements of the range of 5 to 20% of pile diameter. It corresponds to range of settlement from 20 to 200 mm. For evaluation of settlement curve different methods may be applied as empirical or semi-empirical ones, analytical methods and direct measurements during loading tests.

Application of particular method depends on an access to sufficient number of appropriate geotechnical parameters. The method proposed is based on non-linear relations along shaft and under the base of pile. Similar analysis was
presented by Coyle and Reese (1966), Ar- 

In the method a pile is divided on an arbi-
trary number of elastic elements interacting
with surrounding subsoil in terms of non-linear
characteristics t-z and q-z on the shaft and base
of pile, respectively (Gwizdala and Jacobsen

Transfer functions t-z describe dependency
between unit resistance on shaft of pile and
settlement. The q-z function considers the rela-
tion between unit resistance under the base of
pile and its settlement.

The problem relies on calculation of depen-
dency between axial load and settlement of
pile for an arbitrary depth and load in terms of
iterative process. For practical calculations nu-
umerical code PALOS has been elaborated gen-
erating automatically appropriate number of
iterations. The calculation model of pile is
shown in Fig.1.

-- Fig.1 Calculation model of single elastic pile

To determine t-z and q-z functions model and
laboratory tests, theoretical solutions and re-
sults of CPT tests are used. The functions can
be described by following parameters:

-- unit, ultimate resistances \( l_{u, max} \) and \( q_u \) on shaft
and under the base of pile, respectively,

-- calculated or assumed settlements \( z_t \) and \( z_r \),
  corresponding to maximum resistances \( q \) and
  \( t \) for the base and shaft of pile, respectively,

-- non-linear stiffness for settlements \( z \leq z_t \) and
  \( z \leq z_r \),

-- related to kind of soil and stress state,

-- weakness coefficients \( \mu \) and \( \xi \) for shaft resis-
tance for overconsolidated cohesive soils and
very dense non-cohesive soils.

In the paper direct method utilising the resis-
tance \( q_e \) under the cone in CPT test for t-z and
q-z characteristics has been applied.

3. DETERMINATION OF SETTLEMENT CURVES IT TERMS OF CPT TESTS

The employment of test results of CPT is
exemplary presented for large diameter piles in
non-cohesive soils and Fundex piles in strati-

fied subsoil. The first example was extensively
described in the work of Gwizdala, 1984. In
this chapter some basic ideas will be recalled.

It is assumed that the load \( Q(x) \) in head of
pile, being a settlement function is a sum of
base \( Q_b(x) \) and shaft \( Q_s(x) \) resistances:

\[
Q(x) = Q_b(x) + \sum Q_s(x).
\]

(1)

Corresponding resistances of shaft and the base
of pile can be determined by following for-
mlae:

\[
Q_b(x) = q(x)A_b,
\]

(2)

and

\[
Q_s(x) = t(x)A_s,
\]

(3)

where: \( q(x) \) and \( t(x) \) denote resistances under
the base and along shaft of pile being
determined on the basis of cone resistance \( q_e \),
\( A_b \) and \( A_s \) are surfaces of the base and
shaft of pile, respectively.

When assuming that the resistance under the
base of pile can be described in terms of hyper-
bolic curve (q-z function) then on can obtain
following relation:
\[ q(s) = \frac{s}{a^* + \frac{s}{a^*} q_f}; \text{ for } s \leq 2r, \]  
\[ a^* = \frac{\Delta q}{\Delta q} \text{ denotes initial inclination of } q \]

curve, determined on the basis of interpretation of load test of large diameter piles (back analysis).

\[ a^* \] is a correction coefficient for hyperbolic function.

Required values of \( q_f \), \( a^* \) and \( \alpha^* \) were determined from in situ tests: Franke and Garbrecht (1977), Franke (1973), Spang (1972), Touma and Reese (1974). More details can be found in Gwizdala, 1984. The following relations were empirically obtained:

\[ q_f = \frac{1}{l_1 + l_2} \int_{a+h}^{b+h} \frac{1}{s} q_s(h) \, dh, \]  
\[ a^* = 28 - 4q_f \text{ in } \left[ \text{mm MPa} \right]. \]

the relation (6) is valid for sands only in the range of \( 2 \leq q_f \leq 5 \text{ MPa} \), and

\[ \alpha^* = 3 - D_p, \]  
where: \( q_s(h) \) is a resistance under the cone of CPT as a function of depth at the level of base of pile.

\( q_f \) is ultimate average resistance under the base of pile,

\( D_p \) is a diameter of pile base in [m],

\( l_1 \) and \( l_2 \) denotes zones in subsoil included in calculations of base resistance beneath and above its level, respectively:

\( l_1 = 4D_p; \quad l_2 = D_p, \)

\( f_b \) is bearing capacity of base coefficient, according to Table 1.

<table>
<thead>
<tr>
<th>Table 1</th>
<th>Point resistance factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>( q_f ) [MPa]</td>
<td>2.5</td>
</tr>
<tr>
<td>Gravel</td>
<td>0.80</td>
</tr>
<tr>
<td>Coarse sand</td>
<td>0.70</td>
</tr>
<tr>
<td>Medium sand</td>
<td>0.60</td>
</tr>
<tr>
<td>Fine sand</td>
<td>0.60</td>
</tr>
</tbody>
</table>

The relations for \( a^* \) and \( \alpha^* \) were determined on the basis of back analysis of loading test results of 20 piles. Assuming the average relations (6) and (7) and ultimate bearing capacity according to eq (5) the whole curves of base resistance \( Q_D(s) \) were determined (eq(2)).

The results in the form of compatibility coefficient \( \eta_w \) being the ratio of calculated and measured values are showed in Fig. 2.

<table>
<thead>
<tr>
<th>Table 2</th>
<th>Calculation results for large diameter bored piles and Fundex piles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of pile</td>
<td>Measured points</td>
</tr>
<tr>
<td>Large diameter bored piles</td>
<td></td>
</tr>
<tr>
<td>( s = 10 \text{ mm} )</td>
<td>126</td>
</tr>
<tr>
<td>( s = 20 \text{ mm} )</td>
<td>126</td>
</tr>
<tr>
<td>( s = 50 \text{ mm} )</td>
<td>126</td>
</tr>
<tr>
<td>( s = 100 \text{ mm} )</td>
<td>126</td>
</tr>
<tr>
<td>( s = 150 \text{ mm} )</td>
<td>126</td>
</tr>
<tr>
<td>( s = 10 \cdot D_p )</td>
<td>126</td>
</tr>
<tr>
<td>Fundex</td>
<td></td>
</tr>
<tr>
<td>all points</td>
<td>207</td>
</tr>
</tbody>
</table>
The above relations were presented in the form of histogram, which was next approximated by log-normal distribution function. The detailed values for all points investigated and chosen settlements together with parameters describing the distribution assumed are completed in Table 2. For whole curve of base resistance the compatibility coefficient \( \eta_M \) varies from 0.894 to 1.003 when coefficient of variation \( v \) takes the values from 0.131 to 0.342.

![Histogram of ratio of computed to measured base load](image)

**Fig 2** Histogram of ratio of computed to measured base load

The method presented has been verified by author's tests. For shaft resistance and \( \tau-z \) function following relations have been assumed:

\[
\tau_s = \frac{z_s}{z_{\text{max}}} \tau_{\text{max}} \quad \text{for} \quad z_s \leq z_{\text{crit}},
\]

\[
\tau_{\text{max}} = \frac{1}{f_s} \frac{q_s}{\tau_{\text{crit}}},
\]

where \( f_s \) is shaft capacity coefficient according to Table 3, \( z_s \) is a settlement for which the shaft resistance reaches ultimate value, Gwizdala, 1884, 1995.

An example of calculation results for given geotechnical profile and values of \( q_s \) regarding settlement curve \( Q(z) \) for total loads is presented in Fig.3. It can be generally seen that calculated values in comparison to loading test results are on the safe side. The example can be calculated automatically in terms of numerical program PALOS.

<table>
<thead>
<tr>
<th>( q_s ) [MPa]</th>
<th>10.0</th>
<th>15.0</th>
<th>20.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>80</td>
<td>120</td>
<td>180</td>
</tr>
<tr>
<td>Coarse sand</td>
<td>100</td>
<td>150</td>
<td>230</td>
</tr>
<tr>
<td>Medium sand</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fine sand</td>
<td>130</td>
<td>190</td>
<td>300</td>
</tr>
<tr>
<td>Silty sand</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Direct method of utilising the CPT results was additionally used for driven piles of Fun- dex type (Gwizdala and Tejchman, 1986). The analysis was performed for 15 piles of diameter equal to 457 mm, the length of 12 to 16 m driven in stratified subsoil. For load-settlement curve the hyperbolic function has been assumed. Bearing capacity of piles was calculated using CPT results applying the method recommended by Bustamante and Gianeselli (1986). After back analysis the average parameters describing hyperbolic dependence were determined. For whole set of investigated points equal to \( n = 207 \) following values of statistical parameters were obtained: compatibility coefficient \( \eta_M = 0.989 \), standard deviation \( \sigma = 0.288 \) and coefficient of variation \( v = 0.291 \).

Additionally the results of several measurements were approximated in terms of log-normal distribution function, Fig.4. The parameters were drawn up in Table 2.

4. CONCLUSIONS

Direct application of cone resistances from CPT tests in calculations of piles bearing capacity enables the determination of reliable settlement curves.

Utilisation of transfer functions of \( \tau-z \) and \( q-z \) type for shaft and base resistances, respectively allows for calculation of whole settlement curve up to ultimate load. The comparative analysis regarding loading tests shows that hyperbolic and parabolic functions are good engineering approximation of transfer functions. In the paper the characteristic parameters of these functions are estimated for various piles and different types of soils.
The values of compatibility coefficient close to unity and coefficient of variation changing from 0.13 to 0.30 are accurate enough for engineering practice to apply the analytical evaluation of settlement curve on the basis of geotechnical parameters.

Fig. 3. Calculated bearing capacity of piles in sand compared with load tests results.

5. REFERENCES


SIPT-method in determination of soil parameters and point resistance for driven piles

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SYNOPSIS: The stress wave measurements have been largely used in determining point and shaft resistances for driven piles. In Nordic countries stones and cobbles occurring in sand, gravel and till layers often have made CPT less applicable to determine soil parameters for friction piles. Therefore stress wave method has been applied to dynamic-static penetration and dynamic probing tests to determine point resistance in sand, gravel and moraine with small static penetration force (10-20 kN). The new application is called SIPT-method (Static Impact Penetration Test). The point resistance of SIPT has been used to calculate the bearing capacity of piles according to the application of CPT-method. The results have compared with stress wave measurements of piles.

1. INTRODUCTION
Determination of point resistance by CPT has been a useful in-situ method evaluating bearing capacity for friction piles. However in Nordic countries stones and cobbles occurring in sand, gravel and till layers often have made CPT less applicable in such cases.

Therefore a new method called SIPT (Static Impact Penetration Test) has been developed for Nordic circumstances to determine the point resistance also in stony soil ranging from sandy silt to gravel and till. The method enables the determination of point resistance with small static penetration force (10-20 kN).

Since 1989 some 35 SIPT-measurements have been carried out all over Finland together with dynamic probing and static-dynamic penetration tests. Some tests have been made in Germany and Russia, too. Some of the SIPTs have been carried out offshore on a ferry to determine soil parameters in-situ for marine sediments.

The point resistance is determined by measuring the stress wave in connection with driving the rod into soil during dynamic probing or static-dynamic penetration test. The rods are rotated to eliminate the skin friction.

Stress wave measurements are carried out in planned depths. Steel rods can be driven very hard reaching the maximum force of 100 kN. Rotating the rods shaft resistance is reduced and the point resistance can be calculated of the stress wave curve (figure 3) with CASE-method or with CAPWAP-analysis. The method simulates mini-piling giving the basic parameters for the computing of the full-scale piling.

Figure 1. Principal of the SIPT-method
2. STRESS WAVE MEASUREMENTS

2.1 Procedure
After probing has reached to the planned depth the stress waves generated by the driving hammer are recorded in an analyser and settlement of 5 blows is measured. A cushion is used between the rods and the hammer to adjust sufficient impact energy and the length of time. The rods are rotated during the driving to eliminate the skin friction. The fall height of the hammer is adjusted to achieve 2–4 mm penetration/blow. Distances between measuring levels varies usually from 0.5 to 2 m.

The ultimate load $R_u$ is calculated from the stress wave curves of each level by CASE-method. In case of high skin friction a CAPWAP-analysis can be done.

2.2 Point resistance
The total point capacity $R_p$ determined by stress wave analysis, is divided by the point area $A_{pfp}$ (1600 mm$^2$) to obtain the point resistance $q_{pfp}$. Because the area of point angle of SPT differs from those of the standard penetrometer (figure 3), the point resistances have been adjusted to be compatible with CPT results with a correction based on the theory by Durugonolu and Mitchell (1975) (figure 4).
2.3 Results
In addition to the total point capacity and point resistance in each level the results of SIPT also contain dynamic probing or static-dynamic penetration test results and if necessary several parameters derived from CPT cone resistance (figure 5).

\[ Q_p = A_p \frac{q_{cl} \cdot q_{cl}}{2} \]  \hspace{1cm} (1)

in which

- \( q_{cl} \) is the minimum cone resistance below the pile to the depth of 4D,
- \( q_{cl} \) is the average of the cone resistance recorded above the foundation level over a height of 8D,
- \( D \) is the diameter of pile,
- \( A_p \) is the cross-sectional area of the pile.

3.2 Skin resistance capacity
Skin resistance capacity can be computed from

\[ Q_s = \sum Q_s = \sum (q_w \cdot \Delta L) \]  \hspace{1cm} (2)

and from

\[ q_w = \frac{q_{cl}}{\alpha} \cdot q_{max} \]  \hspace{1cm} (3)

in which

- \( Q_s \) is the total capacity of skin friction (kN),
- \( Q_{cl} \) is skin friction in layer i (kN),
- \( q_w \) is skin friction in layer i (kN),
- \( q_{max} \) is maximum skin friction depending on soil and pile types and CPT cone resistance (15...200 kPa),
- \( q_{cl} \) is CPT cone resistance in layer i (kPa),
- \( s_i \) is cross-sectional perimeter of pile (m),
- \( \Delta L_i \) is the length of pile in layer i (m),
- \( \alpha \) is correction coefficient depending on soil and pile types and CPT cone resistance (30...200).

3.2 Comparison with full-scale tests
Examples of computed bearing capacities of single piles with above mentioned method and test results received from full scale tests can be seen in figures 6 and 7.

The summary of the bearing capacities of driven piles computed with SIPT-method and tested with stress wave method in construction phase can be seen in table 1.

Comparison of the computed and full-scale tested piles can be seen in figure 8.
Figure 6. Computed bearing capacities of driven pile by SIPT-method and the bearing capacity of a test pile according to the stress wave measurements after the rest period (a) in sand layers, Vihti, Finland and (b) in moraine layers, Jyväskylä, Finland.
Table 1. Summary of the test pile results computed with SIPT-method and measured with stress wave analysis during the construction phase.

<table>
<thead>
<tr>
<th>Location</th>
<th>Year</th>
<th>Soil type</th>
<th>Pile length (m)</th>
<th>Bearing capacity (kN)</th>
<th>Computed</th>
<th>Measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete piles 250x250</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Porvoo</td>
<td>1991</td>
<td>Sand</td>
<td>13</td>
<td>667</td>
<td>696</td>
<td></td>
</tr>
<tr>
<td>Askola</td>
<td>1992</td>
<td>Sand</td>
<td>11</td>
<td>397</td>
<td>403</td>
<td></td>
</tr>
<tr>
<td>Vihti</td>
<td>1993</td>
<td>Silty Sand</td>
<td>14</td>
<td>494</td>
<td>541</td>
<td></td>
</tr>
<tr>
<td>Vihti</td>
<td>1993</td>
<td>Silty Sand</td>
<td>16</td>
<td>535</td>
<td>534</td>
<td></td>
</tr>
<tr>
<td>Vihti</td>
<td>1993</td>
<td>Silty Sand</td>
<td>18</td>
<td>735</td>
<td>805</td>
<td></td>
</tr>
<tr>
<td>Vihti</td>
<td>1993</td>
<td>Silty Sand</td>
<td>18</td>
<td>859</td>
<td>847</td>
<td></td>
</tr>
<tr>
<td>Concrete piles 300x300</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Imatra</td>
<td>1991</td>
<td>Sand/Till</td>
<td>13</td>
<td>1030</td>
<td>1311</td>
<td></td>
</tr>
<tr>
<td>Imatra</td>
<td>1991</td>
<td>Sand/Till</td>
<td>13</td>
<td>1030</td>
<td>1113</td>
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<tr>
<td>Imatra</td>
<td>1991</td>
<td>Sand/Till</td>
<td>13</td>
<td>1030</td>
<td>1082</td>
<td></td>
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<tr>
<td>Imatra</td>
<td>1991</td>
<td>Sand</td>
<td>14</td>
<td>1345</td>
<td>1360</td>
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</tr>
<tr>
<td>Imatra</td>
<td>1991</td>
<td>Sandy Gravel</td>
<td>17</td>
<td>1010</td>
<td>1204</td>
<td></td>
</tr>
<tr>
<td>Imatra</td>
<td>1991</td>
<td>Sand</td>
<td>17</td>
<td>1403</td>
<td>1642</td>
<td></td>
</tr>
<tr>
<td>Imatra</td>
<td>1991</td>
<td>Sand</td>
<td>18</td>
<td>1427</td>
<td>1675</td>
<td></td>
</tr>
<tr>
<td>Lahd</td>
<td>1992</td>
<td>Sand</td>
<td>20</td>
<td>1102</td>
<td>914</td>
<td></td>
</tr>
<tr>
<td>Jyväskylä</td>
<td>1994</td>
<td>Till</td>
<td>14</td>
<td>1675</td>
<td>1677</td>
<td></td>
</tr>
<tr>
<td>Jyväskylä</td>
<td>1994</td>
<td>Till</td>
<td>14</td>
<td>1958</td>
<td>2065</td>
<td></td>
</tr>
<tr>
<td>Steel pipe piles 250x250</td>
<td>1995</td>
<td>Till</td>
<td>20</td>
<td>7785</td>
<td>7542</td>
<td></td>
</tr>
</tbody>
</table>

*Measured before rest period. 15% added to the result.

Figure 7. Computed (SIPT-method) and measured (stress wave analysis) values of bearing capacities in driven piles.

4. CONCLUSIONS

The comparison of the computed bearing capacities with full-scale measurements has proved that reliable results can be obtained with SIPT-method.

SIPT has also proved to be applicable in-situ method in circumstances where static penetrating method can not be used as in soils with stones and cobbles or when probing off-shore on a floating ferry. The same light weight multifunctional drilling rig that
is used for ordinary investigation work can be used for SIPT-measurements with the same stress wave analyser that is used for pile driving control.

Further development is needed with the equipment to make the measurements and registration part of the routine measures.

Further research is needed to study the possible pore pressure rise in silty and clayey soils because of dynamic penetration.

SIPT enables serious computing of point capacity and skin friction of piles with reliable parameters even in Nordic stony soils.

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Evaluation of Trench Backfill Compaction Using CPT - A Case Study

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SYNOPSIS: This paper presents the results of a relative compaction evaluation of a 2,000-foot-long (610 m) segment of a recently completed water transmission pipeline trench backfill. The trench was backfilled with slightly cemented well-graded gravelly sands. The compaction of the trench backfill along this pipeline segment, which was located along a city street, was in question due to inadequate observation and testing during backfilling and compaction operations. A simplified methodology for evaluation of quality of the compacted trench backfill by utilizing Cone Penetration Test (CPT) test data is presented in this paper. The CPT offered a number of advantages over conventional methods for the evaluation of the backfill compaction. The simple methodology presented can be readily utilized by the practicing engineers involved in evaluation of the quality of compacted fill deposits. Similar future studies on different types of soils will provide wide acceptance of the CPT by the practicing engineers as a powerful tool for the evaluation of the quality of compacted soils.

1. INTRODUCTION
In 1993, a Southern California water agency which supplies water to the residents of a desert area constructed a 76-mile-long (122 km) water transmission pipeline. The pipeline consisted of a 60-inch-diameter (152 cm) cast iron pipe. The water agency hired an outside construction manager for this project. Converse Consultants Inland Empire (CCIE) was the project geotechnical consultant and prepared the preliminary soils investigation report. The project specifications allowed the construction contractor to hire his own geotechnical firm to observe and test trench backfill. The construction manager evaluated the work of the contractor's geotechnical consultant but also retained CCIE for consultation as necessary.

The construction of an approximately 2,000-foot-long (610 m) section of the pipeline traversing through the High Desert city of Hesperia was not observed by the construction manager. A review of the field density test results supplied by the contractor's geotechnical consultant indicated that the number of field density tests conducted within this segment of the pipeline trench was far less than that required by the project specifications. The project specifications required that at least one field density test be performed for every 250 linear feet (76 m) and two vertical feet (0.61 m) of compacted trench backfill. The specifications further require that the Sand Cone Method (ASTM Standard D1556) or the Nuclear Gauge Method (ASTM Standard D2922-81) be used to evaluate field density and that at least one field density test should be performed by utilizing sand cone for every four consecutive tests.
The trench section in question was located along a residential street that was to be paved following the completion of the pipeline construction. Initial field density testing by the owner’s geotechnical consultant utilizing the nuclear gauge test method at a number of locations within the segment of the trench in question indicated lower relative compaction than the 90 percent minimum required by the project specifications. Based on these results, the adequacy of backfill compaction along the entire segment of the pipeline was in question. Accordingly, the owner decided to perform further investigation to estimate the relative compaction of the backfill for the entire trench.

2. INVESTIGATION METHODOLOGY
Field density tests utilizing either Sand Cone Method or Nuclear Gauge Method are commonly used by the industry to evaluate in situ densities of compacted soils. These tests were performed at randomly selected locations along the trench section to evaluate the degree of compaction of the fill.

2.1 Field Density Testing
The field density tests were performed within two pits excavated along randomly selected location along the pipeline alignment. These trenches were excavated on one side of the pipe up to the depth of the invert. Topography along this segment of the pipeline is relatively flat. The depth of the pipe invert was about 12 feet (3.66 m) below final ground surface along the alignment. The field representatives for the Contractor, the Contractor's geotechnical consultant, the construction management firm and its geotechnical consultant were present during excavation and field density testing. All field density tests were performed side by side by the same technicians, one from the contractor's geotechnical firm, and one from the owner's geotechnical firm, each with more than eight years of experience.

Figure 1. Gradation Range of Backfill Soils

Representative bulk samples of the trench backfill soils were retrieved to perform laboratory tests. This trench section was backfilled with excavated soils comprising gravelly sands with silts. These native alluvial soils comprised mainly subangular to angular quartz particles and were slightly cemented. Before placing as trench backfill, the excavated soils were made free of particles larger than three inches in maximum dimension. Much of the cementation which is typical of native soils in desert areas was destroyed due to remolding actions during excavation, screening, moisture conditioning and compaction processes.

The range of gradation of the trench backfill soils is shown in Figure 1. The coefficient of uniformity (C_u) and the coefficient of curvature range from 10 to 36 and 1.0 to 1.6, respectively. The trench backfill soils are classified as well-graded sand (SW) with gravel and silt. The specific gravity of the soil particles is 2.68. The maximum dry density and the optimum moisture content determined in accordance with the ASTM Standard Method D-1557.
were found to be about 130pcf (20.4 kN/m³) and 8.5 percent, respectively. Based on confined compression tests performed during the preliminary soils investigation along this segment of the project, the coefficient of compressibility (Cₜ) of the native soils ranged from about 0.005 to 0.008 ft²/ton (0.05 to 0.08 m³/MN). This range of Cₜ value indicates that the trench backfill soils have low compressibility.

The relative compaction versus depth profile established from the results of the field density tests performed at one of the test pits are shown in figures 2 and 3. Figure 2 shows the results obtained at the north side of the east-west-oriented trench. The Sand Cone and Nuclear Gauge tests yield similar results. These results also show that relative compaction of the trench backfill decreases with depth and that at depths about 6.0 feet (1.83 m) below surface grade, the compaction achieved does not meet the project requirement of at least 90 percent relative compaction.

Figure 3 shows the results from the north side of the trench. At this location, the relative compaction indicated by the two types of tests differs substantially. The relative compactions obtained by the Sand Cone method are consistently lower than those obtained by the Nuclear Method. These results, together with similar experiences with other projects, show that field densities of compacted soils performed by the same technicians at close locations may differ substantially when two different field density test methods are used. The differences can be attributed to a number of factors, including weather, local variations in compaction achieved, soil type, proximity of the test location to the pipe and instrument and/or human error. The Nuclear Gauge, in particular, provides erroneous readings when used near pipes. These test results provided further evidence that the trench backfill might not have received adequate compaction. Further testing, however, was necessary to
Figure 4. Results of CPT at Station 188 + 00, South Side of Pipe

Figure 5. Results of CPT at Station 191 + 00, South Side of Pipe
evaluate backfill compaction for the remainder of the 2,000-foot-long (600 m) trench segment.

However, there was disagreement between the parties involved regarding which field density test method would be utilized for further investigation. Experience shows that, when performed properly, the Sand Cone Method provides more accurate measurements of in situ density than the Nuclear Gauge Method. However, determination of field density by the Sand Cone Method is more laborious, time-consuming, and, therefore, costly.

For these reasons, the Nuclear Gauge Method is usually utilized when field densities are to be evaluated in compacted fill at a relatively large number of locations. The Sand Cone Method is utilized occasionally or when the results of a test performed by the Nuclear Method are in doubt. Both of these test methods, however, determine field densities at discrete locations. Also, to determine the field densities of compacted backfill, as in the present case, excavation, removal and recomposition of the backfill soils above the tests elevations are required. Thus, even the Nuclear Method rendered itself economically inappropriate for testing field densities of completed large fill.

2.2. Cone Penetration Testing
For granular backfill soils, CPT provides a convenient, nondestructive and economical method for establishing continuous relative compaction profiles. Relative compaction profiles at a large number of locations along the alignment can be established by utilizing CPT results in a relatively short period of time.

A standard electrical CPT program was, therefore, undertaken as part of the field investigation to assist in the evaluation of the trench backfill compaction. A total of 20 soundings at intervals of about 100 feet (30.5 m) were performed along the trench alignment. Laterally, the sounding locations were carefully selected as close as possible to the pipe, but far enough away to avoid hitting the pipe itself. The maximum depths of probe penetration were based on the as-built pipeline profile. Both cone tip resistance, \( q_c \), and sleeve friction, \( f_s \), were measured. Typical test results are presented in figures 4 and 5.

3.0 ANALYSIS AND INTERPRETATION OF CPT RESULTS
Data obtained during CPT tests can be used for profiling and identification of trench backfill. Soil classification charts for standard electric friction cone are presented by Douglas and Olsen (1981) and Robertson and Campanella (1983). The simplified relationship by Robertson and Campanella is shown in Figure 6. In the present case, the classification of the fill soil is known beforehand from samples obtained during preliminary soils investigation and during field density testing by Sand Cone and Nuclear Gauge method. This has provided an opportunity to examine the applicability of relationships presented in Figure 6. Based on
the range of measured values ($q_4$) and $f_1$ as seen in figures 4 and 5, and the chart in Figure 6, the backfill soils can be classified as sands. This finding agrees with the known classification.

The main purpose of the present investigation, however, is to establish relative compaction profiles along the trench alignment based on the cone tip resistances. The backfill soils along the trench segment are granular and dry. Various researchers (Chapman and Donald, 1981, Baldi et al., 1982, Schmertmann, 1978, Robertson and Campanella, 1983, Jamiołkowski, 1985) have presented correlations between relative density ($D_r$) and tip resistance ($q_4$) for such soils. The cone tip resistance ($q_4$) data, after suitable calibration, can be used directly to determine relative density profiles. Once the relative compaction profiles are known, the information can be utilized to determine the relative compaction ($R_c$) profile. This would require establishment of an appropriate relationship between the relative density ($D_r$) and relative compaction ($R_c$) for the given soil type.

The currently proposed relationships between $q_4$ and $D_r$ are based mainly on data obtained from CPT tests on calibration chambers. The most comprehensive of the relationships is the one suggested by Jamiołkowski (1985). This relationship between cone tip resistance and relative density, as presented by Meligh (1987), is included as Figure 7. This relationship is based on results from cone penetration tests performed on uncedmented, normally consolidated, mainly quartz sands placed in a calibration chamber. The correlation in Figure 7 is valid for drained conditions and is based on the following assumptions:

1. A linear relationship exists between $D_r$ and $\log_{10}\left(\frac{q_4}{(\sigma''_w)^{0.5}}\right)$
2. As a first approximation, the value of $\alpha$ is assumed to be 0.5 and
3. The uncertainties involved in the measurement of $q_4$ and $\sigma''_w$ during CPT tests in calibration chambers are negligible compared to those involved in determination of relative density ($D_r$).

The relevant properties of the sands in Figure 7 are presented by Meligh (1987). The soils within the pipe trench are normally consolidated as these have been recently excavated, backfilled and compacted. The remolding actions during these processes have also destroyed the cementation of backfill soils. Therefore, it is suggested that among the various suggested correlations between $q_4$ and $D_r$, the one presented in Figure 7 would be most applicable to the present case.

The data in Figure 7 is bounded by the two correlations corresponding to high compressibility and low compressibility sands, respectively. In general, the correlation can be expressed as follows:

$$D_r = \Delta + 66 \log_{10}\left(\frac{q_4}{(\sigma''_w)^{0.5}}\right)$$

(1)

where the correlation coefficient, $\Delta$, depends on the compressibility of the given sand. The unit of $q_4$ and $\sigma''_w$ is kN/m$^2$.

Equation 1 is based on tests performed in calibration chambers. A chamber size correction is required to be applied to Eq. 1. It is applied by dividing measured cone tip
resistance \(q_r\) in the field, to obtain \(q_r\) in Equation 1, by the correction factor, \(K_q\). The correction factor \(K_q\) is given by the following relationship:

\[
K_q = 1 + 0.2 \left(\frac{D_r - 30}{60}\right)
\]  

(2)

One method of estimating the coefficient, \(\Delta\), is to determine the relative density of the penetrated soil at a known depth. This would require the determination of the field density at that location. The relative density can be determined from a knowledge of the measured field density and the maximum and minimum index densities of the given soil type. The minimum index density \(\gamma_{\text{min}}\) of the backfill soils was determined in the laboratory to 94 pcf (14.8 kN/m³) following ASTM Standard Method D4253-83. The maximum index density \(\gamma_{\text{max}}\) of the backfill soils was estimated to be 135 pcf (21.2 kN/m³) from Lambe and Whitman (1969). As presented above, the maximum dry density \(\gamma_{\text{max}}\) of the backfill soils as per ASTM Standard D-1557 was 130 pcf (20.4 kN/m³). From a knowledge of the index densities and the maximum dry density, the relative compaction and the corresponding relative density \(D_r\) for a given dry density \(\gamma_d\) can be evaluated from the following two relationships:

\[
R_c(\%) = \frac{\gamma_d(100)}{\gamma_{\text{max}}}
\]  

(3)

\[
D_r(\%) = \frac{\left(\frac{1}{\gamma_{\text{min}}} - 1\right) \times \gamma_{\text{max}}}{\left(\frac{1}{\gamma_{\text{min}}} - 1\right) \times \gamma_d\left(\frac{1}{\gamma_{\text{max}}} - 1\right) + 100}
\]  

(4)

The relationship between the relative density \(D_r\) and the relative compaction \(R_c\) for the backfill soils is presented in Figure 8. It is seen that for all practical purposes, a relative density of 60 percent corresponds to about 90 percent relative compaction for the backfill soils.

In Figure 2, the measured relative compaction at a depth of about 6.0 feet (1.8 m) below ground surface is seen to be 90 percent and thus corresponds to 60 percent relative density. The measured cone tip resistance \(q_r\) at the same depth, as seen in Figure 4, is about 93 tsf (908 tons/m²). From Eq. 2, the chamber correction factor \(K_q\) is found to be 1.1. The corrected cone tip resistance \(q_r\), corresponding to 60 percent relative density is about 85 tsf (830 tons/m²).

From Figure 3, it can be seen that the trench backfills above this test depth are compacted, on average, to about 93 percent relative compaction, which corresponds to a dry density of about 121 pcf (19.0 kN/m³). At the time of the field density testing, the average moisture content of the backfill was about 4.0 percent, resulting in an average wet density of about 125 pcf (19.6 kN/m³). The effective vertical overburden pressure at a depth of 6.0 feet (1.8 m) equals about 0.375 tsf (3.66 kN/m²). Substituting these values in Eq. 1, the value of the correlation coefficient, \(\Delta\), is found to about (–116). Finally, the correlation between the relative density \(D_r\) and cone tip resistance \(q_r\) for the backfill soils becomes:

\[
D_r = -116 + 66 \log_{10} \left[\frac{q_r}{(\sigma_0')^{0.5}}\right]
\]  

(5)
Figure 9. Theoretical Tip Resistance Profile

This correlation plots close to the lower limit of the correlations shown in Figure 7 but falls well within the low compressibility range. This observation corresponds with the measured low compressibility of the backfill soils.

For a given relative density, the cone tip resistance profile can be established from the following relationship, which is obtained by rearranging terms in Eq. 5 and applying the chamber size calibration factor $K_\alpha$:

$$q_{\alpha} = K_\alpha (\sigma'_w)^{0.4} \log_{10}(D + 116) \quad (6)$$

The purpose of the present analysis is to determine depths within the trench at which the relative density is less than 60 percent. In this case, Eq. 6, simplifies to:

$$q_{\alpha} = 464(\sigma'_w)^{0.4} \quad (7)$$

The estimated cone tip resistance ($q_{\alpha}$) profile for a homogeneous fill compacted to 60 percent relative density is shown in Figure superimposed over Figure 4. From such

Figure 10. Comparison of the Measured Tip Resistance and Theoretical Tip Resistance Corresponding to 90 Percent Relative Compaction (Station 188+00)

plots, the depths at which the backfill compaction is less than 60 percent relative density, and hence, less than 90 percent relative compaction, can be clearly identified. Figure 10 shows that at Station 188+00, the compaction of the backfill between the depths of about 6 feet (1.8 m) and 13.5 feet (4.1 m), which is the bottom of the trench, does not meet project specifications of at least 90 percent relative compaction.

A similar plot in Figure 11 shows that at Station 191+00, the backfill compaction between the depths of 6.5 feet (2.0 m) and 15.5 feet (4.7 m) below ground surface does not meet project specifications. Similar analysis was performed for all 20 sounding locations along the trench alignment to identify the depths of compaction non-compliance. It was shown that the relative compaction of trench backfill within the pipe zone, defined as the zone from the 9. Figure 10 shows the same profile at bottom of the trench to about 1.0 feet (0.3m) above
4. CONCLUSIONS
The usual practice of determining relative compaction of compacted fill by utilizing sand cone and nuclear methods is not suitable for evaluating field densities of completed large fills. These methods are destructive and involve test pit excavation, backfilling and recompaition, thus rendering themselves economically impractical for such large fills. Furthermore, the field densities obtained by utilizing the two test methods at the same location can be substantially different and may result in questionable compaction evaluation.

The cone penetration test provides a fast, economic and accurate means of evaluating the compaction of recently completed large fills comprising granular soils. After suitable calibration for the type of soils under consideration, the correlations presented in Figure 7 can be used with acceptable accuracy for evaluating relative densities of such compacted fill.

The simple methodology presented in this paper can be readily utilized by the practicing engineers to estimate densities of completed fill deposits. Similar future studies on different soils will provide wide acceptance of the cone penetration test by the practicing engineers as a useful tool for evaluation of the quality of compacted fills.

5. REFERENCES


Electrical Conductivity Cone Testing used in an Integrated Site Investigation

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Fagro Limited, Hemel Hempstead, England

Satar Hasan
Fagro Limited, Hemel Hempstead, England

SYNOPSIS: The measurement of a material’s electrical properties can be used to determine if it is contaminated. The electrical conductivity cone provides standard in situ data on soil strength/stratigraphy together with in situ measurements of the ground’s electrical conductivity. The use of this cone, as an integral part of a site investigation of a site contaminated with acidic tars, produced detailed information on the extent of the migration of the contaminant. Using a high proportion of cone testing to soil sampling significantly reduced the disruption of the site and the fieldwork period. There was also a reduction in the risks to site personnel and the local environment of exposure to harmful contaminants. The resulting site investigation provided both comprehensive coverage of the site and accurate information on the spatial extent of the contaminated soils.

1. INTRODUCTION
Site investigation of contaminated sites has been traditionally carried out using intrusive and disruptive drilling and excavation methods. Robertson et al. (1995) describes the technical and economic benefits of using the Cone Penetration Test method with the geo-environmental cones to investigate potentially contaminated sites.

The measurement of in situ electrical properties to detect soil contamination plumes is now frequently being used. The use of the electrical resistivity or conductivity cone for detection of groundwater contaminant plumes has been researched extensively (Hornell 1988; Campsells and Weersmees, 1989; Woeller et al., 1991, Strutynsky et al., 1991; Tonks et al., 1993). Figure 1 shows the layout of a 10cm² friction conductivity cone penetrometer. The electrical conductivity cone works on the principal that the measured voltage drop across a pair of electrodes at a certain current is proportional to the electrical resistivity of the soil. Resistivity (R) is equal to the reciprocal of conductivity (C) as given in Equation 1.

\[ C \text{ (mS/m)} = 1000 / R \text{ (Ω.m)} \]  

An investigation of a site in the UK was carried out to determine the extent of contamination of the ground from previously buried tar. The tars are known to be acidic in nature and had been placed some 35 years ago and evidence suggested that the tar had migrated since placement. A programme of investigation was planned to avoid disruption and damage of the site.

To achieve these aims a desk study was carried out followed by a programme of site investigation. The site investigation commenced with non intrusive geophysical surveying of the sub surface to map the contamination. The results of the geophysical mapping were used to locate the Electrical Conductivity Cone Penetration Tests (EC-CPTs).

The data from the EC-CPTs at this site were employed to determine the precise zones of discrete undisturbed soil sampling programme using the ‘Mostap’ sampler rom the CPT
vehicle. These samples were taken for geotechnical description and chemical analysis. The results of which were used to correlate the soils and contamination data from the programme of CPTs. This approach to investigation of a contaminated site resulted in detailed information on the spatial extent of the contamination and significantly reduced the damage and disruption to the site.

2. SITE HISTORY
At the turn of the century the site was used to extract sand. When the sand extraction was finished the sand pits were filled with sludge from the nearby sewage works as well as material from a nearby brass foundry. One of the pits was filled with waste tar from a nearby benzole plant until the 1960's. The site was then landscaped and part of these works involved excavating a trench to the south of the tar pool to allow tar to drain from the pool into one of the southern sludge beds. Figure 2 shows the layout of the site prior to landscaping.

Figure 1. Layout of Conductivity cone

Figure 2. Site Investigation Plan
3. SITE GEOLOGY
The local lithology consists of alluvial deposits, predominantly of peaty clay and peat with sand horizons, overlying glacial sand and gravels which in turn overlie Sandstone. Fill materials, placed as part of the landscaping works, are present across the whole site, thickening to about 8m in the centre of the investigated area. The present distribution of the soils reflects both the original geology and geomorphology and the effects of man’s activities on the area.

4. INVESTIGATION PROGRAMME
When the tar was discovered at ground surface some distance from the original tar filled areas it was obvious that the extent to which the buried tar material had migrated both horizontally and vertically was not known.

In order to avoid the excessive disruption which a conventional large drilling and sampling programme would have caused a less intrusive investigation was carried out.

The acidic nature of the tar was used to locate the extent of the contamination in plan using electro-magnetic (EM) and electro-resistivity (ER) geophysical investigation techniques.

Subsequent to the desk study, a site investigation programme consisting of four phases of work was carried out, these were:

- topographic surveying
- geophysical surveying
- cone penetration testing, using a variety of cone types
- push-in soil sampling

5. GEOPHYSICAL SURVEYING
The principle behind the EM technique is the introduction of a primary electromagnetic field at a specific frequency into the ground, which induces a secondary electromagnetic field in the subsurface. With a portable EM meter the resultant EM field is measured. The presence of geological features or man-made structures generally give rise to a contrast in the electromagnetic field. The whole of the site was covered by the EM survey on a 5m grid spacing. This system investigated the average apparent conductivity of the ground over a depth range of approximately 5m.

With the EM technique an electrical current, is introduced into the ground via a pair of metal stakes. Variations caused by geological or hydrogeological conditions will effect the subsurface current flow and alter the electric potential patterns which are measured via a second pair of metal stakes within and in-line with the first pair. Variations in the ground’s apparent resistivity with depth is measured by increasing the spacing between the electrodes. A total of five EM soundings were carried out in a line running West to East 100m north of the grid baseline. This line went through the expected location of the tar pool.

6. EM & ER SURVEYING RESULTS
The result obtained from the EM survey is presented on Figure 3 as a contour plot of apparent conductivity. The range of the measured apparent conductivity was between 20 and 300 millisiemens per metre (mS/m) occasionally much greater than 300. As this surveying method is taking an average of the electrical properties of the ground the trigger value of conductivity is lower than the actual values measured directly in the laboratory or with the EC cone.

A trigger value of approximately 60 mS/m was used to identify areas for further investigation for this site. Figure 3 shows several areas which are above the trigger value of apparent conductivity. There are both localised and linear trends in the high apparent conductivity contour signatures.

These anomalous areas were further investigated by the CPT method, to positively identify their nature. The location of some of the EC-CPTs are shown on Figure 3.

7. CONE PENETRATION TESTING
The CPT work was carried out using a number of cone penetrometers, a standard friction cone, a piezo-friction cone and a conductivity friction cone. The locations of these CPTs are shown on Figure 2 with test suffix F, denoting the use of a friction cone, P for piezo-friction cone and C for conductivity friction cone.
Figure 3. Results of EM Surveying

The friction cone was used initially to provide general information on the stratigraphy of the soils at the site. The piezo-friction cone was then used to give more detailed data on the strength and organic content of the soft clay deposits as well as the presence of sand layers in the clay strata. The conductivity friction cone tests were located to delineate the extent of the contaminant plume associated with the acidic tar. Their locations were designed using the results of the geophysical surveying and where tar was seen exiting at the ground surface.

The friction cone data clearly identified the natural soils and fill materials at the site. The summary of the variation of the stratigraphy at the site is presented in Table 1.

The fill materials were found to be granular. The alluvium deposits included peaty clays, peat and sands. Beneath the alluvium the sands were coarse in nature with some gravel. This deposit was considered to be the weathered material from the underlying sandstone bed-rock.

<table>
<thead>
<tr>
<th>STRATA</th>
<th>BASE DEPTH</th>
<th>BASE DEPTH</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum (m)</td>
<td>Maximum (m)</td>
</tr>
<tr>
<td>Fill</td>
<td>1.00</td>
<td>7.20</td>
</tr>
<tr>
<td>Alluvium</td>
<td>5.30</td>
<td>8.50</td>
</tr>
<tr>
<td>Sand</td>
<td>7.40</td>
<td>8.30</td>
</tr>
</tbody>
</table>

The results of the EC-CPTs are presented as profiles of measured conductivity with depth on Figure 4 for tests carried out in North South and East West directions. Also presented on Figure 4 is the interpreted soil stratigraphy from the friction cone data. These tests provided detailed data on the vertical extent of the contaminated areas highlighted by the EM and ER work. Whereas the resolution of the geophysical work is some 5 metres vertically and horizontally the spacing of the electrodes on the EC cone are 5 cm giving precise resolution of contaminated material.

In order to quantify the amount of contaminated soil from the EC-CPTs the back ground or clean values of measured
Figure 4. Cross Sections of EC-CPT Results

Conductivity from the EC-CPTs needs to be established. The values of measured conductivity made by the EC cone in uncontaminated soil are shown in Table 2. These have been compiled by correlation with the soil sampling.

A geological cross section is presented on Figure 5, which represents an East-West cross section line AA shown on Figure 3.

<table>
<thead>
<tr>
<th>STRATA (un-contaminated)</th>
<th>Conductivity (mS/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill</td>
<td>&lt;150</td>
</tr>
<tr>
<td>Alluvium</td>
<td>50-100</td>
</tr>
<tr>
<td>Sand</td>
<td>20-120</td>
</tr>
</tbody>
</table>

Proceedings CPT'95
Figure 5. Geological Cross Section
This section was inferred from the CPT results and the soil sampling. The section shows the interpreted location of the tar pit and the lateral extent of contaminated strata. The EC-CPTs indicate that the contamination has migrated laterally via the granular strata in both the made ground and the alluvium. There is also some downward migration from the base of the alluvium sands into the top of the underlying alluvial clays.

Values of conductivity from the ER surveying are presented as apparent resistivity on Figure 5. Due to the averaging of apparent resistivity the ER technique reveals the presence of high conductivity not the precise distribution of the contamination. The contaminated strata are shown on Figure 5.

9. REFERENCES


Piezocone settlement prediction parameters for embankments on alluvium.

Gary Jones
Stellen Robertson and Kirsten, Johannesburg, South Africa.

Eben Rust
University of Pretoria, South Africa.

SYNOPSIS: A database has been built up of road embankment performance over deep alluvial deposits over a period of twenty years in South Africa. This information comprising settlement monitoring records has been correlated both with the original preconstruction investigation laboratory test results and also with piezocone tests.

From these three data sets it has been shown that laboratory $m^*$'s have given reliable predictions of settlements but that laboratory $c_v$'s have predicted rates of settlement between five and ten times slower than measured.

The measured settlements and settlement rates have been correlated with piezocone tests and this shows that for the recent alluvial deposits the constraint modulus coefficient, $\alpha_m$ is given by:

$$\alpha_m = 2.75 \pm 0.55$$

The data also shows that:

$$c_v = 150t/50$$ ($c_v$ in m$^2$/year and t in minutes)

where $c_v$ represents the coefficient of consolidation back analysed from measured embankment settlement rates.

1. INTRODUCTION

Piezocone testing has been carried out in South Africa for about twenty years. The primary reason for its development was a surge in freeway building along the east coast of Natal in the early 1970's; this involved the investigation of about forty river crossings over a length of about 300 kms. A typical crossing comprises two approach embankments to a central bridge with each embankment averaging about 100m in length and 7m height overlying very soft to soft estuarine deposits of sands, silts and clays about 20m deep. Because of the potential problems the pre-construction geotechnical investigations were thorough and the original field and laboratory test data is available. At many of the embankments reliable settlement monitoring has been undertaken since construction began. This indicates settlements varying from 0.3m to 3m.

This data has been collated and piezocone testing has subsequently been carried out both at sites which had and had not previously had piezocone testing (Rust and Jones, 1990; Jones and Rust, 1992).

In essence, therefore, three data sets were obtained, viz. measured settlements, laboratory tests and piezocone tests. From these, comparisons have been made between measured embankment performance and the laboratory-based predictions of settlements and also rates of settlement, and correlation coefficients established to enable performance predictions to be made from piezocone data.
2 GEOLOGY
At most of the embankment sites the underlying rock comprises Ecca shales or Dwyka tillites of the Karoo sequence. In some cases these strata are overlain by cretaceous rocks and in others the Karoo sequence is absent and the recent alluvial deposits are underlain directly by either Gondwana Cape Supergroup quartzitic sandstone or the older Proterozoic granites. The Karoo sedimentary shales and mudstones are extensively intruded by dolerite sills and dykes which in many cases have controlled the river courses. The river estuaries have been formed by the global climate and eustatic sea level changes and hence have a general similarity along the Natal east coast in both shape and size and in the estuarine deposit materials. In view of their geological historical conformity the sites may be considered as belonging to a single group. Nevertheless the sites have marked differences in material types and extents which reflect their complex deposition and erosion histories. It is indeed the complex layering which makes piezocone testing particularly appropriate for their investigation.

3 METHODOLOGY
Comparisons of measured settlements with laboratory test-based predictions and with piezocone-based predictions require a definition of settlement and its components.

Measured total settlements were divided into local yield, immediate settlement, primary consolidation and secondary compression. In some cases this sub-division was possible on the basis of availability of detailed field and laboratory results and in other cases some assumptions were necessary regarding stress conditions, relative values of drained and undrained moduli and Poisson’s ratio. For these materials the results show that the overall conclusions are not sensitive to these assumptions; nevertheless the distinction of the individual settlement components is important.

Prediction of moduli, and hence settlement, from cone penetration test data generally utilizes the following equation in which $M$ represents the combined immediate settlement and primary consolidation and $\alpha_m$ is a material dependent parameter:

$$\frac{1}{M} = m_1 = \frac{1}{\alpha_m q_c}$$

Where $M$ is the constrained modulus, $m_1$ is the coefficient of volume compressibility, $\alpha_m$ is the constrained modulus coefficient and $q_c$ is the cone pressure.

Generally the settlement records were analysed using Asaoka’s (1978) method and this enabled a consistent separation to be made between primary consolidation and secondary compression.

Three-dimensional effects were also considered as these could be significant if embankment widths are small relative to the depth of subsoil. However for all cases the freeway dual carriage way embankments were wide (50 - 100m) compared with alluvium depths and adjustment of settlements would be less than 5%; 3-D effects were therefore ignored in the data interpretation. Coefficients of consolidation, $c_v$, have been estimated from piezocone dissipation tests generally using $t_{so}$, the time for half dissipation, in a variety of models. The authors’ experience suggests that a simple relationship is more than adequate for the problem of predicting embankment settlement times viz:

$$c_v = \frac{f(T)}{t_{so}}$$

Where $c_v$ is the coefficient of consolidation, $f(T)$ is a constant for any cone and $t_{so}$ is the time for half dissipation.

Earlier research (Jones and Van Zyl, 1981) had shown that $f(T) = 50$ where $c_v$ is expressed in $m^2/yr$ and $t_{iso}$ in minutes. The work reported here was to redefine $f(T)$ using a data base of measured actual rates of settlement rather than a comparison of $t_{iso}$'s with laboratory $c_v$'s.

It is important to stress that the $c_v$ derived from analysis of the measured settlement is in effect a global or macro $c_v$ which is a resultant of both horizontal and vertical micro $c_v$'s. In prac-
tice it has been found that for the recent alluvial deposits at these sites typically $c_h = 2c_v$. If the findings given here were to be transferred to a situation where the overall geometry of the problem was dissimilar, or where the material permeability anisotropy was more pronounced, then it would be necessary to exercise caution in the definition of the terms used.

4 INVESTIGATION

The field investigation consisted of piezocone testing at existing embankments where settlement records were available. The piezocone itself used a filter element at the base of the 35mm dia 60° cone complying in all respects with the ISSMFE reference test recommendations. The procedure adopted also complied with these recommendations. The data logging system was connected to both a chart recorder and a laptop computer so that instantaneous read out of cone pressures and pore pressures was available. Friction sleeves are not generally used in South Africa since material descriptions have been well correlated with excess pore pressure/cone pressure ratios (Jones and Rust, 1982).

Piezocone testing was carried out adjacent to the selected embankments in order to measure virgin characteristics. This was feasible because the pre-construction investigations had defined the strata in detail. In some cases additional CPTU’s were conducted through the embankments both to check that the external CPTU data could be interpolated through the embankment subsoil and to ascertain whether any excess pore pressures still existed. In addition settlement assessments could be checked by measuring depths to deformed original ground levels.

5 SETTLEMENT

5.1 Field Data

Typical CPTU results are given in Figures 1 and 2 showing respectively a layered and a relatively homogeneous subsoil.

Table 1 sets out a brief summary of each embankment giving the height, clay thickness, typical cone pressure and measured settlement. An estimate is then made of the yield and secondary components of the measured settlement, and these are subtracted from the total settlement so that an $c_{e_{corr}}$ is calculated which represents only the immediate and primary consolidation settlements. At a number of embankments more than one data set is given and this reflects the extent
Table 1. Constrained modulus coefficient, $e_{cm}$, from settlement analyses.

<table>
<thead>
<tr>
<th>Embankment</th>
<th>Height m</th>
<th>Clay Thickness m</th>
<th>Typical Cone Pressure kPa</th>
<th>Measured Settlement m</th>
<th>Yield and Secondary m</th>
<th>$e_{cm}$</th>
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<tr>
<td>Manzambyama</td>
<td>2.5</td>
<td>11</td>
<td>250</td>
<td>1.265</td>
<td>0.45</td>
<td>3.04</td>
</tr>
<tr>
<td>Umlalazi 1</td>
<td>8.4</td>
<td>14</td>
<td>350</td>
<td>1.30</td>
<td>0.165</td>
<td>3.38</td>
</tr>
<tr>
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<td>7</td>
<td>210</td>
<td>1.80</td>
<td>0.40</td>
<td>3.33</td>
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<tr>
<td>Umlhlatuze</td>
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<td>15</td>
<td>200</td>
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<tr>
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<td>Prosprietor Fall</td>
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<td>2.9</td>
<td>11</td>
<td>420</td>
<td>1.2</td>
<td>0.02</td>
<td>1.73</td>
</tr>
<tr>
<td>Umzimba</td>
<td>5.6</td>
<td>13</td>
<td>450</td>
<td>2.6</td>
<td>0.60</td>
<td>2.97</td>
</tr>
<tr>
<td></td>
<td>4.0</td>
<td>13</td>
<td>430</td>
<td>3.13</td>
<td>0.25</td>
<td>2.12</td>
</tr>
<tr>
<td>Umzimba</td>
<td>4.0</td>
<td>6.9</td>
<td>320</td>
<td>1.73</td>
<td>0.25</td>
<td>1.86</td>
</tr>
<tr>
<td></td>
<td>360</td>
<td>460</td>
<td></td>
<td></td>
<td></td>
<td>1.76</td>
</tr>
<tr>
<td>Umhlungase</td>
<td>7.0</td>
<td>12</td>
<td>560</td>
<td>2.94</td>
<td>0.64</td>
<td>1.91</td>
</tr>
</tbody>
</table>

of the available settlement records which could be correlated with either previous or later piezocone results. At Bot River the embankment was built in two stages, initially 3m followed by a further 2m, to allow for dissipation of excess pore pressures, because of the detailed monitoring of sufficient data was available for this site to be used for both stages. Similar interim and final data was available for Goukamma. The mean and standard deviation of $e_{cm}$'s are given in the table, the mean value is 2.63 and standard deviation 0.59.
Analysis of the settlement components shows that the ratio of total settlement to that due to immediate and primary settlements together is 1,15 with a standard deviation of 0,13. In other words yield and secondary settlements combined, average about 13% of the immediate and primary settlements with a range of from about zero to 25%.

5.2 Laboratory Data

The laboratory coefficients of compressibility, $m_v$, were correlated with the field cone pressures, $q_c$, in order to derive laboratory based $\alpha_m$'s as opposed to the totally field based $\alpha_m$'s described in the previous subsection 5.1.

These correlations are shown in Table 2 in the fourth column, the first three columns giving the embankment name, typical laboratory $m_v$'s and typical $q_c$ values. Note that the $\alpha_m$'s are not simply the reciprocal of the product of the typical $m_v$ and $q_c$ values, but are derived from direct evaluation of $m_v$ and $q_c$ values at specific positions and depths.

The table shows that the mean laboratory derived $\alpha_m$'s is 2,97 with a standard deviation of 0,46. This value is similar to that given in subsection 5.1 for the field derived $\alpha_m$'s.

The measured $\alpha_m$'s from Table 1 are also shown in Table 2; these are compared with the laboratory derived $\alpha_m$'s as in the last column of Table 2. This shows that the ratio of the two $\alpha_m$'s is 0,98 with a standard deviation of 0,13.

This comparison of laboratory derived settlements and measured settlements shows that conventional laboratory coefficients of compressibility give a reliable means of estimating settlements in the normally consolidated alluvium.

It should be noted that the mean $\alpha_m$ in Table 1, viz. 2.63, is not the same as that from the same data given in Table 2, viz. 2.92. This is primarily because different data sets were used from the same base data. For example Bot River has ten values in Table 1 but only two in Table 2 and similarly Goukamma has three and one respectively. If this bias is removed then the constrained modulus coefficient, $\alpha_m$, is:

$$\alpha_m = 2.75 \pm 0.55$$

<table>
<thead>
<tr>
<th>Embankment</th>
<th>$m_v$</th>
<th>$q_c$</th>
<th>MPa</th>
<th>Lab/CPTU $\alpha_m$</th>
<th>Measured $\alpha_m$</th>
<th>Mean $\alpha_m$</th>
<th>Std Dev</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manzamuna</td>
<td>1.2</td>
<td>0.2</td>
<td>3.4</td>
<td>3.09</td>
<td>0.89</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Umlalazi 1</td>
<td>0.9</td>
<td>0.35</td>
<td>3.08</td>
<td>3.38</td>
<td>1.10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Umlalazi 2</td>
<td>1.2</td>
<td>0.50</td>
<td>2.94</td>
<td>3.33</td>
<td>1.08</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Umhlaliwe</td>
<td>1.5</td>
<td>0.25</td>
<td>2.67</td>
<td>3.09</td>
<td>1.16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Prospecton</td>
<td>0.4</td>
<td>0.65</td>
<td>3.42</td>
<td>2.99</td>
<td>0.87</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mzimkulu</td>
<td>0.4</td>
<td>0.80</td>
<td>3.79</td>
<td>3.76</td>
<td>0.99</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Goukama</td>
<td>0.6</td>
<td>0.85</td>
<td>2.70</td>
<td>3.01</td>
<td>1.11</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bot River</td>
<td>0.95</td>
<td>0.375</td>
<td>2.81</td>
<td>3.15</td>
<td>1.12</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bot River</td>
<td>0.7</td>
<td>0.42</td>
<td>3.40</td>
<td>2.79</td>
<td>0.82</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Umzimbazi</td>
<td>1.4</td>
<td>0.32</td>
<td>2.23</td>
<td>1.91</td>
<td>0.86</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Umhlangane</td>
<td>0.75</td>
<td>0.56</td>
<td>2.38</td>
<td>1.91</td>
<td>0.80</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td></td>
<td>2.97</td>
<td>2.92</td>
<td>0.98</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Std Dev</td>
<td></td>
<td></td>
<td>0.46</td>
<td>0.55</td>
<td>0.13</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
6 SETTLEMENT RATE

6.1 Field Data

The settlement records for many of the embankments were generally in insufficient detail to enable reliable long term settlement rates to be established and hence coefficients of consolidation to be derived.

These actual embankment performance $c_v$'s were then correlated with piezocene $t_50$'s in order to evaluate the cone time factor $f(T_V)$.

Asoka's graphical method was used and proved to be an extremely effective way of estimating primary consolidation $c_v$'s and the amount of secondary compression. Figures 3 and 4 show two typical examples. Linear regression lines were fitted to the data points and the slope of these lines is a measure of $c_v$. The drainage path lengths have then to be estimated to calculate $c_v$ and since these lengths are squared the resulting $c_v$ is very dependent on the estimation. In practice these are assessed from careful study of the piezocene records and from the materials description charts together with the piezocene dissipation tests. In general it is considered that if the dissipation times vary by an order of magnitude then this represents an effective change in strata.

The field CPTU $c_v$'s and the Asoka analysed $c_v$'s are given in Table 3. In the table the CPTU $c_v$'s are calculated using the earlier cone time factor of 50. The values of the $c_v$'s in the table are typical values for each site, at most sites more than one field (Asoka) $c_v$ can be derived since there are settlement records at more than one position along the embankment. Similarly there are a number of CPTU's and laboratory $c_v$'s available at each site.

Table 3. Coefficients of consolidation from laboratory, CPTU and settlement analyses.

<table>
<thead>
<tr>
<th>Embankment</th>
<th>$c_v$ (or $c_a$) m/yr</th>
<th>Laboratory</th>
<th>CPTU</th>
<th>Asoka</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manzamoyana</td>
<td>1.2</td>
<td>1.0</td>
<td>5.4</td>
<td></td>
</tr>
<tr>
<td>Umhlali 1</td>
<td>2.2/4.4</td>
<td>9.6</td>
<td>6.8</td>
<td></td>
</tr>
<tr>
<td>Umhlali 2</td>
<td>4.8</td>
<td>1.7</td>
<td>4.3</td>
<td></td>
</tr>
<tr>
<td>Umkhlatini</td>
<td>0.5 - 2.5</td>
<td>2.8</td>
<td>8.2</td>
<td></td>
</tr>
<tr>
<td>Prospecto</td>
<td>0.4</td>
<td>1.0</td>
<td>4.7</td>
<td></td>
</tr>
<tr>
<td>Mzimuku</td>
<td>1.8</td>
<td>7.4</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>Gousika</td>
<td>1.4</td>
<td>6.9</td>
<td>4.5</td>
<td></td>
</tr>
<tr>
<td>Bot River</td>
<td>1.5</td>
<td>5.8</td>
<td>11</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.3</td>
<td>1.6</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.8</td>
<td>4.6</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Ungababa</td>
<td>1.6/2.5</td>
<td>1.5</td>
<td>8.8</td>
<td></td>
</tr>
<tr>
<td>Umzimbazi</td>
<td>2.1/7</td>
<td>2.5</td>
<td>4.5</td>
<td></td>
</tr>
<tr>
<td>Umhlangane</td>
<td>0.7/0.5</td>
<td>1.8</td>
<td>3.6</td>
<td></td>
</tr>
</tbody>
</table>

The detailed comparison of CPTU and Field (Asoka) $c_v$ on a site by site or position by position at each site basis shows that:

Field Measured $c_v = 3$ CPTU $c_v$

Since the CPTU $c_v$ was estimated using a cone time factor of 50 this means that in order for the
CPTU $c_v$ to model field measured embankment settlement rates the cone time factor should be modified to:

$$c_v = \frac{150}{t_{50}} \quad (c_v \text{ m}^2/\text{yr} \times t_{50 \text{ minutes}})$$

As previously indicated the earlier correlation, using a factor of 50, was between CPTU $c_v$ and laboratory data including an element of judgment to reduce overconservatism.

### 6.2 Laboratory Data

The table in the preceding section also indicates typical laboratory data as well as the field data and hence these can be directly compared. Detailed comparisons show that: Field Measured $c_v = 6 \text{ Lab } c_v$.

In addition to the consolidation test data from the pre-construction investigations for the embankment sites much other laboratory test information was also available. It was therefore possible to derive various correlations within the data set e.g. compression indices versus moisture content or void ratio and the conventional plasticity index with liquid limit.

Compression index correlates well with both water content and void ratio as shown in Figure 5 and represented by:

$$C_c = 0.0125 \times (w_r - 7.5)$$

and

$$C_c = 0.45 \times (e_o - 0.25)$$

The second data set illustrates the nature of the alluvial materials found along the east coast as shown on Figure 6 in which the data is represented by:

$$I_p = 0.73 \times (w_l - 15)$$

### 6.3 Field Data versus Laboratory Data

In addition to the primary purpose of the project, which was to obtain $\alpha_m$ and cone time factor values for the recent alluvial deposits, correlations were also sought between $\alpha_m$ and laboratory measured parameters and other field information. For example correlations between $\alpha_m$ and cone pressures and between $\alpha_m$ and stress ratios, moisture contents and plasticity were carried out. The former two showed little or no correlation, but $\alpha_m$ showed a weak correlation with moisture content or plasticity which can be expressed as:

$$\alpha_m = 1.5 + 0.04 \times (w_r - 20)$$

Since there was a strong correlation between moisture content and liquid limit, i.e. the liquidity index is typically close to unity for the saturated recent alluvial deposits, then:

$$\alpha_m = 1.5 + 0.04 \times (w_l - 20)$$

Although the above correlations for $\alpha_m$ are not strong they represent trends which are useful in selecting appropriate $\alpha_m$ values from moisture
content or plasticity data for these recent alluvial deposits.

7 CONCLUSIONS
Back analysis of long-term settlement monitoring of a number of embankments over recent alluvial deposits along the South African east coast has allowed various correlations of piezocene test results to be made with both actual embankment performance and with laboratory based predictions.

7.1 Piezocene with Embankment Settlement and Settlement Rate
The data shows that embankment immediate and primary consolidation settlements -represented by coefficients of compressibility, \( m_e \) - can be reliably predicted from piezocene results using:

\[
m_e = \frac{1}{\sigma_m q}
\]

where \( \sigma_m = 2.75 \pm 0.55 \)

It has also been shown that for the embankments studied the combined average amount of yield and secondary compression is about 15% of the total settlement.

The rate of settlement can be predicted from the piezocene dissipation test data using:

\[
c_r = \frac{150}{t_m} (c_v \text{ m}^2/\text{yr} ; 150 \text{ minutes})
\]

7.2 Laboratory Data with Embankment Settlement and Settlement Rate
The comparison of consolidation test data with embankment performance shows that the use of conventional coefficients of compressibility, \( m_e \), gives reliable predictions of settlement. It is however noted that undisturbed sampling of many recent alluvial deposits is difficult; in practice this may lead to unrepresentative samples being available for testing, hence pessimistic settlements being predicted.

Settlement prediction rates based on laboratory measurements of coefficients of consolidation are shown to be extremely conservative viz. Field measured \( c_v \) = 6 laboratory \( c_v \).

Note that the field \( c_v \) is at a macro scale which combines changes in materials and their layering within a single parameter, whereas the laboratory values are at a micro scale. Nevertheless the difference is large and may lead to grossly over-conservative predictions of settlement rates if reliance is placed solely on laboratory testing.

Thus, it is concluded that for reliable prediction of settlement rates on recent alluvial deposits, piezocene testing is essential.

8 ACKNOWLEDGMENTS
Considerable research was required to collect the original geotechnical investigation information, most of which was more than twenty years old, and the settlement data since monitoring has now ceased. The further piezocene work at selected embankments also represented a major effort. All this was made possible through research grants from the South African Roads Board and for the support the authors are indebted to the board.

9 REFERENCES


Three Dimensional Analysis of Point Resistance of Piles in Clay

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SYNOPSIS: The slip - line fields around the pile tip in clay were analyzed under the condition of axisymmetry, based on the observations made from the model tests. The slip - line fields in the intermediate transient region were computed using the boundary conditions similar to those in a retaining wall with a broken back. The analysis was made under the assumption that soil is a perfectly plastic rigid material, following Mohr-Coulomb’s failure criteria and that the stresses satisfy Haar-von-Karman’s condition.

The results of the analysis were expressed in terms of the bearing capacity factor $N_c$ as a function of depth, tip angle and roughness of the pile. Field records and some experimental results were also analysed and reasonably good agreement was found between the theoretical and observed $N_c$ values.

1. INTRODUCTION

The bearing capacity of pile foundations has been presented earlier considering the shear strength of soil above the level of the foundation and treating the case as a two dimensional problem (Meyerhof 1951). According to that theory as the depth of the foundation was increased, the plastic region around the pile tip got extended above the level of the foundation, finally reaching the shaft. Such a phenomenon was observed when space was available between the pile shaft and adjacent soil mass as in the case of a simple cone penetrating into the ground (Koumoto and Kaku 1982). However, if no such space exists for the soil to move, as in the case of a pile, the failure surface cannot extend to the shaft specially in the case of ideally plastic rigid material like saturated clay and dense sand. Consequently, the slip lines rise vertically to a height above the foundation (Koumoto et al. 1985).

In the present investigation, in order to observe the shape of the rupture surface, two dimensional model tests were conducted with and without space between the shaft and the soil mass. Based on the results of these tests, the slip line fields have been analyzed and compared with some published test data and field records.

2. MODEL TESTS

2.1 Soil data

Bentonite clay having a liquid limit of $W_l = 402$ % and plastic limit of $W_p = 35.6$ % was used at a moisture content of $W = 100$ %. The clay was made by mixing the required quantity of water with dry powder in a mixer. The mixed clay was left for two weeks for moisture equalization. After this period it was packed into an acrylic box of 200 x 300 mm and 15 mm deep.

2.2 Pile data

The piles were made of acrylic and were 50 mm wide, 15 mm thick and 200 mm long. Two types of piles were used, one with a constant cross section throughout the depth called pile 1. Pile 2 was 50 mm wide at the toe and tapered symmetrically to 40 mm width at 60° angle as shown in Fig.1(a).

2.3 Test details

The front cover ( 200 x 300 mm ) of the test box...
was removed and the clay was packed into the box. The case of pile at the bottom of a slope was created by packing the soil at the required inclination of 45° and by placing pile 2 in the required position (Fig.1(a)). The case of pile having space between the shaft and the adjacent soil mass (Fig.1(b)) was obtained by placing pile 1 in place, packing clay, and replacing pile 1 with pile 2. The case of pile without space between the shaft and soil mass was created by placing pile 1 at different depths so that $D_p/2R = 0.5 - 3.0$ at intervals of 0.5 where $D_p$ is the depth of embedment and $2R$ is the width at the toe of the pile. After completing the packing of clay the front surface was marked with black lines at 10 mm spacing using a felt pen. The acrylic cover was placed in position over the soil after applying silicone oil to reduce friction. The box was placed over the pedestal of a strain controlled testing machine. The pile was loaded under undrained condition as the pedestal was moving up at a constant rate of 5mm/min. After soil failure was clearly seen the failure surfaces were photographed.

2.4 Test results

In Figs. 1(a) and (b) the rupture surfaces reverted to the shaft of the foundation as suggested by earlier investigators (Meyerhof 1951). However, in Fig.1(c) the rupture surface clearly rose to the ground level even from a depth of 3x2R and did not follow the pattern observed in Figs. 1 (a) and (b). This difference...
in the first two instances (Figs 1 (a) and (b)) there was space between the shaft of the foundation and the adjacent soil mass whereas in the third case (Fig.1(c)) no such space was available for the movement of the soil towards the shaft. This observed mode of failure (Fig.1(c)) was considered in the theoretical analysis as given below.

3. THEORETICAL ANALYSIS

3.1 Basic equations

In terms of cylindrical polar co-ordinates \((r, \phi, z)\), the four stress components \(\sigma_r, \sigma_\phi, \sigma_z, \tau_{rz}\) are radially symmetric with respect to the \(z\) axis, the other stress components being zero. The stress \(\sigma_\phi\) is a principal stress and is considered equal to the intermediate principal stress \(\sigma_2\) in the Cartesian co-ordinate system while \(\sigma_1\) and \(\sigma_3\) are the major and the minor principal stresses, respectively. The stress components satisfy the following equations of equilibrium given by

\[
\begin{align*}
\partial \sigma_r / \partial r + \partial \tau_{rz} / \partial z + (\sigma_r - \sigma_\phi) / r &= 0 \\
\partial \tau_{rz} / \partial r + \partial \sigma_\phi / \partial z + \tau_{r\phi} / r &= \gamma
\end{align*}
\]

where \(\gamma\) denotes the unit weight of the material.

For saturated clay under undrained condition, \(\phi = 0\) and the Mohr-Coulomb failure criterion is expressed by the following equation,

\[
(\sigma_r - \sigma_\phi)^2 + 4 \tau_{rz}^2 = 4 C_u^2
\]

where \(C_u = (\sigma_1' - \sigma_3' / 2

3.2 Boundary conditions

In order to integrate the above equation, the following boundary conditions were considered, based on the observations from the model tests. Two typical tip angles shown in Figs.2(a) and (b) were considered for the boundary conditions,

\[0 < a \leq 45^\circ \quad \text{and} \quad 45^\circ < a \leq 90^\circ\]

respectively where \(a\) represents the semi angle of the pile tip. Two cases of roughness of pile were considered, one in which the pile was perfectly smooth and the second in which it was semi rough. Together with these boundary conditions the plastic regions suggested in the case of a retaining wall with a broken back.
Fig. 2 Plastic zones considered in the case of a pile tip having a semiangle \( \alpha \) of
(a) \( 0 < \alpha \leq 45^\circ \)  (b) \( 45^\circ < \alpha \leq 90^\circ \)

(Sokolovski 1960) were also considered so that,

a) On the ground surface AB (Figs. 2(a) and (b))
\[ \theta = - \pi / 4; Z = 0; \sigma_m / C_u = 1 \]
b) At point A (Figs. 2(a) and (b))
\[ \theta = - \pi / 4 + \delta i / N \]
where N is the total number of divisions.
c) On vertical side AA' (Figs. 2(a) and (b))
\[ \theta = - \pi / 4 + \delta / \text{Number of divisions} \]
d) At Point A'
\[ \theta_1 = - \pi / 4 + (3 \pi / 4 - a + \delta) / N \]

In the case of \( 45^\circ < \alpha \leq 90^\circ \)
\[ \theta_1 = - \pi / 4 + (3 \pi / 4 - \xi - \delta) / N \]
\[ \sigma_m / C_u = 2 \theta_1 / 2 \text{(Figs. 2(a) and (b))} \]
e) On plane AA' (Fig. 2(a))
\[ Z = (1 - r) \cot \alpha + D_b; \theta = - \pi / 4 + \delta + \alpha \]
f) On plane OA' (Fig. 2(a) and (b))
\[ O'A' = 1.0; O'O = D_b \]
g) At point G (Figs. 2(a) and (b))
\[ r = 0 \]
h) On plane GF (Fig. 2(b))
\[ r = 0; \theta = \pi / 4 \]
where \( \delta \) depends on the degree of roughness of shaft so that \( \delta = 0 \) for perfectly smooth and \( \delta = \pi / 4 \) for perfectly rough shafts, \( \zeta \) value depends on the degree of roughness of the base and is discussed in section 4.2 of this paper, \( D_b \) = depth of embedment.

4 ANALYTICAL RESULTS

4.1 Slip line fields
By integrating equations 4 and 5 under the above boundary conditions, the slip line fields around the pile tip were computed. The computed slip line nets for typical values of \( \alpha = 30^\circ \) and \( \alpha = 90^\circ \) are presented in Figs. 3(a) and (b), respectively for both perfectly smooth and perfectly rough piles for \( D_b / 2R = 2 \). In these figures the left side of the pile axis shows the case of perfectly smooth pile while the right side of the axis depicts the case of perfectly rough pile.

4.2 \( \zeta \) Value
\( \zeta \) is the angle of the \( S_2 \) slip line with the pile base in the case of \( 45^\circ < \alpha \leq 90^\circ \). In the present case of \( \phi = 0 \), for a perfectly smooth base \( \zeta = \pi / 4 \). However, in the case of perfectly rough base, \( \zeta \) value is less than \( \pi / 4 \) and should be determined to satisfy the boundary condition on GF, namely the \( S_2 \) slip line has to intersect GF at an angle of \( \pi / 4 \).
The variation of \( \zeta \) value with depth for perfectly smooth and perfectly rough flat bases (\( \alpha = 90^\circ \)) is presented in Fig. 4.

4.3 Bearing capacity factor \( N_c \)

From the normal stress \( P_b \) and the shear stress \( S_b \) on the contact surface \( A/G \) of the cone tip with the soil (Fig. 2(a)), the unit tip resistance \( q_p \) was computed as

\[
q_p / C_v = N_c = (P_b / C_v) + (S_b / C_v) \cot \alpha \tag{6}
\]

\( N_c \) values computed for different values of \( \alpha \) for the case of perfectly rough base and shaft are presented in Table 1 together with those suggested by Skempton (1951), for \( \alpha = 90^\circ \).

As can be seen from this table the \( N_c \) values are increasing with depth for any given value of \( \alpha \). Further, the \( N_c \) value suggested by Skempton was constant below the depth of \( D_b / 2R = 4 \), while the present computations resulted in a continuous increase in the \( N_c \) value below.
the depth of $D_b / 2R = 4$. However, within this depth the present values are very close to those suggested by Skempton.

4.4 Other data

The $N_c$ values from in-situ tests (Skempton 1951, Bjerrum and Eide 1956) and from model tests (Meyerhof 1951) in the case of piles with flat base ($\alpha = 90^\circ$) were presented in Fig. 5 along with those from the present theory for a perfectly rough pile. Reasonably good agreement was noticed between the proposed and the observed values.

CONCLUSIONS

Model tests with piles having flat base revealed that the failure surfaces rise to the ground level when no space exists between the shaft and the adjacent soil mass. Based on these observations the slip line fields were analyzed under the condition of axisymmetry. Consequently, theoretical values of the bearing capacity factor $N_c$ were computed for different values of semi angle $\alpha$ in the case of perfectly smooth and perfectly rough piles. The estimated values of $N_c$ for the case of perfectly rough pile with a flat base ($\alpha = 90^\circ$) were compared with those observed from in-situ and model tests. Good agreement was noticed between the theoretical and the observed values. The present theory is useful in evaluating the in-situ undrained shear strength of clay deposits based on the tip resistance obtained from penetration tests.

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**SYNOPSIS.** After having been considered as a tool for soil characterization during two decades, the piezocone is more and more directly used for design. The paper presents several applications, and also limitations, of the piezocone in sensitive eastern Canada clays such as the determination of a coefficient of consolidation, the evaluation of the preconsolidation pressure, the evaluation of strength increase under embankments, the definition of plastic zones in natural slopes and the evaluation of the friction mobilized along piles.

1. **INTRODUCTION**

   Up to now, the piezocone has been essentially used for soil characterization, and relatively seldom as a design tool. It is particularly true in clays, and this can be explained by the fact that, up to recently, the apparatuses available were influenced by temperature changes, expensive to use and not enough accurate. Now, better tools exist at a lower cost, and it seems time for developing direct, practical and reliable applications. The present paper summarizes practical uses of the piezocone in the sensitive eastern Canada clays resulting from researches performed at Laval University in the last decade or so.

2. **BRIEF DESCRIPTION OF EASTERN CANADA CLAYS**

   The main geotechnical characteristics of eastern Canada clays have been described by Leroueil et al. (1983). These clays of marine or lacustrine origin, have been deposited between 18 000 and 6 000 years B.P., while the Wisconsin glacier was retreating North. The grain size distribution varies from site to site, but mineralogical analyses show that about 50% of the clay fraction (<2μm) is made up of rock flour. The plasticity index IP is smaller than 55%. The liquidity index IL is usually larger than 1.0, but can be as low as 0.7, and as high as 4 in silty deposits. The overconsolidation ratio OCR is usually between 1.2 and 2.5, but can be much larger in deposits which have been eroded in the past. The sensitivity St is generally larger than 15, and can reach values as high as 1000 in some cases. Such high sensitivities can be partly due to leaching, but could also be an inherent characteristic of these soils since sensitivities in the order of 10 were observed in sediments recently formed in salty water at the bottom of a fjord (Locat and Leroueil, 1988). Yielding is well pronounced, giving a brittle behaviour in shear tests and a high compression index in oedometer tests. The friction angle in the non-fully consolidated range is in the order of 30°.

3. **SOIL CHARACTERIZATION**

   Soil characterization remains the most common use of the piezocone. Several charts were proposed for that purpose. The most complete, which takes into account tip resistance qτ, pore pressure u and friction u and friction fτ is that proposed by Robertson (1990).

   Clays are first characterized by simultaneous variations of tip resistance and excess pore pressures. In eastern Canada clays, the parameter

   \[
   B_q = \frac{(u - u_0)}{\left(\frac{q\tau}{\sigma_{vo}}\right)}
   \]

   varies

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between 0.45 and 0.82. As suggested by Krook (1984), $B_q$ should reflect the stress history of clays, but no clear relationship was found (Demers and Leroueil, 1995).

As evidenced by Konrad (1987) and Virely (1992), the friction just behind the tip can be very small, and sometimes equal to zero. Five diameters behind the tip, as measured with the Laval piezocone (Virely et al., 1995b), the friction in sensitive clays remains small, generally smaller than 10 kPa.

More interesting is the fact that the tip resistance, and to a lower extent the excess pore pressure, are related to strength parameters. In particular, $q_T$ can be related to the vane shear strength $v_{v0}$ as follows:

$$q_T = N_{vT} \tau_{v0} + \sigma_{v0}$$  \hspace{1cm} (1)

in which $N_{vT}$ is the cone factor. Its values are generally constant on a given site, but vary from site to site from 11 to 18. Hence, for the time being, $N_{vT}$ has to be defined on every new site.

Results from 21 sites (Demers and Leroueil, 1995) show that $q_T$ can also be related to the preconsolidation pressure $\sigma_{p0}$ (Fig. 1).

$$q_T = N_{vT} \sigma_{p0} + \sigma_{v0}$$  \hspace{1cm} (2)

with $N_{vT}$ equal to 3.6 on average. Eq. 2 should not be used instead of oedometer tests for determining the $\sigma_{p0}$ profile of a clay deposit. However, as for most projects only few oedometer tests are performed, the $N_{vT}$ value specific to the clay under study can be defined, compared to the value deduced from Fig. 1, and used together with the $q_T$ profile to obtain a continuous $\sigma_{p0}$ profile.

4. COEFFICIENT OF CONSOLIDATION

It is theoretically possible to deduce a coefficient of consolidation ($c_l$) from the observation of the pore pressure dissipation around a piezocone. Using the theory proposed by Teh and Houslby (1991), Virely et al. (1995b) showed that the calculated coefficient of consolidation is about the same whatever the position of the porous element and the degree of consolidation reached.

Virely et al. (1995b) evaluated $c_l$ on eight sites where this parameter had been determined by other methods. On 5 of these sites, an in situ value was deduced from settlement observations of embankments for the normally consolidated range; on three of them, an in situ value in the overconsolidated range was deduced from pore pressures generated during the early stages of construction.

Figure 2 summarizes the results. The in situ coefficients of consolidation in the normally consolidated range are typically 10 times larger than the values deduced from oedometer test results, using the Casagrande method, and about two orders of magnitude smaller than the values deduced from in situ observations in the overconsolidated range. $c_l$ values are typically in the order of $10^{-6}$ m²/s for the clays studied, i.e. close to the values deduced from oedometer tests in the overconsolidated range. However, due to viscous phenomena, these last values are generally badly defined in soft clays, and their significance is not clear. When compared to in situ values, which should be the most representative, $c_l$ appears to be somewhere in between the values corresponding to the overconsolidated and the normally consolidated ranges. This result seems reasonable as, pore pressure dissipation involves a volume of soil which is partly intact and overconsolidated, and partly remoulded and
normally consolidated due to the penetration of the probe. The representativity of $\gamma_{ch}$ piezo is thus questionable.

5. STRENGTH INCREASE UNDER EMBANKMENTS

Equation 1 expresses a relationship between the net tip resistance ($q_T - \sigma_{vo}$) and the undrained shear strength of intact clays. Assuming that a similar relationship exists in normally consolidated clays under embankments, the piezocone could be used for evaluating the strength increase under embankments.

Leroueil et al. (1995a) performed vane tests and piezocone profiles on 4 sites where embankments were built, both in the intact clay and under one or several embankments. At St-Hyacinthe (Fig. 3), the strength increase is clearly evidenced in the entire deposit by both tests. At St-Alban, the preconsolidation pressure has been exceeded only in the upper part of the deposit, at elevations larger than 95 m (Fig. 4a); this is confirmed by Fig. 4b showing that the 2 piezocone profiles coincide below this elevation.

$N_{kT}$ values were determined for both the intact clay and the clay under the embankments, and the ratios are shown in Fig. 5. $N_{kT}$ (intact) / $N_{kT}$ (under emb.) is smaller than one, and close to the correction factor $\mu$ proposed by Bjerrum (1972) for analyzing the stability of the first stage construction of an embankment with the vane shear strength. As Canadian experience (Tavenas et al., 1978; Law, 1985) indicates that the vane shear strength measured under embankments has not to be reduced by the correction factor $\mu$ for evaluating the available strength, it seems that the change in $N_{kT}$ is due to the vane shear strength only. As a consequence, the strength increase under an embankment can be directly estimated from the change in ($q_T - \sigma_{vo}$).

6. PIEZOCONE IN NATURAL SLOPES

It is generally thought that landslides in sensitive clays happen very suddenly. However, piezocone profiling in several unfailed natural slopes, but probably of precarious stability, showed the presence of plastified zones, with a net tip resistance and an excess pore pressure locally smaller than those in the intact material.

At Maskinongé where there was a landslide in 1990 in a sensitive clay deposit ($I_p = 42\%$; OCR = 1.35), an investigation was performed both inside the landslide crater and outside, in slopes presenting the same geometry (Deres et al., 1993). Figure 6 shows the piezocone profiles obtained in a cross-section 50 m south of the landslide. The profiles performed close to the river present a tip resistance which is smaller...
Fig. 3 - Vane strength and net tip resistance increases under an embankment at St-Hyacinthe.

Fig. 4 - Vertical effective stress (a) and net tip resistance (b) under an embankment at St-Alban.
than that observed in CPTU18, in the intact clay, far from the slope. These observations, and others made in another cross-section, show net tip resistances lowered by 10 to 50%, indicating that the clay mass is softened near the slope. From observations of the 1990 landslide and other landslides in the area, it appears that the failure surface develops within the softened mass (Fig. 6).

Piezocone profiles in two other natural slopes of precarious stability show a different behaviour. In these cases, the softening of the clay is localized in a band having a thickness of about 70 cm. This has to be studied in greater details; however, it is evident that the practical consequences are important: the piezocone could apparently be used in hazard mapping of landslides, and, if failure surfaces develop in softened zones, existing approaches for stability analysis could have to be re-examined.
7. PIEZOCONE, A MODEL PILE
The piezocone developed at Laval University (Virely et al., 1995a), with its friction sleeve 5 diameters behind the tip, has been used for evaluating the friction mobilized along the shaft of piles. The piezocone is first pushed to a given depth and left there until full dissipation of the excess pore pressure. It is then slowly pushed further down at a small rate of about 3 cm/min, while the various test parameters, in particular the friction, are measured.

A typical result, obtained on the site of Louisville (f_p = 42%; OCR = 2.5-3.0), is shown in Fig. 7. The friction-displacement curve first shows a peak of 23.6 kPa; the friction then decreases to a plateau at 10 kPa, corresponding to the sliding of the friction sleeve in the reconsolidated soil (zone 1); when the friction sleeve starts penetrating the freshly disturbed soil (zone 2), the measured average friction progressively decreases to reach the value measured during standard penetration into the intact soil, i.e. few kPa (zone 3).

Tests were performed at different depths, on 6 sites where friction piles were tested up to failure by compressive or tensile loading. The clays under study had plasticity indices from 10 to 50, and OCRs between 1.15 and 5. The details of the study are presented by Leroueil et al. (1995b). Fig. 8 shows the peak frictions f_s piezocone obtained on the site of Louisville. They increase regularly with depth from 20.5 kPa at 3 m to 36 kPa at 9 m. The average shaft frictions mobilized at failure along piles f_s pile are also shown in Fig. 8. The agreement between f_s piezocone and f_s pile is very good.

The average frictions mobilized on the piezocone and on the friction piles are compared on Fig. 9 for the 6 sites and the 11 tested piles. There is an excellent correlation, with f_s piezocone being on average only 5% smaller than f_s pile. A nearly perfect agreement, indicating that the piezocone could be used for a direct evaluation of shaft friction, in clays of course, but possibly in other materials such as silts and loess for which no reliable design method exists.

Fig. 7 - Friction mobilized after pore pressure dissipation at Louisville (4.25 m).

Fig. 8 - Frictions mobilized on the piezocone after pore pressure dissipation, and along the shaft of piles at failure.
Leroueil et al. (1995b) also showed that $f_{s_{\text{piezocone}}}$ is not simply proportional to the net tip resistance $(q_T - \sigma_{vo})$. As indicated in Fig. 10, the ratio $f_{s_{\text{piezocone}}}/(q_T - \sigma_{vo})$ decreases from 0.1 to 0.055 when the plasticity index increases from 10 to 55. The evidence of the influence of the plasticity index on this ratio could allow an improvement of existing indirect methods using the piezocone for the design of friction piles.

![Fig. 9 - Comparison between $f_{s_{\text{piezocone}}}$ and $f_{s_{\text{pile}}}$.](image)

![Fig. 10 - Variation of $f_{s_{\text{piezocone}}}/(q_T - \sigma_{vo})$ with the plasticity index.](image)

8. CONCLUSION

Researches on practical applications of the piezocone in soft clays have been performed at Laval University in the last decade or so. The main results show that:

- $a(q_T - \sigma_{vo}) - \sigma_T$ relationship exists and can be used for specifying a preconsolidation pressure profile;
- a coefficient of consolidation can be deduced from pore pressure dissipation. It seems to be between the in situ values and the normally consolidated values;
- the strength increase under embankments can be estimated with the piezocone;
- plastified zones in unfaulted slopes can be detected with the piezocone;
- the piezocone can be used indirectly for determining friction along piles.

9. ACKNOWLEDGEMENTS

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10. REFERENCES


Prediction of settlements based on CPT, for a five storey building founded on organic silt and sand

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Yao Yu
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The new requirements in the Eurocode imply that the current computational models must be improved and measurements of deformations must be performed. For foundation problems, methods better describing the interaction between the building and the subsoil are asked for.

The purpose with this project has been to follow up the settlements during the erection, and one year thereafter, of the five storey apartment houses built by the construction company Skanska in the South Harbour (Södra hamnen) in the city of Luleå, Sweden.

The difficulty of taking undisturbed samples of course-grained soils, in order to, for example, determine the oedometer moduli, forced us to perform in situ-tests. Computations based on CPT-tests were then in good agreement with measured results. Such a judgement should be based on at least three CPT-tests according to the method proposed by De Beer (1965). In the future, computations based on the Finite Element Method (FEM) will be more and more common making a variety of assumptions and conditions possible, facilitating the judgement of how different chosen moduli will influence on the result.

A comparison was made between results obtained from penetration tests and from sampling. An interpretation of the soil profile was made from CPT-tests according to methods proposed by Sangerlath (1972) and Larsson (1993). The comparison showed that the CPT-tests coarsely can localize the type and position of different soil layers.

1. INTRODUCTION

There is a great demand for rational methods to determine the stratification and the engineering properties in the various layers in a soil profile. The piezocone is an unsurpassed method for continuous and detailed determination of soil stratification and a number of soil properties can be estimated from the test results. In soft soils, this sets very high demands on the accuracy of the measurements. Because of the geological history of the soft soil deposits, there is normally a great variation of the soil composition with depth.

The organic sulphide silt (clayey silt or silty clay), locally known as "svartmocska" and primarily located along the coast of the Bothnian sea, contains certain organic matters as well as iron sulphide, which give rise to a complex open soil structure and make it more like a soft clay, though the sulphide soil normally has a high content of silt particles. Due to its particle size distribution, silt has many special properties which have caused
geotechnical hazards such as flowing of silt slopes and surface erosion. Difficulties concerning stability are often met, for instance, in designing excavations in silt. In many cases, results from the cone penetration test (CPT) show a peculiar dilatant behaviour in a ground where silt soils appear in a normally consolidated state, indicating that silt can possess both contractant and dilatant properties.

As a special organic soil, the sulphide soil also has some particular characteristics in comparison with glacial and postglacial soft soils in southern Sweden. It has a considerably high compressibility as well as a viscous structure, and its undrained strength characteristics include anisotropy. These properties should be accounted for, for example, in calculations of settlement and stability of embankments on the sulphide soil. In recent years, difficult geotechnical problems in connection with foundation of buildings and civil engineering works in northern Sweden have been faced involving these two soils. Therefore, a better understanding of the genuine geotechnical properties of these two soils is asked for. In fact, while organic sulphide soil has been studied to some extent, silt is one of the least investigated soils up to now. The reason why knowledge about silt is so poor, is partly due to an incorrect judgement made by geotechnical engineers. They claim that the properties of silt could be interpolated between those of clay or sand, and also partly due to the extreme difficulty in sampling and preparation of samples representing the in situ structure of silt. Furthermore, the previous investigations on sulphide soils are limited and incomplete, resulting in an unclear pattern of the intrinsic behaviour of the soil.

Due to the inevitable sample disturbance in this type of soils, soil compressibility based on laboratory tests yields settlements which may be several times the measured values. Consequently, greater reliance should be placed on estimating compressibility from in situ tests or other indirect methods. The success of this approach depends on the database supporting the correlations between soil compressibility and in situ tests. This points out the need to perform settlement measurements on full-scale structures and to compare predicted and measured settlements for improving the correlations between in situ tests and soil compressibility.

The foundation of each of the apartment houses in the South Harbour (Sodra Hamnen), City of Luleå, was designed as a continuous reinforced concrete raft without piles on a subgrade, consisting of from the top, sand, 1-3 m sulphide silt and sand again. This is a soil profile which is quite common along the coastline and in the river valleys of northern Sweden.

For design of the foundation, the stresses in the subgrade was limited to 200 kPa in order not to obtain too large settlements. This condition gave a width of the footing of 1,8 m.

Settlements were calculated during design using cone penetration tests and published correlations between cone resistance and elastic modulus. By comparing measured settlements during the erection of the houses and after with the calculated values, it is shown that a reasonable estimate of settlement was made using only cone data and available correlations.

2 SUBSURFACE CONDITIONS

Soil conditions were investigated by performing two cone penetration tests to depths of 6,1 to 7,0 m below the existing ground surface. Approximately 20 previously performed soil borings, dynamic probing, soil sampling, to depths of 15,2 m were also reviewed.

The cone resistance measurements were carried out using a standard 60° cone, with a cross-sectional area of 10 cm².

Typical cone penetration test and soil boring logs are shown in Fig 1 and 2.
3 SOIL COMPRESSIBILITY

In everyday geotechnical practice settlements, which occur beneath newly built constructions, are calculated using the moduli of compressibility which correspond to the interval of effective stresses, from the initial geological stresses $\sigma_{\text{ini}}$ to the final stresses, as caused by the increase $\Delta \sigma^e$ in the ground. When using the results of laboratory tests the moduli of compressibility at any given stress interval are known. In the case of CPT measurements it is necessary to make an estimate of the moduli of compressibility on the basis of experience.

4 SETTLEMENT CALCULATION PROCEDURE

The settlement calculations were made with four different methods.

The laboratory tests of compressibility were carried out in oedometers on intact specimens taken from one or more than one borehole which had been drilled in the vicinity of the CPT.

A comparison of preconsolidation pressures determined by CRS tests and by conventional oedometer tests is given in Fig 3.

![Graph showing settlement calculations](image)

Fig 3 Results from oedometer tests on sulphide soils from the South Harbour of Luleå.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>W_0 (%)</th>
<th>W_1 (%)</th>
<th>W_r (%)</th>
<th>T_d (%)</th>
<th>T_r (%)</th>
<th>$\sigma_{\text{ini}}$ kPa</th>
<th>$\sigma_{\text{fin}}$ kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.8</td>
<td>104</td>
<td>118</td>
<td>58</td>
<td>36.3</td>
<td>1.63</td>
<td>4.0</td>
<td>76/13</td>
</tr>
<tr>
<td>5.3</td>
<td>74</td>
<td>79</td>
<td>32</td>
<td>46.4</td>
<td>1.67</td>
<td>3.5</td>
<td>75/15</td>
</tr>
</tbody>
</table>

Table 1 Physical properties of the samples.
The resulting parameters from the oedometer tests were:

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$\sigma'_{0}$ (kPa)</th>
<th>$\sigma'_{1}$ (kPa)</th>
<th>$M_{d}$ (kPa)</th>
<th>$S_{L}$ (kPa)</th>
<th>M'</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.8</td>
<td>110</td>
<td>150</td>
<td>2400</td>
<td>1000</td>
<td>16</td>
</tr>
<tr>
<td>5.3</td>
<td>85</td>
<td>137</td>
<td>1600</td>
<td>1050</td>
<td>9</td>
</tr>
</tbody>
</table>

Table 2 Oedometer results.

For piezocene penetration tests, the correlations between cone penetration resistance and the compressibility of the sand in the Buisman-De Beer method (De Beer, 1965) can be used to estimate the settlement under a specified pressure. With this method the $q_{c}$ versus depth $z$ plot below footing is divided into a number of layers of varying thickness. For each layer the value of cone penetration resistance $q_{c}$ is assumed constant.

The division of $q_{c}$ versus depth $z$ plot is terminated at the depth at which the stress increment ($\Delta \sigma$) becomes less than 10% of the effective overburden pressure ($\sigma_{0}'$) at the center of the layer. Using a constant of compressibility from a value of cone penetration resistance, the settlement of each layer is calculated for each footing.

Settlement of a footing on sand consists of $s_{k}$ and post-construction $s_{k}$. The method for computing $s_{k}$ using consolidation settlement CPT $q_{c}$ values, is patterned after the work of Schmertmann (1970). The settlement equation for an embedded footing on sand is:

$$s_{k} = \left( \frac{M_{d}}{\sigma_{0}' z} \right) \sum \left[ \frac{t_{j}}{E_{j}} \right]$$

(1)

Where $E_{j}$ is the drained Young's modulus of the silt. The value of vertical strain influence factor $I_{V}$ for embedded footings is obtained from $I_{V}$ in Fig 4 for footings that are placed at the ground surface, using the correction factor in the inset. Values of $[E_{j}]$ and $[I_{V}]$ are evaluated at middepth of sublayer $j$. The value of $z_{j} = \sum [z_{k}]$ is computed from:

$$\frac{z_{j}}{z} = \left( 1 + \tan \frac{L}{B} \right)$$

(2)

Equation 2 is valid for length, $L$, to breadth, $B$, ratios $(L/B)$ between 1 and 10.

The value of $E_{j}$ is obtained from CPT tip resistance, $q_{c}$. The empirical relationship between modulus $E_{j}$ and cone resistance $q_{c}$, based on settlement of foundations and plate load tests, for circular or square loading conditions is:

$$E_{j} = (1.5 - 0.20)q_{c}$$

(3)

where $q_{c}$ is the weighted mean of the measured CPT $q_{c}$ values of sublayers within the $z_{j}$ defined by $E_{j}$:

$$q_{c} = \sum \left[ \frac{\rho_{j}}{g} \right] [z_{j}]$$

(4)

Fig 4 Strain influence factors.

In the method proposed by Schmertmann (1970) and Schmertmann et al (1978), the settlement is computed by integrating the vertical strain within the depth of influence of the footing. The vertical strain distribution used in this method is based on elastic theory, results of finite element analyses and model studies of shallow foundations. The soil stiffness used in the computation of vertical strain is obtained from a correlation with the cone tip resistance obtained from CPT tests, and correction factors are used to account for the effects of depth of embedment and time after load application.

The twodimensional finite element net (plane deformation) is used to symbolize the system building/subground. The vertical boundaries are constrained, it is only vertical deformations that can occur. The bottom boundary of dense layered sand is constrained in both vertical and horizontal directions.

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The E-moduli for the fill (sand) and the silt are based empirically on the q, from CPT where E<sub>s</sub> is chosen to (1,5-2,0)q, with an estimated value of the Poisson value υ=0.2. It can be mentioned that an E<sub>s</sub>-value of this dimension corresponds to E<sub>so</sub>, which is guilty for most foundation problems (Meigh 1987). The concrete footing is supposed to have a modulus equal to 2,0 GPa.

The E-modulus for the silt can be estimated from the CRS-tests

\[ E_s = \frac{(1+\nu)(1-2\nu)}{1-\nu} \cdot M \]

where \( \nu \) is the Poisson value with a value set to 0.2 and M is an average value for the secant modulus at constrained deformation.

The FEM-calculation is made in such a way that the modulus for the silt can have different values for different stress conditions, the modulus for the sand is kept constant.

You can find that the part of the settlement in the silt clearly dominate the total settlement and that the settlement increases for higher values of the Poisson value.

The parameters, used in the FEM-analysis is shown in Table 3.

<table>
<thead>
<tr>
<th>Layer 1 (fill) MPa</th>
<th>Layer 2 (fill) MPa</th>
<th>Layer 3 (silt) MPa</th>
<th>Layer 4 (sand) MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>E=1,5 q&lt;sub&gt;s&lt;/sub&gt;</td>
<td>E=5,0</td>
<td>E=9,0</td>
<td>E=1000</td>
</tr>
<tr>
<td>υ=0,2</td>
<td>υ=0,2</td>
<td>υ=0,2</td>
<td>υ=0,2</td>
</tr>
<tr>
<td></td>
<td>E=10,5</td>
<td>E=1025</td>
<td></td>
</tr>
</tbody>
</table>

Table 3 Parameters for the FEM-analysis.

Fig 5 Measured settlements

6 COMPARISON BETWEEN COMPUTED AND MEASURED VALUES

<table>
<thead>
<tr>
<th>Method of Calculation</th>
<th>Calculated settlement (mm)</th>
<th>Remarks</th>
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</thead>
<tbody>
<tr>
<td>Oedometer test</td>
<td>34</td>
<td>Structure load</td>
</tr>
<tr>
<td>CPT, Schmertmann</td>
<td>44</td>
<td>1 year</td>
</tr>
<tr>
<td>CPT, De Beer</td>
<td>51</td>
<td>10 year</td>
</tr>
<tr>
<td>FEM</td>
<td>23</td>
<td>10 year</td>
</tr>
<tr>
<td>Average</td>
<td>27 - 35</td>
<td>1 year</td>
</tr>
</tbody>
</table>

Table 4 Calculated and measured settlements at house F, South Harbour, Luleå.

7 INTERPRETATION OF SOIL LAYERS FROM CPT TESTS

Sometimes an evaluation of the stratification in a soil profile is made from the results of CPT-tests. Some researchers have interpreted the stratification from measured values from point resistance q<sub>c</sub>, sleeve friction f, and pore pressure, Sanglerat (1972) and Larsson (1993).

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Sanglerat uses even the friction ratio \( R_f = \frac{L_s}{q_e} \) and Larsson take into account the ratios \( \frac{q_e - \sigma_{uu}}{\sigma_{ve}} \) and \( \frac{f_s}{q_e - \sigma_{uu}} \).

The comparison between results obtained from the CPT tests and from sampling according to the two methods, showed that CPT-tests coarsely can localize the type and position of different soil layers.

8 CONCLUSIONS

Two of the above mentioned methods, generally used to estimate soil modulus for very stiff soils, include correlations with cone penetration resistance.

Although the approach to correlations between the CPT, which is in fact an undrained strength test, and the modulus of compressibility obtained by the mean of a drained deformation test, is, physically, not very evident There is nevertheless a tendency to make as full use as possible of the data obtained during penetration into the ground. The advantage of CPT is that it is carried out in situ, with continuous recording as depth increases. The experimental study described in this paper indicates the following:

- It is important to carry out measurements of the settlement for shallow foundations and compare these results with calculated settlements.

- Settlement predictions, based on CPT, must be related to at least three tests due to the fact that great differences can occur even within limited areas.

- When the conditions between house/ subground are complex a combined calculation with FEM-analysis, based on CPT-results, can be performed.

9 REFERENCES


Proposal on the improvement of Schmertmann et al. settlement prediction method

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SYNOPSIS: Footing load tests are used to investigate the accuracy of Schmertmann et al. (1978) method applied on the settlement prediction of shallow foundation on sand. The test results are used to the method improvement. According to this investigation the strain factor diagrams used in this method are generally indicated in case of foundations on dense sands. In the contrary, their application in case of foundations on loose sands leads to very unsafe predictions. In the paper the test results are presented and modified strain factor diagrams based on the test results are proposed.

1. INTRODUCTION

Due to the difficulty with obtaining intact samples of granular soil, the prediction of footing settlements is based on data of in situ tests. A widely applied method using the $q_c$ value of CPT test is the method of Schmertmann, Hartmann and Brown (1978). This paper intends to investigate the accuracy of the above method based on high accuracy footing load tests and to propose improvements when necessary.

2. THE SCHMERTMANN ET AL. METHOD

Based on theoretical analyses and on load tests on rigid footing models, the above researchers accept that the centerline vertical strain distributions under loaded footings follow the distributions of Figure 1. For a rigid strip footing the peak strain value occurs at depth (measured from the footing base) equal to the footing width $B$ and becomes zero at depth equal to $4B$; for a square footing the respective depths are equal to $0.5B$ and $2B$. Therefore, for

![Figure 1](image)

Fig. 1. Strain influence factor diagrams a) According to Schmertmann et al. (1978) for a rigid strip footing. b) According to Schmertmann et al. (1978) for a square footing. c) Initial proposal by Schmetmann (1970) for all shapes of footings.

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the settlement calculation they use instead of the stress influence factor, a strain influence factor \( I_s \), whose values vary with depth and with applied footing load according to the diagrams of Figure 1. More specifically, they determine the distribution of \( I_s \) as follows: For a strip footing (L/B=10), \( I_s = 0.2 \) at the footing base, \( I_s = 0.5 + 0.1 \sqrt{p / \sigma' v} \) at depth equal to the footing width \( B \) and \( I_s = 0 \) at depth equal to 4B. For a square footing (L/B=1), \( I_s = 0.1 \) at the footing base, \( I_s = 0.5 + 0.1 \sqrt{p / \sigma' v} \) at depth equal to 0.5B and \( I_s = 0 \) at depth equal to 2B. \( p = \) stress increase at the footing base and \( \sigma' v = \) the vertical effective overburden pressure at depth \( B \). For the settlement determination, they divide the subsoil into thin layers \( z_i \) in thickness. For a strip footing and for a square footing the settlements are determined by the relations

\[
\begin{align*}
\eta &= C_1 \eta \sum_{n=0}^{4B} \frac{L_i}{E_{ud}} z_i \\
\varepsilon &= C_2 \eta \sum_{n=0}^{2B} \frac{L_i}{E_{ud}} z_i
\end{align*}
\]

where \( C_1 = \) coefficient expressing the influence of the foundation depth \( D_f \); \( C_2 = 1.0 - 0.5 \ sigma' v / p \); \( C_m = \) coefficient through which additional settlements due to creep phenomena are accounted; \( C_m = 1 + 0.2 \ log(1/p \ sigma' v) \); \( \sigma' v = \) vertical effective overburden pressure at the footing base, \( t = \) time in years after the load application, \( I_s = \) strain influence factor in the middle of each layer, \( E_{ud} = \) screw-plate deformation modulus in the middle of each layer determined by the \( q_k \) values of CPT tests through empirical correlations. For the values of \( E_t \), Schmartmann et al. (1978) propose the following relations:

\[
\begin{align*}
E_t &= 2.5q_k \\
E_t &= 3.5q_k
\end{align*}
\]

The Schmartmann et al. method consists an improvement of the Schmartmann method proposed in 1970. In his initial proposal the \( I_s \) distribution was independent on the footing load and on the footing shape (Fig. 1c); for the determination of \( E_t \), the correlation \( E_t = 2q_k \) was proposed.

3. TEST DESCRIPTION

The tests consisted of six loading tests on a natural scale model. In an experimental container filled by dry sand, successive loads on a rigid strip footing embedded in the sand, were applied and final strip footing settlements were measured. The reinforced concrete strip footing 0.40 m in width and 0.40 m in height consisted of three independent footings: A central one 1.30 m in length and two external footings each one 1.05 m in length. This division in three footings had been performed in order to avoid the friction influences on the measurements which could be developed between the container walls and the external footings during their settlement. Each of the three footings was loaded by three hydraulic jacks placed in a triangular arrangement. The container dimensions allowed the unconfined realization of sand deformation in it for loads up to the failure limit load. The load of the strip footing was applied in the following way: Too small successive settlement increments were defined by means of a computer. The computer connected to the hydraulic system interrupted the feeding of the jacks just when the defined settlement increment was exceeded. At this moment the forces developed in the jacks and the displacements in the loading points were recorded automatically. Prior to the following loading, a waiting time of about ten minutes intervened for the registration of the final values of the displacements. The loading duration was about five to six hours. Six different densities of a uniform medium-grain to coarse-grain sand were examined. The sand deposit preparation was performed in the following way: The sand was transported in the container by a special funnel 1.3 m in capacity and was bedded by a gantry crane in layers of 0.20 m. After the bedding of each layer, compaction by means of a rectangular vibrating plate 170 m in surface followed. The homogeneity of the sand deposits was checked by measurements of sand density with gamma ray detector. The \( q_k \) values and the laboratory soil parameters were determined for each examined sand density.

4. TEST RESULTS AND PROPOSALS

The investigation should be performed for load values covering the range of footing loadings applied in practice. Therefore, the soil behavior was investigated for applied on the footing base stresses whose values ranged between 68% and 54% of the footing ultimate bearing stress \( q_k \) (values of factor of safety, F.S between 14.72 and 1.83). The relation between applied stresses and settlements was investigated for four load levels.
Fig. 2. Observed distributions of centerline vertical strain $e_y$ under the model strip footing for sand densities $D_s=0.91$ and $D_s=0.50$.

Fig. 3. Comparison between measured and predicted settlements by Schmertmann et al. (1978) for the cases $E_s=3.5q_c$ and $E_s=2q_c$. a) Dense sands. b) Loose sands.

in each density.

From the investigation resulted that distinction between dense and loose sands should be made in case studies of shallow foundation settlements on sand. In this paper, the sands with $D_s \geq 0.64$ will be called dense while the sands with $D_s \leq 0.50$ loose.

In figure 2, the distributions of the centerline vertical strain $e_y$ for a dense and a loose sand are illustrated. Their peak values occur at depth equal to 1 to 2.5 of the footing width B. This depth was ranged between 0.75 to 10 of the footing width B in the remaining four densities. Practically this is in accordance with the assumption of Schmertmann et al. (1978) regarding the position of the peak strain. (The application of Boussinesq elastic solution would lead for a constant with depth elasticity modulus and for a value of Poisson's ratio equal to $v=0.33$ to maximum strain depth equal to 0.5 B).

Consequently, for the settlement prediction the strain factor distribution of the Figure 1a is used. In Figure 3 the predictions are compared to the measurement results. The use of the relation $E_s=3.5q_c$ in Schmertmann et al. (1978) method leads to unsafe predictions. In dense sands the observed settlements are in average approximately double the calculated (Table 1); the predictions in loose sands are in average six times less than the measured settlements (Table 2). Therefore this method was also investigated for the case in which the determination of $E_s$ was performed through the relation $E_s=2q_c$ proposed by Schmertmann in the year 1970. In dense sands the results are generally acceptable; the value of the ratio $s_{obs}/s_{calc}$ ranged between 0.82 and 1.67 with a mean value 1.09 and standard deviation 0.27 (Table 1). In contrary, in loose sands the predictions are very unsafe; the values $s_{obs}/s_{calc}$ ranged between 293 and 42 with a mean value
Fig. 4. Comparison between measured and predicted settlements in case of loose sands.

Table 1. Dispersion values of the ratio $s_{u_{obs}}/s_{u_{cal}}$. Case of dense sands.

<table>
<thead>
<tr>
<th>$s_{u_{obs}}/s_{u_{cal}}$</th>
<th>range</th>
<th>mean value</th>
<th>S.D.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Schmertmann et al. (1978):</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$I_w = 0.5 + 0.1 \frac{p}{\sigma_v'}$</td>
<td>$E_i = 3.5q_c$</td>
<td>1.44-2.92</td>
<td>1.91</td>
</tr>
<tr>
<td>$E_i = 2q_c$</td>
<td>0.82-1.67</td>
<td>1.09</td>
<td>0.27</td>
</tr>
</tbody>
</table>

proposed relation:

$I_w = 0.5 + 0.25 \frac{F.S}{F.S. - 1} \log \frac{1}{1 - D_i} \quad E_i = 2q_c$

| 0.91-1.30 | 1.09 | 0.16 |

Table 2. Dispersion values of the ratio $s_{u_{obs}}/s_{u_{cal}}$. Case of loose sands.

<table>
<thead>
<tr>
<th>$s_{u_{obs}}/s_{u_{cal}}$</th>
<th>range</th>
<th>mean value</th>
<th>S.D.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Schmertmann et al. (1978):</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$I_w = 0.5 + 0.1 \frac{p}{\sigma_v'}$</td>
<td>$E_i = 3.5q_c$</td>
<td>5.12-7.37</td>
<td>5.96</td>
</tr>
<tr>
<td>$E_i = 2q_c$</td>
<td>2.93-4.30</td>
<td>3.40</td>
<td>0.43</td>
</tr>
</tbody>
</table>

proposed relations:

$I_w = 0.5 + 1.2 \frac{p}{\sigma_v'} \quad E_i = 2q_c$

| 0.66-0.99 | 0.86 | 0.13 |

$I_w = 0.5 + 4 \frac{F.S}{F.S. - 1} \log \frac{1}{1 - D_i} \quad E_i = 2q_c$

| 0.72-1.09 | 0.97 | 0.13 |
3.4 and standard deviation 0.43 (Table 2).

What creates unease is mainly the unsafe predictions in loose sands. From the investigation it was found that in case of loose sands the relation

$\lambda_p = 0.5 + 0.25 \sqrt{\frac{F.S.}{100 (D_i + 3.5)}}$

should be replaced by the relation

$\lambda_p = 0.5 + 1.20 \sqrt{\frac{p}{p'}}$. The application of this proposed relation led to the $s_{sub}/s_{sub}$ values which ranged between 0.66 and 0.99, mean value 0.86 and S.D. = 0.13 (Table 2 and Fig. 4).

The strain influence factor diagrams proposed by Schertmann et al. (1978) do not coestimate the distance degree of the applied stress increase $p$ from footing bearing capacity which as known influences the stress distribution and the soil deformation parameters considerably. This influence will be different in dense and in loose sands.

Jänke (1978) studied experimentally, the distribution of the introduced vertical stress $\sigma'_v$ along the vertical axis which passes through the middle of a rigid strip footing. He found that the maximum $\sigma'_v$ value appears at depth equal to 0.65B. In loose sands, $\sigma'_v$ stress increases developed in the layer being between 0.125B and 1.25B are greater than the stress increase $p$ applied on the footing base. The values of $\sigma'_v/p$ increase with the load level increase and their maximum values (appearing at depth equal to 0.65B) range approximately between 1.10 and 1.35 for usually in practice applied loads. An increase of $\sigma'_v/p$ value is also observed in dense sands with the load level increase; however, in case of dense sands, $\sigma'_v$ does not overcome the value of $p$. Jänke explains the above remarks as follows: The lateral displacements in loose sands are greater than in dense sands; the small lateral resistances in this case result in stress concentration near the footing middle. In other words, the support of the footing is mainly concentrated near the footing middle. The same, but in a smaller scale, also occurs when the load level increases for a given density. The lateral displacements increase as the applied load approaches the bearing capacity. Jänke's remarks as well as an expected sufficient decrease of the value of $E_i$ with the load level increase explain sufficiently the reasons why the examined method lead to unsafe predictions in case of loose sands. They also explain the settlement underestimation which was observed several times for high values of load level in case of dense sands. Consequently the sand density and the load level are two very significant parameters which should be coestimated in issues referring to the approach of load-settlement behavior.

For the above reasons the connection of $I_p$ to the load level and to the sand density was examined. In this paper the load level is determined by the coefficient $F.S.$ where F.S. is the factor of safety against foundation failure according to DIN 4017. $F.S. > 1$ expresses the applied stress to the footing in percentage of the foundation bearing capacity.

According to this investigation, the following relations are proposed:

$$I_p = 0.5 + 0.25 \frac{\sqrt{F.S.}}{\sqrt{F.S. - 1}} \log \frac{1}{1 - D_i}$$ dense sands

$$I_p = 0.5 + 1.20 \frac{\sqrt{F.S.}}{\sqrt{F.S. - 1}} \log \frac{1}{1 - D_i}$$ loose sands

The proposed relations are valuable for $D_i < 1$. According to these relations the value of $I_p$ becomes infinite for $F.S. = 1$. The values of $D_i$ and of friction angle, necessary for the determination of $F.S.$, can be evaluated through empirical correlations connecting these parameters with $\sigma'$. The Schertmann et al. (1978) method application with the proposed relations led to the values of $s_{sub}/s_{sub}$ ranging between 0.91 and 1.30 with a mean value 1.09 and S.D. = 0.16 for dense sands (Fig. 5 and Table 1) and between 0.72 and 1.09, mean value 0.97 and S.D. = 0.13 for loose sands (Fig. 4 and Table 2).

![Fig. 5. Comparison between measured and predicted settlements in case of dense sands.](image-url)
5. CONCLUSIONS
The results of the investigation presented in this paper are summarized as follows:

In case studies of shallow foundation settlements on sand, distinction between loose and dense sands should be made.

In dense sands the use of Schmertmann et al. (1978) method is generally indicated. However, for the determination of $E_I$, the initial correlation of Schmertmann $E_I = 2d_e$ proposed in 1970 should be used. The $s_{90}/s_{100}$ values ranged between 0.82 and 1.67 with mean value 1.09 and standard deviation, S.D. = 0.27.

The application of the proposal by Schmertmann et al. in 1978 according to which the relation $E_I = 3.5q_s$ should substitute the correlation $E_I = 2q_s$ led to unsafe predictions. The $s_{90}/s_{100}$ values ranged between 1.44 and 2.92 with mean value 1.91 and S.D. 0.46.

In loose sands the application of Schmertmann et al. (1978) method leads to very unsafe predictions. The mean values of $s_{90}/s_{100}$ were equal to 3.4 for the case of $E_I = 2q_s$ and 5.96 for the case of $E_I = 3.5q_s$. Consequently in these sands the replacement of the Schmertmann et al. relation by the proposed relation in this paper is necessary.

Sand density and load level are two very important parameters which influence the load-settlement relation. Their connection to the strain factor distribution led to more accurate predictions.

6. ACKNOWLEDGEMENTS
The tests have been carried out at the Laboratory of the State Institute of Soil Mechanics of Nürnberg. The author wishes to express his gratitude to Professor M. Kany and Dipl. Ing. S. Jänke who have assisted essentially.

7. REFERENCES

Use of cone penetration testing to elucidate bearing mechanisms of steel pipe piles.

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Tatsunori Matsumoto
Department of Civil Engineering, Kanazawa University, Japan

SYNOPSIS: A series of load tests of steel pipe piles was carried out in a soft rock in 1991, in Noto Peninsula, Japan, in association with design of foundation piles of highway bridges. A number of Cone Penetration Tests (CPTs) as well as Standard Penetration Tests (SPTs) and unconfined compression tests were conducted in the test site in order to gather soil parameters for pile design. Pore pressure dissipation tests using CPT were also conducted to predict freezing (set-up) phenomena of the piles. Dynamic load tests and static load tests of the piles showed a notable freezing phenomena which was anticipated from the dissipation tests. It is emphasized that the pore pressure responses around the piles measured during and after pile driving were similar to the results of pore pressure dissipation tests. The bearing capacity of the piles derived from CPT method were in a good agreement with the measured values. The CPT conducted inside the pile (Pile T3) after the tension test showed a marked reduction of $q_c$ values and pore pressure, $u$, at the level of pile base.

1. INTRODUCTION

A new highway route is under construction within the Noto Peninsula, Japan, which is famous for a deposit of diatomaceous mudstone that is a unique homogeneous soft rock. Pile foundations of some bridges will be constructed in this soft rock for the new highway route.

The design of foundation piles for highway bridges in Japan has been generally based on specifications provided by Japan Road Association (Specifications 1990). Empirical design formulas for piles in these specifications use only blow count, $N$, from Standard Penetration Tests (SPTs). However, such pile design method brought troubles of pile driving in this mudstone (Nishida et al., 1985).

A series of load tests of steel pipe piles was carried out in 1991 in association with foundation piles of the highway bridges (Matsumoto, Michi and Hirano, 1995). A number of Cone Penetration Tests (CPTs) as well as SPTs and unconfined compression tests were conducted in the test site in order to gather soil parameters for pile design. CPTs were also conducted to investigate freezing (set-up) phenomena and bearing mechanisms of the steel pipe piles.

2. OUTLINE OF PILE LOAD TESTS

Three test piles ($T_1$, $T_2$ and $T_3$) and five reaction piles ($R_1$ to $R_5$) were driven with a diesel hammer having a rated driving energy of 108kN·m in a narrow area of $9 m \times 9 m$ (Fig. 1).

The test pile specifications are listed in Table 1. The test piles were instrumented with strain gages at 10 levels to measured axial forces in the load tests. Steel channels welded inside the piles for the protection of the strain gages increased the cross-sectional areas of the piles to 0.04m².

Borehole investigations at Points $B_1$ through
Bₜ and CPTs at Points C₁ through C₄ were conducted before pile driving. These investigations at the other points were conducted after the pile load tests.

Four piezometers (electric transducers) p₁ - p₄ were buried in the ground around Pile T₁ at points B₁ through B₄ as shown in Fig. 2.

**Table 1. Pipe pile specifications.**

<table>
<thead>
<tr>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length L (m)</td>
<td>11.0</td>
</tr>
<tr>
<td>Embedment length Lₑ (m)</td>
<td>8.3</td>
</tr>
<tr>
<td>Wall thickness rₛ (mm)</td>
<td>12.1</td>
</tr>
<tr>
<td>Outer radius rₒ (mm)</td>
<td>400.0</td>
</tr>
<tr>
<td>Inner radius rᵢ (mm)</td>
<td>387.9</td>
</tr>
<tr>
<td>Cross-sectional area A (m²)</td>
<td>0.044</td>
</tr>
<tr>
<td>Young's modulus E'(MN/m²)</td>
<td>2.06 × 10⁴</td>
</tr>
</tbody>
</table>

![Fig. 1. Site layout.](image)

![Fig. 2. Instrumentation of the test ground around Pile T₁.](image)

Dynamic load tests with stress-wave monitoring were performed on all the test piles at pile installation process. Compression load tests were conducted on Piles T₁ and T₂ while a tension test was conducted on Pile T₃ more than 2 weeks after pile driving.

**3. RESULTS OF SOIL INVESTIGATIONS**

Fig. 3 shows the soil profile and the results of SPTs at Points B₅ and B₆ and CPTs at Point C₁ conducted before pile driving. The ground consisted of a thick deposit of diatomaceous mudstone with a top soft clay of 1.5 m thick. The mudstone was fully saturated in the natural condition.

The piezoeon probe used had a diameter of 35.7 mm with a cone angle of 60° and was penetrated at a constant penetration rate of 20mm/s. The area of friction sleeve was 150cm², and the filter location of the pore pressure transducers was immediately behind the cone shoulder.

A relatively constant unit cone resistance qₑ of approximately 3.1 MN/m² was measured in the mudstone below elevation 0. The change of qₑ with depth may comparable with that of SPT N’-values.

The distribution of the unit sleeve friction, fₛ had a saw-shape. The penetration of the piezoeon was interrupted at 1-m intervals in order to add CPT rods. The values of fₛ after adding rods were typically twice the values before stopping though the period was only 1 - 2 min. Immediately after the start of penetration, fₛ values returned to the original values before stoppage of continuous penetration process. This phenomena indicates a rapid soil freezing (set-up) of the mudstone.

The pore pressure, u, attained a relatively high value of approximately 2 MN/m² during continuous penetration process.

A total of 32 unconfined compression tests were conducted on the soil specimens sampled from Boreholes B₁ through B₆ before pile driving. A total of 24 out of 34 test data were selected through the primary treatment of the test results.
Fig. 3. Results of SPTs at Boreholes B₂, B₃ and CPT at Point C₁ before pile driving.

Fig. 4. Results of soil tests.

Fig. 5. Frequency distribution of qₜ values and its statistical properties.

Fig. 6. Frequency distribution of unconfined compression strength qₜ. 
in order to grasp the most probable properties of the ground from the experimental data which may contain various errors produced through the testing procedure (Matsumoto et al., 1993).

The variation with elevation of the natural water content, \(w_m\), and wet density, \(\rho_w\), are very small (Fig. 4). The unconfined compression strength, \(q_u\), and the secant modulus, \(E_{50}\), are relatively uniform with elevation although only two data points were obtained from elevations deeper than -4 m. A statistical analysis of the physical properties, \(w_{m}\) and \(\rho_w\), suggests that the variance of the soil parameters, \(q_u\) and \(E_{50}\), of this test site are also uniform up to an elevation of -12 m.

The values of \(q_u\), \(f\), and \(n\) were recorded at 100mm-intervals. The frequency distribution of \(q_u\) in the mudstone is shown in Fig. 5, while that of \(q_u\) is shown in Fig. 6. It is seen that the coefficients of variance (COVs) of \(q_u\) and \(q_u\) are 0.16 and 0.114, respectively. They are comparable and indicate a uniformity of the ground. It is notable that the CPT results at Points C2, C3, and C4 were very similar to the CPT at Point C1.

The pore pressure dissipation tests using the piezocene probe were conducted at elevations of -2.4 m and -3.4 m. Fig. 7 shows the dissipation of the excess pore pressure, \(\Delta u\), with elapsed time, \(t\), after the stoppage of penetration. The excess pore pressure dissipated rapidly with the time for 90% consolidation of only 5 min.

4. RESULTS OF PILE LOAD TESTS

4.1 Dynamic load test results

The pore pressures in the ground were measured during after the pile driving of Pile T1.

Fig. 8 shows the pore pressure responses around Pile T1 at Points P1, P2, and P3 (see Fig. 2). A high positive excess pore pressure was measured at Point P1 which was located near the pile (1 m from the pile surface), while negative excess pore pressures were observed at Points P3 and P4 which were located relatively far (2 m and 4 m, respectively) from the pile surface. It is seen that the excess pore pressure dissipation curve at Point P1 is similar to that of the pore pressure dissipation test using CPT probe although the time for consolidation is longer for Pile T1. The result of the pore pressure dissipation test using CPT probe may be a good measure for the set-up of soil strength around driven piles, if due consideration of scale effect and local variation of permeability are adequately is made.

It is noted that the total driving resistance of Pile T1 at the last driving was 2.5 MN.

![Fig. 8. Pore pressure phenomena around Pile T1 during and after installation process.](image)

4.2 Static load test results

Compression load tests of Piles T1 and T2 were carried out 29 days and 11 days, respectively, after pile driving. The soil inside Pile T1 was excavated and removed using an auger so that Pile T2 had only the outer shaft resistance.
Fig. 9. Load-displacement curves of Piles T1 and T2 from vertical load tests.

Fig. 9 shows the pile head load, $P$ versus pile head displacements, $w$, curves of Piles T1 and T2. The ultimate capacity of Pile T1 was 4.9 MN. It is notable that the displacement of the top of the soil plug, $w_{top}$, inside Pile T1 was also measured in the load test. The displacements of the pile head and the soil plug were almost identical indicating that Pile T1 reached its ultimate capacity with a perfect plugging mode where the pile capacity is the sum of the outer shaft resistance and the bearing capacity of the ground below the 'closed-end' of Pile T1.

It is interesting that the load-displacement curves of Piles T1 and T2 were identical until $P$ reached 2.9 MN. This may imply that only the outer shaft resistance was mobilized in Pile T1 until after $P=2.9$ MN was applied. Thereafter the inner shaft resistance, which was always equal to the end resistance of the ground below the soil plug, was mobilized resulting in a larger ultimate capacity of Pile T1. The end bearing capacity of the ground below the base of Pile T1 was estimated as 1.2 MN, since the ultimate capacity (the ultimate outer shaft capacity) of Pile T2 was 3.7 MN.

Fig. 10 shows the axial distributions of Pile T1. It is seen that the majority of the applied load was supported by the shaft resistance. The axial force distributions of Pile T2 were similar to those of Pile T1 except that the load transferred to the pile toe was 0 in Pile T2.

Fig. 10. Axial force distributions of Pile T1.

The tension load test was carried out on Pile T3 44 days after pile driving. The test results in Fig. 11 showed that Pile T3 had the ultimate tension capacity of 3.6 MN, which was comparable the ultimate compression capacity $P_{u}$ = 3.7 MN of Pile T2. The measured displacement of the top of the soil plug, $w_{top}$, was almost equal to the pile head displacement also in this tension test.

It is described again here that the total driving resistance of the test piles was about 2.5 MN. All the pile load tests showed the increase of the shaft resistance with elapsed time after pile driving as expected by the pore pressure dissipation test using CPT probe.
5. ESTIMATION OF PILE CAPACITIES FROM CONE PENETRATION TEST

Applicability of pile design using CPT method (ISSMFE, 1977) to piles in this mudstone was examined.

The following semi-empirical equations were used for estimation of the shaft resistance, $f_s$, and the end resistance, $q_e$:

$$f_s = \alpha c_u \quad (\alpha = 0.5) \quad (1)$$

$$q_e = N_c c_u \quad (N_c = 9) \quad (2)$$

Here, $c_u$ is the undrained compression strength and was assumed as $c_u = q_u/2$ in this study, $\alpha$ is the empirical value proposed by Burland (1973) and $N_c$ is the coefficient of bearing capacity. The values of $f_s$ and $q_e$ are integrated to obtain the ultimate shaft and end capacities.

In order to estimate $c_u$ values of the test ground, the following relation was used:

$$c_u = q_u/N_c$$

The value of $N_c$ was determined from the results of statistical analyses of the measured $q_u$ and $q_e$ values (Figs. 5 and 6) as follows:

$$N_c = \frac{q_u}{(\bar{q}_u/2)} = \frac{3.06}{(0.846/2)} = 7.23$$

where $\bar{q}_u$ is the mean value of $q_u$ and $q_e$, respectively.

The estimated shaft and end capacities with pile penetration is shown in Fig. 12. The total capacity, $R_u$, stands for the sum of the shaft and end capacities. The total capacity estimated for Pile $T_1$ agreed with the measured value. However, it should be mentioned that the estimated capacity has a COV of 0.196 since COVs of $q_u$ and $q_e$ are 0.16 and 0.114, respectively.

Fig. 13 compares the sleeve friction, $f_s$, from CPT and the shaft resistance, $f_s$, obtained from the static load test of Pile $T_1$. For elevations deeper than -2m, the values of $f_s$ during continuous penetration is about a half of $f_s$. The $f_s$ values during continuous penetration is comparable to the shaft resistance during pile driving. On the other hand, the peak values of $f_s$ measured just after stoppage of penetration of the CPT probe tend to increase to $f_s$. It may inferred that the peak values of $f_s$ would reach to $f_s$ if the time of stoppage of penetration was more longer.

The above results suggest that CPT may be used to estimate the difference between the shaft resistance during driving and that in the static load test by changing the penetration speed of CPT probe for this particular mudstone.

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Fig. 12. Shaft and end capacities with embedment depth derived from CPT method.

Fig. 13. Comparison of $f_s$ values and shaft resistance $f_s$ from static load test of Pile $T_1$. 

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6. USE OF CPT TO RECONFIRM LOAD TEST RESULTS

The CPTs were carried out inside Piles T1 and T3 after the static load tests to reexamine the load test results.

Although the values of $q_c$, $f_s$, and $u$ measured inside the pipe piles may be affected by limited boundaries where the piezocone having a diameter $d=36$ mm was pushed inside the pipe piles having an inner diameter $D=756$ mm ($D/d=21$) as pointed by Mayne et al. (1990), their profiles clearly showed the changes of the soil conditions due to pile installation and the static load tests as mentioned below.

The results of CPT inside Pile T1 are presented in Fig.14. The $q_c$ values inside the pipe pile increased along the lower 2 m of the pile. Also the $q_c$ values increased up to a depth of 4 m below the pile toe. The pore pressures, $u$, along these sections also increased. Such changes in $q_c$ and $u$ may imply that the ground below the pile toe was compressed by the pipe pile with perfect plugging mode in which the inner soil along the lower 2 m of the pile is highly compressed resulting in a large inner shaft resistance.

The values of $f_s$ obtained from the CPT inside Pile T1 have almost the same amplitudes of those from the CPT at Point C5. Hence, effect of limited boundaries seemed be small.

![Fig. 14. Results of CPT inside Pile T1 after the compression load test.](image)

![Fig. 15. Results of CPT inside Pile T3 after the tension load test.](image)
It is very noteworthy that the response of $u$ gives us useful information of pile integrity. The $u$ values sharply decreased when the piezocene passed through the level of the pile toe. Then, the piezocene test may be applicable as an integrity test for confirmation whether the pile toe reaches the design depth.

The results of CPT inside Pile T3 clearly shows a marked reduction of $q_c$, $f$, $u$ and $q$ at the depth of the pile toe (Fig.15). These observations imply that there was no soil just below the pile toe. Therefore, from these results and the measured $w_{pie}$ it is seen again that the whole soil plug was pulled upward with the pile. Then, the ultimate pull-out capacity was equal to the sum of total outer shaft resistance and the weights of the pile and the soil plug. The weights of the pile and the soil plug were 34 kN and 51 kN respectively, the sum of which were only 2.4% of the measured ultimate pull-out capacity of 3.6 MN.

6. CONCLUSIONS

A series of CPTs was carried out in conjunction with the load tests of steel pile piles driven in the diatomaceous mudstone. The results of CPTs have been compared with other soil tests and the pile load test results in this paper.

Main findings from this field study are as follows:
1. The CPT method gives us good soil parameters for pile design in the mudstone, although it is empirical.
2. The observed set-up phenomena of the test piles were similar to those of the piezocene during stoppage of penetration where the increase in the sleeve friction is associated with the dissipation of the excess pore pressure generated during continuous penetration. The dissipation of the pore pressure imply the increase in the effective horizontal stress acting on the friction sleeve.
3. The set-up phenomena was not observed in the cone resistance $q$.

Further studies on the difference of soil deformations around the piezocene and the driven pipe piles, on the difference of interface characteristics of the piezocene sleeve and the actual piles, and on the scale effect between the piezocene and the actual piles may be required to make the cone penetration testing more useful for the purpose of pile design.

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Sounding Application for Pipe Pile Bearing Capacity Evaluation in Clayey Soils.

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SYNOPSIS: Pipe piles with the open and closed bottom end driven into clayey soils are considered. The conditions of their interaction with the soil are investigated. It is stated that when the open end pile penetrating a soil core is formed in its cavity. The regularity of core forming depends on different factors. The interaction between the core height, clayey soil condition and pile parameters is shown. Factors influencing the pipe pile bearing capacity are also shown. Calculation formula is suggested that uses soil static sounding data for pile bearing capacity determination. The programme "Ring" is worked out that automatizes the calculation process.

1. INTRODUCTION
Investigators turned their attention to pipe piles owing to their advantages compared with square piles of solid section. Pipe piles provide less metal and concrete expenses per 1 kN of bearing capacity, allow to decrease energy loss when their penetration, to use the upper pile part as column footing for placing a column in this upper part, to simplify nodes with overground structures. Owing to above advantages the pipe piles have been successfully used at the residential and industrial objects of cities Nizhny Novgorod, Omsk, Riga, St.-Petersburg, Tallinn, Ufa since 1960.

The peculiarity of pipe piles is that the soil inside them is capable to resist at pile penetration. Because of the soil getting into the pile cavity, the consolidation around the pile decreases and the conditions of pile load transfer onto the footing are changed. The conditions of pipe piles interaction with the soil must be taken into account in design formulas.

2. STATE-OF-ART
The method of static sounding is well known in practice of projecting and construction. This method allows to obtain in the shortest time the reliable information about soils in conditions of natural layering at the given frequency by depth. The engineers of the institute BashNIIstro in Ufa carried out the investigations and worked out the methods of pile structures bearing capacity calculation by static sounding data. The method reliability was reached with the help of detailed soil sounding data analysis and pile static tests results.

Pipe piles differ from others with the cavity presence that is filled with the soil when pile driving. In some cases a dense core is formed in the cavity that becomes part of the bearing structure.

The conditions of soil core forming are shown in the scientific works of Luga A.A. (Russia, 1952), Prudentov A.I. (Russia, 1966), Secha K. (Hungary, 1959), Petrashovich G. (Hungary, 1964), Kloss J. (Poland, 1978). Their model and in-situ investigations (Prudentov A.I.) showed that the core forming regularity depends on different factors, they are: soil conditions, pile sizes (length and diameter), penetration method (pressing in, driving), the inclination of inner pile walls. According to
investigation results it is stated that there exist the limit pile penetration depth, at which the increase of soil core height stops. From this moment the open end pile resistance approximates the closed end pile resistance.

The investigations of BashNIstroi Institute showed that the decrease of open end pile resistance compared with closed end pile resistance depends also on soil strength that is expressed as the index of clayey soil fluidity (Figure 1).

3. STATEMENT OF A PROBLEM

For projecting and construction practice it is very important to know the extent of soil core forming at any given penetration depth and the pile resistance corresponding to that depth. The given problem occurs because of pile structures given length, the limit power of construction equipment and given soil conditions in the absence of bearing soil layer.

The soil static sounding method allows to speed up the pile bearing capacity data obtaining. The common principles of pile bearing capacity calculation according to sounding data have been worked out in the Institute BashNIstroi by Kolesnik G.S. and Ryzhkov I.B. (1968). The suggested method is meant for solid square piles. Design formulas for pile pile bearing capacity determination must take into account factors influencing the soil core forming. In the article in-situ piles in casing for reinforced concrete pipes being driven into the clayey soils are considered.

4. METHOD OF CONDUCTING THE WORKS

All the problems set have been decided analytically and experimentally. Proceeding from the equilibrium condition of stressed minimum clayey soil layer in pile cavity a theoretical formula of core height was obtained. The formula of pile bearing capacity was deduced from the condition of soil core presence in the cavity and formation of the consolidated zone of cylindrical form around the pile.

The process of soil core forming was laboratory studied. Pile models were penetrated into different mediums (sand, clay, plasticine). The regularities obtained were tested at the experimental sites:

In-situ piles of 0.5-0.96 m diameter and 2.3-12 m length were used for tests.

Soils at test sites were quaternary alluvial-deluvial stiff clays and loams and alluvial (flood plain) soft clays with the following characteristics: natural humidity \( w = 0.18 \ldots 0.39 \); soil density of natural humidity \( \rho = 1.60 \ldots 2.02 \) g/cm\(^3\); porosity coefficient \( e = 0.71 \ldots 1.0 \); fluidity index \( J_1 = 0.27 \ldots 0.67 \); angle of inner friction \( \phi = 7 \ldots 29 \); cohesion \( c = 0.016 \ldots 0.085 \) MPa; deformation modulus \( E = 5 \ldots 28 \) MPa.

In places of test piles penetration soil static sounding was carried out with the unit C-832, construction of which was worked out at the Institute BashNIstroi.
Figure 2. Calculation scheme for resistance determination under the pipe pile toe: a) without soil in pile cavity; b) soil core at forming stage; c) the soil core is formed.

Piles were driven with pile driver with fall block mass 2500kg. In total 77 piles were driven at 21 experimental sites, 49 piles with an open bottom end 26- with the closed bottom end. The driven piles were penetration tested with static load, several piles were extraction tested. Tests were carried out not earlier than in 7 days after pile driving.

5. BASIC DESIGN PROPOSITIONS
The study of the soil core forming process showed that the core growth decreased with the pile penetration depth increase. The limit core height depends on soil conditions and pile parameters. With the core increase stopping the resistance of pile with the open bottom end approaches that of the pile with the closed end. The concept of extent of soil core K forming completion is introduced. In the absence of soil in the cavity (K=0), the resistance under the pile toe is determined only with the pile circular section (figure 2a). When the soil core is formed (K=1), the resistance under the pile toe is realized with the whole pile section (see fig.2b). In projecting and construction practice a case is known when 0<K<1 (figure 2c). The solving of the theoretical problem of pile bearing capacity allowed to state that the soil core influences the decrease of soil resistance under the pile and along its lateral side. The value of this decrease depends on soil strength and pile sizes (diameter and length). These factors determine the extent of soil core forming in pile cavity, that is taken into account with the coefficient under the pile toe γ_{CR}. The physical essence of this coefficient is that it shows the extent of soil resistance in limits of pile open section. Soil resistance decrease along the pile lateral side is taken into account with the coefficient γ_{rL}, that shows the extent of compression of the pile lateral side with the soil.

In pipe piles with the closed end soil uplifting onto the surface occurs at the first moment of driving, that leads to resistance decrease under the pile bottom end. With pile penetration depth increase the outward soil uplifting stops. The extent of resistance decrease is also taken into account with the coefficient that depends on the relative depth of pile penetration.

When pile bearing capacity determination by static CPT data the common principles are used that help to calculate square piles. The correction coefficients characterizing peculiarities of piles considered are introduced into the formula. The soil resistance under the pile toe is determined with the account for heterogeneity of soil layering in the zone of pile bottom end.

6. THE SUGGESTED METHOD OF PILE BEARING CAPACITY CALCULATION
The particular value of driven pipe limit resistance $F_u$ by static CPT data is determined by formula

$$F_u = \gamma_{CR} \cdot R_s \cdot A + \gamma_{rL} \cdot U \cdot \sum f_{is} \cdot l_i$$  \hspace{1cm} (1)

where $\gamma_{CR}$ - the coefficient of soil performance conditions under the pile toe that is taken for open end piles according to
table 1 in dependence on soil resistance $q_3$ and relative driving depth $ld$, and for closed end piles according to table 2 in dependence on $ld$.

$\gamma_{cr}$ - the coefficient of soil performance conditions along the pile lateral side, that is taken for open end piles according to table 3 in dependence on $ld$, for closed end piles the coefficient is taken to be 1.

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7. SUMMARY
1. The resistance of pipe piles with the open end depends on the extent of soil core forming under the influence of soil conditions, pile parameters, kind of driving.
2. The calculation formula is suggested for pipe piles resistance determination by static CPT data. Computer programme is worked out that allows to speed up the necessary information obtaining.

$R_s$: the limit specific soil resistance under the pile toe according to CPT data;

$f_s$: mean specific resistance of $i$-th soil layer along the pile lateral side determined by CPT data;

$A_{pl}$: area and perimeter of pile cross section;

$\lambda$: thickness of $i$-th soil layer.

Comparison of test and design values of pile limit resistances showed that the discrepancy up to 20% refers to 79% of results and the discrepancy value up to 50% - to 89,6% of results. The correlation coefficient that shows the interaction between the comparable values is taken to be 0,95.

The programme "Ring" is worked out that automatizes the calculation process. The programme determines the following parameters: particular and mean limit penetration and extraction resistances values at each sounding point, soil reliability coefficient, pile bearing capacity for the whole site. The programme allows to calculate piles of 0.3-1.2 m diameter and 2-20 m length.
Experiences of Mechanical Heavy CPT-Soundings in Finland and it’s Vicinity.

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Reino Hiltunen (Section 3)
Funder Ltd, Helsinki, Finland

SYNOPSIS: The first heavy CPT-sounding rig of Finland was acquired 1981 to be used on projects in Iraq and the rig was transferred one year later to Finland, where this sounding method wasn’t common known. The equipment is mechanical and it possess a total weight of 200 kN.

The main targets has been in Russia (also in former Soviet Union) and Finland. The type of the buildings to be designed has been mainly heavy industrial buildings. Due to the high loads the evaluating the soil parameters have the their importance. For heavy structures the limit range of soft soil is relatively more wide compared to usual structures.

A Static Penetration test similar to the CPT-sounding is standardized in Russia which has made possible to find the basis to get the chosen geotechnical calculations to be approved by the client. The procedure of the test and the world wide comparative test results between CPT-soundings and laboratory tests and in-situ measurements give a good background to find a good estimate on geotechnical parameters to be used.

In Finland the use of the CPT-soundings has been depended mainly on the geotechnical designer’s view. In future can be assumed that the closer connection to Europe may strengthen the practice.

1. INTRODUCTION
The CPT-soundings has not been in common use in Finland so far. In the following cases the CPT-soundings have been requested by the designer due to the demands caused by soil conditions and the nature of the structures.

In all of the cases CPT-soundings have completed a soil investigation consisting of other methods too. The following cases doesn’t have any scientific content but they reveal the practice where an local soil investigation practice takes the first steps to global environment. The use of the CPT-sounding has been unavoidable when foreign projects has been carried out to find a mutual understanding.

2. CASE HISTORIES
2.1 Experiences in Russia

2.1.1 General
The CPT-sounding method is standardized in Russia and is described in the Russian code GOST 20069-81. The equipments are mechanical and they called C-979, where the side friction is evaluated base on the load exposed to the rods, and C-832 equipped with a sleeve for side friction measurements. The tips of these equipments are very similar to the European ones which has been presented by De Ruijer in ESOPT II. The disadvantage these Russian rigs is the low maximum total load, usually 50 to 100 kN.

The Russian design code SNiP 2.02.03-85 allows to use the lowest possible safety factor when defining the pile design load if the design is based on the CPT-sounding results.
2.1.2 Furniture factory in Leningrad
In 1989 a “Furniture Factory” was built to the present St. Petersburg. The Russian client had carried out the soil investigations for the project. Due to the following reasons Finnish contractor decided to carry out complementary CPT-soundings using a maximum total capacity of 200 kN:
- the western contractor had to assume the total responsibility of the project
- the use piles with length of 12 m was favourable
- numerous CPT-soundings carried by the Russian rig where abandoned in the upper layers
- CPT was known by all parties

The piles where supplied by the Russian client and in lengthen those piles shall be done by welding the steel plates locating in the pile ends and to be protected by a bituminous cover as a corrosion protection was considered uneconomical and bad in quality. The Russian Code SNIP 2.01.01-83 allows quite high deformations caused by the subsoil and the use of the piles with length of 12 meters was found to be possible.

Only the cone resistance was measured (Fig. 1). The bearing capacity of the piles were calculated according to SNIP Code 2.02.03 using the formula as follows:

\[ N = \frac{Q}{C} \]  \hspace{1cm} (1)

\[ N_c = \text{design load of pile} \]
\[ C = \text{failure load of pile} \]
\[ F_s = \text{safety factor (}= 1.25 \text{ in case of CPT)} \]

\[ C = P_c + P_t \]  \hspace{1cm} (2)

\[ P_c = \beta_1 \times q_s \]  \hspace{1cm} (3)

\[ P_t = \beta_2 \times f_t \]  \hspace{1cm} (4)

\[ P_c = \text{failure load of pile tip} \]
\[ P_t = \text{failure load of pile shaft} \]
\[ \beta_1, \beta_2 = \text{factors according to SNIP 2.02-03-85} \]
\[ q_s = \text{cone resistance of CPT} \]
\[ f_t = \text{shaft resistance of CPT rods} \]

Based on the calculations a design load of 350 kN was chosen for the precast driven 350 mm by 350 mm, length of 12 meters. The loads where controlled by the Dynamic Test. Loading as described in GOST 5686-78, which proved the actual load of the piles to be sufficient.

![CPT-sounding result of Furniture Factory site in Leningrad.](image)

2.1.3 Brewery in St. Petersburgh
The prevailing soil conditions on the site where loose silt and fine sand. The charasteristic features for the project were, that precast piles 350 mm by 350 mm 16 meters in length were available. No pile joint were not permitted due the bad reliability of the welding joint type. The piles were of the maximum length of the pile factory.

As a consequence of the the above mentioned reasons CPT-soundings were performed in 1993. The cone resistance and the total side friction were measured. One of the sounding results is presented in figure 2.

The nominal load (allowable pile load was calculated as presented by Bustamante:

\[ Q_N = \frac{Q_0}{3} + \frac{Q_0}{2} \]  \hspace{1cm} (5)

\[ Q_0 = A \times k_1 \times q_s \]  \hspace{1cm} (6)

\[ Q_0 = A \times L \times f_t \]  \hspace{1cm} (7)
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$Q_n$, allowable pile load
$Q_r$, limit resistance under pile point
$k_n$, bearing capacity factor
$q_c$, cone resistance
$L_s$, shaft length
$f_s$, skin friction
$S_2$, total safety factors

**Cone Resistance MN/m²**

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**Total Side Friction kN**

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**Fig. 2** Representative CPT-sounding log of the brewery site in St. Petersburg.

Based on the calculations an allowable pile load of 350 kN per pile was chosen to be used.

2.1.4 The for Northwest Power Plant in St. Petersburg

The subsoil conditions in this project where changeable, clay-silt layers, stony gravel moraine layer in the middle, soft silt underlaying by moraine moraine. The Finnish contractor was responsible for the site investigations.

The site investigations, 1994, consisted of ram soundings, CPT-soundings and soil sampling.

The significance of the CPT-soundings was:
- giving soil parameters dimensioning for dimensioning pile bearing capacity and settlements
- sensitivity to discover the variations in the encountered soil layers

The CPT-soundings fulfilled well the expectations. When the cone resistance value of 90 MN/m² percussion

**Fig. 3** Soil conditions of the power plant in St. Petersburg.
drilling or casing boring had to be used to penetrate the upper moraine layer. The CPT soundings where abandoned in general a couple of meters higher due to heavy soil conditions compared to ram soundings. The minimum total force of 150 kN was necessary to achieve adequate penetration. None of the used soil investigation wouldn’t have managed as the only acceptable site investigation method.

2.2 Experiences in Finland

2.2.1 General
The prevailing soil investigation practice is far different that of elsewhere in Europe. A good description of that is given by Gardemeister (1974). So far the cosmopolitan soil investigation methods have not reached routine application in Finland.

2.2.2 Paper Mill Projects
In case the paper mill projects the CPT has proved to be useful in at least in two cases, in Joutseno and Veitsiluoto.

The foundation works are requiring and wide. To make possible to find the most economical construction cost an international contract inquiry should be done. In these both cases for the piling work the foreign contractors requested CPT-soundings to be done and in both cases an European contractor, non-Finnish, got at least some of the piling contract. In case of piling the piles are driven to the depth of the soil possessing the con resistance of 30 MN/m² in minimum if possible.

Fig. 4 Comparative CPT-Sounding and Swedish Weight Sound in Joutseno
As can be observed the CPT-sounding can find more accurate the variations of the soil properties as the Swedish weight sounding (SWT), figure 4. In case when the SWT cannot penetrate by turning the sounding diagram don't give any valuable information compared to the CPT. When compared to the ram sounding test has been found that the ram sounding gives the same monotone diagram on the contrary to the CPT.

As can been seen in the figure 4 the soils possess adequate good bering capacity but un case the paper mill the equipment make great demands on the deformations influenced by heavy dynamic forces.

In both cases also CPT-soundings were used to control the results of the heavy tamping. On this purpose CPT-soundings has proved to be the most economical and useful method.

3. PROJECT STOCKMANN-TALLINN, IN ESTONIA

The construction site of a new department store of the Finnish company Oy Stockmann Ab is located in the city center of Tallinn, in Estonia. The piling works were started in April 1995 and the building is scheduled to be completed by April 1996. The detailed programming of the subsoil investigations as well as foundation engineering design was done by Fundus Ltd. As a part of investigations some 30 CPT tests were done in January 1995 Geotek Oy using heavy mechanical equipment.

The soil profile presented in figure 5 was previously known on the basis of the preliminary investigations consisting deep bore holes, soil sampling and laboratory tests as well as CPT tests carried out by light equipment and terminated at the dense intermediate layers.

On the basis of the preliminary analyses on the foundation engineering solutions the use of the heavy CPT tests was seen to be the best method for detailed investigations. The following primary targets were set for these CPT tests:

- Clarifying the density of the silty sand layers No 14 to 16 including its areal and vertical variation at the site.
- Determination of the boundaries of different soil layers arealy on the site.
- Finding out the cone resistance and sleeve friction values from the deeper soil layers.

The final foundation engineering solution consisted of 600 driven precast concrete piles, some 34 meters long and of 21 cast-in-situ piles in the immediate vicinity of an existing building.

The bearing capacity of piles was primarily calculated by using the methods based on the CPT cone resistance and local sleeve friction as well as comparatively using other methods based on the soil parameters. The different calculation methods gave quite identical values. The real bearing capacity

![Fig. 5 CPT-Sounding log in Tallinn](image-url)
checked by static load test after “rest time” was some higher than calculated. The writer has evaluated that this is caused by skin friction. At the CPT test the measuring of the sleeve friction will in soils in question give smaller values than the the “real” friction. The soil around the sleeve is at the the moment of the measurement disturbed by the cone penetration. The “real” friction will be developed during long “rest time” after the disturbing of soil by sounding or piling.

4. CONCLUSIONS
These deductions presented here are based on the use of the mechanical truck mounted CPT-rig having the maximum capacity of 200 kN. The design objects have been mainly industrial buildings having high concentrated loads.

The main the technical aspects are:
- the cone resistance shall reach a minimum value of 40 MN/m² and respectively the total force 150 kN
- the test results give the geotechnical strength and deformation properties of good degree of accuracy
- the cone gives the information from the certain elevation
- reference material between CPT-soundings and other testing methods (soundings, borings, testing, laboratory testing) are abundantly available all over the world
- mechanical cone is serviceable in heavy soil conditions
- due to the static nature CPT-soundings works both in water saturated cohesive and friction soils
- the sensitivity of the mechanical cone in very soft cohesive soils may be insufficient

The special features in Finland are:
- the subsoil conditions are quite easy due the the glacial periods (unbound soil layers are thin)
- soils containing stones and boulders are common (layers are impossible to be penetrated without boring techniques) as well as frost
- the accessibility by heavy truck is difficult in soft soils

- CPT-sounding method is still quite unknown and only very few equipments are available
- the mechanical cone has low operating costs

Based on the above mentioned facts it can be considered that the mechanical cones have still a certain role in the sense of CPT-soundings.

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Application of Static Sounding for Pile Calculation in Soft Soils.

Dr.-Eng. Gotman Nataly
Ufa, Bashkortostan, Russia

SYNOPSIS: Results of in-situ pile performance investigations in soft clayey soils are reviewed. They are used for elaboration of prismatic pile bearing capacity calculation method according to static sounding data.

Data of testing the two tensopiles equipped with the load cells of different types is analysed. As the result, diagrams of soil specific resistance along the pile lateral side and under the pile toe depending on penetration depth are obtained. The data is compared with static sounding data and depending on the results the coefficients of transfer from the soil specific resistance measured on probe surface, to that on the pile surface are determined.

Depending on the experimental tests and statistical analysis of pile test results comparison with calculations by sounding data a method of pile bearing capacity calculation according to static sounding data in soft soils is suggested.

While elaborating the pile bearing capacity calculation method both homogeneous and multilayer grounds are reviewed.

1. INTRODUCTION
At projecting the pile foundations in soft clayey soils the main problems are: pile bearing capacity determination and the choice of its penetration depth. Data of soil static sounding and pile static test are most often used for pile bearing capacity calculation. As the results of static comparable analysis of piles in soft soils show, there are significant discrepancies between the results of pile calculation according to sounding data and according to their static test. In order to improve the reliability of pile bearing capacity calculation methods, experimental and theoretical investigations with the application of strain-gauge measurements have been carried out. In a result of the investigations a calculation scheme of pile performance in soft soils is defined more exactly and new coefficients of transfer from a probe to a pile are obtained. A problem of definition the soil resistance under the pile toe while its penetrating into the heterogeneous soils is theoretically solved.

2. PRECONDITIONS OF PILE CALCULATION IN SOFT SOILS
The total pile resistance consists of its lateral side and toe resistances. Taking into account that in soft clayey soils piles behave usually as "floating", the definition of soil resistance along the lateral pile side is of prevailing meaning. When pile and probe penetration into soft clayey soils, the conditions of soil deformation at the point of contact with the lateral side are different and defined with the character of "pile-probe" interaction with the surrounding soil. Concrete, as pile pore material, absorbs free water. In a result a soil layer, less saturated than the surrounding one, is formed on the pile surface. "Adherence" of soil to a pile occurs. While applying the vertical load onto a pile...
shear takes place along the point of contact “soil-soil”. Water adsorbs at the metallic probe surface and water film is formed that decreases friction resistance. These differences in soil interaction with a pile and a probe take place in soft clayey soils. In usual soils the effect of soil “adherence” to a pile doesn’t occur. That’s why the calculation methods of soil resistance along the pile lateral side in usual clayey soils need specifying for soft soils.

Soil resistance under the pile toe is determined in significant degree with the character of soil layering. Two, the often repeated cases, are characteristic for sites with soft clayey soils:
- when a pile cuts through the thickness of soft soils and deepens by 0.5-1.0 m into firm soils;
- when a pile “floats” in soft soils and under the toe at the distance of 0.5-1.0 m firm soils are situated.

The traditional calculation method of soil resistance under the pile toe doesn’t take into account the possible uplift that appears in a result of soft layer compression in heterogeneous soils.

Not taking this factor into account leads to increasing of the resistance under the pile toe calculated by sounding data, compared with data of in-situ piles.

Thus, while elaborating the pile bearing capacity calculation method in soft soils two main problems were being solved:
- the calculation method of soil resistance along the lateral pile side was specified experimentally;
- the calculation method of soil resistance under the pile toe in heterogeneous soils was corrected theoretically with the help of the known B.I.Dalmatov and F.K.Lapshin (1975) decision.

3. THE RESULTS OF THE EXPERIMENTAL INVESTIGATIONS
The experimental investigations have been carried out at sites with soft soils. At each site a complex of site tests was carried out that included: static investigations of tensopiles with 30 x 30 sm section, static soil sounding by a probe with a friction coupling; soil mass sampling and laboratory investigation of soil properties.

Tensopile construction was designed on the basis of multisectional pile, consisting of separate elements being jacked into the soil stage by stage. In practice such piles are known as “Mega” piles. Tensopile of “Mega” type consists of 30 x 30 sm sections, 60 sm long, connected with bolted joints. Between pile sections and at the end strain-gauges with the circular measuring element have been placed. Piles were jacked by sections.

At one site piles were jacked up to a depth of 6 m, at another site piles were jacked up to a depth of 9 m. Tensopile tests with static jacking load have been carried out at one site at 4 levels of penetration depth, at the other site - at 5 levels of penetration depth (4.5; 6; 7; 8; 9 m). As conditional settlement stabilization its increase by not more than 0.1 mm at the time interval of 15 min was taken.

Compressive forces trasmitted onto the strain-gauges were measured at the process of static tests at each load stage after the settlement stabilization.

According to measurement results the specific soil resistances under the pile toe and along its lateral side were determined. Forces, measured with the strain-gauges during static tests were shown as the overlapping dependencies schedules “load-toe (lateral side) resistance” and “load-settlement”. The results obtained allowed to analyze the dynamic of pile resistance component development in dependence upon pile load. This analysis shows that in contrast to piles in firm soil when the load is transmitted onto the pile toe after the full friction is realized along the pile lateral side, the pile toe in soft soils begins to work already with the loading beginning. At pile limit load both friction along the pile lateral side and resistance under the pile toe are realized.

Taking all those peculiarities into account the problem of pile bearing capacity calculation in soft clayey soils comes to load determination at which the specific frictions along the pile lateral side and under its toe reach the limit values. That’s why in such soils when static sounding is used for pile bearing capacity evaluation, the model, the most near to a pile, is a probe in “equilibrium” when limit specific resistances are measured along the probe lateral side.
and under its tip. Such probe condition is reached at its stopping. The probe "equilibrium" occurs with the help of "damping unit". Below this type of sounding is named sounding "with a probe stopping".

The specific soil resistances along the test pile lateral side and under its toe, measured at limit pile loads, were compared with the results of specific resistances measurement with a probe at continuous sounding and with a probe stoppings (Fig.1). From the schedules on fig.1 follows:
- soil resistances under the pile toe are less than the soil resistances under the probe tip and the firmer the soil the more is the difference;
- the relationships between soil resistances along the pile lateral side (f) and the probe lateral side (f_p) are determined by soil strength; in clayey soils with f_p=0.01-0.02 MPa the values of the soil specific resistances along the pile lateral side are greater than the soil specific resistances measured by a probe, and in soils with f_p=0.03-0.04 MPa the values of the soil specific resistances along the probe lateral side are greater (at continuous sounding) or equal (at soundings with probe stoppings) to that of soil specific resistances along the pile lateral side.

The recently obtained results for usual clayey soils show the reverse picture, i.e. the decrease of soil resistance along the pile lateral side compared with soil resistance measured with a probe (Trofimenkov, 1995). For transfer from a probe to a pile a coefficient is recommended that equals to f_p/f relation, the value of which is in all cases less than 1. However, the same authors note the tendency of transfer coefficient β to increase with the soil strength decrease and the coefficient dependence on the relative depth of the layer considered.

The analysis of experimental results for soft soils showed that in contrast to usual soils the value of transfer coefficient β is as a rule more than 1 and doesn't depend on the relative depth of the layer considered. With the soil strength increase the value of β decreases. These results are shown as the diagram of the transfer coefficient β dependence on f_p and β_p on q_p (Fig.2). The approximation of dependencies shown in figure 2 is carried out and the expressions for transfer coefficient B calculation as parameters q_p and f_p (MPa) functions are obtained:

- at continuous sounding

\[ \beta = \frac{1}{0.4q_p + 1.9} \]  

(1)

- at sounding with the probe stoppings (a probe is in equilibrium)

\[ \beta = \frac{1}{0.5q_p + 1.23} \]  

(2)
4. SUGGESTIONS ON PILE BEARING CAPACITY CALCULATION IN SOFT SOILS

On the basis of experimental tests results it is stated that the pile bearing capacity calculation in soft soils can be carried out by two-term formula, where the bearing capacity \( F \) is defined as the sum of soil resistances along the pile lateral side and under its toe. For this, the peculiarities of pile behaviour in soft soils should be taken into account by means of transfer coefficient \( \beta \) correction. Thus, the suggested formula will be:

\[
F = \beta_R \cdot q_s \cdot A + U \cdot \sum \beta_l \cdot f_{\text{nl}} \cdot l,
\]

where:
- \( U \) - pile perimeter;
- \( q_s \) - soil resistance under the probe tip;
- \( A \) - pile section area;
- \( f_{\text{nl}} \) - soil resistance along the lateral probe friction coupling;
- \( l \) - soil layers thickness;
- \( \beta_R \) and \( \beta_l \) - coefficients of transfer from probe resistances to pile resistances, defined by formulas 1 and 2.

Formula 3 can be recommended for homogeneous soil when soil layers resistances differ from each other not more than 5 times. Otherwise, especially when piles cut through the soft soils thickness and deepen into the firm soils, the coefficient \( K \) should be put into the first term of formula 3. This coefficient takes into account the uplift out from the pile toe at the expense of compression of overlying soft, deformable soils.

Coefficient \( K \) is defined by formula 4:

\[
K = \frac{q_s}{R}
\]

where \( R \) - the actual soil resistance under the pile toe.

While \( R \) definition for the given case a known theoretical solution of B.I. Dalmanov and F.K. Lapshin (1975) was used. In a result, the coefficient \( K \) dependence on soil strength under the pile toe (\( q_s \), MPa) and values of pile penetration depth into the firm soil (\( H \), m) is obtained:

\[
K = \frac{1}{q_s} \cdot (1.12H + 1)
\]

Thus, formula 6 for the pile bearing capacity calculation in soft soils is recommended.

\[
F = K \cdot \beta_R \cdot q_s \cdot A + U \cdot \sum \beta_l \cdot f_{\text{nl}} \cdot l,
\]

In case of homogeneous soils the coefficient \( K \) in formula 6 must be taken equal to 1.

5. REFERENCES


Use of piezocones during earthworks design for the new Kuala Lumpur airport, Malaysia

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SYNOPSIS: The paper describes the use of piezocones during the third phase of the site investigations carried out for the design of earthworks and ground treatment for the proposed airport site. Digital data transfer was employed and a project geotechnical database established. This enabled computerised modelling of sub-surface soil boundaries and automated settlement calculations to be performed. The layout of the piezocones is described in the context of designing earthworks to limit differential settlements under runways and taxiways.

1 INTRODUCTION
The new Kuala Lumpur International Airport is located at Sepang some 45km south west of the capital city, Kuala Lumpur, on the western side of Peninsular Malaysia.

The airport site comprises an area, square in plan, of approximately 100km². The majority of this land area lies on the flat swampy coastal lowlands. The north eastern sector of the site encroaches onto higher ground formed of residual soils. Erosion of the hills has formed valleys which have been infilled with recent soft coastal sediments. A total of five soft ground valleys are present within the site. The layout of the two runways currently under construction is such that they overlie both high ground and valleys.

2 SITE GEOLOGY
In the valleys an irregular sequence of clay, silt and sand of alluvial and colluvial origin, termed the Lower Sediments, occurs immediately above the residual soils. The Lower Sediments are overlain by variable thicknesses of very soft to soft compressible clays which have been generally described as Marine Clays. Overlying the Marine Clays, and forming the low lying swampy areas in the valley floors, is a layer of peat with an average thickness of 2m, although thicknesses of up to 6m have been recorded.

3 SCOPE OF INVESTIGATIONS
The preliminary investigations were carried out in three phases. The first phase was a walk over ground survey to establish an approximate boundary between the swamp and the higher ground and to obtain limited data on the soft clays.

The second phase comprised an investigation using piezocene soundings, Mackintosh probing and a number of boreholes in each of the five main soft ground areas. The intention of this phase was to establish preliminary geotechnical design parameters for the Marine Clays and to define, in detail, the lateral extent of the compressible soils by means of probing, trial excavations and hand augering.

The third phase of investigations provided sufficient information on the soft clays to carry out detailed design for ground improvement, established an indication of the compaction
characteristics of the residual soils for the bulk earthworks and obtained preliminary data for the outline design of the foundations of structures.

The swampy areas were investigated using a combination of piezocone soundings and mud flushed rotary drilled boreholes, sunk using a cross cut bit, to obtain piston samples. The layout of the piezocones was chosen such that the maximum information was obtained beneath the proposed runways and taxiways. Figure 1 shows the density of explorations in one of the five valleys.

Approximately 1200 soundings were carried out on the proposed airport site. This work was undertaken by four machines, three mounted on crawler track chassis and one mounted on a two wheel trailer, the latter machine being used in the more accessible areas. Hogentogler, Gouda and v d Berg equipment were used. Since the dimensions of the cones themselves and speed of penetration are governed by international codes of practice, the only difference between equipment is the manner in
which data are recorded and presented.

3.1 Electronic Data Transfer

The interpretative report of the second phase of site investigations proposed that, for future work, factual data from contractors would be transferred using electronic means. The third phase investigations saw the successful development and implementation of electronic data transfer (EDT) using the format developed by the Association of Geotechnical Specialists, AGS (1994). This was the initial step towards the development of a sophisticated project geotechnical information management system, Nicholls et al. (1995a).

Digital data from the piezocone soundings were received from the various sub-contractors in formats which were dependent on the particular equipment used. The Engineer prepared software translators to convert data from the cone manufacturers’ formats into the format required by the structure of the geotechnical information management system’s database.

3.2 Database and Surface Modelling

At each exploratory hole position the depths to the various sub-surface soil boundaries were assessed, agreed and stored in the project database. These sampled data were then used in the modelling package which employed a kriging technique, Giles (1993), to estimate the sub-surface boundary depths on a 20m x 20m grid of cells across the site. This digital ground model (DGM) was then used to produce contour plans showing variations in the topography of the soil boundaries and to generate isopach of the thickness of peat, Marine Clay, or Lower Sediments.

Strata boundaries at each piezocone location were used in a specially developed iterative settlement module to calculate settlements and surcharge thicknesses. These data were then incorporated into the DGM to allow contours and isopach of estimated settlements and surcharge thicknesses to be generated.

4 INTERPRETATION AND DESIGN

4.1 Piezocone Soundings and Interpretation

Malaysia has a strong documented record of the use of piezocones for profiling of soft soils. They were used extensively during the design phase of the North-South Expressway, PLUS (1990), which extends for some 1000km from Singapore northwards, along the western coastal region of Peninsular Malaysia, to the Thailand border. This experience was an important consideration in arriving at the decision to adopt this technique to investigate the soft areas of the proposed airport site.

The rapid transfer of piezocone sounding data allowed soil profiles to be prepared very early in the third phase site investigation. Within a short period of time profiles along centrelines of runways and taxiways were available to the designers. This information, together with a limited amount of high quality data gathered during the second phase of the geotechnical investigations, enabled some fundamental decisions to be taken at an early stage in the project regarding the likely design problems associated with variable thicknesses of soft compressible clay. This variable thickness of clay was further confirmed when the lateral extent of the soundings had been carried out and the isopach of clay thickness had been produced by the sub-surface modelling capability of the computerised geotechnical information management system.

Since piezocone soundings were made adjacent to the boreholes, from which piston samples were subjected to careful fabric studies, it was possible to build up a high degree of confidence in the interpretation of the graphical profiles. User confidence on a specific site builds up as the general pattern of the lithological profile emerges. The main advantage of the piezocone over the electric static cone penetrometer is the addition of the measurement of transient pore water pressure behind the cone tip. This additional parameter proved to be invaluable in identifying some of the soil type boundaries. For example, the
almost negligible pore pressure response in the peat made the identification of the interface between the peat and the underlying very soft clays considerably easier.

Figure 2(a) shows a longitudinal profile along the centreline of the runway in one of the valleys. The soil type boundaries were derived from sub-surface modelling. Each piezocene sounding shows cone resistance on the right with the local friction to the left of each profile. The presentation of data in this manner allowed trends to be evaluated.

Figure 2(b) shows a typical lateral section across the runway and adjacent taxiways. The variation in the thickness of the soft clay is clearly demonstrated. This variation in soft clay thickness required detailed study in the design phase to minimise differential settlement. The use of piezocene soundings in such situations clearly defines variations in clay strata thickness which would not be apparent from a conventional site investigation using only a limited number of boreholes.

At specific locations pore pressure dissipation tests were undertaken to assess the field values of the coefficient of consolidation at in situ stresses. These data can be considered reasonably representative of a field value of the horizontal or radial coefficient of consolidation, $c_v$, at stresses below the preconsolidation pressure, but are not appropriate for the clay when loaded. Stresses from fills and embankments far exceed the preconsolidation pressure. Therefore, the field $c_v$ relevant to design cannot be measured at the site investigation stage. The field/laboratory ratio of the coefficient of consolidation, at similar stresses, gives an indication of the presence of
thin partitions of more permeable material which form potential drainage paths.

4.2 In situ Permeability Tests

In situ permeability tests were carried out using BAT equipment which is a development of the piezometer tip, Torskensson (1984). The probe was pushed into the soft ground using conventional piezocone equipment. A phial, either pressurised or evacuated, was lowered down the rods and engaged in the tip. The flow of water out of, or into, the tip was calculated from measurements of the pressure changes in the phial with time. The system was used to establish permeability profiles, which was desirable information when designing ground treatment using vertical drains. Figure 3 shows a comparison between the value of permeability assessed using BAT probe and that assessed from the piezocone dissipation tests.

It should be noted that the permeability measured using the BAT system was generally comparable or greater than that assessed from piezocone dissipation tests. This is attributed to the larger filter size of the BAT probe compared with that of the piezocone. The BAT, therefore, tests a larger volume of in situ material and is more likely to contain preferential drainage paths. The difference between these two forms of test may provide another indicator of the extent of soil micro fabric.

4.3 Soil Fabric Studies

Since vertical drains were likely to be adopted as a form of ground improvement it was essential that soil fabric studies were carried out. 76mm diameter piston samples were extruded in the central site sample store, split longitudinally and racked so that some degree of air drying took place prior to the samples being photographed and logged in detail. In addition to detailed descriptions the geologists sketched the observed surface features of the split samples.

The soil fabric studies allowed direct comparison with data from adjacent piezocones. The ability to compare piezocone data with detailed logging at the same locations allows establishment of a good site specific correlation, Nicholls et al. (1995).

5 CONCLUSIONS

The implementation of a geotechnical information management system in conjunction with a project geotechnical database allowed maximum and efficient use of the vast amount of piezocone and borehole information within the short time available on this fast track design phase. It also allowed early and continual interpretation of geotechnical information to be transferred to the design teams.

Confidence in the computerised sub-surface modelling method has grown such that, in future projects, the positioning of piezocone soundings on a systematic grid would be favoured rather than the concentration of investigations along specific corridors.

Piezocones can be used to profile the soft ground areas of a project and boreholes used to confirm their interpretation and generate design parameters. Data from piezocone soundings can be databased, interpreted and sub-surface models generated within a day. Such early information can be vital in fast track projects.
development.
Experience on the Kuala Lumpur airport project suggests that piezocone sounding contracts should start at the earliest opportunity on a systematic grid basis. Once these data have been assessed conventional soft ground borehole investigations can be designed for maximum benefit, which is to obtain specific geotechnical design parameters at specific locations.

6 ACKNOWLEDGEMENTS
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7 REFERENCES


Calibration Chamber Modelling of Grouted/Driven Piles and CPT in Calcareous Soils

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SYNOPSIS: The presently preferred method of securing jacket structures in calcareous soils is by means of driven/post-grouted piles. To date, however, design has been largely empirical, and has not placed much prominence on CPT data, as is the case elsewhere. Much remains unknown about the interpretation of the CPT in crushable and/or silty deposits, because of the special difficulties in handling and sample formation, and in its relationship to open-ended pipe piles, as well as the long term cyclic capacity of grouted piles.

1. INTRODUCTION

The offshore hydrocarbon producing areas of Australia are characterised by calcareous silty sands of varying size and degree of cementation. Driven pile designs for the region were originally developed from conventional interpretations of the CPT test (eg. Focht, 1994), but were found to perform poorly, developing side frictions as low as 10 - 15 kPa as a result of particle crushing during installation (Angenheer et al., 1973, Hyden et al. 1988). Subsequent developments were based on grouted insert piles, the design of which, if not empirical (eg 100 kPa), was largely controlled by grout pressure, placing minimal dependence on soil properties (usually MP from triaxial test, rather than in-situ testing, eg. Abbs 1992). However, while grouted insert piles have performed very well, they are slow and therefore expensive to construct. Consequently, and particularly for the Bass Strait area, more recent attention has focussed on driven post-grouted technology, wherein the loosened soil contact zone is grouted through the wall of the driven casing. Following initial work by Nauroy et al. (1988), Esso Australia Ltd. commissioned an extensive program of laboratory and field studies during 1987 and 88 to develop design procedures and installation methods, particularly for the grouting operation. One sector of this program was undertaken jointly by CSIRO and Monash University, and focussed on adaptations to a large calibration chamber to allow model pile piles to be driven alongside a CPT in prepared samples of calcareous soils.

2. THE CALIBRATION CHAMBER

The essential layout of the calibration chamber, as set up for this investigation, is as illustrated in Fig. 1. Sample size is 1.2m diameter by 1.8m high, with pressurisation possible to 600kPa laterally and vertically, through a side membrane and base piston. Because of the need to realistically represent in-situ seabed deposits and conditions during the driving, grouting and cyclic loading of model piles, it was clear that all samples would need to be saturated. For this purpose, a diffuser of polyethylene tubing was taped to the base membrane and supplied with de-aired water from a storage tank during the saturation stage. Surplus saturation water was collected from the top of the sample and monitored to assess uptake.
In normal practice, formed samples are \( K_o \) consolidated under the air-driven base piston, with the top platen and chamber lid reacting against a hinged structural test frame (Fig. 2). Because this reaction beam is relatively narrow, a test configuration was conceived which allowed three model piles (2 at 100mm and one at 150mm dia.) to be installed off-centre in a sample, subject to maintaining a clearance of >2D from each other and from the boundary of the sample. This required the cutting of clearance holes through the lid and plywood platen at the appropriate positions, with O-ring seals between to ensure that a vacuum cavity could be maintained (with difficulty!) for the purpose of sealing the main lateral membrane. Heavy side brackets were welded to the reaction beam, in the same three positions, to provide anchorage for firstly the pile driving mechanism and secondly the 250 kN Instron test jack. The reaction beam also mounted, on centre, an electro-mechanical driving mast for the CPT: this was driven initially into the test samples and left embedded throughout the subsequent pile driving and testing, rather than create a cavity. It was further necessary to mount the CPT motor drive unit on a hinged table in order to vacate space required for pile driving activity.

3. MATERIALS

For this investigation, interest devolved principally on a medium grained sand, mostly consisting of coral fragments with larger shells, as recovered from the sea floor at the Kingfish B development site (designated Soil A).

Because the laboratory study was to be supplemented by another investigation at intermediate scale at an on-shore site in South Australia, a sample of this material was also supplied for the laboratory program (designated Soil C). Soil C, being of marine origin, was also calcareous, although significantly more silty than Soil A (Fig. 3). Both soils were air-dried and screened through a 4.75mm sieve, but Soil C contained a proportion of cohesive lumps that needed to be broken up by hand.

Information was also sought for an adjacent platform site in Bass Strait where the sediments were known to be considerably finer (and somewhat cemented), but from which a suitable bulk sample (of at least 1m³) could not be provided. A decision was taken to attempt to model this material by grinding Soil A down to a specified grading through a disc mill, with
about 1.5 m³ being derived from a months grinding (designated Soil B). Whilst the grading of Soil B fell within prescribed limits (as in Fig. 3), it could not be expected that the porous nature of particles would be preserved after grinding, as is clearly evident from the density limits of Table 1.

Table 1. Density Limits for Test Soils

<table>
<thead>
<tr>
<th>Soil</th>
<th>G,</th>
<th>ρ min t/m³</th>
<th>ρ max t/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2.71</td>
<td>1.07</td>
<td>1.34</td>
</tr>
<tr>
<td>B</td>
<td>2.71</td>
<td>1.45</td>
<td>1.74</td>
</tr>
<tr>
<td>C</td>
<td>2.68</td>
<td>1.03</td>
<td>1.34</td>
</tr>
</tbody>
</table>

4. SAMPLE FORMATION

By tradition, sample formation has utilised a stationary sand rainer, as described by Bellotti et al. (1982), although no precedent then existed as to how the present silty materials would perform. In the case of Soil A, the fines ensured that only a reluctant flow was achieved from the rainer through a screen of 10mm holes, by using small charges of sand and assisted by vibration. The resulting void ratios (~1.35 or D, ~35%) were appreciably above target (1.20), but were the minimum attainable by this procedure (CH1). Because of the difficulty of drying a large volume of sand under a tight schedule, the bottom 0.6m of a replicate sample (CH4) was placed wet, at a known water content, by stamping, whereupon a much improved void ratio resulted in that portion (1.00).

In the case of the more silty Soil C, a rained sample (CH2A) proved to be so loose that the volume changes during saturation and consolidation caused the rubber membranes to tear from the base piston (which had a maximum stroke of only 67mm), leading to catastrophic loss of the sample. The impracticality of drying ensured that the whole of the replacement sample (CH2B) was placed by stamping to a void ratio of 1.33, although it is clear from the CPT (Fig. 5) that it was not possible in the circumstances to maintain adequate control of water content.

For Soil B, it was clear that new methods were required, and, indeed, at the specified void ratio of 0.95 (and computed w = 35%) the mix was very sloppy. Therefore, it was decided to premix this soil to a water content just sufficient to allow workability into a saturated state with the minimum of volume change (18%), and stamp in place, complemented by gyratory floating with a wooden float (CH3). This procedure was able to work up a latency, indicating fairly full saturation, with a final void ratio of 0.52 (indicating the extent of particle alteration during grinding). It was also necessary that the lower 0.7m of this sample be formed of Soil A, because of insufficient stock of Soil B, an arrangement which allowed 1m embedment for the test piles with two diameters clearance above the interface (where a layer of filtercloth was located).

Because of the relatively modest bursting capacity of the 16 gauge perforated metal sample former, it was necessary to raise the cell water at a rate to match sample height. At the completion of sample forming, it could not be presumed that the cell water would provide adequate support to the near-saturated sample to allow withdrawal of the former and placement of the top platen. Hence vacuum was applied to the diffuser system for 5 days until the developed suction (as measured on a transducer near the fabric interface) reached a level thought sufficient to support the sample (40kPa). This dewatering caused some 15mm settlement, to be made up with new material, with the resulting sample being quite dense and hard in relation to the more sandy ones.

The sealing of the side membrane onto the platen and the sealing of the cell water cavity relies on the maintenance of a vacuum between the platen and the chamber lid for the duration of a test (4 to 6 weeks), a situation made more critical by the loss of cavity space occasioned by the provision of pile-driving ports and the increased number of boundaries to be sealed. Thereafter, procedures for sample consolidation and saturation were initiated. Although normally done under Kc conditions, this was not possible here because of the compressible nature of the materials and the quite limited piston stroke (as well as the need to maintain...
some reserve for pile driving shakedown). Lateral pressure was raised in stages to about 60% of final specified, before raising the piston pressure, with a final isotropic consolidation to working stress level (details: see Parkin, 1991).

5. CPT TESTING

After consolidation, CPT tests were performed on all chamber samples, under Ke conditions, producing outputs of tip resistance as in Figs. 4 to 6. From these outputs, representative maxima have been extracted as in Table II.

Table II. CPT Results

<table>
<thead>
<tr>
<th>Test</th>
<th>Soil</th>
<th>$C_T$ (kPa)</th>
<th>$q_t$ (kPa)</th>
<th>$t_f$ (kPa)</th>
<th>F.R.</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH1</td>
<td>A</td>
<td>275</td>
<td>7.8</td>
<td>69</td>
<td>0.88</td>
</tr>
<tr>
<td>CH4</td>
<td>A</td>
<td>267</td>
<td>7.55</td>
<td>66</td>
<td>0.88</td>
</tr>
<tr>
<td>CH2B</td>
<td>B/A</td>
<td>272</td>
<td>6.35</td>
<td>63</td>
<td>0.99</td>
</tr>
<tr>
<td>CH3</td>
<td>B/A</td>
<td>280</td>
<td>12.7</td>
<td>126</td>
<td>0.99</td>
</tr>
</tbody>
</table>

For the two pluviated samples of Soil A (CH1 and CH4), the variation of tip resistance in Fig. 4 clearly indicates density variations during the 3-stage pouring operation. Such effects are not characteristic, and would not be welcome, in clean sands but derive from the silt fraction and the induced textural variations in pluviated samples. In the case of Soil C (sample CH2B) (Fig. 5), tip resistance closely follows the profile of moisture content, as monitored during sample formation. These severe variations came about from the need to hastily reconstruct this sample from the debris of a failed one. For Soil B in sample CH3 (Fig. 6), the degree of control is clearly better, producing a more consistent record until the effect of the interfacial fabric manifests (and which probably continues to affect the underlying Soil A).

The data of Table II and Figs. 4 and 5 indicate that tip resistances for Soils A and C are basically fairly similar (as are their void ratios), with co-ordinates that plot, respectively, just above and on the sand/silty sand boundary of the Robertson and Campanella classification chart (Jamiolkowski et al., 1985). On the other hand, tip resistance is notably greater for Soil B, with its much finer grading and largely non-porous particles, plotting well into the sand range (although the higher friction ratio is suggestive of more silt). As is normal, representative readings are not obtained before about 3 diameters penetration.

6. PILE INSTALLATION AND LOADING

Model piles were fabricated from steel tube (5mm wall), fitted with a screwed cap and bevelled leading edge, and strain gauged for axial load. A grout injection system was also provided. Piles were driven through a removable guide collar to 0.9m penetration by means of a drop hammer rig fitted to the pile cap (detail: Parkin et al., 1990). During driving,
under an imposed \( K_s \) condition, cell pressure for Soils A and C fell steadily from an initial 280 kPa to around 200 kPa after about half a metre penetration, evidently as a result of particle crushing and perhaps the dissipation of dynamic pore pressures (together with some upward movement of the base piston). As it was felt to be unwise to allow cell pressure to fall any lower, it would then be raised to 225 kPa before continuing driving. For Soil B, pressure loss was much less, presumably due to the less compressible particles.

During driving, pile driving records were obtained, as in Fig. 7, and these may be compared (very approximately) with the previous CPT records. Pile P2CH1, for example, clearly stiffens from 0.4 to 0.55m (where the peak may be accentuated a little by the increase in cell pressure), before falling away to 0.65m penetration. Pile P1CH2B shows a record that fluctuates and which appears to confirm the peak at 0.6m (on Fig. 5).

After driving and removal of the hammer assembly, the height of the internal soil plug was determined by probing through the pile cap, from which it was concluded that there was relatively little base resistance. The sandy materials were found to occupy 70% or more of the driven volume, rising to 110% for the silty Soil B with its greater initial density. It follows that there would have been little remoulding of the external soil, and that the specified 2D clearance between test piles has been adequate to ensure their independent action.

Whilst driving, a nominal average side resistance could be derived from the dynamic (hammer) force at the pile head (as obtained by connecting one of the uppermost strain gauges to a high speed logger) and the embedded area. A resulting plot (Fig. 8) is seen to drop rapidly over the first 1D penetration (as observed also by Poulos, 1988), before stabilising at a value of 80 to 100 kPa, which is reasonably close to the CPT value (63 kPa) from Table II. Whilst particle crushing would undoubtedly contribute to such a response, it is the Authors' belief that it reflects in some measure inertial factors and some measure of base resistance (the initial peak being clearly not side friction).

![Diagram](image)

**Fig. 7. Driving records**

![Diagram](image)

**Fig. 8. Nominal avg. skin friction during driving**

![Diagram](image)

**Fig. 9. Variation of shaft load and friction**

Subsequently and prior to grouting, an Instron jack was fitted to the pile head and a pull test conducted. Load-displacement curves were obtained from the output of the jack load cell, being generally of the form expected, with largest capacity in the 150mm pile (Table III and Parvin, 1991). At maximum load, a shaft load-depth relationship was plotted from strain gauge data, and from this the skin friction variation by differencing (Fig. 9) and the mean skin friction by fitting a straight line to the skin friction values (Table III). Other piles (than
P2CH1 show base loads in tension of 10 to 20 kN for reasons not explainable at this time, although associated with lower derived side frictions.

Table III. Summary, Pile Pull Tests

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Drive Length (m)</th>
<th>Soil Type</th>
<th>Bit Voids (%)</th>
<th>Cell Press Load (kPa)</th>
<th>Peak Load (kN)</th>
<th>Ax. Sk. Fract. (kPa)</th>
<th>Peak Load Def. (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P2CH2</td>
<td>5.5</td>
<td>C</td>
<td>1.33</td>
<td>30.0</td>
<td>125</td>
<td>120</td>
<td>1.5</td>
</tr>
<tr>
<td>P2CH2</td>
<td>5.5</td>
<td>C</td>
<td>1.35</td>
<td>225</td>
<td>35</td>
<td>35</td>
<td>3.0</td>
</tr>
<tr>
<td>P2CH3</td>
<td>6.0</td>
<td>A</td>
<td>1.34</td>
<td>125</td>
<td>45</td>
<td>35</td>
<td>3.0</td>
</tr>
<tr>
<td>P2CH3</td>
<td>6.0</td>
<td>B</td>
<td>0.63</td>
<td>225</td>
<td>22</td>
<td>22</td>
<td>2.0</td>
</tr>
</tbody>
</table>

7. CONCLUSIONS

A large calibration chamber has been modified to allow model pile testing to be undertaken alongside a CPT, in samples of calcareous sand and silt. Traditional methods of sample forming have been shown to be barely, if at all, feasible, with abnormal compressibility and limited piston stroke creating severe problems. An attempt to model calcareous silt by grinding coarser material was not successful in terms of void ratio, but might appear to have not been of great consequence in terms of side friction developed. It would seem that plugging during driving, and therefore also base resistance, is minimal in these materials, so that in general side frictions relate reasonably well to CPT values (although well above some of the more unfortunate field values). The evaluation of base resistance for displacement, or partial displacement, piles remains for future study.

ACKNOWLEDGEMENT

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REFERENCES


CPT SENSORS FOR BIO-CHARACTERIZATION OF CONTAMINATED SITES

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SYNOPSIS: Recent application of CPT to environmental site characterization has generated a wide collection of new sensors, many of which are focused upon characterizing for bio-remediation activities. Recent examples include probes for determination of soil moisture content, pH, temperature and oxidation-reduction potential. These parameters have been determined to be important for the proper design and maintenance of bio-remediation systems along with assisting in transport modelling.

Currently, one probe is used to determine soil moisture through measurement of the dielectric soil properties, capacitance and electrical conductivity. This same probe can be used for detecting contaminates in-situ based upon electrical conductivity contrasts. A second probe is employed to measure the soil and groundwater pH, temperature and oxidation-reduction potential during a single penetration. Descriptions of these probes are discussed along with case studies. Future uses will combine these packages into a single bio-characterization probe.

1. INTRODUCTION
The clean-up of hazardous waste sites represents a significant challenge facing both the US and more recently many other nations. Additional sites are continually being added to the list of known contaminated sites as land becomes more scarce and cities expand. In this expansion process many former industrial sites need to be reclaimed for new non-industrial uses. Many of these industrial sites are highly contaminated requiring remediation of the site prior to reclamation. The first step in the remediation process is site characterization to select the proper remediation technology.

Environmental site characterization presents new opportunities for Cone Penetration technologies in areas of site investigation and remediation planning, installation and monitoring. Many of the newer remediation systems treat the waste in-situ using biological means. Additional site characterization parameters such as contaminant concentration, moisture content, temperature, pH and oxidation-reduction potential need to be determined to select and maintain these new remediation technologies properly. Recently new CPT sensors for measurement of some of these parameters have been developed and tested.

One of the newest contaminant sensors is Laser Induced Fluorescence (LIF). This sensor is described in more detail in another paper of this symposium. In addition to the fluorescence contaminant sensors, new sensors to monitor soil moisture, temperature, pH and, very recently, oxidation-reduction potential (ORP) have been developed. These parameters are all important for the selection and maintenance of micro-organisms within the soil environment which are used to remediate contaminated sites by reducing the waste materials to non-harmful by-products. In addition,
these new sensing packages provide measurements of the soil parameters which can be altered to control and promote remediation of specific contaminants by certain microbial species.

2. RECENT SENSOR DEVELOPMENTS

CPT-technology is widely accepted as a fast and reliable means of assessing the strength and deformation characteristics of soils. International standardization and certification will further enhance the quality, processing and interpretation of the data.

Throughout the years, equipment has been modified such that samples can be taken and other environmental data can be gathered. The latter innovations are aimed at replacing (part of) the drilling/sampling/laboratory testing with more economical in-situ tests. However, often the attempts to perform environmental measurements were stopped after a while because the required resolutions could not be achieved, and/or ineffectiveness.

Nowadays, modern electronics and miniaturization allow for more sophistication than even a few years ago:
- the Soil Moisture Probe (SMP) provides direct measurements at the soil-probe interface. This is now possible due to the development of a miniature three frequency sensor, containing down hole signal conditioning.
- pH and ORP can also be measured continuously at the soil-probe interface nowadays, rather than indirectly using a single water sample that is drawn into the probe;
- the expensive and fragile laser systems and fiberoptics of LIF-systems have been replaced by a mercury bulb in the cone and a photomultiplier.

Two of these new sensors, the SMP and the pH/ORP probe, are discussed in this paper along with some test results.

2.1 Soil Moisture Probe.

Pollutants present in the soil and/or ground water form one of the major problems for the (re-)development of residential and industrial sites. This requires an environmental survey to determine whether the site is safe for use or not. The pollutants are distributed in the soil mass in a complex manner, requiring additional study. Standard environmental surveys comprise drilling and sampling, laboratory testing and engineering analyses. Only after the laboratory tests are performed, presence, type and amount of contamination at the site can be defined. Likewise, the soil moisture determination requires the performance of soil borings, sampling and laboratory testing. Obviously, these methods are time consuming and relatively expensive.

Laboratory research and field tests performed with dielectric sensors (Hilhorst and D'rijk, 1994; Redman and D'Rijck, 1994) showed the close relationship between the dielectric characteristics (electrical capacitance and conductivity) of the soil mass versus the soil moisture and the free ionic concentration. This new technique formed the basis for the development of a dielectric probe, the SMP, (Hilhorst et al, 1995) to measure the soil moisture content and the electrical conductivity.

The SMP is a new geophysical investigation technique which provides additional information during the performance of a CPT. Water content and an indication of the presence of polluted soil layers result from the measurements. Suspect-sites are screened and mapped. Subsequently, soil sampling at pre-selected (SMP data) locations and depths may be carried out with the same CPT unit for further detailed laboratory testing.

The dielectric characteristics of soil are dominated by the water content in a soil mass (Hilhorst, 1994), and is a function of the density, the specific grain size and grain shape as well as the percentage and type of clay and organic matter. Furthermore, these characteristics depend on the operating frequency of the system. When measuring the dielectric constant it is therefore important to know the soil type. Initially, the friction ratio (local
friction over cone resistance) was used to determine the soil type empirically. The author’s have now developed a three frequency sensor that allows a direct distinction between major soil types (Figure 1). Figure 2 shows the SMP, with the two electrodes 50mm apart, mounted on an isolated adaptor as well as a schematic cross section of the SMP.

2.2 pH/ORP Probe
A second new probe being developed jointly by the authors is a soil pH, temperature, and oxidation-reduction probe. The goal of each of these sensors is to provide a means of making screening level estimates of these parameters in real time during a CPT penetration.

For the pH sensor this means an accuracy of 0.5 pH units and a continuous profile in saturated or nearly saturated materials. Standard laboratory pH devices use a glass bulb approach containing both a measurement and reference electrodes (Westcott, 1978). These approaches are not able to withstand the rugged push environment that occurs on the side of a CPT probe. The authors have developed an approach which uses a semi-metallic material as the measurement electrode. This electrode is combined with a silver/silver chloride reference-electrode. The sensor package consists of three small buttons embedded in a milled flat on the side of CPT probe shown in Figure 3. The semi-metallic measurement electrode is the bottom button and the middle button consist of a ceramic material used as the liquid junction. This button covers a reservoir containing a silver wire immersed in a silver chloride solution. The electronic potential between these two electrodes is related to the pH of the soil solution that the CPT probe is penetrating. The top button is a brass plug containing a thermocouple. This button is isolated from the CPT probe body by an insulating material and provides temperature compensation to the pH readings.

This concept of the pH sensor has recently been expanded and further developed by the inventors to add an oxidation-reduction measurement capability to the sensor. Once again, continuous estimates of the oxidation-reduction electronic potential is the desired output from the sensing package. Much like the pH sensor, which measures the availability of hydrogen-ions, the ORP sensor measures the mixed oxidation-reduction potential of the soil environment. The oxidation-reduction potential is the tendency of a substance to donate or accept electrons. This electronic potential is an indicator of the energy that micro-organisms are utilizing to sustain growth. With ORP measurements proper micro-organisms can be selected for a given soil environment or growth conditions, such as the need for nutrients or oxygen can be determined. The ORP measurement is accomplished using an additional metallic button as the ORP reference electrode and the same ceramic button from the pH sensor is used to provide a liquid junction with the ORP reference electrode. For the ORP sensor, the reference junction consists of a Calomel electrode emerged in the silver chloride solution. To date a laboratory evaluation of the approach has been conducted, and a field evaluation program is planned.

3. SOIL MOISTURE CASE STUDY
SMP tests were performed at 7 different sites in The Netherlands. The CPT’s were complemented with undisturbed sampling at depths where pollution was expected. The evaluation of the data showed that clean and polluted sites could be differentiated, easily. For example, a film of pure product just above the ground water table of non-aqueous phase liquids (NAPL’s) such as oil and chlorinated solvents was detected (Figure 4). This was checked against samples taken from the ground water in stand pipes nearby and appeared correct, although it was not detected during an earlier conventional environmental survey due to the limited thickness of the film.

The measured moisture contents are very
realistic and correspond nicely with the data obtained from laboratory testing. In some cases the SMP-moisture content tends to be slightly higher than the value obtained from the standard environmental survey. This may be caused by the decreasing water percentage of a soil sample in the period between the sampling and the actual laboratory test carried-out and the expansion of the sample before the test is performed.

4. **pH CASE STUDY**

To illustrate the utility of the pH sensor, a case study is presented where the pH sensor was effectively used to improve the characterization efforts and accurately identify regions of coal tar from drilling muds which are also present at the site. The site accepted sulphuric acid by-products from the production of aviation fuel between 1942 and 1946. This material was placed in 12 excavated sumps. Three of the sumps were also partially filled with drilling mud. Drilling mud is not known to occur at the other sumps, although soil material has subsequently been used to construct a cap over all the sumps. The site was placed on the U.S. federal Superfund list in 1982.

The authors came to the site in 1993 to assist in the delineation of the coal tar materials from the drilling muds, soil cap and natural soils. Accurate delineation was very important due to the high costs associated with safe disposal of the coal tar materials (Shinn, 1995). Site characterization was conducted previously with a standard CPT, but was unable to accurately distinguish the coal tars and drilling muds. Our subsequent investigation using CPT with resistivity and pH sensors was able to achieve this goal. It was the use of the pH sensor that provided the largest contrast between the coal tars and the drilling muds, although the other sensors were useful for confirmation of the results.

The previous CPT work was conducted by standard CPT without measurements of the pore pressure during the CPT penetration. As stated, this work was unable to distinguish the coal tars from the drilling muds based upon tip and sleeve stress measurements alone. In the second investigation, pore pressure measurements were used to assist in distinguishing between these two materials, along with the use of resistivity and pH sensors.

CPT-resistivity measurements were conducted to exploit a suspected resistivity contrast between the coal tars and the drilling muds. The initial hope was that the drilling muds would be highly conductive in contrast to the coal tars which are normally non-conductive. This would normally provide the necessary contrast needed for accurate identification. However, the coal tars at this site were combined with HSO₃, used in the refining process. The HSO₃ significantly reduced the pH of the tars while increasing their conductivity. Initial calculations using Archie's Law indicated that the necessary resistivity contrast only existed when the degree of saturation of the tar material was below 25%.

The final sensor used was a pH sensor. This sensor was very effective because the HSO₃, added to the coal tars significantly reduced the pH of these materials. In comparison to all the soil type materials at the site (i.e. the drilling muds, soil cap and natural materials) the pH of the coal tars were generally 5 pH units less than the soil materials. This provided an easily identifiable contrast. A typical penetration from the site is presented in Figure 5. All of the key sensor responses are presented. A region of reduced pH is present from a depth of 3.4m to 7.3m clearly indicating the location of a coal tar material. This was confirmed by soil samples. The resistivity profile does not clearly delineate this material, indicating that the HSO₃ has altered the natural conductivity of the tars. An interesting observation from the site was that the temperature in the coal tar regions were slightly increased indicating some exothermic chemical activity was occurring in these materials.

In summary the utility of the pH sensor
was crucial to being able to accurately delineate the coal tar regions within the other soil materials present at the site. Of the 61 penetrations conducted at the site roughly 50% identified coal tar regions. For delineating the coal tar materials, the pH sensor clearly showed more contrast than any of the other sensors. The profiling capability of this sensor was essential in establishing the extent, depth and thickness of the coal tars. Combined use of the pH sensor with the ORP sensors will yield other new and innovative methods for determination of key soil environment and site characterization parameters.

5. CONCLUSIONS
Nowadays modern electronics and miniaturization allow in-situ and on-line environmental measurements in combination with standard CPT-equipment. The sensors discussed in this paper are operational and have successfully been tested both in the US and The Netherlands. Validation tests have taken place in close cooperation with major research institutes and environmental consulting firms.

Soil moisture content, alkalinity and oxidation-reduction potential can now be measured with far more accuracy than before. The systems described herein do not require specialized data acquisition equipment and/or operators. The sensors discussed here are valuable screening tool providing both quantitative and qualitative estimates of chemical contamination and environmental data. Further validation, research and development are in progress to gather information on additional types of contamination in various soil conditions.

Overall, it may be concluded that these new developments provide both geotechnical and environmental contractors with a fast, reliable and economical means of performing soil surveys and map contaminations. Hydrogeologic information is obtained for contaminant transport modelling, risk assessment and the optimization of remediation alternatives (e.g. soil vapour extraction or bioremediation).

6. REFERENCES
Figure 1. Relationship between permittivity and frequency for different soil types.

Figure 2. The soil moisture probe (SMP) and Schematic cross-section of the SMP.

Figure 3. The soil pH, temperature and oxidation-reduction probe (pH/ORP).
Figure 4. Profiles of eight measurements made during a single penetration test on a project detecting non-aqueous phase liquids just above the ground water table.

Figure 5. Profiles of seven measurements made during a single penetration test on a project to delineate acid tars from drilling muds.
Study of Cone Penetration Resistance of Silty Sand in The Calibration Chamber

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SYNOPSIS: The study of cone penetration resistance in the Calibration Chamber for correlation with properties of sands gained popularity due to the possible measurement of the sand strength and other characteristics. This paper presents the result of a study on cone penetration resistance of silty sand in a large calibration chamber. The silty sand was consolidated prior to testing. The cone resistance was found to correlate with the void ratio of the sample and hence with the state parameter. Pore pressures were observed during the penetration of the cone and comparison with the standard cone revealed that no scale effect was detected. The study has added the understanding of the cone penetration response of silty sand.

1. INTRODUCTION

The cone penetration test is a popular in situ test method for the characterization of soils. However, it must be accepted that the cone penetrometer is an empirical instrument. Soil properties are not directly measured with this device, but must be derived from existing empirical correlations or back-calculated from theories of penetration. The use of calibration chambers has proven to be a valuable tool in the development of these correlations.

These chambers have the advantage over field test conditions in that the boundary conditions during penetration (stress or strain) may be controlled and accurately measured. Also, accurate determination of soil density removes an important element of uncertainty in applying the correlations with soil parameters.

While the calibration chamber offers advantages, it should be realized that the calibration chamber sample size and the imposed boundary conditions are important and can bias test data. Boundary conditions in calibration chamber are usually either constant stress or zero lateral strain. In general, the boundary conditions can be expected to be between the limits of constant stress and zero deformation for both the vertical and horizontal directions.

Holden (1977) suggested the following four limiting conditions which were designated as B1 through B4, and these have been adapted by most researchers.

B1: \( \sigma_v, \sigma_h \) constant
B2: \( \varepsilon_v, \varepsilon_h \) constant
B3: \( \sigma_v \) constant, \( \varepsilon_h = 0 \)
B4: \( \sigma_h \) constant, \( \varepsilon_v = 0 \)

Of these possibilities, the most commonly applied boundaries are B1 and B3 (Parkin, 1988).

2. THE CALIBRATION CHAMBER

A schematic of the chamber system assembled for cone penetration testing is shown in Fig. 1. A detailed description of the chamber has been presented by Sweeney (1987). The chamber is capable of housing a 1.5 m dia. by 1.5 m high soil sample. The main part of the chamber is a 2.9 cm thick cylindrical shell bolted to a circular bottom plate 7.6 cm thick. The sample is contained in a membrane and lateral stress is applied to the sample by air pressure. The radius pressure and the vertical pressure can be independently applied to the sample.

The vertical compression of the sample can be measured using a linear variable displacement transformer (LVDT) mounted at the center of the
The cone penetrometer was inserted into the sample by an 8 ton hydraulic piston with a travel of 163 cm. The speed can be regulated by a pressure compensated flow control valve located on the hydraulic line. A steel frame is mounted on the top plate to provide the required reaction during cone insertion. The hydraulic piston is mounted on a trolley which allows lateral mobility of the piston along the reaction frame beam. The frame itself can be rotated around into three positions corresponding to the alignment of the test holes. This arrangement allows seven cone penetration tests to be conducted in one soil sample. Notably, the type of sand being tested and size of cone penetrometer will dictate whether the multiple penetration approach is valid since boundary conditions can affect the test results.

3. THE CONE PENETROMETERS

Standard cone and miniature cone were used in this investigation. The standard cone has 10 cm² projected tip area and a 150 cm² friction sleeve. The miniature is basically a scaled down version of the Fugro electric cone. The miniature has a 4.2 cm² projected tip area and a 63 cm² sleeve area. It has a capacity of one ton, corresponding to a tip resistance of 220 bars, which is sufficient for liquefaction studies involving the penetration of shallow, loose to medium dense sand. Field case studies using the miniature have been presented by Sweeney and Clough (1987) and Dickinson et al. (1988).

4. SOIL SAMPLE

Approximately 90 tons of Yatesville silty sand were acquired for this research. This alluvial soil from Lawrence County, Kentucky, is from the site of the Yatesville Lake Dam on Blaine Creek. Yatesville silty sand contains approximately 40% non-plastic fines and has specific gravity of 2.67. A typical gradation curve is shown on fig 4.

5. SAMPLE FABRICATION

Based on results of the small scale investigations of sample fabrication of Yatesville silty sand in the laboratory, consolidation from a slurry was the technique selected to use in the calibration chamber (Brandon et al., 1991). Before it was consolidated, the slurry was allowed to compress for 24 to 48 hours under its own weight.
Except for the first 30 cm of soil, the degrees of saturation achieved using the slurry consolidation technique averaged 95%. There was some variation of void ratio with depth in the test specimens, with the largest spread of 0.41 - 0.54. This variation of void ratio was most evident in tests CC-3 and CC-4 due to the failure of the bottom drainage during consolidation.

6. PENETRATION TEST RESULTS
6.1. Cone Tip Resistances
A total of 23 cone penetration tests were performed in 5 calibration chamber specimens including 5 standard cone and 18 minicone penetration tests.

A typical piezocone test result is shown in Fig 5 and the results of six minicone tests and one standard cone test in specimen CC-5 are shown in Fig 6. Three important conclusions can be drawn from these figures:
1. The multiple penetration testing method is repeatable and consistent
2. There is no scale effect between the standard cone and the minicone
3. There seems no bias on results due to boundary effects

Upon completion of penetration test, the sample was excavated carefully. One shelly tube sample and several small block samples were taken to measure the density profile of the calibration chamber specimen.
6.2. Use of CPT to characterize soil behavior

Empirical, semi-empirical, and analytical methods are available for interpretation of the cone tip resistance data. While experience exists with interpreting cone penetration tests in sands and clays, it is not clear that these methods are applicable for interpreting cone penetration data in silty soils. Cone penetration tests in clays are usually assumed to be undrained, and tests in clean sands are considered to be drained. However, silty sands have a permeability between that of clay and clean sand. Thus, the evaluation of material properties of the silty sand using procedures developed for the drained behavior of sand or the undrained behavior of clay has to be viewed with skepticism. McNeel and Bugno (1985) suggested that permeability and associated drainage effects are of primary importance to penetration testing results and that strength, density, and overburden effects are of secondary importance.

Robertson and Campanella (1985) emphasized that no unique relationships exist between cone resistance, in situ effective stress and relative density since other factors, such as soil compressibility, also influence cone penetration results. Since the compressibility of silty sands is typically higher than that of clean sands, this also suggests that silty sand may have a lower tip resistance than clean sand.

Been and Jeffries (1986) and Jeffries (1988) have used the state parameter for interpretation of cone penetration data of cohesionless soils, including sands and silty sands. In their method, the tip resistance is normalized by the mean effective stress, \( \sigma'_v \). The use of the state parameter incorporates the void ratio, the effective stress, and the other characteristics of the sand represented by the slope of its steady state line.

As suggested by Jeffries (1988), a single equation can represent the correlation of the normalized tip resistance and the state parameter by inclusion of the slope of SSL of the sand, \( \lambda_d \). This correlation is presented in Fig. 8. It is shown that for a given value of \( \psi \), sands with steeper SSL have a lower tip resistance. According to observations by other researchers (Peoples et al., 1985; Castro, 1987), the slope of the SSL increases with increasing soil compressibility.
Thus, this confirms the previous findings by researchers that at similar relative densities and overburden pressures, a more compressible sand exhibit smaller values of $q$.  

![Fig. 8. Correlation of Normalised Tip Resistance and State Parameter](image)

A better fit to the data acquired in this research would be the log-linear relationship shown in Fig. 9. Been and Jefferies (1987) reported that the tip resistance of all sands and sands with small amount of fines strongly dependent on the state parameter. However, drainage is an important issue for the case of silty sand. The greater the value of the state parameter, the greater the pore pressure that will be generated during the cone penetration for both drained and partially drained conditions. This would result in a lower shear strength. The slope is probably a function of the soil compressibility and the drainage as well, thus incorporating the drainage characteristics during penetration is recommended for soil which is partially or fully undrained upon loading.

6.3. Interpretation of the Sleeve Friction

It is well known through studies by Biggs (1965) that friction ratio, $F_r$, is closely related to the soil type. Calibration chamber results for Yatesville silty sand show that the measured friction ratios are higher than those suggested by the Schmertmann or the Robertson Campanella. The suggested range of friction ratio for silty sand is from 1 to 4%, while most of the data from the calibration chamber show values from 2 to 6% and occasionally as high as 8 to 10%.

Friction ratio is sensitive to lateral stresses and soil fabric (Huntman, 1985; Houlsby et al., 1988). In these calibration chamber tests, a K value of 1.0 was used. Apparently this high value of K could possibly explain the large friction ratio measured.

7. CONCLUSION

Based on the above discussion, it may be concluded that cone tip resistance of silty sand may be correlated to its steady state condition represented by its density and confining pressure. Scale effect was not detected and the friction ratio seems to be higher due to the relatively high lateral confining pressure in the chamber.

This study has suggested that drainage condition is another parameter that need to be quantified for interpretation of cone penetration resistance of sand containing significant amount of fines. In all, this study has added to the understanding of cone penetration resistance of silty sand.

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Field Correlation of Soil Liquefaction Based on CPT Data

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SYNOPSIS: Correlation of soil liquefaction with cone penetration test (CPT) data is studied extensively based on field performance of sandy soil deposits during recent earthquakes in Japan. The CPT tests are conducted at 47 sites where field performance and strong ground motions during the earthquakes are known. Out of 47, soil liquefaction occurred at 25 sites. Based on the above, empirical correlations separating liquefiable from non-liquefiable conditions are presented, in which the dynamic shear stress ratio causing liquefaction is expressed in terms of cone penetration resistance and friction ratio. The boundary lines determined are dependent on friction ratio and are consistent with those determined from laboratory tests on in situ frozen samples. This indicates that only CPT data, i.e., cone penetration resistance and friction ratio, can estimate the liquefaction potential without using physical properties of sand, e.g., mean grain size and fines content.

1. INTRODUCTION
Several practical methods using in situ testing techniques have been presented for evaluating the liquefaction resistance of a soil deposit subjected to earthquake loading. Among these methods, the standard penetration test (SPT) has been widely used for many years in Japan and North America. However, because of the variability of the SPT results and because of the expedience and reliability of the cone penetration test (CPT), the CPT has received increasing interest in recent years in Japan, and empirical correlations using CPT data have been presented. Most of these correlations are however based on the SPT-based correlation together with the second correlation between CPT and SPT, since there is a limited data base to correlate CPT data with liquefaction characteristics.

Although several methods based on field performance data have been proposed (e.g., Shibata et al., 1988), they require physical properties of sand, e.g., mean grain size and fines content, that cannot be obtained from the CPT test alone. This calls for additional field investigation, and thus impairs the expedience of the CPT for liquefaction evaluations. To overcome this disadvantage, Olsen (1994) presented a liquefaction chart that requires the CPT results only, i.e., cone penetration resistance $q_c$ and friction ratio $R_f$. However, this liquefaction chart is also based on the conventional SPT-based correlation, and has not been shown to be effective in estimating actual field performance.

The object of this paper is to study the correlations among liquefaction resistance, CPT data, and physical properties of sand in the hope of establishing a liquefaction chart using CPT data only. For this purpose, the CPT tests are conducted at sites where field performance and strong ground motions during recent earthquakes are known.
2. CPT TESTS AT SITES SHAKEN BY RECENT EARTHQUAKES

2.1 Test sites and strong motions

The CPT tests were conducted at 47 sites in eight regions, Hokkaido and Hyogo prefecture in Japan. For each site, strong motion accelerograms as well as field performance records are available during recent earthquakes, as shown in Table 1. These include the KUSHIRO-Oki earthquake of January 15, 1993; Hokkaido-Nansei-Oki earthquake of July 12, 1993; Hokkaido-Toho-Oki earthquake of October 4, 1994; and Great-Kobe earthquake of January 17, 1995. The CPT tests were made with a penetration rate of 2 cm/sec, which resulted in continuous records with depth of three components, i.e., cone penetration resistance qc, sleeve friction fs, and pore water pressure pw.

The Kushiro-Oki earthquake caused soil liquefaction mainly at the reclaimed lands along the Port of Kushiro in Kushiro City. The strong motion station of Port and Harbor Research Institute (PHRI) at the West Wharf in the Port of Kushiro registered maximum ground surface accelerations of 344-469 gals (Prompt report on strong-motion accelerations No. 41, 1993). After August, 1994, strong motion stations have been temporally set at 23 sites in the city and its vicinity to study site effects during earthquakes. Most of these stations together with the PHRI station successfully recorded the ground shaking during the Hokkaido-Toho-Oki earthquake. The CPT tests were conducted at 22 sites in which liquefaction took place at 8 sites during the Kushiro-Oki earthquake and at 6 sites during the Hokkaido-Toho-Oki earthquake.

In Nemuro City, the above mentioned two earthquakes induced soil liquefaction at the reclaimed land along the Port of Hanasaki. The strong motion station of the PHRI at the Port of Hanasaki recorded maximum ground surface accelerations of 147-162 gals during the Kushiro-Oki earthquake (Prompt report No. 41, 1993) and 346-380 gals during the Hokkaido-Toho-Oki earthquake (Prompt report No. 44, 1994). The CPT test was conducted at the

![image]

Table 1. CPT test sites.

<table>
<thead>
<tr>
<th>Test regions</th>
<th>Number of test sites</th>
<th>Number of liquefiable sites</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kushiro City</td>
<td>22</td>
<td>8(6)*2</td>
</tr>
<tr>
<td>Nemuro City</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Hakodate City*1</td>
<td>8</td>
<td>4</td>
</tr>
<tr>
<td>Mori Town</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Oshamanbe Town</td>
<td>7</td>
<td>4</td>
</tr>
<tr>
<td>Kobe City</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Nishinomiya City</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Amagasaki City</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

*1: Kamiiso Town in the neighborhood of Hakodate City is involved.  
*2: Kushiro-Oki earthquake (Hokkaido-Toho-Oki earthquake)
Soil liquefaction occurred at all sites during the earthquake.

2.2 Evaluation of shear stress ratio
The shear stress ratio, $\tau_d/\sigma'\nu$, that might have developed in the field during an earthquake is estimated from the following equations (Tokimatsu et al., 1983).

$$\frac{\tau_d}{\sigma'\nu} = t_0 \frac{\sigma_{\text{max}}}{g} \frac{\sigma_v}{\sigma'\nu} (1-0.015Z) \quad (1)$$

$$t_0 = 0.1(M-1) \quad (2)$$

in which $\tau_d$=amplitude of uniform shear stress cycles equivalent to actual seismic shear stress time history, $t_0$=correlation factor in terms of earthquake magnitude, $M$=earthquake magnitude, $\sigma_{\text{max}}$=maximum horizontal acceleration at ground surface, $\sigma'\nu$=initial effective overburden pressure, $\sigma_v$=initial overburden pressure and $Z$=depth below the ground surface in meters.

Table 2. Seismic data at test sites.

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>M</th>
<th>Test sites</th>
<th>$\sigma_{\text{max}}$ (gal)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kushiro-Oki</td>
<td>7.8</td>
<td>Kushiro City</td>
<td>150-410</td>
</tr>
<tr>
<td>(6.8)*</td>
<td></td>
<td>Nemuro City</td>
<td>155</td>
</tr>
<tr>
<td>Toho-Oki</td>
<td>8.1</td>
<td>Kushiro City</td>
<td>90-290</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Nemuro City</td>
<td>360</td>
</tr>
<tr>
<td>Nanso-Oki</td>
<td>7.8</td>
<td>Hakodate City*2</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Moriyama Town</td>
<td>175</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Oshamanbe Town</td>
<td>240</td>
</tr>
<tr>
<td>Great-Kobe</td>
<td>7.2</td>
<td>Kobe City</td>
<td>400</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Nishinomiya City</td>
<td>350</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Amagasaki City</td>
<td>300</td>
</tr>
</tbody>
</table>

*1: Correct value by reference (Seed, et al., 1975)
*2: Kamiio Town in the neighborhood of Hakodate City is involved.

Table 2 shows the seismic data of the four earthquakes at the eight regions. Since the duration of the Kushiro-Oki earthquake was short compared with a typical $M=7.8$ earthquake, a correction factor in terms of earthquake magnitude $t_0$ was assigned a value of 0.58 which corresponds to a $M=6.8$ earthquake (Seed et al., 1975). It is difficult to decide the maximum horizontal ground acceleration at test sites during the Great-Kobe earthquake, therefore the maximum horizontal ground acceleration of 400 gal in Kobe City, 350 gal in Nishinomiya City, and 300 gal in Amagasaki City were used tentatively. The average value of two components of the maximum horizontal ground acceleration was used for Eq. (1). The earthquake data shown in Table 2 are tentatively determined. When the digitized records become available, those values may be modified.

3. CORRELATION BETWEEN FIELD PERFORMANCE DATA AND CPT DATA

3.1 Liquefaction resistance
Fig. 1 shows the field correlation between shear stress ratio $\tau_d/\sigma'\nu$ and modified cone penetration resistance $q_{\text{c1}}$ for all data sets. Modified cone penetration resistance $q_{\text{c1}}$, is corrected to an
effective overburden pressure of 98 kPa from the following relationships.

\[ q_{11} = q_1 / (G'/\sigma_r')^{0.5} \] (3)

where \( \sigma_r' \) is an effective overburden pressure in kPa and \( G' \) is an effective overburden pressure of 98 kPa. The solid circles represent the liquefied data, the open circles the non-liquefied data, and open triangles the boundary data. The boundary separating liquefiable from non-liquefiable conditions is obscure in Fig. 1, because the effects of fines content are neglected.

3.2 \( R_f \)-FC relationship

Fig. 2 shows the relationship between friction ratio \( R_f = (q_1/q_2) \) and fines content FC of the samples obtained from the SPT (Toyokatsu et al., 1995). The friction ratio \( R_f \) tends to increase as the fines content FC increases. The friction ratio of 0.5% roughly correspond to a fines content of 5%, and the friction ratio of 1.0% to a fines content of 10%. This suggests a possibility that the fines content FC can be approximately estimated from friction ratio \( R_f \).

3.3 \( q_{11} \)-\( R_f \) relationship

Fig. 3 shows the combined effect of \( q_{11} \) and \( R_f \) on the occurrence of liquefaction. The boundary separating liquefiable from non-liquefiable conditions is still unclear in Fig. 3, however, all liquefied data fall within a limited range approximately defined by \( q_{11} \leq 15 \text{ MPa} \) and \( R_f \leq 1\% \). This fact together with the trend shown in Fig. 2 suggests that \( R_f \) may be used as a substitute for FC in liquefaction evaluations.

The unclearness in boundary line shown in Fig. 3 is probably due to the neglect of the level of dynamic shear stress ratio induced by the earthquake. Thus, the data shown in Fig. 3 are replotted in Fig. 4(a) and (b), according to the level of shear stress ratio. Fig. 4(a) shows the data for sites where liquefaction occurred with \( \tau_d/\sigma_v < 0.15 \), and for sites where no liquefaction occurred with \( \tau_d/\sigma_v > 0.15 \). In a similar way, Fig. 4(b) shows the data for liquefied sites with \( \tau_d/\sigma_v < 0.25 \) and for non-liquefied sites with \( \tau_d/\sigma_v > 0.25 \). These operations could lead to the boundaries separating liquefiable from non-liquefiable conditions for \( \tau_d/\sigma_v \approx 0.15 \) and 0.25, as tentatively drawn in the figure.

The boundaries separating liquefiable from non-liquefiable conditions is much better defined in Fig. 4 than that in Fig. 3. The critical \( q_{11cr} \) value below which liquefaction occurs appears almost independent of \( R_f \) if \( R_f < 0.5\% \), whereas it is significantly dependent on \( R_f \) if \( R_f \geq 0.5\% \).
Fig. 4. Field correlation between modified cone penetration resistance and friction ratio in which the data points are classified on the basis of the shear stress ratio.

3.4 Correlation between $q_{t1}$ and shear stress ratio

Based on the above discussions, the data shown in Fig. 4 are replotted in Fig. 5(a) - (c), in terms of $R_f$, i.e., $R_f < 0.5\%$, $0.5 \leq R_f < 1.0\%$ and $R_f \geq 1.0\%$, respectively. Also shown in Fig. 5(a) and Fig. 5(b) are the boundary lines determined from laboratory tests on in situ frozen samples (Tokimatsu, et al., 1995).

Fig. 5 indicates that:

1. When the data are expressed in terms of $R_f$, the boundaries separating liquefiable from non-liquefiable conditions are well defined.
2. The critical shear stress ratio below which liquefaction occurs for the same $q_{t1}$, tends to

Fig. 5. Field correlation between shear stress ratio and modified cone penetration resistance in which the data points are classified on the basis of the friction ratio.
increase with increasing $R_r$.

(3) The boundary lines determined from laboratory tests on in situ frozen samples (Tokimatsu, et al., 1995) are consistent with the field performance, and can well separate liquefied from non-liquefied sites. The boundary lines for $FC<5\%$ and $5\% \leq FC<10\%$ approximately correspond to those for $R_r<0.5\%$ and $0.5\% \leq R_r<1.0\%$, respectively. Thus, the friction ratio could be used in place of fines content which enable one to estimate liquefaction potential using the CPT data only.

4. CONCLUSIONS

The CPT tests were conducted at 47 sites where field performance during earthquakes are known. Empirical correlations separating liquefiable from non-liquefiable conditions are presented, in which the dynamic shear stress ratio causing liquefaction is expressed in terms of $q_{ct}$ and $R_r$. Based on the above, the following conclusions may be made:

(1) When the data are expressed in terms of $R_r$, the boundary separating liquefiable from non-liquefiable conditions are well defined.

(2) The critical $q_{ct}$ value below which liquefaction occurs appears almost independent of $R_r$ if $R_r<0.5\%$, whereas it is significantly dependent on $R_r$ if $R_r\geq0.5\%$.

(3) The boundary lines determined from laboratory tests on in situ frozen samples (Tokimatsu, et al., 1995) are consistent with the field performance, and can well separate liquefied from non-liquefied sites. The boundary lines for $FC<5\%$ and $5\% \leq FC<10\%$ approximately correspond to those for $R_r<0.5\%$ and $0.5\% \leq R_r<1.0\%$, respectively. Thus, the friction ratio could be used in place of fines content, which enable one to estimate liquefaction potential using CPT data only.

The correlations presented in this paper are tentative, and the data particularly for sands with significant fines contents are still scarce. Thus further refinement of the correlation with further compilation of field performance is required before they are used in practice with sufficient degree of confidence.

REFERENCES


Predicted/Measured Bearing Capacity of Shallow Footings on Sand

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SYNOPSIS: Nine footings bearing on medium dense sand were load tested to failure. The ultimate bearing pressure \( q_u \) is over predicted using \( \phi \) obtained from published correlations that relate cone tip bearing \( q_c \) to \( \phi \) because the sands are lightly cemented. The finite element method (FEM) was used to model the footings, and soil parameters back calculated from the load tests were used to extrapolate the test results to footings of different sizes and embedment. \( q_u \) was non-dimensionalized using a factor \( R_u = q_u/q_c \), considering that \( q_c \) is affected by most of the same parameters that \( q_u \) is affected by. Charts are presented to predict \( q_u \) for sands with similar geologic history.

1. INTRODUCTION
Cone penetration testing (CPT) is perhaps the most economical and practical in situ test for the typical geotechnical investigation. CPT has been shown to be an excellent tool for stratigraphic logging, determination of pile capacity, and for evaluating the bearing capacity and settlement of footings.

Few full size footings on sand have been load tested to failure to compare the accuracy of predicted/measured \( q_u \). Recently, the results of 9 load tests on shallow footings have been reported in the geotechnical literature. The footings varied in width from 1 to 3 meters, and they were loaded to a bearing pressure of 970 to 1830 kPa.

The purposes of this study were to evaluate the accuracy of published procedures for predicting \( q_u \), and to develop a direct method for predicting \( q_u \) using CPT data.

2. SITE LOCATION
Site 1 is located near Bryan/College Station which is in the central part of Texas, USA. This site is one of two National Geotechnical Experimentation sites, and is located at the Texas A&M University Riverside Campus. Site 2 is near Alvin, Texas which is located along the Texas Gulf Coast about 40 kilometers southwest of Galveston. This site is inside a petrochemical refinery.

3. SOIL MODEL
In situ testing included cone penetration, standard penetration, dilatometer and pressuremeter. Laboratory testing included triaxial compression, gradation analysis, and relative density. The in situ and laboratory tests at both sites are described in detail in the ASCE proceedings "Settlement '94", and they are summarized on Tables 1 and 2.

The bearing soils at both sites are medium dense silica sand. The sands at site 1 are Eocene in age, and they were deposited in a coastal plain environment. The sand is lightly overconsolidated by desiccation of the fines and removal of 1 meter of overburden.

The sands at site 2 are alluvial deposits in an abandoned channel of the Brazos River. The channel was cut into older geologic deposits during the last glacial stages, and deposition...
occurred in the Pleistocene period. The sands have not been preconsolidated due to removal of overburden, but they have been densified due to infiltration of rain water, and fluctuations of the water table. The authors believe that the sands at both sites are lightly cemented with naturally occurring salts.

### 4. SOIL MODEL

The strength and deformation properties of a sand mass are complex, and they are affected by the following variables:

4.1 The internal angle of friction ($\phi$) of cohesionless soil is dependent upon the stress level as evidenced by curvature of the Mohr-Columb envelope at high stress. Thus, $\phi$ during a load test on a shallow footing with little confinement can vary significantly from $\phi$ during a CPT with high confining pressures.

4.2 The modulus of elasticity (E) and shear modulus (G) are both stress dependent and strain softening. Thus, the soil modulus in a mass of sand under and around a footing will vary greatly due to the complex state of stress and strain that exists.

4.3 Poisson’s ratio ($\mu$) and the angle of dilatancy ($\psi$) are both stress and strain dependent.

4.4 Natural sands may be cemented with salts. Standard sampling techniques most often break the bonds, and the sand appears to be cohesionless.

4.5 Other factors are the coefficient of lateral earth pressure ($K_c$), footing width ($B$), depth of embedment ($D$), and footing shape which influence confinement of the sand under a footing.

Most geotechnical engineers do not have the means and methods to evaluate all these parameters. The practicing engineer most often

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**Table 1. Summary of Subsoil Conditions—Site 1**

<table>
<thead>
<tr>
<th>Depth</th>
<th>Description</th>
<th>$S_u$</th>
<th>$\phi$</th>
<th>$\gamma_s$</th>
<th>N</th>
<th>$Q_s$</th>
<th>$P_s$</th>
<th>$E_{pmt}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>m</td>
<td></td>
<td>kPa</td>
<td>%</td>
<td>kN/m$^3$</td>
<td>kPa</td>
<td>kPa</td>
<td>mPa</td>
<td></td>
</tr>
<tr>
<td>0 - 3.5</td>
<td>Silty Sand</td>
<td>36</td>
<td>12</td>
<td>14.6</td>
<td>15</td>
<td>7200</td>
<td>700</td>
<td>8</td>
</tr>
<tr>
<td>3.5 - 7</td>
<td>Clean Sand</td>
<td>36</td>
<td>25</td>
<td>14.9</td>
<td>20</td>
<td>9600</td>
<td>1000</td>
<td>11</td>
</tr>
<tr>
<td>7 - 11</td>
<td>Sand &amp; Clay</td>
<td>36</td>
<td>25</td>
<td>14.9</td>
<td>15</td>
<td>7200</td>
<td>4200</td>
<td>14</td>
</tr>
<tr>
<td>11</td>
<td>Clayey Shale</td>
<td>30</td>
<td>50</td>
<td>50</td>
<td></td>
<td>9000</td>
<td></td>
<td></td>
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</table>

**Table 2. Summary of Subsoil Conditions—Site 2**

<table>
<thead>
<tr>
<th>Depth</th>
<th>Description</th>
<th>$S_u$</th>
<th>$\phi$</th>
<th>$\gamma_s$</th>
<th>N</th>
<th>$Q_s$</th>
<th>$P_s$</th>
<th>$E_{pmt}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>m</td>
<td></td>
<td>kPa</td>
<td>%</td>
<td>kN/m$^3$</td>
<td>kPa</td>
<td>kPa</td>
<td>mPa</td>
<td></td>
</tr>
<tr>
<td>0 - 2.7</td>
<td>Sandy Clay</td>
<td>100</td>
<td>18</td>
<td>17.2</td>
<td></td>
<td>2000</td>
<td>550</td>
<td>4.8</td>
</tr>
<tr>
<td>2.7 - 3.7</td>
<td>Silty Sand</td>
<td>34</td>
<td>25</td>
<td>14.8</td>
<td>12</td>
<td>5750</td>
<td>800</td>
<td>6.0</td>
</tr>
<tr>
<td>3.7 - 5.8</td>
<td>Silty Sand</td>
<td>36</td>
<td>23</td>
<td>14.9</td>
<td>15</td>
<td>7660</td>
<td>810</td>
<td>7.0</td>
</tr>
<tr>
<td>5.8 - 15.0</td>
<td>Clean Sand</td>
<td>37</td>
<td>23</td>
<td>14.9</td>
<td>20</td>
<td>18000</td>
<td>880</td>
<td>9.6</td>
</tr>
</tbody>
</table>
relies on conventional formulas or graphs based on empirical observations supplemented with engineering judgement.

The authors have chosen an advanced soil model for the FEM analysis; however, it does not consider all of the above parameters. The initial shear modulus $G_i$ was modeled using a stress dependent modulus (eq. 1). The tangent modulus $G_T$ was modeled using a hyperbolic strain softening relationship (eq. 2).

$$G_i = G_{ref} \left( \frac{P}{P_{ref}} \right)^n$$  \hspace{1cm} (1)

$$G_T = \left[ 1 - \left( \frac{P}{2} \right) \left( \frac{P - \sigma_3}{\sigma_1 - \sigma_3} \right) \right] G_i \hspace{1cm} (2)$$

The commercially available FEM program PLAXIS, specially designed for geotechnical engineering purposes that runs on a desktop computer (386/586), was used to model the load tests. PLAXIS uses 15 noded triangles, and allows for interface elements and tension cutoff. Large strain calculations were performed using an update Lagrangian formulation. The square footings at site 1 were modeled as round footings so that analysis could be performed using axisymmetric computations.

5. ANALYSIS

FEM analysis was performed to model the load tests at site 1. Shown on fig. 1 is the FEM mesh for a typical footing. The soil parameters used for analysis are summarized as follows:

- Reference shear modulus ($G_{ref}$) = 84,000 kPa
- Reference stress ($P_{ref}$) = 300 kPa
- Internal angle of friction ($\phi$) = 37°
- Power ($n$) = .5
- Poisson’s ratio ($\mu$) = .35
- Cohesion (c) = 14 kPa

Results of the load tests and FEM analysis are summarized on fig. 2. Correlations between predicted/measured load settlement response of footings F-1, F-2 and F-5 was excellent. However, FEM over predicted the load settlement response of footings F-3 and F-4. These 2 footings were in the southeast corner of the site, and variations in subsoil conditions probably exist.

$q_o$ for the underreamed footings at site 2 was computed using conventional methods of analysis. This required subtracting friction on the shafts ($P_s$) from the total load ($P_{tot}$) to determine the load carried in end-bearing ($P_a$), and extrapolating the load settlement data to a relative settlement of .05B. Results of the load tests are summarized in Table 3. The values for $q_o$ at site 2 should be considered as lower bound estimates because the footings were bearing on a .4 m. thick layer of stiff clay above the bearing sands.

The most widely used procedure of computing $q_o$ of a footing on sand is the general bearing capacity formula:

$$q_o = cN_c + qN_q + .4\gamma B N_r \hspace{1cm} (3)$$

Soil compressibility is a significant factor, and the general bearing capacity equation does not take this parameter into account. Expansion of a cylindrical or spherical cavity which considers soil compressibility has been used to evaluate bearing capacity. However, neither limit equilibrium or cavity expansion theory correctly models $q_o$ of a shallow footing on sand.
The most widely used procedure for computing $q_u$ using CPT data is to estimate $\phi$ from published charts, and then calculate $N_d$ or $N_q$ to be used in the general bearing capacity formula. $N_d$ and $N_q$ have also been correlated directly to $q_u$. However, most correlations between $q_u$ and $\phi$ are based on calibration chamber tests which are performed on unaged sands. Since many deposits of sand are aged and cemented, $q_u$ could be significantly overestimated.

Shown on Table 4 is a comparison of measured/predicted $q_u$ using 5 different procedures. It should be noted that $q_u$ is significantly over predicted. One reason is that $q_u$ from the load tests was defined as occurring at a relative settlement of .05B, and $q_u$ continues to increase with settlement. Another reason is that $\phi$ was estimated using published charts that correlate $\phi$ to $q_u$ based on unaged, uncedemented sands tested in calibration chambers.

**Table 3. Summary of Load Tests**

<table>
<thead>
<tr>
<th>Site</th>
<th>Footing</th>
<th>$P_{tl}$</th>
<th>$P_s$</th>
<th>$P_u$</th>
<th>B</th>
<th>D</th>
<th>$q_u$</th>
<th>$q_n$</th>
<th>$R_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>F-1</td>
<td>10.3</td>
<td>10.3</td>
<td>3</td>
<td>.76</td>
<td></td>
<td>1.14</td>
<td>7.18</td>
<td>.16</td>
</tr>
<tr>
<td></td>
<td>F-2</td>
<td>2.4</td>
<td>2.4</td>
<td>1.5</td>
<td>.76</td>
<td></td>
<td>1.07</td>
<td>7.18</td>
<td>.15</td>
</tr>
<tr>
<td></td>
<td>F-3</td>
<td>8.9</td>
<td>8.9</td>
<td>3</td>
<td>.89</td>
<td></td>
<td>.98</td>
<td>7.18</td>
<td>.14</td>
</tr>
<tr>
<td></td>
<td>F-4</td>
<td>6.7</td>
<td>6.7</td>
<td>2.5</td>
<td>.76</td>
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<td>7.18</td>
<td>.15</td>
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Table 4. Predicted/Measured q_<i>u</i>

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<th>F-3</th>
<th>F-4</th>
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<td>1070</td>
<td>980</td>
<td>1070</td>
<td>1100</td>
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<tr>
<td>Hansen (1957)*</td>
<td>4170</td>
<td>2730</td>
<td>4170</td>
<td>3700</td>
<td>2230</td>
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<tr>
<td>Balla (1962)*</td>
<td>7060</td>
<td>4190</td>
<td>7060</td>
<td>6100</td>
<td>3160</td>
</tr>
<tr>
<td>DeBeer (1988)</td>
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<td>1410</td>
<td>2230</td>
<td>1550</td>
<td>1010</td>
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<tr>
<td>Schmertmann (1977)</td>
<td>3170</td>
<td>2110</td>
<td>3170</td>
<td>2820</td>
<td>1760</td>
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<td>Meyerhoff (1956)</td>
<td>2250</td>
<td>1350</td>
<td>2250</td>
<td>1950</td>
<td>700</td>
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</table>

* Ø estimated using correlations of Ø with q<sub>u</sub> from Robertson and Campanella (1983)

6. PROPOSED PROCEDURE

q<sub>u</sub> is a function of the following variables (Ø, G<sub>d</sub>, μ, K<sub>R</sub>, D, B). q<sub>u</sub> is also a function of these variables plus the interface angle between the soil and cone. A simplified means to compute q<sub>u</sub> is to assume that there is a direct ratio between q<sub>u</sub> and q<sub>1</sub> (eq. 4). The authors term this ratio N<sub>R</sub>, and N<sub>R</sub> is a function of the footing width (B) and depth of embedment (D). This assumption is not entirely correct because the mode of failure is different between a cone penetrometer and a shallow footing.

q<sub>u</sub> = R<sub>R</sub>q<sub>1</sub> + σ<sub>ω</sub> .................................. 4

FEM analysis was performed to evaluate the effect of embedment and footing size on q<sub>u</sub> using soil parameters back calculated from the footing load tests. Shown as a dash line on fig. 3 is a graph of R<sub>R</sub>/B for 2 common depths of embedment based on FEM analysis. Also plotted on fig. 3 are the values of R<sub>R</sub> determined directly from footing load tests at sites 1 and 2, and two other sites located in Sweden. The solid lines relating R<sub>R</sub>/B are based on our interpretation of the load tests data and FEM analysis.

The values for R<sub>R</sub> are for square or round axially loaded footings. Modifications should be included to account for different footing shapes, eccentric load conditions, and sloping ground surface or footing base.

Prediction of q<sub>u</sub> is only one of the many factors that needs to be considered in the design of shallow footings. Often, settlement considerations determines the allowable bearing pressure used in design. An adequate factor of safety against ultimate failure should be applied because of unknowns in the subsoil stratigraphy, accuracy of the structural analysis, and uncertainties in being able to accurately predict behavior of sand. Typically, a factor of safety of 3 is used for dead plus sustained live load conditions and a factor of safety of 2 is used for dead plus full live load conditions.

Good engineering judgement should be employed when analyzing q<sub>u</sub> for footings in layered soil. For instance, if an extremely weak layer of clay or loose sand was located within the zone of influence below the footing, punch through or excessively large settlements could occur.

Since q<sub>u</sub> is a function of both G and Ø, the values of R<sub>R</sub> in this report are probably not valid for unaged, uncemented sands or heavily overconsolidated, moderately cemented sands. Therefore, the charts relating N<sub>R</sub> to q<sub>u</sub> should only be used for aged, lightly cemented sands with a similar geologic history. q<sub>u</sub> values for such a deposit will probably range from 6,000 to 12,000 kPa, and the friction ratio will be in the order of .3 to 1 percent. Additional field load tests will be required to determine if similar values for R<sub>R</sub> exist for sand with other geologic history.
7. SUMMARY
FEM analysis was used to model load tests of five shallow footings bearing on medium dense sand that is lightly cemented. The predicted/measured load settlement curves were in good agreement. The back calculated soil parameters were used to predict the load settlement curves for footings with varying widths and depth of embedment. \( q_u \), as a relative settlement of 0.05 B was then non-dimensionalized by dividing it by \( q_u \). This ratio is referred to as \( R_u \), and \( R_u \) varies with the width and depth of embedment. The values of \( R_u \) are limited to lightly cemented, medium dense sand with similar geologic history. \( q_u \) of a round or square footing is estimated using:

\[
q_u = R_u q_u + \sigma_m
\]

8. REFERENCES
SURVEY ON SITE OF PORT SALIF IN YEMEN

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SYNOPSIS: The CPT with measurements of two parameters: the temperature and the pore pressure made a basis of complex engineering-geological investigations at a site of the Salif seaport. Dynamic sounding and vibrosounding were used for confirmation of the CPT results. The pile foundation design of hollow steel piles is based on the procedure of plug formation by piles driving; thus the piles are to be calculated as with the closed end. The investigation of the rate of consolidation at the site and the settlements forecast were carried out on the basis of the CPT and a stationary measuring element for pore pressure. During the construction period the pile tests were carried out by means of a Junlann-30PM. The vibromethod was proposed for the arrangement of the territory (the cuirasse distraction) and the port area.

The complex engineering-geological conditions of the construction site in particular the properties of carbonic deposits should be taken into account for the design and calculation of the pile foundations. The properties of these deposits differ greatly from these ones of the quartz sands (J. Hagenaar, 1982). The presence of large electric potentials in the soils, high corrosiveness of the soils in relation to steel, technogenesis factors: blasts which are regularly being produced at the salt pit at a distance of 1.5 km from the deep-water wharf, the application of pipe steel hollow pipes - all these factors taken together put a number of complicated problems by the engineering geological investigations, as well as the design of pile foundations, and an assessment of the construction site and the port area.

Standard and traditional approaches in correspondence with the national and foreign standards (SNiP 1.02.07-87 Engineering investigations in the construction, SNiP 2.02.03-85 Pile foundations, Found-72 France etc), the well known solutions which can be found in science - technical literature on problems of pile foundations design of steel pipes as well as the problems of the territory consolidation and its arrangement can not be completely applied in this specific case.

In 1993 there were bored 17 boreholes 35 m deep and made 16 sound boreholes 29 m deep along the line of the cordon.
In 1994 there were bored 3 sounding boreholes in the area of the set piles.
In both cases probing was carried out by the PIKA-3N field testing set of apparatus for static probing. The principal advantages of the PIKA penetration apparatus are as follows:
- enables to get more information as compared with the analysis;
- the possibility of concordance with any driving devices (static apparatus for static sounding, boring equipment, which enables to apply the technology of combined sounding and perform the investigation to a depth up to 60 m; the rigs in particular at the site of the Salif port - Juntann-30PM); as well as the possibility of investigations of soils on land and sea, what had been demonstrated in Hodeida and Salif not violating by this the standard technology of sounding;
as a result of engineering-geological investigations there had been distinguished the following engineering-geological components (EGC) in the cut:
- EGC-1 - middle sand with shell rocks, loose and of middle density;
- EGC-2 - coarse gravelly sand with shell rocks, mainly loose;
- EGC-3 - dustlike sand with shell rocks, loose, of middle density;
- EGC-4 - fine sand with shell rocks, loose;
- EGC-5 - clayey silt, sometimes loamy and sandy loamy, fluid and secretly fluid;
- EGC-6 - shelly soil with sandy filler;
- EGC-7 - softand fluid plastic clays and loams;
- EGC-8 - includes sands, mainly dustlike, sometimes middle and coarse; include of crushed stone and pebbles of limestone are marked;
- EGC-9 - includes clayey soils, sandy loams; loams that transform into clays; the typical colours are greenish-brown and greyish-brown; include of gypsum pockets; by their consistency the soils are solid and half solid;
- EGC-10 - are of mixed colour: the presence of thin layers of dustlike and fine sand is marked, as well as the crushed rock of gypsum inclusions 5-7 sm in size (10-20 %); the soils by their consistency are from tight-plastic to solid; the soils are very dense; the EGC-10 is 1.6-1.9 m in thickness.

With the aim of simplification of processing the experimental data the 4 layers had been distinguished in the end.
EGC-1, EGC-2, EGC-3, EGC-4 contain sands free dustlike to coarse with shellrocks, from loose to middle density; the norm value of modulus of deformation is $G_{\gamma}=12.5$ MPa; the angle of internal friction is equal to $\Phi = 29^\circ$.

EGC-3A - loamy sand of grey colour, limestone, fluid, seldom plastic, with include of few shell rocks and layers of dustlike sand; the norm value of modulus of deformation is $G_{\gamma}=12.0$ MPa, and the angle of internal friction - $\Phi = 28^\circ$.

EGC-7 - clay and loam, sometimes loamy sand, fluid plastic and soft plastic, of dark grey colour, limestone with few shellrocks and in accumulations. The norm value of modulus of deformation is $G_{\gamma}=7$ MPa, the angle of internal friction - $\Phi = 28^\circ$ and cohesion $C_{\gamma}=0.024$ MPa. The soils EGC-5 (silts) and EGC-7 are the single hemetic type in the cut of sea Holocene deposits and are identical by their composition.

EGC-9 and EGC-10 - are clayey soils of mixed colour: the presence of thin layers of dustlike
and fine sand are characteristic for them as well as the gypsum
covered rock inclusions 5-7 cm in
size (up to 10-20 %). By their
consistence they are from hard
plastic to hard and very hard.
The norm value of modulus of de-
f ormation - EN 142 MPA, the angle
of internal friction - \( \psi = 26 \)
and cohesion - CSN 0,032 MPA.

As far as the EGO-7 can be re-
garded as the weakest unit in the
soilbase and can determine
the settlements during and after
the construction period the in-
vestigations on measuring the
consolidation coefficient C'w in
laboratory (Taylor's method)
and by static sounding (taking into
account G. Sanglerat recomenda-
tions -1065) had been performed.
As a result, the norm value of
C'w was equal to 2,68.10^-5 sm/c
Pore pressure measurements were
performed inside 6 sounding bo-
reholes. There were performed 7
measurements inside each boreho-
le. The field of pore pressure
was calculated. There were no
deviations from the hydrostatic
law of pore pressure distribu-
tion.

The soil massif temperature
was studied inside 7 boreholes
at 3-6 points. It was estab-
lished that the minimum tempera-
ture at the depth of 5-6 metres was
32.8°C, maximum temperature -
38°C at the depth of 26-27 m.
The temperature gradient di-
rected from sea to land and from
the surface to the depth takes
place.

The conducted investigations
and the analysis can be distin-
guished into 3 independent sec-
tions which are never the less
connected by engineering geolo-
gical conditions:
- the bulwark;
- the territory of the port;
- the area of the port.

It was proposed to increase
the bulwark stability by means
of 2 row disposition of piles 20
m long with a diameter of 820 mm
parallel to the line of the cor-
don. These piles from the other
side were necessary for the cra-
ne rails.

The principal task was determi-
ning the pile capacity in the
bulwark.

One of the principal theoreti-
cal and to a large extent expe-
nmental investigations was a
question of soil influence insi-
de the pile on its performance
or so called the plug, so far as
the plug presence or its absence
in its traditional sense raised
questions of determining the
depth of pile sinking. Calcula-
tion of the consolidation in the
area of a set pile, the intro-
duction of the term "pile clus-
ter" in a different way.

The complex investigations al-
lowed to establish that the plug
took place in this specific ca-
se.

The generalisation of the in-
formation, analysis and calcula-
tions had been performed ac-
cording to a special progranme. The
idea of pile calculations is as
follows.

The calculations were per-
formed on the basis of SNIP
2.02.03/85. The formulas from
the SNIP were used for cal-
culating the piles capacity tak-
ing into account the influence of
seismic loads. So far as the
bulwark design was performed of
hollow steel pipes the processes
of pile driving and their per-
formance had been modelled and
analyzed by two versions:

1. The piles capacity was cal-
culated as the sum of soils re-
stance forces under the low
end of the pile, which was in a
shape of a circular section of
the pipe, and the forces acting
on the internal and external
surfaces of the pipe.

2. The plug formation inside
the pile and calculations of the
bearing capacity for the pile.

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with the closed end. The mechanism of plug formation and its destruction had been discovered while analyzing the pile driving process, as well as the results of static probing. The corresponding analytical expressions were obtained. Above that, the quantitative and the qualitative characteristics of the plug formation were determined. These characteristics are influenced by the dynamic of a piles driving process as well as the properties of engineering geological components.

One of the principal conditions of the plug formation for the given site is the transition of the pile open end out of sandy soils into clayey ones while driving. The calculations of the depths where plug formation as well as sliding and the stable plug formation had taken place have been performed. These calculations were based on the data of static probing.

The depth of pile sinking, at which the demanded pile capacity was obtained had also been calculated. On the basis of obtained data the forecast and the evaluation of dynamic processes occurring by piles driving using a Juntann-30PM pile driver had been calculated.

The pile capacity for the depth of sinking of 27.7 m with a small deviation of 0.3 m will be equal: F = 2232.4 kN and taking into account seismicity (8 numbers), F=1451.1 kN.

The developed technique of piles capacity evaluation based on the data of static sounding allowed to investigate the process of plug formation by piles driving as well as the formation of a dead plug. Due to this fact it became possible to propose a new construction technique with the application of steel hollow piles. The new technique provides the increase of piles capacity and optimizes the piles driving process.

The calculations show that by coefficient of consolidation C &=0.69 sm/\text{c} determined by different methods (laboratory and field) and the load P of 3 kg/cm², the layer No7 may give a settlement of 20-30 sm within 2-3 years.

The active method of consolidation with the application of drains couldn’t be used because of economical problems, that’s why the decision was taken to conduct the experimental rounding of the port area by means of fills along with the registration of the settlements of this area and the pore pressure of the layer No7, and thus to test the theoretical forecasts.

The cuirasse is the thick solid cemented layer of middle and coarse sand in 1.0-2.0 m thick on the surface or below the level of the surface of the site. It may be the obstacle for subsidence. However the cuirasse is not distributed along the whole territory of the port. The mechanical properties of the cuirasse can’t be determined by any traditional methods, that’s why the special methods including vibropenetrating and special boring had been used for the estimate.

By loading the large area the cuirasse can’t sustain either, that’s why the settlements observations by loading by sand during a construction period are of great significance.

With this purpose there had been developed “The technique of pore pressure measurement in the layer No7 in the process of consolidation of the soil base of the port Salif territory as well as of the quality of soil consolidation in situ”, the probe and the system of registration (Piezotone-1M) of special design and
measuring elements of pore pressure for fixation and registration of the consolidation process. In addition, the special system consisting of deep reference points and surface ones had been developed for geodesic observations.

For the first time in the native and foreign practice of pore pressure measurements in soil massifs the probe of special design was forced down to the given depth (24.3 m, the layer N07) by the Juntann-30PM pile driver and left in the soil massif. During the construction period the settlements, pore pressure and loads had been registered simultaneously over the fixed intervals of time and thus the information on the process of consolidation at the port territory have been presented graphically.

As far as the measuring element of pore pressure was at a distance of 10 m from the rear wall of the bullwark, the additional loading on the site was of great difficulty (the initial loading = 0.15MPa). It should be noted that the technique and the apparatus for testing the quality of the sandy soil consolidation had been applied for the first time. The technique and the apparatus had no analogs. The portable 9 kg field device gives significantly more detailed express information on the quality of sands consolidation layer by layer as compared with laboratory determinations. These devices and techniques are used in Russia and in the CIS (the Common Independent States) for testing the quality of consolidation of sands in the soils bases as well as at the sites of different types, etc. They are highly valued by foreign specialists. The device and the technique were tested successfully at the port territory in 1993.

Besides, the Vibropenetrometer 3N enables to choose the optimal thickness of the fills layer for machines of special design applied for consolidating the sandy base.

The construction of the port area has been carried out quite intensively. The rate of construction was influenced by the characteristics of the applied apparatus and equipment, and also by the properties of the seabed soils. The Red sea offshore, which is bordered by the coastal plain, is covered with a lot of coral reefs. The reefs run by long ranges along the seashore. There can be distinguished two types of reefs: bordering reefs that run along the shore and the barrier reefs far from the shore (J. Hagenaar, 1982). To our observations the bordering reefs are at a distance of 50-100 m from the shore, their depth is about 9–13 m (the salt wharf and Hodegeida).

Mining of the bordering reef with the purpose of deepening the seabed for navigation could find great difficulties, what already had taken place during a construction period in the area of the port Salif. It should be thought about the equipment which can destroy these reefs. One of the foreign firms used the special equipment for mining the seabed soils of the Red sea.

The other important problem is the cuirasse: its exploitation in the bullwark and in the shoresone. The problem of designing the new technology for hydromechanized method of the cuirasse loosening whilst arranging the port Salif area has aroused due to great technical difficulties connected with the seabed deepening of the port area. So, in the process of the cuirasse mining down from the sea side, the large portion of the cuirasse...
se has broken off from the massif and pressed down the sucking device, resulting in an emergency situation. Besides arising technical problems, the additional measures were to be taken concerning the labour organization and keeping strictly the industrial safety measures.

By engineering-geological investigations in 1983 the cuirasse mining by a boring installation PBU-1 was of great difficulty. In 1984 during the field tests the cuirasse had been driven by a Juntann-30PM. The pipes 168 mm in diameter and 10 mm in the wall thickness were driven; by this, the pipes were either broken more than once at a lower end or bent under the pile driver.

It was supposed that the cuirasse at the shore between the bullwork and the sealvel could be mined by an excavator. However in practice it seemed unrealistic: the excavator wasn't powerful enough for this purpose as well as the bucket design left much to be desired. The most effective method of cuirasse boring is the vibromethod by the Russian researcher D.D. Barkan, which is widely used nowadays both in Russia and abroad. Due to this method, the applied vibrator should be coordinated from the technical point of view with the specially designed cuirasse crusher. This one was designed as a pipe with a cone end 250 mm in diameter, 10 mm in a wall thickness, up to 4000 mm long; 4 blades of a trapezium shape were molded on the pipe from the oblique side at the end; their dimensions were: 1000 mm in length, 800 mm in height and not less than 10 mm in thickness.

The results of the crusher application with the purpose of the cuirasse mining show as following:

1) time expenses per one run along the borehole does not exceed 10 sec;
2) the cuirasse mining on land does not exceed 5 working shifts during the construction period the test static sounding of the soils by a pile driver Juntann-30PM had been performed. The results show that all the requirements of the Internatioanl Standard for static sounding had been observed. In all cases the apparatus for static sounding PIAA-3N had been applied.

Experiences Using CPT Data for Assessing Liquefaction Potential of Mine Tailings

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SYNOPSIS: Empirical correlations which essentially convert CPT to SPT data have been widely used in order to assess the liquefaction potential of soils. Mine tailings, which are comprised of a manmade rock flour/water mixture do not necessarily behave in the same manner as a naturally occurring soil. Geotechnical investigations at nearly twenty sites have provided significant insight into the behavior of mine tailings.

As a result of the vast quantity of test experiences, a better understanding into the behavior of mine tailings has been realized. This improved understanding is particularly applicable to the CPT measurements, and the use of this data for assessing the potential for liquefaction of mine tailings.

1. INTRODUCTION

Mine tailings are a manmade byproduct of the mining and milling industry, the volume currently being produced reaching huge proportions. However, while their earth dam counterparts have received considerable attention by the engineering world, it is not until recently that tailings dams have been the center of focus for engineering. Failures of several poorly engineered and operated facilities have led to a new era in the design of tailings dams (Jayapalan, 1980).

For assessing the behavior of a tailings dam, standard penetration tests (SPTs) are typically carried out. Conventionally, mine tailings are deposited hydraulically, and settlement occurs under water, creating a loose deposit of low relative density. Since tailings are ground to a fine pulp, and because factors such as aging, cementation and soil skeleton effect the SPT blow count, but are likely not present in tailings deposits, the SPT blow count can be very low. The measured SPT values shown in Figure 1 are within the expected range for mine tailings, based on the experience of the authors. In typical tailings...

Figure 1. Typical Plot of Measured SPT Blow Count Values for Mine Tailings
deposits, the measured blow count from the SPT is in the range of zero (weight of rods) to five. Unfortunately, the SPT blow count is relatively insensitive at these low values, the range where this information would be most useful (Poulos, 1988), and the values most often obtained in mine tailings.

The electric piezocone (CPT), when equipped with precision strain gages and pore pressure transducers, has been found to yield accurate and repeatable measurements of tip resistance, sleeve friction and both static and dynamic pore pressure in mine tailings. The CPT has been found to respond rapidly enough to identify thin (less than 0.1 m) layers within the tailings deposit, where such changes are nearly impossible to detect with the SPT.

2. ADVANTAGES OF THE CPT
The CPT offers many obvious advantages over the SPT, including the wealth of information which may be gained in a relatively short time, and at a reasonable cost. The SPT has some distinct disadvantages, including the dependence on equipment and operator, method of boring and borehole fluids. With the CPT, many of these factors may be eliminated, assuming one is using a precision cone instrument and an experienced operator. While the repeatability of an SPT test is often questionable in loose soil deposits, the ability of the CPT to reproduce data can be exceptional. In evidence of this statement, a plot of two CPT probes are shown on Figure 2. The two probes were conducted within 0.3 m of one another, the second probe is indicated with a dashed line in Figure 2. The comparison of the two plots shows remarkable agreement with tip resistance, sleeve friction and pore pressure. It may be noted that the

![Figure 2. Plot of Two Closely Spaced CPT Probes in Mine Tailings](image)
3. MINE TAILINGS

Mine tailings are a ground rock mass which are mechanically ground to a particle size required for successful milling and extraction of the mineral product. As such, the tailings may contain a significant fines fraction, but seldom do these fines exhibit any cohesion. Experience has shown that the majority of tailings may be classified as a silty sand or sandy silt.

Being a ground product, the particles usually are quite angular, and as such, often have very little particle-to-particle contact once deposited, resulting in very high void ratios. Given the above facts, many tailings deposits are susceptible to liquefaction.

4. LIQUEFACTION POTENTIAL

Studies conducted by Seed and DeAlba (1986), indicated that the liquefaction resistance of a soil is proportional to the fines content of the soil under consideration. However, recent work carried out by Vaid (1994) suggests that this concept may not be entirely true for mine tailings. Based on a series of cyclic triaxial tests on tailings samples where the void ratio and silt content of the test samples were varied, Vaid noted that silty sands (at the loosest state achieved in his tests) have similar cyclic strengths regardless of the silt content or void ratio. Additionally, Vaid noted that the cyclic strength of siltly sand appears to decrease with increasing silt contents. It should be noted that in the laboratory experiments, a state of liquefaction was never reached, but rather was defined by a 2.5 % single amplitude axial strain in 10 stress cycles. Findings from this work are shown in Figure 4. Given this information, it would appear that a fines correction for the assessment of liquefaction potential in mine tailings should be applied with discretion, if at all, especially in light of the low SPT blow counts recorded in these deposits.

Seed and DeAlba (1986) presented a method by which CPT data may be used for assessing the liquefaction potential of a soil through a conversion of CPT data to SPT values. Since the background analysis for this method was based on SPT data, this additional step is required. Until a sufficient number of
sites which undergo or are resistant to liquefaction are tested with the CPT, this appears to be the best available technique at this time.

Seed and DeAlba note that the correlation between CPT data and the SPT blow count is dependent on the mean grain size of the soil. It should be mentioned that research by Ulrich and Hughes (1994) suggests that the CPT/SPT ratio is better represented by a constant number for a given mine tailings deposit, and is relatively independent of the mean grain size. This effect may be true for two reasons. First, Seed and DeAlba's correlation is based on many naturally-occurring and engineered soils. Since there are intrinsic differences between a quasi-static CPT penetration and the dynamic SPT, it is logical to state that the soil structure, or skeleton may effect each test slightly differently. Since tailings have little or no skeleton, this absence in itself would cause a variation. Secondly, tailings are mechanically broken particles in which the grain size is dependent not on the particle strength, but rather the milling requirements. As tailings are deposited and settle, finer fractions and coarser fractions tend to separate, resulting in a highly layered deposit. The fine and coarse layers have essentially the same characteristics in regard to the strength and composition of the individual particles. Given this information, it is apparent that the relation between the CPT and SPT may be appreciably different in tailings than in naturally-occurring soils.

The results of the work by Seed and DeAlba is given in Figure 5. Two factors

![Graph showing Cyclic Stress Ratio vs Void Ratio](image)

Figure 4. Liquefaction Resistance of Silty Sand as a Function of Void Ratio (After Vaid, 1994)

![Graph showing Stress Ratio vs Tailings Deposit](image)

Figure 5. Relationship Between Stress Ratio Causing Liquefaction and Resistance for Sands and Silty Sands (Showing Extrapolation of Curves)
which play a major role in the assessment of potential for tailings to liquefy are the fact that tailings contain significant fines, and that the CPT tip resistance may be quite low.

Since the CPT tip resistance obtained in tailings may be considerably less than 40 MPa (40 t/sf), it is quite likely that the data would fall outside the lower range of results given by Seed and DeAlba. This has, in fact, been found to be the norm for mine tailings, rather than the exception. The engineer is then faced with the problem of extrapolating Seed and DeAlba's relationship. On first observation, it would appear that the lower portion of the family of curves is a straight line (refer to Figure 5). Extrapolation of the data in the range of interest would lead to a finite value of cyclic shear strength for a CPT tip resistance of zero. This, in our opinion, is an unconservative assumption. A more reasonable approach is to extrapolate the curves to the origin since a material exhibiting a zero tip resistance should have little or no resistance to liquefaction.

The second point to consider is the fines correction which should be applied. Let's take as an example a tailings material with 35% fines and a modified CPT resistance of 40 MPa. Using Seed and DeAlba's method (refer to Figure 5), one would obtain a cyclic stress ratio of 0.20. Considering the work presented by Vaid (1994), it appears that this fines correction is unconservative for non-cohesive fines. If instead one were to use the curve for a relatively clean sand (and a q'/N_{qv} ratio of 4.4), a cyclic stress ratio of 0.10 is obtained.

5. PORE PRESSURE RESPONSE

One aspect of cone testing which is often nearly disregarded is the pore pressure data. Data on the pore pressure can be very helpful in the analysis of a tailings deposit. The location and behavior of pore fluids in the tailings deposit can have a major impact on the potential for liquefaction. Additionally, since volume change characteristics are of considerable importance with regard to liquefaction potential, the pore pressure response during probing can provide guidance as to pore pressure behavior during earthquake shaking. That is, if positive excess pore pressures are generated during cone penetration, there is at least substantial evidence that pore pressures could build up during earthquake shaking.

Through the use of pore pressure dissipation tests, a relationship may be made between fluid behavior and location of the phreatic surface. By carrying out dissipation tests at various levels and locations in the deposit, a comparison of the dynamic, insitu static and normal hydrostatic pore pressures may be made.

An example of such pore pressure data is shown in Figure 6. This data (obtained at a facility with a full underdrain system) indicated that the phreatic surface is approximately 15 ft. below the ground surface. In addition, the

![Figure 6. Typical Plot of Dynamic, In-situ Static and Normal Hydrostatic Pore Pressures from CPT Probing in Drained Tailings](image-url)
insitu static pore pressures are considerably less than normal hydrostatic pressures. This information is helpful for two reasons: first, the results indicate that the underdrain system is functioning properly, and second, the effective overburden pressures are notably higher since the tailings are not, in essence, fully submerged.

6. CONCLUSIONS

Through the experience gained with nearly twenty geotechnical investigations at tailings facilities, the authors have amassed considerable data pertaining to the static and dynamic behavior of mine tailings. The CPT has been used to provide essential information on various properties, most importantly, including the potential for liquefaction.

The CPT, while capable of providing excellent and reasonable information to the investigator, should be regarded with the necessary amount of caution, and thoughtful logic is needed when interpreting data obtained in tailings.

Since typical tailings deposits are loose, saturated and fine-grained (but non-cohesive) the potential for liquefaction should be regarded as a serious matter.

When applying correction factors to data obtained in mine tailings, especially in estimating the cyclic stress ratio, one should approach published correlations in a logical fashion. This statement is especially true when extrapolation is necessary.

The continued use of the CPT to evaluate the potential for liquefaction should be encouraged. The advantages of this tool, especially when used in tailings deposits, are numerous, and when coupled with reasonable interpretation of the results, this data may be used to establish a very reasonable liquefaction analysis.

Once sufficient information is gathered to generate a true CPT-based liquefaction analysis, it is quite likely that the use of the SPT, as a tool for predicting liquefaction, may become antiquated.

7. REFERENCES


Profiling Mine Tailings With CPT

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SYNOPSIS: Three CPT soundings have been performed to depths of 25 m in the tailings beach of an active copper mine tailings dam constructed by centerline on-dam cycloning. Pore pressure dissipation tests were conducted to interpret horizontal hydraulic conductivity of the tailings. A possible correlation between the normalized cone tip resistance and SPT blow count is investigated. Interpretations of the effective stress friction angle and undrained shear strength are provided for use in slope stability analysis.

1. INTRODUCTION
The results of three piezocene penetration tests (PCPT) conducted in the tailings beach of a copper mine tailings dam are presented. Access to the soft tailings beach was made possible by the construction of a 3.6 m-thick sandfill ramp extending 180 m into the impoundment over a geotextile separation layer. PCPT soundings were conducted at distances of 60, 120, and 180 m from the crest toward the impoundment. Adjacent to each of the PCPT soundings, hollow stem auger drilling was performed to a depth of 40 m using a CME–75 truck-mounted drill rig. Standard penetration testing (SPT) with a CME automatic hammer, open-end drive sampling, and recovery of 168-mm HSA cuttings were performed at selected intervals in the borings.

PCPT were performed according to ASTM D3441–86 Standard using an electric type 2 piezocene with the porous element located behind the tip. The cone was a standard Fugro-type design manufactured by Hogeentogler, having a 60° apex, 10 cm² projected area at the tip, and 150 cm² sleeve. The cone was advanced at a rate of approximately 2 cm/s using EW drill rods and a CME–75 truck-mounted drill rig. The porous element was made of high-density polypropylene, and was saturated in a vacuum chamber using a 50/50 glycerine/water mixture prior to testing. Depth increments for each rod were recorded at the surface using an ultrasonic sensor.

2. SOIL CHARACTERISTICS
The whole tailings slurry stream is approximately 36 to 38 percent solid material (by weight). The current process of cyclone separation of the whole tailings yields about 45 percent tailings underflow (cycloned sands) and about 55 percent overflow (silt-sized particles referred to as slimes).

The sand portion of the tailings is discharged onto the embankment crest, enabling the sands to flow down the embankment slope as excess construction water from the freshly-deposited sands drains through the embankment. The slimes portion of the tailings is discharged onto the tailings beach, which slopes gradually toward the pond located approximately 1800 m upstream of the crest.

Even though two different ore types are mined for copper production, the resulting tailings differ little in fines content and relevant strength parameters. The specific grav-
ity obtained from laboratory tests on whole tailings, cycled overflow, and cycled underflow ranged from 2.66 to 2.72. Figure 1 shows the profiles of unit weight, water content, and fines content obtained from tailings beach samples at various distances from the crest. For the soundings located at 120 and 180 m from the crest, an assumption of hydrostatic conditions below a depth of 3.05 m and a saturated tailings unit weight of 19.6 kN/m³ were used in the calculations described in Sections 3 through 6.

3. TEST RESULTS
The profiles of cone tip resistance (q_c), sleeve friction (J_s), and penetration pore water pressure behind the tip (w_s) are shown in Figure 2. In soft soils, it is important to correct the measured q_c because the pore pressure acts on unequal areas behind the joint of the cone tip. However, at this site, the relatively high values of q_s and small recorded w_s indicate that the corrected cone tip resistance (q_c) is, for all practical purposes, equal to the measured resistance (q_c). This response is characteristic of sandy and silty soils that have moderate to high permeability.

Recent soil classification and interpretation methods for PCPT data can be found in Kulhawy and Maeye (1990), Robertson (1991), and Jeffries and Davies (1993). Classification plots using normalized cone resistance parameters according to Robertson (1991) are shown in Figure 3. These graphs indicate the materials to be sand mixtures, silt mixtures, and clays to silty clays, with evidence of normally-consolidated trends.

During piezocene penetration in the sounding located 180 m from the crest, pore pressure dissipation tests were performed at three depths by maintaining the applied thrust on the rods while the pore pressure decay versus time was recorded, as shown in Figure 4. Methods for evaluating the in situ horizontal hydraulic conductivity (k_h) from the interpolated time required for 50 percent dissipation (t_50) are available (e.g., Leroueil and Jamiołkowski, 1991). The calculated values of k_h were 1.34 · 10⁻⁴ cm/s at a depth of 10.0 m, 1.07 · 10⁻⁴ cm/s at a depth of 13.8 m, and 5.63 · 10⁻⁵ cm/s at a depth of 18.8 m. The interpretation of k_h at a depth of 18.8 m is demonstrated in Figure 4. The calculated values are consistent with published data on copper mine tailings (e.g., Mittal and Morgenstern, 1976; Volpe, 1979).

4. CORRELATION WITH SPT
Numerous studies investigating a possible correlation between the cone tip resistance and SPT blow count corrected to 60 percent of the theoretical hammer energy (N_60) are available (e.g., Robertson, et al., 1992; Jeffries and Davies, 1993). For clean sands, the relationship suggested by Robertson, et al. (1992) is as follows:

\[
q_c/(N_60) = 0.5
\]

where \( q_c = q_c C_N \), \( (N_60) = N_60 C_N \), and \( C_N = \sqrt{p_s/\sigma'_{so}} \), \( p_s \) = atmospheric pressure, and \( \sigma'_{so} \) = effective vertical overburden stress. According to Jeffries and Davies (1993), for sand mixtures (silty sand to sandy silt) and sands (clean sand to silty sand), \( q_c/(N_60) \) ranges from 0.4 to 0.63. A good correlation between the measured and predicted normalized SPT blow count was obtained using Equation 1, as evident in Figure 5.

5. SHEAR STRENGTH
Because of particle angularity resulting from crushing and milling, copper mine tailings have a relatively high effective stress friction angle. It is generally accepted (e.g., Volpe, 1979; Vick, 1983) that copper tailings have a zero effective stress cohesion (c' = 0), whereas the effective stress friction angle (\( \phi' \)) varies within a narrow range of 33 to 37 degrees. According to Kulhawy and Maeye (1990), the effective stress friction angle can be interpreted from the cone tip resistance as follows:

\[
\phi' = 17.6° + 11 \log \frac{q_s/p_s}{\sigma'_{so}/p_s}
\]

Another correlation was suggested by Robertson and Campanella (1985):

\[
\phi' = \arctan \left( 0.1 + 0.38 \log \frac{q_s}{\sigma'_{so}} \right)
\]
Figure 1. Profiles of Unit Weight, Water Content, and Fines Content

Figure 2. Typical Results of CPT Soundings
Figure 3. Soil Classification for CPT Sounding at 180m From Crest

Figure 4. Pore Pressure Dissipation Test
Figure 5. Measured and Predicted SPT Blow Count

Figure 6. Estimates of Drained and Undrained Shear Strengths

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The pore pressure measurements for the sounding at 60 m from the crest were negligible for all practical purposes. Consequently, this test was considered to be drained, and $q_c$ was used to interpret $q_v$. As can be seen from Figure 6, the values of $q_v$ interpreted using Equations 2 and 3 compare well with published values for copper mine tailings.

If the permeability of the soil is relatively low, the cone readings will reflect undrained conditions at a standard penetration rate of 2 cm/s. The undrained shear strength ($s_u$) is indicative of the short-term strength of clays and cohesive silts. The cone tip resistance is traditionally used to evaluate the peak undrained shear strength, as follows:

$$s_u = \frac{q_c - \sigma'_{w}}{N_k}$$

(4)

where $\sigma'_{w}$ = total overburden stress and $N_k$ = cone bearing factor, often taken as 15 for the average strength. Using this approach, the profiles of $s_u$ were established for the soundings located at 120 and 180 m from the crest, as shown in Figure 6. Also shown in Figure 6 are profiles of $s_u$ normalized by $\sigma'_{w}$. The estimated lower bound value of $s_u/\sigma'_{w} = 0.2$ is also recommended by Ladd (1991) for undrained strength analysis (USA) of normally-consolidated soils.

6. CONCLUSIONS

PCPT-based estimates of in situ horizontal hydraulic conductivity of the copper mine tailings compare favorably with published data. The SPT blow count can be estimated from PCPT data as $(N_s)_0 = 2q_c$ (MPa). For slope stability analysis, the strength of the cycloned embankment shell can be characterized by $q_v$ provided that the shell is drained. The estimate of $s_u$ in the tailings beach (Figure 6) represents a peak value than can be notably greater than the residual (or steady-state) shear strength ($s_r$), which is appropriate for static liquefaction triggering or post-earthquake stability analysis. Peak $s_u$ may be used for stability analysis as an intermediate case where there is no risk of liquefaction triggering and deformations are anticipated to be sufficiently low.

7. REFERENCES


Comparison of 3 methods for assessing the capacity of steel H-piles from CPT test results.

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SYNOPSIS: This paper compares the main features of 3 methods for assessing the compressive axial capacity of single, impact driven steel H-piles from the results of CPT tests. Topics like safety factors, pile group behaviour or pile head movement predictions are not discussed within this paper.

The first method has been proposed in Belgium by Prof. de Beer and is based on extensive experimental research in sandy and clayey soils. The second method is described by the French code Fascicule 62-Titre V and is used in France by the Highway Administration for the foundation design of all public works from a civil engineering type. The third method is given by the Dutch code NEN 6742, which is valid for the assessment of the maximum capacity of axially loaded compression piles in the Netherlands. The considered procedures present simple models for a rapid determination of the pile capacity. In view of the differences or, in some cases, the lack in basic design assumptions, the 3 methods may lead to different results. However, the calculated capacities can typically be considered as safe assessments.

1. INTRODUCTION

Due to the intricate shape of steel H-piles, uncertainty exists at the design stage when assessing the base and skin areas capable of transferring the load from the pile into the soil. Although the H-shape itself is fixed prior to installation, it is generally not known beforehand whether or not a plug is formed between the flanges during driving in granular soils or whether or not adherence occurs after some time of wait in cohesive soils. Also, correlation factors are normally proposed in literature for deriving the unit skin friction and the unit base resistance of full displacement piles. Steel H-piles are however small displacement piles which need specific empirical relationships for taking into account the method of installation, the shape and dimensions of the pile and the nature of its skin.

In order to improve the knowledge about the load transfer from the steel H-piles into the soil, 2 major test campaigns, partially sponsored by the European Community (ECSC final reports (1981),(1987)), have been performed in sandy and clayey soils during the years 1980-1987. From these tests, it was possible to derive a comprehensive design method for H-piles based on CPT test results (mechanical cone M4). The procedure is described by De Beer, Scholtes and Carpentier (1982), De Beer (1985), De Beer, Lousberg and Weber (1989).

Part of the data for designing H-piles from CPT results of the French Fascicule 62-Titre V (1993) is also based on above mentioned tests. The Dutch NEN 6743 (1991) standard is not yet adjusted to H-piles; however, general indications for designing full displacement steel piles are given. This standard refers to CPT test results from the electrical cone.

In the following, no correction is performed between $q_s$ values obtained with a mechanical or an electrical cone when they are applied to the different methods.

The purpose of this paper is not to present in detail the 3 design methods, but to compare their main features for assessing the point resistance and the mantle friction of steel H-piles loaded axially in compression.
2. BASIC DESIGN FEATURES
Each of the considered design methods completely separates the base resistance and the skin friction, although there is an interaction in the soil between the two due to the tridimensional state of stresses around the pile toe. Each method gives specific indications for assessing the unit rupture loads below and along a driven full displacement steel pile. The subsequent application of empirical correction factors then allows to consider low displacement H-piles. Specific proposals concerning the assumptions for the H-pile shape are given by de Beer and Fascicule 62-V; de Beer and NEN 6743 also allow to take pile or toe enlargements into account.

2.1 Determination of the H-pile shape
In the de Beer method, two different sets of assumptions concerning the pile shape lead for each design to two successive calculations. In the first case, it is assumed that the H-pile penetrates the soil like a knife. For this hypothesis, the resistance is mobilised only by the steel area at the base and by the full perimeter of the H-section. In the second case, it is assumed that during driving in granular layers, compacted soil partially fills over a height of 10 equivalent pile diameters \( D_i \), the space between the flanges of the H-section. In cohesive soils, the soil plug is assumed to fill the whole volume between the flanges due to adherence after some set-up time. The two assumptions are shown in Figure 1. For the sake of safety, only the lowest value obtained from the two computations will be used in the final pile design. For discussion purposes, the computed results following both assumptions will be shown in this paper. The de Beer method also allows to consider enlarged pile shapes or laggings at any level in the soil.

The assumption concerning the H-pile shape following Fascicule 62-V is shown in Figure 2. Here, the total area of height times width is considered at the base whereas the total perimeter of the H-section is taken as the friction area. Both areas are affected by reduction factors. It is clear here that the rupture models for both the base and the mantle can not occur at the same time in the soil. The assumptions have however been chosen for the ease of computation. Adjustment factors are introduced for obtaining realistic results. No indications are given for considering laggings.
NEN 6743 does not provide any precise indications concerning the shape of an H-pile. Basically, NEN 6743 enables the design of full displacement steel piles and allows to take into account an enlarged pile base. It is up to the designer to decide upon the rupture model in the soil along the H-section. For the purpose of this paper, the two most extreme possibilities are taken into consideration. They correspond to the assumptions 1 and 2 following de Beer for cohesive soils (without and with full plug formation; see Figure 1).

2.2 Determination of the base resistance
The unit rupture load at the base of a pile at a given depth largely depends on the dimensions of the base. Thus, there is generally a marked scale effect. The 3 considered methods have different approaches for taking into account the difference in pile and CPT cone diameters. The details can be found in literature.

After the calculation of the unit rupture load below the base of a full displacement pile, the 3 methods introduce correction factors depending on the shape of the bearing surface of the H-section. A comparison of these factors is shown in Figure 3.

NEN 6743 also introduces an upper limit of the unit base resistance in sand. An additional reduction factor is introduced by de Beer and NEN 6743 in case of a toe enlargement. Without entering into details here, NEN 6743 does not consider a reduction in base resistance when the height of the enlargement is greater than 2 equivalent diameters $D_e$ of the pile whereas de Beer requests this height to be at least 3 $D_e$. Fascicle 62-V does not provide any indication on this topic.

2.3 Determination of the skin friction
The 3 considered methods propose empirical correlation factors for determining the unit skin friction from the $q_u$ values. A comparison of these factors is shown in Figure 4 for granular soils and in Figure 5 for cohesive soils.

Only de Beer proposes in addition a possibility to use the total measured friction on the sleeve of the mechanical CPT apparatus for deriving the unit skin friction. It is felt that the integration of the local friction, measured in some electrical CPT procedures, would result in predicting unrealistically high resistances.

In case of soil compaction between the pile flanges, de Beer provides supplementary information for considering the friction.
reduction above the plug, which is assumed to have a height of 10 $D_c$.

2. COMPARISON OF CALCULATED PILE CAPACITIES
In the previous paragraph, separate features of the three considered calculation methods have been directly compared. It is obvious that there are substantial differences between individual factors from the 3 methods, resulting in different resistance distributions along the pile.

The purpose of each design method is the assessment of the bearing capacity of the piles. Herefore, the different assumptions are linked to one another. In the following, a comparison of the capacities, determined according to the methods and assumptions described in the discussion, is shown for a uniform and an enlarged H-pile in granular soils. The chosen examples refer to well documented test piles.

3.1 Test pile KP1 at Kallo near Antwerp (Belgium)
The test pile has a uniform shape consisting of the original section HP 400x213kg/m (steel yield strength = 355 MPa) and with 4 small diameter tubes and 2 small U-shapes which are fixed between the flanges for measurement purposes. A typical section of the pile is shown in Figure 6. The 16m long pile was driven to a depth of 14m and penetrated ~8m into a dense quartz sand layer.

![Figure 6. Typical section of pile KP1 at Kallo.](image)

The $q_c$ diagram and the computed unit base resistances following the 3 considered methods are shown Figure 7. The conventional rupture load, determined during the load test at a settlement of the pile base of 0.1 $D_c$, was reached between 3.25 - 3.5 MN. The results of the capacity calculations are shown in Figure 8.

![Figure 7. Typical CPT diagram and calculated unit base resistances of pile KP1 at Kallo.](image)

![Figure 8. Comparison of the computed bearing capacities of pile KP1 at Kallo.](image)
3.2 Test pile P3 at the Maasvlakte in the Port of Rotterdam (Netherlands).

The test pile consists of a 31m long section HP320×147kg/m (steel HISTAR 460; yield strength = 460MPa) with a lagging of 1.8m in height, which corresponds to 3D. This toe enlargement is formed by 2 H-segments attached laterally to the flanges of the main element. The lower part of the lagging is closed by a steel plate. Details of the pile enlargement are shown in Figure 9. The pile was driven to a depth of 30 m (level NAP ~25m), penetrating ~4m into a very dense sand layer. The q_u diagram and the computed unit base resistances following the 3 considered methods are shown in Figure 10. The pile was test loaded to the maximum capacity of the installation (6.1MN) without reaching failure. A Static test indicated a maximum load of 7.5 MN and a static resistance of 7.19 MN. The results of the capacity calculations are shown in Figure 11.

Figure 9. Detail of the pile toe enlargement of pile P3 at Maasvlakte.

Figure 10. Typical CPT diagram and calculated unit base resistances of pile P3 at Maasvlakte.

Figure 11. Comparison of the computed bearing capacities of pile P3 at Maasvlakte.
4. CONCLUSIONS
- Steel H-sections are small displacement piles and have an intricate shape. Depending on the type of soil, partial or total soil plugging may occur between the flanges during driving or some time after installation. Especially for bare H-piles, it is important to take this plugging effect into consideration in order to achieve reliable assessments of the bearing capacity and the resistance distribution along the pile.
- The de Beer method allows to consider in great detail soil plugs and pile enlargements of steel H-piles. It is important to note that for each pile, 2 possible rupture models without and with formation of a plug have to be analysed. For the sake of safety, the calculation giving the lowest result will be used in the design. The computations are preferably done by computer. The method leads to reasonably accurate assessments of the H-pile capacity.
- Fascicule 62-V presents an easy to use method with simplified assumptions concerning the H-pile shape. The method leads in general to safe H-pile capacity assessments. In some cases however, the results are too conservative.
- The design of steel piles is possible within NEN 6743; however, specific recommendations concerning the rupture model of the soil around a steel H-pile are not yet included. For the time being, it is up to the designer to decide on the appropriate assumptions. Due to upper limitations of the unit base resistance in dense to very dense sands, capacity assessments following NEN 6743 tend to be overly conservative in those cases. Improvements can be made here in combination with proposals for the H-pile shape.
- The validity of the 3 design methods has been checked against the results of compressive load tests on single steel piles in homogeneous granular and cohesive soil conditions. Additional checks are necessary when assessing the capacity of pile groups. Qualified engineering judgement is also required for the evaluation of the behaviour of H-piles in specific cases like, for example, driving into chalks, the resistance to tension forces or when negative skin friction is likely to occur.

5. REFERENCES
Use of CPT tests in very soft soils

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SYNOPSIS: CPT tests are normally performed using probes with relatively wide measuring ranges which are intended to cover a wide range of soils. In Sweden, where deep deposits of very soft soils are common, probes and procedures designed to meet the special requirements in this type of soil are often used. In the paper, an example is given describing how a "clay probe" was used in an investigation of a landslide in soft clay. Experience from this investigation has led to extensive use of this type of equipment in the subsequent large investigations on slope stability along the Göta River.

1. INTRODUCTION
At the beginning of the 90s, standards for equipment and test procedures together with recommendations for interpretation of test results specially adapted to Swedish soft soils were developed. This has led to the use of CPT tests also in investigation of soft or very soft clays, in practical engineering as well as in research.

In April 1993, the occurrence of a landslide at Ånesberg on the east bank of the Göta River a few miles northeast of Gothenburg, called for thorough investigations in the area in order to clarify the conditions causing the slide and the extent of precautions needed to stop further progression of slides in the area. In conjunction with other more conventional test methods, it was decided also to make use of the CPT test. The main purpose of performing CPT tests was to detect whether thin layers of more permeable soil, silt or sand, exist in the clay profile and also to obtain a continuous picture of the variation in shear strength in different clay layers.

2. GEOTECHNICAL CONDITIONS IN THE TEST AREA
The topography in the area is characterised by a gentle slope down towards the river. Outside the shoreline, the water depth increases slowly from about 1 m to about 3 m, forming a 20-25 m wide shallow shelf, outside which a steep slope takes the river to a depth of 8 m in the main fairway, Figure 1.

At the landslide area, the clay layer is about 35 m thick. Below the clay, thick deposits of friction material can be found. The ground water pressure in the aquifer is artesian, and corresponds to a free ground water level 6 to 8 m above the surface of the Göta River.

The uppermost part of the clay layer, about 13 m, consists of grey, highly plastic clay with shells and organic matter. It lies on a grey plastic clay with frequent sulphide spots. A few inclusions of other materials can be found. The clay was deposited in a marine environment. Mainly because of the artesian ground water pressure in the area, the clay has been leached and is very sensitive.

The geology and geotechnical conditions in the area have been documented in detail by Larsson et al. (1994).
2.1 Field investigations and laboratory tests

Comprehensive geotechnical investigations were carried out in and around the landslide area. Figure 2 shows the positions of the CPT tests that were carried out with a clay probe (test class CPT 3) in the area, together with the other boreholes used for evaluation of soil characteristics.

The first CPT tests were performed at two points 25 m and 50 m behind the backslope of the slide, boreholes SGI 1 and G1 respectively, and at one point 25 m out in the water about 30 m north of the slide, borehole SGI 2. At these points, undisturbed samples were also taken and routine tests as well as oedometer tests were carried out in the laboratory. Figure 3 shows results from routine investigations of soil from SGI 1 and G1, together with the total cone resistance in the CPT test at SGI 1. Sampling at G1 was performed at every metre of the soil profile down to firm bottom, whereas sampling at SGI 1 and SGI 2 was performed more sparsely down to 25 m depth. Vane tests were performed at G1 and SGI 2.

In evaluating the general picture of the properties of the soil in the area, results from other boreholes were also used, especially those of C2, 20 m out in the water about 120 m south of the slide area, and of BAB2 at the upper boundary of the slide area where investigations had been carried out prior to the landslide.
3. RESULTS FROM CPT TESTS

3.1 Soil strata

At borehole SGI 1, two CPT tests were. To investigate the uppermost part of the soil profile, a first test was driven through a 1 m thick fill and the underlying dry crust on top of the clay, down to a depth of 3 m. Since results from testing in soft clay can be greatly affected by zero shifts and hysteresis effects at penetration through dry crust, this test was followed by a second test through the deeper soft soil layers, starting at 2 m depth in a pre-drilled hole.

Figure 4 shows the results from this test, as presented by using the CONRAD programme. CONRAD has been developed at the Swedish Geotechnical Institute for processing and presenting of CPT data and also, as discussed below, for evaluating soil parameters (Larsson, 1993).

Only indications of possible very thin layers of somewhat coarser materials could be detected. The same applied to the results from the other boreholes. The indications did not appear in any specific order with respect to depth below ground surface. It was concluded that no layer of this kind exists, which could have caused propagation of high pore pressures and thereby affected the stability of the slope.

At borehole SGI 2, the water depth is almost 4 m. The upper 5 m of the soil profile here is made up of silty sediments deposited from a brook just north of the slide area. The landslide stopped at the border to this area with silty sediments.

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3.2 Evaluation of undrained shear strength

In order to describe the way in which the shear strength varies with depth along the slope, the stress history of the soil in the area had to be established. This encompassed the investigations of prevailing preconsolidation pressures in the soil profile.

A number of oedometer tests (CRS) were run in the laboratory and the preconsolidation pressures evaluated from these tests were compared with empirical values based on vane shear strength and plasticity of the soil according to the Hambo relation and with those evaluated from a limited number of triaxial tests.

The CONRAD programme can be used for an evaluation of preconsolidation pressures based on the net cone resistance, the in situ effective vertical stress and the liquid limit of the soil according to Larsson & Mulabdic (1991)

$$\sigma'_v \approx \frac{q_c - \sigma_{v,ref}}{121 + 4.4 \cdot W_s} \left( 10^{0.54 \cdot \log OCR} \right)$$

The evaluation of preconsolidation pressures from CPT tests is normally expected to give only approximate values. The recommendation is to use results from CPT tests only as a complement to results from oedometer tests.

Figure 5 shows the preconsolidation pressures as evaluated from laboratory tests, empirical relations and CPT tests. A good agreement between the different tests was found. The CPT tests generally gave values of the same size and the same variation with depth. There was a corresponding good agreement between preconsolidation pressures evaluated by different methods for the soil under the river.

A plausible stress history and relating overconsolidation ratios in the area were established, giving an indication also of the probable variation in shear strength along the slope. The drained shear strength was estimated from triaxial tests and empirical relations based on preconsolidation pressure of the soil. The undrained shear strength was determined mainly from results from vane tests, empirical relations.
based on preconsolidation pressure, plasticity and overconsolidation of the soil and results from CPT tests. The undrained shear strength was evaluated from the CPT tests according to Larsson & Muladhic (1991) by means of the CONRAD programme, as

$$\tau_{\text{us}} = \frac{q_1 - \sigma_{\text{vd}}}{13.4 + 6.65 \cdot w_i}$$

Figure 6 shows the undrained shear strengths estimated from different types of tests and empirical relations. There is good agreement between the different results. An exception is the values from fall cone tests at depths below approximately 10 m, which show greater scatter and lower values than the rest. However, this is the normal result of disturbances and stress relief when sampling at greater depths.

The CPT tests gave fully compatible results and more detailed information on the variation in shear strength for the soil profile, especially regarding the lower part where the relative scatter in test results was greater and the values from other tests more scarce. The CPT test also proved advantageous out in the water, where the movements of the test raft affected the vane tests and also to some extent the sampling, whereas this had no noticeable effect on the CPT tests. However, the evaluation from CPT tests is sensitive to the plasticity of the soil. This can be seen at 19 - 23 m depth, where a layer of clay with higher plasticity was found. This was accounted for in the evaluation of shear strength from the CPT test at G1, but not at SGI 1, where the liquid limits were interpolated from levels just above and below this layer.
4. CONCLUSIONS
The good agreement at this site between the results of the CPT tests and the methods normally used for evaluation of soil parameters paved the way for extended use of CPT tests in the valley of the Göta River. As this case and others have shown, there are great potentials in using the method not only in the traditional "CPT-soils" of silt and sand, but also in soft or very soft clays.

However, in order to take full advantage of the CPT method in soft soils, it is important to use accurate probes and strict test procedures. To evaluate parameters such as undrained shear strength, CPT tests should be combined with sampling and characterisation of the soil profile, particularly in respect to the liquid limit of the clays. For evaluation and presentation, a computer programme, such as CONRAD, offers an efficient means of data processing.

Previously, large investigations in soft soils comprised numerous penetration tests, field vane tests, samplings and laboratory tests and often also pore pressure soundings. When using very accurate CPT-probes, the number of supplementary tests can be greatly reduced and the investigations can be performed more economically and with higher quality.

5. REFERENCES
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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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