Geotechnical Site Characterization

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Pressuremeter testing
Effects of pressuremeter geometry on the results of tests in sand

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ABSTRACT: This paper describes an experimental study of the effect of pressuremeter geometry (i.e., length to diameter ratio) on the results of derived soil properties (i.e., soil strength, stiffness and state parameter). Recent numerical studies indicate that ignoring the finite pressuremeter length effects can lead to a non-conservative estimate of soil strength properties. Although some experimental studies have been carried out in the past on the pressuremeter geometry effect, they are not sufficiently comprehensive for reaching a definite conclusion on the effects of pressuremeter geometry on derived soil properties. In this study, a total of 44 self-boring pressuremeter tests with 4 different pressuremeter length to diameter ratios of 5, 10, 15 and 20 have been conducted in a large calibration chamber located at the University of Newcastle, Australia. Based on the results of these chamber tests, the effects of pressuremeter length to diameter ratios on the soil properties derived from pressuremeter loading curves are quantified.

1 INTRODUCTION

Site investigation and evaluation of properties of soil or rock are important aspects of geotechnical design. Laboratory tests have some shortcomings such as disturbance of the sample and this makes laboratory results seriously questionable. For this reason, over the last three decades, in situ tests in soils have been developed rapidly. Robertson (1986) presented a summary of major in situ methods and their applicability. He concluded that, among the existing in situ testing techniques, the self-boring pressuremeter (SBP) is one of the best in situ devices for application in cohesionless soils.

The remarkable advance of the self-boring pressuremeter as a site investigation tool is due to the fact that the boundary conditions and stress-strain conditions around the soil may be properly defined in theory (Wroth and Hughes, 1973). Therefore it is possible to perform a more rigorous theoretical analysis for the self-boring pressuremeter. Another advantage of the self-boring pressuremeter is that the pressuremeter can be inserted into the ground with minimum disturbance. For this reason, the SBP is one of the most reliable and accurate methods of in situ testing and can be used to determine the in situ horizontal stress, shear modulus and strength parameters for both clay and sand.

Since in most existing methods the pressuremeter length is assumed to be infinitely long, the aim of this paper is to assess the possible effect of finite pressuremeter length on pressuremeter loading results. Apart from the numerical studies, Later (1973) and Fahey (1980) have also carried out some brief experimental studies on the finite pressuremeter length effect. However, until the present study, there has been no comprehensive experimental studies that can be used to verify the numerical findings. This experimental study was undertaken to investigate the effect of finite pressuremeter length on derived soil properties from loading pressuremeter tests.

The results of the loading portion of the pressuremeter tests conducted in the calibration chamber are analysed in this paper. The calibration chamber was considered to be a most appropriate option to calibrate and evaluate the pressuremeter test in cohesionless soils, under controlled stress and density conditions. This series of tests has been conducted under various stress, density and geometric conditions. The experimental results are compared with the results of the existing analytical models and also with the laboratory results on the same sand, and these are then used to develop a correlation factor for field tests.

2 METHODS OF INTERPRETATION FOR PRESSUREMETER LOADING TESTS

The first fundamental interpretation of the pressuremeter expansion test was published by Gibson and Anderson (1961). They assumed that the pressuremeter is infinitely long; therefore the surrounding soil is deformed under the conditions of axial symmetry and plane strain. There is good experimental evidence for this basic assumption from
the radiography of laboratory tests in Kaolin reported by Wroth and Hughes (1973). The Gibson and Anderson interpretation was developed for expansion tests in both cohesion and cohesionless soils. It is assumed that the soil is sheared at a constant effective stress ratio with zero volume change (e.g., Palmer, 1972 and Ladanyi, 1972). This interpretation is suitable mainly for soil with low permeability such as clay. Since cohesionless soils (sand) have high permeability, this assumption is not reasonable because the sample can drain quickly during the pressuremeter test. Therefore, the amount of volume change is considerable and should be taken into account.

Several attempts have been made to include realistic volume change laws in the solution of the expansion of a cylindrical cavity (Fello and Briand, 1986). Ladanyi (1963) analysed pressuremeter tests in the same way as Gibson and Anderson (1961) except that he considered the volumetric strain to failure. Other analyses were also proposed by Bagnold and Jereczek (1973), Prevost and Hogg (1975) and Jakubicki and Lancellotti (1972). Windle and Wroth (1975) developed a method to consider the volume change of a drained pressuremeter test. Windle (1976) concluded that the assumption of zero dilation in the plastic region causes an overestimation of friction angle value for dense sands and an underestimation for loose sands.

Hughes, Wroth and Windle (1977) proposed a method to analyse the self-boring pressuremeter test in dense sand. They modified Gibson and Anderson’s analysis to account for the dilation of soil. The conventional Hughes et al. method for interpretation of pressuremeter test results in sand was useful for deriving the fundamental soil parameters. This analysis was confirmed by a series of calibration chamber pressuremeter tests in dense sand by Fahey (1990). In the Hughes et al. (1977) method, it is assumed that the rate of volume change is constant during cavity expansion and depends on the peak angle of shearing resistance using the concept of stress dilatancy. This means that both friction angle and dilation angle are constant during the pressuremeter test. This assumption was supported by a large number of plane strain shear tests on dense Leighton Buzzard sand in the simple shear apparatus (Sloane et al., 1971). Hughes et al.’s (1977) method is found to be reliable for very dense sands, but less reliable for loose sands.

By using Rowe’s (1972) stress-dilatancy flow rule, Hughes et al.’s (1977) suggested that the values of $\phi$ and $n$ can be directly deduced from the value of pressuremeter loading slope, $S_d$:

$$\sin G_p = \frac{\left[(K+1)S_d - (K-1)S_d^{1.2}\right]}{\sin 2(KS_d^{1.2} - (K-1))}$$

(1)

$$K = \frac{1 + \sin \phi}{1 - \sin \phi}$$

(2)

where $\phi$ is the friction angle at constant volume.

Later Manareso (1980) developed a drained analysis in which he proposed a numerical method based on Rowe’s (1972) stress-dilatancy theory. Similar to the previous method, it also assumed that elastic strains are negligible. This method can be used to produce a complete stress-strain curve for tests in loose to dense sands. By using this method, the peak friction angle of soil can be deduced directly from the stress-strain curve.

Until fairly recently the analysis of cavity expansion problems has been mainly based upon elastic-perfectly plastic soil models. A new analysis of self-boring pressuremeter tests in sand has been developed by Yu (1994), using an elastic-plastic strain hardening-softening model. Yu’s (1994) method is based on the state parameter introduced by Wroth and Bassett (1965) and Been and Jefferies (1985). They defined the state parameter as the difference between initial void ratio and the void ratio at the critical state at the same mean effective stress (more information has been given by Ajalloeian and Yu, 1996). Unlike last methods which used an elastic-perfectly plastic model, Yu’s method is based on a strain-hardening (or softening) plasticity model in which the angle of friction and dilation are assumed to be a function of the state parameter. According to this method, the in situ state parameter of sand can be determined from the results of a self-boring pressuremeter test:

$$\xi_e = 0.59 - 1.85 (S_d)_0$$

(4)

where $\xi_e$ is the state parameter and $(S_d)_0$ the pressuremeter loading slope in the log-log plot corresponding to $L/D = 6$. Yu (1994) also presented the following average correlation for determining plane strain friction angle ($\phi_p$) from pressuremeter loading slope:

$$\phi_p = 0.6 + 107.8 (S_d)_0$$

(5)

In many interpretation methods of pressuremeter tests, the pressuremeter length is assumed to be infinitely long. By means of a numerical study, Yu (1990) found that the finite pressuremeter length has a significant effect on derived soil properties. Since the geometry of most commercial self-boring pressuremeter devices $L/D = 6$, Yu (1994) considered the influence of pressuremeter length to diameter ratio $L/D = 6$, using a finite element solution presented by Yu (1990):

$$\xi_e = 0.59 - 2.2 (S_d)_0 + 0.107 (S_d)_0 \ln (GP_d)$$

(6)

$$\phi_p = 0.6 + 128.3 (S_d)_0 - 6.25 (S_d)_0 \ln (GP_d)$$

(7)

where $P_i$ is initial mean effective stress, $G$ is shear modulus and $(S_d)_0$ is apparent pressuremeter loading slope measured with a pressuremeter with $L/D = 6$.

3 PRESSUREMETER LOADING RESULTS

The results of the loading portion of the pressuremeter tests are presented in this section to quantify possible effects of finite length of the
pressuremeter on the test results. As explained in previous sections, the results of the loading part can be analyzed using the methods suggested by Hughes et al. (1977), Munsenro (1985) and Yu (1994) to obtain values of the angle of internal friction (2) and the angle of dilation (5).

A total of 44 pressuremeter tests have been performed on silica sand to determine properties of the soil. In this series of tests, the pressuremeter has been positioned in the middle of the chamber before sample preparation. This is an idealisation of the self-boring pressuremeter test, because the sample is tested under undisturbed conditions. Four different pressuremeter length to diameter ratios are used. Therefore, by running this series of tests, it is possible to assess the influence of the finite pressuremeter length on the soil properties derived from the pressuremeter tests. Tests were carried out at mean effective stresses (P) of 50, 100, and 150 kPa and at stress ratios (K=σv/σh) of 0.5, 1.0, and 2.0. The density of the sand was controlled by adjusting the intensity and height from which the sample was poured. A detailed information has been given by Ajallooeian (1996).

In this study, four ratios of pressuremeter length to diameter (L/D=5, 10, 15 and 20) have been used. Typical results are presented in Figures 1, 2 and 3, in which the influence of pressuremeter geometry on pressure-expansion curve is illustrated. These results indicate that as the pressuremeter probe length increases, the derived soil strength decreases. It also indicated that this effect is considerably reduced as L/D values change from 5 to 10.

Figures 1, 2 and 3 has been replotted on log-log scale to demonstrate the effect of L/D on pressuremeter loading slope (Figures 4, 5 and 6 respectively). These Figures show that, the geometry of pressuremeter has a considerable effect on the pressuremeter loading slope, 5d.

Table 1 summarise the pressuremeter results obtained from 44 tests with different L/D ratios. To identify each test, a simple system was adopted. For example, SLK1P100 represents the pressuremeter test with L/D=5, loose density sand (L), the stress ratio of 1 (K1) and the mean effective stress of 100 kPa (P100).
Figure 5 Influence of pressuremeter length to diameter in medium density of sand (log-log scale, K=1, P=50kPa)

Figure 6 Influence of pressuremeter length to diameter in loose density of sand (log-log scale, K=1, P=50kPa)

Table 1 Pressuremeter loading test results in different densities of sand

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<th>No.</th>
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<th>No.</th>
<th>Name of Test</th>
<th>Rd</th>
<th>No.</th>
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<tr>
<td>U453.165</td>
<td>0.7</td>
<td>0.92</td>
<td>U531.165</td>
<td>0.6</td>
<td>0.33</td>
<td>U531.305</td>
<td>0.6</td>
<td>0.35</td>
</tr>
<tr>
<td>U453.315</td>
<td>0.7</td>
<td>0.92</td>
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4 ANALYSIS OF RESULTS

This section discusses the analysis and interpretation of the pressuremeter test data to obtain the strength parameters of sand, such as internal friction angle and dilation angle. The methods of Hughes et al. (1977), Manasse (1989) and Yu (1994) are used in turn. Since infinite pressuremeter length is often assumed in many methods of analysis, the validity of this assumption will be discussed in detail.

Figure 7 shows the relationship between Sd (the apparent pressuremeter loading slope derived from log-log plot) and L/D for loose, medium and dense sands. A clear trend of decreasing Sd with increasing L/D is observed in the figure. As can be seen, the trend of variation of Sd vs. L/D at different relative densities are approximately parallel. As a result, it may be reasonable to conclude that the pressuremeter length effect is independent of the density of soil. This is consistent with the numerical studies carried out by Yu (1990) that suggests the variation of friction angle and hence density has very little influence on finite pressuremeter length effects.

Although some scatter is observed in Figure 7, a linear trend of an increase in the ratio L/D with decreasing pressuremeter loading slope exists. By using this figure, it is possible to estimate the value of (Sd) of that would be obtained from an infinitely long pressuremeter (or D/L→0). The ratio between (Sd) and the derived loading slope for the pressuremeter test corresponding to different length to diameter ratio of 5, 10, 15 and 20 may be used to describe the effects due to finite length of pressuremeter:

\[ [(Sd)_{L/D}=1-D/L] \]  

(8)

Figure 7 Effect of pressuremeter geometry on Sd

By using Hughes et al.'s method (equation 1), this correlation (8) can be rewritten in terms of friction angle:

\[ [(\phi_{pm})/\phi_{pm}]=1.08(D/L) \]  

(9)
Regarding to Figure 7 and equation (8), it is possible to establish the following relationship between \((S_D)\), and relative density:

\[(S_D) = 0.4 (R_d) + 0.19 \quad (10)\]

Yu's method for interpretation of the pressurerimeter test follows the method of Hughes et al in the sense that only the slope of the loading portion of the pressurerimeter test curve is used to derive the soil properties. Therefore, regarding to the effect of \(L/D\) ratio, in order to determine the value of state parameter and friction angle using equations 4 and 5, equation 8 need to be used.

According to Manassero's analysis, a computer program has been written to obtain the complete stress-strain response of silica sand from the pressurerimeter test results. This is achieved by firstly describing the pressurerimeter data with a polynomial function. The peak of the stress-strain relationships curve is used as a peak shear strength. By using this method, the results show that the finite pressurerimeter length has a significant effect on drained strength parameter. According to this method, the following equation can be deduced:

\[
\left(\frac{\phi_D}{\phi}_{2D}\right) = 1 - 0.82(D/L) \quad (11)
\]

Interpretation of the results using the Manassero method shows that the value of peak friction angle is dependent on pressurerimeter length with \(L/D\) ratios up to 20. The pressurerimeter with \(L/D\) ratio greater than 20, \(L/D\) ratio has a very small effect on derived peak friction angle.

5 COMPARISON OF PRESSUREMETER TEST RESULTS WITH LABORATORY RESULTS

The triaxial test has been used to investigate the shear strength behaviour of silica sand to compare the results with those of the calibration chamber tests. Triaxial tests consisted of CID and CIU tests on sand at different densities and confining pressures. It was concluded that the strength parameters depend on the values of the initial density and the mean effective stress. Based on Bolton's (1985) empirical equation, the following experimental equation can be used to match the results for silica sand (Ajiakeo et al, 1990):

\[\phi_{2D} = \phi_D + 3(R_d(11.7-\ln p)-1) \quad (12)\]

Plane strain friction angles derived from above mentioned three methods are compared with triaxial results that were obtained from laboratory results. To facilitate comparison between the results deduced from different methods and those of the triaxial tests, the triaxial friction angles have been converted to plane strain friction angle, using the equation suggested by Wroth (1984):

\[\phi_p = \frac{9}{8} \phi_D \quad (13)\]

To compare the three interpretation methods of Hughes et al (1977), Manassero (1989) and Yu (1994), the pressurerimeter test results obtained from \(L/D\)-0 have been analysed. Figure 8 presents the plane strain friction angles derived from these three interpretation methods by using the results of an infinitely long pressurerimeter. Plotted in the same figure are the values of plane strain friction angles converted from triaxial testing.

![Figure 8: Comparison pressurerimeter test results with triaxial test results on silica sand](image)

It is interesting to see that the friction angles obtained from Yu's method are compared very well with those converted from triaxial tests. This is because Yu's interpretation method has properly taken into account the effects of both elastic deformation and strain hardening/softening. Because of failing to take into account of elastic deformation, Manassero's method is found to underestimate the laboratory friction angle slightly. A more significant underestimation of the laboratory friction angle is obtained when Hughes et al's method is used in the interpretation, and this is because in the method of Hughes et al both elastic deformation in the plastically deforming region and strain hardening/softening of the soil have been ignored. These experimental findings are in agreement with the finite element study carried out by Yu (1990, 1993).

6 CONCLUSION

Most of the methods for interpretation of pressurerimeter test results are based on the cylindrical cavity expansion theory which involves important assumptions about material behaviour and pressurerimeter geometry. In most existing methods, the pressurerimeter length is assumed to be infinitely long. This paper has assessed the possible effect of finite pressurerimeter length on pressurerimeter loading test results using current interpretation methods. In agreement with the previous numerical studies, the present experimental study suggests that the
The relationship between Sd (derived from pressuremeter loading slope on a log-log scale) and the different ratio of L/D at various densities was verified. The overestimation factor for Sd is approximately 1.25 with L/D=5. By increasing the L/D ratio, the overestimation factor decreases. If the L/D ratio increases to 20, this factor will be approximately 1.05. It was also concluded that the trend of variation of Sd against L/D is largely independent of the density of soil.

REFERENCES


New design rules for the bearing capacity of shallow foundations based on Menard pressuremeter tests

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F. Baguelin – Terrassol, Montereuil, France
Y. Canéda – Laboratoire Régional de l’Est Parisien, Melun, France
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ABSTRACT: The Laboratoires des Ponts et Chaussées have conducted over 100 full scale loading tests of shallow foundations and as many tests on centrifugal models. This article presents the main results obtained which have been used to develop France’s new design rules (Pasciutti 62, 1993).

1. INTRODUCTION

In 1993, the French Public Works administration issued new technical rules governing the design and computation of structural foundations applicable to all public works contract awards (Pasciutti 62, 1993).

The purpose of this document was to update the previously set of rules prescribed in the document entitled FOND 72 (1972) which contained, in particular, the rules specified by Ménard (1963) which relied on the results of a few full-scale experimental tests.

In the aim of updating the FOND 72’s rules, the Laboratoires des Ponts et Chaussées (LPC) have undertaken a major research program on the behaviour of shallow foundations.

Some one hundred full-scale loading tests, divided among 6 sites representing 4 different types of soils (silt, clay, sand and chalk), were carried out.

Different types of loadings were performed (short and long term static loads, cyclic loads and several configurations were tested (horizontal, vertical, and incline loads, foundation at the crest of slopes).

In order to study specific parameters, these tests were complemented by several series of tests on centrifugal model-scale.

This article describes the entire set of tests conducted and displays the results which were used for the new design rules prescribed in Pasciutti 62 (1993).

2. EXPERIMENTAL SITES AND CONDITIONS

2.1 Description of soils

A geotechnical investigation as complete as possible was made on each experimental site. The different soils were characterised by means of the most commonly field and laboratory tests.

<table>
<thead>
<tr>
<th>Site</th>
<th>Soil</th>
<th>W (%)</th>
<th>W_L</th>
<th>I_p</th>
<th>u_l kN/m³</th>
<th>Eaturity</th>
<th>c’ kPa</th>
<th>c” kPa</th>
<th>φ°</th>
<th>f_kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plaisir</td>
<td>Silt clay</td>
<td>37</td>
<td>35</td>
<td>16</td>
<td>13.0</td>
<td>2040</td>
<td>46</td>
<td>0</td>
<td>34</td>
<td>280</td>
</tr>
<tr>
<td>Jossigny</td>
<td>Silt</td>
<td>22</td>
<td>38</td>
<td>14</td>
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<td>6500</td>
<td>38</td>
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<td>32</td>
<td>510</td>
</tr>
<tr>
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<td>85</td>
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<td>12000</td>
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<tr>
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<td></td>
<td></td>
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<td>8400</td>
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<td>–</td>
<td>1290</td>
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<td>Chalk</td>
<td>27</td>
<td></td>
<td></td>
<td>15.5</td>
<td>19500</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>1350</td>
</tr>
</tbody>
</table>

727
Precaution was taken to systematically carry out a few in-situ measurements of the state of soil (water content, ...) during each series of loading tests.

Table 1 summarizes the identification parameters as well as the average characteristics measured at each of three experimental sites down to a depth of 3 meters, which constitutes the portion of ground concerned by the foundation tests.

2.2 The different types of loading

Different types of loading were performed within this research program. Short-term tests up to failure (R), cyclic-load tests (C), both short-term and long-term creep tests under constant load (F<sub>R</sub>) and (F<sub>C</sub>). Two experimental set-ups, described by Anvar et al. (1987), have been used in order to conduct these tests.

In this article, only the (R) type tests will be discussed, in which the foundation is loaded in steps up to the soil's failure, with each loading step being held constant until the measurements have stabilized or, if not, for a period of at least 30 minutes. Figure 1 shows, as a sample case, the settlement curves obtained from this type of test.

![Typical loading curves](image)

Figure 1: Typical loading curves: 
(a) settlement versus applied load; 
(b) settlement versus time under constant load.

2.3 The different configurations

The influence of a variety of factors on the behaviour of foundations was also investigated relative to embedment D/B, shape L/B, etc. Table 2 illustrates and summarizes the number of tests carried out at each site by type of loading and primary parameters investigated.

Nonetheless, this article will only report the results of on-site tests dealing with the eccentricity (E/B) and the inclination (θ) of loads (Q), as well as the centrifuged-model tests carried out to study the effect of slope (β) on the failure pressure.

Table 2: Type of loading and type of parameter being investigated (number of tests)

<table>
<thead>
<tr>
<th>Site</th>
<th>D/B</th>
<th>L/B</th>
<th>E/B</th>
<th>θ</th>
<th>β</th>
</tr>
</thead>
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<td>-</td>
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<td>-</td>
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<td>3</td>
<td>6</td>
<td>3</td>
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<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Lamberge</td>
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<td>-</td>
<td>3</td>
<td>4</td>
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<tr>
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<td>-</td>
<td>-</td>
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<td>-</td>
</tr>
<tr>
<td>Changy</td>
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<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
<td>Nantes&lt;sup&gt;(10)&lt;/sup&gt;</td>
<td>-</td>
<td>-</td>
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<td>&gt;100</td>
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</table>

<sup>(10)</sup> Tests on centrifuged models.

3. OBSERVED BEHAVIOUR AND RESULTS

3.1 The failure criteria

Loading curves (Fig. 1) in general do not contain any sharp break that would make it possible to determine the load at failure Qr without ambiguity. Furthermore, for some configurations (inclined loads, crest of slope), an important horizontal displacement may have occurred.

We have therefore defined Qr as the lower of the following two values:
- the load Qr, corresponding to a settlement s<sub>r</sub> of the application point of the load equal to 10% of B, and
- the load Q<sub>r</sub> corresponding to an horizontal displacement u<sub>r</sub> of the application point of the load equal to 1% of B.

It is to be noted that in the special case of vertical centered or eccentric loads, Q<sub>r</sub> is thus the load corresponding to a vertical settlement of the application point of the load equal to 0.1 B.

3.2 Centered vertical loads tests

3.2.1 Observed behaviour

Figure 1 shows typical loading curves during a test conducted until failure. It can be observed that the settlements for a given increment increase linearly with the logarithm of time and that the slopes A<sub>r</sub> of the stabilization regression lines obtained increase regularly with the applied load Q.
Table 3: Experimental pressuremeter factors $k_p$ - Labeuse site: sand

<table>
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<tr>
<th>No</th>
<th>Test</th>
<th>$B$ (m)</th>
<th>$L$ (m)</th>
<th>$b$ (m)</th>
<th>$D$ (m)</th>
<th>$q_r$ (kPa)</th>
<th>$P_{el}$ (kPa)</th>
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<tr>
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<td>710</td>
<td>651</td>
<td>0.24</td>
<td>1.65</td>
</tr>
</tbody>
</table>

A closer examination of this figure reveals both that $k_p$ increases with $D_e/B$ and that the average trend line representing this phenomenon can be expressed by the equation: $k_p = 1.03 + 0.39 \times D_e/B$.

Such an interpretation has been performed for each of the sites tested and has shown that for the foundations with no embedment ($D_e/B = 0$), the coefficient $k_p$ is approximately 0.8 for clays, 1 for sands and 1.3 for chalk. These research results, in combination with others (Menard 1963, Antria et al. 1979), have led the French public works administration to propose the new design rates specified in Pascualle 62 (1993).

Figure 2: Evaluation of the experimental load-bearing factor $k_p$ versus the relative settlement $D_e/B$ obtained on the Labeuse site (sand)

3.2.2 Results

Each loading test was associated with a specific pressuremeter profile; an equivalent limit pressure $p_{el}$ was calculated as being the geometric average of the profile between the level of the foundation's base and 1.5 B beneath the foundation (Pascualle 62, 1993). Table 3 summarizes, for the tests conducted on Labeuse site (sand), the resulting values of the failure pressures $q_r$, the equivalent limit pressures $p_{el}$ and the pressuremeter load-bearing factors $k_p$.

Figure 2 indicates, for these same tests, the change occurring in the values of the load-bearing coefficients $k_p$ as a function of the relative equivalent embedment $D_e/B$.

3.3 Eccentric vertical load tests (E/B)

3.3.1 Observed behaviour

About ten tests of this type, split between the Labeuse site (sand) and the Jossigny site (silt) were carried out on square foundations lying on the soil's surface, free to rotate around the load's application point.

It can be noted that the intersection of the successive planes of the foundation's base with the original plane varies in fact very little ($d = constant$) for a given eccentricity E.

Figure 3 shows the evolution of the $d/B$ ratio versus the relative eccentricity $E/B$ for the entire set of tests.

We can note that all these points are very close to the theoretical curve obtained once the assumption of a linear distribution of the stresses under the foundation as well as a proportional relationship between the foundation's displacements and the stresses have been accepted.
3.3.2 Results

The reduction factors due to eccentricity of load \( q_e = q_u \gamma_e \), are presented in Table 4.

<table>
<thead>
<tr>
<th>Ref</th>
<th>( l_e ) (m)</th>
<th>( d_e ) (m)</th>
<th>( E/B )</th>
<th>( q_e ) (kPa)</th>
<th>( l_e )</th>
<th>( d/B )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Innsiøgy Test 10</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>453</td>
<td>0.85</td>
</tr>
<tr>
<td>Test 19</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>444</td>
<td>0.98</td>
</tr>
<tr>
<td>Test 18</td>
<td>0.2</td>
<td>337</td>
<td>0.74</td>
<td>0.47</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test 17</td>
<td>0.3</td>
<td>315</td>
<td>0.70</td>
<td>0.36</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Labenne Test 13</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>887</td>
<td>1.03</td>
</tr>
<tr>
<td>Test 14</td>
<td>0.2</td>
<td>492</td>
<td>0.97</td>
<td>0.03</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test 15</td>
<td>0.2</td>
<td>717</td>
<td>0.81</td>
<td>0.56</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test 16</td>
<td>0.3</td>
<td>553</td>
<td>0.62</td>
<td>0.32</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.4 Inclined centered load tests (6)

3.4.1 Observed behaviour

Half a dozen tests of this type, divided between the sand and silt sites, were conducted. The foundations tested were square foundations lying on the soil's surface, free to rotate around the load's application point.

The most significant results are presented below.

Upon examination of the experimental curves, a gradual displacement of the foundation can be observed in the direction of the load without any significant rotation, with a hinge of the soil in front of the foundation.

\[
\frac{d}{B} = \frac{d}{L}
\]

![Graph showing eccentric loads: Experimental results](image)

A more detailed investigation, with the displacement of the center of the foundation decomposed into two terms, one horizontal \( u \) and the other vertical \( s \), reveals that the progression of both \( u \) and \( s \), as functions of the normal \( (q_u) \) and horizontal \( (q_u) \) components, respectively, is practically independent of the load's angle of inclination.

3.4.2 Results

In the case of an inclined centered load, failure may occur either by punching of the soil's base layer or by slipping of the foundation.

Table 5 presents the values of the load-bearing reduction factors : \( l_e = q_u (6) / q_u (6=0) \).

<table>
<thead>
<tr>
<th>Ref</th>
<th>( l_e ) (m)</th>
<th>( d_e ) (m)</th>
<th>( E/B )</th>
<th>( q_e ) (kPa)</th>
<th>( l_e )</th>
<th>( d/B )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Innsiøgy Test 24</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>350</td>
<td>1</td>
</tr>
<tr>
<td>Test 35</td>
<td>1.1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>350</td>
<td>0.74</td>
</tr>
<tr>
<td>Test 33</td>
<td>0.9</td>
<td>350</td>
<td>0.74</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test 22</td>
<td>0.9</td>
<td>350</td>
<td>0.74</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Labenne Test 13</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>550</td>
<td>1</td>
</tr>
<tr>
<td>Test 14</td>
<td>0.9</td>
<td>550</td>
<td>0.74</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test 15</td>
<td>0.3</td>
<td>240</td>
<td>0.36</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test 16</td>
<td>0.3</td>
<td>240</td>
<td>0.36</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.5 Foundations at the crest of slopes (B)

For this type of configuration, only four full-scale loading tests were actually conducted at the Labenne site (Table 2). In contrast, over 100 tests were carried out on centrifuged models (Bakir 1993).

Results from these tests have been used to define new load-bearing reduction factors (Fascicle 62, 1993). It should be noted that the results from the four tests performed on sites are indeed consistent with those obtained using a centrifuged model.

4. THE RULES PRESCRIBED IN FASCICLE 62

These rules were essentially established on the basis of the experimental results presented above which were complemented, when necessary, by other experimental results (Ménard, Mehs and Weiss, Giraudet, Guerpelmo, Shields et al.) or theoretical results obtained by other researchers (Saleçon).

For greater detail, the reader is referred to Fascicle 62 (1993). Here, only the set of basic rules.
selected for comparison with experimental data are being highlighted.

4.1 Pressuremeter load-bearing factor of \( k_p \)

Table 6 provides, by type of soil, the adopted expressions for the load-bearing factor \( k_p \).

Table 6: Values of the load-bearing factor \( k_p \)

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Formula</th>
<th>( k_p ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft clay and soft silt (A) Soft chalk (A)</td>
<td>( 0.8 - 0.17 \frac{D_n}{B} + 0.08 \frac{D_n}{L} )</td>
<td>&lt; 0.7</td>
</tr>
<tr>
<td>Stiff clays and stiff silts (B)</td>
<td>( 0.8 - 0.17 \frac{D_n}{B} + 0.11 \frac{D_n}{L} )</td>
<td>1.2 to 2.0</td>
</tr>
<tr>
<td>Very stiff to hard clays (C)</td>
<td>( 0.8 - 0.24 \frac{D_n}{B} + 0.10 \frac{D_n}{L} )</td>
<td>&gt; 2.5</td>
</tr>
<tr>
<td>Loose sand and loose gravel (A)</td>
<td>( 1 + 0.3 \frac{D_n}{B} - 0.1 \frac{D_n}{L} )</td>
<td>&lt; 0.5</td>
</tr>
<tr>
<td>Medium sand and medium gravel (B)</td>
<td>( 1 + 0.2 \frac{D_n}{B} + 0.2 \frac{D_n}{L} )</td>
<td>1.0 to 2.0</td>
</tr>
<tr>
<td>Dense sand and dense gravel (C)</td>
<td>( 1 + 0.4 \frac{D_n}{B} + 0.0 \frac{D_n}{L} )</td>
<td>&gt; 2.5</td>
</tr>
<tr>
<td>Weathered chalk (B) Stiff chalk (C)</td>
<td>( 1.3 + 0.2 \frac{D_n}{B} + 0.1 \frac{D_n}{L} )</td>
<td>1 to 2.5 &gt; 3</td>
</tr>
<tr>
<td>Montmorillonite (A) Weathered rock (B)</td>
<td>( 1 + 0.1 \frac{D_n}{B} + 0.1 \frac{D_n}{L} )</td>
<td>1.5 to &gt; 4.5</td>
</tr>
</tbody>
</table>

Figure 4: Load-bearing reduction factors \( i_q \) - experimental data

4.3 Effect of the inclination of load (\( \theta \))

The entire set of experimental data is presented in Figure 5 along with the new curves being proposed. The formulae suggested in the new rules for taking the inclination of load into account are given below.

For cohesive soils, \( i_q = (1 - 0.9 \delta)^2 \), as proposed by Meyerhof (1953).

For friction soils:

\[
i_q = \left[ 1 - \frac{\delta}{Q_n} \right] \left[ 1 - e^{\frac{B_n}{B}} \right] \left[ \max \left( 1 - \frac{\delta}{Q_n} \right) e^{\frac{B_n}{B}} \right]
\]

Figure 2 clearly displays the relative position of the experimental points with respect to the rule adopted for sands.

4.2 Effect of the eccentricity of load (\( E/B \))

Figure 4 shows the values of the load-bearing reduction factor versus the relative eccentricity \( E/B \). On this figure are given the experimental points as well as the line function selected for the fascicule 62 (1993): \( i_q = 1 - 2E/B \)

It should also be noted that all the experimental points are positioned below this line.

Figure 5: Reduction factors \( i_q \)
4.4 Effect of the slope ($\beta$)

Figure 6 shows the values of the load-bearing reduction factor $\eta$ versus the relative distance $d/B$ of the foundation from the crest of the slope.

On this figure, dealing with a strip foundation laying on the crest of a sand embankment ($\beta = 0.5$), are given the experimental points obtained from centrifuged models as well as the function selected for Fascicule 62 (1993):

$$
\eta = \frac{1}{1 + \frac{B}{L}}
$$

Figure 6 : Load–bearing reduction factor $\eta$ versus the distance of the foundation from the crest of an embankment built with a 2/1 slope.

CONCLUSION

The analysis of the experimental data presented herein enables drawing the following conclusions:

-- the bearing capacity of a shallow foundation subjected to a centered vertical load can be estimated in a satisfactory manner on the basis of limit pressures $p_l$ measured underneath the foundation with the Ménard pressuremeter;

-- the bearing capacity of foundations subjected to eccentric or inclined loads or lying near the crest of slopes can be determined from the bearing capacity of the same foundation under vertical centred load and applying reduction factors.

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Meyehoff G. (1953).

*Der Einfluss von Neigung und Ausmitigkeit der Last auf die Grenztragfähigkeit flach


Terashi M., & M. Kitasume (1987). Bearing Capacity of Foundations on Top of Slopes. 8th Asian Regional Conf. on SMFE, Kyoto.
Properties of clays from Menard pressuremeter test results

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Institut de Génie Civil, Université Montaud Manoueri, Tizi-Ouzou, Algeria

ABSTRACT: The results of a series of Menard pressuremeter tests (MPT) carried out in stiff Bordj-Menall clay in Algeria are presented. Six different methods are available to interpret the results of pressuremeter tests in order to determine the undrained shear strength $c_u$. The undrained shear strength values obtained with these different methods and those obtained from static cone penetration tests are compared and discussed. Correlation equations between limit pressure, creep pressure and cone resistance are given.

1 INTRODUCTION

The use of the Menard pressuremeter as an in-situ testing tool has become increasingly popular in recent years in Algeria because of its potential for obtaining stress-strain relationships. Up to now, Menard pressuremeter tests have been carried out over many parts of Algeria to characterize the mechanical behaviour of different soils ranging from very soft clays to soft and weathered rocks. The design foundation based on pressuremeter test results can be performed by using pressuremeter rules that require soil parameters such as the limit pressure and a modulus of deformation, or by deriving parameters such as undrained shear strength, the cohesion, or the angle of internal friction from the Menard pressuremeter test data; these parameters are used in the usual bearing capacity and settlement equations.

This paper presents the results of a series of Menard pressuremeter tests performed in February 1996 on a stiff clay of the Bordj-Menall site located about 60 km east of Algiers, Algeria. The obtained pressuremeter expansion curves have been analysed with different methods, empirical, analytical and numerical, in order to determine the undrained shear strength $c_u$. In this present study the following methods have been tried, Amar and Jezéguel (1972), Salençon (1966), Baguelin et al. (1972, 1978), Prevost and Hoeg (1975) strain softening analysis, a numerical method developed at the Ecole Centrale de Lyon, France, which use the Duncan model (Bouazza 1990, Cambou and Bahar 1993), and the one developed by Oliviari and Bahar (1995). (Abed 1997). The last one uses the generalised Prager model associated to the Von Mises criterion. The undrained shear strength obtained with these different methods and those obtained from empirical methods using static penetration tests are compared and discussed. Correlation equations between limit pressure, creep pressure and cone resistance are given.

2 SITE INVESTIGATION

The Bordj-Menall site is located about 60 km east of Algiers. The site was an area about 100 by 60 m (Figure 1). The site investigation included boring, sampling, Cone penetration tests (CPT) and Menard pressuremeter tests (MPT). The soil consists of clay deposit described as stiff. A summary of the soil properties is shown in Table 1. The ground water table was about 2.6 m depth from ground surface during testing. The nature water content $\omega$ varied between 17 and 24 %. The plasticity index varied between 22 and 30%. Figure 2 shows the results of five static cone penetration tests performed next to the borehole B11.

2.1 Menard pressuremeter tests

44 Menard pressuremeter tests were carried out in stiff Bordj-Menall clay in four boreholes, B11, BH2, BH3 and BH4, located as shown in Figure 1, the test being at depths between 1 and 12 metres. Pressuremeter testing carried out at 1 m depth...
3 INTERPRETATION OF THE PRESSUREMETER CURVES

3.1 Pressuremeter modulus

The pressuremeter modulus $E_m$ was determined from the slope of the linear portion of the corrected pressure versus corrected volume. $E_m$ is given by the equation:

$$E_m = 2 (1 + v) \frac{\Delta p}{\Delta V_v}$$  \hspace{1cm} (1)

where $v$ is Poisson's ratio, assumed 0.5 for undrained tests, $\Delta p$ and $\Delta V_v$ are the differences in pressure and volume, respectively, between two points taken along the straight line portion of the curve. $V_v$ is the average volume of the cavity measured at midpoint of straight line.

In Figure 4 the moduli obtained from MPT tests in the four boreholes are plotted against depth. A wide scattering is obtained, most probably depending upon local variations in soil properties in the horizontal direction and due probably to the disturbance of the soil during the preparation of the boreholes. Modulus is very sensitive to disturbance.

3.2 Limit pressure

The limit pressure $P_l$ is defined at the pressure reached when the initial volume of the cavity has been doubled. Profiles of limit pressure values for the four boreholes are shown in Figure 5. A wide scattering is obtained for depths more than 7 m, most probably depending upon local variations in soils properties in the horizontal direction.

---

Table 1. Soil properties.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td>1.4</td>
<td>3.7</td>
<td>5.4</td>
<td>2.0</td>
<td>4.0</td>
<td>6.6</td>
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<tr>
<td>$Y_d$ (kN/m$^3$)</td>
<td>18.2</td>
<td>16.2</td>
<td>17.5</td>
<td>17.0</td>
<td>16.2</td>
<td>17.7</td>
</tr>
<tr>
<td>$w_c$ (%)</td>
<td>17.2</td>
<td>21.3</td>
<td>19.4</td>
<td>20.2</td>
<td>23.9</td>
<td>18.7</td>
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<td>$S_i$ (%)</td>
<td>94</td>
<td>93</td>
<td>96</td>
<td>93</td>
<td>97</td>
<td>96</td>
</tr>
<tr>
<td>$Y_s$ (kN/m$^3$)</td>
<td>21.2</td>
<td>20.1</td>
<td>20.9</td>
<td>20.5</td>
<td>20.1</td>
<td>21.0</td>
</tr>
<tr>
<td>$w_s$ (%)</td>
<td>54</td>
<td>45.5</td>
<td>44</td>
<td>54.5</td>
<td>59</td>
<td>45.5</td>
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<tr>
<td>$l_p$ %</td>
<td>28.2</td>
<td>24.3</td>
<td>22.8</td>
<td>28.2</td>
<td>29.6</td>
<td>24.4</td>
</tr>
</tbody>
</table>

---

Figure 1. In situ testing boreholes.

Figure 2. Bordj-Menail: static cone penetration tests.

Figure 3. Typical pressuremeter test curves.
Figure 4. Pressmeter modulus versus depth.

Figure 5. Limit pressure versus depth.

3.3 Unstrained shear strength

Several approaches exist to obtain the unstrained shear strength, $C_u$, from a pressmeter curve. Amar and Jézéquel (1972), Salençon (1966), Baguelin and al (1972-1978), Prevost and Hong (1975), Press'Ident (Boubanga 1990, Cambou and Bahar 1993) and Olivari and Bahar (1995). These approaches differ from each other in the assumptions made in soil condition and stress-strain behaviour. However, these six methods made the same basic assumptions of plane strain radial expansion and unstrained condition for analysing the problem. The Baguelin et al (1972) method is referred as the unstrained analysis.

- Salençon analysis: If the Poisson's ratio for the soil is assumed to be 0.5 (unstrained behaviour), the Salençon equation reduces to:

$$C_u = \frac{P_L - P_H}{1 + \log 5 G_u}$$

(2)

where $P_L$, $P_H$, $G_u$, and $C_u$ are limit pressure, in situ total horizontal stress, pressmeter modulus and unstrained shear strength respectively.

- Amar and Jézéquel relationship is given by:

$$C_u = \frac{P_L - P_H}{10} + 25 \text{ (kPa)}$$

(3)

- Press'Ident (pressmeter identification) is a numerical program developed at the Ecole Centrale de Lyon, France, which uses the Duncan model (Boubanga 1990, Cambou and Bahar 1993).

- The method developed by Olivari and Bahar (1995) uses the generalised Prager model associated to the Von Mises criterion. Using an analytical representation of the total stress-strain curve obtained during an unstrained triaxial test given by equation (4), the model takes into account only three parameters which are the elastic shear modulus, a shape parameter characterising the curvature of the test curve, and the unstrained shear strength.

$$\sigma_u = A \left[ \ln(1 - R) + (1 - 2R) \frac{R}{1 - R} \right]$$

(4)

$$R = \frac{\sigma_2 - \sigma_1}{(\sigma_2 - \sigma_1)_0}$$

(5)

Where $\sigma_1$ and $\sigma_2$ are the principal total stresses, $(\sigma_2 - \sigma_1)_0$ is the asymptotic value of stress difference which is related closely to the strength of the soil and $A$ is a parameter defining the curvature of the curve.

Figure 6 shows an identification example for the depth of 5 meters performed in the borehole BHI using Olivari and Bahar analysis (Abel 1997).
3.3.1 Results and discussion

Profiles undrained shear strength values $C_u$ for the boreholes BH1, BH2, BH3 and BH4 obtained for each test by the different methods of interpretation are shown in Figures 7, 8, 9 and 10. It can be seen in these figures that:

- for depths less than 6 meters, Olivari and Bahar, Salençon, Subangent and Press’Ident gave, in general, very similar results;
- for depths more than 6 meters Olivari and Bahar method fitted best with the method by Salençon and gave similar results, with Press’Ident values generally slightly higher;
- The Prevost and Hoeg softening gave far greater scatter and was, in general, much higher. The $C_u$ values measured by Amar and Mézéquel method were, in general, lower than those measured by the other methods. For depths more than 7 meters, there are significant differences in the results. The different assumptions made by each method in the stress-strain behaviour can be considered to be the main reason for the significant differences in the results.

Profiles of the undrained shear strength determined from cone penetration tests performed next to the boreholes 1, 2, 3 and 4 are also given in Figure 7,8,9 and 10. The undrained shear strength values were calculated from static penetration tests using an empirical relationship $C_u = (q_c - p'_{cl})/N$, $q_c$, $p'_{cl}$ and N are cone resistance, in situ effective horizontal stress and an empirical factor respectively.

The value of N ranges between 10 and 15 for a cohesive soils (Casas, 1988). In this present study, N is assumed equal to 15. It can be noted in Figures 7, 8, 9 and 10 that, for the boreholes BH1, BH3 and BH4, the undrained cohesion values obtained from cone penetration tests are relatively similar with those obtained by the above methods. For the borehole BH2, there are significant differences for depths more than 7 meters.
Figure 9. Undrained shear strength versus depth, Bordj-Mennel clay, borehole BH3.

Figure 10. Undrained shear strength versus depth, Bordj Menel clay borehole BH4.

Figure 11. Correlation between limit pressure and creep pressure.

Table 2. Relationship between limit pressure and creep pressure.

<table>
<thead>
<tr>
<th>Site</th>
<th>Bordj-Mennel</th>
<th>Issers</th>
<th>Rou Regh</th>
<th>Bordcuaou</th>
<th>Bourdounia</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tests</td>
<td>43</td>
<td>48</td>
<td>94</td>
<td>28</td>
<td>21</td>
</tr>
<tr>
<td>$P_L/P_C$</td>
<td>1.57</td>
<td>1.60</td>
<td>1.73</td>
<td>1.80</td>
<td>1.69</td>
</tr>
</tbody>
</table>

4. CORRELATION EQUATIONS

4.1 Correlation between limit pressure $P_L$ and creep pressure $P_C$.

A data base of Menard pressuremeter test results and other test data was formed. The pressuremeter data was collected over the last three years on various consulting projects. 39 pressuremeter borings were located in the north of Algeria, with seven clay sites. A total of 269 tests are accumulated. It is therefore useful to explore the relationship between limit pressure and creep pressure which is shown in Figure 11. Table 2 indicates that, for the six clay sites, the $P_L/P_C$ ratio lies between 1.6 to 1.8. For the entire data base, the Figure 11 indicates that there is a constant ratio between limit pressure and creep pressure $P_L/P_C$ is approximately equal to 1.7 which is in agreement with the value obtained by other authors in clays (Cassan, 1988).
4.2 Correlation between limit pressure $P_c$ and cone resistance $q_c$

For Bordj-Menail site, next to the PMT boreholes, 21 static cone penetration tests were performed. Figure 12 shows the correlation between the cone resistance and cone resistance $q_c$. It can be seen a great scatter in the correlation.

![Figure 12. Correlation between limit pressure and cone resistance, Bordj-Menail clay.](image)

CONCLUSION

The undrained shear strength of Bordj-Menail clay were predicted for each test by different methods of interpretation. Of the six methods reviewed, the method by Olivari and Bahar, Saleçon and Press'ident gave, in general, similar results and had reasonable values when compared with other in situ tests results. The Prevoit and Hoog softening gave far greater scatter and was, in general, much higher. The $c_u$ values measured by Amar and Jézéquel method were, in general, lower than those measured by the other methods. The results of pressuremeter tests carried out on several sites confirm the correlation between limit pressure and creep pressure given by many authors for cohesive soils.

ACKNOWLEDGEMENTS

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Laboratory pressuremeter tests on reconstituted silt samples

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ABSTRACT: The differences in pressuremeter strength parameters obtained from the various interpretation procedures are often attributed to the soil disturbance associated with the pressuremeter installation techniques. The results of an experimental investigation on the effect of pressuremeter expansion rate and drainage conditions upon a low plasticity cohesionless silt are presented. To avoid any soil disturbance prior to shearing, soil sample are reconstituted in a large laboratory calibration chamber, from a silt slurry, around the pressuremeter probe. Different interpretation techniques of pressuremeter data are used. Estimated pressuremeter parameters, particularly the lift-off pressure and the peak strength, are found very sensitive to the interpretation method. Loading rate as fast as 10 minutes to double the initial volume of the probe is found not fast enough to prevent drainage.

1 INTRODUCTION

In situ pressuremeter testing offers an attractive opportunity to obtain strength parameters and in-situ stress strain relationship; however the various available interpretation techniques of pressuremeter data and the various results to which they lead introduce enormous difficulties in selecting design strength parameters.

Often, the differences in strength parameters obtained from the various interpretation procedures are related to the soil disturbance associated with the pressuremeter installation techniques; and most theoretically based pressuremeter interpretation methods assume no volume change during testing.

To satisfy this assumption in field testing, a relatively fast rate of loading is selected by practitioners.

To compare pressuremeter strength parameters derived using different interpretation procedures, a series of undrained and drained laboratory pressuremeter tests are conducted on fully and partially saturated isotropic 30.5 cm diameter and 61 cm high silt samples formed by reconstitution from silt slurries around a 2.5 cm diameter pressuremeter probe. This technique eliminates any sample disturbance prior to shearing. The advantage of this laboratory testing procedure lies in the ease with which sample consistency, homogeneity, testing techniques, drainage conditions, and stresses can be controlled compared to field conditions.

All samples are sheared at water content above the liquid limit (wL=28%). Isolation/definition pressuremeter tests are conducted with various expansion rates to examine rates effects upon strength parameters obtained using a perfectly inserted pressuremeter data. The time to the pressure limit varied between 10 and 25 minutes.

To examine the drainage effect on strength parameters, "perfectly" inserted pressuremeter tests are conducted on large silt samples in the calibration chamber with open valves on the pedestal during shear.

2 SOIL CHARACTERISTICS

The soil investigated is a medium to fine silt, the Atterberg limits indicate that the soil is slightly plastic (wL=28%, wPl=99=4%). On the Casagrande plasticity chart, the soil plots as an ML soil. The soil investigated classifies as a gray cohesionless inorganic silt (ML).
3 TESTING PROCEDURE

3.1 Membrane and equipment calibration

As the probe is inflated, the pressuremeter membrane tends to resist expansion. The pressuremeter membrane reaction forces are affected by the rubber membrane quality, as well as fatigue (Obaya et al. 1982). The pressurermeter membrane tends to become more elastic with repeated inflation - definition (Obaya et al. 1982; Elliott 1981). The pressuremeter membrane used in the laboratory loses most of its hysteresis after two cycles of loading-unloading. The tendency of the pressuremeter membrane to return, once softened, to its original elasticity after having been idle requires two cycles to estimate before each test the pressuremeter membrane's reaction forces.

All pressure transducers, and pressure gages are calibrated using a standard dead weight tester of 0.1% accuracy. Since the calibration curves are linear, only transducers' outputs corresponding respectively to zero and maximum pressure are checked, at the end of each test. If a noticeable difference is found, the transducer is recalibrated.

3.2 Sample preparation

To form a sample 30.5 cm in diameter by 61 cm in height, the silt slurry is formed by mixing 77 Kg of air dried silt at a water content of 65% (2.5 times the liquid limit), homogenized by hand mixing, and allowed to hydrate for at least 10 hours (most commonly allowed to hydrate overnight, 16 hours). This high water content is required to obtain a good workability of the silt slurry mix and to ensure the ability to mix the silt slurry with an electrical pump. After the hydration period, the slurry is re-mixed using an electrical pump and then pumped into the slurry cylinder around the pressuremeter probe. During the pumping stage, vacuum is applied, on the lateral sides of the lower part of the slurry cylinder, to hold the membrane against the slurry cylinder. The soil is allowed to sediment for a period of 16 to 24 hours.

In the second stage, the large silt sample is subjected to a one-dimensional consolidation by applying increments of vacuum at the bottom of the sample. During this process, to prevent drying of the top of the sample, a 5 cm high water layer, exposed to atmospheric pressure, is always kept on the top of the silt sample. 100% primary consolidation is reached under each increment of vacuum. The maximum vacuum pressure used is 96.5 KPa.

Further consolidation using vacuum is carried out after sizing the sample, by trimming, to about 61 cm in height, and sealing the top of the sample with a 5 cm thick, plastic plate. Increments of vacuum are reapplied at the bottom of the silt sample, and the volume of water drained and time are recorded.

After assembling the large triaxial chamber, a 34.5 KPa isotropic pressure is applied to the sample, and the specimen under suction is rebounded by letting it suck distilled water. Further consolidation is carried out against a back pressure of 275.8 KPa for the fully saturated samples, and no back pressure for the partially saturated samples. For the fully saturated samples, the back pressure is maintained for at least one hour. The instantaneously measured B values of the Sherrington parameters are greater than 97%. All samples were consolidated to an effective all around pressure of 206.8 KPa, applied in three stages (rebounding under 34.5 KPa an around pressure, consolidation under 137.9 KPa effective pressure), and the pressuremeter tests are conducted under a back pressure of 413.7 KPa for fully saturated samples, and under no back pressure for partially saturated samples.

In some tests Shelby tubes are pushed into the sample at the end of the consolidation stage. Water content tests, from both the Shelby tubes and the sides of the sample, are performed. The water content along the height of the sample varied within 2% range.

3.3 Pressuremeter test

In order to eliminate systematically the effects of disturbance associated with the drilling process, soil sample is reconstituted, by consolidation, around the pressuremeter probe. This technique is intended to simulate a perfect insertion of the pressuremeter.

The laboratory pressuremeter tests were performed on partially and fully saturated samples, with either pervious or impervious boundary conditions. Different loading rates, defined by the time required to double the initial volume of the pressuremeter probe, are used. During the tests, total pressures, volume of water entering the probe, and pore pressures (for tests with impervious boundary) or sample volume change (for tests with pervious boundary) are recorded. Laboratory pressuremeter tests with pervious boundary conditions are intended to assess whether or not the loading rates used would constitute undrained conditions in the field. Different interpretation procedures of pressuremeter data are used.

4 TESTS RESULTS

Results of the three interpretation procedures (Meesad 1957, Baguelin et al. 1972, and Windle and Wrench 1977), are presented. Samples pretreated are consolidated under 206.8 KPa isotropic effective pressure. Tests are conducted at a constant rate of volume change. Stresses in all tests are corrected for
membrane stiffness. Samples A, B, and C are partially saturated (62%-85%), while samples D and E are fully saturated (85%-95%).

Method-1: the undrained strength is related to the limit pressure and to the lift-off pressure (Menard 1957) as:

\[ C_s = \frac{P_L - P_0}{N_p} \]  

(1)

where \( N_p \) is a constant related to soil type. For the soil investigated a value of \( N_p = 3.5 \) is used.

Method-2: based on the assumptions of plane strain, undrained, and axially symmetric deformations about the pressurermeter axis, an analytical relationship between the shear stress and the pressurermeter curve is obtained (Baguelin and al. 1973). This method has a sound theoretical basis, however the graphical interpretation of this method involves substantial subjectivity, and depends on the operator’s judgment. Due to the sensitivity of the differenciation process, use of the finite difference technique results in large scatter of the data. To circumvent this problem, the following fitting procedures (Baguelin and al. 1972)

\[ P = P_0 + A \ln (1 + \frac{c}{c'}) + B \ln (\tan c) \]  

(2)

\[ q = q_0 + c' \left( 1 + \frac{2}{3} \frac{dP}{d\varepsilon} \right) \]  

(3)

are used to evaluate the lift-off pressure \( P_0 \) and the peak strength \( q_{max} \), A and B are fitting parameters.

Method-3: a plot of the radial pressure versus \( \log(\Delta 
 V) \) is fitted with a linear relationship (Windle and Wroth 1977) as:

\[ P = P_0 + C_r \ln \left( \frac{\Delta V}{V} \right) \]  

(4)

This plot easily permits to estimate the peak strength \( C_r \) and the limit pressure \( P_L \).

5 COMPARISON

5.1 Peak strength

The results show that the interpretation method has a great influence upon the estimated pressurermeter shear strength. Method-1 (Menard 1957) and method-2 (Windle and Wroth 1977) seem to yield comparable results, the difference between these two methods is less than 13% for partially saturated samples and is less than 10% for fully saturated samples. However, the differences between Menard's and Baguelin's procedures can be greater than 40%. The simplicity of the Windle and Wroth procedure makes it very attractive.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Partially saturated samples (62%-85%)</th>
<th>Fully saturated samples (85%-95%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>( \sigma_1 ) (KPa)</td>
<td>206.8</td>
<td>206.8</td>
</tr>
<tr>
<td>( \sigma_1' ) (KPa)</td>
<td>206.8</td>
<td>206.8</td>
</tr>
<tr>
<td>( \varepsilon_{\text{lim}} ) (%)</td>
<td>4.6</td>
<td>4.6</td>
</tr>
<tr>
<td>Values:</td>
<td>closed</td>
<td>closed</td>
</tr>
<tr>
<td>( P_L ) (KPa)</td>
<td>289.6</td>
<td>317.1</td>
</tr>
<tr>
<td>( P_c ) (KPa)</td>
<td>737.7</td>
<td>744.6</td>
</tr>
</tbody>
</table>

\( \sigma_1 \): Total cell pressure
\( \sigma_1' \): Back pressure
\( \varepsilon_{\text{lim}} \): Effective consolidation pressure
\( \varepsilon_{\text{lim}} \): Rate of strain (%)
5.2 Horizontal stress

Method-2 (Baguelin 1972) overestimates the observed lift-off pressure by 5% to 24%. For all laboratory pressurerometer tests, the applied total horizontal stress is known, equal to the total cell pressure used. The effective pressurerometer "lift-off pressure" or "initial pressure" should be equivalent to the effective horizontal stress. The results, observed or estimated using different interpretation methods, has not confirmed such expectation. First, it has been believed that the technique used to simulate the perfectly inversed pressurerometer by consolidating the soil around the pressurerometer, induced a stress concentration at the vicinity of the probe. A test in which the soil is consolidated around a slightly inflated pressurerometer probe has been performed. During the consolidation stage, the pressure around the probe is monitored. The pressure near the probe never exceeds the total applied cell pressure. This observation has ruled out the hypothesis that a stress concentration may have been the reason for the difference between the applied effective horizontal stress and the observed "lift-off" pressure.

5.3 Limit pressure

The limit pressure estimated with method-3 (Windle and Wroth 1977) and the observed limit pressure are comparable. The difference is less than 10% for either fully or partially saturated samples with the exception of sample A for which the difference is about 30%. This pressurerometer parameter is the least affected by the method of interpretation.

5.4 Influence of extension rate

The soil undrained shear strength is loading rate dependent. The faster the loading, the higher is the shear strength. Peak strength of Albysh and Boden silts, Sweden, shows a rate dependency, function of the clay content, expressed in terms of the plasticity index (Borgesson 1978). For the cohesionless inorganic silt, $I_p = 4\%$, it is found that the loading rate has a low influence upon the derived peak strength. This is in agreement with the results obtained on Norbotten county silt, Sweden (Borgesson 1978). However, the initial part of the stress-strain curve, up to 2% strain, is loading rate dependent, this section of the stress-strain curve seems to be affected by the initial structural arrangement.

5.5 Influence of saturation

The peak shear strength of partially saturated samples ($62\% = E_b = 65\%$) is higher than that of fully saturated samples; however, it is in the same order to that obtained from an "open valve" test of a fully saturated sample (sample E). The air filled section of the partially saturated sample may have constituted a drainage area during shear.

5.6 Influence of drainage

For soil samples at the same initial state, the peak strength obtained from an undrained test would be lower than that of a drained test. It is always recommended that the field pressurerometer would be conducted "fast enough" to prevent drainage. The motivation behind performing "open valve" laboratory pressurerometer is to determine for the soil investigated, whether the normally suggested loading rate would constitute an undrained test in the field. By performing an undrained test and an "open valve" test on fully saturated samples, using a loading rate as fast as of 4.7% e/min - 10 min. to double the initial volume of the probe, a sample volume change is measured during the "open valve" test. This indicates that for the soil investigated loading rate this fast would not constitute an undrained test in the field. For fully saturated samples, drainage increases the limit pressure and the peak shear strength respectively by 25% and 40%; while for partially saturated samples drainage increases pressurerometer parameters by less than 5%.

6 CONCLUSIONS

The results obtained using different loading rates, different drainage conditions, and different interpretation methods show that for the cohesionless inorganic silt investigated:
1. the peak shear strength is rate independent.
2. Loading rate is fast as 4.7% e/min (10 min to double the initial volume of the probe) is found not fast enough to prevent drainage.
3. Drainage has increased the peak strength by 2% for partially saturated sample, and by 40% for fully saturated sample.
4. The interpretation technique has a great influence on the derived peak strength.
5. The comparison of the lift-off pressure and the applied cell pressure led to the most controversial result. The observed effective lift-off pressure can be 60% greater than the applied effective horizontal stress. Such results indicate that the effective lift-off pressure is influenced by some, not yet well defined, factors.

7 REFERENCES

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Belliaccini, S. 1988. Laboratory study in a calibration chamber of a pressuremeter test on silty silt. Ph.D. Thesis, Dept. of Civil Eng., Tufts University, Massachusetts, USA.


Using pressuremeter to obtain parameters to elastic-plastic models for sands

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ABSTRACT: This paper suggests how to estimate fundamental soil parameters to be used in constitutive laws by using pressuremeter test data. An elastic perfectly plastic theory is used involving dilatation and an increase of the soil stiffness with the increase of the mean normal stress level: the PLAXIS Advance Mohr-Coulomb model of a F.E.M. code. This way, eight parameters can be identified, each of them having a physical meaning to civil engineers. This constitutive law is used to model triaxial tests on the same sands which were later tested by pressuremeter in a calibration chamber. It is shown that it is possible to characterize densification and dilatation during these tests in the 10⁻¹ strain domain.

1 INTRODUCTION

The pressuremeter test (PMT) is part of the in situ tests for which a theory can be built up since boundary values are known. Consequently PMT data should help derive fundamental parameters for soil constitutive laws. However, as for any other in situ tests, stresses and strains around a pressuremeter probe are not uniform and research workers must revert to numerical analysis to achieve their task. Several results have already been presented by Prevost and Haag (1975), Camblou et al. (1993), Shanor et al. (1995), Hicher et al. (1995), Gambin et al. (1996).

In this paper the behaviour of the soil around a pressuremeter is recreated using an elastic perfectly plastic model with dilatation threshold according to the PLAXIS F.E.M. code available to every geotechnical engineer. This procedure will later help them to directly use the results of our research work. Further the classes model, namely the “Advanced Mohr Coulomb” one is a simple model only involving soil parameters which have a physical meaning for the engineer.

During the first part of our work, having obtained the stress-strain parameters of the sand by triaxial tests, one series of PMT carried out in a calibration chamber at the “Laboratoire Solis, Solides, Structures - Lab. 3S” of Grenoble, by Mokarran (1990) is modelled. Conditions of placement of the sand and boundary conditions were perfectly controlled. In a second stage and using a back analysis relationships were set up between the PMT data and model parameters.

It must be pointed out that due to the careful placement of the sand around the pressuremeter probe in the calibration chamber our analysis is not affected by the usual drawbacks of PMT, namely stress relief and remoulding by drilling of the bore wall walls. Then it is judicious to compare pressuremeter modulus and soil modulus in the first part of the PMT loading, i.e. for 10⁻¹ strain level. For a higher strain level, that is 10⁻¹, it is also possible to compare the pressuremeter limit pressure and the critical state of the sand, as long as the numerical analysis involves a readapted procedure.

2 THE NUMERICAL ANALYSIS

We have already mentioned that the analysis uses the PLAXIS F.E.M. code developed at the DelB Technical University (Vermter et al, 1996) available to geotechnical engineers since the late 80’s, and more specifically the “Advanced Mohr Coulomb” model.

2.1 The constitutive law

The “Advanced Mohr Coulomb” model is an elastic perfectly plastic model which exhibit two additional features:

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• non linear elasticity, i.e. stress-dependent stiffness modulus;
• dilation cut-off, which is required for dense sands and for low stress level after extensive shearing.

The elasticity parameters of the model are: \( \nu \), Poisson ratio and \( E \), the stress-dependent modulus, formulated as a power law:

\[
E = E_{ref} \left( \frac{\sigma}{\sigma_{ref}} \right)^n
\]

where: \( E_{ref} \) is the reference modulus corresponding to a reference stress \( \sigma_{ref} = 100 \text{ kPa} \); \( \sigma \) is the effective mean normal stress: \( \sigma = (\sigma_1 + \sigma_2 + \sigma_3)/3 \) (for cohesionless materials) and \( n \) is the power factor.

It must be noted that equation (1) does not affect the volume change which is only a function of \( \nu, \psi, n_l \) and \( n_{max} \).

The plasticity parameters are:
- \( n_l \): cohesion (zero for sand);
- \( \psi \): the friction angle at peak;
- \( \psi \): the dilation angle;
- \( n_l \), \( n_{max} \): are the initial and maximum soil porosity.

Since \( c \) and \( \phi \) fulfill the Mohr-Coulomb perfectly plastic criterion, the ultimate deviator \( q_f \) is:

\[
q_f = (c \cos \phi - \sigma_3) \frac{2 \sin \phi}{1 - \sin \phi}
\]

Soil volume changes are governed by \( \psi \) in the plastic potential functions whereby the dilation takes into account the energy which is dissipated along sliding surfaces during failure. Based on laboratory tests on various sands, the following formula between \( \phi \) and \( \psi \) was proposed by Bolton (1980):

\[
\phi = 0.8 \phi_{cr} + \phi_{se}
\]

where \( \phi_{cr} \) is the critical friction angle measured during a drained triaxial test on a sample which exhibits a zero dilation rate in the plastic phase.

The initial and final porosity parameters \( n_l \) and \( n_{max} \) help obtain the constant dilation level which correspond to the critical state of the material:

\[
\Delta n_l = \frac{b_l f(1 - n_l)}{b_l f(1 - n)}
\]

when \( n = n_{max}, \psi = 0 \) in the PLAXIS model.

### 3.2 Triaxial tests

A serie of conventional triaxial tests carried out on fine Hostun sand (RF), with different relative densities between 2 and 98 per cent, confining pressure \( \sigma_3 = \sigma_1 \) being 100 and 300 kPa, was collected from Bouquet et al (1993).

Figure 1 presents these triaxial results in a reference scale, which is very appropriate to check triaxial test results.

![Figure 1. Triaxial test results.](image)

#### 3.3 PMT in calibration chamber

The calibration chamber is a cylinder 1.2 m in diameter and 1.5 m high. Through confining rubber membranes it is possible to apply vertical pressures (by upper and lower flat membranes) and horizontal pressure (by a lateral cylindrical membrane) up to 600 kPa. Outside shell and lids of the chamber are perfectly rigid.

The pressuremeter probe is 55 mm in diameter and 160 mm long, made of a latex membrane supported by a rigid cylindrical core. The probe is placed vertically in the centre of the chamber and the sand, in a dry condition, is phrased from a constant
height around and also above the probe to achieve a constant density. Thus the sand is moulded around the probe.

A series of tests were performed by Mokran and Foray (Mokran, 1991) on the fine Hostan sand (RD), with a vertical stress varying between 100 and 500 kPa and relative densities between 40 and 88 per cent.

Contrary to the usual Menard pressuremeter curves, the curves in the calibration chamber (figure 2) exhibit neither a recompression phase nor a quasi linear segment. So, in this situation the G modulus is calculated in the initial part of the curve, pressure loss and volume loss being taken into account.

Figure 2. Typical PMT results in calibration chamber.

4 SAND BEHAVIOUR MODELLING

4.1 Triaxial tests modelling

Analysis is made in axisymmetrical small strains; the sand being assumed drained.

During the fitting operation we privileged the best fitting of volume change as a function of strain, since dilatation plays an important role in the PMT data. For the 10° strain level the best fit is of the (ε, ε) curves requested to use the Eor value as a secant modulus at 50 per cent of the maximum deviator.

Table 1 gives the fitting values for triaxial tests. One can note the expected increase of the Eor modulus when density increases and decrease of ϕ and ψ when the p value increases.

It must be noted the low values of Eor obtained from modelling, varying from around 10 to 30 MPa. As was already mentioned this modulus is a reference secant modulus for 50 percent of the maximum deviator stress at a confining pressure (p′/ γ′ v− p) of 0.1 MPa. In consequence this modulus is much lower than the elastic modulus obtained by precision triaxial tests for 10^5 to 10^7 strain levels. The analysis of triaxial results in the same sand, expressed in terms of $\frac{E^p}{2\gamma}$ as a function of log $\varepsilon_i$ (figure 3), shows that the reference modulus correspond to a strain level around 2x10^7. The Figure 3 may also be used to determine the appropriate secant modulus value for a particular geotechnical work, say for 10^7.

<table>
<thead>
<tr>
<th>Hostan R1</th>
<th>$\varepsilon_{or}=0.99$</th>
<th>$\varepsilon_{or}=0.69$</th>
<th>$D_{or}/D_{&lt;p}=2.2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1°</td>
<td>0.82</td>
<td>0.76</td>
<td>0.69</td>
</tr>
<tr>
<td>5°</td>
<td>0.85</td>
<td>0.78</td>
<td>0.71</td>
</tr>
<tr>
<td>10°</td>
<td>0.88</td>
<td>0.79</td>
<td>0.73</td>
</tr>
</tbody>
</table>

Figure 3. Method for extrapolating conventional triaxial test results for strain levels < 10^1 to 10^7.

4.2 Pressuremeter test modelling

Again the analysis is in axisymmetry. There are 192 triangular elements, each with 4 nodes, Dimensions and boundary values are those of the chamber.

4.2.1 Parametric analysis

In our model the parameters which mostly affect the pressuremeter data are Eor, v, ϕ and ψ as well as K0, the pressure at rest coefficient. The density parameters seem to play a secondary role.

The fitting procedure was as follow:

- the K0 coefficient is taken as 0.35, the mean value experimentally obtained;
- the density parameters n, n0, and n200 interpolated from the triaxial tests and the Poisson ratio are kept constant during the fitting;
- the Eor modulus then is selected to obtain the best fit at the origin of the pressuremeter curve;
- $\psi$ and $\gamma$ parameters are chosen to have a best
5 COMPARISON BETWEEN EXPERIMENTAL AND MODELLING RESULTS

5.1 Comparisons of E moduli

Figure 5 shows the correlation between the PMT E modulus ($E_{p}$) and those obtained by the fitting ($E_s$). We can observe that the values are very close, showing that the procedure is appropriated.

Using data from Suarez and Hicher (1994) and Chaif (1995) which give the variation of $E$ with strain level for the dense sand, Figure 6 demonstrates that the E modulus obtained perfectly fit the curve for $10^{-2}$ strain.

In consequence, if another strain level is more representative for the geotechnical design, another modulus must be chosen, for instance from data presented in Figure 3.

5.2 Comparison of limit pressures

The conventional limit pressure as proposed by Louis Menard and his co-workers is obtained for a volume of the cavity twice the initial volume. Since this value is not obtained during most of the tests, the limit pressure is estimated here by log-log method and the inverse volume method (ASTM D 4719, 1980). Comparison with limit pressures derived from model is shown on Figures 7 and 8. For one part the good agreement between the two limit pressures is due to the way the fitting was obtained. Still it is only for the sand samples in loose condition that the two asymptotes are superposed.

5.3 Comparison of fundamental parameters

The fundamental parameters derived from modelling ($E_{c}, v, q, W$ and $p_l$) are in good agreement with those obtained by triaxial test results as well by the classical interpretation of PMT data. The Figures 9, 10 and 11 shows some
6. CONCLUSIONS

By using the PLAXIS F.E.M. code it was possible to show that fundamental soil parameters can be found to be in good agreement when they are derived either from triaxial tests or from pressuremeters tests. Dilation is well modelled in both cases, whatever the level of strain, since the model has a limit to dilation corresponding to the critical state with no volume change.

Still the back analysis of the PMT data requests either to have some additional information from laboratory tests or rely on parameter correlations such as the Bolten equation to be finalised.
<table>
<thead>
<tr>
<th>Reference</th>
<th>Characteristics</th>
<th>K0 (MPa)</th>
<th>n</th>
<th>E (GPa)</th>
<th>( \phi_{90} )</th>
<th>( \gamma )</th>
<th>( \psi )</th>
<th>n1</th>
<th>n2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Briot-d4</td>
<td>D=0.07; ( \sigma_{a}=300 \text{kPa} )</td>
<td>33.2</td>
<td>0.83</td>
<td>82.6</td>
<td>41.6</td>
<td>0.38</td>
<td>13</td>
<td>0.016</td>
<td>0.056</td>
</tr>
<tr>
<td>Briot-d2</td>
<td>D=0.08; ( \sigma_{a}=300 \text{kPa} )</td>
<td>18.3</td>
<td>0.83</td>
<td>46.5</td>
<td>40.3</td>
<td>0.55</td>
<td>11.6</td>
<td>0.018</td>
<td>0.053</td>
</tr>
<tr>
<td>Briot-d5</td>
<td>D=0.09; ( \sigma_{a}=300 \text{kPa} )</td>
<td>30.5</td>
<td>0.83</td>
<td>75.9</td>
<td>41.3</td>
<td>0.01</td>
<td>14.6</td>
<td>0.019</td>
<td>0.067</td>
</tr>
<tr>
<td>Briot-d18</td>
<td>D=0.02; ( \sigma_{a}=100 \text{kPa} )</td>
<td>9.65</td>
<td>0.83</td>
<td>9.62</td>
<td>23.2</td>
<td>0.22</td>
<td>0</td>
<td>0.020</td>
<td>0.070</td>
</tr>
<tr>
<td>Hensel-31</td>
<td>D=0.02; ( \sigma_{a}=100 \text{kPa} )</td>
<td>9.65</td>
<td>0.83</td>
<td>23.6</td>
<td>34.2</td>
<td>0.22</td>
<td>0.3</td>
<td>0.020</td>
<td>0.070</td>
</tr>
<tr>
<td>Hensel-10</td>
<td>D=0.03; ( \sigma_{a}=100 \text{kPa} )</td>
<td>9.65</td>
<td>0.83</td>
<td>23.6</td>
<td>34.2</td>
<td>0.22</td>
<td>0.3</td>
<td>0.020</td>
<td>0.070</td>
</tr>
<tr>
<td>Pressuremeter results (Rc=0.35)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>D=0.83; ( \sigma_{a}=100 \text{kPa} )</td>
<td>22</td>
<td>0.45</td>
<td>13.0</td>
<td>43.3</td>
<td>0.73</td>
<td>10</td>
<td>0.031</td>
<td>0.031</td>
</tr>
<tr>
<td>12</td>
<td>D=0.83; ( \sigma_{a}=100 \text{kPa} )</td>
<td>26</td>
<td>0.45</td>
<td>27.5</td>
<td>41.5</td>
<td>0.35</td>
<td>9</td>
<td>0.041</td>
<td>0.046</td>
</tr>
<tr>
<td>13</td>
<td>D=0.83; ( \sigma_{a}=100 \text{kPa} )</td>
<td>35</td>
<td>0.45</td>
<td>30.3</td>
<td>39.7</td>
<td>0.34</td>
<td>7</td>
<td>0.041</td>
<td>0.046</td>
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<tr>
<td>10</td>
<td>D=0.80; ( \sigma_{a}=100 \text{kPa} )</td>
<td>39</td>
<td>0.45</td>
<td>62.3</td>
<td>38.3</td>
<td>0.32</td>
<td>5</td>
<td>0.041</td>
<td>0.046</td>
</tr>
<tr>
<td>15</td>
<td>D=0.8; ( \sigma_{a}=100 \text{kPa} )</td>
<td>3</td>
<td>0.00</td>
<td>4.9</td>
<td>33.0</td>
<td>0.28</td>
<td>0</td>
<td>0.047</td>
<td>0.045</td>
</tr>
<tr>
<td>14</td>
<td>D=0.8; ( \sigma_{a}=100 \text{kPa} )</td>
<td>9</td>
<td>0.00</td>
<td>9.5</td>
<td>35.0</td>
<td>0.36</td>
<td>-1</td>
<td>0.047</td>
<td>0.048</td>
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<tr>
<td>15</td>
<td>D=0.5; ( \sigma_{a}=100 \text{kPa} )</td>
<td>10</td>
<td>0.00</td>
<td>16.3</td>
<td>36.0</td>
<td>0.34</td>
<td>-2</td>
<td>0.047</td>
<td>0.048</td>
</tr>
<tr>
<td>16</td>
<td>D=0.5; ( \sigma_{a}=100 \text{kPa} )</td>
<td>14</td>
<td>0.6</td>
<td>28.1</td>
<td>29.9</td>
<td>0.25</td>
<td>-3</td>
<td>0.047</td>
<td>0.048</td>
</tr>
</tbody>
</table>

Acknowledgement

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Quantification of the soil disturbance generated by self-boring pressureometers

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ABSTRACT: This paper introduces a new approach to numerically quantify the disturbance generated during the insertion process of the self-boring pressuremeter. This disturbance invariably affects the pressuremeter's results, reliability and field usage time, since an optimization routine has to be carried out in the field during its insertion. The optimization is needed to allow the engineer to select the optimum insertion variables, hence reducing the disturbance imposed in the virgin soil. By numerically assessing this disturbance the optimization routine can be better planned and time shortened, thus reducing the field costs involved with the deployment of this tool and allowing the engineer to scientifically evaluate its obtained results.

1 INTRODUCTION

In situ testing methods are basically divided into two general groups, namely logging and specific methods (Campanella and Robertson 1982). The logging methods were primarily developed for stratigraphic profiling determinations. They are fast and relatively economical in comparison to the specific in situ methods. The later ones were developed in order to extend the particular knowledge of some soil property at specific locations defined by the logging tools. In general these methods are much more specialized and often slower to execute than the logging methods. Although the specific methods can also rely on empirical relationships to define the soil parameters, they have the potential to define the soil properties in terms of a particular stress-strain model.

It was in the context of the specific in situ methods that the pressuremeter was developed. Similarly, like all other specific in situ tools, the pressuremeter does not directly measure any soil parameter, but rather the pressure and the change in volume or radius of an expanding cylindrical membrane. However, the major attraction of this test is the fact that it constitutes a simple boundary value problem in soil mechanics. It can be theoretically modeled by the expansion of an infinitely long cylindrical cavity, where boundary conditions are well defined and controlled. This offers the possibility of the simultaneous derivation of both in situ deformation, lateral stress and strength parameters when applying any of the several available cavity expansion theories.

The self-boring pressuremeter (SBPM) was devised to eliminate disturbance problems caused by the soil probing adopted by the original (pioneer) Méandri pressuremeter probe, since it can ideally 'self bore' itself into the ground without stress and density changes. However, due to the close scrutiny of a number of researchers (Hughes 1973, Mair and Wood 1987 and others) it has been found that the results of the SBPM test are also extremely sensitive to installation techniques, as well test procedures and methodologies of interpretation.

In practice, as mentioned before, the installation of the self-boring pressuremeter is carried out in a trial and error basis, aiming to balance the removal of soil in front and inside the cutting shoe of this equipment with the speed of advancement and other drilling variables, like mud pressure and velocity. The optimum insertion variables are generally unknown at each new site, and shall be obtained in advance prior to further SBPM tests which are aimed to the design stage of the geotechnical work.

The main effect of disturbance on the testing curve is the change of its shape in relation to the 'undisturbed' shape. Since this 'undisturbed' shape is unknown, it becomes difficult to assess the degree of disturbance present in the curve due to the improper insertion process. If the numerical degree of disturbance is known, after each field trial with the self-boring pressuremeter, it becomes easier to readjust the drilling variables towards the 'optimum' general combination. Several testing
results originating from different field trials can be successively assessed to indicate which combination of drilling variables minimizes the disturbance of the testing curve. In order to do that a procedure to numerically quantify the disturbance of the testing must exist, and shall preferably rely on the methodology of interpretation of the testing curve.

The emphasis of Cunha (1994) research thesis was placed on the visualization, assessment and discussion of such interpretation methodology. This author concluded that, from the existing interpretation methodologies for self-boring pressuremeter testing curves, the one denominated “curve fitting” is the best to accomplish above objective. As well, this is less sensitive to the disturbance “built in” the testing curve interpretation methodology, allowing the determination of more reliable soil parameters from the SIBPM testing interpretation.

The curve fitting methodology is briefly presented in a companion paper of this same Conference (Cunha and Camporella 1998), and was already presented in detail somewhere else (Cunha 1996). This approach basically consists in the comparison of the field testing curve with some idealized pressure expansion testing curve, based on an elastoplastic cavity expansion model. The idealized model curve is fitted to the field one after several trials in which the input variables of the model are gradually changed. Once the fitting is over, both model and field curves lay together, it is possible to establish the soil parameters and numerically quantify the soil disturbance.

2 DISTURBANCE QUANTIFICATION

The major benefit from an optimization routine carried out in the field is the faster selection of the optimum drilling variables, which, at moment, depends on the subjective opinion of the pressuremeter operator in regard to defining the “undisturbed” testing curve. As an initial basis for what can be defined as “undisturbed” it is common to use the accumulated experience reported in literature. This experience indicates that high quality curves in both sands and clays are invariably characterized by a continuous and smooth curvilinear shape throughout the strain range of the test.

Denby (1978) and Benoit (1983) observed that disturbance in pressuremeter tests in clays tends to “flatten out” the initial stages of the pressure expansion curve, leading to an almost linear shape. This observation concurs with the opinions of so many others in relation to the less disturbed part of the testing curve. In general it is reasonably well agreed that the latest stages of the testing curve are better for the soil parameters evaluation (with some important details discussed in Cunha and Camporella 1998) than the initial stages.

Robertson (1982) noted that there is no generally recognized criterion for the assessment of the quality of SIBPM testing curves. In general a subjective approach by the “visual inspection” of the testing curve is used. Based on his experience he tried to compile the basic requirements that would serve as guide for the visual selection of “undisturbed” testing curves. However, the broadness and subjectiveness of such requirements turn them of difficult use among different pressuremeter practitioners, since the major drawback of the visual inspection routine is the lack of a recognized “undisturbed” reference curve shape upon which one can visually rely on.

Findlay (1991) presented a methodology to overcome the pitfalls of the visual inspection routine and numerically quantify the disturbance of pressuremeter tests in clays. With this objective he defined an empirical coefficient that was related to the initial slope of the testing curve. The major drawback, however, was the fact that the suggested range of empirical coefficients was site specific, and therefore typical undisturbed values for one particular clay deposit couldn’t be used at a different site. This drawback removes the universality of Findlay’s suggested approach, and may not lead to a much better criteria than the one previously used for clays and sands based on a visual inspection of the curve.

On the other hand, with the concepts stated by Findlay (1991) and the aforementioned curve fitting technique to interpret SIBPM testing curves, one can formulate a disturbance criteria that would be universally applicable. With the curve fitting technique the reference “undisturbed” curve is known, since it is defined by the idealized cavity expansion model curve. This curve will differ for each site and depth in accordance with the fitted soil parameters.

The higher the disturbance the higher will be the influence in the initial stage of the field curve, and hence the greater the deviation from the idealized (“undisturbed”) model curve. If this initial deviation, or differential area as presented in Figure 1, can be numerically quantified, than it should be possible to quantify the disturbance of the test. Similar to the empirical coefficient of Findlay, a numerical coefficient of disturbance (CD) is devised, which measures the average deviation at 1 and 3 ½ circumferential (sands) or radial (clays) strain of the testing curve. This range is based on the experience of the present author and others (as stated before) regarding the zone most affected by disturbance. It is proposed that the CD parameter is defined by the following equation:

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\[ CD = \frac{1}{2} \times 100 \% \]

where \( P_{ci} \) is the idealized internal pressure given by the model at the given strain and \( P_c \) is the equivalent experimental internal pressure at the same strain.

The CD parameter varies between 0 % (fully undisturbed) to 100 % (highly disturbed SBPM tests). Since this coefficient is defined as a normalized ratio it can be used with any of the existing cavity expansion models available in literature (which, in turn, can be used in the framework of the curve fitting technique to interpret the test). The use of a stress ratio, rather than a direct measurement, of an area provides an analogous yet simpler evaluation of CD in the field.

With a computer hooked up to the SBPM probe, and a proper data acquisition system, it is possible to determine the CD parameter in real time after each pressuremeter expansion test. This information can be subsequently used as an input to improve the self-boring quality to the next testing depth.

It can be noted that CD is not an independent variable, but rather a variable that depends on the strain range of curve fitting. As a general rule, and based on an extensive testing research carried out by the author, it is recommended that the curve fitting shall be performed between \( \pm 5 \% \) circumferential / radial strain for either undisturbed or disturbed data. Once the model curve is matched with the field one a value of CD can be devised. It is suggested elsewhere (Cunha 1994) that for CD's below 10 % the field curve can be considered of high quality ("low" disturbance). For CD's from 10 to 30 % the field curve is of good to medium quality ("medium" disturbance). For CD's higher than 30 % the quality of the field curve is low ("high" disturbance).

The assessment of the usefulness of the aforementioned advocated numerical disturbance quantification criteria is presented next, with the use of SBPM field data from a granular deposit close to Vancouver, Canada.

3 ASSESSMENT OF THE CD PARAMETER WITH FIELD DATA

The field examples presented herein were collected by Cunha (1994) during his extensive testing program (with more than 100 self-boring pressuremeter tests) at one of the University of British Columbia well known research sites. The objective of such tests was to understand the effects of several testing variables, as the occasional plugging of the cutting shoe, dimensional differences along the probe's shank, etc.

The chosen site is denominated "Laing Bridge South" and is located in the Fraser Delta, near the city of Vancouver and its international airport. The site has a stratigraphy comprised of a thin surface (2 to 3 m) layer of sandy silt material underlain by a post glacial Holocene strata of fine to medium sand with 15 to 20 m in thickness which, in turn, is underlain by a thick layer of normally consolidated clayey silt to silty clay. The sand strata has not been mechanically overconsolidated by ice load and is relatively homogeneous with an approximate 5 % of silt content. It is mainly composed by quartz (67 %) and can be classified as SP to SF-SM by the Unified Soil Classification system. Further geotechnical and geological details can be found in Bertok 1987.

The likely effect of partial plugging during self-boring is addressed in the next example. In this one the effects of shoe plugging were considered, by comparing two test soundings 5 m apart (SBP11 and 12) in which the plug in the field, estimated by visual inspection of the shoe after withdrawal of the probe, varied from 25 (SBP12) to 75 % (SBP11) of the cross-sectional area of the cutting shoe. Similar equipment and drilling variables (rate of penetration and flow) were adopted for both soundings, and plugging occurred due to inadequate drilling variables adopted during self-boring of the superficial sandy silt material. Typical SBPM curves at 10.4 m testing depth are compared in Figure 2, where it can be noted that the larger the plug the larger will be the lift off stress, indicating high residual stresses set up at the soil-probe interface. As well, the larger will be the average CD parameter along depth, as demonstrated by Figure 3.
The following and last example refers to testing soundings in which the effects of dimensional differences along the shaft were studied. The dimensional tolerances along the shaft of the pressuremeter can be of extreme importance in the process of soil disturbance as already observed in literature (e.g., Faleay and Randolph 1984), and can be simulated by adopting oversized lantern retainers to hold the pressuremeter's lantern.

For this example the testing results of the pressuremeter soundings SBP16, SBP17, SBP18, and SBP19 were used. In SBP16 and 17, as schematically shown in Figure 4, the sounding was carried out with an oversized lantern retainer that had a larger diameter difference with respect to the diameter of the steel lantern (1.5 mm) and a lesser difference to the diameter of the cutting shoe (0.5 mm). In soundings SBP18 and 19 a new lantern retainer was used, in order to reduce the dimensional differences in relation to the remaining parts of the pressuremeter unit. In this case a tapered retainer, with diameter varying from 74 mm at the shoe connection (same diameter of the shoe) to 74.5 mm at the steel lantern connection was devised. A lesser diameter difference of 0.5 mm with respect to the diameter of the lantern was obtained at the top of the retainer, because two rubber membranes were used underneath the steel lantern.

Figure 5 presents the variation of the CD parameter along depth for the aforementioned sounding cases. It can be readily seen that the disturbance imposed by the SBP16 and 17 soundings (classified as "high") was in average 3 times higher than the disturbance caused by the insertion of the SBP18 and SBP19 soundings (classified as "low"). Indeed, during the insertion of both cases a small loading and subsequent unloading of the soil in the vicinity of the pressuremeter shaft took place.

However, for the SBP16 and 17 soundings an initial outward soil displacement of 0.67% of the diameter of the shoe took place, followed by a final and abrupt inward displacement of 2.0% of this same diameter. In the case of SBP18 and 19 a similar (and smaller in magnitude) loading and unloading mechanism was imposed in the surrounding soil. In this later case there was a gradual outward displacement of the material up to only 0.67% of the shoe diameter, followed by a final inward soil displacement of similar magnitude.

In Figure 6 the effects of dimensional differences are shown for a typical test of the aforementioned soundings. The low CD parameter of 6% again suggests almost no disturbance for the SBP19 sounding. This is also reflected by its curvilinear and smooth shape. A much higher CD parameter of 40% was observed for the SBP17 test, which is also reflected by its almost entirely linear shape.

This last example demonstrates that the soil surrounding the pressuremeter can have a distinct behavior depending on small dimensional tolerances adopted for the probe, and its sensitivity to loading and unloading mechanisms during the probe’s insertion. With the aid of the numerical quantification criteria advocated in this paper it was possible not only to numerically assess the extent of disturbance in each sounding but also gradually guided the author into the solution’s direction.
Figure 4. Dimensional characteristics of the distinct lanterns used

Figure 6. Typical comparison of curves from tests with different dimensional tolerances

Figure 5. Influence of dimensional tolerances of the probe on the CD parameter

4 CONCLUSIONS

Disturbance generated during selfboring is a complex combination of the influence of variables related to the drilling parameters adopted during insertion, to the equipment used, to random field occurrences (such as shoe plugging), as well as to secondary unknown factors.

Using the curve fitting approach to interpret selfboring pressuremeter testing curves it is possible to devise a coefficient of disturbance CD to numerically quantify this disturbance, thus removing the subjectiveness that existed so far in the assessment of the quality of SBPM testing curves. This coefficient can be used with typical (proposed) ranges of expected values, corresponding to different degrees of curve quality, to assess the testing results and define the best insertion conditions in the field. As demonstrated herein, the CD parameter has been shown to properly identify the degradation of the quality of the pressuremeter testing curve caused by inadequate drilling variables adopted in the field and inadequate dimensional tolerances of the probe.

The adoption of a field optimization routine with the use of such coefficient shall invariably reduce the costs involved with the deployment of this complex testing tool, encouraging a higher use of the SBPM in current geotechnical designs. Moreover, it will help to establish insertion standards, or guidelines, in continuously studied sites.

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Interpretation of self-boring pressuremeter tests using a curve fitting approach

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ABSTRACT: This paper presents the concept of interpreting self-boring pressuremeter testing curves by using a curve fitting approach, which is seldom used nowadays to derive soil parameters from this in-situ testing tool. This approach has the great advantage of easily establishing useful soil data even when testing curves are initially disturbed. It has been found, however, that the quality of the soil parameters predicted by the curve fitting approach is also dependent on factors other than the initial conditions of the soil, like the ability of the idealized cavity expansion model to simulate the pressuremeter expansion process and the strain range of curve fitting. Thus, this paper addresses the strain range factor and gives some general guidelines for the interpretation of self-boring pressuremeter testing curves with the curve fitting approach.

1 INTRODUCTION

Since Menard's pioneering work there have been major developments in the pressuremeter, especially in the 70's and 80's. These developments can be subdivided into areas of pressure and strain measurement, probe insertion, analytical and numerical interpretation and new types of pressuremeter. The self-boring pressuremeter was devised to eliminate the disturbance problems caused by the preboring method adopted for the Menard pressuremeter.

With the self-boring method to minimize disturbance the test results can be properly analyzed in the light of cavity expansion theories, where the initial conditions are well established. However, Hughes (1973) demonstrated in the laboratory with the X-ray technique that even under "perfect" self-boring conditions a disturbance of at least 0.5% of the pressuremeter radius can be induced in sand. Thus, when self-boring in-situ an even higher disturbance percentage can be expected.

The current traditional methodology of analysis, well discussed by many authors (Benoit and Clough 1986, Fidlony 1991 and others), is extremely sensitive to any disturbance "built into" the testing curve. Therefore, the disturbance affects set up during the self-boring pressuremeter installation may limit the usefulness of this tool, at least in sands. In order to overcome this aspect, emphasis is placed on the development of improved ways of interpreting the data which are less sensitive to disturbance. The presentation of this less sensitive interpretation approach, and its usefulness in interpreting both disturbed and undisturbed testing curves are demonstrated using field examples and are given next.

2 CURVE FITTING OF TESTING CURVES

The curve fitting approach basically consists in the comparison of the field testing curve with some idealized pressure expansion testing curve based on some sort of constitutive model, as demonstrated by Figure 1. The idealized model curve can be interactively changed by changing the input parameters that constitute its rheological relationships. This is carried out until a "match" or "fitting" between this model curve and the field one is established.

The closer the rheological relationships and assumptions of the adopted cavity expansion model are to the real shearing phenomena, the closer will be the agreement of the idealized curve with respect to the field one (assuming no disturbance). As well, the lower the number of input model parameters, the easier and faster will be the curve fitting. In this respect, Figure 1 illustrates a curve fitting example in which only 3 parameters of the adopted model (c, or effective lateral stress, G or shear modulus and θ or friction angle) are varied during the curve fitting.

The curve fitting concept is not new in the pressuremeter technology, although it is almost
3 STRAIN RANGE OF CURVE FITTING

The data presented herein was obtained during an extensive field testing programme in the UBC research site called "Laing Bridge South", for which details can be found in Berok (1987). This data was interpreted by a new cavity expansion model (Cunha 1994), although any of the existing cavity expansion models in literature for drained materials could be used for this purpose. Since the emphasis of this paper is not placed on theoretical aspects of the modelling itself, but rather on the concept of test interpretation and its refinements, it is felt that the usage of such a model does not hamper the conclusions yet to be drawn.

The analysis is carried out with both undisturbed as well as disturbed testing data, trying to cover all possible classes of pressuremeter testing curves which are obtained in the field. The chosen set of data originates from test soundings 8 m apart at the site, and are related to the same (3.3 m) depth.

3.1 Undisturbed Testing Curve

Typical self-boring pressuremeter tests are expanded to cavity (circular in sand) strains in the range of 10%. Under the traditional interpretation methodology only the last loading points obtained within this testing range are used to define the slope in the log-log graph, hence the friction angle. This is so because it is argued that the initial stages of expansion can be considerably affected by the disturbance generated prior to the test. Following this same reasoning, it may be also argued that if the approach followed by full-displacement pressuremeter tests is adopted for the self-boring, pressuremeter, with a testing stage carried out to a considerably higher circumferential strain, then it will be possible to predict truly undisturbed soil parameters from the interpretation of the latter stages of the field data.

In order to assess this hypothesis a series of curve fitting interpretations were carried out with an undisturbed field curve. This curve had its quality assessed by the visual quality assessment criteria put forward by Robertson (1982), and was expanded to a circumferential strain around 20%. The fitting ranges chosen for the match of both field and analytical (model) curves varied from 0 to 5%, 5 to 10%, 10 to 15% and above 15% as demonstrated by the top plot of Figure 2.

The interpretation analysis varied the parameters $\phi$, $G_i$ and $\delta$ during the fitting, and adopted the constant values of 34° and 0.25 for the constant-volume friction angle and the Poisson's coefficient, respectively. These latter values were experimentally obtained in drained triaxial tests with undisturbed samples of this site.

Figure 2 also shows the curve fit results (predicted parameters) for each of the chosen strain ranges, rendering the following comments:

1. The predicted soil parameters are not unique and depend on the range of match adopted during fitting. Nevertheless, for curve fittings between 0 to 5% or 5 to 10% the same idealized curve suffices to represent the measured experimental
3. The predicted soil parameters for the curve match above $\approx 10\%$ do not seem to be realistic. The predicted effective lateral stresses appear to be extremely overestimated (consider, for instance, that the effective vertical stress at this depth is around 60 kPa). The predicted friction angles appear to be extremely underestimated. This is so because friction values below the constant volume angle are predicted, suggesting a contractive behavior during shear rather than dilatant. However, a fully contractive behavior was not observed in the triaxial laboratory tests with the undisturbed samples of this site.

The findings above suggest that for undisturbed self-boring testing curves meaningful parameters from the fitting approach can be solely obtained if the match is carried out with the initial loading points of the testing curve, between 0 to $\approx 10\%$ (cylindrical strain). This finding is directly in opposition to the common belief that only the latter stages of the testing curve are useful for the interpretation analysis, but agrees with the usual practice of expanding self-boring pressure sensors to cavity strains up to around 10%.

Perhaps the information experimentally measured for circumferential strains above $\approx 10\%$ is influenced by external factors that are not considered in the existing cavity expansion models. Indeed, two possible factors are prone to hamper the interpretation analysis at these high strain levels, as follows:

1. End Effects: The UHE self-boring has an expanding section only 6 meters (12 radii) long and it is unlikely that a plane strain solution will be applicable for strains much larger than 10%. Given the possible non-cylindrical shape of the plastic zone at these latter stages, a compromise between the cylindrical and the spherical cavity expansion theory would have to be developed for the interpretation analysis.

2. Strain Softening of the Sand: The typical shear behavior of the sand at a constant confining pressure was discussed by Vaid et al (1980). Over a considerable range of strain, both initially loose dense samples undergo volume expansion, and at very large shear strains tend to approach an ultimate strength and critical void ratio, at which the sand shears with no change in volume or stress.

In the latter stages of the test the expansion also takes place with the imposition of very high levels of shear stress and strain in the sand surrounding the cavity. Based on the experimental sand behavior previously mentioned, it may be possible to speculate that during this stage an annulus of sand at critical state conditions will be developed between the cavity wall and the elasto-plastic boundary. In this case the expansion process can be understood as the expansion of a two-layered system, composed of an inner layer shearing at

![Figure 2. Fitting analyses and results on undisturbed testing curve](image-url)
constant volume conditions encompassed by an outer (plastic) layer where dilative volume change takes place. The response of this critical strate
annulus of sand has a dominant effect on the
measured testing response at the latter stages of
expansion. Thus, if reliable soil parameters are
sought with curve fitting interpretation at high strain
levels, then the cavity expansion model has to be
modified to account for the strain softening of the
sand. Failure to do so results in the prediction of
unrealistically high $\phi_s$'s and low $\phi_s$'s, such as those
presented in Figure 2.

3.2. Disturbed Testing Curve

Disturbance affects the shape of the testing curve,
and consequently should also influence the final set of
predicted soil parameters.

In order to assess the likely influence of
disturbance on the predicted parameters another
series of curve fitting interpretations was carried
out. For this purpose a disturbed field curve was
selected. The disturbed characteristics of the chosen
curve are assessed based on the following evidence.

1. The shape of the curve does not follow the
"high quality standards" put forward by
Robertson (1982) with his visual quality assessment
criteria. This is, however, circumstantial evidence of
the disturbed characteristics of this field curve;
2. The chosen curve came from the test sounding
SBP19, at a depth of 5.3 m in the testing site. This
particular sounding consisted of 2 insertion trials at
the same borehole. The first trial was carried out up
to 5.7 m at a high penetration rate, resulting in the
plugging of the cutting shoe to an extent of 70% of
its sectional area. This invariably disturbed the
surrounding sand up to 6 m deep. This is strong
evidence of the disturbed characteristics of this
particular field curve.

The interpretation analysis was conducted
similarly as before, but in this case a higher number
of fitting ranges were selected for the interpretation
analyses. The fitting ranges chosen for the match of
both field and analytical curves varied from 0 to
3%, 3 to 6%, 4 to 7%, 5 to 8%, 6 to 9%, 7 to
10% and 9 to 10%. Given the previous findings the
interpretation analysis was carried out up to a
circumferential strain of 10%.

Figure 3 shows the chosen field curve and the
obtained results. The top plot presents the fitting
between both field and idealized analytical curves.
For clarity, only 3 analytical curves are shown. The
bottom plot shows the predicted parameters for each
fitting case.

For this disturbed testing curve the following
comments apply:
1. The quality of curve fitting is excellent in
either the initial or in the latter stages of the field
curve. This does not mean that the predicted soil
parameters (for each of the fitting cases) are equally
reliable.
2. As for the undisturbed testing curve the
parameters obtained are not unique, and depend on
the range of fitting. Nevertheless, for each of the
predicted parameters, the variation with the range of
curve fitting "levels off" for matches at the latter
stages of the test (close to 10%). For curve fittings
in the initial ranges of the test, from 0 to = 5% it is
not possible to obtain a unique set of predicted
parameters;
3. The predicted parameters for curve fittings
between = 5% and 10% (average $\phi = 44^\circ$,
$\phi_0 = 37$ kPa and $G_L = 11.3$ MPa) agree well with the parameters obtained with the fitting interpretation of the initial stages of the undisturbed curve.

4. The predicted soil parameters for the curve fitting in the initial stages of the field test do not seem to be realistic. This observation was also given with the results of the analysis carried out in the latter stages (beyond 10%) of the undisturbed curve. With the disturbed curve, however, it appears that overestimated friction angles (above 50°) and underestimated effective lateral stresses (below 20 kPa) were predicted.

The findings above suggest that for disturbed self-boring testing curves meaningful parameters from the fitting approach can be solely obtained if the testing is solely carried out with the latter stages (above 5% and below 10%) of the field curve. This is caused by the fact that disturbance affects the initial shape of the testing curve, reducing its "roundness". At the latter stages of this same curve, below a circumferential strain of approximately 10%, the effects of disturbance are decreased and may be even erased.

Basically when disturbance is generated during the self-boring process an annulus of disturbed and loose soil is formed around the probe. The diameter of this annulus is unknown and will depend on the degree of disturbance generated prior to the testing stage. The response of this speculated annulus of soil influences the measured response of the test. The expansion process, therefore, can be also understood as the cavity expansion in a two-layered system, one looser close to the probe's shaft and another denser around this first layer. The self-boring testing curve will initially follow the path defined by the looser (disturbed) annulus of sand, therefore reducing its initial smooth "roundness".

As the plastic zone grows in the latter stages of expansion (beyond 5%), the effects of the disturbed annulus on the measured response are continuously decreased. This results from the fact that a larger zone of undisturbed soil starts to be encompassed by the expanding plastic zone. Hence, the measured cavity response at the latter stages of expansion predominantly reflects the shearing response of this undisturbed zone of soil. However, even if the disturbance on the testing curve is erased beyond circumferential strains above 10% other factors start to dominate (as noted before) hampering the fitting interpretation analysis.

4 CONCLUSIONS

The interpretation methodology presented herein allows the prediction of reliable soil parameters for either undisturbed or disturbed data in sands. This represents an advance in relation to the traditional interpretation methodologies, that could only be applied in perfectly undisturbed self-boring pressuremeter curves. Nevertheless, the reliability of the predicted soil parameters may be expected to be directly proportional to the quality of the testing curve. For undisturbed or slightly disturbed testing curves the reliability of the predicted parameters is high. For disturbed curves the reliability of the parameters is somewhat reduced. As a general rule, for either undisturbed or disturbed data the curve fitting should be carried out in the latter stages of expansion (between $\approx 5\%$ to $\approx 10\%$).

As a final comment, it shall be noted that, with the aforementioned interpretation methodology, it is also possible to obtain a good indicator of fundamental soil parameters from prebored or Ménard pressuremeter test results, as long as the curve fitting can be applied between 5 and 10% strain. Reasonably reliable deformation and strength parameters have already been obtained with Ménard pressuremeter tests in the residual (unsaturated) porous clay of Brasilia, as presented by Ortigao et al (1996).

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The Menard pressuremeter for quality control of soil densification

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ABSTRACT: Any soil improvement technique needs to be associated with specific testing equipment for acceptance of the works. When speaking about soil improvement by mechanical means, static loading tests have been proved to be the most representative tests to obtain the improvement ratio. Can the Menard pressuremeter test which is a static loading test be used in this event? Theoretical as well as practical aspects of this test are analysed here and compared with other types of soil testing techniques.

1 GENERAL

A cost effective monitoring of a soil improvement job by mechanical means such as vibrocompaction, dynamic compaction or compaction grouting is possible by a continuous recording of the volume change of the treated soil during the improvement process. However the engineer must be aware of the final values of the stress-strain parameters which are necessary to design the foundations of the proposed construction.

The choice is between laboratory testing on so-called undisturbed samples and in situ testing. Preliminary conditions to a reliable testing scheme are:

a. to involve soil volumes as big as possible so as to get an overall view of the improvement,
b. to avoid the disturbance of the new soil structure before or during testing.

It can immediately be seen that laboratory testing is eliminated:

- one sample has a very small size and the testing of many samples can be considered too costly
- remoulding of samples is not avoidable unless exceptional precautions are taken to keep the overconsolidation effect at the time of core retrieval. Further it is easy to demonstrate that solely evaluating index properties such as the void ratio on samples is not accurate enough to reach the proposed target (Gambin 1991).

Consequently the Engineer is lead to revert to in situ testing. What sort of in situ testing?

2 IN SITU TESTING AS ACCEPTANCE TESTS

In situ tests can be divided into three main categories:

- penetration tests
- geophysical tests
- static loading tests

Each category involves several types of tests.

Before ranking these tests for our present purpose we must go back to basic soil mechanics.

In most cases soil stiffness governs the design of shallow foundations. This stiffness involves many soil properties such as:

- the mineralogy, the angularity and the grading of the soil constituents
- the gain arrangement and orientation
- the stress-strain history of the material including its present mean effective stress and its shear strain plastic hardening
- the drainage conditions
- the time effects: loading rate, creep and aging.

Can this stiffness be measured during the penetration of a hammer-driven split spoon sampler
(SPT) or a steadily pushed cone (CPT)? Experience has shown that the penetration resistance cannot be analysed in terms of fundamental soil parameters. Since this would involve too complex boundary value problems (i.e. the stress distribution on the cone surface at a given penetration velocity). Moreover, below a pushed cone as well as below a loaded rigid plate, soil elements follow different effective stress paths according to their location with respect to the cone or the plate and according to the magnitude of the applied load.

What we know is that the SPT or CPT penetration resistance is primarily governed by the soil void ratio (or more practically the relative density or density index Ip) and the effective stress. Their combined effect is visualised by the state parameter as defined by Been and Jeffery (1985).

For the SPT:
\[ I_D = \sqrt{\frac{N}{C_0 \sigma_{vo} + C_1}} \]

where \( C_0 \) and \( C_1 \) slightly vary from Gibb's and Holtz to Skempton, for freshly deposited sands.

For the CPT:
\[ I_D = \frac{1}{C_2} \ln \left( \frac{q_c}{C_0 \sigma_{vo}^{n} \sigma_{o}} \right) \]

where \( C_{vo} \), \( C_1 \) and \( C_2 \) are different for normally consolidated sands and overconsolidated sands (Pano et al. 1995), \( C_1 \) still remaining close to 0.5.

When using the stress parameter \( \psi \), one can write:
\[ q_c - \sigma_{vo} / \sigma_{m} = a \exp (n \psi) \]

where: \( \sigma_{vo} \) is the total mean stress, \( \sigma_{m} \) the effective mean stress, \( a \) and \( n \) factors approximately equal to 35 and -9 respectively.

The previous relationships are given for clean sands. As soon as the percentage of fines increases and also in the event of more crushable sands the value of relative density is underestimated (Tatsuoka et al. 1978).

If in a calibration chamber a sand is subjected to a cyclic pretesting along different stress paths, values of the cone resistance \( q_c \) and tangent Young's modulus \( E_t \) before and after treatment exhibit very different improvement ratios:

Table 1: Comparison of \( q_c \) and \( E_t \) before and after cyclic stressing (in MPa).

<table>
<thead>
<tr>
<th></th>
<th>( q_c )</th>
<th>( E_t )</th>
<th>Ratios</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before</td>
<td>6.8</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>After</td>
<td>8.1</td>
<td>132</td>
<td>1.2</td>
</tr>
</tbody>
</table>

The same observation applies to normalised value of \( q_c / \sqrt{\sigma_{vo}} \) and \( E_t / \sqrt{\sigma_{vo}} \) which respectively increase by 10 - 30 percent and 3-5 times.

As a matter of fact the large strains created during the penetration of devices like SPT, CPT (and also the Marchetti's flat dilatometer or DMT) obliterate to a large extent the effect of the stress-strain history in the soil. Recommended values for \( E_t / q_c \) are given in table 6 (Pano et al 1995) for silica sands. They show large variations:

Table 2: Present recommended values of \( E_t / q_c \) in sands.

<table>
<thead>
<tr>
<th></th>
<th>( E_t / q_c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freshly deposited sand</td>
<td>2.0 - 3.5</td>
</tr>
<tr>
<td>Aged normally consolidated sand</td>
<td>3.5 - 6.0</td>
</tr>
<tr>
<td>Overconsolidated sand</td>
<td>6.0 - 12.0</td>
</tr>
</tbody>
</table>

We have made a long way since the early research work of Schmertmann (1970) later improved by Leonards and Frost (1988).

Still a better correlation seems to be obtainable between \( q_c \) and \( G_0 \), the purely elastic shear modulus for 10^6 strains, but since \( G_0 \) is mostly affected by the relative density, and not so much by the stress-strain history of the soil, this way seems to lead to a dead end. 2.1 Dynamic penetration testing

It cannot be denied that during a standard penetration test or any dynamic cone testing which stresses the soil to failure the new soil structure is largely disturbed; no valuable data on overconsolidation ratio can be obtained. Further in its chapter 9 the ASTM standard method D 1586 pinpoints a very poor accuracy of the blow count. A more restrictive suggested practice was
envisioned at a time to make the blow count more reliable, especially to detect liquefiable soils. At least Standard Method D 4633 was issued to direct how to measure the stress wave energy at each blow and eliminate part of the bias.

2.2 Static cone testing

The two main parameters which can be derived from this test are totally empirical, since \( q_c = \frac{Q_a}{A} \), where \( Q_a \) is the soil failure load for a penetration velocity of 20 mm/sec and \( A \) is the cross section of the cone (usually 10 cm²). Similarly the friction resistance \( Q_f \) is measured on a conventional shaft, \( Q_f \) being equal to \( Q_{total} - Q_{cone} \), although \( Q_f \) is only a few percent (1 to 5) of \( Q_{cone} \). Further, to make comparison of tests before treatment and after treatment the same type of cone must be used, (Nyrens 1995).

Instead of using \( q_c \) directly for foundation design through an adequate bearing factor, since the OCR value cannot be known unless other types of tests are also performed, many practitioners (Zambello and Pasquale 1992) proposed to derive relative density index from \( q_c \) and check that performance criteria originally expressed by this index are fulfilled. This fulfilment is not always easy (Debat and Sims 1997).

2.3 Geophysical tests

Geophysical tests do not directly measure the stiffness of the soil. However, due to the relationship between the G modulus at very small strain and the propagation velocity of true elastic waves in soils, the improvement of stress-strain parameters in treated soils is more and more checked using one of the following techniques:

- seismic refraction
- surface wave propagation (by sometimes using the dynamic consolidation tamper as the vibration source, in the event of this technique being used)
- crosshole testing or downhole testing
- seismic cone
- spectral analysis of surface waves (Haegensel and Van Impe 1995).

Analysis is based on the use of the formula giving the shear wave velocity \( V = \sqrt{G_p/\rho} \) whereby the soil density \( \rho \) exhibiting a very small change during improvement, the G modulus at small strain \( (G_s) \) varies as the square of the shear wave velocity. However these methods are indirect methods, they are costly and not always easy to perform. Results yielded are again on the conservative side since the variation of \( G_s \) does not seem to be so much affected by the stress-strain history of the soil, especially in clean silica sands (Shibuya et al 1992).

2.4 Static loading tests

These tests encompass:
- embankment tests on large areas
- prototype footings loading tests which also involve large volumes of soil.

During any static loading test and along the first pressure increments, strain readings will make it possible to evaluate the new stiffness of the improved soil, which must be used for foundation design.

Embankment test results can be analysed by the theory of elasticity but for rigid footings tests one must revert to cruder formulae. In both cases the stiffness is obtained for a range of strains equal to that of the test.

3 THE PRESSUREMETERS

These devices developed by Menard in the late 50's for the prebored one and in the mid 70's by two research teams, one in France, one in the UK for the self boring one, fulfill the 2 initial conditions put forward:

- one test involves a volume of soil of 0.1 m³
- the slowly increasing rate of stress does not disturb the new soil structure.

Moreover boundary values problems can be solved since pressure at the borehole wall is known and constant at every instant, and pressure at a distance is always equal to \( \sigma_m \). Also all soil elements which are strained during the test follow very similar effective stress paths. Consequently by assuming a non linear elasticity theory it is possible to derive a variable G modulus for various strain levels, a value theoretically independent of the drainage conditions. Experience has shown that with the prebored Menard pressuremeter, only one G value can be obtained which corresponds to a 10⁻⁴ strain, i.e. the strain observed below a well designed footing (Menard 1961).

Regarding derivation of shear strength assumptions must be made about drainage conditions plastic and stress-strain relationship.
avoid these difficulties Menard and his coworkers proposed to conventionally fix the limit pressure to the pressure for which the cavity exhibits a volume twice as large as the original cavity. Consequently the Menard limit pressure is also a function of the soil straining.

4 JOB SITE COMPARISON

The comparison between CPT and Menard PMT results before and after soil improvement which is reported here below is related to a dynamic consolidation job in South West of France completed in 1993.

Main geotechnical properties of the alluvial clayey soil are given in Table 3.

Table 3: Geotechnical properties of the alluvium (stresses in MPa)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>LL</th>
<th>PI</th>
<th>C_F</th>
<th>CPT</th>
<th>Menard PMT</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 - 6</td>
<td>58</td>
<td>26</td>
<td>0.045</td>
<td>1.2 - 3</td>
<td>2.5 - 3.0</td>
</tr>
<tr>
<td>6 - 12</td>
<td>58</td>
<td>15</td>
<td>0.22</td>
<td>1.1 - 5</td>
<td>2.5 - 3.0</td>
</tr>
</tbody>
</table>

(ground water level at 2 m depth)

The settlement of a test embankment 15x15 m in area, bringing a pressure of 40 kPa was estimated at 180 mm at the end of primary consolidation.

After treatment which also included stone piers (Gambin, 1984) under structural loads the following acceptance tests were carried out:

- one conventional series of CPT's
- one series of CPTU's (piezocone) with some holding tests to observe excess pore water pressure dissipation time,
- one series of Menard PMT's
- one test embankment

The first series of CPT's, 1-2 weeks after completion of the job did not exhibit any improvement, the ratio between penetration resistance after and before treatment being a few percent.

Consequently two additional series of tests were performed: one with CPT's, one with Menard PMT's. Results were as shown in Table 4 and Fig. 1.

Table 4: Acceptance test results (in MPa)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>CPT</th>
<th>Menard PMT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>q_u</td>
<td>E</td>
</tr>
<tr>
<td>2 - 6</td>
<td>0.8</td>
<td>1.5</td>
</tr>
<tr>
<td>6 - 12</td>
<td>&lt; 0.8</td>
<td>11 - 30</td>
</tr>
</tbody>
</table>

Fig 1. Comparison of CPT and PMT results before and after treatment at job site.
One can see that post treatment $q_c$ values are smaller than the original ones (as a matter of fact several types of instruments were used, which may also influence the results). On the contrary PMT E - moduli are 3 times higher in the upper layer and between 5 and 10 times higher in the lower layer.

PMT limit pressures only exhibit moderate improvement up to 3 times the pretreatment value.
The ratio $E_d/\gamma$ being between 12 and 19 means a highly overconsolidated soil. (Fig 2).

It is interesting to mention that the upper layer which, before treatment, would not show stable slopes even for small slope angles, could later sustain vertical slopes up to a depth of 5 m without observing ground water seepage.

Additional observation with the CPTU were as follows:
- same range of $q_c$ values as for CPT
- large values of $A_u$ during cone penetration:
  - $A_u < 0$ up to 6 m depth pinpointing an overconsolidated soil in which dilation occurs,
  - $A_u > 0$ below 6 m depth, as in a typical clay layer.

- large stabilised values of $A_u$ after conventional dissipation which are not due to an excess pore pressure in the soil but may be due to the combination of two factors. Cone penetration creates shear stresses which in turn temporarily increase the excess pore water pressure in clayey soils. Further during dynamic consolidation tamping gas microbubbles existing in the lower silt layer formation may migrate to form bigger bubbles due to a temporary permeability increase caused by treatment. These bubbles will move upwards but they are trapped below the less pervious upper layer. Consequently their density of accumulation decreases when depth increases like the resulting stabilised $A_u$ (Fig 3).

The test embankment 11 x 11 m was extended over 4 stone piers to simulate the final loading configuration for the proposed shopping center. For a load equal to 40 kPa too, the final settlement was estimated at 25 mm, i.e. 7 times less than the pretreatment value, confirming the post to pretreatment Menard E values ratio.

Further this value of 25 mm was considered a typically acceptable value to fulfill the differential settlement criterion.

It can be seen that here only static loading tests, represented by embankments and Menard PMT’s could help conclude that the treatment was effective.

The shopping center was inaugurated in Summer 1995 and structure has not shown any sign of distress since then.

Other examples could be given. Dumas et Morel (1995) gave tables summarising types of dynamic consolidation jobs and number of pressuremeter tests performed to date in Canada for this purpose.
CONCLUSION

Among the various techniques which can be used to carry out acceptance testing for soil densification, especially in formations exhibiting some fine fraction, static loading tests have been proved the more realistic. Among these tests, pressuremeter tests are cost effective as exemplified by a typical job of dynamic consolidation. Louis Menard could not have pioneered this technique without the invaluable help of his own pressuremeter.

REFERENCES

Gambin, M. 1991. Lateral static densification at Monaco, design, construction, and testing. Deep Foundation Improvements. ASTM, STP No. 1089, Philadelphia (Especially Table 1).
A new method to estimate the angle of internal friction of sand using a pressuremeter test

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ABSTRACT: An estimation method of the angle of internal friction using a pressuremeter test was newly proposed in this paper. The new method combines the Hughes et al method (1977) with the Ohta & Fukagawa method (1984). In order to validate the new method, the theoretical stress-strain response based on an elastoplastic model considering dilatancy and strain-softening etc. was applied to the new method. As a result, the effectiveness of the proposed method was verified.

1 INTRODUCTION

The pressuremeter test has been applied to estimate not only deformability but also strength properties of soils, e.g. the undrained shear strength of clay, c’u, and the angle of internal friction of sand, φ. Since the determination of φ of sandy soils is a difficult task for engineers, if φ can be easily derived from in situ tests, that would be useful information. Of course, cone penetration tests or the standard penetration test etc. are used to estimate φ. However, these are empirical estimation methods so that there is little mechanical background and this results in low reliability. Some estimation methods of φ from the pressuremeter test have been proposed so far (e.g., Brim, 1992). Hughes, Wroth and Windle (1977) proposed a method to evaluate the peak angle of internal friction, φu, but their method was to evaluate or assume one of the parameters required in their analysis. Ohta and Fukagawa (1984) have proposed an estimation method of φ for sands which shows almost no volume change at failure. Although the method is very simple and φ can be derived very easily, the usage of this method is much restricted considering the assumption. A new method to estimate φ of sandy soils is therefore proposed in this paper. The new method combines the Hughes et al method with the Ohta & Fukagawa method, and it was finally concluded that this method gives reasonable results.

2 ESTIMATION METHODS OF φ BY USE OF PRESSUREMETER DATA

2.1 Hughes et al method

Hughes, Wroth and Windle (1977) proposed the method to estimate φ considering the dilatancy characteristics of sandy soils. The material around the pressuremeter probe satisfies the following conditions: 1) perfectly drained condition, 2) deformation under axi-symmetry and plane strain conditions, 3) constant φ and the angle of dilation at failure. When the stress-strain data based on these assumptions are plotted on logarithmic scales, lnσ = lnεj, relations shown in Figure 1(a), where εj is the inflating pressure, and σj is the circumferential stress at the inner wall of the borehole. The gradient of the straight line, S, in logarithmic scales is given theoretically by the following equation:

$$S = \frac{(1 + \sin \phi) \sin \phi}{1 + \sin \phi}$$

where φ is the angle of dilation and φ is the angle of internal friction corresponding to the peak strength.

Moreover, Rowe (1962)'s stress-dilatancy equation is as follows:

$$1 + \sin \phi = \frac{1 + \sin \phi}{1 - \sin \phi} \frac{1 + \sin \phi}{1 - \sin \phi}$$

where φ is the angle of internal friction mobilized when continuing constant volume flow is occurring. The Hughes et al method is to estimate φ by use of Eq(1) and (2). However, since Eq(1) and (2)
Figure 1. An estimation method newly developed and the comparison with conventional methods

2.2 **Obata and Fukagawa method**

Obata and Fukagawa (1984) proposed a method to estimate $\phi$ at residual state, $\phi_{res}$, in which the volume change of the sample at failure is very small. Other assumptions required in the method are almost the same as the Hughes et al method. Final $\ln \sigma_T - \ln e$ relations are shown in Figure 1(b). If the gradient corresponding to the residual state is denoted as $S_{res}$, $\phi_{res}$ is theoretically obtained by the following equation:

$$\sin \phi_{res} = \frac{S_{res}}{1 - S_{res}}$$

(3)

If there is no volume change in shear, then $\phi=0$ in Eq. (1) so that the same equation as Eq. (3) can be derived from Eq. (1).

This method is effective in the estimation of $\phi_{res}$ and is also applicable to the materials which do not show much dilatancy and strain softening after failure such as loose sands since $\phi_{res}$ is nearly equal to $\phi_{cr}$ in these materials. However, in considering the usage of this method against actual sandy soils, the availability of this method is not very high.

2.3 **A new method to estimate $\phi_{res}$**

A new method proposed here is to combine Hughes et al method with the Obata & Fukagawa method. The basic assumptions are as follows: 1) perfectly drained conditions, 2) deformation under ad-symmetry and plane strain conditions, 3) having dilatancy at peak strength, but no volume change at residual state.

Based on these assumptions, $\ln \sigma_T - \ln e$ relations are shown in Figure 1(c). An important assumption to validate the new method is required, that is $\phi_{res}$ at residual state is nearly equal to $\phi_{cr}$. Some researchers refer to the relation between $\phi_{cr}$ and $\phi_{res}$ and regarded as $\phi_{cr}=\phi_{res}$ (e.g., Yagi, 1976, Fedda, 1982) so that the assumption seemed to be acceptable.

The determination procedure of $\phi_{res}$ is explained as follows:

$$\sin \phi_{res} = \frac{S_{res}}{1 - S_{res}}$$

(4)

$$1 + \sin \phi_{res} = \frac{1}{1 - \sin \phi_{res}} - 1$$

(5)

$$\sin \phi_{res} = \frac{1 - S_{res}}{S_{res} - 2}$$

(6)

where $S$ is the gradient at peak strength on $\ln \sigma_T - \ln e$ plots, which ordinarily corresponds to $e_{cr}=2^\frac{1}{2} \phi_{cr}$ and $S_{res}$ is the gradient at residual state, which ordinarily corresponds to $e_{res}=10^\frac{1}{2} - 15%$.

The merit of this method is that no extra experiment is required to estimate $\phi_{res}$ at sands, since all
3. EVALUATION OF NEW METHOD BY USE OF ELASTO-PLASTIC THEORY

In this section, theoretical stress-strain relations are derived based on the elasto-plastic constitutive model. This model also assumes dilatancy at plastic and strain softening regions and no volume change at the residual region. The new method to estimate $\phi_\psi$ will be applied to the theoretical stress-strain relations obtained in this section.

3.1 Stress-strain relations based on elasto-plastic theory

A thick cylinder type model shown in Figure 2 is assumed under plane strain and axisymmetry conditions. The stresses applied to the inner and outer walls are $p_1$ and $p_6$ respectively, and the distance from the center to the inner and outer walls are $r_1$ and $r_6$ respectively. As the stress level of $p_1$ increases, the plastic region, strain softening region, and residual region are assumed to be generated in order. Critical stresses when the boundary was generated are $\sigma_{pl1}$, $\sigma_{pl6}$, $\sigma_{pl}$ for the plastic region, strain softening region and residual region respectively, and the corresponding distances from the center of the cylinder to each boundary are $r_{pl1}$, $r_{pl6}$, $r_{pl}$ in order.

Figure 3 shows the assumed stress state and volume change for each region. The input parameters of this model are Young's modulus $E$, Poisson's ratio $\nu$, the outer pressure to thick cylinder $p_6$, $\phi_\psi$ at plastic region $\phi_\psi$, $\phi$ at residual region $\phi_{res}$, the gradient of $q$ at strain softening region $g_e$. The stress-strain relations were finally formulated based on the equilibrium conditions, the continuity of the stress and strain etc. Please refer to Fukugawa, Maro and Hino (1995) about the details of the modeling procedure.

3.2 Verification of new method by use of elasto-plastic model

The stress-strain relation corresponding to the pressuremeter test was first obtained from the proposed elasto-plastic model and then the new method was applied to the calculated pressuremeter expansion curve.

Input parameters are $\phi_\psi = 40(\text{deg})$, $\phi_{res} = 30(\text{deg})$, $\nu = 0.3$, $g_e = 180(\text{kN/m}^2)$, $r_1 = 3(\text{cm})$, $p_1 = 98(\text{kN/m}^2)$. Other parameters required in the theory can be determined by these parameters. Figure 4 expresses the stress-strain relations obtained from the theoretical equations for each region. Figure 5 is obtained when the stress-strain relation is expressed in logarithmic scales. The gradients corresponding to the plastic and residual regions are demonstrated in Figure 5, and then from Eq.(4),

$$\phi_{\text{new}} = \sin^{-1} \left( \frac{S_m}{1 - S_m} \right) \sin^{-1}(0.5) - 30$$

and this value coincided with the initial value.

Next $\phi_{\psi}$ is replaced with $\phi_\psi$, and Eq.(6) is applied, then the following relation was obtained:
When the data are plotted on logarithmic scales:

$$\ln p = \frac{1 - K_2}{n + 1} \ln (\varepsilon_f - 1) + C$$  \hspace{1cm} (C: constant, plastic region) \hspace{1cm} (12)

$$\ln p = \frac{1 - K_2}{2} \ln (\varepsilon_f - 1) + C$$  \hspace{1cm} (C: constant, residual region) \hspace{1cm} (13)

Accordingly when the stress-strain data obtained from the pressuremeter test are arranged by logarithmic scales, the straight lines having the following gradients are theoretically obtained at the plastic and the residual regions:

$$S = \frac{1 - K_2}{n + 1}$$  \hspace{1cm} (plastic region) \hspace{1cm} (14)

$$S_{\text{res}} = \frac{1 - K_2}{2}$$  \hspace{1cm} (residual region) \hspace{1cm} (15)

Moreover the following equations are obtained from Eq (11):

$$S = \frac{(1 + \sin \delta) \sin \phi_f}{1 + \sin \phi_f}$$  \hspace{1cm} (plastic region) \hspace{1cm} (16)

$$S_{\text{res}} = \frac{\sin \phi_{\text{res}}}{1 + \sin \phi_{\text{res}}}$$  \hspace{1cm} (residual region) \hspace{1cm} (17)

Eq (16) coincides to Eq (1) proposed by Hughes et al. Since the Ohta & Fukagawa method assumes that there is no volume change at failure, when $\delta = 0$ is substituted in Eq (16), this will give Eq (3) if it can be assumed that $\phi_f = \phi_{\text{res}}$. This assumption will result in Eq (17) at the residual region.

It could be verified from these results that the new method, which uses the Hughes et al method at the plastic region showing dilatancy properties and the Ohta & Fukagawa method at residual region showing no volume change, will give reasonable estimated values of $\phi_f$ and $\phi_{\text{res}}$.

4 CONCLUSIONS

The main conclusions obtained from this study are as follows:

1. By use of the Hughes et al method and the Ohta & Fukagawa method, a new method to estimate the angle of internal friction $\phi_f$ was proposed. In this method, since $\phi$ at peak strength is determined by the Hughes et al method and $\phi$ at residual state is determined by the Ohta & Fukagawa method, no extra experiment is required to estimate $\phi$ of sand.
2. The pressuremeter expansion curve was formulated based on a kind of elasto-plastic model. The model also assumes dilatancy at plastic and strain softening regions and no volume change at residual region. The new method was applied to this theoretical expansion curve (logarithmic scales) and the result showed the effectiveness of the new method.

REFERENCES


A new approach to the Menard PMT parameters

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ABSTRACT: By comparing results obtained with a Menard pressuremeter and with a self-boring pressuremeter it is possible to propose another way to analyze the volume expansion curve and to derive both the Menard G modulus and the Menard limit pressure. These new definitions make easier a quality control procedure for the test involving a data logger and simple softwares.

The Menard pressuremeter test is the geotechnical in situ test which is the more often carried out prior to the design of foundations in France. Several hundreds of thousands tests of this type are performed each year, involving about 500 crews of site investigation.

This attraction comes from the fact that this test performed with a tricell probe can reasonably yield a G modulus and a limit pressure and that Louis Menard in the early 60's developed rules to design foundations on the basis of criteria for general stability and allowable differential settlements involving this limit pressure and the Menard G (or E) modulus (Baguelin et al. 1978, Briand 1992, Clarke 1995).

According to a recently proposed categorization of geotechnical in situ testing (Still 1993) the Menard pressuremeter is a class 2 equipment since the soil parameters obtained are derived from a simple theory: the expansion of a cylindrical cavity in an elastic-plastic medium. The G value is obtained during that part of the test where of the strain relationship is assumed to be elastic, although it has been shown that out of the times the soil behaviour is micro-plastic (Menard & Rousseau 1962, Gambin et al. 1994). Further, to simplify the derivation of the limit pressure \( P_e \) which would compete to choose between theoretical assumptions such as plastic strain hardening or strain softening models, the Menard conventional value is the pressure at which the volume of the original cavity has doubled at the level of the central cell.

These features are allowed for in the French AFNOR standard P 94-110 of July 1991 and the ASTM standard D 4719-87 (Figure 1). The pressuremeter curve is divided into 3 parts, according to the analysis of the creep curve obtained by plotting the creep data \( V_m(P_m) - V_m(P_{10}) \) where \( P_e \) is one of the pressure levels applied, \( V_m \) and \( V_m \) being the readings respectively 60 and 30 seconds after the \( P_e \) level has been reached.

\[
G_{meas} = \frac{(V_{m0} + V_{m1}) \Delta P \Delta V}{\Delta V}
\]

(1)

where: \( V_c \) is the probe central cell volume at rest, \( V_m \) is the mean volume reading in the \( P_1, P_2 \) range \( \Delta P \) and \( \Delta V \) are given on Figure 1.

![Figure 1. A typical Menard pressuremeter curve with the creep curve (ASTM 1987).](image-url)
for \( P > P_e \), micro-plastic soil response develops around the probe.

According to this analysis, \( P_e \) is called the creep pressure. Beyond \( P_e \), the volume readings do not stabilize as a function of time. The conventional Menard limit pressure is attained when the volume of the cavity is twice as much as the volume for \( P_e \).

It must be noted that both standards specify the types of drilling tools and probe emplacement techniques to be used so as to get reliable and repeatable tests results.

The present drawback, being naturally that more and more tests may not be adequately run, it becomes necessary to envision a stricter means of quality control for the test.

The recent survey of several hundreds of tests shows 2 main results:
- the number of pressure increments is often much less than the suggested average of 10;
- the volume expansion of the measuring cell never reaches twice the volume of the original cavity and this often by far.

The origin of these deficiencies is the operator's willingness to avoid probe bursting which would result in site investigation delays. Then the limit pressure is underestimated, whatever method of extrapolation is used.

1 ANOTHER WAY TO ANALYZE THE PRESSUREMETER CURVE

In the early 70's more elaborated equipments were developed in France and in the UK: the self boring pressuremeters. The French one called PAF (for pressiomètre auto-forçeur) still measures volume changes of the cavity by displacing water (Jezquel et al, 1988) and the British one called SBP measures radial displacement by sprung feeler arms (Wood & Hughes 1972). Although in France this equipment is mostly considered as a research tool, its greater merit is to help understand the Menard pressuremeter test.

On Figure 2 is shown an expansion curve obtained with a PAF in a soft clay, exceptionally using the same procedure as with a Menard test, i.e. by applying pressures increments and reading volumes as a function of time over a period of 60 seconds. The curve is shown on a typical Menard P-M-T graph since pressure is the variable and volume the function. As it can be seen the creep curve (Figure 3) does not exhibit the same shape as for a Menard pressuremeter test: the curve continuously increases during the whole test, without any kink like the second one shown on the creep curve of Figure 1. Early F.E.M. analysis of the test (Waschowski 1976) lead to the conclusion that this kink was to be more ascribed to the probe geometry than to the soil response against an inflating cylinder of infinite length.

Figure 2. A stress controlled PAF curve.

Figure 3. The creep curve for Figure 2 test.
the volume expansion curve has a concavity towards
the increasing volumes.

There is only one singular point on the pressure-
meter curve and this is where the soil behaves the
closest to its non remoulded condition.

Consequently the conventional Menard G modu-
lus of the soil must be derived in the vicinity of point
6 on the curve (Figure 4). However, in order to get a
sort of average value for G, instead of simply using
ΔV/ΔP measured between points 5 and 6 in formula
(1), a value which may be subjected to a measure-
ment bias, it is suggested to consider an extended
section of the volume expansion curve from point 4
to point 7, for which the ΔV/ΔP data fall between
the minimum value and 1.2 times this minimum va-
value. This operation can be processed by a simple
software.

This way to derive the G (or E) modulus is pro-
posed to AFNOR - the French Standard Organiza-
tion - for the next revision of the Menard pressure-
meter practice standard. It is already exemplified in
Figure 4.3 of Eurocode 7, part 3 « Geotechnical de-
sign assisted by field tests ».

2. ANOTHER WAY TO DERIVE THE LIMIT
PRESSURE

With any PAF or SBP equipment it is possible to
simulate one of the various effects of preboring.

If the cutting shoe is slightly over-sized in diame-
ter, that is if the shoe side protrudes 0.1 , 0.5 or
1 mm beyond the flush shaft (Figure 5), then the vo-

Figure 4 Proposed readings range to obtain
the Menard modulus.

Conversely in most soils ΔV/ΔP values for each
segment of a Menard pressuremeter curve can be
plotted as shown on Figure 4 (Jerequiel et al. 1974).
In the zone where ΔV/ΔP is minimum there is an
inflexion point on the volume expansion curve. Let
call P<sub>E</sub> the horizontal coordinate of this point, then:

- for P < P<sub>E</sub> the pressuremeter curve exhibits a
downward concavity
- for P > P<sub>E</sub> the pressuremeter curve exhibits an
upward concavity.

This means that the volume expansion curve
should only be divided into 2 parts:
- in a first section, P < P<sub>E</sub> , a soil ring around the bo-
rehole which was remoulded by the drilling op-
terations and stress relief is pressed between the probe
cover and the non affected body of soil in the dis-
tance, the expansion curve has a concavity towards
the increasing pressures,
- in a second section, P<sub>E</sub> < P , the ring of remoulded
soil cannot exhibit any more volume readjustment
and the response of the non remoulded soil appears :
lume expansion curves exhibit reverse curvatures as the Menard curves (Figure 6). This effect is due to a slight stress relief of the soil around the borehole. How do the G values for these 3 curves compare with that one obtained for the true PAF curve, i.e.

G = 9.2 kPa ?

Using the inflexion point method described above, but this time for strain controlled tests, the inflexion point being located by the horizontal arrows, the following values can be derived:

G = 9.8 kPa for a 0.1 mm over-sized cutting shoe
G = 7.0 kPa for a 0.5 mm over-sized cutting shoe
G = 5.3 kPa for a 1 mm over-sized cutting shoe

As it can be seen these G values decrease when the over-size increases but not as much as when compared to prebored pressuremeter G values.

Consequently stress relief is certainly not the major factor of soil remoulding. More disturbance is probably caused by the drilling tool tearing off action.

As it can be seen this overall disturbance does not seem to affect so much the conventional limit pressure (for ε = 100 %, i.e. far away from the Figure 6 graphs limit). This is in agreement with the proposed new view of the pressuremeter test.

Incidentally the analysis of these 4 curves (Figure 6) to attempt deriving the horizontal pressure at rest \( \sigma_{h0} \) exemplifies the uncertainties regarding its value. First there is no reliable confirmation that the value \( \sigma_{h0} = 185 \) kPa on the PAF curve is equal to the undisturbed soil horizontal pressure at rest and this comment would also apply for radial displacement probes. Then on the other 3 curves performed at the same depth, the horizontal pressures at rest, following the Briand’s method (1992) are:

\( \sigma_{h0} = 134 \) kPa
\( \sigma_{h0} = 140 \) kPa
\( \sigma_{h0} = 143 \) kPa

which are all different from the PAF value.

Neither the AFNOR standard nor the ASTM one quote any recommendation to derive the horizontal pressure at rest from a Menard pressuremeter since any value would be even more dubious than with the PAF or the SIBP. Strangely enough both standard still assume that one can have some knowledge of either the volume for which or the pressure at which the probe cover comes into contact with the borehole wall in its remoulded state. Since this knowledge stays messy the idea comes to slightly alter the definition of the Menard limit pressure.
3 PROPOSALS FOR QUALITY CONTROL CRITERIA

As for any test which is operator dependent the client needs to receive some quality assurance that the tests are correctly performed.

Three classes of factors govern the quality of the test:
- the metrological factors: pressure and volume measurement systems must exhibit the proper accuracy level, which here cannot be high, simply because the volume expansion of a cavity in a non homogeneous soil can only yield an averaged curve. Also pressure and volume measurement systems must be regularly checked;
- the loading process factors: number of increments up to limit pressure and duration of each pressure level are stated in the standards as well as the pressure lag at the monitoring box level between the central measuring cell and the guard cells according to the design of the probe, the quality of rubber membrane used and to the depth of the test. However expansion up to the limit pressure is mandatory, consequently this value is seldom reached to prevent probe cover failure. It must often be derived by an extrapolation procedure which has to be carefully validated;
- the drilling factors: they are the most difficult to appreciate. Recommendation is not to try to perform a test on an undisturbed soil but to follow drilling rules so as to test a soil in a similar condition to that one Menard would have obtained at the time he elaborated his design rules. Also the test must be performed as soon as the cavity is completed.

Since most of the time the Engineer only gets a log of the G (or E) and $P_2$ values as a function of depth for each borehole, he has no assurance that the standard rules were fulfilled. This is why several additions to the standard are suggested:
- The pressure and volume readings must be recorded against time with an approved computerized data logger. This logger must contain a fully secure clock, a printer, and involve a memory card. Such a logger has already been proposed by French manufacturers for the last 6 years. It has been proved to be very effective and also very useful for the training of new operators (Gamba & Pot 1995).
- Objective criteria for Menard PMT quality assurance must be adopted, some of them based on the analysis of the logged data, to check that the standard procedure is strictly followed, others on the analysis of the cavity expansion curve. Among these criteria one can quote:
  - existence or not of pressure calibration and volume calibration performed prior to testing;
  - how much time elapsed between drilling and testing (this criterion can be better appreciated when the drilling rig is fitted with a drilling data logger, common feature in France presently);
  - pressure lag value between central cell and guard cells circuits;
  - number of pressure increments;
  - was expansion of the cavity permitting the extrapolation of the curve so as to obtain $P_2$? It can be noted that this criterion is easier to check and to be accepted by the contractor than a criterion based on the maximum allowable bore hole diameter.

Each penalty can have a certain weight and partially invalidate the test. One large weight penalty or several small weight penalties may fully invalidate a test.

This invalidation procedure must be carried out by an approved purpose built subroutine which is part of the Menard PMT software.

This addition to the standard is also proposed to AFNOR.

CONCLUSION

The present proposals are prompted by the development of new technologies. They are not aimed at altering the values of the parameters derived from a Menard PMT but to make easier the set up of quality control criteria softwares. Several surveys have shown that values obtained by the proposed methods are not much different from those obtained by the previous methods, and regarding the G modulus the values are probably closer to the exact value.

Further it was checked that these values do not make obsolete the Menard design rules, especially in their latest version (Gamba & Frank 1995).

The quality control / quality assurance of the Menard PMT requests that field data be recorded by a computerized logger, as is now commonly used for the CPT and will become generalized for the SPT in a near future.

The proposed system of penalty does not mean that any invalidated test at a job site a discount is not paid to the contractor, but that above a given number of these invalidated tests a discount may be applied to the final invoice. This number may be agreed between the client and the contractor as a function of the difficulties which can be met in the performance of the tests in some specific formations such as gravelly soils.
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Strain level dependency of pressuremeter shear modulus of clay

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ABSTRACT: The pressuremeter test, especially the self-boring type pressuremeter (SBP) test, is expected to give useful information on the mechanical properties of ground. However, there are some unknowns even in the evaluation of the measured deformability so that the aim of this paper is to propose a principle of evaluation of test results, that is, the strain level dependency. Many SBP tests were carried out at both a normally consolidated and at an over consolidated clay ground, and the test results were compared with those from pre-boring type pressuremeter (PBP) tests, elastic wave exploration and unconfined compression tests based on the strain level dependency.

1. INTRODUCTION

The Pre-Boring type Pressuremeter (PBP) and the Self-Boring type Pressuremeter (SBP) tests are typical pressuremeter testing methods. It is well known that the cavity stress - cavity strain curve (so-called pressuremeter expansion curve) derived from PBP tests is apparently different from those of SBP. The expansion curves obtained from PBP have a clear linear part known as a quasi-elastic region, while the expansion curves from SBP show a strong non-linearity, especially in the initial part. Although these differences are annoying, there are few studies which reasonably explain the difference. The first aim of this paper is therefore to explain the difference of the deformability derived from SBP and PBP from the view point of "strain level dependency". The second aim of this paper is to make clear the interrelationship between shear moduli obtained from some testing methods based on the strain level dependency.

2. TEST FIELDS

2.1 Normally consolidated clay ground

The SBP tests were carried out at a ground near Yokohama in which the thickness of alluvial clay is about 30m. In this field, so far various kinds of laboratory and in-situ tests have been carried out (Taniguchi et al., 1994), so that this ground can be regarded as a good reference field. Some laboratory tests including unconfined compression tests, triaxial tests (UU,CKoU) and consolidation tests were conducted to investigate the deformation and strength properties of the ground, and also some in-situ tests including PBP, cone penetration tests, dilatometer tests and elastic wave exploration were carried out. From these test results, it was made clear that the ground is almost under normally consolidated condition. Figure 1 shows the profiles of the test ground. It can be seen that the ground condition is very simple and clear as the deformation and the strength properties increase almost linearly with depth.

2.2 Over consolidated clay ground

The profiles of an over consolidated clay ground tested in Yokohama are shown in Figure 2. Some tests were carried out at the depth range from 15 to 30m. The field condition changes at about 25m depth, that is, the upper layer includes more sandy soils more than the lower layer, although the ground can be almost classified as silty clay.

3. PRESSUREMETER TESTS

3.1 Testing device

The SBP used in this study was CAMBRIDGE INSTRUMENT MARK VII self-boring pressuremeter. The SBP is a mono cell type and the diameter and the length of the inflating probe are 83mm and 624mm respectively. The PBP used in this study is a typical mono cell type.
Figure 1. Profiles of test field of normally consolidated clay ground

## 3.2 Testing procedures

(1) Normally consolidated clay ground

10 SBP tests and 12 PBP tests were carried out at various depths shown in Figure 1. The SBP tests consisted of two kinds of tests. PBP simulation tests were conducted to investigate the effect of stress release due to pre-boring. Other tests are standard tests. The cutting shoe used in the simulation tests, shown in Figure 3, has a 2 mm larger radius (equivalent to 5% cavity strain). The loading method of SBP is stress controlled and 2 min holding time, but stress controlled with constant stress increment. The loading rate was set as 20 kPa/min to maintain an undrained condition.

(2) Over consolidated clay ground

8 SBP tests were conducted as shown in Figure 2. The SBP tests consisted of two kinds of tests which
have different holding time, 30 minutes or 16 hours. However, there was less difference between the results, so that the result was shown in Figure 2 without distinguishing the test type. The loading method of SBP was almost the same as those in the normally consolidated ground.

4. DEFINITION OF SHEAR MODULUS

4.1 Pressuremeter tests

In the case of the pressuremeter, the shear modulus \( G \) is expressed by \( G = \frac{\Delta e}{e_a} \), where \( \Delta e \) : the increment of cavity stress or infilling pressure of the probe and the suffix 0 denotes the values at the borehole wall, and \( e_a \) : the initial radius of the borehole. Consequently, the shear modulus \( G \) from the pressuremeter tests are categorized in the following three ways according to the determination of \( \Delta e \) and \( e_a \) (see Figure 4): 1) secant shear modulus \( G_s \), 2) tangent shear modulus \( G_t \), 3) cyclic shear modulus \( G_c \).

When \( G \) is determined from the gradient of the straight line connecting two points on the expansion curves, the \( G \) is called secant shear modulus \( G_s \). This was well used to determine \( G \) from the quasi-elastic region of the expansion curves of PBP. Of course, this method is sometimes applied to obtain \( G \) from SBP, but engineers are confused about which two points on the SBP expansion curves should be selected because the expansion curves show strong non-linearity. Therefore, \( G \) of SBP was determined as the tangent shear modulus \( G_t \) in this paper. It is well recognized that the repeated or cyclic loading process of pressuremeter tests gives linear and stable stress-strain relations so that the cyclic shear modulus \( G_c \) is defined.

Figure 4 also shows the corresponding strain for each shear modulus. When the strain levels are determined from these strains, \( \varepsilon_s \), \( \varepsilon_t \), and \( (\varepsilon_a - \varepsilon_s) / 2 \) should be selected for \( G_s \), \( G_t \), and \( G_c \), respectively.

4.2 Elastic wave exploration

The shear modulus \( G_s \) obtained from the elastic wave exploration was determined from \( G_s = \frac{V_c^2}{g} \), where \( V_c \) : elastic wave velocity, \( g \) : gravitational acceleration. The \( G_s \) from the elastic wave exploration is thought to correspond to infinitesimal strain level (about 10^-4 order) and was usually employed in considering the dynamic deformation of soils.

4.3 Uncrushed compression test

Secant shear modulus \( G_{0s} \) corresponding to half of the undrained shear strength was used based on the data from uncrushed compression tests. \( G_{0s} \) is related to the so-called \( E_{0s} \) by \( G_{0s} = E_{0s} / \varepsilon_{0s} \).

5. TEST RESULTS AND CONSIDERATIONS

5.1 Pressuremeter expansion curves of SBP and PBP

Typical examples of SBP and PBP tests carried out at the same depth are demonstrated in Figure 5 as the relationship between the loading pressure \( \sigma_p \) and the cavity strain \( \varepsilon_{cp} \). In this figure, the reference state of the strain was at the start of inflation of the probe for SBP, while that for PBP was at the state corresponding to the static earth pressure \( \rho_p \). PBP showed good linearity in the so-called quasi-elastic region (about from 0-3 \% strain range), while SBP showed non-linearity in
difficulty in evaluating $G$ from SBP. Therefore, tangent shear modulus $G_s$ was determined from SBP instead of secant shear modulus $G_s$ such as in PBP and they were compared with each other based on the same strain level. The strain level corresponding to the $G_s$ of PBP was regarded as the average strain of the quasi-elastic region of PBP expansion curves. The reference states of the strain was both assumed to be at the state related to the static earth pressure $p_0$.

Figure 6 is a typical example of comparison of $G_s$ from SBP, $G_s$ and $G_s$ from PBP, in which the $G_s$ were generalized by $G_s$ obtained from the elastic wave exploration tests. In this figure, SBP gave many plots because $G_s$ was determined from the tangent of expansion curves and each plot corresponded to a strain level. As clearly shown in Figure 6, the relationship between $G_s/G_s$ and $e_{0pp}$ had a tendency toward linearity in log-log scales at strain ranges from about 0.1% to 4%. $G_s$ and $G_s$ obtained from PBP were also plotted in Figure 6. Both values showed good accordance with $G_s$ values based on the same strain level.

Figure 7 shows the relations between $G_s$ from PBP and $G_s$ from SBP. The strain levels corresponding to each PBP and SBP data set are of course adjusted to become the same. Both values show good accordance so that it can be clearly concluded that the strain level plays a very important role in evaluating the deformability.

5.3 Effect of overburden pressure on strain level dependency

Since a linearity could be confirmed in $\ln(G_s/G_s)$ - $\ln(p_0)$ relations as shown in Figure 6, the effect of the overburden pressure on the gradient of the linearity was next examined in the normally consolidated clay ground. The gradients of $\ln(G_s/G_s)$ - $\ln(p_0)$ data con-
responding to each test depth determined by the least square method are plotted against the test depth in Figure 8. The gradients were almost distributed between 0.8 and 1.0, and it can be concluded that the linearity in \( \ln(G) \sim \ln(e) \) \( \ln(\varepsilon) \) relations is almost constant irrespective of the overburden pressures. Zen et al. (1987) pointed out that if a clay ground has a plasticity index beyond 30, the \( \ln(G) \sim \ln(\varepsilon) \) \( \ln(\varepsilon) \) shear strain relation does not depend on the effective overburden pressure. The result in this study assures their conclusion.

The results from O. C. clay are also demonstrated in Figure 9. The values of the gradients of \( \ln(G) \) \( \ln(\varepsilon) \) \( \ln(\varepsilon) \) relations are slightly different of those in Figure 8. This may be caused by the difference of O.C.R. and soil type.

5.4 Repeated loading process

In the case of SBP, 3 times of repeated loading were carried out at about 0.5, 5 and 10% cavity strain. Figure 10 shows the comparison of the shear moduli corresponding to the initial and the repeated loading processes. The reference states of strain for the repeated loading processes are just where the reloading starts. It can be seen from this figure that these shear moduli show good accordance each other in strain ranges from about 0.1 to 4% cavity strain. Accordingly, if the test quality of SBP is good, the information from the initial loading process seems to suggest the response at the repeated loading processes.

5.5 Simulation of PBP by use of SBP test apparatus

Two SBP tests shown in Figure 1 were the simulation tests of PBP by use of SBP test apparatus. In these tests, the diameter of the cutting hole was a slightly greater than that of the SBP prove. A typical example of the PBP simulation tests was shown in Figure 11. Compared with the normal expansion curves related to other standard tests, it can be seen that the expansion curve obtained from the simulation test is similar to those of PBP tests, in which a clear linear part could be confirmed in its initial part. Since the simulation tests were conducted for the borehole wall with 5% greater diameter than the SBP stress prove, the inner wall of the borehole would swell at nearly the initial inner wall of the prove. This means that the borehole wall would swell at
range of 5% would be a reloading process against the loosened surrounding ground. This phenomenon resembles the so-called "quasi-elastic response" of conventional PBP tests. Although $G$ of PBP simulation tests is a slightly greater than the value from $G$ of PBP tests, this maybe caused by the difference of the degree of the borehole wall disturbance.

5.6 Comparison of $G$, $G_{opt}$ from SBP and $G_{opt}$ from unconfined compression tests

The result of comparison between $G$, $G_{opt}$ from SBP and $G_{opt}$ from unconfined compression tests was shown in Figure 12. This figure includes the data from normally consolidated and over consolidated clay grounds. In this figure, the strain levels of SBP were adjusted to the average strain level corresponding to $G_{opt}$. As clearly shown from this figure, when both values were compared based on the same strain level, both values showed good accordance irrespective of the ground stress history. But if mechanically clear explanation is required for this accordance, there are many issues which must be solved, such as the effect of stress release or disturbance etc., in the unconfined compression tests.

6. CONCLUSIONS

The main conclusions obtained from this study are as follows:

1. When the stress-strain data from the self-boring type pressuremeter (SBP) tests are plotted based on the strain level, the result was almost arranged as a unique straight line in logarithmic scales for the range from about 0.1 to 4% cavity strain, and the data from pre-boring type pressuremeter (PBP) tests could be also plotted on the straight line. This linearity was almost constant irrespective of the overburden pressure for normally consolidated clay ground.

2. The shear moduli $G$ obtained from repeated loading processes of SBP showed good accordance with $G$ from the initial loading process.

3. The so-called quasi-static region of PBP seemed to be a "reloading process" from the loosened state based on the simulation tests of PBP by use of SBP test apparatus with cutting shoe having a slightly larger diameter than the probe.

4. An almost unique relationship between the shear moduli and the strain level was obtained irrespective of the testing methods and the ground stress history. This means the strain level is a very important factor to estimate the deformability of the ground.

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Pressuremeter testing for drilled shafts in gravelly clay

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ABSTRACT: Underlying the proposed foundations at the Nooksack River, there is a substantial thickness of gravelly clay. In these gravelly clay materials, it was very difficult to obtain undisturbed samples from which reliable test data could be obtained to determine the likely settlement characteristics. This paper describes the results of deep pressuremeter tests (33m-46m) conducted in these gravelly clays to obtain an indication as to whether the ground was in an overconsolidated state and to determine the likely elastic settlement of the proposed drilled shafts.

1 INTRODUCTION

This paper outlines the salient features for the foundation investigation for proposed bridge across the Nooksack River in Washington State.

The foundation conditions underlying the proposed Nooksack river bridge consist of an upper 30 m of recent gravel and sand alluvium. Below this zone is a thick layer of a dark grey clay (glacio-marine drift) with some interbedded silt layers near the top of this zone. These conditions continue to the bottoms of the borings at over 80 m below the ground surface. The material in this lower layer contain a large quantity of gravels, which makes conventional sampling exceedingly difficult.

Laboratory tests on the samples obtained in the initial investigation indicated the possibility of the clay being in a normally-consolidated state. If this was in fact the case, the likely settlements of any foundation supported within this lower layer would be unacceptable and foundation extending to 60m or more in depth would be necessary.

In view of the cost implications of placing the foundations at 60m or more below the surface, and the uncertainty of the laboratory test results, a load test was proposed to assess the settlement behavior of a drilled shaft founded in the gravelly clay layer at the 35-40m level.

When the proposed costs for this load test were found to be unacceptably high, a second attempt was made to define the material properties more accurately using in-situ testing techniques and improved sampling methods. The supplementary investigation consisted of drilling and coring, with an unsplit core liner, and performing three pressuremeter tests. In addition a 3m length of cone trace were obtained in the gravelly clay from 30 to 46 m in depth. This depth was the desired bearing depth of the foundation if testing indicated overconsolidation of the gravelly clay layer. In addition index and physical laboratory tests were completed on samples obtained in this new boring. (Shannon and Wilson 1997)

In-situ testing was carried out over two days during this field investigation. The results of all of the testing in the second phase were remarkably consistent. The laboratory tests on the core extruded from the unsplit core liner, the short cone trace, and the three pressuremeter tests all indicated that the underlying glacio marine drift was indeed over-consolidated, to the extent that the calculated settlements would be within acceptable limits.

This paper outlines the results of the pressuremeter data as they apply to the design of the proposed drilled shafts.
2 THE PRESSUREMETER

The pressuremeter used at this site was a single-cell pressuremeter, in which the pressures and displacements were measured electronically inside the instrument. With electronic measurements taken inside the pressuremeter there is no limitation on the depth to which tests can be conducted. The pressuremeter was expanded by manually regulating the flow of compressed nitrogen to the bladder. During the pressuremeter tests, all of the data was displayed and stored on a computer. The schematic details of the instrument are given in Figure 1.

The hole for the pressuremeter was drilled using a NQ core barrel. The recovery from the core barrel showed that there was over 10% gravel imbedded in the clay.

Three pressuremeter tests were undertaken at depths of 33m, 38m and 46m, for a total testing time of less than 12 hours from the time the hole had been advanced to the top of the clay level at 30 m depth. Figure 2 is a plot of the test at 46m depth.

The tests were conducted to try and determine the following:
- An indication of the limiting pressure, $P_{\text{lim}}$, i.e. the maximum pressure at which indefinite expansion would occur.
- An indication of the existence of creep or consolidation in the gravelly clay.
- An indication of the modulus that might be appropriate to assess the likely settlement of the drilled shafts.

To try and achieve these objectives within the very limited time frame, the three tests were conducted in a similar manner. The pressure was continuously increased until the borehole wall had moved outward by about 2%. The pressure was then held constant for five minutes prior to an unload-reload cycle. This procedure was repeated at least twice, until a strain of between 10 and 12% was reached.

In view of the heterogeneous nature of the material, the results of the three tests were surprisingly similar. The standard pressuremeter parameters defined from the test are presented in Table 1.

Table I Standard Parameters determined from the pressuremeter tests.

<table>
<thead>
<tr>
<th>Test</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td>33</td>
<td>38</td>
<td>46</td>
<td></td>
</tr>
<tr>
<td>Limit Pressure, $P_{\text{lim}}$ (kPa)</td>
<td>3,200</td>
<td>3,800</td>
<td>4,200</td>
<td>3,700</td>
</tr>
<tr>
<td>Undrained Shear strength (kPa)</td>
<td>500</td>
<td>750</td>
<td>970</td>
<td>740</td>
</tr>
<tr>
<td>Shear Modulus from unload-reload loop (kPa)</td>
<td>76,000</td>
<td>167,000</td>
<td>185,000</td>
<td>142,000</td>
</tr>
<tr>
<td>Measured Modulus (kPa)</td>
<td>44,000</td>
<td>38,000</td>
<td>44,300</td>
<td>42,000</td>
</tr>
</tbody>
</table>
Table 2. Material parameters determined from a cohesive model analysis.

<table>
<thead>
<tr>
<th>Test</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td>33</td>
<td>38</td>
<td>46</td>
<td></td>
</tr>
<tr>
<td>Cohesion (kPa)</td>
<td>500</td>
<td>740</td>
<td>870</td>
<td>703</td>
</tr>
<tr>
<td>Total Lateral Stress (kPa)</td>
<td>750</td>
<td>700</td>
<td>709</td>
<td>716</td>
</tr>
<tr>
<td>Secant Shear Modulus (kPa)</td>
<td>34,000</td>
<td>27,000</td>
<td>23,000</td>
<td>28,000</td>
</tr>
</tbody>
</table>

Figure 2. Pressuremeter test results at 46 m

An alternative method of analysis of the pressuremeter test is to determine the set of material parameters which can be used to derive an ideal pressuremeter curve which best matches the pressuremeter data obtained in the field. This is achieved by interactive computer graphics in which the parameters are manually adjusted to produce the bests match to the field data (Hughes 1997). If a single cohesive model is chosen a minimum of three parameters are required to describe the behaviour of the material—a linear secant modulus, from zero strain up to the start of shearing, a total in situ lateral stress and a value of the undrained shear strength. The material parameters which best match the pressuremeter data are summarized in Table II.

3 ULTIMATE LOAD ON THE DRILLED SHAFT

To provide an estimate of the lower limit of the ultimate load, consider a block of soil immediately below the base of a drilled shaft of the same diameter, and having a depth of one diameter (Figure 3). If the maximum radial stress on this element of soil is assumed to be the limit pressure determined from the pressuremeter (Table 1), then from the Molt’s Circle consideration of the stress immediately below the footing, the ultimate load will be given by:

\[ P_{vu} = 2S_u + P_{cm} \]  

(1)

For the average conditions given in Table I or II, \( P_{vu} \) is on the order of 5000 kPa.

The design working load for the drilled shaft is 8000 kN. Hence, on a shaft 2 m in diameter, if all of the load was transferred to the base, the stress on the soil would be 2500 kPa. Hence there is at least a factor safety of 2 on loading (assuming no side frictional support).

4 ELASTIC SETTLEMENT UNDER WORKING LOAD

An indication of the approximate elastic settlement...
shear stress. At a radial pressure of 1094 kPa — the pressure necessary to resist the static working load (Figure 4) — the soil on the vertical sides of the block is still in the assumed elastic range. Hence, a shear modulus of 28,000 kPa is appropriate to obtain an estimate of the elastic settlement.

The elastic displacement \( d \) for a rigid plate of radius \( r \) with a load of \( P \) deeply embedded in the ground with a shear modulus \( G \) is quoted by Poulos and Davis 1974 as:

\[
\frac{d}{r} = \frac{P}{12G+cr} \text{ for a Poisson's ratio of 0.3.}
\]

Therefore, for a 8000 kN load and a 2m diameter drilled shaft, the elastic displacement will be 20 - 25 mm.

As an alternative approach, an approximate indication of the settlement can be obtained by considering the compatibility of the average strains within the block of soil below the base of the drilled shaft.

The radial strain at 1094 kPa (Figure 5) is 0.7%. Hence, the average vertical strain will be 1.4%. For a block of soil 2 m in depth, the settlement will be 2
3 CONCLUSIONS

The above very simple approach to the determination of the ultimate load and the elastic settlement was obtained from a simple in-situ testing program, in which three remarkably consistent pressuremeter tests were obtained in the gravelly clay. The results from the subsequent laboratory testing on the higher quality samples, obtained during the second phase of the testing using an unsplit core liner, confirmed the conclusions derived from the pressuremeter testing that the gravelly clay was indeed over consolidated. The three metres of cone testing in the top section of the clay layer also confirmed the existence of material much stiffer and more overconsolidated than estimated from the initial site investigation.

6 ACKNOWLEDGEMENTS

The field work for this in-situ testing study was carried out under the direction of Mr. S. Puri, of Shannon and Wilson Inc. The drilling was performed by the Washington Department of Transportation. The authors are grateful to the director of the Washington State Department of Transportation for permission to publish the results of this site investigation.

7 REFERENCES


Development of automatic cyclic pressuremeter and its application to evaluation of deformation and strength of ground

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OYO Corporation, Fukuoka, Japan

ABSTRACT: Cyclic loading self-boring pressuremeter test device has been newly developed. An actuator, which generates sinusoidal wave pressure, is connected directly to a probe to obtain high precision load control. This device is applied for evaluation of non-linear deformation properties and also for cyclic strength of geomaterials. Preliminary results are summarized which indicated agreement to those obtained from laboratory triaxial test and borehole PS logging.

1 INTRODUCTION

In recent years, laboratory test techniques with high accuracy have been remarkably developed, and it has become possible to know more detailed properties of the deformation of geomaterials at the small strain level (Tainaka et al., 1994). And after Hyogoken-nanbu earthquake in Japan, there has been a need for detailed evaluation of liquefaction resistance of dense sand (Matsuo, 1996). In the background of these movements, the way to evaluate the properties of geomaterials by in-situ tests have been reviewed, and much attention is especially focused on self-boring pressuremeter tests, as well as PS logging (Koga et al., 1994). Authors have especially focused on cyclic loading self-boring pressuremeter tests, and have developed testing devices. Applicability of these devices and analysis methods have been studied through laboratory model tests and in-situ experiments (Fujitani et al., 1994, Koike et al., 1995). This paper presents outline of the cyclic loading self-boring pressuremeter, and also some examples of its applicability on evaluation of non-linear deformation properties and cyclic strength of geomaterials.

2 OUTLINE OF CYCLIC LOADING SELF-BORING PRESSUREMETER

2.1 Hardware

The measurement system is based on the RSBP (Clarke et al., 1989). It is improved to be equipped with a cyclic loading actuator by option. Data acquisition system and drilling bits are also newly developed. Figure 1 shows the configuration of the whole system, and table 1 shows the specifications. The actuator generates the cyclic load of the sinusoidal wave. A built-in piston is coupled to twin electric motors and it is precisely controlled under the signal from the pressure cell in the probe (Figure 2). The delay of the pressure transfer

![Diagram of cyclic loading self-boring pressuremeter](image_url)
Table 1 Specification of cyclic loading self-boring pressuremeter

<table>
<thead>
<tr>
<th>Device</th>
<th>Subject</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ingrounded RSDP System</td>
<td>Supply of pressure</td>
<td>Oil</td>
</tr>
<tr>
<td>Max Pressure</td>
<td>20MPa</td>
<td></td>
</tr>
<tr>
<td>Displacement</td>
<td>3 mm, half cell</td>
<td></td>
</tr>
<tr>
<td>Probe size</td>
<td>L = 120cm, φ = 7.36cm</td>
<td></td>
</tr>
<tr>
<td>Dan trigger</td>
<td>16 bits A/D converter, sampling ratio 2ms/msec</td>
<td></td>
</tr>
<tr>
<td>Twin rods</td>
<td>Outer rod φ = 50mm, inner rod φ = 28mm</td>
<td></td>
</tr>
</tbody>
</table>

**Actuator**

<table>
<thead>
<tr>
<th>Size</th>
<th>1 = 320cm, φ = 97mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum pressure amplitude</td>
<td>6MPa</td>
</tr>
<tr>
<td>Cyclic frequency</td>
<td>0.5 ~ 0.01Hz</td>
</tr>
<tr>
<td>Poisson capacity</td>
<td>82%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>1 = 343cm, φ = 6 ~ 7cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow rate</td>
<td>76 ~ 106 l/min (catalogue specification)</td>
</tr>
<tr>
<td></td>
<td>70 ~ 81 l/min (actual result)</td>
</tr>
<tr>
<td>Rotation</td>
<td>810 ~ 1100 rpm (catalogue specification)</td>
</tr>
<tr>
<td>Torque</td>
<td>38N·m (catalogue specification)</td>
</tr>
</tbody>
</table>

Figure 2 Cyclic loading system of actuator

or pressure losses produced by the viscous resistance in the pressurized medium can be minimized by connecting the actuator directly to the probe. Its outer diameter is made to the size of 97 mm by using the planet gear system (Figure 3). The actuator generates only cyclic loads and a pump on the ground supplies static load.

The mud motor is introduced instead of twin rods drilling method to promote handling efficiency especially in deep drilling and also to reduce bending of the inner rods which affects non uniform rotation of drilling bit. The mud motor is a kind of tip driving type of boring machine. Pressurized mud water rotates a spiral rotor which is connected to a bit. This time, the bit is removed and the rotor is connected to inner rod of the probe. While the mud motor has some problems, such as difficulty in delicate control of rotation of bit and pressure of mud water, disturbances of borehole wall have been quite minimum in field-proven performances in dense sands or in mud stone. When it is used at depths of 50 meters or more, the required time for a
test operation can be reduced by 50–60% compared to the time by using the twin rod.

2.2 Main field-proven performance

Table 2 shows the field-proven performance of the cyclic loading pressuremeter tests equipped with the actuator. Maximum depth of measurement has reached to 350 meters from ground surface.

3 NON-LINEAR DEFORMATION PROPERTIES OF GEOMATERIALS

Figure 4 shows example of pressuremeter curve measured for homogeneous sedimentary soft rock. Pressure controlled stage loading method is used. The cyclic load with eleven waves in a given amplitude is set as the first stage and the amplitude is gradually increased in following stage. A hysteresis loop of the applied pressure P – cavity strain εc for a cyclic load is also shown in the figure. Figure 5 shows time t – cavity strain εc relationship. Those figures demonstrate that strain measurement at in 10^{-6} % strain level is constantly obtained.

The unload/reload modulus G_{ur} is calculated for the hysteresis loop of the tenth wave at each loading stage. Then the secant shear modulus G_{sc} – shear strain γ relationship is calculated using method of Wood (1990). In this case, the hyperbolic function is applied for G_{sc} – γ relationship. Stress correlation function proposed by Robertson (1982) for drainage condition is not considered because coefficient of permeability is relatively small compared to sand and drainage condition during cyclic loading is obscure.

Figure 6(a) shows G – γ relationship of artificial soft rock (Kolke et al., 1995), which is obtained through model test in calibration chamber. Results of cyclic triaxial tests are also indicated. G_{sc,ref} is shear modulus calculated from ultrasonic velocity of shear wave. It is measured under isotropic triaxial condition. Good agreement for results of those tests is observed. One reason for good agreement is considered as ideal conditions of model test.

Figure 7 shows Cyclic frequency.0.05Hz

Applied wave for hysteresis loop is shown in Figure 4.

Figure 5 Record of cavity strain for time.

Hysteresis loop for the applied wave is shown in Figure 4.

<table>
<thead>
<tr>
<th>Site</th>
<th>Maximum depth (m)</th>
<th>Ground type</th>
<th>Mud motor</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>100</td>
<td>ms</td>
<td>not used</td>
</tr>
<tr>
<td>b</td>
<td>50</td>
<td>ms</td>
<td>not used</td>
</tr>
<tr>
<td>c</td>
<td>30</td>
<td>sand, clay</td>
<td>used</td>
</tr>
<tr>
<td>d</td>
<td>180</td>
<td>sand, ms</td>
<td>used</td>
</tr>
<tr>
<td>e</td>
<td>30</td>
<td>alt</td>
<td>used</td>
</tr>
<tr>
<td>f</td>
<td>30</td>
<td>alt</td>
<td>used</td>
</tr>
<tr>
<td>g</td>
<td>250</td>
<td>ms</td>
<td>used</td>
</tr>
<tr>
<td>h</td>
<td>20</td>
<td>gravel</td>
<td>used</td>
</tr>
<tr>
<td>i</td>
<td>20</td>
<td>sand</td>
<td>not used</td>
</tr>
<tr>
<td>j</td>
<td>20</td>
<td>Sand</td>
<td>not used</td>
</tr>
</tbody>
</table>

ms: mud stone  ss: sand stone  alt: alternation of sandstone and mudstone
Figure 6
(a) Relationship between shear strain and shear modulus for artificial soft rock in calibration chamber.
(b) Relationship between shear strain and damping ratio for the same test.
(after Koike 1995)

Figure 7(a) shows the relationship between the shear modulus and shear strain for the measurement of sandy mudstone (depth 110m). Pressuremeter curve is shown in figure 4. The ○ mark is the unload reload modulus $G_u$ and the bold curve is the sonic shear modulus $G_s$. The $G - \gamma$ relationship which is obtained from the cyclic triaxial test and shear modulus $G_s$, which is calculated from shear wave velocity in borehole PS logging are also indicated. These results disagree with each other. In actual ground, conditions such as effect of drilling in pressuremeter test, disturbance or release of in situ stress of cored sample are considered as reasons of difference. Figure 7(b) shows normalized shear modulus $G/G_0$ and shear strain $\gamma$ relationship. $G_0$ values are determined for each $G - \gamma$ relationship. In this case, $G/G_0 - \gamma$ relationships agrees for each type of test.

Figure 8 shows the relationship between the shear modulus $G_s$ and $G_0$ for various kinds of ground. It is clear that $G_0$ values are a little smaller than $G_s$ values however there are in good conformity. After arranging results of various measurements, such kind of tendency is generally observed and it is summarized as following relationship.

$G_0 \leq G_0(\text{cyclic SPT}) \leq G_0(\text{cyclic triaxial test})$

Figure 6(b) and figure 7(c) show relationship

Figure 8 Relationship of shear modulus calculated from velocity of shear wave and estimated from results of cyclic pressuremeter test for various ground
between damping ratio \( \beta \) and the shear strain \( \gamma \). The values also correspond to the cyclic triaxial test, and it means that the damping ratio can be measured with the cyclic pressuremeter test.

4 CYCLIC SHEAR STRENGTH OF DENSE SAND

4.1 Liquefaction strength of sand evaluated laboratory tests

In chapter 3, it is clarified that proper stress - strain relationship is obtainable from cyclic pressuremeter test. This means that it is possible to estimate cyclic shear strength of geomaterial using pressuremeter test. Pressuremeter tests are performed against dense sand (Fine content : 10-30%, SPT N value : 20–over50) and cyclic strengths are compared with liquefaction strength which are evaluated by laboratory cyclic triaxial tests.

Figure 9 shows relationship between stress ratio (\( \tau_{uu}/\sigma'_{v} \)) and corrected N value (N1) for various grounds studied by Matsuo(1996). Samples for cyclic triaxial tests are obtained by cone tube sampling with ground freezing method. Mark X are results which are just close to depth of pressuremeter tests marked as □.

4.2 Cyclic shear strength estimated from dilation angle obtained by monotonic loading pressuremeter test

Robertson(1982) proposed a method for estimation of liquefaction potential of sand using monotonic loading pressuremeter tests. Using pressuremeter test curve, he calculated tangent dilation angle at normal pressure at 100kPa and shear strain at 10%, and correlated it to relationship of liquefaction potential and dilation angle proposed by Vaid et al.(1980). Results obtained in this study are plotted as mark □ in figure 9. Those values are almost a half of those obtained by laboratory tests. Because the relationship of Vaid is based upon remolded samples, detail examination on effect of difference between in-situ undisturbed sample and remolded sample considering locality of soil is the subject for a future study.

4.3 Cyclic shear strength estimated from cumulative strain obtained by cyclic pressuremeter test

Robertson(1982) also focused on cumulative strain during slow cyclic pressuremeter test and correlated it to dilation angle and cyclic strength of Vaid. He suggested that 10 cycles should be performed maintaining an approximately constant strain amplitude of 0.2%. There is difference between Robertson’s cyclic loading condition and those adopted in this study, this method is not applicable to this study in a strict sense. Furthermore although stage loading with comparably rapid loading is adopted in this study, effect of static creep is included to the cyclic cumulative strain. As a consequence this method brought insufficient estimation of cyclic strength.

4.4 Cyclic shear strength estimated from cyclic pressuremeter test curve

Figure 10 shows a example of cyclic loading pressuremeter curve (Depth:12m, Fc:198s, SPT N value:32). The cyclic stage test is introduced in which loading pattern is similar to that of cyclic triaxial test. It is possible to define a skeleton curve and cyclic shear strength are
Figure 10  Cyclic pressuremeter test curve of dense sand

estimated from the curve. Analysis method is as follows.
(1) Focus on any cyclic number N at each loading stage in the cyclic pressuremeter curve and create a linking curve connecting peak points of each stage. This curve corresponds to the skeleton pressuremeter curve (see figure 10).
(2) Apply Palmer's method (Palmer 1970) to the curve and calculate shear stress \( \tau \) - cavity strain \( \varepsilon_c \) relationship.
(3) Determine maximum shear stress \( \tau_{max} \).
(4) Calculate stress ratio \( \tau_{max}/\sigma_0 ' \) which indicates a normalized cyclic shear strength to the cyclic number N. Average stress \( \sigma_0 ' \) is determined as \( (P - \tau_{max})/n \).

In this study, tenth cycle is applied for calculation. Results of stress ratio are plotted as mark \( \square \) in figure 9. However there is a scatter in those values, results of pressuremeter are larger than those of cyclic triaxial tests in general. To obtain best estimate of cyclic shear strength of sand considering drainage condition, more detailed experimental research on loading pattern, loading speed, drainage effect during cyclic loading are required.

5 CONCLUSION

The conclusions are as follows.
(1) The cyclic loading self-boring pressuremeter testing device which is developed in this study can be easily handled. Maximum depth of measurement has reached to 350 meters.
(2) This device is applied for evaluation of non-linear deformation properties of geomaterials and shear modulus and shear strain relationship that reaches to \( 10^5 \) strain level are able to be defined. This relationship agrees to result of the laboratory cyclic triaxial test.
(3) The shear modulus of the small strain level estimated from cyclic pressuremeter test almost agrees to those calculated from the in-situ velocity of shear wave.
(4) Methods for estimation of cyclic shear strength of dense sand are examined. To obtain best estimate of strength considering drainage condition, more detailed experimental research on loading pattern, drainage effect during cyclic loading, relation between stress ratio and dilation angle of undisturbed sample are required.

REFERENCES

Determining the characteristics of self-hardened suspension by pressuremeter tests

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ABSTRACT: The paper deals with changes of characteristics of self hardened suspensions with respect to time according to pressuremeter test results. By comparing with triaxial tests, the Poisson ratio ν, coefficient β and the geological factor α were determined.

1. INTRODUCTION

In the construction of diaphragm walls employed either as bearing structures or impermeable cutoffs, different types of design methods are used for self hardened suspensions. Researches have been carried out and are still going on these areas. However firms protect the different components and also the resulting characteristic parameters after suspension hardening. In a cooperation with other organizations, we participated in a research of such components and their characteristics after suspension hardening.

2. RESEARCH METHOD

Samples were prepared with different composition and tests were carried out using the following methods to study their mechanical properties:

- uniaxial strength tests and their changes in time (according to STN - Slovak Technical Standard 72 2429)
- tensile test under bending and modulus of elasticity (according to STN 72 24 50 ) and Young’s modulus (according to STN 73 13 19)
- triaxial tests according to STN (Slovak Technical Standard 72 10 31)
- pressuremeter tests according to STN 72 1004 and the regulations of EC 4 ISSMFE.

Janček, V. et al 1987, deals in detail with the methods of laboratory tests.

3. PRESSUREMETER TEST METHODS AND THEIR EVALUATION

The mechanical properties of self hardened suspensions were determined using pressuremeter tests. Tests were carried out after six, eighteen and thirty months after construction of test piles. Pressuremeter tests were carried out according to STN 72 1004 and EC - 4. The values of mechanical properties of the following parameters were registered and studied in different test stages (Fig.1):

- \( P_{\text{un}} \) - Menard limit pressure [MPa]
- \( E_{\text{p}} \) - Pressuremeter modulus [MPa]
- \( E_{\text{r}} \) - Rebound modulus [MPa]
- \( E_{\text{c}} \) - Cyclic modulus [MPa]

Comparing test results from triaxial and pressuremeter tests for the same stress value and a
determined Poisson's ratio $\nu$, we found out the relation coefficient $\beta$ between oedometric modulus $E_{om}$ and deformation modulus $E_{od}$ from equations (1 and 2).

$$\beta = 1 - (2 + \sqrt{1 - \nu})$$  \hspace{1cm} (1)

$$E_{od} = \frac{E_{om}}{\beta}$$  \hspace{1cm} (2)

From the well known correlation between oedometric modulus $E_{om}$ and pressuremeter modulus $E_p$ (3), the value of geological factor $\alpha$ is calculated. $E_{od}$ value was determined from triaxial tests.

$$E_{od} = \frac{E_p}{\alpha}$$  \hspace{1cm} (3)

From the measured and calculated values of $E_p$ and $E_{od}$, $\alpha$ is determined according to equation (4).

$$\alpha = \frac{E_p}{E_{od}}$$  \hspace{1cm} (4)

4. TEST RESULTS

The average values of tests results determined from pressuremeter tests or triaxial tests, for the suspension with hydraulic matter content 350 kg/m$^3$ are presented in Table 1.

<table>
<thead>
<tr>
<th>Testing after</th>
<th>Average values [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>construction</td>
<td>$E_{om}$</td>
</tr>
<tr>
<td>6 months</td>
<td>2.20</td>
</tr>
<tr>
<td>18 months</td>
<td>$&gt;2.5^3$</td>
</tr>
<tr>
<td>30 months</td>
<td>3.70</td>
</tr>
</tbody>
</table>

$E_{od}$ modulus derived from triaxial tests

$E_p$ determined from triaxial tests

$\nu$ pressuremeter apparatus used with a monometer $\pm 2.5$ kPa.

The average value of the Poisson ratio $\nu = 0.14$ was determined from several triaxial tests and for four different stress values. The average value of the coefficient $\beta = 0.95$ was calculated according to equation (1) and the geological factor $\alpha = 0.47$ according to equations (1 - 4). Details are discussed in [Matys, M. 1985].

The average values of tests results, for the suspension with hydraulic matter content 210 kg/m$^3$ are presented in Table 2.

Figure 1. Scheme of piles and places of pressuremeter test performance. a) pile with higher content of hydraulic admixture, b) pile with lower content of hydraulic admixture.
Table 2

| Testing after  | Average values (MPa) |
| construction   | $P_{th}$ $E_g$ $E_i$ $E_{req}$ $E_{rel}$ |
| 6 months       | 0.77 13.2 51.9 31.9 53.5 38.6 |
| 18 months      | 1.32 29.1 150.7 75.5 126.5 91.3 |
| 30 months      | 1.23 21.2 53.5 41.9 92.2 66.6 |

The average value of the Poisson ratio $ν$, was determined from several triaxial tests for four different stress values and was calculated to be $ν = 0.31$. Coefficients $α$ and $β$ were calculated according to equations (2 - 4), where $β = 0.722$ and the rheological factor $α = 0.23$.

5. DISCUSSION

It is possible to recommend according to gained experiences the preparation of holes for pressuremeter tests, using the steel tube before filling with self hardening suspension to the bore hole (diameter of the hole must be in tolerance with the probe used). If this is not possible it is needed to use core drilling and make pressuremeter tests after some days. Disturbed and plastified layers on the wall of the bore will dry, fall off, and do not influence test results.

Self hardened suspension with higher contents of hydraulic admixture under water and entree of air leads to improved mechanical properties probably to 18 months. On further suspension hardening under water, though the $P_{th}$ is improved, the deformation characteristics do not show distinct improvement (Table 1).

Hardened suspension with lower contents of hydraulic admixture leads to an improved mechanical properties likely to 18 months. Further hardening do not improve all the mechanical properties, instead lower values were measured after 30 months of suspension hardening. This was probably caused by non homogeneous suspension mixing, influence of borehole wall and other factors.

For self hardened suspension with higher content of hydraulic admixture, the values of Poisson’s ratio were derived and calculated to be $ν = 0.14$, coefficient $β = 0.95$ and the rheological factor $α = 0.47$ according to pressuremeter and triaxial tests.

For self hardened suspension with lower content of hydraulic admixture, the Poisson’s ratio $ν = 0.31$, coefficient $β = 0.722$ and the rheological factor $α = 0.23$ were found.

6. CONCLUSIONS

I hope that the values found out concerning the properties of self hardened suspensions using pressuremeter tests will contribute to the global understanding of their characteristics and their change in relation to the time of hardening. If it will be possible, we will make further pressuremeter tests for the confirmation of the conclusions brought up.

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6. STN 72 1319 - Determination of modulus of deformation and strength of concrete (in Slovak)
7. STN 72 1031 - Triaxial tests (in Slovak)
8. STN 72 1004 - Pressuremeter tests (in Slovak)
Incorporating the unloading portion in the interpretation of pressuremeter test in cohesive soil

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ABSTRACT: Interpretation of self-boring pressuremeter (SBPM) in cohesive soil using the combined expansion and contraction phases of test data is presented in this paper. Three type of shear stress-strain models (elastic-perfectly plastic, strain hardening, and strain softening) for cohesive soil were included in the analysis. The closed form solutions for these models are presented along with their numerical implementation. The assumptions associated with the stress-strain models under shear stress reversal mode are discussed. An in-situ SBPM test data was analyzed using the three types of soil models, and the conclusions related to the interpreted model parameters (initial shear modulus, undrained shear strength, and total horizontal stress) are summarized.

1 INTRODUCTION

The results of pressuremeter tests in cohesive clays are often evaluated using the theory of cylindrical cavity expansion. The various theoretical interpretation techniques developed to date for use in pressuremeter testing have focussed predominantly on the probe expansion curve. As a part of self-boring pressuremeter test, the probe contraction data is also routinely acquired from the field at the end of expansion stage. It is often true that the unloading portion of the pressuremeter curve is less affected by the installation procedures of the probe as opposed to the loading portion. Thus, it is beneficial to develop procedures for including the interpretation of the unloading portion. Houlsby and Withers (1988) first proposed such an approach for the full-displacement pressuremeter tests in clay using an elastic-perfectly plastic assumption for modeling soil shear stress-strain behavior. Jeeferies (1988) also used elastic-perfectly plastic model for interpreting self-boring pressuremeter using a trial and error approach for obtaining model parameters using visual computer aided modeling approach (CAM). Ferreira and Robertson (1992) extended this analysis to include the response of soil stress-strain variation represented by a hyperbolic function (strain-hardening). In this paper, interpretation of unloading portion of a pressuremeter test was further extended to include clay stress-strain behavior that has strain softening features. Closed form solutions for all the three soil models were derived and implemented numerically using Marquardt-Levenburg (Marquardt 1963) algorithm. The proposed approach of interpreting the total horizontal stress, initial shear modulus, and undrained shear strength using the combined loading and unloading portion of the pressuremeter test data are demonstrated using an in-situ test data.

2 SBPM INTERPRETATION FOR CLAY SOIL

The interpretation of pressuremeter test is an inverse boundary value problem where the expansion and contraction probe pressure and associated radial/circumferential strain values are measured in the field. The objective of the interpretation is to deduce the values of total horizontal stress, and the model parameters that describe the shear stress versus strain variation from this measured probe pressure-strain variation. For clays, this boundary value problem can be modeled using cylindrical cavity expansion/contraction theory under plane strain condition. The background theory for the interpretation procedure for the combined loading and unloading portion of the pressuremeter curve is presented below for a strain softening stress-strains model. The correspondence between the coefficients obtained from the computer program that does numerical implementation and the soil properties is also presented.

The equation of equilibrium for a cylindrical
cavity is given by equation 1.

\[
\frac{d\xi}{dr} = \frac{\sigma_2 - \sigma_1}{\tau} = 0
\]  
(1)

In pressuremeter testing, the circumferential strain \(\varepsilon_\theta\) is referred to as the cavity strain, \(\varepsilon_c\). The probe expansion under undrained condition gives the relationship between radial distance, \(r_c\), and the cavity strain, \(\varepsilon_c\), as:

\[
\frac{dr_c}{r_c} = -\frac{d\varepsilon_c}{\varepsilon_c (1 + \varepsilon_c) (2 + \varepsilon_c)}
\]  
(2)

The following equation was used to model the non-linear strain softening type shear stress-strain behavior (Arnold 1981) for cohesive soil.

\[
\tau = \frac{(\sigma_2 - \sigma_0)}{2} = \frac{abc}{(a + \varepsilon_c)^2}
\]  
(3)

For small values of \(\varepsilon_c\), \((1 + \varepsilon_c)(2 + \varepsilon_c) = 2\). The cavity strain at the pressuremeter boundary is \(\varepsilon_c\). Using equations 1, 2, 3, the small strain assumption, and the boundary conditions of a pressuremeter test \((\sigma_1 = \sigma_2)\), the probe pressure at \(\varepsilon = \varepsilon_c\), and \(\alpha_1 = \alpha_2\), at \(\varepsilon = 0\), the following equation is obtained for \(P_r\) as a function of cavity strain, \(\varepsilon_c\).

\[
P_r = \frac{(\sigma_2 - \sigma_0) + b\varepsilon_c}{(a + \varepsilon_c)}
\]  
(4)

From the pressuremeter test data, the measured probe pressure - cavity strain \((P_r, \varepsilon_c)\) data during the loading portion is now used for fitting equation 4 to obtain the three parameters \(\alpha_1\), \(\alpha_2\), and \(b\). The initial shear modulus, \(G_i\), is then calculated using equation 5.

\[
G_i = \frac{d\tau}{d\varepsilon} \bigg|_{\varepsilon = 0} = \frac{b}{\alpha}
\]  
(5)

For \(\tau = \tau_{\text{ult}}\), \(d\varepsilon = 0\), and corresponds to the occurrence of maximum shear stress at \(\varepsilon_c = a\).

\[
\tau \bigg|_{\varepsilon = \tau_{\text{ult}}} = \frac{b}{4}
\]  
(6)

Thus, the undrained shear strength of clay is obtained from equation 6. The total horizontal stress is obtained from \(\sigma_{ho}\), and the complete shear stress - cavity strain variation is obtained using equation 3. This approach has been used to date for interpreting only the loading portion (expansion phase) of pressuremeter data (Penman and Champeau 1995). The following section presents an approach for including unloading portion (contraction phase) for interpreting the complete probe pressure-strain curve.

Rewriting equation 3 in terms of undrained shear strength \(\tau_c\) and initial shear modulus \(G_i\), the following equation 7 is obtained. For undrained cylindrical cavity expansion, the shear strain is simply twice the value of the cavity strain (\(\gamma = 2\varepsilon\)).

\[
\frac{(\sigma_2 - \sigma_0)}{2} = \tau_c = \frac{8\varepsilon_c^2 G_i}{(2\tau_c + G_i)^2}
\]  
(7)

\[
e = \frac{(R - R_o)}{R_o}
\]  
(8)

\(e\) is the cavity strain during expansion phase, \(R_o\) is the initial radius, and \(R\) is the current radius of the pressuremeter during loading phase. For unloading phase of the pressuremeter curve,

\[
\tau_c' = \frac{8\varepsilon_c'^2 G_i}{(2\tau_c' + G_i)^2}
\]  
(9)

Where \(\varepsilon_c'\) is the cavity strain during unloading phase of the pressuremeter, \(R_{\text{max}}\) is the probe radius at the end of loading phase, \(R\) is the current radius of the probe, \(\tau_{\text{ult}}\) is the ultimate undrained shear strength for unloading, and \(\tau_c\) is the mobilized shear stress. The cavity strain during unloading (contraction) and loading (expansion) are related from the geometry by the following equation 11.
\[ e^* = \frac{(e - e_{max})}{(1 + e_{max})} \]  

(11)

Pressuremeter expansion and contraction can be considered as a plane strain problem where the stress path in unloading is the reverse of loading. Since the plane strain stress-strain envelope is approximately equal for loading and unloading, it is reasonable to assume a similar condition for pressuremeter stress-strain path during loading and unloading. The effect of shear stress reversal on clay strength was reported by Jefferys (1988) for a total stress path similar to the one a soil element would experience adjacent to a pressuremeter using undrained stress-controlled triaxial test at constant mean total stress with shear stress reversal. The undrained strength in unloading was found to be 83% of the undrained shear strength during loading for this clay. For relating the loading portion of pressuremeter curve with the unloading portion, a factor \( R_i \) (Pereira and Robertson 1992) is introduced as defined in equation 12.

\[ R_i = \frac{\tau^*}{\tau^*_{max}} = \left(1 + \frac{e_{max}}{e^*_{max}}\right) \]  

(12)

For clay soil having elastic-perfectly plastic or strain hardening (hyperbolic) stress-strain variation, the value of \( R_i \) is approximately equal to 2. However, for strain softening model introduced by the author in this paper, \( R_i \) value depends on the peak value of cavity strain that corresponds to the starting phase of unloading and is found out from equation 12 after obtaining the model parameters of equation 9. Using the approach identical to deriving probe expansion curve for loading phase as demonstrated above, the closed form solutions for loading (equation 13) and unloading phase (equation 14) are derived describing the pressure-cavity strain relationship.

\[ P^* = \frac{4\pi R_s G e}{G(\tau^*_{e0} + R_s)} \]  

(13)

\[ P_{max}^* = P_{max} \]  

(14)

The approach employed in this paper was to use equation 14 initially to curve fit the experimental probe pressure-cavity strain data of the unloading portion \( (P^*_{max} vs. \xi) \) using the Mavjud-Rouenberg algorithm to obtain the two model parameters \( \tau^*_{e0} \) and \( \xi \). At this stage, the value of \( R_i \) was evaluated by numerically solving equation 12. Using these values of \( \tau^*_{e0}, G, \) and \( R_i \), the experimental probe pressure-cavity strain data of loading portion \( (P \text{ Vs. } \xi) \), and using Mavjud-Rouenberg algorithm, the value of total horizontal stress \( \sigma_h \) is determined from equation 13.

Using the procedures described above, equations similar to 13 and 14 were now derived using the following two other stress-strain models: 1) strain hardening using hyperbolic response (Pereira and Robertson 1992), 2) elastic-perfectly plastic (Gibson and Anderson 1961). For brevity, the relationships were not included in the present paper. The objective was to include comparative interpretation results for the three possible stress-strain variations commonly observed for clays using combined analysis of expansion and contraction of the pressuremeter test data.

3 APPLICATION EXAMPLE

A previously published test data shown in Fig. 1 (Jefferys 1988) was used for applying the above three soil models for interpreting the combined loading and unloading portion of the SBPM data. The reasons for using this test data was two fold. A portion of the interpretation results can be directly compared with the trial and error computer aided modeling (CAM) technique that is currently being used in practice (Jefferys 1988; Jefferys and Shuttle 1995). Secondly, for this silty clay soil, information on the ratio of undrained shear strength from unloading to loading was experimentally found to be approximately equal under stress path soil elements are subjected to adjacent to a SBPM.

Using the basic assumption of isotropy of undrained shear strength (loading versus unloading), a value of \( R_i = 1 \) was used for the elastic-plastic and strain hardening stress-strain model. For the test data shown in Fig. 1, using \( P_{max} = 2,267.8 \text{ kPa} \) and \( e_{max} = 0.079 \) in equation 14, the values of \( \tau^*_{e0} \) and \( \xi \) were determined and using equation 12 the value of \( R_i \) was calculated to be 1.416. Using these model parameters, the value of total horizontal stress \( \sigma_h \) was evaluated using the entire probe expansion test data. The results of interpretation using combined analysis of contractions and expansion portions of the test data are summarized in Fig. 2. The model parameters in terms of total horizontal stress, initial shear modulus, and undrained shear strength are shown in Table 1. The resulting stress-strain variation for all the three models are shown in Fig. 3.
Table 1: Interpreted Model Parameters

<table>
<thead>
<tr>
<th>Model</th>
<th>$G_i$ (kPa)</th>
<th>$S_u$ (kPa)</th>
<th>$\sigma_{zz}$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic-Plastic</td>
<td>34800</td>
<td>103.2</td>
<td>1668</td>
</tr>
<tr>
<td>Strain Hardening</td>
<td>59080</td>
<td>123.8</td>
<td>1660</td>
</tr>
<tr>
<td>Strain Softening</td>
<td>34426</td>
<td>184.5</td>
<td>1564</td>
</tr>
</tbody>
</table>

From Table 1, it is clear that for the same test data there is a considerable variation in the interpreted values of undrained shear strength (103 kPa to 185 kPa) and initial shear modulus (34 MPa to 59 MPa) depending on the assumptions associated with the type of stress-strain model used in the analysis. However, the total horizontal stress value was less affected by the choice of the stress-strain model and can be reliably estimated using any of the above models from the combined analysis of pressuremeter expansion and contraction data.
The advantage of the methodology presented in this paper is that only partial probe expansion data can be used in the interpretation without considering the initial disturbed portion (due to self-boring procedure) of the test data.

4 SUMMARY AND CONCLUSIONS

A procedure for interpreting the pressuremeter test data incorporating the combined expansion and contraction test data was presented. This procedure eliminates the need for trial and error approach currently used in the practice employing iterative forward modeling image matching techniques. Three types of soil stress strain models (elastic-plastic, strain hardening, and strain softening) were included in the present formulation for the interpretation of SIBPM in cohesive soil. These were implemented on a personal computer using Marquardt- Levenburg algorithm. The application of these models was demonstrated by considering an in-situ SIBPM test data in silty clay. A new procedure for interpreting the pressuremeter curve was developed using a strain softening type soil shear stress-strain model. The following can be concluded based on the present analysis:

1) Considerable variation was observed in the deduced values of initial shear modulus and undrained shear strength depending on the assumptions related to the stress-strain model.
2) Interpretation of total horizontal stress was relatively insensitive to the assumed stress-strain model when the complete probe expansion and contraction test data was used in the analysis.

Several inferences on the interpretation of both loading and unloading portions of the pressuremeter test data were also presented in the paper along with recommendations for use of these models for interpreting the field test data.

5 REFERENCES

6 APPENDIX I: NOTATION

\( \sigma_r \) = radial stress
\( \sigma_\theta \) = circumferential stress
\( r \) = radial distance of the soil element
\( p_r \) = maximum probe pressure
\( P_r \) = probe pressure during loading
\( P_u \) = probe pressure during unloading
\( \varepsilon_r \) = radial strain during loading
\( \varepsilon_e \) = cavity strain during loading
\( \varepsilon_c \) = cavity strain at the probe boundary
\( G_I \) = initial shear modulus of clay
\( R \) = radius of pressuremeter during loading or unloading
\( R_0 \) = initial radius of the pressuremeter before loading phase
\( R_u \) = ratio of shear strength of unloading with loading
\( S_u \) = undrained shear strength of clay
\( \sigma_h \) = total horizontal stress
\( \tau_u \) = ultimate shear stress during loading
\( \tau_c \) = ultimate shear stress during unloading
Assessment of soil properties in cohesive-frictional materials
with pressuremeter tests

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ABSTRACT: An interpretation method based on elasto-plastic soil behaviour is used to analyse pressuremeter test results in cohesive-frictional materials. A software developed for personal computers performs a curve-fitting analysis from which strength and deformation parameters can be predicted and the in situ horizontal stress can be assessed. Both self-bored and pre-bored pressuremeter tests carried out on weathered soils have been used to verify the proposed methodology. The method was considered a promising framework to derive soil parameters but at present other field and laboratory data are necessary to support interpretation.

1 INTRODUCTION

Compacted soils, cemented materials, soft rocks and deposits of unsaturated soils are all examples of materials that exhibit cohesive-frictional behaviour. The mechanical behaviour of such soils is generally very complex and is influenced by the stress path, structure breakdown, suction, and geological and pedological conditions, among other factors.

The study of the stress x strain x strength behaviour of these materials has been traditionally supported by extensive laboratory observations on compacted samples which are tested in triaxial cells. Inherent difficulties of sampling on natural deposits are often bypassed, even though it is accepted that the influences of cementing agents, weathering processes and spatial heterogeneity are indispensable for geotechnical design purposes. The influence of these factors should be evaluated from field performance data using techniques which allow for the in situ measurement of the stress x strain behaviour of the soil, the results of which can be rationally interpreted by means of analytical and numeric methods. The pressuremeter appears to be a natural option for site investigation since, in theory, the boundary conditions are controlled and well defined.

In the present research both pre-bored and self-bored techniques are used with the aim of assessing and developing methods of interpretation of pressuremeter test results in cohesive-frictional soils. The discussion focuses on the assessment of shear strength parameters from curve fitting analysis, as well as the measurement of shear stiffness and in situ horizontal stress from unload-reload loops and lift-off pressure, respectively.

2 INTERPRETATION

Interpretation of in situ tests requires either a closed-form solution or a numerical technique to model the test. A numerical technique using a finite element method was performed successfully to model pressuremeter test results in cemented soils (Mántaras et al., 1997). However a closed-form solution is considered more convenient since it can be implemented in a simple software for analysing experimental data. The solutions provided by Carter et al. (1986) and Yu and Houlsby (1991, 1995) can accommodate the cohesive-frictional nature of soils and for this reason have been investigated.

The analytical development of the methods considers a cylindrical cavity of infinite length expanding in a homogeneous and isotropic soil mass. The adopted hypothesis are: expansion occurs in triaxial symmetric, the soils is initially in isotropic state of stresses, a linear elastic-perfectly plastic model is used in a Mohr-Coulomb failure criterion and the elastic phase responds to the Hooke's law. Implications of the different mathematical treatment given to the analytical equations by Carter et al. (1986) and Yu & Houlsby (1991, 1995) is here explored throughout comparisons.
It is worth noticing that in cylindrical cavity expansion formulations, the Mohr-Coulomb failure criterion ignores the possible effects of intermediate principal stress by assuming that in loading the soil fails with the radial stress as the major and the circumferential as the minor principal stress. When the initial in situ stress state satisfies the condition that soil behaviour can be treated in terms of $\sigma_0$ stresses, the isotropic stress state can be assessed from the lift-off pressure. There is no accumulated experience on the possible range of $K_0$ ($\sigma_0'/\sigma_0$) values for cemented soils and a possibility exists for the vertical stresses to remain as the minor principal stress at failure. In this case the formulation can no longer be applicable to estimate shear strength parameters.

The methodology for interpreting the pressuremeter data is similar to that proposed by Jeffreys (1988) and Ferreira and Robertson (1992) for clays. The analytical equations for the loading (Carter et al., 1986; Yu and Houldby, 1991) and unloading (Yu and Houldby, 1995) curves were derived and used to visually match experimental data. The analytical methods were implemented in a mathematical package (MathCad), the applicative program executes calculations for loading and unloading equations, performs a curve fitting analysis and provides a visual evaluation of the comparison between experimental and theoretical curves at the computer screen. The parameters that produce an analytical curve that adjusts satisfactorily experimental results are in theory representative of soil behaviour.

Drainage conditions are assumed in the analysis either because these lightly cemented soils are a product of an in situ weathering process which generally increases porosity (e.g. Leroueil & Vaughan, 1990) or because the presence of permeable discontinuities greatly increase the ratio of pore-water pressure dissipation (e.g. Haberfield, 1997). Since drained conditions are assumed, the analysis was made considering effective stress parameters.

Application of the proposed methodology is clearly subjected to criticism despite of being supported by a sound theoretical background which is based on analytical cavity expansion/contraction equations. Most pressuremeter results are subjected to some degree of disturbance which may affect the shape of the initial loading portion of the pressuremeter curve. A suitable approach would be the one of putting greater reliance on the final portion of the loading curve and on the unloading portion of the pressuremeter test (Ferreira and Robertson, 1992). However due to both the possible physical effects of soil arching and structure breakdown and the simplifying assumptions adopted to express the governing equations in a sufficiently simply form, the unloading portion may not be suitable for interpretation purposes. The main objective of the study is to evaluate the usefulness and restrictions of the proposed approach to cohesive-frictional materials.

3 SELECTION OF PARAMETERS

The use of implemented elasto-plastic models in the applicative software involves six variables: internal friction angle $\phi'$, cohesion $c'$, horizontal stress $\sigma_{ho}$, shear modulus $G$, coefficient of Poisson $\nu$ and dilatancy $\psi$. The Coefficient of Poisson has little influence on results and for this reason is estimated from general background (assumed to be between 0.2 and 0.3 for practical applications). The values of $G$ and $\sigma_{ho}$ are measured directly from pressuremeter data, whereas shear strength parameters are assessed from the proposed approach. Each of these variables affects the simulation in a distinct way. For this reason a thorough description of the parameter selection process is presented hereinafter, in such a way as to provide the engineer with guidance in the use of the fitting process.

3.1 Horizontal Stress

Several methods have been developed to assist in the prediction of horizontal stress from SBP curves in both clay and sands (e.g. Matesand and Randolph, 1977; Jeffries et al., 1985). It is a common sense that selection of $\sigma_{ho}$ is subjective since interpretation depends upon the insertion of the probe into the ground. More importantly, there is no accumulated experience that can be applied to cohesive-frictional materials.

It is well recognised from the various theories that information on the initial horizontal stress remains embedded in the value of limit pressure, in which case it can be assessed from the interpretation of the entire experimental curve. It is therefore suggested to estimate $\sigma_{ho}$ from the lift-off pressure and to use this point as a reference value to perform a curve fitting analysis that will refine the estimate of $\sigma_{ho}$ once an appropriate matching is produced.
3.2 Shear Modulus

The soil shear modulus is the parameter of most geotechnical interest when pressuremeter tests are accomplished (Wroth, 1984). It is relatively simple to obtain G values from the measured pressure versus volume or radius change of an expanding cylindrical membrane. It is therefore suggested to initiate the curve fitting using stiffness values from unload-reload cycles, G, with a shear strain amplitude of about 0.2 to 0.3%.

3.3 Strength Parameters

Two strength parameters (c' and $\phi'$) have to be obtained simultaneously through the curve fitting of the plastic phases of the pressuremeter test. Two alternatives are subject to consideration:

a) adopting pairs of values for $\phi'$ and c' which adjust the theoretical curve to the limit pressure and to the plastic segment at the end of unloading. It is possible that more than one pair of values offer a perfect fitting, which will force an experienced engineer into some sort of geotechnical judgement as to the choice representative values of soil behaviour, or

b) measuring the shear strength parameters independently from complementary laboratory or in situ tests and adopting these values as input data in the curve fitting. A good match would enhance the reliability in selecting this set of parameters as representative of field conditions for design purposes.

It is interesting to note that the degree of cementation has little effect on $\phi'$ but has a marked influence on c' (e.g. Clough et al., 1981; Consoli et al., 1996). If several pressuremeter tests are carried out in the same soil, it is possible to select a single value of $\phi'$ representative of the soil matrix and assess the magnitude of c' from each individual pressure expansion curve. Changes in the values of c' would reflect possible changes in cementation with depth.

As for dilation, it is generally recognized that Rowe's dilatant theory can be applied, as already recommended by Haberfield (1997) for soft rocks. The relationship proposed by Schanz & Vermeer (1990) is adopted.

\[ \sin \psi = \frac{\sin \beta - \sin \phi'}{1 - \sin \beta \sin \phi'} \]

replacing the critical state friction angle $\phi''$ for the residual friction angle $\phi'$ (Haberfield & Johnston, 1990).

In summary, with the fitting technique a coupled set of six parameters should be used to describe soil behaviour; to reduce ambiguities some constrains have to be imposed to the analysis. The estimate of $\sigma''_{0}$ is limited to the range of values of lift-off pressures measured by the three strain arms. The pressuremeter modulus can vary only within the range of measured unload-reload loops G. Shear strength values c' and $\phi'$ are allowed to vary during the fitting process or can be estimated directly from complementary laboratory tests. In the last case all 6 parameters are pre-selected and if the fit of model to data is considered adequate then the set of parameters is unquestionable.

4 APPLICATION OF THE PROPOSED METHODOLOGY

Traditional interpretation methodologies for the analysis of SBP data in sands and clays are applicable to extremely high quality testing curves. The concepts of the curve fitting to interpret disturbed data in clays have been recently explored by Jeffries (1988) and Ferreira & Roberton (1992). This paper addresses the possibility of extending this approach to different materials and also to analyse data from other pressuremeter probes. No attempt is made here to cover a full investigation testing programme but to give examples of interpretation of pressuremeter data on cohesionless soils and non-standard materials of cohesive-frictional nature.

4.1 SBP tests in sands

To highlight the relevance of the proposed approach, a number of reported calibration chamber tests in sand has been interpreted by Määttä (1997). Some experience gained in sand was essential for supporting the interpretation of testing data in cohesive frictional soils. Furthermore, in many saturated weathered profiles the shear resistance due to the frictional component of shear strength would govern, with the cohesion intercept being close to zero (Baig et al., 1995; Gan & Fredlund, 1996). Under these conditions the experience gained in sand
may be particularly appropriated.

A typical plot showing the comparison between experimental and analytical curves is presented in Figure 1 for a test carried out by Ajd-lowiahs (1990) with a small pressuremeter of length to diameter ratio of 20, ideally installed (cast) in the sample. The curve fitting is very good over the entire range of cavity strain both on the loading and unloading portion of the pressuremeter curve. Similar agreement was observed for three different in situ tests in sands (Fehey & Randolph, 1984; Houlsby et al., 1985; Wroth, 1982) from which the following conclusions were established:

![Figure 1: Loading and unloading curve matching from a calibration chamber test on medium dense sand](image)

- In general, the quality of the match is irrespectively to the initial density of the sand. It is acknowledge that Yu & Houlsby's interpretation method has properly taken into account the effects of both elastic deformation and plastic volumetric behaviour. Carter's method disregard the elastic deformations in the plastic region which may have some effects on predicted results for sands with a high angle of friction and low stiffness (Carter et al., 1980; Collins et al., 1992).
- Values of stiffness derived from the loading and unloading analysis are very similar and compare well with the magnitude of Ge.
- Soil friction angles derived from the loading analysis are comparable to measured triaxial peak friction angles, whereas the values of $\phi'$ yielded from the unloading are of the same order of magnitude as the residual friction angle. This observation has already been suggested by Houlsby et al. (1985), but contrasts with some previously published work.

- A remarkable agreement is observed between the values of $\phi'$ derived from the methods proposed by Carter et al. (1986) and Yu & Houlsby (1991).

4.2 SBP tests in weathered gneiss from São Paulo, Brazil

Early results from SBP suggest that an approximately analysis using a curve fitting technique is able to give good estimates of strength and stiffness, as well as an assessment of the magnitude of the in situ horizontal stress.

A representative pressuremeter test on weathered gneiss is presented in Figure 2, in a test carried out at 0.20 m depth. The equipment used is a standard Camkometer probe. The data were plotted using the average strain from the three arms from which the unload-reload loops have been intentionally removed. The model of the complete test was applied to the experimental data using $\alpha_n$, $\phi'$ and G as constrained parameters and $c'$ as a free parameter. A good fit has been achieved between theory and data over the whole curve. However, the analytical solution usually lies below the field data at the start of the test which may suggest that disturbance caused during installation is not very severe. When the fit is performed over the complete loading curve, the match at large strains is not representative of the initial stiff response of the structured soil before yielding.

The model predictions of the unloading portion of the test are consistent with the residual triaxial strength. It may be indicating that either the bounds of the structure have already been completely destroyed by large strains and reverse shear at

![Figure 2: Comparison of experimental and fitted pressuremeter response in weathered gneiss](image)
cotraction or simple that the pressure-expansion curve is not well represented by the elasto-plastic
model adopted in the analysis.

4.3 Ménard pressuremeter tests in weathered granite from Southern Brazil

Several pre-bored pressuremeter testing programmes have been carried out in southern Brazil to support
research on weathered cemented soils (Bosch, 1996) and more recently in unsaturated soil conditions
(Farias, 1997). A standard GA Ménard pressuremeter probe is used and test procedure
generally follows the recommendations from the French Standards (e.g. Bagueau et al., 1978).

A typical pressure-expansion curve is presented in
Figure 3; the experimental and theoretical curves are shown to compare well over the complete field of
cavity strains. The unloading portion of
pressuremeter tests appears to be insensitive to disturbance caused by installation (e.g. Bellotti et al.,
1986; Schmidt & Hoadley, 1992). It would
therefore, appear logical to model MPM and SPM
unloading indistinctively. As for the loading portion,
the datum associated to \( \alpha' \) is not identifiable
and the limit pressure is rarely achieved.

5. CONCLUSIONS

An attempt is made to address the problem of interpreting self-boring and pre-bored pressuremeter results
in cohesive-frictional soils. The proposed interpretation method consists on a curve fitting technique to model the loading and unloading
portions of the pressuremeter curve. Validation of
the method has been obtained from the interpretation
of tests carried on decomposed lightly structured soils from Southern Brazil.

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Deformation and in-situ stress testing
Measurement and prediction of effective stress in an overconsolidated clay

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ABSTRACT: An extensive laboratory investigation has recently been carried out at Queen's University Belfast to study the yield characteristics of Belfast Upper Boulder Clay. As part of this investigation an attempt was made to examine the influence of the presence of layers of silt and fine sand on the estimation of the effective stress in the samples. A pressure plate apparatus developed at Queen's University was used to measure the effective stresses in the samples. Measured values of effective stresses were compared with values predicted using cross-anisotropic elastic theory. Samples with no trace of sand or silt layers showed good agreement between measured and predicted values of effective stresses. However, the comparisons were less good with samples containing traces of silt and sand. Nevertheless the results are encouraging and confirm the validity of the assumption of cross anisotropic elastic behaviour during the sampling process.

1 INTRODUCTION

Recent developments in site investigation practice appear to be moving towards the situation where more sophisticated testing procedures are applied to individual and possibly fewer samples. In this situation it is a matter of much greater significance to be able to trace the actual stress path followed by a particular sample throughout the sampling and testing procedure. The present paper describes an attempt to elucidate this process. The work is limited to saturated samples taken from considerable depths where overburden pressures and insitu pore water pressures are known with reasonable accuracy.

When a saturated sample of soil is removed from the ground, the total applied stresses are reduced to zero and the effective stress acting on the sample is equal to the negative pore water pressure in the sample. Various methods of determining the negative pore water pressure were discussed by Skempton, (1961) using measurements obtained with the triaxial apparatus. Recent work by Ridley, (1993), Ridley & Harland (1993) shows the growing interest in the measurement of effective stress in the sample using the pressure plate apparatus and similar more sophisticated devices. Measurements of negative pore water pressure have been compared with calculated values in order to assess the degree of disturbance of natural clay samples (Ladd & Lambe, 1963; Hight, 1993). These authors made use of equations developed by Skempton (1961) or Skempton & Sowa (1963) and the clays considered were mostly normally or lightly overconsolidated. It is important to note that this work was based on the assumption that clay in an isotropic material. The present paper is based on the assumption that throughout the sampling process clay behaves as a cross anisotropic elastic material and 'perfect' sampling conditions are assumed. In other words changes in pore water pressure due to the shearing action inherent in the sampling process are ignored and it is assumed that the changes in effective stress which occur are due simply to the changes in total stress which are an inevitable part of the sampling process.

It is important to remember that natural samples almost invariably contain imperfections. Occasional stones and/or thin layers of silt and sand frequently occur and these may have a significant effect on the results obtained.

2 ANISOTROPY

All soils are anisotropic in structure or loading or both. Generally speaking cross anisotropy may be assumed and this is done in the present paper. In an
important paper Graham & Houlsby (1983) propose a particular form of anisotropy which when applied to stress conditions similar to those in the triaxial test lead to the equations

\[ \Delta \sigma' = K' \Delta \varepsilon_v + J \Delta \varepsilon_s \]  
\[ \Delta \tau = J \Delta \varepsilon_s + 3G' \Delta \varepsilon_v \] 

where \( \varepsilon_v \) = volumetric strain, \( \varepsilon_s \) = shear strain, \( K' \) and \( G' \) are the bulk modulus and shear modulus respectively and \( J \) is a coupling parameter. \( \nu' \) is the mean effective stress and \( q \) is the deviator stress. In the case of isotropic clays \( J = 0 \) and the above equations reduce to the isotropic elastic equations.

In the undrained condition \( \Delta \varepsilon_v = 0 \) and equations 1 and 2 reduce to

\[ \frac{\Delta \sigma'}{\Delta q} = J(3G') \]  

Using equation 1 and following the procedure developed by Skempton (1954) the change in pore water pressure consequent on a change in effective stress is given by

\[ \Delta u = (\Delta \varepsilon_s + 2J \Delta \varepsilon_v)3 - (J(3G'))(\Delta \varepsilon_s - \Delta \varepsilon_v) \]  

where \( \varepsilon_s \) and \( \varepsilon_v \) are the effective vertical and horizontal stresses. This equation can be applied to the sampling process and again following the procedure adopted by Skempton (1961) we obtain

\[ \nu'_c = (\varepsilon_s + 2\nu \nu' \nu - J(3G'))(\varepsilon_s - \varepsilon_v) \] 

where \( \nu'_c \) is the effective stress (isotropic) on the sample and \( \varepsilon_s \) and \( \varepsilon_v \) are the effective vertical and horizontal stresses in the ground before the sample is taken. It should be noted that while this equation is similar in form to that proposed by Skempton, the sign of \((\varepsilon_s - \varepsilon_v)\) is not modified. The determination of the term \( J(3G') \) is discussed in a later section. A detailed deviation and discussion of the new pore water pressure equation in sampling is given in Johnson, 1997.

3 MEASUREMENT OF EFFECTIVE STRESS IN THE SAMPLE

3.1 Pressure plate apparatus

The detailed description of the pressure plate apparatus is shown in Fig. 1. It consists mainly of a high air entry filter (5 bar), Bodeenberg gauge, GDS and a pressure transducer. The measurement of effective stress in the pressure plate apparatus involves two stages:

(a) saturation of the high air entry filter.
(b) measurement of effective stress in the sample.

Complete saturation of the high air entry filter was necessary to achieve the specified air entry value. If the filter was not de-aired properly then the air would blow through the filter and get into the water drainage system.

For de-airing, the porous ceramic and pedestal assembly were fixed to the base of the pressure plate apparatus. The apparatus was filled with de-aired water and the drainage valves through the base pedestal were flushed with freshly de-aired water. The chamber cell was pressurised using a GDS. The chamber pressure was increased to 900kPa and left for two days with the valve on the drainage line closed. The drainage line valve was then opened and water was allowed to drain through the high air entry filter for one day at a cell pressure of 900kPa. The drainage valve was closed and the system was left for another day at a pressure of 900kPa. Then the cell pressure supply was gradually reduced to zero over a period of about 12 hours.

The porous ceramic was re-saturated after each test. If the porous ceramic was kept under water after each test it was sufficient to re-saturate the porous ceramic in 1-2 days. The saturation of the porous ceramic was checked by closing the drainage valves and monitoring the response of the water pressure in the drainage system. If the porous ceramic was fully saturated, the pressure in the drainage system rose to a value equivalent to the chamber pressure within about 3 seconds.

The measurement of effective stress in the sample using the pressure plate apparatus may be affected by two factors: (a) excess water left on the surface of the high air entry filter (b) the small flow of water due to the formation of meniscus (Fig. 2). Therefore an experimental evaluation was carried out to assess the relevant influence of these effects.

Once the saturation of the high air entry filter was assured the water in the chamber was drained and the system was dismantled. Inside the chamber was
wiped with dry cloth and the surplus water on the
higher entry filter was carefully removed leaving a
thin film of water. Then the system was assembled
again without the specimen and the drainage line
was connected to a GDS. The air pressure in the
chamber was increased to 100kPa while the
drainage line remained open to GDS. Once
equilibrium was achieved the volume readings on
the GDS were noted. Then, the air pressure in the
chamber was increased to 450kPa in increments of
50kPa. During each increment the system was left to
equilibrante and the amount of water drained into
the GDS was noted. Fig. 3 shows the graph plotted
between the amount of water drained into GDS and
the differential pressure between air pressure in the
chamber and water pressure in the drainage line (u1,
u2). The amount of water drained into the GDS is
equivalent to the amount of water replaced by the
meniscus caused by the pressure difference (u1−u2).

This process was carried out before every test,
however each time the air pressure was increased to
a value equivalent to the expected value of effective
stress in the sample. Once equilibrium was reached
the chamber was de-pressurised under undrained
conditions expecting that the meniscus would
remain the same shape and size. Although it was
expected that the above mentioned process to expel
all the excess water on the porous ceramic and
create an appropriate meniscus, it carries a possible
disadvantage. De-pressurisation of the chamber

under undrained condition may induce negative
water pressure in the drainage system and quite
possibly it may lead to cavitation. If that were to
occur then the purpose of creating an appropriate
meniscus prior to each test is ruined. However a
counter argument can be made that the cavitation
may disappear during the re-application of the
chamber pressure during the test in which the
sample is in the apparatus.

Prior to every test the above mentioned procedure
was repeated; the porous ceramic was saturated, air
pressure in the chamber was increased to a value
equivalent to the expected value of effective stress
in the sample while keeping the drainage valve open
and the chamber pressure was released under
undrained conditions. Following this procedure the
system was dismantled and the specimen was placed
on top of the porous stone. A small amount of hand
pressure was applied to the sample to make sure a
good contact between the specimen and the porous
ceramic was obtained. The system was left to
equilibrante and this took about 16-15 hours to
complete. The response of the water pressure in the
drainage system was monitored throughout this time
and the effective stress in the sample was measured
as the difference between the air pressure and the
final water pressure in the drainage system. At the
end of the test, the chamber pressure was de-
pressurised with the drainage valve closed and the
final moisture content the specimen was determined.

In order to check the above procedure, reconstituted samples of kaolin were isotropically
consolidated in the triaxial cell to effective pressures of 250, 300 and 400 kPa. When tested in the
pressure plate apparatus they gave p’c values of 225,
280 and 374 kPa respectively. The agreement
between the initial isotropic consolidation pressure
and the final measured effective stress is very
promising and the marginal difference can be
attributed to the nature of the kaolin.

3.2 Tests on undisturbed samples

An extensive laboratory investigation into the
geotechnical properties of Belfast Upper Boulder
Clay has been in progress for several years. This
material is a fluvi-glacial deposit (Dougan, 1992)
laid down in Glacial lake Belfast. It is a firm to stiff
overconsolidated red brown clay, often fissured
particularly when it occurs near ground surface and
containing horizons of laminated clay, usually near
its base. In spite of its name it is virtually stone free
and its natural moisture content is rarely more than
5% in excess of its plastic limit. The average liquid
and plastic limits are approximately 65% and 25%
and the average undrained shear strength and moisture content are 100 kPa and 30% respectively.

As part of this investigation a detailed series of tests was carried out in order to examine the yield characteristics of several samples of the material from the same boring. The opportunity was taken to study the variation in measured \( p_u \) values in the same samples. In this particular fill and estuarine alluvial deposits extended beyond a depth of 15 m and were underlain by Upper Boulder clay to a depth of 43 m. The Upper Boulder clay was underlain by bedrock till which was not fully penetrated in the boring. The underlying bedrock is Sherwood Sandstone.

Samples referred to in the present paper were obtained using the standard U100 sampling process at depths of 22.6m, 28.0 and 38.8m below the ground surface. Layers of silt and fine sand varying from 2 to 10mm in thickness occurred in the 22.6m sample and from 0.5 to 2mm thick in the case of 38.8m sample. In the 28.0m sample very thin silt layers were visible only after the samples had been oven dried.

![Fig. 4 Typical test result on the pressure plate apparatus](image)

Fig. 4 shows a typical result (Test no. 4, 22.6m), in which the water pressure in the drainage line and air pressure applied in the chamber are plotted. When the chamber pressure was released with drainage line closed prior to the test (formation of appropriate meniscus without the specimen in the chamber), the water pressure in the drainage line dropped to 50 kPa. When the air pressure was re-applied (with the specimen in the chamber), the water pressure in the drainage line increased steadily. The water pressure reached a peak value within 6 hours and then dropped to a stable value. The difference between the air pressure applied and the final water pressure in the drainage line yields the magnitude of the effective stress in the sample (\( p_u \)). The detailed test results obtained are given in Table 1.

### Table 1.a Measured values of \( p_u \) (22.6m)

<table>
<thead>
<tr>
<th>No.</th>
<th>Air pressure</th>
<th>Water pressure</th>
<th>( p_u ) kPa</th>
<th>Moisture content %</th>
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<tr>
<td>1</td>
<td>325</td>
<td>330</td>
<td>33.12</td>
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<tr>
<td>2</td>
<td>357</td>
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<td>30.28</td>
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<tr>
<td>7</td>
<td>395</td>
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<td>29.91</td>
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<tr>
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<tr>
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<tr>
<td>Average</td>
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</tbody>
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### Table 1.b Measured values of \( p_u \) (28.0m)

<table>
<thead>
<tr>
<th>No.</th>
<th>Air pressure</th>
<th>Water pressure</th>
<th>( p_u ) kPa</th>
<th>Moisture content %</th>
</tr>
</thead>
<tbody>
<tr>
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<td>330</td>
<td>33.12</td>
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### Table 1.c Measured values of \( p_u \) (38.8m)

<table>
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<tr>
<th>No.</th>
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<th>Water pressure</th>
<th>( p_u ) kPa</th>
<th>Moisture content %</th>
</tr>
</thead>
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<tr>
<td>Average</td>
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<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

4 PREDICTION OF EFFECTIVE STRESS IN THE SAMPLE

Prediction of the effective stress in the sample using Equation (5) requires a knowledge of the in situ vertical effective stress, the in situ horizontal effective stress and the anisotropic parameter \( J/(3G') \).

The in situ vertical effective stress can be calculated with reasonable accuracy from the borehole records using density measurements but the estimation of the horizontal effective stress in overconsolidated clays is subject to considerable uncertainty since it depends on an estimate of the maximum vertical preconsolidation pressure \( \sigma_{vp} \).
4.1 Determination of maximum vertical preconsolidation pressure \( \sigma_{vp} \)

From the depth of the sample and its moisture content the initial effective stress and the void ratio can be calculated. The present state of the sample in the ground is then represented on a pressure void ratio diagram by a point such as A in Fig. 5, which illustrates the procedure for the sample at a depth of 28.0m. The swelling Index \( C'_{s} \) obtained from an oedometer test on an undisturbed sample enables the line \( AX \) to be drawn and the point representing the sample at \( \sigma_{vp} \) must lie on this line.

\[
\text{Fig. 5 Determination of } \sigma_{vp} \text{ from } e - \log \sigma_{vp} \text{ graph}
\]

The position of the Normal consolidation line can be approximated by making use of Burland’s (1990) concept of Intrinsic Properties. The Intrinsic Consolidation Line (ICL) can be obtained either from tests on reconstituted samples or by using the correlation with liquid limit given in Burland’s paper. (Tests carried out at Queen’s University have confirmed that these correlations apply reasonably to Belfast Upper Boulder clay although a somewhat better result is obtained using Nakane et al., 1988). Using the ICL together with the results of oedometer tests on undisturbed samples a value for the maximum overconsolidation pressure can be assessed.

4.2 Determination of the in situ horizontal effective stress

Meyerhof (1976) proposed a relationship between the earth pressure coefficient \( K_e \) and over consolidation ratio OCR given by

\[
K_e = \frac{K_{oc}}{OCR^{(b)}}
\]

where \( K_{oc} \) is the earth pressure coefficient of the normally consolidated soil given by Jaky’s equation

\[
K_{oc} = 1 - \sin \phi'
\]

where \( \phi' \) = angle of internal friction. Using these relationships and the value of \( \sigma_{vp} \) obtained as described in 4.1 above, the value of the in situ horizontal effective stress can be calculated from the equation

\[
\sigma' = K_{oc} (\sigma_{vp}/\sigma_{vp})^{(b)}
\]

4.3 Determination of the anisotropic parameter

The cross anisotropic parameter ratio \( J(3, G') \) was obtained using Equation 3, in which \( J(3, G') \) is defined as the ratio between \( A p' \) and \( A q' \); i.e., the initial slope of undrained effective stress path with respect to the \( q \)-axis in \( p'-q' \)-space yields the value of \( J(3, G') \). Alternatively \( J(3, G') \) can be estimated from a plot of change in pore water pressure against change in total mean stress. The value at each depth was obtained using the data of a standard undrained triaxial test conducted on samples at each depth (22.6, 28.0 and 38.8m). Each sample was isotropically consolidated to a pressure equal to the effective stress in the sample \( p^* \) (150, 325, and 295kPa respectively), prior to undrained shearing. The calculated values of \( J(3, G') \) are shown in Table 2.

4.4 An alternative approach is to make use of the empirical relationship proposed by Shoehet 1995 which enables the values of the in situ horizontal effective stress to be determined. This enables the effective stress in the sample to be calculated. The results obtained using the two methods are given in Table 2.

| Table 2: Prediction of the effective stress in the sample, \( p^* \) |
|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| depth \( w \) | LL | PL | \( \alpha' \) | \( \phi' \) | \( C' \) | \( J(3, G') \) | Estimated \( \sigma_{vp} \) | Estimated \( \sigma' \) | Estimated \( p^* \) |
| (m) | (%) | (%) | (%) | (%) | (%) | (kPa) | (kPa) | (kPa) |
| 22.6 | 26.0 | 39.7 | 25.4 | 220.8 | 139.7 | 22 | 0.04 | 0.20 | 1245 | 328 | 363 | 271 | 288 | 2224 |
| 28.0 | 32.1 | 69.7 | 29.9 | 276.3 | 173.5 | 21 | 0.10 | 0.26 | 1300 | 384 | 376 | 320 | 317 | 319 |
| 38.8 | 26.7 | 54.3 | 24.1 | 386.5 | 176.6 | 24 | 0.08 | 0.24 | 1410 | 435 | 434 | 407 | 407 | 350 |
5 DISCUSSION

From the test results given in Table 1 it is evident that there is a particularly wide scatter in the tests on samples from the depths of 22.6m and 38.8m. It is generally recognised that the presence of silt and sand layers will have a significant effect on the negative pore water pressure developed on the sample since moisture from the silt and sand will be absorbed by the intervening clay layers. This will clearly give rise not only to the considerable scatter observed but will also result in a general reduction of measured effective stress. For this reason the probable $p'_u$ values which are given in Table 2 are greater than the maximum measured values.

On the other hand the test results for the sample at 28.0m which contained no significant sand and silt layers show relatively little scatter and the average value is given in Table 2.

Turning to the predicted values of $p'_u$ given in Table 2, the agreement between the values obtained from the Intrinsic Consolidation Line method and from Shobet’s method (making use of the assumption of cross anisotropic elastic behaviour) is remarkably good. This to some extent confirms the validity of the various assumptions made.

The agreement between the predicted and measured values of effective stress in the case of the 28.0m sample is excellent, particularly in view of the fact that the sample was recovered by the standard U100 sampling procedure.

6 CONCLUSIONS

The authors consider that the results are encouraging in spite of the obvious problems which arise in dealing with natural clays. They confirm the validity of the assumption of cross anisotropic elastic behaviour during the sampling process and illustrate the value of the pressure plate apparatus which enables overburden pressure, over consolidation pressure and cross anisotropic properties to be related.

ACKNOWLEDGEMENTS

The samples on which the work described in this paper was based were provided by Glover Site Investigation Ltd, Ballymoney, Northern Ireland. The authors are very grateful for the assistance received. Mr. A. S. Johnson is grateful to The Queen’s University of Belfast and Overseas Research Award Scheme for providing the financial support for the study. The Authors wish to express their appreciation to Mr. Alan Murray at the Department of Civil Engineering, QUB, Belfast, UK.

REFERENCES


Evaluation of coefficient of lateral subgrade reaction based on horizontal loading test for piles

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OYO Corporation, Chubu Office, Nagoya, Japan

ABSTRACT: For design of pile foundations, coefficient of lateral subgrade reaction of ground is often investigated by in-situ measurement using a pressuremeter or the other type instruments. However, the coefficient obtained from the tests have a trend to conservative estimation for the actual foundations. So, engineers often perform in-situ horizontal loading tests on driven and/or buried piles and confirm adequacy of the coefficient accepted to the design. It is well known that obtained load-displacement relationships includes characteristics of the pile. Especially in case of concrete piles, elastic-plastic behavior due to breakout and processing of cracks affects on the relationship. This research is performed to develop a simple method for reasonable evaluation of lateral subgrade reaction based on pile loading tests for cast in place reinforced concrete pile.

1 INTRODUCTION
The coefficient of lateral subgrade reaction of ground is needed parameter for design and static and dynamic analysis of pile and the other type deep foundations. Generally, the coefficient is determined from results of in-situ lateral loading test in a borehole by using pressuremeter or the other type instruments. However, it is often pointed out that the lateral subgrade reaction evaluated from the pressuremeter tests is somewhat conservative for the actual foundations. Therefore, horizontal loading tests on actual piles are often performed for direct evaluation to the coefficient of lateral subgrade reaction.

However, obtained load-pile displacement relationships does not represent characteristics of the ground itself, because it includes the effect of mechanical characteristics of the piles. Especially in case of concrete piles, the horizontal load displacement relationships are affected strongly by elastic-plastic behavior due to breakout and progressing of cracks in the pile during the loading tests. Then, to evaluate pure characteristics of lateral subgrade reaction of ground surrounding the piles, the load-displacement relationship should be analyzed by eliminating the effect of pile characteristics.

This research is performed to develop a method to evaluate lateral subgrade reaction of ground based on pressuremeter tests in boreholes and in-situ horizontal loading tests against a driven steel pipe pile and a cast in place reinforced concrete pile.

2 HORIZONTAL LOADING TEST FOR PILES
2.1 Profile of ground at testing sites
The ground of testing sites consists of sandy soil and silty clay layers at site A and of silty clay layers at site B as can be seen in Figs. 1 and 2. The profiles of the ground at these sites are similar to each other at the depth more than 10 m from the ground surface. The N-values of SPT were 1 to 6 for the sandy layer at the site A and around 1 to 3 for the clayey layers at B sites. Pressuremeter tests were performed in boreholes of which diameter was 86 mm for each site.

The coefficient of lateral subgrade reaction, k_h, obtained from the pressure-meter tests are also shown in this figures.

2.2 Summary of pile loading tests
Horizontal loading tests were performed for two types of piles; i.e. a steel pipe pile and a cast in place reinforced concrete pile. These piles were driven or constructed through the subsoil soft ground as shown in Figs. 1 and 2. The diameters of both piles were 1,500 mm. The loading tests were carried out in accordance with JGS (The Japanese Geotechnical Society Standard). Applied load to the pile head and horizontal displacement at the pile head were measured for each load step. Holding time at each loading
step was kept 30 minutes. Strain at 15 locations along a vertical line through the loading point of the steel pipe pile and those along a vertical line at opposite side were measured. For the cast in place concrete pile, axial stresses of reinforcement were measured at approximately corresponding measuring locations with the steel pipe pile.

2.3 Results of loading tests

Fig. 3 and 4 show relationships between horizontal load and displacement for each pile. From these figures, we can find that yielding load of the steel pipe pile is among 900 to 900 kN for the steel pipe pile and among 600 to 700 kN for the cast in place concrete pile, respectively.

Typical relations of the fiber stress of steel pipe pile and of the axial stress in the reinforcement of cast in place concrete pile against horizontal load at each depth are shown in Figs. 5 and 6. Within the applied load, the fiber stresses at both sides showed approximately linear response, while the axial stress of reinforcement at loading side increased significantly after the stress reached to about 25,000 kN/m², as shown in these figures. Tensile strains of concrete around the reinforcement at this stage was estimated to the strain which induces tensile cracks in concrete. This trend was observed prominently at depths between 2 and 6 meters from the ground surface. From the result mentioned above, the authors guessed that the yield of the pile was mainly induced by outbreak and progress of cracks of concrete.
3 PROCEDURE OF REVERSE ANALYSIS

3.1 Limitation of the conventional procedure for reverse analysis

In reverse analysis to evaluate the coefficient of lateral subgrade reaction based on horizontal loading tests for concrete piles, it should be considered the reduction of cross section of the pile because of outbreaks and progress of cracks in the pile.

Two procedures for reverse analysis were examined. One is based on Chang’s method in which the pile behaves as elastic notwithstanding the yield of pile and total area of cross section of the pile is effective to external forces. The other is assumed that the pile can resist to the horizontal load by elastic effective cross sections, which are defined from observed stresses of reinforcements at each depth.

In the following section, procedure for evaluating the effective cross section will be mentioned.

3.2 Evaluation of effective cross section

Fig. 7 is illustrative diagram to compute effective cross section of the cast in place concrete pile. By equating resultant forces of compressive and tensile stress, the effective cross section which behaves elastically can be easily determined. Location of the neutral axis, Xo, at each section of the pile can be determined based on measured tensile and compressive stress of the reinforcements. Equilibrium condition of forces on a cross section can be expressed as:

\[ C_3 + C_4 - T_x = T_y = 0 \]  

(1)

where \( C_3 \) is resultant force of compressive stress acting on concrete portion of a section, \( C_4 \) is resultant force of compressive stress acting on reinforcement portion of a section, \( T_x \) is resultant force of tensile stress acting on concrete portion of a section and \( T_y \) is resultant force of tensile stress acting on reinforcement portion of a section, respectively.

Fig. 8 is an illustrative diagram for calculating resultant forces of stresses. Each resultant forces in Eq. 1 can be expressed as follows:

\[ C_3 = \int_0^\theta \sigma \cdot r \cdot dA = \frac{\sigma_{cr}}{X_t} \cdot \frac{X_0}{X_t} \cdot \int_0^\theta X_0(\theta) \cdot dA \]

\[ C_4 = \int_0^\theta \sigma \cdot r \cdot dA = \frac{\sigma_{cr}}{X_t} \cdot \frac{X_0}{X_t} \cdot \int_0^\theta X_0(\theta) \cdot dA \]

\[ T_x = \int_0^\theta \alpha r \cdot dA = \frac{\sigma_{cr}}{X_t} \cdot \frac{X_0}{X_t} \cdot \int_0^\theta X_0(\theta) \cdot dA \]

\[ T_y = \int_0^\theta \sigma \cdot r \cdot dA = \frac{n \cdot \sigma_{cr}}{X_t} \cdot \frac{X_0}{X_t} \cdot \int_0^\theta X_0(\theta) \cdot dA \]

where \( \sigma \) and \( \alpha \) are normal stress acting on concrete, axial stress of reinforcement and fiber stress of concrete, respectively. \( n \) is a ratio of Young’s modulus of steel to that of concrete. \( \sigma_{cr}, \sigma_{cr}, \sigma_{cr} \) and \( G_0, G_0 \) are geometrical moment of compressive cross section corresponding to the concrete portion and the reinforcement, geometrical moment of tensile cross section corresponding to the concrete portion and the reinforcement, respectively. The other notations are illustrated in Fig. 8.

Substituting Eq. 2 into Eq. 1, we get

\[ \sigma_{cr} + \sigma_{cr} - \sigma_{cr} - n \sigma_{cr} = 0 \]  

(3)

Height of the effective cross section, Xo, can be determined from Eq. 3 by iterating as shown in Fig. 7.

3.3 Bending moment considered crack progression

Bending moment on the effective cross section can be calculated by following equations:

\[ M = M_r + M_t \]

\[ M_r = \frac{\sigma_{cr} \cdot I_1}{X_t} \]

\[ M_t = \frac{n \cdot \sigma_{cr} \cdot I_1}{X_t} \]

\[ I_1 = \int_0^\theta \left[ X_0(\theta) \right]^2 \cdot dA \]

\[ I_2 = \int_0^\theta \left[ X_0(\theta) \right]^2 \cdot dA \]
where $M$ is total bending moment on the cross section, and $M_{c}$ and $M_{b}$ are bending moment on the portion of concrete and on the reinforcement at the cross section, respectively. $I_{c}$ and $I_{b}$ are geometrical moment of inertia for the portion of concrete and of reinforcement at the cross section, respectively.

3.4 Coefficient of lateral subgrade reaction

Fig. 9 shows a flow of the reverse analysis for the results of horizontal loading tests. The coefficient of lateral subgrade reaction was calculated for each pile and for each loading step by finite element analysis. The pile was assumed as a beam of which bending rigidity was equivalent to that of the pile. Considering that the beam is supported laterally by springs corresponding to the surrounding ground as shown in Fig. 10, the governing equations of this model can be expressed by

\[ F = S \epsilon \\
\epsilon = A'X \\
\]

where $F$ is internal force matrix, $P$ is external force matrix, $\epsilon$ is internal member displacement matrix, $X$ is external nodal displacement matrix, and $S$ is stiffness matrix.

From the above equations, we can get

\[ X = \left( A S T \right)^{-1} P \\
P = A S A'X \]

(6)

Iterative calculation was continued by changing the value of $k$ until the analyzed pile displacements coincide with the measured value.

4 RESULTS AND DISCUSSION

4.1 Analyzed coefficient of lateral subgrade reaction

For the steel pipe pile, no yielding was observed through the horizontal load test. Therefore, the authors assumed that the total area of each cross section through the pile length behaved elastically. Then analysis was performed by both the conventional method and finite element analysis. These method resulted to give almost same coefficient of subgrade reaction for each other.

Fig. 11 shows a relationship between horizontal displacement of the steel pipe pile at a location of the ground surface, and analyzed coefficient of lateral subgrade reaction of ground, $k$. The analyzed coefficients corresponds to secant modulus of the load-displacement curve. As can be seen, we can find that the values of $k$ are good proportional with $y$ to the power of 0.5.

For the cast in place reinforced concrete pile, the finite element analysis described previously was applied to the reverse analysis. To realize the validity of the concept on reduction of the effective cross section due to growth of cracks, two kind of analyses were made; i.e. the one in that all area of cross section is effective and the other is that the area of cross section decreases with growth of cracks.

Fig. 12 shows relationships between horizontal displacement of the pile at a location of the ground surface and analyzed coefficients of lateral subgrade reaction. A dashed line in the figure represents the results assumed that the total area of cross sections are effective for bending of pile. From this line we can judge that the pile-ground system might be yielded at

![Figure 9: Calculation of coefficient of lateral subgrade reaction based on the results of horizontal loading test](image-url)

![Figure 10: Model of beam-spring element](image-url)
7.5 cm of horizontal displacement. It is remarkable that this displacement agree well with the displacement corresponding to yielding load, $P_y$, shown in Fig. 4 and this load is correspond to 25 MPa of the tensile stress of reinforcement, from which the increase of tensile stress at each cross section becomes rather predominant than compressive stress shown in Fig. 6.

In Fig. 12, when the horizontal displacement is smaller than 0.75 cm, the values of $k$ are proportional with $y$ to the power of -0.5. However, the power decreased to -0.91 for $y$ larger than 0.75 cm. It should be noted that decrease of the value of power is due to outbreak of cracks in the pile. Therefore, we can conclude that the dashed line is not representative for the coefficient of lateral subgrade reaction of the ground itself surrounding the pile. Solid line in Fig. 12 shows the results of analysis in which the reduction of the effective cross section were took into account. It is interesting that the analyzed values of the power became to -0.5 throughout the horizontal displacement analyzed. This suggests that the effect of damages of the pile was eliminated by considering the effective cross section.

Figs. 13 and 14 show comparisons of analyzed bending moments corresponding to deep with moments calculated from the measured strain of the piles. In the calculation, the equations relating the values of $k$ and $y$ defined in Figs. 11 and 12 were used.

From these figures, we can conclude that it is necessary to introduce the concept of effective cross section in stress analysis for the concrete piles.

4.2 Comparison of coefficient of lateral subgrade reaction

For each site, the pressuremeter tests were carried out from the ground surface to a depth around 15 m, because it was predicted that the lateral subgrade reaction to the pile deeper than 15 m might not affect so much to the behavior of pile for both sites. Values of measured coefficient of lateral subgrade reaction, $k_x$, are shown in Figs. 1 and 2.

For practical purpose, the value of coefficient of lateral subgrade reaction for design of a pile foundation is expressed by

$$k_0 = k_y y^{0.5}$$

$$k_y = a k_x$$

$$a = f(C_o, B, etc.)$$

where $k_y$, $k_0$, and $k_a$ are subgrade reaction for design of the pile, coefficient of lateral subgrade reaction normalized to unit displacement of the pile at $a$
Table 1 Comparisons of $k_h$ with $k$ corresponding to 1 cm-pile displacement obtained by reverse analysis of the horizontal loading tests

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>$k_h$ obtained by pressuremeter method of Imai</th>
<th>$k$ by reverse analysis of pile</th>
<th>$k_h / k$</th>
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<tr>
<td>CP</td>
<td>JRA's method (method)</td>
<td>JRA's method (method)</td>
<td>Total section</td>
</tr>
<tr>
<td>CP</td>
<td>11.8</td>
<td>5</td>
<td>18</td>
</tr>
<tr>
<td>SP</td>
<td>26.2</td>
<td>77.3</td>
<td>21</td>
</tr>
</tbody>
</table>

In a column of $k_h / k$, the upper numbers in the ratio against the effective section and lower asterisk corresponds to total section.

CP: cast in place concrete pile
SP: steel pipe pile

location corresponding to the ground surface and measured coefficient of lateral subgrade reaction by the pressuremeter test, respectively, $y$ is horizontal displacement of the pile at a location corresponding to the ground surface, $x_p$ and $d$ are initial radius of the pressuremeter and radius of the pile, respectively.

Many researchers suggested formulae regarding the above equation. Imai (1970) and JRA (Japanese Road Association, 1982) derived semi empirical formula, respectively, which can apply to pressuremeter (LLT) used in this research.

Comparisons of $k_h$ calculated by Imai’s and JRA’s method with $k$ corresponding to 1 cm-pile displacement shown in Figs. 11 and 12 are tabulated in Table 1.

For the case of cast in place concrete pile, values of $k_h$ by Imai’s and JRA’s method are considerably conservative. On the other hand, for the case of steel pipe pile, the value of $k_h$ by Imai’s method is 27 % greater than the value of $k$, but JRA’s method gives 18 % smaller than the value of $k$. The cause of difference of $k_h$ between the cases of cast in place concrete pile and the steel pipe pile is not rather the difference of pile than the soil layer near the ground surface surrounding the piles; for the cast in place concrete pile, the soil layer predominates clayey soil and for the steel pipe pile, sandy soil is predominated.

From described above, it can be concluded that the value of ratio $k_h / k$ varies from 0.25 to 0.7 for clayey layer and from 0.8 to 1.3 for sandy layer.

5 CONCLUDING REMARKS

Two types of pile were tested in this research. The one was a cast in place concrete pile and the other was a steel pipe pile of which diameters were 1,500 mm and their length were about 50 m. Strains at 30 locations for each pile were measured at each loading step during the tests. Main features obtained from this research were concentrated as follows;

1. In the case of concrete pile, difference on stresses in the reinforcement between the compressive and tensile side of the pile was observed after that the tensile stress reached to 25 Mpa, and the difference increased with increase of the horizontal load. It can be guessed that breakout of cracks of pile started at this stress and that the yield of pile is characterized by the breakout of cracks and their progress.

2. In order to eliminate the effect of change in the characteristics of pile, consideration of the effective cross section of pile is useful for reverse analysis of pile-ground interaction.

3. Notwithstanding the yield of concrete pile, the analyzed coefficient of lateral subgrade reaction was still proportional to the horizontal displacement of pile at the ground surface to the power of 0.5 by considering the effective cross section of the pile. It is remarkable that the amount of this power coincides with those before yield and with that of the steel pipe pile which behaved elastically through the loading test.

4. Considering the effective cross section and assuming the reverse analyzed coefficient of lateral subgrade reaction, the bending moment at each depth of the pile by P.E.M. analysis agreed well with that calculated based on the measured strain of the reinforcement.

5. The coefficient of lateral subgrade reaction, $k_h$, did not agree with the reverse analyzed coefficient, $k$, for the piles tested in this research. Imai’s method resulted to underestimate for clayey layer and to overestimate for sandy layer. On the other hand, JRA’s method gives underestimation both for clayey and sandy soil layer. However, Imai’s method gave closer estimation than JRA’s method in this research.

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A new self boring in-situ friction testing technique

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ABSTRACT: This paper presents a new in-situ test technique, namely Self Boring In-situ Friction Test (S.B.I.F.T.). As a multiple-purpose technique, it is aimed to evaluate soil for sub-structure interaction strength, deformation parameters as well as earth pressure at rest. Here, it will be concentrated on the deformation modulus of soils. The device is described, and loading methods are proposed. Theoretical background for their interpretations required is discussed. Examples will be given to illustrate the tested results.

1 INTRODUCTION

In recent years, the growing need to build on more and more poor soils has led us to use more approaches to investigate the soil behavior. We have learned that it is necessary to find more reasonable methods to estimate soil for sub-structure interaction behavior from elastic to large deformation fields. The design of structures in soils requires that their strength and deformation parameters, and earth pressure at rest be evaluated for the in-situ conditions. It may be difficult or even impossible to create in-situ conditions in the laboratory, due to severe static, groundwater situation, specimen disturbance and size effect. For instance, earth pressure at rest can only be evaluated in-situ but not in the laboratory.

Varieties of in-situ test technique have been developed to interpret the soil behavior. In-situ tests have the advantage of keeping the material in situ circumstance, and are not affected by sampling and preparing of specimen. Compared to the specimen volume of the laboratory element test, the volume in-situ tested is much larger, which incorporates the heterogeneity and anisotropy of material. It could be expected that in-situ tests lead more reasonable results.

However, the stress and strain induced in-situ test that it undergoes are generally not in uniform, and pore water drained condition is difficult to be controlled. Undoubtedly, the measured displacement is affected owing to pre-drilling and the not fully contact at the interface. It is considered that parameters estimated by in-situ test be a kind of boundary problem values. Besides, one in-situ test technique is usually designed for single purpose, that leads us difficulty to make comprehensive analyses.

Here, a new in-situ test technique, namely Self Boring In-situ Friction Test (S.B.I.F.T.), has been conceived and introduced to interpret the site characteristics of soils. Self-boring system is employed to minimize the disturbance of soils as small as possible. A piezometer is annexed to monitor the changing of the pore water pressure while loading, which provides us more information about the soil characteristics. Normal stress σn, normal to the wall of boring, is applied by multiple cycles. With the certain normal stress, parallel stress σ0, parallel to the wall of the boring, is loaded. According to the applied stresses and deformations and their relationships, strength and deformation parameters of soils can be interpreted by this technique.

We do every effort to make S.B.I.F.T. as a multiple-purpose in-situ test instead of single purpose. It is attempted to interpret:

(1) the strength of the interface of soils;
(2) the subgrade shear reaction coefficient (Crs);
(3) the deformation modulus (E);
(4) the earth pressure at rest (p0).

The strength of the interface c and τ) of soils (c and τ) interpreted by S.B.I.F.T. has been discussed by Maeda et al. (1997b) and Xu et al. (1997a).

The subgrade shear reaction coefficient (Crs) interpreted by the technique was discussed by Maeda et al. (1997c). This paper will be concentrated on the deformation modulus of soils. In order to get high reliable results, loading methods are also proposed. Existing examples are shown the site deformation characteristics obtained in S.B.I.F.T.

2 SKETCH AND CONFIGURATIONS OF S.B.I.F.T.

Figure 1 shows the newly developed S.B.I.F.T. device. The S.B.I.F.T. device consists of the following four main components:

(1) a self-boring system.
(2) a probe;  
(3) a surcharge applying system; and  
(4) a data acquisition system.  
The technique configurations of S.B.I.F.T. are  
(1) the maximum of a vertical stress (normal stress) \( q_v \) applied by a hydraulic pump through membrane tube is up to \( q_{v,\text{max}} = 1.5 \text{MPa} \);  
(2) the maximum of a parallel stress (shear stress) \( q_d \) applied by a hydraulic pump through a central half jack is equal to \( q_{d,\text{max}} = 0.65 \text{ MPa} \);  
(3) the maximum shear displacement for one loading cycle is \( \delta_{\text{max}} = 30 \text{ mm} \);  
(4) the diameter of the boring hole is to 116 mm, and  
(5) the maximum depth of the measurement can be reached to \( D = 50 \text{ m} \).  

2.1 Self-boring system  
To minimize the disturbance of soil caused by insertion, various studies have been carried by different models of operation and design of equipment. Here, a self-boring technique has been adopted to minimize the disturbance of soil and to improve the accuracy of the measurements in S.B.I.F.T. by an order of magnitude or more. The self-boring diameter is 97.5 mm, and the maximum depth of the self-boring hole can be able to reach \( D = 50 \text{ m} \).  

2.2 Probe of S.B.I.F.T.  
The probe of S.B.I.F.T. shown in Figure 2 is of 200 mm length and of 100 mm diameter. The superficial loading plate has three separated subplates. With the expansion of the chamber, the hydraulic pressure pushes the loading plates to go outward. In other words, a normal stress acts on the interface between the probe and surrounding soils.  
The interface roughness effect has been investigated by many researches (for example, H. Kishida and M. Uesugi, 1987). The friction characteristics of the interface between the probe and the soils vary with the combinations of the structures and the soils. When the roughness represented by an indicator, \( R_{\text{wmax}} \), is larger than a critical value, friction resistance will be not affected by the roughness. It means the frictions at the interfaces equal those of soils. Due to the considerations, different types of the superficial loading plates have been designed. That makes conveniently in practicing to choose the superficial loading plates for different situations.  
Two often used types of the superficial loading plates are shown in Figure 3.  
Type I is a kind of the smooth superficial shape. It is made of aluminum. This superficial shape corresponds to those of, for example, the interactions of the steel pipe pile structures.  
Type II is a kind of the indent superficial shape. It is also made of aluminum. The spacing between the indents is 10 mm, and the height and width of the indents are 1.5 mm and 2.0 mm, respectively. It makes the interface rough enough, so that the friction angle of the interface, \( \phi \), can be considered as to approximately be the internal friction angle of soil, \( \phi \).
2.3 Surcharge applying system

The surcharge applying system of S.B.I.F.T. includes: (1) the vertical stress (normal stress) \( q_n \) applied system, and (2) the parallel stress (shear stress) \( q_y \) applied system.

The vertical stress (normal stress) \( q_n \) in the probe of S.B.I.F.T. is applied by hydraulic pressure through a inflatable membrane shown in Figure 2.

The principle of this is the same as that of the pressuremeter, which was first conceived by Mennard in 1954. The stress is measured at the ground surface by monitoring the pressure and the volume of the fluid used for inflation.

A basic expansion test of the probe have to be conducted firstly in order to calibrate the vertical stress (normal stress) \( q_n \), which really acts on the interface between the soils and the probe of S.B.I.F.T.

The parallel stress (shear stress) \( q_y \) applied system is applied by a gear hydraulic pump through a center hole jack, which is tied by a outer rod holder. The magnitude can also be known by the reading of a load cell attached. The surcharge can be loaded by controlling of a constant speed of displacement, which is monitored by a dial gauge (Figure 1).

2.4 Data acquisition system

All of the measurements are acquired by a computer. Analog channels are connected to the computer through a data logger. The testing data is scanned and recorded at certain rate.

3 TEST PROCEDURES AND LOADING PROPOSALS

3.1 Loading cycles

After the S.B.I.F.T. probe is placed at the prescribed depth, a initial normal stress is applied. Keeping this normal stress a constant, the probe is up exhaded or down driven. That means a parallel force is applied and gradually increased till a peak mobilized resistance at interface occurs or the parallel displacement surpasses 10 mm for the given normal stress. Then, a normal stress increment is applied, and again a new parallel force at the new normal stress is applied. Repeat this procedure, a series of load-displacement for the given normal stresses can be recorded. Thus, with these curves, we can deduce the reaction responses at interface.

Considering the efficient and the scatter of testing, we suggest 5 cycles of loading in S.B.I.F.T. testing. Usually, 5 cycle loading can obtain satisfactory results.

3.2 Initial normal stress

It is known that the soil characteristics varies with the stress path formed in the geological history. For example, the subgrade shear reaction coefficient depending upon the stress path. It changes at some extent within the over consolidated region, especially when the OCR value is great (Xu et al., 1997).

Consequently, in order to interpret the behavior values of soils with good precision, it is very important to set the applied stresses, especially the initial normal stress in the probe. To estimate earth pressure at rest, it must be needed to apply the normal stress carefully. If the normal stress is set too small, it is possible that the testing is conducted within over...
consolidated region. On the other hand, if the normal stress is too large, it is possible that the surrounding soil will fall into failure. In principle, the initial normal stress can be assigned to

\[ q_{eq} = p_0 + \alpha \] (kPa) \hspace{1cm} (1)

where \( p_0 \) is the horizontal stress at history.

If the property values of soils, such as the density and the coefficient of pressure at rest \( K_{os} \) are known, it is estimated by Equation (2).

\[ p_0 = K_{os} \gamma_i Z_i \] (kPa) \hspace{1cm} (2)

If the property values of soils are unknown, it is also estimated by Equation (2), but \( K_{os} \) is approximately estimated by Iaby Equation. That is

\[ K_{os} \equiv 1 - \sin \phi' \] \hspace{1cm} (3)

\[ K_{os} = 0.5 (\phi' \equiv 30'), \gamma_i \equiv 18 \text{ (kN/m}^3) \hspace{1cm} (4)

where \( \phi' \) is the internal friction angle, \( \gamma_i \) is the density of soil at water, \( \gamma_i \equiv (\gamma - 9.8) \text{ (kN/m}^3) \); and \( Z_i \) is the depth of overburden strata, (m).

### 3.3 Subsequent normal stresses

Accounting to the applying load capacity, the strengths of soils and other factors, the normal stress increment for each loading cycle is suggested bellow.

\[ q_{eq} = q_0 + \alpha \quad q_{eq} \equiv 1.25 \cdot q_0 \] \hspace{1cm} (5)

### 4 INTERPRETATION OF DEFORMATION MODULI

Deformation modulus, \( E \), is a very important parameter for deformation and stability analysis of structures. It is usually estimated by the following four methods.

(1) Plate load test;
(2) Pressuremeter test in bore hole;
(3) Unconfined compression or triaxial compression test; and
(4) Standard penetration test.

Unfortunately, the deformation moduli estimated by these methods are usually different from each others. That is because (1) precision and error in different tests are different, and (2) different tests are based on different principles. Therefore, the deformation moduli estimated should be modified for design purpose (Japan Highway Association, 1995).

Here, we discuss the possibility of deformation modulus estimated by S.B.I.F.T. .

#### 4.1 Background theory

Figure 4 shows the stress state for a axial symmetry cylinder model. The stress equilibrium equation can be expressed in polar coordinate as

\[ \frac{\partial \sigma_r}{\partial r} + \frac{\partial \sigma_z}{\partial z} + \sigma_r - \sigma_n = 0 \] \hspace{1cm} (6)

\[ \frac{\partial \tau_{rz}}{\partial r} + \frac{\partial \tau_{rz}}{\partial z} = \gamma \] \hspace{1cm} (7)

The stress state corresponds to the stress state in S.B.I.F.T., in which the normal stress \( q_e \), and parallel stress \( q_n \) are applied simultaneously. When the normal stress \( q_n \) is only applied, the stress state reduces to a two dimension problem if the soil weight is neglected (\( \gamma = 0 \)). It is just the stress state in pressuremeter test as shown in Figure 5. In this case, the stress equilibrium equation can be reexpressed as

\[ \frac{\partial \sigma_r}{\partial z} + \sigma_r - \sigma_n = 0 \] \hspace{1cm} (7)

The deformation moduli can be estimated by the slope of the curve of the normal stress \( q_n \) and displacement \( \delta \), or the normal stress \( q_n \) and fluid inflated volume, \( V \). Figure 6 presents a typical curve of the normal stress \( q_n \) and displacement \( \delta \). At the start of the fluid inflated, the membrane expands,
and the probe fits the loading plate and has the same diameter. In theory, no displacement should be detected until the normal stress \( q_0 \) is equal to the earth pressure at rest in the ground in the contact with the probe. In reality there is some compliance due to the probe itself and the soil disturbance. Until at point B, the soil starts to deform regularly under increasing normal stress \( q_0 \). The deformation modulus \( E \) can be estimated by the relationship between the normal stress \( q_0 \) and the change of borehole \( e_b \) in segment BC. With elastic theory, and let Poisson’s ratio be \( v \), the deformation moduli can be given by equation (8).

\[
E = (1 + v)v_0 \frac{a_0}{dr}
\]

where, \( a_0 \); the normal stress increment at segment BC, (kPa); \( dr \); the displacement increment under \( a_0 \), (cm); \( v_0 \); the middle radius at segment BC, (cm).

Assume the loading plate length of the probe be \( L \), and the volume change \( dV \) due to \( dr \) will be

\[
dV = \pi (r_0 + dr)^2 L - \pi r_0^2 L
\]

(9)

Let \( dr^2 \geq 0 \), the volume change \( dV \) can be rewritten as

\[
dV = 2\pi r_0 L dr
\]

(10)

From equation (8) and (10), it becomes

\[
dV = \frac{1 + v}{E} 2\pi r_0^2 a_0 dr
\]

(11)

Because \( \pi r_0^2 L \) is the initial volume due to the borehole radius \( r_0 \), equation (12) can be obtained

\[
E = 2(1 + v)r_0^2 \frac{da_0}{dV}
\]

(12)

or,

\[
E = 2(1 + v)r_0^2 (V_c + V_f) \frac{da_0}{dV}
\]

(13)

where, \( V_c \); the initial cell volume of the probe, (cm³); \( V_f \); the mean fluid inflated volume at segment BC, (cm³); \( da_0 \); the slope of normal stress - volume change at segment BC, (kPa/cm³).

From above, the deformation modulus \( E \) can be estimated by equation (8) or equation (13).

It should be noted that Poisson’s ratio \( v \) is predetermined for calculating deformation moduli. For the interpretation of undrained condition test in clay, it is assumed that the clay behaves as a perfectly elastic-perfectly plastic inelastic characterized by \( v = 0.5 \). In fact, Poisson’s ratio \( v \) is very difficult to be determined. For the convenience of the design, it is usually assumed \( v = 0.33 \) or \( v = 0.3 \).

4.2 Example

The comparisons of shear strength in S.B.I.F.T. to those obtained from triaxial compression tests were given by Xu et al (1997) and Mueda et al (1997). Here, another examples tested in S.B.I.F.T. are given in Figure 7 and 8, in which the in-situ test were performed at the 2nd Mektoa Tobishima Foundation Engineering, Aichi Prefecture, Japan. Type II, indend superficial shape, was used in the in-situ test.

The geology of the site is oceanic sedimentary. From the ground to depth 15.5m, it consists of loess silt, fine sand or medium size sand. The SPT N-value
of the strata is equal to 6–3. The shear strength of the strata is 14.8 kPa of cohesion, and 9.2° of friction angle in S.B.I.F.T. (Figure 7).

From 15.5 m to depth 30.6 m, it consists of clay or clay with silt. The SPT N-value of the strata is equal to 4–5. Figure 8 shows the soil strength characteristics obtained in S.B.I.F.T. together with those of undisturbed samples tested by the unconfined triaxial compression test and unconfined compression test.

Figure 9 shows the curve of the normal stress and the fluid inflow volume at depth 6.5 m. 5 loading cycles were applied in this S.B.I.F.T. test. For each loading cycle \( q_v \), the deformation moduli interpreted are 4.3 MPa, 8.6 MPa, 11.6 MPa, 9.9 MPa and 10.3 MPa, respectively. It should be noted that \( q_v \) for the first cycle loading is 4.3 MPa, and the mean value for the subsequent cycles is equal to 10.0 MPa, i.e., about twice as the one for the virginial. In the triaxial compression test, it is often observed that the virginial loading is about half of the reloading one.

5 REMARKS AND CONCLUSIONS

As a multiple-purpose approach, S.B.I.F.T. is introduced to interpret not only the site shear strength characteristics, but also the site deformation characteristics. Moreover, it is possible to interpret the earth pressure at rest in this technique. It can provide us more information of site characteristics. Thus comprehensive judgment can be made for design purpose.

In S.B.I.F.T., it is important to set out the procedures and the magnitude of the loading. The loading method is proposed, and 5 loading cycles are recommended for the efficiency and the scatter of soils.

To interpret the deformation moduli of soils, the background theory is reviewed. Deformation moduli can be calculated from the relationship between the normal stressess and the inlet fluid.

With the regarding to the reliability of the technique, like other techniques, it depends on the precision of the calibration test at the great extent, and others.

REFERENCES


Seismic flat dilatometer tests in Piedmont residual soils

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ABSTRACT: A hybrid test combining the conventional flat dilatometer with downhole geophysical testing has been developed and referred to as the seismic flat dilatometer test (SDFM). Downhole seismic velocity measurements have been incorporated with a flat dilatometer by placing a velocity transducer in a connecting rod just above the blade. The seismic dilatometer test has the exceptional advantages of determining both estimates of soil properties and stratigraphic information, while also measuring the shear wave velocity ($V_s$) within a single sounding. The hybrid test is rapid, simple, and cost-effective requiring essentially no more time than a conventional dilatometer sounding. The seismic dilatometer was field tested in the Piedmont residual soils of eastern Alabama with the shear wave velocity results compared with adjacent seismic piezocene profiles, crosshole geophysical arrays, and spectral analysis of surface waves (SASW).

1 INTRODUCTION

The seismic flat dilatometer test provides excellent potential for efficient geotechnical site investigation. Many field and laboratory studies within the past sixteen years have shown the dilatometer interpretations of stratigraphy, material classification, and strength parameters to be reasonable and reliable with site specific calibration (Lutenegger 1988). With the addition of a velocity transducer, nondestructive properties such as shear wave velocity, shear strain, and small-strain shear modulus can be measured directly with minimal additional time and effort. The seismic measurements provide information about the small-strain behavior of the soil which is essential for analysis of monotonic loading (such as foundations), liquefaction analysis, evaluating dynamically loaded foundations, and earthquake engineering problems. A prototype seismic dilatometer was fabricated at Georgia Institute of Technology (Kates 1996). The seismic dilatometer was field-tested in the Piedmont residual soils near Opelika, Alabama, with the shear wave velocity results compared with adjacent seismic piezocene profiles (SPT), crosshole geophysical arrays (CHT), and spectral analysis of surface waves (SASW).

2 SEISMIC DILATOMETER TEST SETUP

The test apparatus of the seismic flat dilatometer consists of a traditional standard stainless steel high-strength blade, internally-wired plastic tubing, and a dual-gage pressure control panel. Regulated compressed nitrogen is used to expand the membrane and obtain the lift-off ($p_b$) and expansion ($p_e$) pressures. Seismic capabilities are facilitated by the addition of a velocity transducer positioned 0.25 m above the center of the dilatometer membrane. The receiver geophone is a Geospace type 14 model 1.9 with a natural frequency of 28 Hz and sensitivity of 0.236 volts/cm/sec. The trigger geophone, which is a Mark Products model 1-15A, is attached to the source plumb to determine the start time of the shear wave. Both geophones are connected to an oscilloscope to evaluate the travel time of the wave from the source to the receiver. A portable matrix dot printer is used to print each wave for later reference. A schematic of the field setup for the seismic dilatometer is provided as Figure 1.

The co-axial receiver geophone cable has been taped to the standard DMT tubing so that the threading process through the push rods takes minimal additional time. The receiver geophone is added to the adaptor rod (which connects the blade...
to the push rods) by dismantling a section that was machine-cut from the rod. The plate is removed simply by loosening two set screws. With the geophone secured against the inner wall of the rod, excellent soil-to-receiver contact is assured. Once the geophone is inserted and cable connected, the test is ready to proceed.

The seismic dilatometer test progresses as a traditional flat dilatometer sounding with the wave travel time measurements taken at the short pause for each successive push rod to be added. The flat blade is pushed vertically in the ground, usually by hydraulic force, stopping at particular intervals to measure the lift-off ($p_0$) and expansion ($p_1$) pressures of the membrane (Schmertmann 1988). During the addition of each push rod (generally 0.9 m intervals), shear waves are sent through the subsurface media to determine the wave travel time. The shear wave source used for the seismic dilatometer tests was a horizontal wood board. A static load over a wooden plank assists in producing waves rich in shear and weak in compression. Prior to loading the source, any thick surface vegetation or gravel particles should be removed so that the plank is in direct contact with the soil.

All measures of the shear wave source with consideration of the orientation of the geophone is essential to receive a signal with minimal interference from the faster compression waves. Because shear waves propagate perpendicular to the direction of particle motion, the shear wave source should be aligned perpendicular to the direct travel path of the shear waves to the dilatometer hole as seen in Figure 1. The amount of compression waves detected by the receiver geophone is minimized with this arrangement because compression waves propagate parallel to particle motion away from the receiver.

The trigger geophone is attached to the wooden plank to signal the start time of the impulse and the receiver geophone within the adapter rod signals the arrival of the shear wave down the hole at the known depth. The travel time of the wave is determined as the time difference in the initial responses of the trigger and receiver signals. Several travel time measurements at each depth were made to assure repeatability and reliability of the data. The wave signals at each depth were recorded for later reference by downloading to a printer.

The shear wave velocity determined from this version of the seismic dilatometer is the pseudo-interval velocity. That is, the difference in travel
distance at two consecutive test depths is divided by the difference in arrival time for the two depths. Travel times are determined from two separate wave measurements with the same receiver in different vertical locations. Another method to calculate the shear wave velocity is the true-interval method. In order to obtain the true-interval velocity, two receivers located at a set distance apart are used simultaneously. The difference in the initial responses from the two receivers can be measured directly on the oscilloscope from a single impulse. Less test error is associated with the true-interval method because the travel time is determined from a unique wave (Burgin and Zlotnicki 1991). Additionally, the travel time does not have to be assayed from the more disrupted first arrival. The first arrival of the shear wave is often difficult to choose due to interference from the faster compression wave and other subsurface waves (refracted waves, Love waves, etc.). Using the true-interval method can also eliminate errors associated with the trigger signal. A major drawback of the true-interval method is the added complexity and cost due to the necessity of at least two receiver geophones and coaxial cables as well as a multi-channel oscilloscope. Differences between results using the pseudo- and true-interval velocities were found to be less than 10% by Robertson et al. (1986).

The pseudo-interval velocity is a much more cost-effective means to evaluate the shear wave velocity of soils. Only one geophone and cable is added to the test set up and threading process. Also, simple signal conditioners may be used rather than the more complex and expensive multi-channel oscilloscopes required for the true-interval method. However, some errors may be introduced due to trigger delays, interpretation of the first arrival of the shear wave, and variable impulses when utilizing the pseudo-interval method.

3 FIELD TESTS AT OPELIKA, ALABAMA

The geology of the Opelika test site is referred to as the Piedmont province (Figure 2). The Piedmont region is bordered by the Blue Ridge on the west and Atlantic coastal plain on the east and extends from Pennsylvania southwest into Alabama. The subsurface materials are generally composed of silty to sandy residual soils underlain by partially weathered rock. Residual soils have resulted from the chemical and physical weathering of the underlying parent rock formations which consist of Paleozoic metamorphic and igneous rocks, primarily schists, gneisses, and granites. The weathering process is accelerated in the southeast due to the temperate to warm climate, abundance of rainfall, established vegetation, and lack of glacia tion of the region (Sowers and Richardson 1983). The subsurface profile in the Piedmont province generally consists of an upper zone of completely weathered soil, an intermediate zone made up of saprolite with soil texture, a partially weathered zone with alternate scours of soil and rock, followed by a natural to slightly weathered rock (Martin, 1977). At the Opelika test site, the overlying silty to sandy soils are believed to be derived from schists and gneisses.

Figure 2. Location of the Piedmont Province.

Three seismic dilatometer soundings were performed at the Opelika test site in August of 1996. A Diedrich D50 drill rig was utilized to push the blade from the surface to test depths of approximately 8 m. The blade was not pushed to further depths because the drill rig was light and could not supply an adequate reaction force or provide sufficient lateral support to prevent buckling instability of the push rods. The dilatometer can generally be pushed to the vicinity of bedrock based on previous experience with larger drill rigs more commonly used for in-situ push tests.

Lift-off (pL) and expansion (pD) pressures were measured at 0.3 m intervals. The shear wave velocity (V_s) was calculated based on the responses of the trigger and receiver geophones, as previously described. Figure 3 provides the raw data from the three seismic dilatometer tests performed at the Opelika test site.
The Opelika test site was advantageous for demonstrating the applicability and validity of the SDMT for in-situ characterization due to alternate data collected for cross-comparison. Soil borings, cone penetrometers, and laboratory tests were performed to evaluate the subsurface stratigraphy and strength characteristics for the site. Several tests in addition to the SDMT were performed to evaluate the seismic characteristics of the soils.

The shear wave velocities measured using the seismic dilatometer were compared with results from seismic piezocene tests, a geophysical crosshole test, and a SASW survey. Both the SCPTs and SDMTs were performed in a downhole manner with the source remaining at the surface and a single receiver pushed to various depths in the subsurface. Due to using only one receiver, the velocity calculated is the pseudo-velocity. The shear waves for the two direct-push seismic tests were instigated by striking a horizontal plank with a sledge hammer. Therefore, the waves were horizontally-polarized shear waves (SH-waves). Perhaps the most significant advantage of the SCPT and SDMT is that stratigraphic and strength information are collected in conjunction to the shear wave velocity measurements in a single sounding.

The crosshole tests consisted of three aligned boreholes cased with plastic pipes and grouted in place. The downhole source and two receivers were lowered into the holes to equal depths and clamped in place using pneumatically inflatable rubber packers. The source produced vertically-polarized shear waves (SV-waves). The shear wave velocities computed from the crosshole tests are the direct-velocities. That is, the velocity is calculated as the measured travel time divided by the measured travel distance for a single impulse. The shear waves are assumed to travel along a horizontal path from the source to the receiver. Crosshole tests are generally considered the bench mark test for shear wave velocity measurements but can sometimes be costly and time consuming due to the laborious procedure of drilling, casing, and grouting the boreholes.

Spectral analysis of surface waves (SASW) is a noninvasive method of measuring the shear wave velocity profile. Geophones are placed at multiple spacings to evaluate surface waves (Rayleigh waves) with a range of frequencies to decipher the shear wave velocity. A numerical inversion of the data is required for interpretation.

4 INTERPRETATION OF RESULTS
Stratigraphic information gathered from the dilatometer tests agree relatively well with laboratory tests and other in-situ tests performed at the site. The visual classification from the boring logs generally identified the upper two meters as a
very stiff clay and silt with an average blow count (N) value of 13 blows/0.3 m. The material index values from the dilatometer indicated this region to be composed of silty sand to silt. Laboratory analyses of samples retrieved from depths of less than 3 m generally indicated a silty fine sand material, agreeing with the classification from the dilatometer. Laboratory tests and boring log information denote the soil in the 2 to 7 m zone to be primarily composed of a silty sand. The dilatometer classification and laboratory analyses registered the material in this zone as a sandy silt.

Seismic dilatometers SDMT1 and SDMT2 were performed approximately 1.5 m away from two seismic piezocone tests. The SDMTs were placed very close to seismic piezocene soundings in an attempt to decrease the impact of soil variability on the measured shear wave velocities of the two tests. These tests were also relatively close to the crosshole and SASW tests.

Figure 4 presents a comparison of the shear wave velocity profiles determined using the seismic dilatometer with the profiles from the seismic piezocene penetrometer, crosshole test, and SASW test. Shear wave velocity measurements from seismic dilatometer tests agree well with results from the other seismic tests performed at the site. Because the seismic core and the seismic dilatometer are both downhole tests with a single receiver measuring horizontally polarized shear waves (SH-waves), the velocity profiles are expected to compare favorably.

Shear wave velocities measured by the SDMT compare favorably with neighboring crosshole results displaying a similar trend and values. However, the measurement from the crosshole test are not expected to display an exact agreement with the SDMT due to differing test setups (downhole measurements with a single receiver versus crosshole measurements with two receivers) and measuring waves that propagate in different directions.

The SASW measurements agree well with the SDMT results as seen in Figure 4. In a more non-homogeneous material, the velocity measurements from downhole tests and SASW would most likely not agree as well due to the averaging mechanisms of SASW and analyzing larger volumes of the material. Additionally, SASW evaluates a different wave form than the SDMT test.

Velocity profiles from each test method are shown together in Figure 5. The general shear wave velocity trend for the upper 8 meters is around 260 m/s. The difference between the shear wave velocity measured using the SDMT and other test methods was generally less than 25 m/s and
never more than 75 m/s. Considering soil variability, error introduced by averaging mechanism of pseudo-interval method, resolution limitations of electronics, operator judgment of picking the first arrival, and different operators for each test, the velocity profiles for the Opelika test site display acceptable agreement. The validity of the downdrill measurements of the seismic dilatometer test for evaluating the shear wave velocity of soils is confirmed by the correspondence with existing geophysical tests such as the seismic piezocene, cross-hole test, and SASW.

5 APPLICATIONS OF THE SDMT

The seismic flat dilatometer test provides a simple and cost-effective means for stratigraphic delineation and assessing soil stiffness and strength parameters for low-, intermediate-, and high strain conditions. The conventional flat dilatometer test provides an evaluation of the undrained shear strength (s,0) in clays and the effective friction (P') angle in sands. With the addition of a velocity transducer, downdrill geophysical measurements can be performed to directly measure the shear wave velocity (V_s). Shear wave velocity of soil is a nondestructive property that is useful for solving problems involving earthquake engineering, dynamically-loaded foundations, assessment of liquefaction susceptibility, and for determining the small-strain shear modulus (G_mod) of the soil. In addition to dynamics applications, the stiffness of soils at low strain levels is also relevant to monotonic deformation problems, such as foundation settlement (Burland, 1989).

6 CONCLUSIONS

A seismic dilatometer apparatus was constructed and field tested at a residual silty sand site in Opelika, Alabama. Conventional dilatometer data compared well with material classification and strength parameters from field and laboratory tests. Validity of downdrill shear wave velocity profiles was confirmed by comparison with existing in-situ tests such as seismic piezocene tests, cross-hole arrays, and SASW surveys.

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Measurement of matric suction in compacted soils used modified tensiometer

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ABSTRACT: Measurement of matric suction of soils in the laboratory or in the field is important for dealing with unsaturated soil problems. A tensiometer is one of several devices which can be used for the measurement of negative pore-water pressures in a soil. This paper presents a study on a modified tensiometer. The theoretical concept of measuring negative pore-water pressure in the soils through the use of the modified tensiometer is similar to that proposed by several other researchers. Results from this study suggest that the modified tensiometer is suitable for measuring negative pore-water pressure in soils.

1 INTRODUCTION

Cronin (1952) found that the role of matric suction was closely related with to practice in geotechnical problems. Jennings (1961) and Atchison (1961) recognized that matric suction was valid in explaining unsaturated soil behavior. Matyas and Radhakrishna (1968) described the volume change of an unsaturated soil subjected to increasing loading by controlling the matric suction and confining pressure. Fredlund, Morgenstem and Wiberg (1978) defined the equation for the shear strength of an unsaturated soil using matric suction and confining pressure. As shown above, matric suction is necessary for engineering practice in unsaturated soils.

1.1 Purpose of this study

In this study, the negative pore-water pressure in compacted silty soils at various water contents were directly measured using a modified tensiometer. Measurement of the negative pore-water pressure in a compacted soil was conducted at the surface of the compacted soils and within the compacted soils. This study investigate both the equilibrium time and stability of the long time measurement with respect to the negative pore-water pressure using the modified tensiometer.

1.2 Review of tensiometer

Matric suction can be measured either in a direct or indirect manner. Tensiometer directly measure the negative pore-water pressure in a soil. The measurement of negative pore-water pressure through the use of a tensiometer can reveal the vertical profiles or gradients of matric suction in the field. Tensiometers have been used to measure negative pore-water pressure for numerous geotechnical engineering applications (Sweeney 1982, Krah, Fredlund and Klassen 1989). The tensiometer, formally named by Richards and Gardner (1936), has undergone many modifications for its application to specific problems. The tensiometer usually consists of a porous ceramic cup connected to a pressure measuring device through a small bore tube. However, the basic components have remained unchanged. There are several types of tensiometers; for example, there are the jet tensoimeter, small tip tensiometer and the Quick draw tensiometer. A variety of tensiometer designs are available. Some tensiometers can be purchased commercially. The principle of measurement of negative pore-water pressure in the soil is similar for each tensiometer. Stannard (1992) described the theoretical and practical considerations pertaining to the successful on-site use of commercial and fabricated tensiometer.
The tube and the ceramic cup in the tensiometer are filled with desired water. The ceramic cup is used as an interface between the soil and the pressure measuring system. As water in the ceramic cup acts as a link between the pore-water in the soil and the water in the measuring system, a direct measurement is possible for the negative pore-water pressure. The pore-water pressure that can be measured in a tensiometer is limited to approximately 50 kPa negative due to cavitation of the water in the tensiometer. The most common air entry value for tensiometer ceramic cup is one bar (100 kPa). The use of a ceramic cup with an air entry value greater than 100 kPa will not extend the measuring range of tensiometers since water in the tube will always cavitate when water pressure approaches approximately -90 kPa.

The continuous hydraulic connection between the water in the porous cup and the water in the soil is essential to the collection of accurate tensiometer readings. Gaps between the porous cup and soil increase the tensiometer response time by reducing the effective area of the porous cup. In the worst case, when no hydraulic connection exists between the porous cup and the soil, soil-water tension cannot be measured. Installation of the tensiometer must ensure hydraulic connection with minimum disturbance of the soils. Therefore, the installation of the tensiometer requires care and attention to detail.

1.3 Future of the modified tensiometer used in this study

In this study, a regular tensiometer is modified. The components of the modified tensiometer are the ceramic disk, the electrical pressure transducer with no plastic tube being used. There is a small space between the ceramic disk and the pressure transducer in the tensiometer. The small space is filled with desired water and has a volume of 1.92 cc. The modified tensiometer uses a ceramic disk (instead of a ceramic cup), and so the boundary between the soil and the surface of ceramic disk is plane. The plastic tube used in a regular tensiometer is relatively permeable to air (Stanwood 1992). The water in the modified tensiometer is covered with metal, and therefore air diffusion is eliminated. The modified tensiometer is made mostly out of metal. The tensiometer can be buried in soils. The modified tensiometer does not affect the hydraulic pressure since there is no tube from which to pull the water. The modified tensiometer is equipped with an electrical pressure transducer, and so the modified tensiometer is suited for remote automatic collection of large quantities of data.

1.4 Theory of soil suction

The soil suction theory was mainly developed in relation to the soil-water system (Richards and Fiereman (1943), Corey and Kemper (1961)). The soil suction is related to the free energy of the soil water. The free energy of soil water is called total suction. Total suction consist of matric suction and osmotic suction. Total suction is expressed as equation (1).

\[ \psi = (\psi_m - \psi_a) + \pi \]

where

- \( \psi \) = total suction
- \( \psi_m - \psi_a \) = matric suction
- \( \psi_a \) = pore-air pressure
- \( \psi_m \) = pore-water pressure
- \( \pi \) = osmotic suction

The matric suction is associated with the capillary phenomenon. The capillary phenomenon is caused by surface tension created by the intermolecular forces in the air-water interface. The magnitude of surface tension is 72.75 mN/m at a temperature of 20 degrees. The capillary water has a negative pressure with respect to the air pressure, which is generally atmospheric (i.e., \( \psi_a = 0 \)) in the field. The surface of water in the capillary tube in the soil is in the form of a meniscus.

2 TEST PROGRAM

2.1 Soil

The soil used in this study is a silty soil. The physical properties of the soil are shown in Table 1. The soil is a non-plastic material. The soil has a uniform grain size distribution (Fig. 1).

2.2 Tensiometer

Fig. 2 shows an illustration of the modified tensiometer. The length of the tensiometer is 100 mm. The diameter of the tensiometer is 40 mm.
The air entry value of the ceramic disk used in the tensiometer was 200 kPa. The ceramic disk is 20 mm in diameter and 5 mm in thickness. The water confined in the space between the electrical transducer and the ceramic disk has a total volume of 1.92 cc. The pressure transducer is connected to a data logger so that data can be collected automatically.

2.3 Test procedure / TEST 1

The container used in this test has a diameter of 475 mm and a height of 580 mm. The soil was compacted in the container. The compacted soil had a height of 80 mm. A modified tensiometer was buried in the soil. The container was covered, and so the water content was kept unchanged during measurement of the negative pore-water pressure in the soil. The negative pore-water pressure was measured for twelve days in the laboratory.

2.4 Test procedure / TEST 2

The compacted soil was prepared in the container used in TEST 1 which had a height of 400 mm. Four modified tensiometers were buried in the soil at depths of 80 mm, 160 mm, 240 mm and 320 mm from upper surface of the soil. The container was covered, and so the water contents in the soil were kept unchanged during the tests. The measurement of the negative pore-water pressure was conducted over seven days in the laboratory.

2.5 Test procedure / TEST 3

The compacted soil was prepared in the container used in TEST 1 which had a height of 400 mm. Four modified tensiometers were buried in the soil, similar to TEST 2. The container was not covered, and so the water content was allowed to decrease during the tests. A thermocouple and a hygrometer were installed near the upper surface of the compacted soil. The relative humidity in the laboratory and the negative pore-water pressure in the soils were measured over eight days.

2.6 Test procedure / TEST 4

The soils were compacted in a compaction mold at various initial water contents. The compacted soils and the compaction molds were
put into the container. The compacted soils had a diameter of 100 mm. A modified tensiometer was placed against the surface of compacted soil with minimum disturbance to the hydraulic continuity in the soil. The container was covered, and so the water content in the soil specimens were kept unchanged during the tests. The negative pore-water pressure in the soil specimens were measured in laboratory.

3 TEST RESULTS

3.1 Change in the negative pore-water pressure at constant water content (TEST 1, TEST2)

The air-water interface in the unsaturated soil or compacted soil has a negative pore-water pressure. The surface tension is generated along the contractile skin in the soil. The pore-water pressure in the soil is related to the soil water content. The pore-water pressure indicates a highly negative values for low degree of saturation.

The compacted soils were prepared in TEST 1 with a water content of 0.6 % and 19.0 %. The negative pore-water pressure in the soils at a water content of 0.6 % was measured by the modified tensiometer. The change in the negative pore-water pressure with elapsed time is shown in Fig. 3. The decrease in the pore-water pressure shows a linear form on a logarithmic scale of elapsed time. Water in the tensiometer was attracted by strong tension to the soil particles, and so a high negative pore-water pressure was measured. The pore-water pressure that can be measured in the modified tensiometer is limited to approximately negative 90 kPa. The tensiometer indicated -93 kPa at 4500 minutes. Subsequently, the pressure quickly increased toward zero (i.e., atmospheric pressure). Air in the soil diffused through the pore in the ceramic disk. When the tensiometer was removed from the soil it was found that the all of the water in the tensiometer has been replaced by air.

The measurements for the pore-water pressure in the soils at a water content of 19.0 % is shown in Fig. 4. Fig. 4 indicates a stable collection of data with respect to time. The response time of the tensiometer is associated with the sensitivity of pressure transducer and the permeability of the ceramic disk.

Fig. 5 shows the change in the negative
pore-water pressure in the soils against depth with time. The modified tensiometers were exposed to atmospheric conditions before installation in the soil. Water evaporates from the tensiometer and a negative pressure was registered. The tensiometer registering negative pressure was installed in the soil. The data shows scatter after ten minutes as shown in Fig. 5.

The measured pressure indicated a stable value with elapsed time. The modified tensiometer buried in the soil worked well for a long time. After seven days, each tensiometer indicated -29.2 kPa, -29.8 kPa, -37.3 kPa and -30.2 kPa of pore-water pressure in order of depth. The results obtained from the four tensiometers were similar because the water contents in the soil were constant with depth. Results show that the four tensiometers buried in the soils had good continuous hydraulic connection between the ceramic disk and the water in the soil.

3.2 Change in the negative pore-water pressure due to drying (TEST 3)

The difference between TEST 2 and TEST 3 is that the compacted soils were allowed to change in water content as the container was not covered in TEST 3. Fig. 6 shows the change in the negative pore-water pressure in the soil. The negative pore-water pressures remained constant for about five hundred minutes. After five hundred minutes, the pore-water pressure became more negative, regardless of depth. The result shows that the water content in the soils decreases due to drying from the surface of the compacted soil. A decrease in the pore-water pressure in the soils continued for eight days. The negative pore-water pressure changes linearly against elapsed time on a logarithmic scale.

Fig. 7 shows measurements of temperature and relative humidity near the upper surface of the compacted soils in the laboratory. The temperature changed from 28 degrees to 36 degrees. The relative humidity changed from 47% to 79%. A decrease in the pore-water pressure in the soils was maintained in the tests regardless of the fluctuation of both temperature and relative humidity.

3.3 Relationship between matric suction and water content (TEST 4)

The relationship between water content and dry density in compacted soils is shown in Fig. 8. The negative pore-water pressure was measured at the upper surface of the soil specimens. On the assumption that the pore-air pressure is atmospheric, the measured negative pore-water pressure is numerically equal to the matric suction. The relationship between matric suction and water content in compacted soils is shown in Fig. 9. It appears that the matric suctions are dependent on the water content of the soil. The matric suction increased due to a decrease in water content. However, when the water content was less than 5%, the matric suction decreases. The continuous hydraulic connection between the ceramic disk and the soil is not adequate or gaps have appeared between the ceramic disk and the
soils. The measured negative pore-water pressures were uncertain due to poor contact.

4 CONCLUSIONS

The negative pore-water pressure in a compacted soil was measured using a modified tensiometer. The results of this study are as follows. 1. The measurement of the negative pore-water pressure by the modified tensiometer is stable at a constant water content. 2. The modified tensiometer can measure the changes in the negative pore-water pressure in the compacted soils due to drying. 3. The negative pore-water pressure in the soil depends on the water content. 4. Continuous hydraulic connection between the ceramic disk and the soil is important for the obtaining correct negative pore-water pressures in the soil. 5. The modified tensiometer can be used to measure negative pore-water pressures in a soil.

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Tensiometers; their design and use for civil engineering purposes

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ABSTRACT: The design of tensiometers is reviewed with particular reference to their use in engineering soils. A new tensiometer is described that can for the first time measure absolute pore water suction in excess of 100 kPa. Tests are presented in which comparisons are made between measurements using a normal tensiometer and those using the new tensiometer.

1. INTRODUCTION

Over the past few years a great deal of effort has been given to developing techniques for measuring soil suction and within the subject of soil science a few have evolved that can make a successful and reasonably accurate measurement (Ridley and Wray, 1995). However only the tensiometer is capable of making a direct measurement (i.e., an absolute tensile measurement) under normal atmospheric conditions.

Stansard (1992) gave a comprehensive review of the construction of standard, commercially available tensiometers and it is not our intention to include any unnecessary repetition here. Rather, our aim is to highlight a few misconceptions regarding the theory of operation of tensiometers and the potential pitfalls that can be encountered when using them for engineering purposes.

2. CONSTRUCTION

A tensiometer consists of a porous filter and a means of measuring stress which are separated by a fluid (usually water) filled reservoir. It works by allowing the water to be extracted from the reservoir in the tensiometer into the soil, until the stress holding the water in the tensiometer is equal to the stress holding the water in the soil (i.e., the soil suction). When this condition is present no further exchange of water will occur between the soil and the tensiometer. The suction will then manifest itself in the reservoir as a tensile stress in the water and can be measured on the stress measuring instrument (e.g., a manometer, a vacuum gauge or an electronic pressure sensor).

In specifying the design of a tensiometer one can vary (a) the location of and type of pressure sensor, (b) the porous filter, (c) the volume of the fluid reservoir and (d) the material from which the tensiometer is constructed.

Whilst researching the movement of moisture beneath covered areas Black, Crouse and Jacobs (1958) used a tensiometer that had a glass tube with a fused bore, a sintered glass filter with an average pore size of 1 micron and a mercury manometer.
located remote from the filter (figure 1). Using this arrangement they were able to make suction observations up to about 94 kPa and state that this was extended by up to a further 100 kPa using filters with a smaller average pore size.

( Ridley and Burland, 1993 ) with a vastly reduced reservoir (~15 mm³) made from stainless steel, a pressed kaolin ceramic filter with a pore diameter of about 0.2 microns and a high range electronic pressure sensor located close to the filter. Ridley and Burland (1993) improved the design of their suction probe by reducing the volume of the reservoir to about 3 mm³ (figure 3). Using a specific preconditioning of the suction probe direct measurements of suction in excess of 1800 kPa have been made.

Figure 3. Suction Probe (after Ridley and Burland, 1995)

3. RANGE OF MEASUREMENT

The four factors that restrict the measurement range of tensiometers are (i) the procedure used to remove air from the reservoir, (ii) the volume of the fluid reservoir, (iii) the material used to manufacture the reservoir and (iv) the pore size of the filter.

The range of operation of all tensiometers is limited by the formation of vapour cavities within the fluid reservoir. In the field of soil mechanics the phenomenon of cavitation has been used to explain the formation of air in pore pressure measuring systems that are subjected to a pore water tension.

Cavitation is the formation of vapour cavities within the liquid itself or at its boundaries with another medium. The theory for the tensile strength of pure liquids predicts that a vapour cavity will only form when the liquid is placed under extremely high tension (i.e. about 50 MPa), or when the liquid is supersaturated. Since neither of these conditions exist within the tensiometer when air forms within the reservoir, its presence cannot be due to a rupture of the molecular bond between adjacent water molecules. However, imperfections that exist in the surface of objects, even after the finest machining or polishing, provide an ideal trap for tiny amounts of air that can remain after thorough de-airing using

More recently the design of commercially available tensiometers has changed to make use of modern and cheaper materials. The most common construction uses a transparent stiff nylon tube with a 9.7mm bore, a pressed kaolin ceramic filter with a pore diameter of about 2 microns and a differential vacuum gauge located remote from the filter. In addition to the normal attributes this type of tensiometer often incorporates a reserve supply of water mounted in a storage container at the top of the tensiometer (figure 2). This is used to replace any air that may form within the measuring system with water. This type of tensiometer has received widespread use and can, with careful preparation, make measurements up to about 90 kPa. However, in some circumstances they will give quite misleading measurements, as will be discussed later.

In 1990 the senior author first observed the ability of water to sustain tensile stresses in excess of 100 kPa. This led to the design of a new tensiometer

Figure 2. Vacuum gauge tensiometer
A vacuum pump is then applied to the top of the tensiometer and used to remove any large air bubbles from the reservoir and gauge prior to fixing the storage container to the top of the tensiometer.

The suction probe is assembled dry, and water is introduced into the filter and the reservoir by (i) placing the tensiometer in an evacuated chamber, (ii) allowing the chamber to fill with de-aired water such that the filter is immersed and (iii) allowing the chamber to return to atmospheric pressure whilst keeping the tensiometer immersed. After this the tensiometer is removed from the chamber and placed in a manifold that allows a hydraulic pressure of 4000 kPa to be applied for a period of about 24 hours.

Even in a thoroughly de-aired tensiometer, air can form in the fluid reservoir when the difference between the tensile stress in water within the reservoir and the atmospheric air pressure outside the tensiometer is equivalent to the air entry value of the porous filter. Ridley and Burland (1995) demonstrated this to be so by inserting a range of different ceramic filters into their suction probe and allowing water to evaporate from the exposed surface of the filter.

4. INSTALLATION

The mercury manometer tensiometer was installed inside a 4 inch diameter borehole with the porous filter placed firmly in contact with the soil at the bottom of the hole. The remaining space inside the borehole was backfilled with soil cuttings to the same density as the in situ soil.

The vacuum gauge tensiometer is installed inside a 7/8 inch diameter (tight fitting) hole with the vacuum gauge just above the ground surface. Contact between the ceramic filter and the soil can be enhanced by using a slurry of cuttings removed from the hole that is poured into the base of the hole and used to seal the filter in.

Ridley and Burland (1996) installed their suction probe inside a 2 inch diameter borehole lined with PVC tubing. This was done to facilitate (i) the easy removal of the tensiometer and (ii) in order that small moisture content samples could be gathered prior to making a suction measurement.

5. FIELD MEASUREMENTS

The operation and interpretation of measurements made using two different designs of tensiometer will now be presented. Unfortunately no glass mercury...
manometer tensiometers were available for use and therefore the observations are restricted to those made with a nylon vacuum gauge tensiometer and a suction probe.

A number of vacuum gauge tensiometers were installed in a 70 year old vegetated embankment that consisted of clay fill overlain by a variable thickness (up to 3m) of granular fill. The ceramic filters were located approximately 50 cm into the clay fill. Figure 5 shows observations made over an eight month period on a tensiometer buried at a total depth of 1.21m and with the vacuum gauge deliberately left protruding 50 cm above the ground surface. Therefore gauge measurements have been reduced by about 17 kPa to give the measurements presented in figure 5.

![Image of Vacuum Tensiometer Setup]

Figure 6. Test arrangement for laboratory comparison

In the first comparison the ceramic filter of a carefully de-aired vacuum gauge tensiometer and a suction probe were buried in a clayey silt within a chamber that was subjected to bottom drainage (Figure 6). Hydraulic continuity was maintained at the bottom of the sample and a tension was applied to the water in the drainage system by reducing the pressure in the air above the water surface in a stepwise manner. The exposed surface of the soil was covered with a perspex disc and a layer of lead shot.

Although the suction measured by the tensiometer are entirely believable the presence of air within the tensiometer gave some cause for concern. When the air was removed from the tensiometer by flushing from the storage container, more air would quickly form. This means that the no flow condition required for an accurate suction measurement was not being achieved. Furthermore, in some instances, if a tensiometer was left without flushing air from the reservoir, the quantity of air would continue increasing until eventually the tensiometer would record a zero suction, a measurement that was obviously incorrect. The concern with these observations was that it is normal practice to bury a tensiometer of this type so that the vacuum gauge is located just above ground level. Under such circumstances the presence and amount of air in the reservoir could be uncertain and the measurements could be misleading.

![Graph of Air Flushing from Tensiometer]

Figure 5. Field measurements with a vacuum gauge tensiometer

6. COMPARISONS

As a result of the field observations made using vacuum gauge tensiometers a series of controlled laboratory comparisons of measurements made with a vacuum gauge tensiometer (fitted with an electronic sensor) and a suction probe were undertaken.
When the suction applied to the soil was increased in increments of 5 kPa, good agreement was obtained between the two tensiometers (figure 7) and no air was observed to form in the reservoir of the nylon tensiometer at suction up to 25 kPa. However, at a suction of 25 kPa an air gap was observed to form at the top of the reservoir in the nylon tensiometer. Following each increment of suction up to a value of 50 kPa the air gap eventually ceased increasing in size (suggesting that a flow equilibrium was being reached) and the agreement was still good between the two tensiometers. After a few hours at a stable suction of 50 kPa the size of the air gap in the vacuum tensiometer and the suction recorded by the sensor started increasing again (figure 8). At the same time the tension in the drainage line started decreasing and large amounts of air were observed flowing through the drainage line. During this stage of the test, the suction measured by the suction probe decreased, but only very slightly. This was probably caused by the flow of water out of the vacuum tensiometer.

![Figure 8 Observations at a suction of 50 kPa](image)

Either the air entry value of the soil had been reached or shrinkage cracks had formed in the soil, allowing air to pass through it. Laboratory tests on the soil indicated that the air entry value was in excess of 150 kPa. To overcome the problem of air passing through the soil a vertical stress was applied to the top of the sample and a wax seal was poured over the top cap. Following further increments of the suction, agreement between the two tensiometers remained good up to a suction of 75 kPa. Throughout this series of tests the response time of both tensiometers was quick but a small temperature sensitivity was observed in the suction probe.

In a second comparison the tip of a carefully desired vacuum gauge tensiometer was buried in a compacted clay that was known to have a suction in excess of 500 kPa. The response of the tensiometer was almost immediate with the gauge measuring a tension of about 60 kPa (figure 9) and an air gap was observed to emerge at the top of the reservoir. The air gap was observed to increase in size whilst the tensiometer continued to record a suction of about 60 kPa. When the water level in the tensiometer had dropped below the vacuum gauge the reservoir was vented to atmosphere through the storage container but it was deliberately not filled with water. Upon closing the vent, the reading on the gauge increased at a much slower rate, eventually reaching a maximum value of 44 kPa. The quantity of air in the reservoir continued to increase and eventually the water level in the tensiometer disappeared below the surface of the clay. After 29 days the reading on the vacuum gauge dropped rapidly to zero (a value usually associated with a very low suction or a positive pore water pressure) and when the tensiometer was removed from the test pit it was found to have completely emptied of water. The suction measured by a suction probe installed adjacent to the vacuum tensiometer was 570 kPa (figure 10).

![Figure 9 Observations with a vacuum tensiometer in a high suction environment](image)

![Figure 10. Suction probe measurement](image)
DISCUSSION

These results presented in this paper highlight some important situations where the observations made with a large volume nylon tensiometer can be misleading. Firstly, at suction in excess of that required to cause cracks to form in the soil air can readily form in the reservoir of nylon type tensiometers.

A crack will form within the soil mass when the horizontal total stress reduces to zero. Therefore, for a given vertical total stress the suction that will cause cracking is given by

\[ \sigma_h = K_0 \sigma_v + u_w (1 - K_0) \]

For normally consolidated soils \( K_0 = 1 - \sin \psi \) and when \( \sigma_v = 0 \) the above expression becomes

\[ -u_w = \sigma_v (1 - \frac{1}{\sin \psi}) \]

It can be seen that for most soil soils the suction required to cause cracking at relatively shallow depths will be low (eg. less than 100 kPa).

Although the suction required to cause cracks to form in stiffer soils should be higher than this the slurry paste that is often used to provide a good contact between the in situ soil and the ceramic tip of the tensiometer is still likely to crack at quite low suction. Once a crack has formed, contact between the ceramic tip and the soil will not be good. Uncontrolled evaporation could then occur from the ceramic tip into the cracks. This would cause the tip to dry and increase (i) the potential for the loss of more water from the tensiometer and (ii) the suction recorded by the tensiometer. A prolonged loss of water from the tensiometer may eventually lead to a localized increase in the water content of the soil and a commensurate decrease in the suction. Therefore the cracks could close and contact between the ceramic tip and the soil might be re-established.

As the nylon tensiometers remain buried it is impossible to detect the presence of cracks in the paste. However, it has been shown that the presence of such cracks can lead to misleading observations.

A second and potentially more worrying observation is that, if a large volume nylon tensiometer is installed in a soil with a suction much higher than 100 kPa, the quantity of air in the tensiometer will increase in an uncontrollable manner and the sensor reading will rise to a perfectly believable, but radically incorrect value.

CONCLUSIONS

The presence of cracks in the soil adjacent to the ceramic tip of a large volume tensiometer can result in misleading observations of suction. Such cracks can form at relatively low values of suction.

In a high suction environment, large volume tensiometers can display very believable, but radically incorrect values of suction.

The suction recorded on a large volume tensiometer can only be correctly interpreted by careful observations of the quantity of air inside the tensiometer.

The practice of flushing air from the tensiometer reservoir without first noting its rate of growth could lead to a serious misjudgment of the suction in the ground.

REFERENCES


International development of the field compressometer

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ABSTRACT: The field compressometer (or screw plate) test was originally developed at the Department of Geotechnical Engineering of the Norwegian Institute of Technology (now the Norwegian University of Science and Technology, NTNU). After an initial research phase of about 15 years in Norway, interest shifted abroad. Two decades of international research activity have produced significant advances in the test equipment and techniques which complement the original Norwegian techniques. This paper reviews the field compressometer test methods, equipment and interpretation procedures developed outside of Norway.

1 INTRODUCTION

The field compressometer test (FCT), or screw plate test, is an in situ plate load test originally developed in Norway for the in situ measurement of deformation characteristics. The initial development stage of the FCT ended in 1970, and no further FCT research was conducted in Norway until the early 1990s. The FCT is undergoing renewed research at the NTNU, and the Norwegian development of the FCT from inception to the present is presented in another paper to this conference.

Although the FCT was dormant in Norway from 1970 until about 1990, the international research community was very active with the test method and has contributed significantly to the equipment, test procedures and interpretation theory. The first international activities can be characterized as extensions and improvements to Norwegian practice for testing in sand. The next major step was the development of an elasticity-based interpretation theory as an alternative to the semi-empirical plasticity approach used in Norway. In conjunction with this, the use of the FCT was then expanded from sands and silts to include clays and silty soils. The most recent international research evaluates plate flexibility and installation disturbances.

2 ADAPTATIONS TO NORWEGIAN THEORY

The first use of the FCT internationally appears to be in the United States. Schmertmann (1970) recommended standards for screw plate testing based on his research experience from the University of Florida during the late 1960s. These recommendations included installation methods, the use of a seating load, a minimum vertical distance between the tests, and some equipment details.

Schmertmann (1970) noted that the plate should be installed using controlled advancement, such that the downward progression of the plate matches the pitch of the blades. After installation, a plate seating load of about 50 kPa helps remove any prestressing effects caused by installation of the plate. Subsequent tests in a profile should be separated by at least 3 plate diameters to avoid the influence zone of the previous test. The rods should have a locking connector allowing rotation in both directions, and the rods should be counter threaded to the screw plate pitch, allowing the equipment to be recovered if the locking connector fails (Schmertmann, 1970).

Dahlberg (1974) developed an FCT interpretation theory for overconsolidated sands loaded beyond the preconsolidation pressure ($\sigma_c'$), allowing the identification of $\sigma_c'$ and the evaluation of both normally consolidated and overconsolidated soil moduli. Dahlberg (1974) proposed to determine the $\sigma_c'$ from the FCT data as the intersection of two straight lines fitted to the 'recompression' and the 'virgin loading' portions of the curve. The net stress increase $q$, is divided into two load increments $q_1$ and $q_2$ at the $\sigma_c'$, as shown in Figure 1.

The stress increase $q_1$ is used to develop a vertical stress increase profile, area $A_1$ in Figure 2. The stress profile for the net stress increase $q_2$ is similarly calculated and is shown in Figure 2.
where the \( \varepsilon \) is the vertical strain caused by the stress increase \( \Delta q_2 \) in Figure 2. The deformation \( \delta_{NC} \) are the integral of the vertical strains, which are a function of the stress increase \( \Delta q_2 \) and the stiffness of the soil, thus the normally consolidated modulus (or modulus number) can be derived from (2.2).

Dahlberg (1974) conducted FCT tests in a sand deposit where 16 m of overburden had been removed. The soil was assumed to have been normally consolidated before the excavation. The \( \sigma' \) profile found using the FCT was less than calculated values; Dahlberg (1974) attributes this to 'stress relaxation' following excavation.

3 ELASTICITY BASED INTERPRETATION THEORY AND FCT USE IN CLAY SOILS

Selvadurai and Nicholas (1979) developed an interpretation theory for the FCT based on elasticity. Eight cases reflecting approximations to FCT conditions were considered, and either exact or approximate solutions presented. These analytical models provide bounding solutions for Young's modulus, \( E \). The elasticity problem was based on a circular plate embedded in an elastic medium, with variations in the elastic medium, the stiffness of the plate and contact in the soil-plate interface. The analytical models considered for the elastic analysis are described in Table 1 and Figure 3. The solutions of these analytical models yielded expressions for Young's modulus \( E \) of the form

\[
\frac{\sigma}{r \cdot q_0} - \frac{\lambda}{E} = \delta
\]  

(3.1)

where \( \delta \) is the center deflection of a plate with radius \( r \) subjected to the uniform stress \( q_0 \). The factor \( \lambda \) is a function of Poisson's ratio \( v \). For undrained loading (\( v = 0.5 \)), Selvadurai and Nicholas (1979) considered model (d) in Figure 3 to yield a representative upper limit with \( \lambda = 0.75 \), and model (b) the lower limit with \( \lambda = 0.525 \). As model (b) is based on the stringent assumption of zero soil stiffness above the plate, Selvadurai and Nicholas (1979) recommend 0.6 \( \leq \lambda \leq 0.75 \) as an appropriate range of values for undrained loading. Tests in soft clay were interpreted using this theory, and the estimates of \( E_o \) were in good agreement with values obtained from other tests (Selvadurai, et al., 1980).

Selvadurai, et al. (1980) estimated undrained shear strength using a bearing capacity formulation:

\[
u_u = \frac{q_f}{N}
\]  

(3.2)
where \( N \) is a factor ranging in value from 9.00 to 11.35 and \( q_w \) is the failure load for the plate. The failure load was determined from a semi-log plot of stress versus deformation expressed as a percentage of plate diameter (normalized deformation). The failure load is identified as either the peak load or the point were the curvature changes abruptly.

**Table 1. Analytical models in Figure 3 (Selvadurai and Nicholas, 1979).**

- a. Uniform flexible circular lead
- b. Fully bonded rigid circular plate
- c. Smoothly embedded partially separated rigid circular plate
- d. Bonded partially separated rigid circular plate
- e. Bonded thin flexible plate
- f. Rigid spherical inclusion in bonded contact
- g. Deep borehole uniform flexible circular load
- h. Deep borehole rigid circular plate

![Figure 3. Analytical models for the screw plate test, Selvadurai and Nicholas (1979).](image)

The tests could be controlled as either constant rate of deformation or constant load. The constant rate of deformation tests allowed estimates of the undrained Young's modulus \( E_u \) and the undrained shear strength \( \phi_u \), and the constant load tests gave estimates of the drained Young's modulus \( E_d \) and the coefficient of consolidation, \( c_v \).

Kay and Parry (1982) used (3.1) and (3.2) for the determination of \( K_0 \) and \( q_w \). The failure load in (3.2) was defined as the ultimate load adjusted for in situ pressure, \( q_t = q_u + q_{rep} \). The ultimate load \( q_{ult} \) is the asymptote to a log(\( r \)) - normalized deformation plot (same plot as used by Selvadurai, et al., 1980). If the ultimate load \( q_{ult} \) was not reached during the test, Kay and Parry (1982) predicted \( q_{ult} \) by extrapolation using an exponential fit to the data:

\[
\sigma = q_w - ce^{-5}
\]  

where the constants \( \alpha \) and ultimate load \( q_w \) are determined using two (\( \alpha, 5 \)) test data points.

Kay and Avallie (1982) felt that the radial drainage model for consolidation proposed by Enslin (1970) was not physically appropriate, and that
interpretations using three dimensional isotropic consolidation would be better. Blot consolidation theory and a diffusion solution matched well with field data, and Kay and Avallone (1982) proposed an interpretation model based on these theories.

An estimate of the settlement at 70\% consolidation ($\delta_{70}$) is obtained from the time rate of deformation data using a graphical construction on an $\delta$-\(\bar{t}\) plot, cf. Figure 5. The $\delta$-\(\bar{t}\) curve is extrapolated using an $s$-shaped curve to estimate the initial deformation $\delta_0$. A tangent from $\delta_0$ defines a reference slope, and a second line with a slope ratio of 1:1.28 to the reference defines $\delta_{10}$ and $\delta_{20}$. With plate diameter $B$ and time factor $T_{10} = 1.24$, the consolidation coefficient is (Kay and Avallone, 1982):

$$c_v = \frac{1.24 (B)^2}{T_{10}}$$

(3.4)

Kay and Avallone (1982) calculated the drained Young's modulus using (3.1) with the deformation at 100\% consolidation ($\delta_{100}$) and the coefficient $\lambda$ calculated for an appropriate value of Poisson's ratio. The deflection $\delta_{100}$ was obtained by a linear extrapolation from the $\delta_{10}$ shown in Figure 5.

Kay, et al. (1983) conducted tests in sand with the Australian equipment. The tests were conducted in a predrilled hole stabilized using drilling mud. The plate had a diameter of 100 mm. The tests were conducted using constant rate of deformation, but with the low deformation rate the leading was considered as drained. The drained modulus was calculated directly from the $\alpha$ - $\delta$ data, and a subsequent settlement analysis using these moduli overpredicted measured settlements by 18%.

Bodare (1983) investigated the dynamic response of a screw plate system to evaluate theoretical solutions for the dynamic shear stiffness ($G$) of soil. The theoretical models considered were a flexible plate, a rigid plate and a rigid sphere fully embedded in a medium subjected to a dynamic vertical load. Bodare (1983) found theoretically that the dynamic response of a screw plate system was the same for either sinusoidal or random excitation for the same kind of screw plate, but that smaller plates gave higher values of $G$. The initial shear modulus $G_0$ calculated at small excitation levels was dependent on the choice of the theoretical model and the Poisson's ratio. The theoretical solutions were compared to experimental results, and good agreement was obtained for small frequencies but not at higher frequencies. Installation disturbance was problematic, but if the plate was allowed to remain in place for several days before the testing began, then the effect of the installation disturbance was apparently reduced (Bodare, 1983).

The screw plate test was evaluated in soft Bangkok clay in a series of field and laboratory tests in the latter part of the 1980's. Bergado and Huan (1987) compared screw plate test results to the pressurometer and vane shear, and to laboratory tests including unconfined compression, UU triaxial and oedometer tests. An extension of this calibration work was completed by Bergado, et al. (1990) using stress path controlled triaxial tests. Malaysian testing procedures and equipment design were used. The plate had a diameter of 25.4 cm and 3 cm pitch. The results of these comparisons are given in Table 2.

| Table 2. Comparison of interpreted parameters reported by Bergado and Huan (1987) |
|-----------------------------------|----------|----------|
| Screw plate (sp) | $E_s/E_0$ | $E_s/E_0$ |
| Pressurometer | 0.75 | --- |
| Vane shear | 0.78 | 0.85 |
| Unconfined compression | 1.18 | --- |
| UU Triaxial | 1.1 | 1.15 |
| No clear relationship between $c_v$ from oedometer and $c_v$ from screw plate |

4 PLATE FLEXIBILITY AND INSTALLATION DISTURBANCE

Brown (1992) studied the effect of plate flexibility on the determination of the undrained modulus $E_u$. The screw plate had two half hexx flights, which for the analysis was modeled as a flat plate with two opposing gaps, cf. Figure 6. The response of the model embedded in an elastic medium is compared to that of a whole plate under identical conditions.
Brown's (1992) solution was presented as a dimensionless plot for a given Poisson's ratio, reproduced here as Figure 7. This figure may be used to develop interpretation curves for specific plates by the substitution of the appropriate plate geometry. The results presented are for \( v = 0.3 \) in the soil, but Brown (1992) indicates that other \( v \) values may be accommodated by recognizing that the center deflections are proportional to \( v \) by

\[
\frac{(3 - 4v)(1 + v)}{(1 - v)}
\]

The assumptions of a rigid disc or a whole flexible disc may be unsatisfactory, as these assumptions cause an underestimation of the soil modulus (Brown, 1992). This is evident in Figure 8, which contains calibration curves calculated from Figure 7 for a specific geometry.

Brown and Reyno (1996) studied the effect of controlling the plate installation rate on the modulus estimates. A built-in flat plate provided reference values, and the effects of either controlled pitch screwing or self advancement of the plate were investigated. The tests were conducted with miniature plates in dry Sydney quartz sand.

The plates were 30 mm in diameter, the screw plate was 1.5 mm thick with a pitch of 4.2 mm. The screw plate was split into two half flights, similar to the plate sketched in Figure 6. The plates were tested at a depth of 150 mm. A center displacement of 0.01 mm was used to evaluate Young's modulus.

Brown and Reyno (1996) found that the measured moduli depended on soil density, plate shape and installation method. Higher densities gave higher moduli for all plate shapes. Self advancement of the screw plate yielded moduli higher than a built-in flat plate, ranging from nearly equal at the minimum density to approximately 300% larger at the maximum density. Controlling the rate of penetration reduced the moduli values, and at the maximum density the modulus was 55% larger than for the flat plate.

Brown and Reyno (1996) attribute these variations in moduli to dilation during shearing, as well as to the stress increase on the upper side of the
plate during insertion under self advancement. Brown and Reyno (1996) felt the shear strain could be reduced by altering the shape of the plate, and their proposed screw bar is shown in Figure 9. The dimensions are given for the miniature model used. Initial tests of the screw bar at a depth of 20 mm yielded moduli within 10% of flat plate values, suggesting that dilation effects have been reduced or largely eliminated (Brown and Reyno, 1996).

5 SUMMARY

The field compressometer test, originally developed in Norway, has undergone a significant amount of research and development both in Norway and abroad since its inception in 1953. The interest in the instrument has been cyclic, reaching high points in Norway in the early 1970s, abroad in the 1980s, and presently under review again in Norway.

The principle of the test has been consistent throughout the years: an at depth plate load test for measuring deformation moduli. The actual equipment and test methods have varied. Plate dimensions have ranged from miniature laboratory models to 30 cm diameter plates, and shapes from simple screws to single and double flight flat helical plates. Installation of the plates has been either by screwing from the surface or from the bottom of a predrilled borehole. The test procedures have varied as well, from the original incremental load control to constant rate of deformation and dynamic loading.

Tests have been successfully conducted in a wide variety of soil types, from dense homogeneous sand to soft clays. The interpretation methods give estimates of drained and undrained Young's modulus, the constrained modulus, shear modulus, the consolidation coefficient and the undrained shear strength. The theoretical background has included both elastic and plastic approaches.

6 REFERENCES


Figure 9. Screw bar, Brown and Reyno (1996).
The field compressometer test in Norway

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ABSTRACT: The field compressometer (or screw plate) test originated in Norway in the early 1950's. The Norwegian research and development of the instrument has occurred in two phases, the first from 1953 to 1970, and the second from 1970 to the present. This paper presents an overview the development of the field compressometer test in Norway from 1953 to the present.

1 INTRODUCTION

The field compressometer test (FCT), or screw plate test, was developed to measure vertical deformation moduli of soils in situ. The test method originated in Norway in 1953, and the two decades following these tests saw the development of the FCT equipment and interpretation theory as used in Norway.

Following this initial flurry of research, interest in the FCT waned in Norway, and from 1970 until the early 1990's little use of the FCT was made in Norway. During this period interest in the FCT grew abroad; the international research contributions from 1970 - 1996 are described in a companion paper to this conference.

In 1990 renewed interest in the FCT in Norway led to the instigation of a significant research effort at the Norwegian University of Science and Technology (the former Norwegian Institute of Technology, NTNU). This research is an ongoing fundamental revaluation of the test equipment, procedures and interpretation theory for the FCT.

This paper presents an overview of the development of the FCT in Norway, from inception in 1953 to the present day laboratory activities.

2 INITIAL DEVELOPMENT 1953-1970

The first tests using a screw shaped at depth bearing plate were conducted in 1953 for the foundation design of a storage tank on loose sand in Drammen, Norway. Measurements of compressibility were required, and since laboratory samples could not be taken a simple in situ test was devised. The field tests were conducted by the Norwegian Geotechnical Institute (NGI) and described by Kurnmeeje (1956).

Ordinary plate load tests were deemed inadequate since the large diameter of the tank (29.4 m) would induce stresses and subsequent deformations to much greater depths than were economically (or perhaps even technically) feasible to test using an ordinary plate load test. The tests were conducted using a screw shaped plate 25.2 cm in diameter, made of 3/8" steel plate welded to a 5/8" steel rod. Load was applied at the surface; the plate load was corrected for shaft friction and the measured deformations corrected for elastic rod deformation.

Initial settlement estimates were made from the load-deformation curves using an empirical method and elastic solutions. The settlement estimates ranged from 2.4 to 48 cm. An additional estimate using elasticity, an empirical correction for the effects of plate size, yielded an estimate of 16 cm which was comparable to subsequently measured deformations of 13-19 cm.

The experience from these tests indicated that an economical in situ measurement of compressibility in loose sand at large depths was possible, and for this purpose a screw plate test was promising. However, these tests also showed that a rational methodology for evaluating compression in soils from load-deformation tests was needed.

The first screw plate tests were thus influential in the development of Jansho's modulus concept for settlement prediction. In turn, the modulus concept was incorporated in the Norwegian FCT interpretation theory. The modulus concept is fully described by Jansho (1967); only the adaptation for the FCT is presented here.
The constrained modulus \( M \) is expressed as a function of stress using two material constants, the modulus number \( m \) and the stress exponent \( a \):

\[
M = m \sigma_a^{a-1} \quad (2.1)
\]

where \( \sigma_a \) is a reference stress (100 kPa), The stress dependency is described by \( a \), and relative stiffness by \( m \). For sand \( a = 0.5 \), giving \( M \) as a quadratic function of \( \sigma' \). Integrating over the vertical effective stress increase \( \Delta \sigma' \) gives the vertical strain:

\[
\frac{\Delta \sigma'}{\sigma_a} = \frac{\Delta e}{M} = \frac{\sigma_a}{M} \int_0^\Delta \sigma' \frac{d \sigma'}{d e} \quad (2.2)
\]

Substituting (2.1) into (2.2) and integrating over the influence depth gives the vertical settlement \( \delta \):

\[
\delta = \frac{\Delta e}{M} = \frac{\sigma_a}{M} \int_0^\Delta \sigma' \frac{d \sigma'}{d e} \int_0^e \frac{d \sigma'}{d e} \quad (2.3)
\]

The elements required for a settlement analysis are the material constants \( m \) and \( a \) and the vertical effective stress profiles, cf. Figure 1a. The difficulty for loading on finite areas lies in determining the \( \Delta \sigma' \) profiles. The Norwegian approach is to calculate the \( \Delta \sigma' \) profile using vertical equilibrium with the shear stresses \( \sigma_v \) mobilized along the walls of a cylindrical volume beneath the plate, cf. Figure 1b. The \( \sigma_v \) profile is evaluated by elasticity or plasticity. Assuming that \( \Delta \sigma' \) can be expressed as a function of the load intensity \( q \), on a circular area of diameter \( B \), a simplified expression appropriate for the FCT is derived from (2.3):

\[
m = \frac{B^2}{\pi} \frac{q}{\sigma_a} \quad (2.4)
\]

The settlement number \( S \) contains the stress dependency of the constrained modulus and the assumptions for the vertical effective stress profile. Values for \( S \) may be calculated and plotted for various soil types (values of \( a \), initial stress \( \sigma_a' \), and applied load \( q_a \). The modulus number \( m \) is then easily determined from the FCT data.

With this theoretical basis and the promising results from the tests in Drammen, research was initiated at the NTH to develop the FCT. A series of studies using various prototypes were conducted from 1967 to 1970 by Masters students at the NTH.

The first NTH tests were conducted in a medium quick clay by Haavardsholm (1967). The tests used a small screw tipped plate mounted in place of the vane in a standard vane shear device, as sketched in Figure 2. This device was pressed down to 50 cm over the test depth, and the plate then screwed down to the test depth in the undisturbed soil below. The time required per load increment was assumed to be 3-4 days, however the time-settlement curves drawn during testing indicated that 3-4 weeks would probably be required in this material (Haavardsholm, 1967). The tests were characterized as a failure, and Haavardsholm (1967) reported that no reasonable conclusions could be drawn from the results. These were the only tests conducted in clay at the NTH.

Haavard (1967) investigated the effect of plate size and conducted field tests in cohesiveless soil to evaluate settlement estimates from field test data. The version of the equipment for these tests was constructed to be screwed from the surface to the test depth. The load was applied at the surface via an inner rod, which was free to move within the outer rod. The bearing plates were plate steel formed into a screw shape and welded to flat cylindrical center piece. The plate was keyed to fit the outer rod to transfer torque during installation. The plate was broken off and abandoned after testing.

The laboratory tests seemed to indicate that larger plates produced stiffer response (higher \( m \) values). It was also noted that the \( m \) obtained depended on the stress increment \( q_a \), when theoretically \( m \) is a material constant. Haavard (1967) attributed this to contact problems in the plate/soil interface, and suggested that larger plates have less 'contact errors'.

In the field tests, Haavard (1967) noted that the plate did not advance at the same rate as the pitch of the blade. The interpreted \( m \) values varied widely. Settlements recorded from a circular test foundation were used to back calculate an \( m \), which Haavard (1967) found to be lower than the screw plate values.

Haavard (1967) felt that the problems encountered in the field and the variations in the modulus number could be due to friction in the rod system and the form of the screw plate. Haavard (1967) recommended...
These recommendations were incorporated in the construction of the first full version of the FCT during 1968. The instrument was a custom hydraulic jack mounted on a concentric two rod system. Hydraulic pressure was supplied via a hydraulic line run through the center of the inner rod. The outer rod (from the 54 mm piston sampler) provided the reaction for the jack and was anchored at the ground surface by an anchor frame. The inner rod (from the piezometer) was connected to the piston of the jack (thus the plate itself), and the movement of the top of the rod was measured at the ground surface. The screw plate was 16 cm in diameter with a pitch of 4.5 cm. The plate was attached to the jack by a weakened center screw, allowing it to be broken off and abandoned at the completion of a test profile.

The new version of the FCT equipment and the proposed interpretation theory was evaluated in the autumn of 1968. Berle (1968) reviewed the theory and prepared charts of the settlement number S. Fremstad (1968) performed laboratory tests, and together they performed field tests with the FCT. Settlement estimates from the FCT data were controlled using a 1 m² foundation and a 200 m² fill.

Berle (1968) prepared the S plots based on the assumptions of a quadratic root modulus (2.1 with $\alpha = 0.5$), homogeneous soil to the influence depth of the loaded plate, and distributions of $\Delta m$ based on elasticity and plasticity. Berle (1968) noted that the work curve obtained by the FCT is not in accord with the assumption of a constant $m$, but surmised that this may be explained by elastic deformations of the plate, rather than the contact problems proposed by Hanan (1967). Berle (1968) also presented some concerns with the interpretation theory: The screw plate may not be directly comparable to a flat plate, the installation of the plate may create a special stress field around the plate, the soil may not follow the model proposed by the modulus concept, and that the assumptions inherent in the elastic and plastic models used for the $\Delta m$ distribution may not be correct. Despite these concerns, this analytical model has endured and is still in use in Norway.

Fremstad (1968) measured the deformation of the plate, shown in Figure 3. The deformations varied greatly and were not particularly repeatable. Under the first load cycle it was always the deepest wing which deformed the most, and after several cycles the deformations evened out. The load is not evenly distributed over the plate, and the deformations are large enough to be of concern (Fremstad, 1968).

Fremstad (1968) and Berle (1988) concluded that the FCT equipment in its current form was acceptable, with the exception that a better system for maintaining constant oil pressure should be developed. Berle (1968) proposed that the regularity of the work curve may allow fewer load steps. Fremstad (1968) proposed a relative criteria for establishing the stop time for each load increment, where the next load increment is applied when the settlement rate is small compared to the total settlement incurred in the current load increment.

Determination of stress under the plate during a load increment was deemed problematic. Soil arching over the plate may reduce the net stress increase below the plate, since a portion of the applied load would then function as the in situ overburden stress. Fremstad (1968) recommended correcting the applied load for in situ vertical effective stress $\sigma_e = \sigma_1 - \sigma_3'$ of Figure 4.

The last work during this period at the NTH was by Enslid (1970), who extended the settlement number charts to include other soil types. Enslid (1970) prepared S charts for NC clays and OC soils ($\alpha = 1$ and $\alpha = 0$ in 2.1), but indicated that in most cases the 'smf' model with $\alpha = 0.5$ is preferred. The load increment $\Delta q$ is adjusted for in situ overburden (cf. Figure 4) for the modulus number calculation.
Ennlid (1970) also developed an interpretation theory for the consolidation coefficient. Ennlid (1970) evaluated axial and radial drainage models for a cylindrical volume of consolidating soil. The radial drainage model in Figure 5 was found to be more acceptable than the axial model. The radial consolidation coefficient $c_r$ is estimated as

$$c_r = 0.335 \frac{R^2}{t_{90}}$$  \hspace{1cm} (2.5)

where $R$ is the length of the drainage path (radius of the plate), and $t_{90}$ the time for 90 percent consolidation for the load increment.

The $t_{90}$ is obtained from a $\ln t - \ln \Delta t$ plot, or preferably, from a $\ln t - \Delta t$ plot. A graphical construction similar to Taylor's method is used, see Figure 6. A similar construction to Figure 6 is used for the $\ln t - \Delta t$ plot, however the initial slope is taken tangent to $\Delta t_{90}$ and the slope multiplier is 1.31. The two methods yield approximately the same $t_{90}$.

Ennlid (1970) performed field and laboratory tests to verify the interpretation theory. The $m$ and $c_r$ obtained from the FCT were in good agreement with $m$ and $c_r$ from oedometer tests. Settlement estimates using FCT data corresponded with those measured in a construction project, but time rate of settlement estimates were not as successful. Ennlid (1970) felt this could be attributable to uncertainty in the assumptions for the calculations, and not sufficient grounds to discard the theory.

This concluded activity with the field compressometer at the NTB for this period. The FCT equipment as described by Fremstad (1968) and the interpretation and testing procedures from Ennlid (1970) were used essentially unaltered for the next two decades in Norway.

**Figure 4. Determination of the modulus number.**

**Figure 5. Radial consolidation model, Ennlid (1970).**

**Figure 6. Graphical construction for $t_{90}$.**

Kvalsvik (1991) investigated the effect of plate diameter and embedment depth on the interpreted modulus values for flat bearing plates. Two plate diameters (16 and 30.7 cm), two embedment depths (1 and 2 m) and three soil densities were used. Hoff (1992) also conducted bearing plate tests, using 16 cm diameter plates at 1 m embedment for three densities. Kvalsvik (1991) reported that the small variation in embedment depths had no effect, and that the larger plates gave larger settlements. Modulus estimates were prepared using the Norwegian interpretation theory for the FCT. The modulus number estimates from Kvalsvik (1991) and Hoff (1992) are compared to those interpreted from oedometer tests in Figure 7. The bearing plate modulus are much lower than from the oedometer.
During 1991-1992 an alternative version of the field compressometer was developed, and initial proof testing performed by Hoff (1992). The basic concept of this prototype field compressometer was to apply the load at the surface which was then transferred to the plate through a single set of stiff drill rods. The load reaching the plate was measured at the plate by a load cell positioned between the drill rod and the plate. The instrument was installed and extracted using a rotary drill rig, and the plate was recovered at the end of the test.

Hoff (1992) conducted the first tests of the prototype FCT in dry sand in the large scale testing facility at NTNU. These tests were reported as successful, and Hoff (1992) determined modulus numbers similar to those obtained from flat plates; these are also plotted in Figure 7.

Subsequent testing under both laboratory and field conditions has revealed a curious flaw with the prototype instrument. These control tests were conducted using both the load cell at the plate level, and an additional load cell at the top of the drill rods where the load was applied. Load and deformation levels were significantly smaller than used by Hoff (1992), with maximum deformations under 5 mm (Hoff, 1992 reports 25-30 mm). Unload-reload cycles were also used in these tests.

The use of two load cells allows three forces to be measured: at plate level, at the top of the drill rods, and the friction along the drill rods (difference between the first two forces). These forces may be expressed as 'equivalent plate stresses' by dividing the force by the area of the plate. Figure 8 presents plots for each of these quantities versus measured vertical deformation for data from a typical test.

The first plot gives what could be described as an 'ordinary' FCT curve - the initial portion of the curve and the rebound loop exhibit stiffer response than the 'virgin' curve. The second plot, using friction as an equivalent plate load, shows that 0.3 to 0.4 mm of deformation is required to fully mobilize the available friction. The third plot, based on the stress at the plate level, gives a curve that is recognizable as an FCT data set.

![Figure 7. Modulus comparisons, data as reported by Kvalsvik (1991) and Hoff (1992).](image)

![Figure 8. Test data from prototype FCT.](image)

The effect of friction along the drill rods is significant. The center plot in Figure 8 shows that the stress transfer to the plate for lower load levels is a function of deformation. Maintaining constant load at the level of the plate was found to be extremely difficult; the load reaching the plate at any time was a function of the mobilized friction and the deformations occurring in the soil surrounding the drill rods. A complex equilibrium is established, where the load transfer mechanism depends on friction mobilization, the strength and deformability of the soil around the drill rods, and the stress-deformation response of the soil beneath the plate.

The first curve in Figure 8 (using the load measured at the top of the drill rods) is the only curve resembling FCT data. It is in fact a load test - not of the soil under the plate but rather of the entire soil system in contact with the plate and drill rods. The response of the soil beneath the plate is not
separable from the response of the whole soil system. Based on these control tests, the concept was viewed as a failure and the instrument abandoned in 1994 (Strout, 1994).

Although the prototype FCT was abandoned, the drill rod system constructed for the device was successful. New drill rods were constructed for the prototype FCT to allow the instrument to be installed and extracted using a rotary drill rig. This required rod connections which could withstand compression, tension and torsion in both directions. A connection detail (Figure 9) using heavy steel rods was constructed. The ordinary FCT has been adapted to use these rods, and the use of a drill rig has proven to be a satisfactory improvement in the test procedure.

Figure 9. Drill rod connection details (cut away).

An alternative reference level was also developed in connection with the control tests of the prototype FCT. The reference level is a constant tension wire, see Figure 10. The free hanging weight maintains constant tension (and constant height) despite possible thermal expansions. The wire can be located at any convenient height above the work area to avoid interference with equipment or personnel. This reference system was satisfactory during field testing, and no apparent disturbance due to wind or temperature effects was noted in the measurements.

Figure 10. Tension wire reference level.

4 SUMMARY

This paper presents the history of the field compressionometer test development in Norway. The FCT, originally developed in Norway, has undergone a significant amount of research and development both in Norway and abroad since its inception in 1953. The interest in the instrument has been cyclic, reaching high points in Norway around 1970, abroad in the 1980s, and presently in focus again at the Norwegian University of Science and Technology in Norway.

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Push-in total stress cells for horizontal pressure measurement in clay

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ABSTRACT: The paper discusses the use of push-in total stress cells for the evaluation of in situ lateral stresses in the ground. Details of the stress cells and modifications made are given. Temperature and pressure calibrations are reported and the importance of these corrections are discussed. The installation and data recorded from several push-in total stress cells at clay sites in the Lower Mainland of British Columbia are presented and the results are compared with other independent measurements of in situ horizontal stress. Laboratory measurements are also compared with the field data.

1 DETAILS OF TOTAL STRESS CELL

The spade-shaped push-in total pressure cells (TSC) used for this study were purchased from Solinst Canada Ltd. The spade cell is a plate 6-4 mm thick with a pressure sensitive area of dimensions 100 mm by 200 mm. The rectangular oil-filled chamber is formed of two thin steel sheets welded at the edges. The pressure sensitive area is welded to a support plate. The cavity so formed is pressurized to maintain plate separation. The welded plates are strengthened by a solid metal strip which is welded on to the cell perimeter. The oil pressure in the chamber is connected via a short length of steel tube to a pneumatic transducer located on a connector boss behind the support plate (Fig. 1). A ceramic porous disc is also located on the support plate and connected hydraulically to a second pneumatic transducer which is tandem-mounted behind the first. Both transducers are protected within a steel sleeve adaptor which connects the spade cell to the installation rod.

A pre-set baseline (zero reading) and calibration is supplied for each cell by the manufacturer. The zero reading corresponds to the oil pressure in the chamber formed by the two steel plates. The manufacturer recommends an initial storage life to check that no baseline changes occur.

Twin nylon tubes, sheathed in polyethylene, are attached to the compression fittings located on each of the pneumatic transducers. Quick release couplings are attached to one of the nylon tubes at the other end of the twin tubing. The quick release couplings are used to connect the down pressure line to the pressure readout box. The twin tubing lines are usually cut at lengths determined by the depth at which the spade cell is to be installed in the ground.

The cell and pore pressure measurements are taken using a portable pneumatic readout box. The readout unit contains a compressed nitrogen pressure bottle which is used to obtain field measurements. With the quick release coupling connected to the readout box, the pressure valve on the box is opened and a gradually increasing pressure is applied to the spade cell pressure transducer. When the applied pressure just exceeds the pressure in the cell, the diaphragm in the transducer deflects and vents the applied pressure to the return line (the second nylon tube). The readout box then measures the gas pressure required to just maintain a continuous flow through the diaphragm chamber. The same technique is used for reading both the oil chamber pressure (which corresponds to the total lateral stress acting on the spade cell) and the pore water pressure transducers. The pressures are measured at the surface by a Dureck electronic...
transducer with a 0 to 2000 kPa (kN/m²) range. Resolution of the transducer is ±0.05% full scale, i.e. ±1 kPa.

Prior to installation in the field, minor modifications were made to the cells and calibration checks were performed. Because the pressure cells are oil-filled and sealed, the differing temperature characteristics of the cell components will cause the baseline to be sensitive to variations in temperature. This is recognized by the manufacturer but no data have been presented to evaluate the effects. Furthermore, for none of the cases reported in the literature, are the pressure cell data corrected for temperature effects. To provide data on the in-ground ambient temperature and its variation during the period when the cells are installed, platinum RTD temperature sensors were installed in several of the cells. The RTD sensors were installed adjacent to the internal pressure cell body (Fig. 1). The electrical cables from the sensor were taken up to the ground surface through the return pressure line attached to the pressure cell transducer. The presence of the two thin wires did not restrict the venting action required for diaphragm movement during readout.

The stabilized temperature for each set of pressure calibrations was measured by the RTD sensor attached to the face of the blade. A temperature range of 0° to 20° C was used for both cooling and warming temperature cycles. To ensure that stable temperature variations were achieved, a set of pressure and temperature readings took between 12 and 24 hours to complete. Typically, a series of cell and porewater pressures were taken at nominal chamber pressures of 0, 50, 100, 150 and 200 kPa for both loading and unloading cycles for both warming and cooling temperature cycles.

The results of the pressure and temperature calibration for one of the total stress cells (blade cells) are presented in Fig. 2. From the results of the calibration it is evident that:

- the total stress cells have an internal pressure at zero applied confining stress which must be subtracted from the actual reading to give the stress increase resulting from the increase in external pressure,
- an offset in the internal cell pressure occurs (baseline drift) as the temperature of the blade changes. This concurs with results presented by Felix and Bauer (1986) for other types of pressure cell,
- the baseline drift resulting from the temperature change is essentially independent of the external applied pressure and can be related linearly to the temperature change. This facilitates easy correction of field measurements since the temperature adjustment does not vary with the in-ground stress acting on the blade.

The temperature drift for all the blades initially used is shown in Fig. 3 for the condition of zero applied chamber pressure. The temperature coefficient, B, for the cells is listed in Table 1. Temperature coefficients of up to 1.5 kPa/°C were measured although average values are around 0.5 kPa/°C. Since temperature changes of 10° C or more may occur between the laboratory and field
Fig. 2 TSC temperature and pressure calibration

Fig. 3 Temperature dependence for all TSC blades

The temperature corrections become appreciable, especially where low stresses are being measured.

Calibration measurements were performed on the pressure cells before field installation and again after the cells had been recovered from the ground; the latter calibration was used for data interpretation.

The baseline pressure (at zero chamber or confining pressure) for the individual total stress cells given in Table 1 is governed by the arbitrary choice of the reference temperature. For this study, all temperature corrections to the in situ blade pressures were made with respect to the equilibrium ground temperature, as measured by the RFD sensor installed on the cell. At any one site, a sufficient number of cells were instrumented so that a representative temperature profile could be obtained. At depths where blades without temperature sensors were installed, the ground temperature was estimated by interpolation from the other temperature measurements. Thus, the measured blade pressures from in situ measurements can be corrected according to:

$$\sigma_{TSC} = \sigma_m - \sigma_r \left[ (T_r - T_i) B_r \right]$$  \hspace{1cm} (1)

where:

- $\sigma_{TSC}$ = temperature corrected net total blade pressure (kPa),
- $\sigma_m$ = measured total blade pressure (kPa),
- $\sigma_r$ = baseline total pressure at reference temperature (kPa),
- $T_r$ = reference temperature (°C),
- $T_i$ = in-ground temperature (°C),
- $B_r$ = cell pressure temperature calibration factor (kPa/°C).

Similar baseline readings were also determined for the pneumatic pore pressure transducers; these transducers were not found to be temperature sensitive.

3 TSC INSTALLATION PROCEDURE

Total stress cells of the push-in type are normally installed in the base of an existing borehole (Todd and Charles 1981, 1983); this reduces the risk of

<table>
<thead>
<tr>
<th>Spade Cell No.</th>
<th>Reference Temperature $T_r$ (°C)</th>
<th>Baseline Pressure $\sigma_r$ (kPa)</th>
<th>Factor, $B_r$ (kPa/°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TSC1350</td>
<td>22</td>
<td>130</td>
<td>+0.45</td>
</tr>
<tr>
<td>TSC1537</td>
<td>9.5</td>
<td>156</td>
<td>+1.25</td>
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<td></td>
<td>(9.5)</td>
<td>(147)</td>
<td>(+0.67)</td>
</tr>
<tr>
<td>TSC1538</td>
<td>8.0</td>
<td>179</td>
<td>+0.58</td>
</tr>
<tr>
<td></td>
<td>(9.5)</td>
<td>(164)</td>
<td>(+0.47)</td>
</tr>
<tr>
<td>TSC1539</td>
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<td>138</td>
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</tr>
<tr>
<td>TSC1540</td>
<td>9.3</td>
<td>146</td>
<td>+0.14</td>
</tr>
<tr>
<td>TSC1541</td>
<td>10.5</td>
<td>135</td>
<td>+0.48</td>
</tr>
<tr>
<td></td>
<td>(9.5)</td>
<td>(136)</td>
<td>(+0.25)</td>
</tr>
<tr>
<td>TSC1542</td>
<td>9.1</td>
<td>139</td>
<td>+0.91</td>
</tr>
</tbody>
</table>
damaging the cell. Due to the high costs involved in the boring operation and the availability of an alternative technique, minor modifications were made to the spade cells to facilitate installation using the University of British Columbia (UBC) Geotechnical Research Vehicle. This involved the machining of a steel sleeve adapter to connect the spade cell to the installation rods. The adapter also serves as a protective housing for the pneumatic transducers. One end of the adapter is screwed on to the cell connector boss (Fig. 1) while the other accepts the AWL casing (4.7 mm OD, 4.5 mm wall thickness) that was used to push the spade cell into the ground. High buckling strength rods were required to avoid rod damage due to the large loads necessary to push the cell assembly, most of the resistance resulting from the larger diameter sleeve adapter. Unlike the borehole situation, where the TSC is only advanced 0.5 m to 1.0 m below the base, the use of the UBC vehicle (normally used for penetrating CPT equipment) required the cells to be pushed from ground level to their final depth. To avoid buckling and breakage of the rods, it was decided to use AWL casing for installation. The 3.5 mm OD of the rod permits easy passage of the two lines of twin tubing from the cell to the surface.

The TSC blade itself is most susceptible to breakage, under axial loading, at the weld where the two plates connect to the support plate. To reduce the axial loads on the pressure cell during installation, a dummy plate was pre-pushed to a final depth approximately 1.0 m above the planned depth for the TSC. In this way, the TSC was only pushed in virgin soil for a depth of about 1.0 m. In order to obtain more information on soil variations at the instrumented location, the dummy push was performed using the standard dilatometer (DMT) and data taken every 0.2 m (thrust, P, P2, P5).

After installation of the total pressure cell, the lateral stress and pore pressure were monitored with time until a stable equilibrium value was reached.

4 OVERVIEW OF TSC EXPERIENCE

The concept of the push-in spade-like total pressure cell to measure in situ horizontal stress was first utilized by Massarsch (1975) in a soft clay. The Gloetzl cell used was 4 mm thick and was pushed into the ground protected within a steel casing. The casing frame was withdrawn about 0.3 m above the intended depth and the cell alone advanced and left in the ground until a stable stress equilibrium was reached. The maximum membrane deflection of the Gloetzl cell is about 5 μm (negligible in soft soils). Use of this full-displacement method gave consistent Ks values for the normally consolidated deposit tested. Satisfactory results with push-in total stress cells have also been reported by Massarsch et al. (1975), Tavenas et al. (1975), Massarsch and Brnns (1976) and Massarsch (1979). Reported multiple measurements at one depth were within 1 kPa (Tavenas et al. 1975).

During installation of the TSC, the soil is displaced and excess pore pressures are generated which then subsequently decay with time. Once these excess pore pressures have dissipated, the lateral stress acting on the blade should still be higher than the pre-installation value. However, if the viscous characteristics of the soil permit, the lateral stress increment induced by TSC installation may also dissipate so that no additional stress (over and above the original Ks stress) remains. Under differing conditions, the stress increment will remain and the measured lateral stress acting on the TSC will require some correction in order to obtain an estimate of the pre-installation horizontal stress.

\[
\sigma_{TSC} = \sigma_{TSC0} + \Delta \sigma(t) + u_0
\]

\[
\Delta \sigma(t) = u(t) + \Delta \sigma(t)
\]

\[
u(t) = u_0 + \Delta u(t)
\]

\[
\Delta u(t) \to 0 \text{ and } u(t) \to u_0
\]

where:

- \(\sigma_{TSC0}\) is the in situ pre-penetration horizontal stress,
- \(u(t)\) is the time dependent pore pressure measured by the TSC,
- \(\Delta \sigma(t)\) is the time dependent stress increment induced during installation of the TSC,
- \(u_0\) is the equilibrium in situ pore water pressure,
- \(\Delta u(t)\) is the excess pore pressure induced during TSC installation.

In soft clays, it is generally accepted that no correction to the final equilibrium measured blade pressure is required since stress relaxation is assumed to occur, i.e.

\[
\Delta \sigma(t) \to 0 \text{ for } t > 1 \text{ to } 2 \text{ months}
\]

In fact, \(\Delta \sigma(t)\) may seldom be zero, but may be small enough so as to not to cause significant deviations from the expected Ks value.
Results obtained with push-in pressure cells in stiff overconsolidated soils by Tedd and Charles (1981, 1983), however, indicate that the TSC overreads by an amount approximately equal to one half the undrained shear strength, $S_u$ (as determined from unconsolidated undrained triaxial compression tests). The reference lateral stress used to evaluate the amount of overread was taken as that obtained from self-boring pressure meter tests. While Tedd and Charles (1983) argue that it should be possible to relate the magnitude of the overread to the soil modulus, they suggest that, due to the impracticality of deciding upon a relevant modulus value, it is more realistic to empirically correlate the overread to $S_u$. Data presented by Powell et al. (1983) for a stiff glacial till confirm the magnitude of the correction suggested by Tedd and Charles (1981).

For soft soils with $S_u < 30$ kPa, no correction is recommended, but for $S_u > 30$ kPa, the suggested in situ total horizontal stress is given by:

$$\sigma_{ho} = \sigma_{TSC} + 0.5 S_u$$

(7)

A review of all published data where stress history (OCR) and $K_s$ (from TSC) are available, leads to the following correlation (Fig. 4):

$$K_s^{TSC} = 0.581 (OCR)^{0.432}$$

(8)

Equation (8) suggests an average drained friction angle of 25° for the reported data.

The data in Fig. 4 are from both Glenside and Solfen type pressure cells, corrected according to the recommendation of Tedd and Charles (1981), developed initially for the Solfen cells. The data would suggest that the corrected lateral stress from TSC data represents fairly well the actual in situ lateral conditions, as referenced by the self-boring pressure meter.

5 INTERPRETATION OF TSC DATA

The TSC instruments described above were installed at two UBC clay research sites, namely Lr. 232 St. and Strong Pit. Both clays are overconsolidated, at Strong Pit the overconsolidation arises from unloading resulting from quarrying activities, while at Lr. 232 St. the stress history is more uncertain, being a combination of unloading due to construction activities and drying/wetting cycles due to ground water level fluctuations.

Typical field data obtained at Strong Pit are presented in Fig. 5 where it is apparent that for this particular deposit, monitoring periods in excess of 60 days are necessary before the stress measurements stabilize. At Lr. 232 St. similar periods of time were also found to be necessary. At Strong Pit, the equilibrium pore pressure throughout the clay profile is approximately zero. The results in Fig. 5 indicate this condition since the total and effective horizontal stresses become equal as the excess pore water pressure dissipates to the in situ equilibrium value. The corrected net blade pressure is also presented on Fig. 5 - this is the measured blade pressure corrected for overread according to the Tedd & Charles (1983) method.

At the Strong Pit site, the undrained shear strengths of the clay silt are in the range 100 kPa to 175 kPa. Hence at the surface, the correction for overread may be as much as one half of the initial measured blade pressure value. At Lr. 232 St. the undrained strengths are much lower (20 kPa to 40 kPa), but so too are the measured field stresses (100 kPa to 300 kPa).

Hence two major problems are considered to exist in relation to the use of the push-in TSC for engineering measurements:

(i) the long delay required for the pressure cell to come to equilibrium in the ground after insertion, and

(ii) the large correction required to the
measured horizontal pressures and particularly the uncertainty or error involved in the adjustment.

5.1 Dissipation Modelling of Stress Change

A typical result from the TSC’s installed at Strong Pit is shown in Fig. 5. The effective horizontal stress is obtained by subtracting the pore water pressure measured on the blade ($\sigma_{w}$) from the temperature corrected net blade pressure ($\sigma_{n}$). After the long dissipation period, the pore pressure at this site is zero and $\sigma_{n}$ is equal to $\sigma_{n}^{*}$.

The dissipation of the total stress data has been evaluated using a power function relationship of the form:

$$\sigma_{n}^{*}(t) = \alpha_{n} t^\beta$$

(9)

where $\sigma_{n}^{*}(t)$ is the time dependent stress measured with the TSC, $\alpha_{n}$ is the value of $\sigma_{n}^{*}$ at $t=1$, $t$ is the time after installation and $\beta$ is the exponent which controls the rate of stress relaxation.

Data from both Strong Pit and Lr. 232 St. were evaluated using Eq. (9) and corresponding $\alpha_{n}$ and $\beta$ values were obtained. The variation in both $\alpha_{n}$ and $\beta$ is presented in Fig. 6 as a function of depth - the relationship is remarkably linear. It is also interesting to note that:

- $\alpha_{n}$ is greater for the stiff clay at Strong Pit than for the soft to firm clay at Lr. 232 St. This is intuitively correct since larger pressures will develop due to full-displacement penetration in stiffer soils;
- $\beta$ for Strong Pit is more largely negative than for Lr. 232 St. which implies a more rapid post-installation reduction in $\sigma_{n}^{*}$ for stiff clay than and may also indicate a larger degree of disturbance.

The rate of change of the measured total stress incorporates both the pore pressure dissipation and the soil relaxation. In stiff soil, the effect of stress relaxation may be such that the final corrected lateral stress may be very close or equal to the in situ value. This is not likely to be the case for stiff soils since some amplification of the equilibrium horizontal stress will certainly remain.

5.2 Validation of Overread Correction

Since the stress history at this site is reasonably well documented and is confirmed by laboratory test results, it was decided to try and back figure the overread correction based on laboratory derived $K_{c}$ - OCR relationships, obtained from Lateral Stress Oedometer tests performed on undisturbed samples recovered at the site. In situ vane shear tests were also performed and the data used to verify the stress history profile (Sully and Canpanella 1989). Using the OCR profile, an estimate of the in situ $K_{c}$ was made using a normally consolidated value of 0.54 and exponent of 0.38 (average from 4 tests). This $K_{c}$ value was used to calculate the in situ horizontal effective stress. The magnitude of the overread associated with the push-in total stress cell was then calculated as the difference between the net corrected measured total blade pressure and the calculated in situ horizontal total stress obtained from the $K_{c}$ - OCR relationship. The overread of the lateral stress was found to be approximately 75 kPa which corresponds on average to one half of the undrained shear strength of the soil as determined from the in situ vane test. This is in agreement with the Tedd & Charles correction method discussed earlier.

6 CONCLUDING COMMENTS

The use of push-in total stress cells for measuring in situ horizontal stresses was implemented with varying degrees of success at a total of four sites in the Lower Mainland of British Columbia. At Strong Pit where the glaciomarine clay contains occasional cobbles and boulders, three of the nine blades installed were damaged. The blades usually break at the end of the stress sensitive section as this is the weakest point. Blade breakage is an important consideration as the TSC equipment is expensive. In the soft to firm clay at Lr. 232 St. no problems were encountered and all blades were installed, worked
This was reflected in many of the pore pressure measurements which varied considerably throughout any one profile. Similarly, the final dissipated pore pressures do not generally agree with expected values based on knowledge of the equilibrium pore pressure at the sites.

The spade cells were calibrated in the laboratory prior to installation and again after recovery from the ground. Small variations in the baseline readings occurred between the two calibrations. The latter calibration was used for data interpretation. In the case of the Strong Pit cells, after recovery it was found that the temperature calibration factors had changed. The calibration changes for three of the cells are shown in Table 2. The change is thought to arise as a result of wear on the blade during installation in the stiff clay. At Lr. 232 St. no changes were noted on recovery of the cells.

Table 2 Calibration changes for spade cells before and after installation at Strong Pit

<table>
<thead>
<tr>
<th>Spade Cell No.</th>
<th>Base Pressure $\alpha_0$ (kPa)</th>
<th>Temp. Factor $B_\gamma$ (kPa°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before</td>
<td>After</td>
<td></td>
</tr>
<tr>
<td>Installation</td>
<td>Installation</td>
<td></td>
</tr>
<tr>
<td>TSC1537</td>
<td>156</td>
<td>1.35</td>
</tr>
<tr>
<td>TSC1538</td>
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<td>0.58</td>
</tr>
<tr>
<td>TSC1541</td>
<td>133</td>
<td>0.48</td>
</tr>
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<td>164</td>
</tr>
<tr>
<td></td>
<td>TSC1541</td>
<td>136</td>
</tr>
</tbody>
</table>

7 REFERENCES


Dilatometer tests in a Leda clay crust

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ABSTRACT: The dilatometer test results have recently been applied in foundation design for prediction of settlement and bearing capacity problems. The equipment, its calibration, test procedures and test data interpretation are simple. These advantages seem to explain the increasing use of the dilatometer (DMT) test as a routine technique for subsoil investigation. The DMT test boreholes were carried out on the grounds of the National Research Council (NRC) in Ottawa. Several test results based on laboratory, and other in-situ tests available in the literature for the Leda clay deposit in Ottawa area were used to provide correlations between geotechnical properties and soil index parameters as proposed by Marchetti (1980). More appropriate relationships, even though preliminaries, are presented for the crust layer.

INTRODUCTION

The development of micro-electronics, microprocessors, and of digital computers have propelled prominent advances in the devices and techniques for physical measurements in engineering. As a result, relevant soil data for design and analysis of foundation problems can presently be assessed from laboratory, in-situ testing, and from performance of instrumented structures. The range of applicability of in-situ tests has been broadened such that the suitability to test discontinuous soil deposit, such as fissured and stiff clays, has justified the use of the in-situ test techniques.

The present paper deals with the flat, plate dilatometer (DMT), which has been developed by Marchetti (1975). Due to the fact that, the dilatometer equipment, testing procedures are very simple and; repeatability and reproducibility of the test are very high, this technique has gained a wide acceptance and use in routine soil exploration programmes. Several empirical correlations between DMT test data and soil engineering properties have been proposed by Marchetti (1980) and, Marchetti and Crapps (1981). Since then, a great deal of improvements of those relationships and/or new proposals have been presented (Lunne et al., 1989; Riaud and Miran, 1992; and others). More recently, the DMT has proven that is not only an efficient investigation tool but also, a valuable alternative in the prediction of the axial and lateral load capacity of piles, settlement of shallow foundations and, control of compaction of cutfills soils (Riaud and Miran, 1992).

In this view, it is found appropriate to present DMT test data more applicable to foundation problems dealing with stiff, fissured clays.

TEST SITE CONDITIONS

The subsoil in the tested area is originated in the marine environment of the Champlain Sea and is also known as Leda clay deposit. The superficial layer of this deposit is, in general, a desiccated and fissured crust. It is of great interest to investigate its in-situ engineering properties because it may extend down to a depth of 6 meters from the ground surface (Eden and Crawford, 1957).

A total of five Standard Penetration Tests (SPT) were performed in a nearby test area of 12 by 23 m near the building M20 of the Institute for Research in Construction on the grounds of the National Research Council (NRC) of Canada in Ottawa. Also, disturbed and undisturbed soil samples were retrieved for laboratory tests. The in-situ test program comprised the utilization of four different techniques, whose test data results have been presented by Bauer and Tanaka (1988).

The geotechnical subsoil profile based on these test results is shown in Figure 1. The top layer
FIGURE 1 - GEOTECHNICAL SOIL PROFILE

consists of a medium to stiff, brown, silt clay and is about 2 m thick. Below this, is found a fissured and stiff, brown, silt clay layer of thickness of 1 m. Between the depths of 3 and 4 m below the ground surface, the silt clay gradually changes from firm to a soft, grey, clay. Thereafter down, is found a soft, grey, and sensitive clay.

The liquid and plastic limits of consistency are nearly constant from the ground surface down to a depth of 3.5 m; and averaged to 55 and 30 percent, respectively. The natural water content is close to the plastic limit near the ground surface and is approximately equal to the liquid limit at a depth of 3.5 m. Thereafter down, the natural water content becomes greater than the liquid limit by about 40 percent. And finally, it is found that the overconsolidation stress ratio (OCR), and the preconsolidation pressure present a rapid increase from the depth of 4 m towards the ground surface. At that depth, the preconsolidation stress is about 300 kPa, and the OCR of about 7.

THE DMT TEST AND INTERPRETATION

The DMT device consists of a flat plate of thickness, width and length, respectively, equal to 14, 95 and 220 mm. An expandable steel membrane of 60 mm in diameter is located in one face of the blade. The plate is pushed down into the ground at a rate of 10 to 20 mm/s. At every 100 to 200 mm depth interval the penetration is stopped and three pressure readings are taken: i) the first one, is related to an internal pressure necessary to just lift off the membrane from the sensing disc; ii) the second one, is the pressure required to push the center of the membrane by one millimeter into the surrounding soil; and iii) the third one, the pressure when the membrane is deflected. These readings are then, corrected for effects of offset in the measuring gauge; and membrane stiffness to yield the corrected readings, \( p_u, p_i \) and \( p_v \) respectively (Riaud and Miran, 1992).

The DMT test data in the present study were interpreted according to Marchetti (1980); and, Marchetti and Crapps (1981). Thus, the data reducton are based on \( p_u \) and \( p_i \) vanues; and the material index, \( I_0 \), horizontal stress index, \( K_p \), and dilatometer modulus, \( E_D \), are expressed as:

\[
I_0 = \frac{p_i - p_u}{p_o - p_u} \quad \text{(1)}
\]

\[
K_p = \frac{p_u - p_o}{\sigma_v} \quad \text{(2)}
\]

\[
E_D = \frac{E}{(1-v^2)} = 38.2 \times (p_i - p_u) \quad \text{(3)}
\]

where:

- \( p_u \) = in situ, static pore water pressure;
- \( p_o \) = overburden, vertical effective stress;
E = Young’s modulus of elasticity; and v = Poisson’s ratio.

The significance of these index parameters is that, I₀ is primarily related to the prevailing grain size fraction of the soil; K₀, with the in situ horizontal stress and stress history and, E₀ with soil stiffness.

Over the years, a series of correlations and/or charts relating these index parameters with identification of the soil deposit, its stress history, stiffness (deformation and constrained modulus) and shear strength characteristics have been developed (Marchetti, 1980; Lønne et al., 1989; Rissand and Miran, 1992).

DILATOMETER TEST RESULTS

A total of 14 DMT boreholes were performed in the test site area. The rate of the blade penetration was adjusted to 20 mm/s, and the DMT tests were carried out at every 200 mm depth interval.

The material index, I₀, horizontal stress index, K₀; and, dilatometer modulus E₀ with depth for all 14 DMT tests are shown in Figures 2, 3, and 4, respectively.

The I₀ values greater than one correspond, according to Marchetti and Crapps (1981), to values for silt and silty sand soils. Thus, the soil response in the first 1.5 m of the crust appeared to resemble to that of a cohesionless soil. Between the depths of 1.5 and 4 m, the I₀ values decreased from about 0.6 to 0.25; which correspond, respectively, to silt clay and a clayey soil. Below a depth of 4 m, the I₀ values averaged to 0.25, which is related to a clayey soil. The K₀ profile with depth decreases considerably in the first 3.5 m. It varies from about 60 to 10. Thereafter down, there is an almost linear decrease to a value of 5 at a depth of 10 m (Figure 3). It should be mentioned that K₀ profile obtained in this investigation is systematically higher than those obtained by Marchetti (1980), regardless the state of consistency and stress history of clayey soils. As pointed out by Marchetti (1989), the validity of his results are restricted to cohesive deposits of low sensitivity, non cemented, and for those that have only experienced stress history changes due to mechanical (unloading) effects. This fact could probably explain the influence of these factors in the high difference in K₀ values found for the Leda clay deposit.

The dilatometer modulus, E₀ profile shown in Figure 4, appears to increase from the ground surface down to a depth of 1.0 m. Between this and the depth of 4.0 m a significant reduction occurs. Thereafter down to the end of the tests, there is a very slight decrease of the E₀ values. The K₀ and E₀ profiles with depth are very similar and appear to well characterize the dependence of the soil deposit on the stress history.

The undrained shear strength, cᵤ, profile as shown in Figure 5; delineates a substantial reduction from the ground surface down to a depth of 4.0 m; which has corresponded to a decrease from about 240 to 75
Below that depth, there is a slight decrease and at a depth of 10 m below the ground surface the $c_u$ value is about 60 kPa.

The comparison of the $c_u$ (Figure 5) with $K_0$ and $E_0$ profiles in Figures 3 and 4, respectively; despite of the different factors which may affect the estimation of $c_u$ values, has also indicated that these soil index parameters have a consistent significance with the stress history of the tested clays.

The correlation between $K_0$ and OCR is shown in Figure 6. The consolidation tests carried out in the present study were complemented with those presented by Eden and Crawford (1957), Eden and Law (1980), and Hailo (1978). The relationship obtained in the present investigation is defined by:

$$OCR = (0.3 \cdot K_0)^{1.36} \quad \ldots (4)$$

The Marchetti's (1980) relationship, as has been indicated in Figure 6, clearly shows that his proposal highly overpredicts the preconsolidation stress for the crest, particularly for depths above 4.0 m; where, $K_0$ values were greater than 10. Lacsace and Lumme (1988) have found for several tests done in Norwegian clays that the best fit for Equation 4 is such that the multiplier constant of $K_0$ is equal to 0.225 and the exponent is between 1.35 and 1.67. Powell and Uglow (1988) found for clays tested in the United Kingdom that the multiplier constant and the exponent in Equation 4 can be taken, respectively, equal to 0.24 and 1.32. These results have evidenced that Marchetti's (1980) correlation requires a continuous validation even for cohesive soils, which are not subjected to sensitivity and cementation effects.

The relationship between $K_0$ and the at rest, coefficient of earth pressure, $K_a$, which has been depicted in Figure 7, can be expressed as:

$$K_a = (K_0 / 8.27)^{0.617} \quad \ldots (5)$$
It should be mentioned that the regression analysis is valid for $K_0$ values ranged between 4 and 40. The compressibility and consolidation soil parameters are based, as mentioned before, in the consolidation test results executed in the present study and, those by Eden and Crawford (1957), Eden and Law (1980) and Haile (1978). The $K_0$ profile (Figure 3) decreases linearly with depth below the depth of 4.0 m. The DMT tests were done to depths above of 10.0 m and thus, the proposed relationship (Figure 7) is valid for the clay crust. Marchetti’s (1980) proposal, which is indicated in this Figure, involves $K_0$ values lower than 4 for slightly overconsolidated and normally consolidated clays. It is hoped that, as the data bank increase, these differences can be explained by the influence of sensitivity and effects of consolidation.

The relationship between the ratio of the undrained shear strength, $c_u$, of a soil and the effective vertical stress, $\sigma_v$, has yielded the following equation:

$$\left( \frac{c_u}{\sigma_v} \right) = 0.14 \left( K_0 \right)^{1.55} \quad \text{..... (6)}$$

This expression is shown in Figure 8. The dependence of the ratio $(c_u/\sigma_v)$ with the stress history (OCR) for the tested clay is found using the correlation obtained in Equation 4; and, the ratio between the undrained shear strength, $c_u$, and the vertical effective stress, $\sigma_v$, which relates an overconsolidated and a normally consolidated clay. This ratio is assigned to be equal to 0.22 (Mesri, 1975). A regression analysis with the experimental data of the present investigation has provided:

$$\left( \frac{c_u}{\sigma_v} \right) = 0.22 \left( 0.3 K_0 \right)^{1.09} \quad \text{..... (7)}$$

Lacasse and Lunne (1988), Powell and Uglov (1988) and, Jamiolkowski et al. (1988) have found that Marchetti’s (1980) relationship, shown in Figure 5, is reasonably well verified for soft clay deposits. In the case of stiff clays the difference is of the order of two. Despite the limited shear strength data, Equation 7 can lead to an estimate for the clay crust of about 40 to 50 percent lower than those given by Marchetti (1980). It appears that Equation 6 is valid for the underlying soft, silt clay. The comparison between $K_0$ and $R_{0c}$, being this parameter the ratio between the constrained modulus, $M$ and the dilatometer modulus, $E_{Dc}$ is given in Figure 9.
It should be pointed out that the bounds depicted in this Figure are those proposed by Marchetti (1980), whose data are mostly based on laboratory chamber test and in situ test results for cohesionless soils, and slightly overconsolidated (OCR less than 3) clays. Therefore, it should be kept in mind that Marchetti (1980) has provisionally used chamber test results of overconsolidated granular soils to derive an equivalent correlation, which was admitted valid for overconsolidated clays. Furthermore, the slightly overconsolidated clays tested by Marchetti (1980) are based on $K_0$ values between 0.3 and 0.4; whereas, these values in the present study varied from 5 to 40 (Figure 3), and are valid for OCR ranging from 30 to 2.

As can be seen in Figure 9, the scatter is considerable and no definite relationship is found to exist between $K_0$ and $R$. The stiff and fissured clay samples were very difficult to trim in the laboratory. Sample disturbance effect has, quite likely, contributed to the low values of the constrained modulus for the crust. Schunertmann (1981) has obtained $M$ values of about two times greater than the laboratory ones, for no sensitive and non cemented cohesive soils. The scatter of the data suggests further investigations to provide a valid correlation to predict the constrained modulus, $M$.

CONCLUSIONS

The dilatometer test can provide quick estimates of soil parameters for a fissured and stiff clay. The reproducibility and repeatability of the DMT test have been highly consistent.

The confidence level of the proposed correlations, which relate the Marchetti (1980) indeces with geotechnical soil parameters, will be increased as more data bank become available for the crust layer.

To date, the prediction of the constrained modulus from Marchetti (1980) index parameter's is very poor. A more detailed investigation of the influence of stress level on the stiffness characteristics of the crust will reduce the uncertainties in the reference values of the constrained modulus.

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In situ determination of $c_h$ by flat dilatometer (DMT)

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ABSTRACT: This paper deals with the use of the flat dilatometer test (DMT) for evaluating the in situ horizontal coefficient of consolidation ($c_h$) by dissipation tests. Two different procedures developed for this purpose, the DMT method by Robertson et al. (1988) and the DMTA method by Marchetti & Totani (1989), are briefly recalled and some comparative results discussed. Four case histories are presented where $c_h$ values determined by DMT dissipation tests performed at different well documented NC to OC clay sites in Italy are compared with the corresponding values determined in the laboratory or backfigured from interpretation of field measurements.

1 INTRODUCTION

Different procedures have been developed for evaluating the horizontal coefficient of consolidation ($c_h$) by means of DMT dissipation tests (Robertson et al. 1988; Schmertmann 1988; Lubetregger 1988; Marchetti & Totani 1989). Particular reference is made herein to the “DMTC” method developed by Robertson et al. (1988) and to the “DMTA” method proposed by Marchetti & Totani (1989). The procedures and formulations adopted by the two methods are briefly recalled and some results comparatively discussed in this paper. Other procedures, such as the “A2” method used by Schmertmann (1994) (consisting in a series of A readings after one A-B-C cycle), are just mentioned, since no direct comparative experience is available to the authors at the present time.

In all the proposed methods, the DMT penetration is stopped at a given depth, then some form of decay with time of the total contact horizontal stress ($\sigma_h$) is observed and plotted to infer $c_h$. The target of these procedures is $c_h$ since, as shown by the piezocene (CPTU) research, the water flow occurs predominantly in the horizontal direction.

Research has also indicated that $c_h$ determined this way generally applies to the overconsolidated (OC) range. Expected values of the correction factors for estimating $c_h$ in problems involving loading in the normally consolidated (NC) range have been indicated by Schmertmann (1988) and Marchetti & Totani (1989).

2 DMT DISSIPATION TESTS: PROCEDURE AND INTERPRETATION

2.1 DMTC dissipation test

This method, developed by Robertson et al. (1988), consists in stopping the blade at a given depth and taking, at different times, the sequence of readings A-B-C. C is the closing pressure, determined by slowly defining the membrane after the B reading, until the contact is reestablished.

The DMTC method is based on the assumption that $p_f$ ($p_f = \text{closing pressure}$ C corrected for membrane stiffness) is essentially the DMT penetration pore pressure in the soil facing the membrane, hence the resulting $p_f - \log t$ plot is treated as the dissipation curve of the excess pore pressure and the final $p_f$ after complete dissipation represents the equilibrium piezometric pressure ($\sigma_h$).

This assumption was verified in a number of soft NC to slightly OC clay sites by researchers at the University of British Columbia and the Norwegian Geotechnical Institute.

The procedure recommended by Robertson et al. (1988) for estimating $c_h$ is the following: (a) plot the $p_f$ values obtained from A-B-C cycles versus $\log t$; (b) identify the time for 50% dissipation ($t_{50}$); (c) use equation (1) to estimate $c_h$:

$$c_h = \frac{R^2}{t_{50}}$$

where $R = 20.57$ mm (equivalent radius for a standard 14 mm by 95 mm DMT blade) and $t_{50} = 4$ (DMT time factor obtained from comparison
with the theoretical solutions for CPTU), hence $K' T_p = 17$ cm$^2$
presented a similar procedure for evaluating $c_s$ from
DMTC dissipation tests: (a) plot the $C - t$ curve; (b)
identify $G_p$, (c) use equation (1) to estimate $c_s$,
assuming $R' = 600$ mm$^3$ and $T_p$ = time factor
depending on the rigidity index ($E_i/k_n$) ranging between 1.5 - 2 (from Gupta 1983), hence $R' T_p = 9$
+ 12 cm$^2$ versus 17 cm$^2$ proposed by Robertson et al.
(1988).

2.2. DMTA dissipation test

This method (Marchetti & Totani 1989) consists in
stopping the blade at a given depth, then taking a
sequence of $A$ readings at different times (usually
0.5, 1, 2, 4, 8, 15, 30, 60 etc. minutes after stopping
the blade at the required depth, until stabilization).
Only the $A$ reading is taken, without performing the
expansion to $B$ as in the standard DMT test, i.e.
deflating immediately as soon as $A$ is reached (this
method is also called "A & deflate" dissipation).

The steps suggested by Marchetti & Totani
(1989) for evaluating $c_s$ are: (a) plot the $A - log t$
curve; (b) identify the contractions point in the
curve and the associated time ($t_{Cg}$) (c) use equation (2)
for an average estimation of $c_s_{AV}$
(Marchetti 1997):

\[ c_s_{AV} = \frac{7}{t_{Cg}} \]  \hspace{1cm} (2)

3. DMT DISSIPATION TESTS IN DIFFERENT
CLAY SITES IN ITALY

3.1 Fucino

Several standard DMT and DMTA/DMTC

dissipation tests were performed at the Fucino site,
in center Italy (Totani & Marchetti 1988; Marchetti
& Totani 1989), as a part of an extensive
investigation program. A detailed characterization of
the soil (a thick, quite homogeneous deposit of soft,
highly structured, cemented, NC lacustrine clay) is
DMT test are shown in Figure 1.

A comparison between decay curves obtained
from DMTA ($A - log t$) and DMTC ($p_d - log t$

dissipation tests performed at about the same depths
(5 - 10 - 15 m) is shown in Figure 2.

The in situ $c_s$ values determined by DMTA and
DMTC are plotted versus depth in Figure 3, compared
to the values of $c_s$ and $c_s$ determined in the

Figure 1. Fucino - Typical DMT profiles

Figure 2. Fucino - Comparison between decay
curves from DMTA and DMTC at the same depth
Figure 3. Fusaro - Comparison between in situ $c_0$ by DMT and laboratory $c_v\cdot c_f$ by oedometer tests (after A.G.I. 1991)

Laboratory by oedometer tests at effective stresses equal to twice the yield stress (A.G.I. 1991), hence in the NC range.

The laboratory $c_0$ values (horizontally trimmed specimens), though limited in number, indicate a ratio $c_0 / c_v \approx 1 \pm 1.5$, hence a nearly isotropic behavior of the clay.

The ratio between in situ $c_{0,OC}$ from DMTA and laboratory $c_{0,NC}$ is nearly constant with depth (reflecting high reproducibility and stability of DMTA results) and equal to $3 \pm 4$. This could be explained as an effect of the different stress range (OC versus NC); anyway, the ratio $c_{0,OC} / c_{0,NC}$ in this case is less than presumed previously (Schmertmann 1988; Marchetti & Totoneri 1989).

The $c_v$ values from DTM, determined according to Robertson et al. (1988), are one order of magnitude higher than the laboratory $c_0$ and $= 4$ times $c_v$ from DMTA at the same depths, thus indicating a discrepancy between the two methods.

3.2 Garigliano River

Two standard DMT tests and two DMTA dissipation tests were performed in 1994 in an extensively investigated site located in center-south Italy, where a new bridge over the Garigliano River had to be constructed. The tests were performed nearby a large approach embankment, 6 to 8 m high, instrumented with piezometers and extensometers (Figure 4). Details about site characterization and instrumentation are reported by Mandolini & Viggiani (1992). The results of one DMT test are shown in Figure 5.

The soil is a thick, slightly OC (OCR = 6 \pm 8) in the upper 20 \pm 30 m, decreasing with depth) silty sandy clay deposit. Considering the stress level involved by the problem, consolidation occurs predominantly in the OC range.

Since calculations carried out based on an average $c_v$ determined from laboratory tests ($c_v = 4.5 \cdot 10^{-5}$ cm/s) had indicated a very long time to achieve a significant amount of consolidation, prefabricated vertical drains, 12 to 25 m long, had been installed in order to fasten settlements below the embankment.

An average value of the in situ $c_0 = 0.9 \pm 1.9 \cdot 10^{-5}$ cm/s of the laboratory $c_0$ was backfigured by Mandolini & Viggiani (1992) from interpretation of the radial (due to the vertical drains) consolidation process based on piezometers and extensometers measurements.

The in situ $c_0$ values determined from the two DMTA dissipation tests, performed at 13.90 m and
21.40 m depth (c\textsubscript{R} = 0.4 \times 10^{-3} - 2 \times 10^{-3} \text{ cm/s}) fall in the range of the in situ c\textsubscript{R} values estimated by back analysis. The difference between the two c\textsubscript{R} values from DMTA probably reflects, in this case, some small scale heterogeneity of the deposit. This points out the necessity of performing an adequate number of dissipations, particularly in non homogeneous sites, in order to obtain representative data.

3.3 Santa Barbara mine

In 1994 three standard DMT tests and one DMTA dissipation test were performed in a large clay waste disposal (Forestello) located in the area of Santa Barbara open-pit mine (center Italy), where a considerable consolidation process, monitored over a long period by piezometer cells installed at different depths and locations, was still in progress (Figure 6).

The disposal is formed by the material resulting from excavation of the mine slopes, a heavily OC jointed pliocenic clay, which, due to softening and water content redistribution, is turned into a reconsolidated, isotropic, compressible NC clay (D’Elia et al. 1994). In this case, consolidation occurs in the NC range.

The results of a typical DMT test are shown in Figure 7. The A - log t curve from DMTA (33.20 m depth) is shown in Figure 8.

The values of c\textsubscript{R} determined from laboratory tests on undisturbed clay samples taken from the disposal and backfigured by modeling the consolidation process based on piezometer measurements (D’Elia et al. 1994) are plotted versus the effective vertical stress (c’\textsubscript{v}) in Figure 9. The in situ c\textsubscript{R} from DMTA (c\textsubscript{R,OC} = 1.2 \times 10^{-3} \text{ cm/s}) is also indicated in Figure 9, falls within the range of c\textsubscript{R} (NC) values determined in the laboratory and estimated by back analysis at the same stress level (c\textsubscript{R} = 1-3 \times 10^{-3} \text{ cm/s}).

In this case, since the clay may be considered isotropic (c\textsubscript{R} = c\textsubscript{v}), the ratio c\textsubscript{R,OC}/c\textsubscript{R,NC} is = 1.

Figure 7. Santa Barbara - Typical DMT profiles

Figure 8. Santa Barbara - Decay curve from DMTA dissipation test

3.4 Parma

In 1997 a comprehensive site investigation program, including several standard DMT and DMTA dissipation tests, was carried out near Parma, in the Po River plain (north Italy), in order to characterize a site where a new important railway structure had to be constructed.

The soil is a slightly OC clayey silt with frequent thin sandy layers. The results of a typical DMT test are shown in Figure 10. Figure 11 shows a sample DMTA A - log t curve.

The values of c\textsubscript{R} from DMTA have been compared with the values of c\textsubscript{R} determined in the laboratory from oedometer tests on undisturbed samples taken at the same depth at c’\textsubscript{v} in the recompression (OC) range (Figure 12). Despite the large scattering of the data, reflecting the marked

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Figure 9. Santa Barbara - Comparison between $c_v$ from laboratory tests, in situ $c_v$ from back analysis (D’Ellis et al. 1994) and in situ $c_{v,OC}$ from DMTA

Figure 10. Parma - Typical DMT profiles

Figure 11. Parma - Simple DMTA decay curve

heterogeneity of the deposit, an average ratio $(c_{v,OC})_{DMTA}/(c_{v,OC})_{LAB} \approx 1.5$ may be detected in this case.

4 CONSIDERATIONS ABOUT DMT DISSIPATION TESTS RESULTS

Favourable aspects of DMT dissipation tests are believed to be:

- absence of problems of filter smearing / loss of saturation / clogging (the DMT membrane is anyway a non draining boundary);
- while pore pressures $u(t)$ vary from point to point, settlements (like in the cedometer) and total contact horizontal stresses $\sigma_{h} (t)$ (the membrane can be regarded as a "mini lateral embankment") have a more stable trend, being some kind of integral, thus providing a more stable evaluation of $c_v$ (see Fig. 6 by Marchetti & Totani 1989).

Comparisons between DMTA and DMTC have been attempted, whenever possible. The two methods differ for test procedure, type of "reference time" and formulation. Since only a few data are available of parallel $c_v$ from DMTA / $c_v$ from DMTC (generally one opts for DMTA or DMTC) in sites where reliable reference $c_v$ are available too (as at the Fucino site), it is not possible today to evaluate comparatively the quality of the two methods. However, the authors preference goes to the DMTA method for the following reasons:

- DMTA is perfectly analogous to the well established pressuremeter "holding test" (in the case of the DMT, the fixity of the probe is 100 % insured, being the DMT blade a solid object). While theoretical $c_v$ decay curves are not available for the DMT shape, such curves are expected to be similar to the theoretical curves found for the cylindrical case, since the phenomenon is the same. Waiting for the theory, the most effective way appears to use an experimentally calibrated relation such as the above equation (2).
- The DMTC method relies on the assumption (which may not always be sufficiently approximate) that C = n, using C(t) = u(t) decay curves in an attempt to reproduce the interpretation of C from CPTU.

- In DMTC the C reading is affected by the travel of the membrane during the expansion (Powell & Uglow 1988), which suggests some "procedure dependence" of the results.

- Being the DMT blade rectangular, the selection of the DMT equivalent radius involves further uncertainty, requiring an experimental calibration anyway.

- Determining $t_{eq}$ (DMTA) is, generally, simpler and more stable than $t_{o2}$ (DMTC); usually $t_{eq}$ is very clearly identifiable (except in non-S-shaped curves), while $t_{o2}$ is often of dubious determination due to the uncertainties which in many cases exist at start/end of the decay curves. Comparing equations (1) (assuming an average product $R^2 = T_4 = 14$ cm) and (2), it can be noted that they imply $t_{eq}$ (DMTC) = 2 $t_{o2}$ (DMTA). The authors have attempted, based on a few available data, some comparisons $t_{eq}$ (DMTA) - $t_{o2}$ (DMTC), in order to evaluate if and to what extent the relation $t_{eq}$ (DMTC) = 2 $t_{o2}$ (DMTA) is verified. It has been observed that these values are usually similar, but not systematically one larger than the other. Some cases are reported where the ratio $t_{eq}$ (DMTA) / $t_{o2}$ (DMTC) is found to be $\leq$ 1 (at Furno $t_{eq}$ (DMTA) / $t_{o2}$ (DMTA) = 1; Chang (1994) illustrates a case in Singapore marine clay where this ratio is $\approx$ 1.6 - 5.6). Obviously, larger amount of parallel DMTA-DMTA data is needed in order to draw general conclusions about this point.

5 CONCLUSIONS

- The persisting scarcity of adequate reference data still does not permit to evaluate adequately the $C_b$ predictions by DMTA. However, in the cases presented in this paper, the values of $C_b$ from DMTA are in relatively satisfactory agreement with the corresponding values determined in the laboratory or backfigured from interpretation of field measurements.

- The comparative data presented in this paper, obtained for both NC and OC clay sites, show that $C_b$ values from DMTA dissipations are highly reproducible, stable, procedure and operator independent.

- The ratio $t_{eq}$ (DMTA) / $t_{o2}$ (DMTC) has been found to vary significantly from site to site, in the range 1 to 3 - 4 (i.e. DMTA predictions 1 to 3 - 4 times faster than predictions based on laboratory). Such range of variation is high, but not as high as previously estimated (Schmertmann 1988; Marchetti & Totani 1989). Further insight is needed into the problem of the identification of "correction factors" from OC to NC state (and, in general, from field to laboratory).

- In the examined cases, predictions of the settlement rate based on $C_b$ from DMTA, without applying any correction factor, would have been not too far from field behavior, being the DMTA predictions slower by a factor 1 to 3 $C_b$ from "real life" back calculations = 1 to 3 times $C_b$ from DMTA.

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A large scale plane strain test facility

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ABSTRACT: The test unit described herein is designed to calibrate and test a specially designed flat plate dilatometer under conditions of plane strain. To simulate plane strain field conditions in the laboratory, a prismatic sample is enclosed in a box shaped rubber membrane and on two opposite ends by rigid steel plate sections. The open sides of the membrane impose controllable horizontal stress conditions while the rigid steel sections are used as restraint to ensure plane strain conditions. The prismatic unit is contained within a cylindrical chamber which is connected by flanges to two circular shaped end plates at the top and bottom of chamber. The sample is formed by deposition of sand, falling through air from an overhead hopper. Vertical stresses are applied to the specimen by an air bag, placed on top of the sample top plate while horizontal stresses are applied by water pressure, which enters the chamber through the base plate.

INTRODUCTION

In situ tests and particularly penetrometers provide rapid estimates of soil parameters for predicting the performance of real structures. However, the capability of the tests to provide optimum interpretation of results is hampered by the lack of experimental data obtained under controlled conditions. A need exists for further study of the performance of such tests in the laboratory under controlled boundary conditions and known applied stresses. Calibration chambers have been used extensively to help in the process of developing correlations between in situ penetrometer test results and different soil parameters for conditions of axial symmetry. In this case, similar comparisons will be made but under conditions of plane strain. The test chamber described herein has been designed and fabricated at the University of Hong Kong.

The major effort in the testing program will be to evaluate a new flat dilatometer designed with a cross section geometry that minimizes penetration disturbance in the vicinity of the test diaphragm. The facility will permit, in the first instance, evaluation of the capability of the new design to measure horizontal stress, in this case the applied cell pressure. If successful, it is proposed to use the facility to examine the use of the dilatometer unit for measurement of several other parameters including relative density and constrained modulus of sand.

GENERAL DESCRIPTION OF TEST SYSTEM

The chamber was designed to allow independent application of vertical and horizontal stresses to the sand specimen while preventing strain in the third direction. In general, tolerable deflections governed the design of the unit rather than allowable stresses. The specimen is rectangular, with cross section 1 m x 0.6 m, height 0.876 m and weight approximately 1 ton. The prismatic shaped sample is enclosed in a rubber membrane and on two opposite ends by rigid steel plate sections. The general arrangement of the test system is shown in Fig. 1 and a more detailed representation of the test chamber is shown in Fig. 2.

The dilatometer is pushed into the sample by the hydraulic cylinder mounted on the loading frame to various depths of penetration for testing. The cylinder is double acting and has a stroke of 600 mm. The cylinder is mounted on the loading frame at a sufficient height above the top of the chamber to allow push rods to be connected to the cylinder for pushing the dilatometer into the sample. The frame
is connected to channel sections at the top and bottom of the chamber.

The main rubber membrane is made from 1.5 mm thick natural rubber sheet, lap-vulcanized to form an open ended rectangular box. It is glued to the sample base support and clamped to the sample top cap by 3 mm thick steel strips, as shown in Figure 3.

The surrounding steel frame at the top of the end sections (Figs 2, 3) acts as a support which prevents lateral movement and tilting of the air bag during inflation.

Sand sample preparation

The sample is formed by the deposition of sand falling through air from an overhead hopper and then through two levels of No. 10 wire mesh sheeting, as shown in Fig. 4. The sand hopper is mounted on a frame with wheels at the base to enable positioning over the chamber.

The jets of sand originating from a perforated plate in the base of the top storage bin are dispersed into a uniform rain by sieves placed in their path. This follows methods used previously by (Kolbuszewski & Jones 1961, Chan 1995) and aids in producing repeatability for preparation of a relatively homogeneous sand specimen at a controlled density.

PILOT STUDY

In order to obtain the details for the larger scale design for the sand used, a pilot study was
performed using a 150 mm diameter tube system as shown in Figure 5. The sieves were placed below the perforated plate at a spacing of 100 mm.

Control of specimen density was obtained by varying (a) the diameter and spacing of the holes at the base of the sand hopper and (b) the fall distance from the underside of the sieve to the specimen surface. The pilot study provided a very simple means to determine the required parameters for the full scale tests.

The replaceable perforated plates used in this study, as shown in Figure 6, were 4 mm thick circular perspex plates with a series of machined holes that fall within the 150 mm diameter outline of the tube system. The resulting densities produced by the respective plates for various heights of fall are shown in Figure 7.

The results of the pilot study confirmed that, in order to obtain low densities, the flow quantity must be maximized while the fall height is minimized. To achieve the desired conditions, it is required that the sieves move upwards during the specimen preparation process to maintain a constant height above the sand surface. This presented a major difficulty owing to the lack of access to the sand bed during deposition. An electronic sensor for detection of bed height as shown in Fig. 8 is to be installed at the base of the sieves to solve this problem.

**SUMMARY**

A large scale test chamber has been designed to test a modified flat dilatometer in the laboratory under plane strain conditions.

Details are given of (a) the design of the test chamber, (b) the facility to be used in placing a controlled density with relatively homogeneous test specimens and (c) a pilot testing programme to enable execution of the detailed design of the controlled placement facility.

The chamber will permit testing under a range of controlled vertical and horizontal stresses, in which the vertical stresses are applied through an air bag and lateral stresses through cell water in a similar way to that of the conventional triaxial test.
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Long-term set-up of driven piles in non-cohesive soils evaluated from dynamic tests on penetration rods

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ABSTRACT: Long-term increase in bearing capacity of driven piles in non-cohesive soils can be substantial as several studies have shown. This paper describes a method to evaluate the set-up of driven piles and is based on performing dynamic loading tests on penetration rods at different times after installation. The method was studied in a field test involving 21 rods and three concrete piles. The piles showed a set-up of 40% on average per log cycle of time and for the rods this was slightly less. However, the scatter was large. Nevertheless, the method clearly demonstrated the possibility of it being a valuable tool for evaluating set-up of driven piles.

1 INTRODUCTION

Long-term set-up is defined as an increase in bearing capacity with time that takes place after pile installation and the dissipation of excess pore pressures induced during driving. Long-term set-up in non-cohesive soil can roughly be divided into two main time-dependent causes, based on Schmertmann (1991) and Chow et al. (1996, 1997):

1. Stress relaxation in the surrounding soil arch (creep), which leads to an increase in horizontal effective stress on the shaft.
2. Soil aging, which leads to an increase in stiffness and dilatancy of the soil.

Long-term set-up of driven piles in non-cohesive soil can in many cases be substantial as several studies have shown. The problem is, however, to predict the size of the set-up at a specific site and its time-dependent characteristics, which would enable practical use of the full effect. Chow et al. (1997) summarized the results from 10 studies including case histories with long-term set-up effects from pile driving in sand. The database shows that the long-term set-up is normally in the region of 50 to 150 percent over a 100-day period, although the scatter is large.

Denver & Skov (1988) presented a linear relationship between pile set-up and the logarithm of time based on three case histories of load tests on driven piles in sandy, clayey and calcareous soil. For the piles in sandy soil the load tests were performed between 0.5 and 23 days after end of driving. Case histories presented by Svinkin et al. (1994), Pelletier et al. (1989) and others confirm this relationship, at least up to a month from end of driving. Åsotd et al. (1994) stated, on the other hand, that it continues for several months.

Axelsson (1998) measured the increase in horizontal stress on the shaft due to stress relaxation on two concrete piles, instrumented with earth pressure cells on the shaft, and driven in loose to medium-dense sand. It was observed that the increase in horizontal stress on the pile shaft after driving followed the relationship by Denver & Skov during the whole measuring period of 72 days, however, a slight tendency to level off was noted. Furthermore, the rate of this increase was clearly dependent on depth. It was concluded that the most probable cause for the observed long-term set-up, evaluated from dynamic loading tests performed at different times, was primarily soil aging in combination with soil particles interlocking with the surface roughness. This caused increasing dilatant behavior with time and generated a strong increase in horizontal stress on the shaft during loading. That, in turn, was indicated by the very low horizontal effective stresses measured on the shaft at rest, although very high shaft capacities were mobilized during the actual loading. Also, it was observed that only a minor part of the set-up was due to stress relaxation.

Chow et al. (1996), on the other hand, argued that the observed long-term set-up of some open-ended pipe piles driven in a dense sand was due mainly to stress relaxation. Furthermore, only a third of the set-up was attributed to changes in dilution and stiffness through soil aging.
This paper describes a method, in which dynamic loading tests are used on ram penetration rods to evaluate the long-term set-up of driven piles. The method is based on determining the relative increase in total capacity of the rods at different times after installation. It is primarily used in non-cohesive soils, where the excess pore pressures normally dissipate within a few hours. The method has previously been studied by Astfelt & Holm (1999) in connection with 5 piling projects in Sweden involving 13 rods and 15 piles in total. Dynamic testing on ram penetration rods can also be used to estimate the driveability and the total capacity of piles as shown by Erkasson (1992), although this is a more difficult task than measuring a relative increase in bearing capacity.

An evaluation of the method is based on a field test involving both driven piles and penetration rods. Dynamic testing was performed on three 235 mm square concrete piles, and twenty-one 32 mm diameter steel penetration rods, at different times after installation. They were all driven in a loose to medium-dense glacial sand to a depth of 19 m. The main objective was to compare the set-up for the piles and rods, to find out if the method would produce relevant results, and if there is a direct relationship between the set-up of the piles and the rods. The site and the piles are the same as in the study by Axcelsson (1998) mentioned previously.

The soil investigation consisted of one CPT-test, 21 dynamic penetration tests with the 32 mm penetration rods and soil sampling at six levels. Furthermore, the pore pressures were measured using one piezometer and one open stand-pipe.

2.1 Soil conditions

The soil consisted of more than 40 m of loose to medium-dense glacial sand. The results from the CPT and the dynamic probing show a soil that is relatively homogenous with respect to the penetration resistance as indicated in Figures 2 and 3. The groundwater table lies approximately 2.0 m below ground level. Measured and calculated soil properties are presented in Table 1. The soil is well-grained and varies between a silty sand and a gravelly sand. The relative density (D_r) is estimated to lie between 35-50 % from the CPT using a relationship from Jamiołkowski et al. (1985). The sand is considered to be normally consolidated with respect to its geological history. Furthermore, the sand consists mainly of hard minerals, such as quartz and feldspars.

![Fig. 1 Plan view over the test site.](image)

![Fig. 2 CPT results.](image)

### Table 1. Soil properties.

<table>
<thead>
<tr>
<th>Sampling depth</th>
<th>Soil type</th>
<th>Uniformity coefficient D_30/D_60</th>
<th>Mean particle size d_(50)</th>
<th>Relative density D_r [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.1</td>
<td>Silty sand</td>
<td>13</td>
<td>0.16</td>
<td>35</td>
</tr>
<tr>
<td>10.1</td>
<td>Silty sand</td>
<td>9</td>
<td>0.15</td>
<td>35</td>
</tr>
<tr>
<td>13.2</td>
<td>Sand</td>
<td>7.5</td>
<td>0.25</td>
<td>40</td>
</tr>
<tr>
<td>15.8</td>
<td>Gravelly sand</td>
<td>6</td>
<td>1.0</td>
<td>50</td>
</tr>
<tr>
<td>18.2</td>
<td>Sand</td>
<td>5</td>
<td>0.55</td>
<td>40</td>
</tr>
</tbody>
</table>
3 TESTING PROCEDURE

3.1 Rod Installation

The rods were installed with the Swedish standard equipment used for performing dynamic penetration tests (dynamic probing). In this test the penetration resistance is registered in blows per 20 cm ($N_0$). The weight of the hammer is 63.5 kg and the drop height is 50 cm. The rod is normally a 32 mm diameter solid steel rod with an enlarged 45 mm diameter toe to minimize the shaft friction. However, to resemble the behavior of normal pile driving, the dynamic probing in this study was performed without an enlarged toe. Furthermore, in order to keep the rods straight, they were rotated every meter.

3.2 Pile Installation

The three concrete piles, 235 mm-square, were installed under easy driving with a 4-ton hydraulic hammer and a drop height of 20 cm. Figure 4 shows the driving resistance of the piles. First Pile A then Pile B were driven to a depth of 19.1 m. Several months later, after the test program for Piles A and B was concluded, Pile C was driven to the same level in between both the piles.

A piezometer was placed at a depth of 7.1 m in the silty sand at a distance of approximately one meter from where Pile A was driven. A sudden increase in pore pressure was registered when the pile tip passed this level. However, after further driving, the excess pore pressure dissipated rapidly. Furthermore, it was estimated that the excess pore pressures dissipated completely within 5-10 minutes after the driving had ended.

3.3 Dynamic testing

Dynamic testing with stress wave measurements was performed on the three piles and the 21 rods, at different time intervals from the end of driving, up to a maximum of 216 days for Pile B and 69 days for rods H13-H15. The tests were performed using a Pile Driving Analyzer (PDA), together with strain gages and accelerometers attached to the pile top. The drop-height for the piles was increased with every test in order to take the set-up effect into account and to fully mobilize the capacity. The permanent settlement for each blow was measured.

To determine the static capacity and its approximate distribution along the shaft and on the toe, CAPWAP analysis (stress-wave signal matching) of the measured signals was performed for seven of the rods (H2, H5, H6, H13, H15, H19 and H20) and all the piles. The static capacity was also calculated for all the tests using the Case Method (RMAX). For this, Case damping factors of $J = 0.7$ for the piles and $J = 0.3$ for the rods were chosen. These damping factors produced capacities that best corresponded with the CAPWAP results.

4 RESULTS

The rod capacities are summarized in Fig. 5. It is interesting to note the increasing scatter with time. Part of the scatter can be explained by the fact that a few of the rods probably were disturbed by the installation of nearby rods. It is estimated that this affected the capacity within a radius of approximately two meters. In the figure the rods that might have been affected by the installation of a nearby rod (within a distance of two meters), are presented in black. Another possible explanation for this large scatter in the deflection of the rods during
installation, and this possibly having a greater influence on the capacity with time.

The test results show an increase in capacity for both the piles and the rods over the whole measuring period. In Fig. 6 this increase is presented as a factor $(Q/\bar{Q})$ versus the logarithm of time $(\log_{10} T)$. Here, $Q_0$ is the capacity at the time $T_0$, which for these tests was one day, and $Q$ is the capacity corresponding to the time $T$. Furthermore, the relationship by Denver & Skov is presented for $A=0.2$ and $A=0.8$. The increase in capacity was approximately linear with the logarithm of time for both the piles and rods. The total pile capacity increased approximately 40% $(A=0.4)$ on average per log cycle, whereas the increase in capacity for the undisturbed rods was slightly less. However, the scatter was large. Overall, the set-up for the undisturbed rods was somewhat smaller than for the piles. Whereas, a few of the disturbed rods showed a greater set-up than the piles.

For the rods the static toe capacity evaluated from the CAPWAP analyses was approximately 5 kN (6 MPa) and roughly constant over time. However, the scatter was large. These results correspond very well with the CPT results, which showed a $q_t$ of 5-7 MPa at the same depth. They also correspond very well with the toe capacity of the piles, which showed an average of 355 kN (6.1 MPa). Also, the toe capacities of the piles were more or less constant over time, and consequently, this implies that the set-up primarily takes place along the shaft.

Furthermore, the average shaft capacity of the rods, between one and 30 days, increased from approximately 15 kN (8 kPa) to 25 kN (13 kPa), which is 67%. Whereas, the capacity of the piles increased from approximately 600 kN (34 kPa) to 1100 kN (62 kPa), which is 83%, during the same period of time. A more detailed description of the results from the dynamic tests on the piles is given by Axelsson (1992).

Figure 7 shows that there is a non-linear relationship between the rod capacity and the permanent settlement per blow during driving and restrike. From this one may gather that it is not possible to evaluate the degree of set-up solely on measurements of permanent settlement.

5 DISCUSSION

The fact that the piles show a slightly higher set-up effect than the rods is in line with the results from field tests in silty and sandy soils by both Eriksson (1992) and Åstedt & Holm (1995). The difference in set-up between piles and rods can be explained by the two main pile and rod parameters, the size (diameter) and the surface roughness.

Firstly, the size has an influence on the degree of soil arching and the overall soil disturbance, and consequently also on the nature of the stress relaxation.
Fig. 6. The increase in pile (P) and rod (R) capacity.

\[ \Delta \sigma_v = \frac{2\mu G}{R} \]  \hspace{1cm} (1)

where \( G \) is the shear modulus for an elastic soil mass and \( R \) is the pile radius. Equation (1) implies, that an increase in horizontal stress during loading due to dilation is inversely proportional to the size (radius). This means that a change in dilatancy and stiffness through soil aging has a greater influence on a rod than on a pile.

Secondly, the interlocking of soil particles with the surface roughness has an influence on the interface dilatancy, and consequently also on \( \Delta \sigma_v \).

To be able to make full use of the results from set-up tests it is important to understand the basic mechanisms behind set-up and their influence on the capacity. Chow et al. (1997) and Axelsson (1998) give a rational explanation of the mechanisms involved during set-up. In Table 2 some conclusions are presented as to how the main pile and rod parameters may affect the degree of set-up.

The method has clearly demonstrated the possibility of it being a valuable tool for evaluating the set-up behavior of driven piles. However, as this and previous studies show, a large scatter in the capacity should be expected. Further, the method is cost- and time-efficient compared to the alternative, which today is full scale pile testing.
Table 2: Hypothesis of how the basic mechanisms may influence the degree of set-up for piles and rods.

<table>
<thead>
<tr>
<th>Size (Radius)</th>
<th>Steel rods</th>
<th>Concrete piles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>An increase in slenderness and brittleness (soil aging) has a greater effect on a small rod</td>
<td>Large soil disturbance during installation - strong welding effect and stress relaxation</td>
</tr>
<tr>
<td>Surface roughness</td>
<td>Weak interlocking between soil particles and rod</td>
<td>Strong interlocking between soil particles and rod, leading to large dilation effects during loading</td>
</tr>
<tr>
<td>Expected degree of set-up</td>
<td>Medium</td>
<td>High</td>
</tr>
</tbody>
</table>

Rausche et al. (1995) performed 20 torque and uplift tests on SPT rods in clayey sandy soils. The results indicated that there is a linear relationship between the torque and the static shaft capacity. This suggests that torque tests also can be used to evaluate pile set-up.

6 CONCLUSIONS

- The results from the present and previous field studies indicate that there is a relationship between the set-up of piles and rods, and consequently that it is possible to assess the set-up of driven piles from dynamic tests on penetration rods.

- Furthermore, the understanding of how the set-up mechanisms affect piles and rods suggests that this relationship will vary with both soil conditions and pile type.

7 ACKNOWLEDGMENTS

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8 REFERENCES

Comparison of SPT energy measurement methods

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ABSTRACT: Standard penetration test can exhibit considerable variability and this is partially due to differences in the energy delivered to the drill rod during driving. A standard delivered energy has been established and procedures for measuring energy have been standardized by the American Society for Testing and Materials. The procedure specified for computing energy, integrating the square of the impact force, has been controversial. Measurements made on 44 drill rigs obtained during routine drill rig calibration, have been collected and re-analyzed using two other computational methods. A total of more than 9000 hammer impacts were processed. It is concluded that the use of the integral of the product of the impact force and velocity gives more reliable results for the drill rod energy than the integral of the force squared. The force squared method produces results that are 10% higher than the force velocity method but the variability of the results is much greater.

INTRODUCTION

The standard penetration test is still the most common in situ test employed for foundation and stability analyses worldwide despite its variability and frequent lack of repeatability (Coduto 1994; Kovacs & Salomone 1982). A primary reason for the variability of performance is the wide range of energy that the different SPT systems deliver to the drill rod. The key bit of data that results from the standard penetration test is the penetration resistance, or N-value. Many empirical correlations have been established, in which the N-value is used in predicting several soil properties, including allowable bearing capacity, shear strength, relative density, settlement, and compressibility. It is also used in corrections for cone penetration tests (Mayne & Kemper 1980), Becker penetration tests (Ny & Campanella 1994), and in liquefaction analyses (Seed et al. 1985).

Before the N-value can be used in such predictions, the input energy should be normalized to a standard value so that its use is consistent everywhere. This results in a corrected N-value reflective of the efficiency with which energy was transferred to the sampler during driving. The de facto U.S. standard is the N60, corresponding to 60% of the potential energy of the hammer at the top of the specified stroke. All specific efficiencies, as measured, are corrected to this value for comparison. Because it would be easier to refine and control this test than to overhaul the entire industry’s practice, it also makes sense that accurate and reliable energy measurement should be utilized.

There are two computational procedures used in practice today to calculate energy delivered to the drill string, the Fp and Fv methods and there are limitations and problems associated with each method. However, there has been no study that compared the two methods when used on production measurements. In order to sort out the differences between the Fp and Fv methods, a study was made to statistically compare energy measured by each (Butler 1997). A third hybrid method, the Fv-corrected, was also investigated.

The data was collected from field measurements of SPT energy transfer during drill rig calibrations performed by Goble Rausche Likins and Associates, Inc. (GRL) over the past four years. The compilation of this database also allowed for further documentation of characteristic drill rig efficiencies, and for verification of the theoretical limitations of each method. The energy transmitted
by these different configurations was computed by
the $F^v$ and $F^e$ methods.

The objectives of this study are summarized as
follows:

1. To provide a comparative analysis of the two
   methods of energy measurement, based on
   theoretical limitations of each method, and a
   statistical analysis of the data.

2. To study the effects of different drill rod
   configurations on the energy transmission, as
   measured by the $F^e$ and $F^v$ methods.

3. To determine the characteristic efficiencies of
   the particular drilling systems studied in
   this project.

Only the result of the evaluation of the
computational methods will be presented in this
paper.

BACKGROUND

The energy transmitted into a rod during impact is
(Timoshenko and Goodier 1970)

$$\Gamma_N = \int^2_0 F(t) v(t) \, dt$$  \hspace{1cm} (1)

where $F$ is the impact stress wave force measured
in the rod, usually near the top, $v$ is the particle
velocity measured at the same location in the rod,
and $t_{max}$ is the time at which the integral has its
greatest value. Schmertman and Palacios (1979)
made the first measurements of drill rod energy and
in that project they encountered difficulty in making
velocity measurements with the transducers that
were available at that time. They avoided this
problem by using the relationship

$$\Gamma_i = \frac{c}{EA} \int^2_0 F(t)^2 \, dt$$  \hspace{1cm} (2)

where $E$ is the modulus of elasticity of the rod
material, $A$ is the rod area, $c$ is the velocity of
wave propagation, $L$ is the length of the rod, and $c$
is the velocity of wave propagation in the rod.

This computational approach depends on the force
velocity proportionality given by

$$F = \frac{EA}{c} \nu$$  \hspace{1cm} (3)

This expression holds where the wave is
propagating in the positive direction only. Thus,

Eq. 2 is also limited to cases where the wave is
propagating in the positive direction (downward)
direction only. Reflections that travel in the
opposite direction up the rod can come from cross
section changes due to connectors or loose
connections, and shaft resistance. In an SPT drill
rod, there should be no shaft resistance since testing
is done in an open drill hole. However, reflections
may come from loose connectors, from the
additional area of the connector, and from the end of
the drill rod and these reflections will be superimposed
on the downward traveling wave, thus destroying proportionality. Since the end of
the rod causes a large reflection Eq. 2 is integrated
to the time of the arrival of the rod tip reflection,
a time equal to $2L/c$. The basic method of Eq. 2
was used when SPT energy measurement was
standardized in ASTM D4633-86.

If the drill rod is of sufficient length, then the
energy in the ram will have been transferred in
the initial wave cycle. However, for shorter rod
lengths, energy transmission is not complete at the
$2L/c$ time. A correction factor, based on rod
length, is then included in Eq. 2. Other corrections
were included in ASTM D4633-86 but they will not
be discussed here.

Last revised in 1986, the ASTM (1992)
standard is currently under revision because of
deficiencies in the old standard, the appeal of the
$F^e$ method of energy calculation, and the reliability
of new technology that allows for more accurate
velocity measurements. This study concentrated on
comparing the two methods of energy measurement
proposed in the new standard. This is done
through comparison of measured energy, transfer
efficiencies, velocity measurement, and kinetic and
potential energy. Although many factors have been
found to affect energy transfer, only a few of them
are presented in this study.

DATA COLLECTION

The basic data used in this project was collected by
GRL during routine drill rig calibration testing, at
various sites located across the United States. Each
drill rig, with different SPT systems, was tested at
varied depths and geology. The measurements
were made with the Pile Driving Analyzer (PDA),
a portable field instrument that collects, processes,
and analyzes data in real time during pile driving or
drilling operations (Hussein & Likins, 1995).
During this testing, the goal of the test was to
determine the existing efficiency of the SPT
system. No effort was made to improve its operation or correct problems.

The PDA conditions, processes, and stores measurements made on a short, instrumented section of SPT rod, which is connected into the drill string immediately below the anvil. It is desirable that this section be of the same area as the rod being used so that wave transmission will not be affected. For the data collected in this study, this was not always the case. In most cases, the force measurement was made with a drill rod section that was instrumented with foil strain gages and calibrated. However, for some data the force measurements were made by attaching strain transducers. In all cases, accelerometers were attached to the same drill rod section and velocity was obtained by integrating the measured acceleration. During the SPT test, these measurements were displayed on the PDA screen as time history traces of force and velocity so that the PDA operator can continuously evaluate the measurement quality. Concurrently, the record from each blow is stored to a disk for possible re-analysis and qualitative evaluation.

A software version of the PDA (called PDAPC) was used on a personal computer to retrieve the force and velocity records that had been stored on disk. Each record was accessed via the personal computer where a series of qualitative refinements and evaluation could be done before the data was used quantitatively. The desired energy values (as calculated by the PDAPC using the \( F_v \) and \( I^2 \) methods) were saved to an ASCII file.

**DATABASE COMPILATION AND ANALYSIS**

A spreadsheet was chosen to compile the data collected with the PDA into a data base so that could be organized in a way that allowed for it to be clearly seen, and that permitted easy numerical manipulation. This database is comprised of 273 records of data consisting of from 3 to 168 individual hammer blows. The size of a record usually depended on the driving resistance. A record was obtained from one sample depth (or one rod length). There are 44 sets of records, each of which was collected with a particular SPT system.

A total of 9,919 blows were individually analyzed during the creation of the data base. An individual record includes a description of the drilling system, a qualitative evaluation of the data quality, the mean measured energy, the efficiency and the coefficient of variation of the data for each computation method.

**QUALITATIVE ANALYSIS**

The PDAPC software was used for the qualitative analysis of the data. The purpose of doing a qualitative analysis was to refine the data so that energy from the two methods could be accurately and fairly compared. This involved the elimination of certain bad records that occurred as a result of an erroneous initial blow, a bad strain transducer or accelerometer, or simply a poor electronic connection. Once the data had been "pruned" and imported into the spreadsheet, the force and velocity histories were visually inspected in accordance with the following criteria:

1. Force - velocity proportionality over the first 21\% of time
2. Velocity adjustment magnitude
3. Strain transducer - accelerometer functioning
4. Amount of unusable data
5. Expected energy magnitudes

Using these criteria, each record was given a subjective data quality rating, to "weight" the data during qualitative evaluation and statistical analysis. The Subjective Data Quality Rating given in the Project Database is a function of the five criteria described above. These criteria are listed in order of importance, with proportionality being first. Hence, the rating is a good indication of proportionality, and was used as such in the statistical analysis of the energy measurement methods.

**QUANTITATIVE ANALYSIS**

Following the qualitative assessment of the data the energy measured by the \( F_v \) and \( I^2 \) methods was imported into the spreadsheet. Anomalous data had been removed so that all that remained was a block of energy data based on measurements judged to be correct. From this block, the mean, standard deviation, and coefficient of variation were computed for each method, and transferred to the Project Database. The amount of data discarded for the \( I^2 \) and \( F_v \) methods was recorded. Overall, about 5\% more blows were discarded for the \( I^2 \) method than for the \( F_v \) method. The energies and associated efficiencies computed by the \( I^2 \) and \( F_v \) methods will be denoted by subscripts \( r \) and \( v \), respectively.

The \( F_{enc} \) energy measurement method is unique
to this study. This method involved manually scaling the energy computed by the Fv method on the computer screen at the 2L/c cutoff time, from the display of the PDAPC. Three measurements were selected at random from the blows in each record, the method was applied, and the three results averaged. The average was then multiplied by the corresponding ASTM correction factor \(K_s\). Both \(F_{v,corr}\) and \(\Gamma\) are measured at the 2L/c time, and both are multiplied by the ASTM correction factor \(K_s\).

The value of \(K_s\) is highest for short rod lengths, dropping exponentially to 1.00 for longer rod lengths. For an AW rod, \(K_s\) equals 1.00 for rod lengths \(\geq 13.9\) meters; for an NW rod, \(K_s\) equals 1.00 for rod lengths \(\geq 7.4\) meters. The larger the rod cross-sectional area, the shorter the rod length needed for \(K_s\) to equal 1.00. Thus, for the same range of SPT rod lengths, \(K_s\) has less of an effect on NW rod than on AW rod.

RESULTS AND DISCUSSION

Extensive statistical studies were performed on the data by Butler (1997). These studies were much too extensive to present in this brief paper. The most interesting data from the point of energy measurement will be summarized and discussed. A total of 44 different rigs and SPT systems were examined. Between one and 13 records were measured on each rig with an average of about six.

The averages of the energy measured for each record were averaged to obtain the measured rig performance. These values were then examined to obtain the means and coefficients of variation. The results are given in Table 1.

Table 1. Means and dispersions for the measured energies for each computation method.

<table>
<thead>
<tr>
<th>Calculation</th>
<th>Mean</th>
<th>COV</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fv</td>
<td>61</td>
<td>23</td>
<td>85 - 51</td>
</tr>
<tr>
<td>Fv,corr</td>
<td>60</td>
<td>25</td>
<td>82 - 19</td>
</tr>
<tr>
<td>(F^2)</td>
<td>67</td>
<td>40</td>
<td>156 - 19</td>
</tr>
</tbody>
</table>

It can be seen that the average energy ratio was 10% higher for the \(F^2\) method than for the Fv method. The difference between Fv and Fv,corr was small. The variability in the computational procedure is given by the range of average energies obtained for each of the three methods. The range of average efficiencies for the \(F^2\) method was much larger than for the other two methods with extreme values that were both larger and smaller than those obtained for the other two methods. This same information is also contained in the coefficients of variation. It is interesting to note that of the 44 results the Fv method gave larger results than the \(F^2\) method for 18 rigs while \(F^2\) was larger for the other 26 cases. The \(F^2\) gave values for efficiencies greater than 100 percent for eight of the rigs. Abou-matar and Goble (1997) have given explanations based on one dimensional wave mechanics for the problems with the \(F^2\) method.

CONCLUSIONS

From this study, the following is concluded:

1. Changes in SPT rod cross-sectional area from connectors or slack joints result in a wave reflection that disrupts the force-velocity proportionality within the time interval from 0 to 2L/c. When the proportionality is disrupted, the energy measured by the \(F^2\) method is inaccurate. Disruptions in force-velocity proportionality do not affect the Fv method.

2. For high penetration resistances resulting in blow counts above 50, the \(F^2\) method over-predicts energy transfer because of a high force reading at the 2L/c time. This results from a compression wave being reflected from the rod string toe rather than a tension wave. It is difficult to determine the appropriate 2L/c cutoff time due to the rise time in the force at impact.

3. For long rod lengths, normal blow counts, and good proportionality, the values of energy measured by the two methods agree very well. However, this conclusion was reached without minimizing the effects of hammer type, drilling method, drill rig type, and drill rod size.

4. Quality control of energy measurement should be implemented and adhered to. If the first zero force is not within the time range of 0.9 to 1.2 times 2L/c, the energy value measured by the \(F^2\) method cannot be used. This should be checked regularly during testing activities. If the velocity measurement for the Fv method is not close to the zero axis late in the record, its value of energy measurement may be unreliable. This should also be regularly checked in the field. Force velocity proportionality with due consideration of the effect of reflections from cross section change should be carefully evaluated.
5. For the data set analyzed, the relationship of energy predicted by each of the three methods is as follows: \( \Gamma_n < \Gamma_{E_n} < \Gamma_n \). This relationship indicates that energy measured by the Fv method is the lowest, and will thus result in the lowest \( N_0 \). The efficiency of the Fv method will average 10% higher than the Fv method but it can produce results that are smaller than Fv so the results cannot be easily corrected by a simple multiplier.

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CPT-SPT correlations for some Brazilian residual soils

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ABSTRACT: Correlations between CPT and SPT for different types of residual soils from different parent rocks have been established. The correlations presented a large scatter, mainly due to intrinsic heterogeneity of residual soils. Different parent rocks generally produced different correlations for the same soil type as far as particle size distribution is concerned. A general trend of decreasing values of q/N with the decrease of grain size was found, but the approach of correlating q/N with D_60 did not provide good results.

1 INTRODUCTION

The Standard Penetration Test (SPT) is the most common in situ test for foundation design in Brazil. The SPT equipment used is described by e.g. Lima (1979) and Décourt et al (1989). Pin-weight hammers, hand lifted, are used in almost all cases. The anvils are light, about 0.5 to 3 kg (Décourt et al 1989, Moodie 1997). Standard rods 25 mm have a mass of about 3 kg/m. The SPT is performed every meter.

Measurements carried out in 2 series of tests have suggested that the soil energy ratio of a typical, correctly performed, SPT is about 72% of the theoretical free fall energy (Décourt et al 1989) for long drive rods.

Mechanical Cone Penetration Test (CPT) has generally been used as a complementary tool for the design of piles (Velloso 1959) or for the design of shallow foundations (e.g. Burata & Danziger 1995). Electrical CPT's have been used for offshore soil investigation (Mello & Bogossian 1997) and recently for research purposes (Rocha Filho & Carvalho 1988, Schmied et al 1995).

There are a number of sites where SPT and CPT are available, but very few of the existing correlations correspond to residual soils.

The paper presents a database of CPT-SPT correlations for residual soils from different types of rocks. A total of 49 SPT and 59 CPT profiles, corresponding to 374 data pairs, have been analyzed. The energy delivered to the rods, considering the length of the drive rods, was taken into account.

2 PREVIOUS CORRELATIONS BETWEEN CPT AND SPT

Correlations have been proposed between the blow count N from the SPT and the cone resistance q, from the CPT all over the world (e.g. Meyerdijf 1956, Velloso 1959, Danziger and Velloso 1995). Moreover, some authors have collected existing correlations from different places (e.g. Sanglerat 1972, Robertson et al 1983). Most of the correlations, however, do not take into account the differences from country to country (and even inside one country) related to the energy delivered to the SPT drive rods, which have been found since the late 70's and early 80's to be the most important factor related to the measured blow count (e.g. Schmertmann and Palacios 1979, Robertson et al 1983, Skempton 1986). Even when the energy ratio is taken into account in some way, the variation of energy delivered to the rods due to the drive rods length is generally neglected.

Some correlations have been proposed for Brazilian soils in Rio de Janeiro and São Paulo (e.g. Costa Nunes and Fonseca 1959, Alencar 1980, Menezes et al 1991, Danziger & Velloso 1995). However, just a few includes residual soils.

3 SOME CHARACTERISTICS OF RESIDUAL SOILS

The action of intensive in situ weathering of rocks results in a profile of weathering composed of three elements (Vargas 1953, Sandroni 1985):

(i) a mature soil in which all traces of the macrofabric of the parent rock have been destroyed;
(ii) a young soil in which to a greater or lesser extent the macrofabric of the parent rock subsists;  
(iii) an altered rock (or transition to a sound rock).
Residual soils occur to some degree in most countries of the world. The greater areas and depths are normally found in tropical humid areas, such as Brazil, Ghana, Malaysia, Nigeria, southern India, Sri Lanka, Singapore and the Philippines (Brand and Phillipson 1983).

The term residual soil itself is not a consensus throughout the world and not even in Brazil. In fact, Brand & Phillipson (1983) have gathered definitions from different countries and found that differences in the meaning of the term residual soil do exist. As an example, there are some authors that consider colluvium soils on the top of in situ weathered soils also as residual soils. Moreover, some (e.g. Sandroni 1985) consider as residual soil only the young soil where the macrofabric of the parent rock is kept - also named saprolitic soil - and the mature soil, also known as lateric soil (e.g. Vargas 1988) where the macrofabric have been destroyed. In the present paper, and also following Deere & Patton (1971) both soils will be classified as residual soils, namely mature residual soil, or lateric soil, and young residual soil, or saprolitic soil. The transition between the young residual soil and the weathered rock is many times difficult to characterize. Deere & Patton (1971) suggest that the saprolitic soil can be defined as having less than 10% by volume of corestones. Sandroni (1985) states that rock in accordance with engineering usage is a material not penetrable with SPT or almost not penetrable with wash-boring equipment, or which cannot be excavated without the aid of explosives.

It must be pointed out that in a Brazilian SPT a soil is generally classified as residual when there are no doubts concerning its origin. Since the macrofabric of the parent rock is a clear identification of the origin of the soil, most soils classified as residual soils are young residual soils.

Residual soils are generally well graded, as shown by Vargas (1953) for different kinds of parent rocks. This characteristic results in difficulties regarding soil classification according to particle size distribution, as sand, silt and clay are present in many residual soils in similar proportions. Moreover, classifying a residual soil solely by particle size distribution is not sufficient to represent its complex characteristics.

Mass heterogeneity is a typical characteristic of residual soils. However, as pointed out by Sandroni (1985), it has a different connotation with respect to the sedimentary soils. A sedimentary deposit is called heterogeneous if layers of different characteristics exist in the mass. The term erratic is used if the contrast is not between layers but if there are pockets, scours, etc. In residual soils it is common that the variability of a certain property in a restricted portion of the mass (like a block sample) is similar to the variability of this same property in the whole mass. Vargas (1970), quoted by Sandroni (1983), stated that “residual soils are uniformly heterogeneous”.

Water table is in many cases deep in the profile, hence the soils are generally partially saturated. Penetrability is an advantage of the SPT in respect to the CPT in residual soils. CPT generally needs friction reducers, and bent of rods sometimes occur.

4 PREVIOUS CORRELATIONS BETWEEN CPT AND SPT FOR RESIDUAL SOILS

There are very few publications relating correlations between CPT and SPT for residual soils, and Table 1 summarises most of the available data. Care must be taken when comparing data from Table 1 due to 2 reasons: the first one is the lack of information regarding the energy delivered to the drive rods during SPT; the second is that just mean values of qcu/N are included in the table, and the scatter can be significant.

In fact, Ajayi & Balegum (1988) presented data for Nigerian soils where significant scatter was found, even for the same soil type. Although scatter was significant in all cases, average values of qcu/N were in a narrow range, 3.2-4.5.

Chang (1988) has reported CPT-SPT correlations with much less scatter than the ones obtained by Ajayi & Balegum (1988). Although this may be attributed to less available data, it may have also happened due to less horizontal inhomogeneity.

5 ANALYSIS OF THE DATABASE

The available database are mainly from 15 sites in the Rio Grande do Sul State, 14 where mechanical CPTs were carried out as a complementary tool for the design of piles. The 15th is a test site owned by the Rio Grande do Sul Imerpower Company (CEIEF) also used by the Federal Univ. of Rio Grande do Sul, UFRGS (e.g. Cudnami 1994, Schnaid et al 1995, Averbeck 1998).

Data from 2 sites in Rio de Janeiro and 1 site in Sao Paulo are also included in the database. The Rio de Janeiro sites are the Catholic University, PLU-Rio test site (Surb 1986, Rocha Filho & Cavalc 1988, Áboze 1995) and an experimental site for testing transmission line tower foundations at Adriâpolis owned by Light Electrical Services (Barata et al 1978, Dansiger 1983). The data from the site in Sao Paulo were reported by Barata et al (1970).

No site analyses are available for the data from 14 sites in Rio Grande do Sul and the Sao Paulo data, and soil classification was only obtained by inspection of SPT samples. Actually, the classification of
residual soils as fine as grain size distribution is concerned is a difficult task, due to the mentioned well graded soil characteristic and to special features generally included in the classification, as illustrated in Table 2.

The samples from SPT have provided a large number of soil types, grouped according to Table 2. Some tests have not been included in the analysis because classification did not allow any idea about the particle size distribution.

For each adopted soil type, the mean value of q/N was evaluated. When one soil type was relative of more than one parent rock, the analyses were carried out for the whole data for each parent rock.

The values of the q/N ratio correspond to intervals of one meter, once the SPT is performed every meter in Brazil. Since the SPT is carried out in a length of 30 cm after a seat of 15 cm, and that penetration starts at a full meter, i.e. SPT blow count corresponds to 1.15 m-1.45 m, 2.15 m-2.45 m and so on, values of q, at a full meter plus 0.3 m were taken for the analysis. The obtained values are shown in Table 3 in the all data columns, together with the standard deviation (in brackets).

After these first calculations, some data were eliminated by considering only the data inside plus and minus one standard deviation. This procedure resulted in a partial data analysis, included in the partial data columns, which was also used for the remaining analysis. The data are relative to the Brazilian SPT, i.e. with no corrections as far as rod energy ratio is concerned.

Next calculations take into account the rod energy ratio, however not considering the energy reduction relative to the rod length less than about 10 m, as theoretically shown by Schuerenmann and Palevski.
(1979). An energy ratio of about 75% was considered in the analysis, slightly different from Décout et al’s (1989) suggestion, but consistent with the lighter anvil considered (Muxfeldt 1997). These values are included in Table 3 named no length effect.

The energy ratio decrease due to the drive rods length can be different for very shallow depths, if the values from the theoretical curves by Schermerhorn and Palacios (1979), or the suggestion by Skempton (1986) are considered. Two calculations were carried out, one with the theoretical values taken from a table by Décout (1989) and another with Skempton’s (1986) suggestion, extrapolating the correction factor of 0.75 for 3-4 m also for 1-2 m. This is actually one subject that deserves more research, since the final energy ratio can vary by a factor of almost 3, and is particularly important for the design of shallow foundations. Both calculations are included in Table 3. Mechanical and electrical CPT’s have been indistinctly analysed since no data were found relating \( q_c \) in residual soils from both types of penetrometers. Some comments and conclusions can be drawn from Tables 2 and 3.

Only one sample was classified as sand. Actually, pure sand is very seldom found in residual soils.

i) Only one sample was classified as sand. Actually, pure sand is very seldom found in residual soils.

ii) Obviously, the standard deviation has reduced from the all data analysis to the partial data analysis. The subsequent reduction from the Brazilian N to N\(_{sp} \) is only due to the corresponding energy correction.

iii) The remaining 2 columns refer to energy correction in N due to length effect. It can be seen that the standard deviation in all cases has increased, in many cases significantly, when the theoretical values were introduced. As far as Skempton’s (1986) suggestion is concerned, the values of standard deviation have been about the same as the obtained values for no length effect. Hence, recognizing the importance of the length effect, the last column of Table 3 has been used for the preparation of Figure 1. Although this analysis was not available for most cases as mentioned before, a qualitative classification similar to the one presented by Dangert & Velhos (1995) is shown in Figure 1.

It can be seen from Figure 1 that:

i) Different kinds of parent rock have produced similar values of q\(_c\)/N\(_{sp} \) for silty sands. However, in the other cases quite different values of q\(_c\)/N\(_{sp} \) were
generally obtained for the same soil type. Therefore, it seems that correlations for each soil type and each parent rock must be always considered.

ii) There is a general trend of decreasing values of \( q/N_0 \) when the mean grain size decreases, as generally found for sedimentary soils. However, unexpected results are sometimes found. For instance, clayey silty sands have provided much lower values of \( q/N_0 \) than clayey sands. Moreover, silty clays have provided unexpected high values of \( q/N_0 \), and sandy clayey silt from basalt have provided too low values of \( q/N_0 \). These 2 values can be seen with caution, since they were obtained from a small number of data pairs (see Table 3). This discrepancy can also be attributed to the chosen classification, not able to properly represent the soil tested. Specifically in the case of silty clay, the clay fraction may have had the behaviour of sand, packed together and cemented.

Table 3 also includes data from a test site at Porto, Portugal (Viana da Fonseca 1996), a well documented site, allowing a similar analysis to the one carried out for the Brazilian data. According to Viana da Fonseca (1996), the Dando equipment delivers about 60% of the theoretical free fall energy to the drive rods, i.e., it does not need any correction for obtaining \( N_0 \) in the case of long rods. Hence, just length correction was carried out. It can be seen from Table 3 that the silty sand from the Porto granite has provided higher values of \( q/N \) than the Brazilian corresponding data.

For 4 test sites (Table 4) the particle size distribution is available. The corresponding range of \( D_{50} \), together with the range of \( q/N_0 \), is shown in Figure 2 (from Robertson et al. 1983). It can be seen that this approach is not able to properly represent the behaviour of residual soils, even the trend of the mean values. It seems that other parameters need also to be included in the analysis.

### Table 4. Test Sites

<table>
<thead>
<tr>
<th>Test site</th>
<th>Parent rock</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adramatopolis, RJ</td>
<td>gneiss</td>
<td>Baraki et al. (1978), Dauplais (1983)</td>
</tr>
<tr>
<td>Porto, Portugal</td>
<td>granite</td>
<td>Viana da Fonseca (1996)</td>
</tr>
</tbody>
</table>

Fig. 1. Values of \( q/N_0 \) for different types of residual soil.

Fig. 2. \( q/N \) versus \( D_{50} \) (Robertson et al. 1983).
6 CONCLUSIONS

Correlations between CPT and SPT for different types of residual soils from different parent rocks have been established. The correlations presented a large scatter, mainly due to intrinsic heterogeneity of residual soils. Different parent rocks generally produced different correlations for the same soil type as far as particle size distribution is concerned. A general trend of decreasing values of q/N with the decrease of grain size was found, but the approach of correlating q/N with D_50 did not provide good results.

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A more rational utilization of some old in situ tests

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ABSTRACT: SPT and Plate Load Test are among the most used in situ tests, at least as far as foundation design is concerned. This paper presents improved versions of these old tests which were lately used in our professional routine practice. The SPT has been replaced by the SPT-T which is the SPT complemented by torque measurements. Notwithstanding costing about the same, the reliability of the SPT-T is many folds higher than that of the traditional SPT. The conventional Plate Load Tests are being replaced by similar tests on mini piles, 5" (12.7cm) in diameter, carried out either above or below the water level, within a 6' (15.24cm) in diameter cased hole.

1. SPT - A BRIEF RETROSPECT

The predecessor of the SPT was introduced early in this century, 1902, by GwG. The sampler, as it is used today, in the work of Mohr and Fletcher, 1927. The first attempt for the standardization of this test happened in 1939 by Fletcher and Mohr, but its popularity became evident only after the publication of the book Soil Mechanics in Engineering Practice by Terzaghi and Peck, 1948. Along this almost a century of existence three State of the Art Reports have been presented, de Mello (1971), Nixon (1982) and Décourt (1989). Some very good reviews should also be remembered like for example Skempton (1986) and Stroud (1988).

The most important single factor affecting the energy that effectively reaches the sampler and therefore the N values has been properly analyzed among others, by Schustermann and Palacios (1975) and Kovacs (1994). In Brazil this subject has been analyzed principally by Beltranta (1985-1998), Décourt et al (1989). It is also worthwhile to mention that an ISSMFE Committee chaired by S. Thorburn, finally succeed in elaborating an International Reference Procedure for the SPT, Décourt et al (1988).

It must be recognized that in these papers almost all the factors affecting the penetration resistance have been recognized and properly analyzed, leaving little to be done in the future, except, perhaps, the analysis of the behavior of sands in calibration chambers which, notwithstanding being highly recommended, hasn't been done yet.

2. THE SPT-T, THE ULTIMATE IMPROVEMENT OF THE SPT

Ranisz (1988) suggested that torque measurements might be added to SPT in order to obtain more reliable values of the adhesion between piles and soil, using the adhesion between the SPT sampler and the soil as an intermediate step. But the first SPTs with torque measurement were carried out only in early 1991 by Engesolsos, the first results being reported by Décourt and Queirazan Filho (1991). This new test was denominated SPT-T, by these authors.

Since that time, the author has no more used the SPT in his design jobs, using instead only the new SPT-T. In Brazil there are presently two trends as far as the practical utilization of SPT-T data is concerned. The pioneer work of Décourt and Queirazan Filho (1991-1994) has been towards a better understanding of the soil behavior through the determination of the “static” and therefore much more reliable T values, as compared with the “dynamic” Nₜₚ values. Even the identification of the collapsibility of soils may be easily made through comparison between T and Nₜₚ, what is an application of this test formerly hardly believed to be possible. The updating of all the formerly existent correlations via the Nₑq concept, to be further defined, was also considered to be a major concern. The other
trend limits the use of this test for pile design only, Almeida (1995).

3. EXECUTIVE PROCEDURE

Upon the conclusion of the penetration of the sampler into the ground (the \( N_{TPP} \) determination) the soil is removed and an adapter is placed. The idea is solely to create a stable type connection in order that a torque moment could be applied, using common torque meters. The maximum value of this moment, designated by \( T \), is registered. In Brazil, where this moment is read in kgf cm units, the order of magnitude of \( T \) and \( N_{TPP} \) turns to be the same. It is important to observe that the value of \( T \) is always referred to a 45 cm penetration of the sampler into the ground. Sometimes the residual value of \( T \) is also recorded.

4. THE TORQUE RATIO

Since long ago it has been said that the intricate properties of soils, specially those presenting a cohesive-frictional behavior, could not be assessed via a single parameter, in this case the penetration resistance \( N \). The SPT-T provides two independent measurements of the soil resistance and what is of paramount importance, practically in the same place.

The SPT is well known to be a highly disrupting test. In structured soils it is therefore reasonable to suppose that it will measure the behavior of a remolded soil rather than that of an almost intact one. The torque \( T \) is a measurement of the tortional effort required to overcome the friction between the sampler, driven 45 cm into the ground, and the soil. It is reasonable to suppose that the region where this side resistance is measured is much less disturbed by the penetration of the sampler into the ground than the region just below its tip.

For this reason, it seems reasonable to suppose that the ratio \( T/N \) would probably be an indirect measure of the soil structure. The ratio of \( T \) by \( N \) was called Torque Ratio (T/N).

5. A NEW SOIL CLASSIFICATION.

The knowledge of both \( T \) and \( N \) values of the SPT-T provides the basis for a new soil classification, differently from the formerly extant classification systems, which are based on remolded soil properties, this classification takes into account the soil structure. The first soils investigated were those belonging to the Tertiary Sedimentary Basin of São Paulo (TSBSP) which are supposed to be little sensitive. These soils are typically sandy silty clays and silty clayey sands.

For this soils the average value of the torque ratio was found to be about 1.2.

The same order of magnitude was also initially found for the torque ratio of sedimentary over consolidated sands of this basin, known as basal sands. For the residual soils of the city of São Paulo, which are predominantly derived from granitic, granulitic and migmatitic rocks, the average value of T/N is about 2.0.

For the unsaturated collapsible clays of the city of São Paulo (av. Paulista region) this ratio is about 2.5. For clayey collapsible soils of the interface of São Paulo T/N is 2.5 or even higher, approaching sometimes 5.0.

For medium sensitive saturated soft clays from the quaternary period, largely found along the littoral of the state of São Paulo this ratio is about 3 to 4.

These information renowned the knowledge available up to four years ago and a table summarizing all these data was presented by Décourt and Quaresma Filho (1994). This same table, updated is presented in fig. 1.

Fig. 1 Soil classification on the basis of the torque ratio (T/N)

6. THE EQUIVALENT N CONCEPT (Neq).

Most of the correlations often used in Brazil have been established for the soils of the Tertiary Sedimentary Basin of São Paulo (TSBSP). For these soils and for \( N_{TPP} \) values neither too high nor too low it was found that the \( T \)-values, measured in kgf cm units, were, on average, 1.2 times the \( N_{TPP} \) values.

Décourt (1991b) postulated that even for soils outside the TSBSP the correlations established for the soils of the TSBSP could be used, provided \( Neq \) values were used instead of the directly measured \( N_{TPP} \) values.

The \( Neq \) was defined as the torque \( T \) measured in kgf cm divided by 1.2.

\[ Neq = \frac{T}{1.2}. \]
In other words, what is being postulated is that it is the torque $T$ rather than $N_{SP}$ that is effectively related to the soil behavior.

7. SPT-T IN SEDIMENTARY SANDS

7.1 THE INFLUENCE OF THE MAGNITUDE OF $N_{SP}$ VALUES.

More recently, a certain amount of $T$ and $N_{SP}$ data on sedimentary sands became available. These soils, originally considered to be of a simple behavior, are in reality the most complex ones, at least as far as torque ratios are concerned. Apparently similar soils may present $T/N$ values ranging from as low values as 0.30 to as high values as 10, what means a more than 30 folds difference.

At this very moment, there isn’t any definitive explanation for this broad range of values of the Torque Ratio.

Bélincourt (1996) followed by Décourt (1996) pointed out that the torque ratio was not a constant as have been previously assumed but it is rather a function of the $N_{SP}$ value.

This observation is particularly important in the case of sedimentary sands. To check the correctness of this observation the results of tests in four sites are presented in Figure 2. Three of them were along the shore of São Paulo state (Santos, Hortolândia and São Sebastião) and the fourth in Mogi das Cruzes, 40 kilometers from the shore. All samples were classified as fine silty sands.

The tendency for decreasing the torque ratio values and therefore $N_{SP}/N$ with the increase of $N_{SP}$ was clearly evidenced.

For comparison purposes, the correction of the $N_{SP}$ values proposed in the first edition of Soil Mechanics in Engineering Practice (1948) was also plotted. It must be stressed that for allowing fair comparisons to be made, the original Terzaghi and Peck correction was adapted for the much more efficient Brazilian SPT ($E_s = 75$ kPa) as compared to the old candle American SPT ($E_s = 46$ kPa - 50 kPa) which probably was the one considered in that book

$$N_{SR} = N_{SP} / 1.5 \quad (N_{SR})_{min} = 10 + (N_{SR} - 10)/2$$

Just for sake of curiosity this expression was also extended to low values of $N_{SP}$, what definitely was not intended to be done by Terzaghi and Peck. These authors are to be commended for their fantastic feeling. Fifty years ago they predicted a tendency very

![Figure 2 Variation of $N_{SP}/N_{SR}$ and $N_{SP}/N_{SR}$ with $N_{SR}$](image1)

![Figure 3 Variation of $N_{SP}/N_{SR}$ and $N_{SP}/N_{SR}$ with $N_{SR}$](image2)
similar to that presented in figures 2 and 3. By the
way, unfortunately this correction was omitted in the

A similar result was obtained for the basal sands of the
TSHSP and is presented in figure 3.

All these data clearly suggest that the direct use of
unconnected N_T values in sands leads to unreliable
conclusions, varying from conservative to very
conservative for low N_T values, to unsafe, for higher
N_T values.

The unreliability of the low N_T values was also
pointed out by Stroud (1988).

7.2 THE INFLUENCE OF THE STATE OF
CONSOLIDATION.

It is postulated that the state of consolidation of sandy
sediments may also have an important influence in the
torque ratio values. A tendency seems to exist of
higher Torque Ratio values for higher over
consolidation ratios, however definitive conclusions
would only be possible upon carrying out comparative
tests in calibration chambers.

8. LOAD TESTS ON MINI PLATES; EXECUTION
PROCEDURE

These tests have been performed on circular steel
plates with 5" (12.7 cm) in diameter inside a 6"
(15.2cm) diameter casing. The presence of the water
level is not a restriction for carrying out these tests
what represents a major advantage over the former
plate load tests.

For applying loads two systems are in common use.
When pressures up to 500 kPa are sufficient for
defining the foundation rigidity the loads are applied
using weights, like for instance the weights used in
odometer tests. When higher loads are required, jacks
have to be used. More details about these procedures
may be found in Décourt and Quaresma Filho (1996).

9. UNIQUENESS OF THE NORMALIZED
STRESS-SETTLEMENT CURVE IN A LOAD
TEST.

The pioneer work of Briaud and Jeanjean (1994),
followed by Décourt (1994-1995) have demonstrated
that the stress-settlement curve for shallow
foundations is unique, provided the stresses were
normalized by the conventional failure stresses and the
settlements, were normalized by the equivalent widths
of the foundations. The conventional failure stress is
declared as the stress corresponding in the load-test, to
a settlement of 10% of the equivalent width of the
foundation (N_0). The equivalent width of the
foundation is the width of a square foundation with the
same area of the foundation being tested.

10. THE BASIC CURVE

Since four years ago a simple but very reliable method
developed by the author for predicting settlements has
been tested. The soil response is recognized to be
highly nonlinear.

A law of variation of the settlements normalized by
the equivalent width of the foundation with the stresses,
normalized by the stress corresponding to the
conventional bearing capacity (q_u) has been established.

Some examples of the successful application of the
basic curve for predicting settlements may be found in

For sands, q_u in kgf/cm^2 units (kPa x 10^3) may be
assessed as a function of T, in kgf/cm units.

shallow foundations ............... \( q_u \approx T \)

This formula is an extension of the originally proposed
by Décourt (1991a) \( q_u \approx 1.2N \), considering the
recommended use of N_q derived from T instead of N
and that N_q = T/1.2 as shown in item 6.

For deep foundations like deep plates and non
displacement piles with, D/B \( \geq 5 \), one has:

\[ \text{deep foundations} \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \]

\[ q_u = 1.5 \ T \]

For intermediate embedments (D/B) the rate of
increase \( q_u \) with D/B is given by:

\[ \log F = 0.09 + 0.125 \log D/B \]

11. A PRACTICAL EXAMPLE.

One of the most problematic soils found in Brazil has
been chosen to illustrate the potential use of these
tests. It is a silty sand, from the Cenozoic period, that
probably hasn't been pre-stressed and that is well
known to be a badly collapsible material. This soil
covers an important part of the inland of state of
São Paulo, including the city of Barueri where the
Estadual University of São Paulo has a Foundation
Experiment Field.

Agelli (1992) performed in the area a number of
conventional plate load tests (steel circular plates with
80.5 cm in diameter). The results of these tests may be
found in Agelli (1992) and Agelli and Aliberti
(1994).
In order that comparisons between the results of load tests performed conventionally be made with the results of load test carried out with mini plates, six of these later tests were performed in this same soil, very close to where the tests by Agnelli (1992) were carried out. Moreover, considering that it is fundamental to perform SPT-Ts instead of SPTs, what hasn’t been done by Agnelli, three SPT-Ts have been carried out in the area by Engesols, very close to where the load tests were performed.

The results of these tests are presented in figure 4. It is clear from these test results that there are two distinct regions, one for depths from zero to about 12.0 m and another for depths greater than about 12.0 m. It may be observed that all strength measurements like $N_{eq}$ and $T$ increase approximately linearly with depth, showing a typical Gibson soil behavior. Also shown are the regressions established for depths from zero to 12.0 m. The Torque Ratio ($T/N$) is on average 0.71.

In figure 5 the results of the twelve load tests are presented. The stresses normalized by the conventional failure stresses and the settlements, normalized by the equivalent widths of the plates ($B_{eq} = \sqrt{A}$) are presented.

For the Agnelli tests, the failure loads were obtained through extrapolation procedures, using statistical regressions that presented very high values of $R^2$.

For predicting the rigidity of the foundations on the basis of $T$ and $N$ values of the SPT-T, it is suggested that the representative values of the resistance parameters of these tests were those corresponding to a depth of 0.7 $B_{eq}$ below the plate level. This assumption is based on previous comparative analyses carried out by the author.

12. CONCLUSIONS

Significant improvements made in the SPT and in Plate Load Tests allow a much better understanding of the soil behavior, without introducing increase in costs.

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Summary of Standard Penetration Test (SPT) energy measurement experience

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ABSTRACT: The purpose of this paper is to review experience with SPT energy measurements. SPT data are important for determination of most all engineering properties in sands and are especially critical in determining earthquake liquefaction resistance. It is common practice to normalize SPT data to 60% drill rod energy delivery. Measurements have been performed since the late 1970's. Early SPT energy measurements were taken using force transducers in the drill string. These methods were standardized in the 1980's in ASTM D-4633. In the 1990's new instrumentation allows for measurement of acceleration on the drill string. New measurements are being performed by integration of the product of force and velocity. These are sometimes significant differences in the two methods. This paper will evaluate any differences in the energy measurement methods, and will make recommendations for correction of penetration resistance data. Practitioners currently are revising the ASTM standard. The paper will also make recommendations on future standardization. For some equipment and systems which are operated correctly, assumed energy transmission values can be used.

1 INTRODUCTION

A new method for measuring energy was developed in the 1990's. This method allows for measurement of acceleration on the drill string. Data is being collected and evaluated. The older force measurement data differs from the new force velocity approach in many cases. These differences may affect normalization methods. This paper will review the problems with both methods and make recommendations for their future use.

2 FORCE SQUARED (F2) METHOD

Schumer and Paliacas (1977) explored the F2 method in 1977. For perfect one dimensional wave prolongation, the energy consists of equal components of strain and kinetic energy. Efforts to use accelerometers in those days were not successful, so they integrated the square of the force. They used strain gage load cells placed at both the top and bottom of the drill rods. An extensive series of measurements with differing drill rods, and hammers to depths of 75 ft. were conducted. They found that the majority of sampler penetration occurred during the first wave pulse. This was fortunate, because the F2 method cannot be applied past the tensile wave return. After a study of energy transfer, they concluded that the energy measured in the top cell was inversely related to SPT N values. A wide variation in energy for differing hammer systems was discovered.

The F2 method is defined as follows:

\[ E_2 = \frac{cK}{AE} \int (F(t))^2 dt \]  \hspace{1cm} (1)

where:
- \( A \) = cross-sectional area of the drill rods above and below the force transducer,
- \( c \) = velocity of the compression wave in the drill rods = \( (\alpha p)^2 \), approximately 5120 m/s for steel,
- \( E \) = modulus of elasticity of the drill rods,
- \( E_t \) = maximum energy transmitted to the drill rod during the impact event,
- \( F(t) \) = dynamic force in the drill rod as a function of time.
Figure 1 - Summary of ER, Data for Safety Hammers by Kovacs (1983)

\[ K_c = \frac{E_r}{E_n} \]

where \( E_r \) is the ratio of energy expressed as a percent of \( E_n \) and \( E_n \) is the nominal kinetic energy for the SPT.

At shallow depths, \( E_r \) is lower because the hammer input energy is terminated by the reflected wave. This trend is confirmed by observation of \( N \) values at shallow depths in homogeneous materials. In order to correct \( E_r \) to a nominal value, the factor \( K_c \) is used.

3 SUMMARY OF PRACTICE - F2 METHOD

Measurements were continued through the 1980’s using the F2 method. Most of the data collected in North America was procured with a Binary Instruments device developed by John Hall. Kovacs, (1983) summarized data collected in the early 1980’s. Kovacs compared Binary Instruments data to analog data and verified the validity of the Binary Instruments device. He discovered that under hard driving conditions, the Binary Instruments device would integrate portions of a reflected compression wave, almost doubling the true energy content. Timers were obtained and Kovacs observed that the true cutoff time to zero load was often longer than theoretical and he proposed a correction factor, \( K_c \).

Today it is recognized that the \( K_c \) factor is no longer necessary. Test method D 4633 was passed in ASTME (1985) which included this correction. Energy measurements were also conducted by Kovacs (1984) in Japan due to the importance of obtaining energy adjustments when performing liquefaction investigations.

Summaries of the findings of Kovacs are shown on figures 1 through 3. Data on the figures is corrected for the \( K_c \) factor of about 0.95. Examination of the data for safety hammers indicates a very wide variation in \( E_r \). Data for donut hammers revealed lower energies averaging 55%.

It should also be noted, that in the early 1980’s, strain gage load cells were being replaced by piezoelectric load cells. Most of the data for figure 3 was collected with piezoelectric load cells except for the data for one wrap which was collected with Senotec strain gage load cell. Close examination of the field measurement data indicates that the piezoelectric load...
cell ER, ranged from 5 to 10% higher than strain gage cell ER, for the same equipment. This apparent difference in instrumentation has never been systematically examined.

In 1984 Dr. H. Boltoon Seed recommended correction of SPT data to 60% ER. Although some safety hammer data is higher than this value, it would be conservative to assume U.S. practice safety hammers deliver 60% ER. Since this time, it is common practice in the U.S. to assume safety hammers deliver 60% drill rod energy.

In the late 1980's I had the opportunity to use the binary Instruments device and piezoelectric load cell on numerous hammer systems. Findings from this study indicated that when using large NW rod, there were numerous early zero load cutoffs. The problem was alleviated with use of smaller AW rod. NW rod energy data, with early cutoff time was generally within 3 to 5% of data with correct cutoff times.

Through the 1980's numerous measurements were made on new automatic hammers (Riggs 1984). Although some of this data was confirmed to suffer from excessive travel times, many more measurements confirmed that the hammer system was very efficient (Farrar 1990). Based on these studies, it was assumed that a CME automatic hammer operated at 55 blows per minute, would deliver a drill rod energy ratio of ER = 95%.

4 FORCE VELOCITY (FV) METHOD

The force velocity method is defined as:

$$ E_F = \int F(t) v(t) dt $$

Where:

- $E_F$ = the maximum energy transmitted to the drill rod during the impact event,
- $F(t)$ = the force measured by the instrumented drill rod as a function of time,
- $v(t)$ = the particle velocity measured in the instrumented rod as a function of time, and
- $\int$ = the time interval of interest.

A typical force velocity trace is shown on figure 4. Proponents of the FV method state the advantages of this methods are, 1) The integration is through the complete waveform capturing complete energy in the system, and 2) If proportionaltiy is not present in the first 20% time interval, the F2 method assumptions are not correct and integration of FV gives the correct energy of the system.

Successful FV energy measurements were performed by Dr. George Goble in 1990. Through the 1990's Goble, Rausche, Likins, and Associates (GRL) performed numerous tests for several agencies (Goble (1993), Botchelor (1995), Miner (1995)). Those tests
were performed with Pile Dynamics Inc. pile driving analyzers, with an instrumented drill rod sub-assembly containing two strain gages, and two accelerometers. Either piezoelectric (3,000 g, 7 kHz) or piezoresistive (10,000 g, 3 kHz, 5 kHz damped) accelerometers are used. Various types of PDI analyzers have been used. Currently, it is preferred to use a PDI PAK analyzer with a sampling rate of 15 to 20 kHz with 12 bit A/D resolution. The drill rod assemblies are most often two foot long sections. These assemblies are calibrated using two methods to assure proper results. Results of GRL measurements and other comparison studies are given in Table 1.

The in situ testing group at the University of British Columbia with assistance from B.C. Hydro, also performed FV energy measurements. Alex Sy (1991) presented some comparison data of FV and F2 data collected with piezoelectric load cells and accelerometers sampled with a Nicolet digital oscilloscope sampled 15 bit resolution and 0.01 ms intervals. He concluded FV data was typically larger than F2 methods by 5 to 15%.

In 1992 Dr. David Frost performed a comparison study of FV and F2 methods when measuring a Dietrich automatic hammer. He used a Binary Instruments device and a PDI pile driving analyzer. In these measurements FV data are typically lower than F2 data. There were problems with some of the F2 data due to reflected compressive wave integration.

Scott Jackson of Northwood Instruments developed a Dynamic Energy Monitor N100 to obtain FV and F2 data. This equipment uses piezo-resistive accelerometers, a sampling rate of 100 Hz, and an A/D resolution of 12 bits. Acceleration data are filtered at 48 Hz. Jackson (1995) compared FV and F2 data for Keeneyside Dam for B.C. Hydro. In this report, energy on safety hammers with the F2 method was lower than FV data by 6 to 9%. This lower energy in the F2 method was explained as being caused by reflections or loose joints. There were problems obtaining good acceleration data on an automatic hammer system - due to very high acceleration and high frequency associated with that system. Measurements were taken to depths of 25 meters. Ken Luan (1997) of B.C Hydro reports that one of three other projects F2 data was 2 to 12% higher than FV on one project and was 2 to 8% lower on two other projects. The reason for these differences are not known.

Rich Lamb, of the Minnesota Department of Transportation, performed energy measurements using a PDI analyzer. He found that one CME automatic hammer, when operated too fast delivered lower energy. This finding has been confirmed by CME which states that if the hammer is operated at rates exceeding 58 blows per minute, the chain can hit the hammer on the downstroke, interrupting free fall. Lamb found that the F2 data collected was up to 35% lower than FV data. This was apparently due to the use of incorrect area assumption for the drill rod. However, F2 data can be corrected for incorrect area, even after the fact. Lamb used an instrumented sub-assembly provided by PDI.

Recently, GRL Associates funded a study of their recent data which was performed by Joshua Butler (1997) of Utah State University. The goal was to examine the differences between F2 and FV data. This study statistically analyzed the complete GRL/PDI data, most of which is summarized in Table 1. The findings of this study are:

- F2 data is lower than FV data for short drill rod lengths and requires correction with $K_e$ factor.
- Loose connectors or reflections from rod joints can cause a loss of proportionality in F and V and result in high F2 data.
- For N>50 blows, F2 data can be abnormally high.
- For N<10 blows, FV data over predicts energy transfer due to additional hammer impact.
- For long rods, normal blow counts, the F2 and FV methods agree rather well.
- Depending on the combination of rod length, proportionality, and blow count range, F2 and FV methods can differ as much as 15%.
- For cases of short rods, poor proportionality, or high blow counts, the F2 method can give results from 2 to 15% higher than the FV method.

5 REVIEW COMMENTS

Review of the data in Table 1 indicates a wide variation in the differences of the F2 and FV data. A large majority of GRL data is collected with short rod lengths which result in more complex wave forms to analyze. Also, GRL F2 data is not integrated correctly. In their analysis they integrate over the time period 21/c instead of to the zero load cutoff time. For good wave forms, zero load cutoff occurs slightly later than later than 21/c and their F2 data could be low for smaller rods. In smaller rods, energy
Table 1 - Summary of Recent Energy Measurements Using the FV and F2 Methods.

<table>
<thead>
<tr>
<th>Date/Author/Agency</th>
<th>Series</th>
<th>Hammer Type</th>
<th>Bed Type</th>
<th>Hammer Rate</th>
<th>Avg ER, FV</th>
<th>Avg ER, F2</th>
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</thead>
<tbody>
<tr>
<td>1990, Geble CDOT-USBRR</td>
<td>CME Automatic</td>
<td>AW</td>
<td>86</td>
<td>85</td>
<td></td>
<td></td>
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<tr>
<td>1991, Sp, UBC study</td>
<td>Safety Hammer NW guide</td>
<td>AW</td>
<td>60</td>
<td>51</td>
<td></td>
<td></td>
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<td>1992, Frost, Diedrich Drill</td>
<td>Diedrich Automatic</td>
<td>RWF</td>
<td>14</td>
<td>89</td>
<td>100*</td>
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<td>1992, Frost, Diedrich Drill</td>
<td>Diedrich Automatic</td>
<td>AW</td>
<td>14</td>
<td>64</td>
<td>85</td>
<td></td>
</tr>
<tr>
<td>1993, GBL Texas A&amp;M</td>
<td>Safety Hammer</td>
<td>N</td>
<td>48</td>
<td>50</td>
<td>51</td>
<td></td>
</tr>
<tr>
<td>1994, GBL ASCC Seattle</td>
<td>BK-81 Auto</td>
<td>AWJ</td>
<td>59</td>
<td>66</td>
<td>67</td>
<td></td>
</tr>
<tr>
<td>1994, GBL ASCC Seattle</td>
<td>A-2</td>
<td>Safety Hammer</td>
<td>BW</td>
<td>53</td>
<td>51</td>
<td>56</td>
</tr>
<tr>
<td>1994, GBL ASCC Seattle</td>
<td>A-3</td>
<td>CME Auto</td>
<td>AWJ</td>
<td>51</td>
<td>81</td>
<td>81</td>
</tr>
<tr>
<td>1994, GBL ASCC Seattle</td>
<td>B-1</td>
<td>Safety Hammer Spooling Winch</td>
<td>NWJ</td>
<td>45</td>
<td>23</td>
<td>21</td>
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<tr>
<td>1994, GBL ASCC Seattle</td>
<td>B-6</td>
<td>CME Auto</td>
<td>AWJ</td>
<td>50</td>
<td>73</td>
<td>74</td>
</tr>
<tr>
<td>1995, GBL Oregon DOT</td>
<td>E-89-1</td>
<td>Safety</td>
<td>AW</td>
<td>21</td>
<td>82</td>
<td>61</td>
</tr>
<tr>
<td>1995, GBL Oregon DOT</td>
<td>B-83-1</td>
<td>Safety</td>
<td>BW</td>
<td>37</td>
<td>61</td>
<td>64</td>
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<tr>
<td>1995, GBL Oregon DOT</td>
<td>A-10-1</td>
<td>Safety</td>
<td>BW</td>
<td>27</td>
<td>78</td>
<td>82</td>
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<tr>
<td>1995, GBL Oregon DOT</td>
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<td>Safety</td>
<td>BW</td>
<td>39</td>
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<td>A-B7-1</td>
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<td>27</td>
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<td>1995, Jackson, B.C. Hydro</td>
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<tr>
<td>1995, GBL Oregon DOT</td>
<td>C-85-1</td>
<td>Mobile Auto</td>
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<td>1995, GBL Oregon DOT</td>
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<td>BW</td>
<td>58</td>
<td>82</td>
<td>95</td>
</tr>
<tr>
<td>1995, GBL Oregon DOT</td>
<td>B-84-1</td>
<td>CME Auto</td>
<td>BW</td>
<td>54</td>
<td>78</td>
<td>93</td>
</tr>
<tr>
<td>1995, GBL Oregon DOT</td>
<td>A-82-1</td>
<td>CME Auto</td>
<td>BW</td>
<td>56</td>
<td>76</td>
<td>118</td>
</tr>
<tr>
<td>1995, GBL Oregon DOT</td>
<td>A-81-1</td>
<td>CME Auto</td>
<td>BW</td>
<td>56</td>
<td>82</td>
<td>102</td>
</tr>
<tr>
<td>1997, Lamb, Minn. DOT</td>
<td>CME Auto</td>
<td>N</td>
<td>&gt;58</td>
<td>66</td>
<td></td>
<td></td>
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<tr>
<td>1997, Lamb, Minn. DOT</td>
<td>CME Auto</td>
<td>N</td>
<td>55</td>
<td>78</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1997, Lamb, Minn. DOT</td>
<td>CME Auto</td>
<td>N</td>
<td>55</td>
<td>86</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1997, Lamb, Minn. DOT</td>
<td>B给人 Spooling</td>
<td>N</td>
<td>72</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1997, Lamb, Minn. DOT</td>
<td>Mobile Spooling Winch</td>
<td>N</td>
<td>43</td>
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<td>1997, Lamb, Minn. DOT</td>
<td>Safety Hammers (3)</td>
<td>N</td>
<td>40-50</td>
<td>87</td>
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<td></td>
</tr>
</tbody>
</table>

* area probably wrong?

A wide variety in data instrumentation and calibration methods have been used. It is not clear whether the variations in equipment, sampling, and processing methods can explain differences in F2 and FV data.

The trends for automatic hammer system data are most disturbing. These systems, with very efficient hammers, have been assumed to deliver high energy according to the F2 method. This trend has been confirmed in numerous cases comparing N values of automatic hammers and rope and cable operated...
hammers. The new FV data implies that these hammers deliver less energy than assumed from previous F2 work. Both Jackson and GRL have noted difficulties in obtaining good acceleration data for the automatic hammers. Differences as much as 20% between F2 and FV data have been reported. This could be due to the fact that for these hammers, the measurement equipment is too close to the shot, solid, impact anvils. This causes all kinds of detrimental end effects, including higher frequency vibrations. Sometimes, data is reported for a hammer operating incorrectly. There should be a nominal energy for these autohammer hammers, when operated correctly. It should not be necessary to measure energy for these systems as often. If operated properly, a nominal energy could be assumed as long as rate and drop height are periodically confirmed. The GRL radar system can provide useful data in energy measurements and in periodic performance checks.

It appears that the F2 and FV data agree better with safety hammer systems where the measurement instruments are further from the impact surface.

The GRL conclusion that F2 data is higher than FV data due to reflection or loose rod joints is contrary to the data reported by Jackson. Jackson's data was taken with long rod lengths and AW rod. His reasoning for high FV data was loose joints, but it is doubtful the rods joints were always loose in the numerous tests performed at Keenleyside. The difficulty of obtaining data for larger NW rods using the F2 method was also reported in my earlier measurements. Undoubtedly, the NW rod with massive coupling reflective surfaces may be a problem in SPT testing.

6 CONCLUSIONS AND RECOMMENDATIONS

There is a wide variation in F2 and FV data to date. There is evidence that the F2 method may not be correct when proportionality is violated. This is especially true with NW rod or with loose couplings. Most people involved with these measurements believe that FV and F2 data should be collected to evaluate potential problems with F2 data. I concur with these recommendations.

It is apparent that additional controlled studies should be performed. Many of the existing measurements have been performed for clients requiring energy correction. Often the personnel performing these measurements have constrained budgets. Hopefully, research funding could be procured to do more detailed work.

The ASTM task group on energy measurements is working on a new standard for drill rod energy measurements. Currently ASTM D4633 is not the book of standards under the task group can determine the requirements for the new FV method and the differences between these methods. From the appearance of the differences in this report, it seems there are problems with both methods. It appears that there are significant differences in the methods which will require continued explanation. Some recommendations for future measurements include:

- Tighten up equipment tolerances and calibration requirements. It would be good to pick constant accelerometer type, sampling rates and resolutions, and filtering techniques. Calibration methods should be the same. Drill rod area must be known.
- Develop better guidance on waveform interpretation, especially the ability to detect unreliable data.
- Use long rod lengths to avoid complex waveforms which stress the equipment.
- Use smaller drill rods to avoid errors caused by reflections.
- Avoid testing in abnormally high or low blow count materials.
- On automatic hammers, locate the measurement equipment further from the impact surface.
- Strive to determine the nominal energy transmission of systems according to manufacturers instructions.
- For automatic hammers there should be more carefully controlled studies to resolve larger differences seen in these systems.

Nominal hammer energy should correspond to the significant portion of energy causing sampler penetration. Both wave equation and field data conclude that over 95% of sampler penetration occurs during the first major pulse of energy traveling to the sampler. Secondary impacts are much smaller and not of interest. The claim that F2 data does not always give complete nominal energy is not convincing because of this fact. If collected correctly, FV data integrated throughout the complete waveform, should exceed F2 data because small portions of energy from secondary and even tertiary impacts are collected. Engineers adjusting SPT data need an indicator which can be used to adjust N.
values - it is not our goal to determine the ultimate energy of the system.

When performing energy measurements using the F2 method, use of the Kf factor is not required as long as the measurement location is sufficient distance from the impact area. It is possible that the FV data collected near the impact surface for autohammers is low and may require correction similar to Kf, which was based on St. Venants principle. Use of the Kf factor is not required. Short cutoff time for larger rod systems may be acceptable because the tails of these waves are rather flat. Data should be inspected to assure reflected compressive waves are not integrated. Since the zero load cutoff is typically longer than 21\%
 of the data with cutoff times exceeding 21\% is acceptable. The measuring instruments should not cause area changes, and thus strain gages bonded to drill rod sections are preferred. In any F2 measurement, the area must be measured and cannot be assumed. Drill rod manufacturers vary the wall thickness and textbook values cannot be used. The rod section must be physically measured.

For those who use the energy data, the data must be carefully planned and reviewed. Your chances for better data can be improved by insisting on smaller rods with lengths exceeding 10 m. The basis of our current correction is in the earlier F2 database and in many cases the FV and F2 data will be close enough to feel comfortable with the adjustments. Engineers need to assure that rod joints are tightened and should be sure operators know the operating guidelines for hammer systems. For those who have short rods lengths and nominal energy data, the N data can be decreased by the inverse of the Kf factor. If you are going to measure energy with short drill rods and wish to make direct correlations, either the raw ER, from the F2 method, uncorrected for Kf, or FV data integrated over the 21\% time period can be used. Typical Kf factors can theoretically calculated for different drill rod types.

The use of smaller drill rod to depths up to 30 m is in accordance with a newly written ASTM standard on Normalized Penetration Resistance Testing of Sands (ASTM D-6066). There are currently studies underway of large long drill rods underway. It is quite possible that large N rod may be less efficient on energy transmission than assumed.

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Calibration of the blunt tipped dynamic penetrometer for silica sands

Stephen Fityus
The University of Newcastle, Department of Civil, Surveying and Environmental Engineering, Callaghan, N.S.W., Australia

ABSTRACT: A calibration chart is presented for a light weight dynamic penetrometer device. The blunt tipped “Perth Sand Penetrometer” is used extensively in Australia for characterisation of silica sand sites prior to the construction of light weight domestic structures. The hand operated device, which can be readily deployed in fine granular soils at depths up to 5 m, is routinely and successfully used to infer in situ density and internal friction angles in extensive coastal sand deposits. The charts presented here are based on the results of 18 calibration chamber tests, performed in a specially constructed chamber, 1m high and 1m in diameter. Tests were performed on silica sand samples consistently prepared to 3 densities using the sand raining or pluviation technique. The prepared samples were pressurised during testing, to simulate overburden effects due to burial at depths up to 5m. Particular account is taken of the effects of rod friction.

I. INTRODUCTION

The in-situ density of sand foundations is commonly assessed by penetrometer testing. Current international practice uses both static (pushed) and dynamic (driven) penetration devices. The most commonly employed dynamic method is the “Standard Penetrometer Test”, or SPT, which involves the driving of a split spoon sampler using a 63.5 kg. drop weight (Palmer et al., 1957). While such devices are useful tools for larger scale site investigations, and to depths of tens of metres, their applicability to scale projects is limited by the scale of the apparatus and its relative insensitivity at shallow depths.

Most of the larger cities in Australia are situated on the coast and many have extensive coastal deposits of clean, aeolian and alluvial sands, of Quaternary age. These are commonly exploited for residential developments comprising one and two storey residential structures and light traffic pavements. In many instances, the natural density of these deposits is spatially inconsistent, with relative densities from 30% to 80% commonly encountered within 1.5m of the ground surface. Also, many sand soil areas are low lying or dunal, and require reclamation and/or regrading.

Whether natural or regraded, there is a need to characterise the density of sand sites prior to the construction of roads, dwellings and light weight commercial and industrial structures. As the depth of interest is usually no more than about 3m, the expense of heavy, truck mounted equipment is seldom justified. In order to reliably and efficiently characterise sand sites, Australian geotechnical practice has adopted a light weight, hand operated dynamic penetrometer device, the Perth sand penetrometer (PSP).

With this device, sites as small as as single residential block can be thoroughly characterised to a depth of 2 or 3m by an unaided technician, in 1 or 2 hours, and with minimal equipment hire and running costs.

Despite its extensive use over the past 30 years, only one paper to date (Glick and Clegg, 1965) has attempted to provide a quantitative calibration of blow counts to in-situ density; and to depths of only 0.75m. More commonly the device is employed to assess compliance with empirically developed industry standards. For example, the Australian residential foundations code requires that a minimum of 7 blows per 300mm be achieved in sand fill up to 0.8m deep, to indicate an adequate foundation density for residential construction, (AS2870–1995). Many of the larger Australian consulting firms have developed their own in-house density calibrations, however these remain closely guarded and unpublished.

This paper presents a calibration chart for the Perth Sand Penetrometer in a silica sand. It relates the blow count, as a function of depth, to the relative density of shallow, non-soil layers.
2. THE PERTH SAND PENETROMETER

The Perth Sand Penetrometer is described in detail in Australian Standard AS1289.3.3 (1984). Its physical arrangement is summarised here, and illustrated in Figure 1. It consists of a 9kg sliding weight which delivers a measured quantity of energy by falling through a height of 600mm onto an anvil block. This energy is used to push a 16mm blunt ended steel rod into the ground. The steel rod is usually scribed at increments of 50mm and the results expressed as a number of blows required to drive the rod through a distance of 150mm.

The total mass of the device is less than 20kg, making it relatively portable. Raising and releasing of the weight is achieved by hand, with a certain amount of care required to ensure that:

- the weight is lifted through the full 600mm height,
- there is negligible impact on the upper stop at the top of the lift, and
- the weight is released cleanly and allowed to fall without interference.

The sounding rods are configured so that once driven, the hammer can be removed and additional rods added to enable testing to continue to depths of several metres. In the authors experience, maximum practical depths are in the order of 5 or 6m in exceptionally loose conditions. Beyond these depths, difficulties in the retrieval of rods and the risk of lost rods through damage become too great.

Australian practice also employs a variation of the PSP in the characterisation of pavement subgrades and mixed soils. The variation is known as the Dynamic Cone or Scaula Penetrometer (Scaula, 1956, Chiu, 1988, AS1289.3.2, 1984). Its principal differences are a falling weight distance of 510mm and a conical point with a maximum diameter of 20mm. The enlarged point is intended to reduce shaft adhesion in more cohesive soils. Whilst sometimes deployed in sands, it will not be considered further in this discussion.

3. CALIBRATION PROCEDURE

Calibration of the PSP was achieved using a purpose built calibration chamber at the University of Newcastle. The chamber was originally constructed to facilitate research into penetrometer calibration (Ajalleoian, 1996, Ajalleoian et al., 1997). A full description of its arrangement and sample preparation techniques are presented in Ajalleoian (1996). Only a brief summary of these details is presented here.

3.1 Calibration Soil

Because of the abundance of clean, poorly graded (well sorted) silica sands around the Australian coastline, a representative sand of this type was employed in the calibration. A Holocene dune sand was selected from the Stockton beach. This sand was characterised by extensive laboratory testing by Ajalleoian et al. (1996), who reports that it is composed of 98.8% quartz fragments, with minor heavy minerals and rock fragments.

The particle size distribution of Stockton beach sand is presented in Figure 2.

Figure 2. Particle size distribution for Stockton beach sand. (After Ajalleoian et al., 1996)

Ajalleoian also reports that the sand has a roundness of 0.41, a sphericity of 0.72 and maximum and minimum dry densities of 1.77 and 1.49 t/m³, respectively.
3.2 Calibration Chamber

The calibration procedure was conducted using a cylindrical calibration chamber which was designed and constructed at the University of Newcastle. It has a height and diameter, both of 1 m. The chamber was lined with 2 inflatable rubber membranes; one in the base of the chamber; the other around its internal circumference. Each of the membranes was connected to a separate, passive inflation/pressurisation system. Water was used as the pressurisation fluid. Vertical pressurisation was achieved by inflation of the base membrane to raise the sample into contact with the lid. Testing was conducted through a 20 mm diameter hole in the centre of the lid.

3.3 Sample Preparation

Samples were prepared to one of 3 target densities using the sand raining or pluviation technique (see discussion in Ajaloiaiz, 1996). This involves raining of the sand into the chamber, from a hopper at a fixed height, through a series of perforated plates and diffuser screens. The size of the perforations and arrangement of screens controls the resultant sample density. Consistency in prepared samples is improved by having a hopper large enough to contain the entire volume of the test sample, avoiding the need to interrupt the pluviation procedure. It was found that the raining times for samples of a particular density were similar, suggesting good consistency in the density of prepared samples. This is confirmed below.

For this project, the target sample densities were nominally chosen to be 'loose', 'medium' and 'dense'. Measurements at a variety of positions within prepared samples yielded the following typical statistics:

Table 1. Typical density statistics of prepared samples.

<table>
<thead>
<tr>
<th></th>
<th>loose</th>
<th>medium</th>
<th>dense</th>
</tr>
</thead>
<tbody>
<tr>
<td>number of tests</td>
<td>10</td>
<td>21</td>
<td>28</td>
</tr>
<tr>
<td>Bulk densities kg/m³</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>max</td>
<td>1.555</td>
<td>1.657</td>
<td>1.731</td>
</tr>
<tr>
<td>min</td>
<td>1.579</td>
<td>1.690</td>
<td>1.759</td>
</tr>
<tr>
<td>average</td>
<td>1.570</td>
<td>1.678</td>
<td>1.739</td>
</tr>
<tr>
<td>relative densities %</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>average</td>
<td>31</td>
<td>63</td>
<td>87</td>
</tr>
</tbody>
</table>

3.4 Testing

Because of the dynamic nature of the PSP test, it was expected that some amount of sample densification might occur during the course of each test. It was thus decided that only one sounding profile could be reliably measured in each prepared sample. Testing was carried out in the centre of the chamber, to a depth of 0.85 m.

For each of the 3 sand densities described above, tests were carried out under applied pressures to simulate depths up to 5 m. The density data of Ajaloiaiz (1996) was used to estimate the appropriate vertical pressures, while the friction angle data was used to calculate the earth pressure coefficients. The values employed are given in Table 2.

Table 2. Schedule of test pressures.

<table>
<thead>
<tr>
<th></th>
<th>loose</th>
<th>medium</th>
<th>dense</th>
</tr>
</thead>
<tbody>
<tr>
<td>unit wt. (kN/m³)</td>
<td>1.54</td>
<td>1.64</td>
<td>1.71</td>
</tr>
<tr>
<td>friction angle (°)</td>
<td>42.5</td>
<td>36.5</td>
<td>32.0</td>
</tr>
<tr>
<td>孔隙水压力 (kPa)</td>
<td>0.47</td>
<td>0.41</td>
<td>0.32</td>
</tr>
<tr>
<td>孔隙水压力 (kPa)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>孔隙水压力 (kPa)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>孔隙水压力 (kPa)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.5 Measurement of Pull-out Resistance

Whilst able to simulate deep in situ conditions at the tip of the penetrometer rod, the chamber fails to account for the increasing amount of resistance due to rod friction which would occur in deep test situations. That is, in a simulated 5 m test, only 1 m of rod was actually embed- ded, so that 4 m of side friction on the rods was un- accounted for, rendering the results potentially un- conservative.

Two approaches were adopted to account for this. Firstly, the relative side friction on embedded rods in loose, medium and dense samples, and simulated depths up to 5 m, were assessed by measuring the quasi-static pullout resistance of the rods after driving. This was achieved simply by incorporating a spring balance between the embedded rod and a pillow cable. The load on the balance was recorded as the rod was extracted at a steady rate of about 100 mm per sec- ond.

The second approach involved attaching the PSP slide hammer to the end of the embedded rod, via a rope over a large, free running pulley. This effectively facilitated driving of the rod in the reverse direction, allowing the number of blows to overcome side friction to be estimated directly. The arrangement is illustrated in Figure 3.
4. RESULTS AND DISCUSSION

4.1 Preliminary Results

As expected, the number of hammer blows to achieve a nominal rate of penetration generally increased with the density of the sample and increased in sample confining pressure. During the test procedure, results were recorded as a number of blows per 50mm. In analysing the data, blow counts per 150mm were obtained by summing any 3 consecutive 50mm values. The results are presented in Figure 4.

It is apparent from this figure that only the results between 0.4 and 0.85m within the chamber have been plotted; those outside this range are omitted because of possible boundary effects.

The results show a significant amount of scatter, possibly resulting from several effects. These include:

- The effect of vibrations on the sample due to dynamic testing. The principal effect is that of sample settlement, causing the sample volume to decrease. This results in the upper surface of the sample falling away from the lid of the chamber. As the pressurisation is passive (not self maintaining) it results in a drop in the vertical confining pressure. While this behaviour was anticipated and efforts were made to account for it by continual adjustment of the pressures, it is likely to have had some effect on the consistency of the results.

- The effect of slight differences in the prepared densities of the samples. Despite reports of only...
small statistical differences in measured densities between, and within, prepared samples (AJalbotian, 1996), the results here suggest that there may be some variations between successive sample variations.

Whilst the extent of scatter appears to be of considerable significance, it is not inconsistent with the results which are routinely obtained from natural sand deposits. In this study, it is addressed by maintaining conservatism in the adoption of interpretative trend lines. In Figure 4.a, the trend lines (dotted) are drawn to link the isolated raw data groups, and are positioned to serve more as upper bounds, than as averages.

4.2 Correction for missing Shale Friction.

As discussed in section 3.5, tests performed in a 1m deep chamber to simulate deep conditions (>1m), are potentially unconservative because they do not account for frictional losses along the length of rod which would be embedded in the ground (above test level) under actual field conditions. An attempt to account for this has been made by conducting pullout tests on the driven rods at the end of each test.

In the case of both quasi-static and dynamic pullout tests, it was observed that the pullout resistance was relatively steady after a peak pullout load had initially been applied to reverse the driven rod through a short distance of about 50mm. By comparing both the peak and steady resistances of the quasi-static tests, the following trends were established:

- For tests at any particular depth, the resistances in loose and medium sands were in the order of 25% and 60% respectively, of values in dense sand.

- For tests at any particular density, the resistances increased steadily in proportion with increasing confinement, with the values for 0-1m of confinement being about 35% of those for 4-5m of confinement.

Based on the results of 5 dynamic pullout tests from dense samples under 4.5m of confinement, the following values were established:

- The pullout distance for a single upward blow varied from 150 to 300mm.

- The average pullout distance for a single upward blow was 200mm, corresponding to a maximum correction of 0.75 blows per 150mm in dense sand at 5m.

On the basis of this information, it is possible to estimate the number of blows which would be required to advance the rod through overlying depths of soil, if it were present. The estimated correction is shown in Figure 4.b. Note that there is no correction for unconsolidated samples representing the upper 0-1m of soil profile. When added to the interpreted trends in the raw data, the solid lines in Figure 4.a, are obtained.

4.3 Calibration chart for the PSP.

By interpolating the corrected blow count data in Figure 4.a, a calibration chart relating the density of sand to testing depth and blow counts/150mm has been developed. It is presented in Figure 5. The plotted points in Figure 5, have been taken from the corrected results in Figure 4.a.

Comparisons of the chart with the results of Blick and Clegg (1965) are difficult because of the limited depths considered in that study. However, it is interesting to note that the quoted blow counts for depths of sand between 200 and 750mm, at 80% relative density, are a constant value of 8/150mm. By comparison, this study determined blow counts ranging from 3/150mm at 200mm to 8/150mm at 750mm in sand at the same density. The constant blow count value for shallow depths in the 1965 study is inconsistent with the results of this study, and extensive field experience of the author.

5. CONCLUSIONS.

The blunt tipped, dynamic PSP is an extremely useful and efficient tool for the characterisation of sand sites. Prior to this study, the limited results of Blick and Clegg (1965) were the only generally available data for the interpretation of the results of this test.

In this study, a calibration chart has been developed for use in the density characterisation of clean, dry silicic sand to 5m deep. Whilst the trends in the raw data showed an amount of inconsistency, it is expected that the conservative approach adopted in the analysis of this data has preserved a satisfactory degree of conservatism in the resultant calibration chart.

Nevertheless, the inference of in situ density using the PSP and the chart developed here, must in all cases, be tempered by engineering judgement and experience. Those seeking to characterise sand sites with the PSP must appreciate that the chart is the product of interpolation of a limited data set, and can thus give only a guide to the likely magnitude of in situ densities. In particular, one must be aware that only a small amount of data for loose sands at shallow levels has been used in the formulation of the chart. Thus, any interpretations made in this regard must be assumed to have a significant degree of uncertainty.

6. ACKNOWLEDGMENTS.

The efforts of Mr Colin Fisher, in performing most of the testing described here, are recognised and appre-
associated. Also, this research has been carried out with financial support from the Mine Subsidence Board of New South Wales, Robert Carr and Associates and the Australian Research Council.

7. REFERENCES


AS1289.F3.2 1984 *Determination of the Penetration Resistance of a Soil using the 9kg Dynamic Cone Penetrometer*, Standards Association of Australia.


SPT N-value and S-wave velocity of gravelly soils with different particle gradings

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Chuo University, Tokyo, Japan
Yasuo Yoshida
Central Research Institute of Electric Power Industries, Aomori, Japan

ABSTRACT: Properties of gravelly soil layers liquefied during recent earthquakes are first reviewed to find that SPT N-value and S-wave velocities of gravelly soils are quite low despite their high soil density. Series of large soil container test are carried out for various gravelly soils with different particle gradings to formulate SPT N-value and S-wave velocities as functions of the uniformity coefficient, the relative density and the confining stress. These values are found very sensitive to the difference in particle gradings and can be as low as loose sands if the soils are relatively loose. Empirical formulas proposed for N-values and S-wave velocity for gravelly soils are found consistent with a previous research based on field data.

1 PROPERTIES OF RECENTLY LIQUEFIED GRAVELLY SOILS

In the Hyogoken Nanbu earthquake [1995], heavy damages were caused by extensive liquefaction in fill layer consisting of gravelly soils of decomposed granite called Masa. In some other recent earthquakes, namely the 1993 Hokkaido-Nansei-Oki earthquake in Japan and the 1989 Loma Prieta Earthquake in USA, it has been found that loosely deposited gravel layers liquefied causing differential settlements or flow slides (Kokusho et al. 1995 and Andrus 1994).

Fig.1 indicates the mean grain size (D50) versus uniformity coefficient (Uc) relationship for recently liquefied gravelly soils for which soil data are available. So far the upper limits for D50 and Uc are about 20mm and 300 respectively, but no reasonable limit may be justified, indicating that whatever coarse gravelly soils can liquefy under loose enough conditions. Dry densities of these gravelly soils are rather high (1.7-2.0t/m3 for Kobe Port Is. and 2.0-2.1t/m3 for Hokkaido Mori) due to large uniformity coefficients, much higher than typical liquefiable coarse sands. Fig.2 shows the range of the SPT N-value (5 to 16) and the S-wave velocity (60 to 210m/s) for these liquefied gravelly soils. These values, though not normalized for the effective confining stress and the driving energy, indicates that the liquefied soils exhibited quite low N and Vs values despite high soil densities. These recent case histories are posing a challenging geotechnical problem that well-graded gravels can take unexpectedly low N-values and S-wave velocities and can liquefy despite its much higher dry density than typical liquefiable sands.

Fig.1 Mean grain size D50 versus Uc relationship for gravelly soils recently liquefied

Fig.2 SPT N-values versus S-wave velocity relationship for gravelly soils recently liquefied

Compared to typical sands little is known so far about the SPT N-value and S-wave velocity of gravelly soils with regard to their density, particle grading, etc., though they have significance in engineering
design not only in dynamic but also in static problems. Gravels are normally better graded than sands in natural deposits (Kobus et al. 1995). In other words, gravels are under most natural conditions actually the mixture of gravels, sands and sometimes even finer materials. Therefore gravel layers can be densely packed and is normally believed to be stiffer and seismically stabler than sand layers. However these well-graded gravelly soils as previously mentioned can have low N-value and S-wave velocity if they are relatively loose. As a basic step to understand fundamental properties of such well-graded soils and to develop simplified empirical formulas to correlate N-value and S-wave velocity with fundamental soil parameters, large scale soil container tests for sands and gravels with varying particle gradings have been carried out in this research.

2 TEST METHOD

The circular steel container used in this test was 2.0 m in inside diameter and 1.5 m in inside height as shown in Fig. 3. Artificial soil layers were made in this container, saturated and vertically loaded hydraulically with a given overburden by a rubber bag installed just beneath the container cap. The soils were placed in the container with various initial density either by foot-tamping or by a mechanical tamper. The initial density was calculated by the total weight of the soil layer, its average water content and the total volume. The change of the volume due to changing overburden was monitored and taken into considerations in the data analysis. The overburden stress was initially set at 50 kPa and then raised step by step. In the first series of test (called LC series), the maximum overburden was 200 kPa, however in the later test series (HC series) the container was reinforced to raise the maximum overburden up to 1 Mpa. SPT was carried out through five openings in the container cap and into soils loaded with various overburden stresses. The rod length for SPT between the tip and the knocking head was only 3 m in the test. Previous researches indicate that, for the rod length shorter than 12 m, the driving energy efficiency of SPT is reduced and for 3m rod length the reduction factor of 0.75 is recommended (e.g. Skempton 1978). However, in one case of this research the rod length was elongated from 3m to 6m, but no meaningful difference of N-value was found. Therefore no modification of measured N-value for the rod length was made in this research.

A steel rod penetrating the both sides of the container wall was buried in the lower part as a wave source for S-wave velocity measurements. The horizontal shear wave, the frequency of which was around 1.5 kHz, generated by hitting the both ends of the rod and propagating upwards was measured by four sets of wave sensors 25 cm or 35 cm apart to each other.

![Fig. 3 Circular soil container used in the test](image)

![Fig. 4 Particle properties of five tested soils](image)

Table 1 Physical properties of tested soils

<table>
<thead>
<tr>
<th>Soils</th>
<th>N - value</th>
<th>S-wave velocity</th>
<th>Void ratio</th>
<th>SPT Ref.</th>
<th>SPT Value</th>
<th>Mass of soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>G matrix</td>
<td>1.05</td>
<td>1.15</td>
<td>1.06</td>
<td>6.40</td>
<td>950</td>
<td>0.354</td>
</tr>
<tr>
<td>G matrix</td>
<td>1.05</td>
<td>1.15</td>
<td>1.06</td>
<td>6.40</td>
<td>950</td>
<td>0.354</td>
</tr>
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<td>1.06</td>
<td>6.40</td>
<td>950</td>
<td>0.354</td>
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<td>0.354</td>
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<tr>
<td>G matrix</td>
<td>1.05</td>
<td>1.15</td>
<td>1.06</td>
<td>6.40</td>
<td>950</td>
<td>0.354</td>
</tr>
</tbody>
</table>

Five different soils with different particle grading as shown in Fig. 4 were used in the test; two kinds of sand with different mean grain size (called here as TS and TK5 sands) and three kinds of gravels (called here as G5, G50 and G75 gravels). Particles composing these gravels are mostly of round shape, mainly of chert, andesite and sandstone and hard in quality. Physical properties of these soils are tabulated in Table 1. The minimum void ratio, e_min, of the five kinds of soils were determined by using a large size mold of 30 cm in diameter and a cap with a mechanical vibrator on it, while the maximum void ratio, e_max, were determined by gently filling the same mold by a hand shovel (Kobus et al. 1995).

Stress distribution in the container was monitored with the eight vertical and three horizontal pressure cells. The average vertical and horizontal stresses were calculated at the mid-height level of the tested soil layer by considering vertical and horizontal variations in stresses monitored by these pressure cells. These stresses at the mid height level are referred as
"vertical stress" and "horizontal stress" ($\sigma_v$ and $\sigma_h$ respectively) hereafter and correlated with the S-wave velocity and the N-value measured at almost the same level. The details of the test method are available in other literature (Kokusho and Yoshida 1997).

3 TEST RESULTS ON S-WAVE VELOCITY AND ITS FORMULATION

Based on previous researches (e.g. Roessler 1979), $V_s = V_s(\sigma_v/\sigma_h)^{\alpha_v}$ may be assumed to evaluate the S-wave velocity based on stresses, $\sigma_v$ and $\sigma_h$, and void ratio, $e$. Here, $V_s(\sigma)$ is the S-wave velocity under unit vertical and horizontal stress for a given void ratio, $e$. By using this equation, the S-wave velocity is normalized as $V_s(\sigma_v/\sigma_h)^{\alpha_v}$ and plotted against void ratio as shown in Fig.5 for the five tested soils. The plotted relationship obviously indicates the S-wave velocity can be almost linearly correlated to void ratio and approximated by the linear regression lines drawn in the figure. Despite possible discrepancies between the high capacity and low capacity tests, there can be seen an obvious trend that the correlation systematically moves leftward as the soil is getting coarser and better graded, indicating that the S-wave velocity is not determined by an unique function of void ratio as proposed by Hardin and Richard (1963) but is highly variable with difference in particle gradation. In order to totally evaluate the S-wave velocity for soils with different gradations, the normalized S-wave velocities corresponding to $\sigma_{vm}$ and $\sigma_{hm}$ previously mentioned, $V_{s\text{vm}}$ and $V_{s\text{hm}}$ respectively, are introduced here. These values can be determined as the crossing points of the straight lines with $\sigma_{vm}$ and $\sigma_{hm}$ as plotted in Fig.5 with open circle and square marks respectively for each soil material. It is noteworthy in the figure that $V_{s\text{vm}}$ does not change much for different soil materials while $V_{s\text{hm}}$ remarkably increases with decreasing $\sigma_{hm}$. In other words, the minimum S-wave velocity of gravels corresponding to the maximum void ratio will stay at almost the same level as that of sand despite much lower void ratio than sand. On the other hand, the maximum S-wave velocity of gravels corresponding to the minimum void ratio tends to considerably increase with the increase of the uniformity coefficient.

These values of $V_{s\text{vm}}$ and $V_{s\text{hm}}$ are plotted against $U_r$ in Fig.6, which may be approximated by simple equations indicated on the graph. Values of the power $m$ in Eq. $(1)$ obtained in the series of tests are plotted in Fig.7 against the relative density $D_r = (\rho_{\text{vm}}/\rho_{\text{hm}})(\sigma_{vm}/\sigma_{hm})$. Obviously, $m$ is decreasing a little with $D_r$ but it may also be assumed that $m = 0.125$ constant. Then, the power $k$, connecting the initial shear modulus $G_0$ and the overburden stress $\sigma_0$, as $G_0 = 4(\sigma_0/\rho)^{3/2}$. 

\[ V_s = V_{s\text{vm}} (V_{s\text{hm}}/V_{s\text{vm}}) D_r (\sigma_v/\rho)^{3/2} \]  

From Figs.6 and 7, $V_{s\text{vm}}$ is taken as 120 m/s and $m=0.125$ as an average, and $V_{s\text{hm}}$ is approximated.
as $V_s_{\text{max}} = 420U_c/(U_{c+1})$. Substituting these values and correlations into Eq. (1), the S-wave velocity of gravelly soils can be written as:

$$V_s = (120 + 420U_c/(U_{c+1}) - 120)D_{50}/(gV_s)^{1.115}$$  \(2\)

In Fig. 8, the estimated $V_s$ by Eq. (2) are compared with the values measured in all the tests. It is evident that the empirical equation can estimate the tested results within the deviation of 1.2 to 1/1.2.

4 TEST RESULTS ON SPT N-VALUE AND ITS FORMULATION

N-value may be correlated to the mean stress, $\alpha_{\text{op}}$, and void ratio, $e$, as

$$N = N_e((1 + 2\lambda)\alpha_{\text{op}})^{2}$$

where $p_0$ is an unit pressure for normalizing the stress taken here as 98 kPa and $n$ is the power. Based on numerous test results for the four kinds of soils in the HC and LC tests, the relationship between void ratio, $e$, and $N_e$ was plotted as shown in Fig. 9 where $\alpha_{\text{op}} = (1 + 2\lambda)/3$ is the mean pressure. It has been found that the N-value normalized for an unit confining pressure is linearly correlated with the void ratio on the full logarithmic chart but its location is quite different on the chart due to the difference in soil particle grading. This indicates that void ratio can not be used to evaluate N-value if gravelly soils with different grading are concerned and other indicator like relative density based on maximum and minimum void ratios may be more appropriate.

In Fig. 9, crossing points for the eight straight lines with $e_{\text{max}}$ and $e_{\text{min}}$ mentioned above are plotted with solid circles and squares. Except for G7S, these plots are almost linearly aligned on the chart, indicating that the maximum possible N-value, $N_{\text{max}}$, corresponding to $e_{\text{max}}$ tends to linearly increase for materials with higher uniformity coefficient, while

Fig. 8 Comparison of S-wave velocity between empirical formula and test results

Fig. 9 Normalized N-value versus void ratio relationships for four soils and max/min normalized N-value, $N_{\text{max}}$ and $N_{\text{min}}$

Fig. 10 Normalized N-value versus relative density relationship

Fig. 11 $N_{\text{max}}/N_{\text{min}}$ versus uniformity coefficient relationship and their approximation lines

The minimum possible N-value, $N_{\text{min}}$, seems to stay almost constant. Although the trend was obviously different for G7S gravel probably due to greater difficulties in conducting test for coarser soils like particle segregation etc., the previously mentioned linear alignment of $N_{\text{max}}$ or $N_{\text{min}}$ was assumed to
hold even for this material for subsequent data processing. Thus, $e_{\text{max}}$ and $e_{\text{min}}$ were modified for G75 as shown in the parentheses in Table 1. Despite the smaller reliability of G75 data, Fig.9 clearly indicates that the N-value can be as small as loose sands in loose gravelly soils despite a large difference in void ratio and grain size, whereas it can be much greater than that of dense sands if they are dense enough.

Fig.10 shows the relationship between $N_e = N(N_e \ln(d_0) / \ln(d_m))$ and relative density $D_r = (\varepsilon_{\text{max}} - 0.5) / \varepsilon_{\text{min}}$, indicating that well-graded gravels can take wider range of N-values than poorly graded sand and that for $D_r$ less than 40%, the difference due to different grading may be negligible.

Based on these considerations, $N_{\text{max}}$ and $N_{\text{min}}$ are correlated with the uniformity coefficient, $U_e$, as shown in Fig.11. Furthermore, $N$ for a given void ratio can be expressed by the following formulation based on the linearity of $N_e$ versus $e$ relationships on the full logarithmic chart.

$$N = N_{\text{min}} (N_{\text{max}} / N_{\text{min}})^{2.0} \left( c_{\text{sy}} / p_{\text{m}} \right)^{0.5} \left( \varepsilon_{\text{max}} / \varepsilon_{\text{min}} \right)^{0.5}$$

(3)

$D_r*$ is a logarithmic relative density and defined here as $D_r* = \log(e_{\text{max}} / e_0) / \log(e_{\text{max}} / e_{\text{min}})$ and $n(D_r*)$ is the power for the confining stress $c_{\text{sy}}$ and plotted against $D_r*$ as shown in Fig.12 based on numerous test data. Despite rather large data scatter, the relationship can be formulated for all the four materials as $n(D_r*) = -0.27(D_r*)^{-0.8}$. It is evident that the power $n$ tends to monotonically decrease with increasing relative density. On the other hand, the minimum and maximum N-value can be correlated with the uniformity coefficient, $U_e$, as $N_{\text{max}} = 58$ and $N_{\text{min}} = 42.6$ as indicated in Fig.11, therefore the following empirical equation can eventually be drawn.

$$N = 58 \left( 42.6 \ln(c_{\text{sy}} / p_{\text{m}}) \right)^{0.5} \left( c_{\text{sy}} / p_{\text{m}} \right)^{0.5} \left( e_{\text{max}} / e_{\text{min}} \right)$$

(4)

In this equation, N-value can be correlated simply with the uniformity coefficient $U_e$, the logarithmic relative density $D_r*$ and the mean pressure $e_{\text{sy}}$. The

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Fig.10 Power $n$ versus logarithmic relative density $D_r*$ relationship and its approximation curve

Fig.11 Comparison of N-value between empirical formula and test results

Fig.13 S-wave velocity versus N-value relationships compared with previous researches

Fig.14 S-wave velocity versus N-value relationships compared with empirical formula proposed in this research

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5 S-WAVE VELOCITY VERSUS N-VALUE RELATIONSHIP

Based on the test results on S-wave velocity and N-value for four kinds of soils, the relationship between N and Vs, both of which are normalized for the confining stresses, can be obtained and linearly approximated on the semi-logarithmic chart as shown in Fig.14. Although N and Vs have normally been linearly correlated on the full logarithmic graph in most engineering practice, semi-logarithmic linearization is chosen here based on the difference in the equation forms in Eq.(2) and Eq.(4). Despite some scattering in test data, it is evident from the chart that gravely soils tend to have larger Vs than sand for the same N-value. In the same figure is also shown the curves based on the empirical equations between Vs and N-value proposed by Ohita and Goto(1978) based on field investigation data for Holocene fine sand and gravel for the depth D=10m, indicating a similar trend.

In Fig.15, N versus Vs relationships obtained in the test are compared with those based on empirical equations proposed in this research in Eqs.(2) and (4). It may be said that, at least for sand and two kinds of gravels, the empirical formulas proposed in this research are also in accordance with the test results in terms of N versus Vs relationships.

6 CONCLUSIONS

1) Gravely soils liquefied during recent earthquakes had quite low S-wave velocities and SPT N-values despite higher density than sand due to better particle grading.
2) S-wave velocity for each kind of soil is almost linearly related with void ratio, e, while for N-value log(N) is linear with log(e). However, relationships between Vs versus e and N versus e for different kind of soils are highly dependent on particle gradation and cannot be uniquely determined by void ratio.
3) The lowest possible values of S-wave velocity and N-value corresponding to 0% mean to stay almost constant even for soils with quite different particle gradation, while the highest values corresponding to 100% remarkably increases with increasing uniformity coefficient. Thus a well-graded gravel with much higher dry density than sand may actually be able to exhibit S-wave velocity and N-value as low as that of poorly graded loose sand if it has a low enough relative density. In other words, gravely soils, in contrast to sandy soils, show larger changes in S-wave velocity and N-value with a small change of void ratio.
4) Empirical formatters are proposed in which Vs and N are respectively correlated with basic properties of gravelly soils; the uniformity coefficient, the relative density and the confining stresses. A direct correlation between Vs and N derived from the above formulas has been found compatible with an empirical formula based on field data by other researchers.
5) These empirical formulas have been derived solely from the large container test using limited numbers of gravelly soils. The constants presented in Eqs.(2) and (4) should therefore be revised according to field data on natural gravelly soils. However, the basic forms of equations as presented in Eqs.(1) and (3) are to be borne in mind in evaluating soil properties of gravelly soils.

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Standard penetration tests with torque measurement

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ABSTRACT: Recently, geotechnical engineers in Brazil have made the suggestion to supplement the conventional split spoon Standard Penetration Test (ASTM D1586) with a measurement of torque after driving. To perform the torque measurement, a rod adapter is attached to the top of the drill string and a simple torque wrench is used to rotate the spoon to obtain the maximum torque. The torque resistance is assumed to derive only from the interaction between the spoon and the surrounding soil. This procedure in no way compromises the current standard SPT practice, but provides an additional quasi-static measurement following the dynamic measurement of the spoon penetration. The torque measurement may have direct application in estimating skin friction for the design of driven pipe piles. A comparison of SPT-Torque tests and pile load tests performed in a sand is presented in this paper.

1 INTRODUCTION

A series of tests were performed to investigate values of unit skin friction obtained from the rotation of a Standard Penetration Test (SPT) split spoon to evaluate the rotational skin friction obtained from the measurement of the corresponding maximum torque. Tests were conducted at a site consisting of medium dense uniform fine to medium sand. Torque measurements were obtained in several test borings down to depths of 6.4 m using 50.8 mm spoons of different configurations, i.e., with and without liners, and with a spoon fitted with a solid 60° point to simulate a fully plugged spoon. Additional tests were performed using a larger 76.2 mm spoon to evaluate scale effects. In order to provide a comparison of the skin friction obtained from the SPT torque measurements with the dynamic advance of the spoon, tests were also conducted by installing the spoon with quasi-static penetration, rather than by driving. The force vs. penetration record of the spoon advance allows for an independent measurement of the spoon skin friction developed in the vertical direction. The test results are presented along with results of the lab and field characterization of the site and a comparison is made between the unit skin frictions obtained using the various spoon configurations. A final comparison is presented between the skin friction results obtained from the torque measurements and the results of steel pipe piles installed at the site and loaded in tension to failure.

2 SPT-T TEST - BACKGROUND

According to Decourt (1989), Ramzine (1988) was the first to suggest a simple modification to the SPT in which the traditional SPT is complemented by a torque measurement. That is, after driving the split spoon, torque is applied to the top of the drill rod string in order to rotate the spoon. Decourt and Filbo (1994) have shown that the ratio of torque to SPT blowcount, N, (T/N with T measured in kgf-m) has proven useful in practice.

It can be argued that an advantage of the torque measurement is to add a static testing component to a test which results initially from a dynamic phase. While most of the soil structure may be destroyed during installation of the spoon, the
torque measurement may act in a region where the soil retains much of its original fabric and is only partially remolded. The torque measurement appears to be a novel addition to the SPT which does not detract from the standard test procedure and requires only minimal additional effort. In fact, the test only takes about another minute to perform. It seems logical that while the actual N value obtained from the SPT may be subject to wide variations because of differences in test equipment and field practice, the torque measurement may be subject to less variability. The torque may be affected if the spoon or rod wobbled and contact between the spoon and soil is lost.

The torque measurement may have direct application for estimating skin friction of driven piles. Using the moment arm as the distance from the center of the spoon (where torque is applied) to the outside diameter, and neglecting any contribution from the soil at the end of the spoon, the unit skin friction may be given as:

$$\tau_s = \frac{C}{T} \times dL$$  \hspace{1cm} [1]

where:

- $T$ = measured torque
- $d$ = diameter of spoon
- $L$ = length of the spoon driven

Equation 1 also neglects any contribution of skin friction from the inside of the spoon, which is likely to be the case if the spoon is used without liners and has an internal relief.

3 FIELD INVESTIGATION

A field investigation was performed to evaluate the unit skin friction measurements obtained with SPT-T tests and other variations of the SPT and to compare the test results with skin friction obtained from pile load tests. The test site and the tests performed are described in this section.

3.1 Test Site

The site investigated is located at the former U.S. Air Force Base in Plattsburgh, New York. The site is dominated by glacial and proglacial outwash and lake deposits. The surficial layer consists of a poorly

graded fine to medium sand with a trace of silt and represents a glacial meltwater delta deposit. The sand is on the order of 27 m in thickness. The ground water table is typically located at a depth of around 12 m below the ground surface.

A large number of test borings were performed at the site to determine the characteristics of the deposit and to obtain samples for laboratory testing. A total of 141 sieve analyses were performed on individual samples collected at the site. The results indicate that the sand deposit is very uniform with $D_{60} = 0.4$ mm, $C_3 = 2.0$, $C_2 = 1.6$, $C_1 = 0.4$, $C_0 = 4$, $G = 2.67$, $c_{sus} = 0.46$, $c_{ext} = 0.91$. The estimated in situ relative density is about 69%. The water content in the upper 10 m is on the order of 6%.

3.2 SPT-T Tests

Torque tests were conducted by attaching a drill rod adapter to the top of the drill string immediately after driving the spoon a distance of 45 cm. A direct read torque wrench with a capacity of 41.5 kgf-m and a precision of 0.7 kgf-m was then connected to the drill rod adapter and the operator rotated the assembly (rods and spoon) to produce a failure. Occasionally, if the drill rod connections were not tight, some initial hand tightening of the string was needed. The assembly is usually rotated a full 180° until a maximum value of torque is obtained.

In some situations the soil was insufficiently stiff or dense so that the torque wrench capacity was reached before a failure occurred. In these cases, a torque transducer was attached to the top of the drill string and a long handle was then used to rotate the assembly. The torque is then read from a digital strain indicator. A schematic of the test arrangement is shown in Figure 1.

In most cases, it was found that the use of the direct read torque wrench was sufficient. The technique is simple to perform, inexpensive and easy to deploy and is therefore attractive for a field engineer. No attempt was made to regulate the rate at which the torque was applied as the focus of the torque measurement is on simplicity and rapid testing so as not to overcomplicate the procedure. Given the perceived crudeness of the SPT in general, this was considered sufficient. Torque was applied in a slow and steady manner so that rotation would be complete in about 15 sec. In order to be able to make a comparison between the SPT blowcount and the torque measurement, all SPT's reported in this paper...
were conducted by driving the spoon a distance of 45 cm using a CME automatic hammer.

![Diagram of SPT-T test arrangement](image)

Figure 1. Schematic of SPT-T test arrangement.

3.3 SPT-THRUST Tests

SPT-Thrust Tests were also conducted in order to obtain a comparative measurement of the quasi-static thrust required to advance the spoon. Similar tests have been reported by Schmertmann (1979) and to the authors’ knowledge are the only such tests reported in the literature. After lowering the spoon into the borehole, a strain-gauged load cell was threaded onto the top of the drill string. Thrust was applied by the hydraulic feed of the drill rig and the quasi-static pushing thrust was measured at each 76 mm of penetration using a digital readout strain indicator. This arrangement is shown in Figure 2.

Schmertmann (1979) suggested that if a plot of thrust vs. penetration were made, the results would show a linear relationship between the thrust and penetration. As the spoon is penetrated into the subsurface, the contact area of the spoon with the soil increases thus increasing local skin friction. Extrapolation of the data to zero penetration would then give an estimate of the thrust needed to overcome the end bearing produced from the projected end area of the spoon. The difference between the extrapolated end thrust and the total thrust after a full 45 cm penetration would be the total thrust attributed to the local skin friction on the outside of the spoon. (This neglects a small amount of thrust from the internal part of the shoe.)

3.4 SPT-PULL Tests

In some cases, after either applying the torque in an SPT-T test or after conducting an SPT-Thrust Test, the load cell at the top of the drill string was used as a tension load cell to measure the maximum pulling force required to remove the spoon as shown in Figure 2. After pulling the spoon, the assembly was suspended above the bottom of the borehole to obtain a measure of the total mass of the assembly. Since there is no end bearing when the spoon is in tension, all of the pulling force can be attributed to local side friction acting on the outside of the spoon. These measurements provided yet another comparison for spoon skin friction obtained from both the SPT-T and SPT-Thrust Tests.

Figure 2. Test arrangements for SPT-Thrust and SPT-Pull tests with load and frictional forces acting on the spoon.
3.5 Pile Load Tests

In order to provide a basis for comparison of the skin friction values measured in the field tests, a series of prototype-scale tension pile load tests were conducted on several small diameter pipe piles installed at the site. Pile characteristics are given in Table 1. Both open end and closed end piles were tested. All piles were allowed to rest for a period of 30 days prior to testing. Axial tension tests were then performed until failure. Table 2 gives a summary of the load tests and the calculated average unit skin friction based on the measured failure load and the calculated external surface area.

Table 1. Summary of pipe pile characteristics.

<table>
<thead>
<tr>
<th>Pile</th>
<th>L (m)</th>
<th>OD (mm)</th>
<th>ID (mm)</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.1</td>
<td>73.0</td>
<td>-</td>
<td>closed</td>
</tr>
<tr>
<td>2</td>
<td>6.1</td>
<td>88.9</td>
<td>-</td>
<td>closed</td>
</tr>
<tr>
<td>3</td>
<td>6.1</td>
<td>114.3</td>
<td>-</td>
<td>closed</td>
</tr>
<tr>
<td>4</td>
<td>6.1</td>
<td>73.0</td>
<td>60.3</td>
<td>open</td>
</tr>
<tr>
<td>5</td>
<td>6.1</td>
<td>88.9</td>
<td>76.2</td>
<td>open</td>
</tr>
<tr>
<td>6</td>
<td>6.1</td>
<td>114.3</td>
<td>101.6</td>
<td>open</td>
</tr>
<tr>
<td>7</td>
<td>6.1</td>
<td>88.9</td>
<td>-</td>
<td>closed</td>
</tr>
<tr>
<td>8</td>
<td>6.1</td>
<td>44.5</td>
<td>-</td>
<td>closed</td>
</tr>
</tbody>
</table>

Table 2. Pile failure load and unit skin friction.

<table>
<thead>
<tr>
<th>Pile</th>
<th>Failure Load (kN)</th>
<th>Fb (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>208</td>
<td>148.7</td>
</tr>
<tr>
<td>2</td>
<td>86</td>
<td>50.5</td>
</tr>
<tr>
<td>3</td>
<td>197</td>
<td>99.0</td>
</tr>
<tr>
<td>4</td>
<td>232</td>
<td>165.9</td>
</tr>
<tr>
<td>5</td>
<td>484</td>
<td>284.3</td>
</tr>
<tr>
<td>6</td>
<td>234</td>
<td>106.9</td>
</tr>
<tr>
<td>7</td>
<td>398</td>
<td>233.8</td>
</tr>
<tr>
<td>8</td>
<td>109</td>
<td>127.9</td>
</tr>
</tbody>
</table>

4 COMPARISON OF SKIN FRICTION

Figure 3 presents the results from typical SPT-T tests obtained using an unlined spoon. The values of unit skin friction obtained from Equation 1 are shown in Figure 3 and are similar in magnitude to the unit skin friction obtained from electric CPT’s performed at the site. The average recovery recorded from the SPT’s presented in Figure 3 was 90%, indicating that the spoon was advanced unplugged in this sand. Similar tests using a spoon with brass liners gave nearly identical recovery, blowcounts, and torque results suggesting that in this sand, the use of liners does not significantly affect either N or f.

To investigate the influence of spoon plugging on the measured unit skin friction, a 60° conical point was fabricated and threaded onto the end of the spoon in place of the standard shoe. This allowed the spoon to act in a completely plugged manner. Figure 4 shows a comparison of the unit skin friction obtained from the unplugged and plugged spoons. There is very little difference in the recorded values, and in fact the spoon with the conical point actually gives lower skin friction in many cases.

A comparison of the SPT-T, SPT-Thumb, and SPT-Pull tests for unplugged spoons is shown in Figure 5. It can readily be seen that the pull and torque tests give similar values of unit skin friction throughout the profile, but the thrust tests tend to give consistently higher values of unit skin friction. It is suspected that in this case, the extrapolation technique does not give an accurate measure of spoon end bearing. Recall that this procedure assumes that the end bearing will be a constant with advance of the spoon; the only increase in thrust being the result of the accretion of side friction. If the end bearing is not constant but increases as spoon penetration proceeds then the extrapolated end bearing will be too low and the calculated side resistance will be too high. This appears to be an area that requires more detailed work.

In order to investigate if the unit skin friction is a fixed quantity or is dependent on spoon size, a series of SPT-T tests were conducted using a 76.2 mm spoon without liners. A comparison between the standard 90.8 mm spoon and a 76.2 mm spoon unit skin friction is shown in Figure 6. With the exception of some points in the lower part of the profile, there appears to be only a minor effect of size on the unit skin friction in this sand.
It can be seen from comparing the results of the pile load tests given in Table 2 and the results presented in Figures 3, 4, and 6 that the measured average unit skin friction values obtained from the prototype pile tension tests are within the range of unit skin friction measured in the SPT-T tests. This is encouraging and suggests that the SPT-T data may be used for estimating the skin friction of driven piles in sand. However, it should be noted that the pile tests themselves produced some uncertain trends and the influence of pile size, pile plugging, L/D, and other factors should be taken into account for design.
Empirical approaches to the design of driven piles based on SPT N values have been suggested, e.g., Meyerhof (1976) based on observations of full-scale performance. Figure 7 shows a comparison of measured unit skin friction from the SPT-T tests and the 60% energy corrected SPT blowcount, N_60. The recommendation presented by Meyerhof (1956, 1976) for estimating unit skin friction from SPT N values is shown for comparison. The average trend line suggested by the SPT-T data from this site gave a unit skin friction about 2 times that suggested by Meyerhof (1956, 1976). One possible explanation for this is that the SPT N values used by Meyerhof (1956, 1976) were likely obtained in earlier years using either a Donat or Pinweight Hammer and therefore the N values would be artificially high as a result of lower energy levels produced by these two types of hammers. Additionally, Meyerhof suggested the correlation shown in Figure 7 essentially represents a conservative "lower bound" value for bored piles. Inspection of the test data presented by Meyerhof shows that the majority of driven pile data fall above this line.

Figure 5. Unit skin friction determined by the SPT-T, SPT-Thrust, and SPT-Pull tests.

Figure 6. SPT N-values and unit skin friction determined torsionally for 50.8 and 76.2 mm split spoons.
Figure 7. Comparison of unit skin friction versus $N_0$ values with the Meyerhof (1976) correlation.

5 SUMMARY AND CONCLUSIONS

The results of SPT-Torque tests conducted in this fine to medium sand suggest that the calculated unit skin friction values are similar in magnitude to average unit skin friction obtained from prototype scale pile load tests. The torque test is simple to perform and does not detract from the standard SPT test procedure. In the sand tested, changes in the spoon geometry or plugging did not have a pronounced effect in measured spoon skin friction. That is, there is strong evidence that the dynamic measurement of the test is directly correlated to the static skin friction acting on the outside of the spoon.

This was initially suggested by Schnuerlein near 20 years ago. The results presented in this paper and performed at a number of other sites by the authors may help to provide a justification for the empirical use of SPT results for design of piles. Ongoing additional research will help further verify this phenomenon.

REFERENCES


The practice of the standard penetration test in degradable soils

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ABSTRACT

The paper presents the obtained experiences in the use of the standard penetration test to determine the resistance characteristics of the puniceous materials that form the subsoil of the city of Guadalajara, the state capital of Jalisco and the second largest city of Mexico. In the development of the paper, are presented the most important characteristics of the urban subsoil: the observed behavior of some important structures of the city, the results of the load tests and is concluded that in spite of its limitations, the standard penetration test is a very valuable tool for the study of puniceous soils and is proposed a methodology for the shallow foundations design, based on the obtained experiences to date.

1 INTRODUCTION

The city of Guadalajara, counts currently with about five million of inhabitants and it is located in a great valley, limited by mountains and hills. The greater part of the superficial deposits under the urban area are formed by gravel and sands of puniceous origin.

In the city for many years, the standard penetration test (SPT), it has been used for the foundations design with satisfactory results; it is pointed out that, it given the impossibility from obtaining undisturbed samples in these soils and that the accomplishment of other penetration tests as the Dutch cone results very costly and difficult by the great frictional resistance of these materials; the SPT is practically the only one test used by the technicians in the foundations design of this city.

In this article is emphasized the use of the SPT, for the shallow foundations design, while in other work (Padilla 1994), was presented the case of deep foundations.

2 SUBSOIL CHARACTERISTICS

2.1 Geotechnical description

The subsoil of the city of Guadalajara is very uniform and is formed by powerful deposits of pyroclastics materials, made on pre-existing basin, which in some cases they have been transported and redeposited by the water and the wind, with intercalations of fluviatile deposits clay layers, or formations of basaltic rock, which are the product of the water action, as of the different cycles of volcanic activity of the region.

The pyroclastics deposits are made by sands and gravels, which particles are formed by punices lined in strata of variable thickness and slightly horizontal. From the surface up to the depth, from 4 to 8 m, the sands prevail with small quantities of silt or clay and relative density of loose to medium (2 to 30 blows in the SPT); to lower levels the relative density increases up to higher levels than the 50 blows of the referred test, eventually, lenses of fine materials are found (silt or clay) intercalated.

In the geotechnical exploration, basalticas formations have been found on depths that varies from outcrops
in some zones of the city, to more than 30.0 m in others.

The groundwater levels have been located at depths that vary from the 2.0 to more than 20.0 m, founded with some frequency isolated aquifers on intercepted watertight layers, intercalated in the big pyroclastic landfill. In Figure 1, a typical stratigraphy of the city is presented, where an important bridge was constructed.

![Figure 1. Typical soil profile.](image)

2.2 Laboratory tests

In regard to laboratory tests, in Table 1, are presented typical values of index properties.

<table>
<thead>
<tr>
<th>Index property</th>
<th>Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SM</td>
</tr>
<tr>
<td>Natural water content %</td>
<td>20.0-40.0</td>
</tr>
<tr>
<td>Void ratio</td>
<td>0.81-1.62</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.00-2.47</td>
</tr>
<tr>
<td>Bulk unit weight kN/m³</td>
<td>8.14</td>
</tr>
</tbody>
</table>

In regard to laboratory strength tests, the information is very poor due to the difficulty, that it has to be elaborate or reproduce undisturbed samples of these materials. In Table 2, are presented some of the obtained values.

<table>
<thead>
<tr>
<th>Test</th>
<th>Internal angle of friction φ</th>
<th>Cohesion kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined compression</td>
<td>∞</td>
<td>60-112</td>
</tr>
<tr>
<td>Unconfined compression</td>
<td>20°-42°</td>
<td>30-150</td>
</tr>
</tbody>
</table>

In synthesis the subsoil of the city is formed by pumiceous materials and are presented in the form of gravels and sands, their principal characteristics are: highly frictional, lightweight, degradable and of high absorption. One of the characteristics that influence significantly the deformability of a foundation, is its degradability for which a series of special tests have been made to try to analyze this phenomenon and establish criteria to estimate quantitatively the effect of the rupture of grains.

2.3 Degradation tests

Two series of tests were made on typical pumiceous materials, one gravel (GW) with 30% of sand and less than 5% of fines, and silty sand (SM) with 20% of fines. The study for each soil was made from its initial grain size distribution compacting with static load a series of specimens with contact pressure of 14.0 MN/m² (Porter static compaction test), 10, 5, 2.5 and 1.0 MN/m², having at the end of this test, another grain size distribution test.

The results are presented in Figure 2 in which, the degradation is measured with the breakage index of grains Bg*, as Marsal (1980) defined it. In the graphic analysis it is shown that for high contact pressures (5.0-14.0 MN/m²), the degradation is a lot larger on the coarse material but as the axial pressure decreases these differences are lower and from 2.0 MN/m² or less, are of the same order and minor quantities.

![Figure 2. Degradation test results.](image)

Bg*: numerically equal to the sum of differences with positive sign between the initial and final percentages by weight retained in a standard sieve series.
These results confirmed quantitatively that the pumiceous soils are degradable and that for the minor pressures to the 2.0 MN/m² the particles rupture is small, this coincide with the results founded in other studies.

3 LOCAL PRACTICE IN THE USE OF THE RESULTS OF SPT

Below is described briefly the generally use of the SPT results, although some specialists of the region use other procedures.

For the superficial foundations design, once accomplished the tests and corrected the number of blows N', either by presence of the groundwater level or by effective overcharge pressure, is opted for anyone of the following procedures:

1. It is estimated the internal friction angle $\phi$ of N', and with that parameter, the characteristics of the foundation and the unit weights of the soil, is calculated the allowable bearing capacity, applying the equation and the coefficients of Terzaghi and Peck (1967) or Vesic (1973) and a safety factor of 3.0.

2. With the geometric characteristics of the foundation, N' and the graphics of Terzaghi and Peck (1967) or Peck et al. (1974), is calculated the allowable pressure for a maximum settlement of 25 mm.

The described uses, cause that as a rule the recommended bearing capacities vary from 40 to 200 kN/m², for depths of foundations smaller to 2.0 m and, from 200 to 100 kN/m² for levels from 2.0 to 6.0 m. As is observed, these results are generally rather conservative.

With respect to use of the results of the SPT for the deep foundations design, basically are continued the Meyerhofer’s criterion (1970).

Other use very important of the SPT is the location of low resistance strata or zones with caves, due to deficient placing fillings or for erosion effects due to seepage; both situations are found frequently in the zones toward where grows the city.

4 FOUNDATIONS BEHAVIOR

As a rule in the city are not carried settlement records in the constructions, except in some works in which its importance, have been demanded. However, within the experience of the author can be said that given the low levels of bearing capacity that are used, few buildings have had serious problems by settlements and when they have been presented, in most cases, they have been by buildings which were build on deficient placing fillings or on zones eroded by seepage.

The author has participated in the foundations design for buildings from 4 to 6 levels supported on bound isolated foundations or by continuous foundations, built in different zones of the city, where the settlements were measured during the construction process. Of approximately 50 buildings, the settlements varied from 6 to 14 mm, with maximum differential settlement of 4 mm, in spite of that in all the cases the forecasts were of 25 mm.

In relation to historical cases, it has information of the underpinning of the church of Santa María de Gracia (Padilla 1990), where it was carried a careful control of the settlements in 10 different points of the building. The new foundation is formed by continuous foundations with a wide from 3.0 to 4.0 m, depths from 6.0 to 6.2 m and contact pressures from 220 to 280 kN/m².

The Figure 3 shows the settlement observed from the beginning of the construction process until two years after ended and the settlement forecasts calculated with the results of the SPT. Upon revising the

![Figure 3. Comparison between measured and predicted settlements for the Santa María de Gracia church.](image)
graph, is appreciated that the forecasts are rather conservative.

Other case was the underpinning of the Degollado Theatre, one of the historical buildings more valuable of the city (Padilla, 1990). In this work was carried a settlement control in 25 points of the building. The new foundation is formed by isolated foundations with horizontal dimensions of 5.0 x 3.75 m, depth of 7.0 m and work pressures until of 520 kN/m², with maximum settlement forecasts of 26 mm. In the results of the measurements, it was no point detected with a total settlement up to 12 mm during a period that comprised from the construction process until two years after.

In the case of deep foundations, the experiences measured also reveal that the forecasts made for bearing capacity, as well as for settlements are rather conservative (Padilla 1994).

5 LOAD TESTS

To obtain more behavioural information about from these soils, they have been accomplished some series of load tests, that are described below.

5.1 Plate bearing tests

They were accomplished in the first stage 12 plate tests in a site where the thicknesses of pumiceous sands vary from 5.0 to 18.0 m. The tests were accomplished on natural terrain with the use of circular seat plates of 450 and 300 mm of diameter, with the procedure to determine the reaction module, until a maximum load of 75 kN. Additionally, near each plate of test, SPT were accomplished to estimate the resistance of the terrain under the test plate.

In the Figure 4 are presented the settlements measured in the plate tests versus the settlement forecasts, based on the values obtained, according to two of the current procedures in use; the examination of the figure shows that the forecasts are excessively conservative; equal situation is presented for the case of the bearing capacity (Figure 5).

Figure 4. Comparison between measured and predicted settlements for the plate bearing tests.

Figure 5. Comparison between measured and computed bearing capacity for the plate bearing tests.
axis. This yielding is the result of fracturing of individual soil particles, which permits large relative motions between particles. The realized analysis showed that considerable degradation is produced.

3. The fracturing of the particles permits still tighter packing of the new and remaining particles. Since the number of grains has now increased, the average force has actually decreased. Thus the sand once again becomes stiffer and stiffer as the stress increases still further.

From the above, the degradation phenomenon shows that it is important for two reasons; one, when the rupture occurs, significant volumetric changes can occur and the second that exhaustive studies (Masait 1980), show that when the degradation increases, the resistance decreases as is shown in Figure 6.

![Stress-deformation curve, plate bearing test](image)

Figure 6. Stress - deformation curve, plate bearing test.

5.2 Micro piles tests

These tests were accomplished in 12 elements, with diameters from 150 to 200 mm and length from 2.5 to 3.0 m; the results showed a measured bearing capacity bigger than three times the theoretical, with a load-deformation behavior similar to the Figure 7, having its first curvature change (particles breakage), about 2200 kN/m² (Padilla 1994).

6 DISCUSSION

In the degradation test was observed the great influence of the size and shape of the mineral particles; so that if porous material is more coarse, it is less rounded and for that reason it is more susceptible to break, that it is the case of the tested gravel (Figure 2).

The measurements of the structures in service as well as the results of the load tests showed that the theoretical bearing capacity as well as the settlement forecasts are too conservative, in relation to the real resistance characteristics found in the studies; due to the fact that the work pressures or, of test, they are less than the breakage level of the grains of the soil (Figures 3, 4 and 5).

In the plate test to greater load, there was no confinement and less compact soil. In these conditions, there was sliding of particles, increasing the resistance of the terrain until a pressure of 1200 kN/m², where occurred a notable grains breakage (Figure 6).

In the micropiles tests, there was found a greater observed bearing capacity than the theoretical and a pressure of breakage particles nearly 2000 kN/m².

It has been found, that to greater depths the subsoil of the city has bigger relative density which indicates that it has suffered a previous degradation. This situation, together with the confinement pressure, makes it a major resistance material to the grains breakage than more superficial soils.

From the exposed, the importance of the grains breakage in the behavior of porous soils is evident, whose critical pressure is in function of the maximum size and hardness of the mineral particles, the grain size distribution, the relative density of soil and the confinement pressure.

7. CONCLUSIONS AND RECOMMENDATIONS

1. The SPT is simple, rapid, low-cost and it must be followed using in the exploration of porous soils, since permits to zone adequately the subsoil and to obtain an index from the resistance characteristics of the terrain; furthermore, the recovery of soil samples, provides very useful information about the nature of the crossed layer and in the case of the subsoil of the city of Guadalajara, there is not other test more rapid to determine soft strata and zones with caves.

2. The most important disadvantage of the SPT in
these degradable materials, it is that to be driven the sampler breaks particles of the soil, what by a part hinders notably the interpretation of the results, but by the other, it can be taken this circumstance as an additional safety factor.

3. Since that the empirical methods to estimate the bearing capacity of the settlement forecasts in base to the results of the SPT, and due to the reasons before mentioned, are too conservative. It is proposed for current practice the use of the criterion proposed by Sowers (1979) for the superficial foundations design. In the Figure 7 is presented the previous criterion, whose graph corresponds sensibly to a regression study, accomplished with the data of the plate tests and the measurements of the referred buildings, with a confidence level of 97% and a safety factor of 3. The use of the Figure 7 permits to increase the admissible bearing capacity in approximately 30% in relation to described current practices.

4. For the deep foundations, in other event, criteria were provided to improve the geotechnical design of these elements (Padilla 1994).

5. Finally, it is considered necessary to work with other type of field tests as the cone penetrometers, pressuremeters, etc., to characterize more widely the subsoil of the city and through additional studies, improve the current methods for the foundations design.

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Spatial distribution characteristics of soil strength in embankment estimated by portable dynamic cone penetration test

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ABSTRACT: Focusing on slope surface of railway embankment, we discussed the spatial characteristics of soil strength in the railway direction using the autocorrelation coefficient, which is expressed as a function of soil strength in the slope surface, which remains small at high soil strength and grows large length between soundings. We statistically analyzed shapes of embankments which occurred in rainfalls. Then we proposed a more reasonable interval of soundings which corresponds to the autocorrelation and collapse width of embankment slope along railway in Japan.

1 INTRODUCTION

Structures of the old type in Japan Railways are represented by embankments and cut slopes which were constructed not to the current technical design standards. Therefore many slope failures along the railway have occurred during rainy season or typhoon.

To determine the causes of the failures and to estimate the soundness of the slopes, a portable dynamic cone penetration is conducted as one of several sounding tests, which are the standard penetration test and the Swedish weight sounding test etc. As the portable dynamic cone penetration uses a portable rig and light weight, it is generally used in the slope investigations in the railway.

In this paper, focusing on the slope surface of railway embankment, we discuss the spatial soil strength characteristics in a vertical direction of embankment surface and a surface plane direction of embankment using the portable dynamic cone penetration. We also analyze the length dependence in railway direction of the soil strength using the autocorrelation coefficient.

Lastly, we propose an estimation method of a sounding interval for embankment strength.

2 BACKGROUND OF RESEARCH ON DISTRIBUTION CHARACTERISTICS OF SOIL STRENGTH

To express a space statistical characteristics of soil strength, we use mean and variance and resort to a regression equation. But to express spatial nonhomogeneity of the ground by area dimension, we use autocorrelation coefficient.

Regarding the correlation coefficient in the vertical direction, Matsu(1984) analyzed the characteristics of unconsolidated undrained strength of marine clay deposits, and he showed transient characteristics of undrained strength before and after consolidation of an embankment on soft clay. From these results he gave the autocorrelation coefficient as follows,

$$r_{i}\tau \approx \exp(-i / l)$$

(2.1)

where $r_{i}$ is the distance from place $i$ to $(i+\tau)$, $l$ is the correlation distance. Wu(1974) and Alonso(1975) gave a similar formula for correlation in the vertical direction.

Meanwhile, autocorrelation between soil properties in the horizontal direction shows a relatively high value because of the sedimentary process of natural ground. Tang(1979), using a parameter $h$ produced the following expression,
3.2 Characteristics of soil property

(1) Collapsed embankments during heavy rainfalls
The distribution of frequencies for $N_r$ of the portable dynamic cone penetration tests is shown in Fig.2 (Okada et al., 1992) for the collapsed embankments at 67 points, in which $N_r$ is obtained average as values between the slope surface and the 3m depth. The average embankment strength of the slope surfaces is $N_r=5.4$ with the standard deviation $\sigma=2.6$.

The grain size distribution is shown in Fig.3 for the samples taken at the sounding test sites. The dotted lines are $\pm 1 \sigma$ (standard deviation).

(2) Test sites of embankment
One of the test sites to make survey (called site-A) is about 14m high with a single track railway embankment constructed about 90 years ago. Soil samples were taken at 80cm deep from the embankment surface. The sample had a uniformity coefficient $U_c=111.3$, a curvature coefficient $U'=0.89$. This soil belongs to SM in the Japanese Method of Classification of Soils for Engineering Purpose.

The other site (called site-B) is about 5m high with a single track railway embankment constructed 25 years ago. Soil samples were taken at 30cm deep from the embankment surface at 50cm intervals. Some soils belong to CH, most soils belonging to SV.

$$r(r) = \exp(-r^2/\sigma^2)$$

But there is no quantitative treatment in the vertical direction and on the slope plane. This is one of the bottlenecks to estimating embankment stability.

3. DESCRIPTION OF SOUNDING TEST FOR A RAILWAY EMBANKMENT

3.1 Sounding test
Sounding tests were carried out on embankments which were collapsed during rainfall in the past and two test sites. The test instrument designed by the Public Works Research Institute of the Ministry of Construction (JAPAN) is called a portable dynamic cone penetration test instrument because it uses a portable rig, light weight and small in size as shown in Fig.1. The instrument whose total mass is about 15kg consists of a cone head connected with a rod and a driving weight for penetration. The angle of the cone is 60 degrees, and the diameter is 16mm. The number of strokes $N_r$ when the cone is penetrated 10cm and a mass of 5kg drops from a 50cm height, is counted.

Fig.1 Portable dynamic cone penetrometer

Fig.2 Histogram of soil strength $N_r$ (collapse embankment)

Fig.3 Grain-size distribution

\begin{tikzpicture}
\begin{axis}[
    width=\textwidth,
    height=0.5\textwidth,
    axis lines=left,
    xlabel={$D (\text{mm})$},
    ylabel={$\text{Per cent passing (%)}$},
    xtick={0.001,0.01,0.1,1.0,10.0},
    ytick={0,20,40,60,80,100},
    xticklabels={0.001,0.01,0.1,1.0,10.0},
    yticklabels={0,20,40,60,80,100},
    xticklabel style={align=center},
    yticklabel style={align=center},
    legend style={at={(0.5,0.75)},anchor=west},
    legend columns=2,
    legend entries={Unit of A.F. M., Single Silt, A typical embankment},
]
\end{axis}
\end{tikzpicture}
The grain size distribution of soils at site-A and site-B, shown in Fig.3 by the solid line and the broken line, are included in the range of grain size distributions of the collapsed embankments along all the Japan Railways.

Soil strengths in which $N_c$ was obtained as average values between the slope surface and the 3m depth at site-A and site-B, which matter will be treated, are $N_c=7.3$, 6.3 respectively. These strengths are in the range of standard deviation ($1\sigma$). Therefore, we conclude that embankment materials and soil strengths are the same as those of past collapsed embankment.

For the sounding tests, the grid points on eight survey lines 3m apart in the direction of the railway were designed and six at the same distance apart in the slope direction at site-A. At these grid points portable dynamic cone penetration was made. And at site-B, it was made at 0.5m intervals at 27m length along top of embankment slope.

4 DISTRIBUTION CHARACTERISTICS OF STRENGTHS ON EMBANKMENT SLOPE

4.1 Distribution of soil strengths in the vertical direction

At site-A, mean of all the data from the portable dynamic cone penetration test for strength $N_c$ at site-A is shown in Fig.4(a). In this figure a standard deviation $\sigma$ is also shown. Because of the light weight, $N_c$ is a very large value because the dynamic compressive pressure and the shear strength affect the penetration. It is also affected by grain size and characteristics. By abandoning these singular data, we obtain mean values for all the measured points as shown in Fig.4(b).

On the embankment surface up to a depth of 60cm, $N_c$ is 1~2 and the measured standard deviations are almost equal. But deeper than 60cm, $N_c$ increases linearly and the standard deviations also increase, but not linearly. For the section from the surface up to a depth of 60cm, therefore

$$N_c=1.63m+6.3$$  \hspace{1cm} (4.1)

where $\sigma$ is 1.00 and $m$ is a constant. For the section of 60~600cm,

$$N_c=0.054h+21.2m+6.3$$  \hspace{1cm} (4.2)

and the standard deviation is a function of the depth $h$.

$$\sigma=0.023h+0.03$$  \hspace{1cm} (4.3)

Correlation coefficients for Eq(4.2) and (4.3) are 0.97 and 0.98 respectively.

Neglecting the constant 0.03 in Eq(4.3), because of its small value, the strength at depths greater than 60cm can be expressed by Eq(4.4) from Eq(4.2) and Eq(4.3).

$$N_c=0.054h+0.023m+1.21$$  \hspace{1cm} (4.4)
slope. Many points have values $N_r > 7$. It seems that this shows variations in compaction at the time of embankment construction.

The average strength over the whole slope is nearly constant, in the range of $N_r = 4$ to $8$.

Change of soil strength $N_r$ to distance is measured for further examination as follows.

5 DEPENDENCE OF DISTANCE ON SOIL STRENGTH OF EMBANKMENT SLOPE

We analyze dependence of distance on soil strength of embankment slope by autocorrelation.

Assuming a random variation $x(t)$, an autocorrelation $r(t)$ is given by the formula.

$$r(t) = \lim_{T \to \infty} \frac{1}{T} \int_{-T/2}^{T/2} x(t)x(t + \tau) \, dt$$  \hspace{1cm} (5.1)

A discrete expression of Eq.(5.1) over a distance $l$ and data $x_n (n=0,1,2,\ldots,N-1)$ are given by a sequence of numerous $n$.

$$r_n = \frac{1}{l} \sum_{i=0}^{l-1} x_{n+i}$$ \hspace{1cm} \hspace{1cm} (5.2)

Then, normalizing Eq.(5.2) by $n$ at $j=0$,

$$\rho_j = \frac{1}{\sigma_j^2} \sum_{i=0}^{l-1} x_{n+i} x_{n+j}$$ \hspace{1cm} \hspace{1cm} (5.3)

This $\rho_j$ is called an autocorrelation coefficient for discrete data.

Examples of soil strength change in the railway direction at site-B are shown in Fig.7. In this figure, the change of soil strength is at the deep point is larger than at the shallow one.

Seeking an autocorrelation coefficient $\rho_j$ of railway direction at site-B, Fig.8 is gained. The tendency of the one at site-B is just about the same. Autocorrelation coefficient $\rho_j$ tends to decrease
as distance \( l \) is longer, and approximation formula is obtained as follows:

\[
\rho (l) = \exp(-Al^{-\nu})
\]  \hspace{1cm} (5.4)

where \( A \) is a constant. This formula shows the autocorrelation coefficient of railway embankment is similar one of the natural sedimentary ground.

Eq.(5.4) shows the autocorrelation coefficient \( \rho \) is smaller in proportion to the depth \( z \) and soil strength \( N_c \) of embankment. \( N_c \) is in proportion to depth \( z \) in Figs.4 and 5. Therefore, the relation between constants \( A \) and soil strength \( N_c \) as to each depth \( z \) as shown Fig.9. Marks ○ and ● shows at site-A and site-B respectively at Fig.8. The constant \( A \) and soil strength \( N_c \) are mutually related, and the two relation are given as follows:

\[
A = 0.0074N_c + 0.0163
\]  \hspace{1cm} (5.5)

The broken line shows 95% trust area in Fig.9.

Therefore, from Eq.(5.4) and Eq.(5.5), the autocorrelation coefficient \( \rho \) and soil strength \( N_c \) can be expressed as follows:

\[
\rho (l) = \exp(-0.0074N_c + 0.0163) \left( \frac{1}{l^{\nu}} \right)
\]  \hspace{1cm} (5.6)

6 EVALUATION OF SOUNDING INTERVAL FOR SOIL STRENGTH

6.1 Collapse shape of railway embankment during rainfall

From the data of the 67 embankment collapses mentioned above, the mean and histograms of the collapse shape at collapse depth \( z \), width \( B \) along the railway, length \( L \) along the slope and solid volume \( V \) are shown in Table 1 and Table 10. The data in Table 1 and Table 10 include various embankment collapse types from a surface failure to a deep circular slip collapse; the scales range from a relatively small failure to a large collapse whose failure soil volume was more than 2000m³.

6.2 Evaluation of sounding interval for soil strength

Because the autocorrelation coefficient of embankment strength is a function of distance \( l \) and soil strength \( N_c \) as shown in Eq.(5.6), relation between autocorrelation coefficient and soil strength \( N_c \) are gained as a parameter for distance \( l \).

For instance, the mean and standard deviation of collapse width \( B \) on the railway embankment are \( B=20.7m \) and \( \sigma =14.3m \) respectively, and when the range of collapse width dispersion is 6.4m to 35.0m (\( B \pm 1 \sigma \)) is considered, the relation between soil strength \( N_c \) and autocorrelation coefficient \( \rho \) is shown in Fig.10. In Fig.11, histogram is a theoretical formal distribution of soil strengths \( N_c \) as shown in Fig.2, and the broken lines are 95% reliable lines in Eq.(5.6).

Because soil strength \( N_c \) is dispersed as mentioned above, autocorrelation coefficient \( \rho \) changes against \( N_c \). Minimum value of autocorrelation coefficient \( \rho \) is 0.61 at \( N_c=80 \) as
Fig. 11: Relation between \( N_c \) and autocorrelation \( \rho \) considering collapse width \( (B \pm 1 \sigma) \).

Fig. 12: Relation between \( N_c \) and autocorrelation \( \rho \) shown in Fig. 11.

It shows that the autocorrelation coefficient is maintained over \( \rho = 0.61 \) at a dispersion \( 1 \sigma \) of soil strength \( N_c \) and collapse width \( B \). In other words, if a sounding is done at intervals of 35m, that is \( (B + 1 \sigma) \), the autocorrelation for the investigated soil strength remains more than \( \rho = 0.61 \).

The autocorrelation coefficient \( \rho (l) \) of the soil strength for longitudinal distance of railway is shown in Fig. 12 in which the dotted lines are 95% confidence levels. If the soil strength \( N_c \) and the distance become large, \( \rho (l) \) gradually decreases. By using Fig. 12, we can determine the necessary sounding interval for an arbitrary autocorrelation coefficient according to embankment soil strength \( N_c \). For example, we obtain a sounding interval \( L=50m \) for autocorrelation coefficient \( \rho = 0.7 \) and soil strength \( N_c = 5.4 \).

7. CONCLUSIONS

We discussed the spatial characteristics of soil strength in the railway embankments using a portable dynamic cone penetration and also analyzed the distance dependency in railway direction of soil strength by the autocorrelation coefficient. The results obtained in this paper are summarized as follows:

1. Soil strength \( N_c \) is 1 to 2 on the embankment surface up to a depth 60cm. But deeper than 60cm, the soil strength increases nearly in proportion to the depth.

2. Horizontal distribution of soil strengths is almost constant. Autocorrelation coefficient \( \rho (l) \) of the soil strength for distance is shown in Eq. (5.6) which is expressed as a function of soil strength. It gradually decreases as the soil strength and the sounding interval increase.

3. Autocorrelation coefficient \( \rho (l) \) is shown in Fig. 11 for dispersions of collapse width \( B \) and soil strength \( N_c \). In the case of a variation \( 1 \sigma \) for both \( B \) and \( N_c \), for example, the minimum \( \rho (l) \) may be 0.61 at the sounding interval of \( l=35m \).

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Static penetration testing
Site characterization of a lacustrine very soft Rio de Janeiro organic clay

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ABSTRACT: Site characterization for the design of an embankment on a very soft organic deposit included a number of in situ and laboratory tests. Soil profiling was well identified by means of piezocone and continuous index tests. Magnitude and rate of settlements were computed respectively by means of good quality oedometer tests and piezocone dissipation tests. Stability analyses made use of corrected vane measurements. Actual field behaviour presently monitored shows that parameters and hypotheses adopted were realistic.

1. Introduction

An embankment on a very soft organic clay was planned to be constructed in Barra da Tijuca, Rio de Janeiro to be the headquarters of SENAC, an educational centre.

A comprehensive site investigation for the design of the embankment on soft clay started in 1994 and included a number of in situ tests such as SPT, vane and piezocone tests, as well as laboratory investigations comprising index tests, oedometer and triaxial tests. It is the purpose of this paper to describe this site investigation and how it was used in design.

Embarkment construction was planned to be performed in two stages. Prefabricated drains with 1.7m spacing in a triangular pattern were adopted to accelerate settlements. The first stage construction reached an embankment height of 2.4m. A detailed instrumentation to monitor settlements and pore pressures (Almeida et al, 1996) has been installed at the site.

2. SOFT CLAY LAYERS

2.1 SPT boreholes and water content data

The embankment has been built in an area of about 100,000 m² as shown in Figure 1 and location of SPT boreholes is also indicated in the same figure. A typical cross section, shown in Figure 2, indicates a soft clay deposit with an average depth of 4.5m and a

Figure 1 - Embankment site and SPT boreholes

water level close to the surface. A fine to medium sand layer was found below the clay deposit.

Water content data of four boreholes, shown in Figure 3, indicate three soft clay layers: a 3m thick organic clay crust followed by two clay layers, each about 4m thick. Atterberg limits and water content presented in Figure 4 show less clearly the three layers. Water content is consistently greater than liquid limit, thus the liquidity index is greater than unity. Overall data of plasticity index including data
of other boreholes in the soft clay deposit show \( q_c \) in the range 100% - 250%. Water content measurements showed values as high as 600% at the crust and average values of \( w = 200\% \) and \( w = 120\% \) at the two clay layers below the crust.

### 2.2 Piezocone data

Soil profiling was performed using a piezocone equipment developed at COPPE (Sousa et al., 1987).

Five piezocone soundings have been performed in this area. One of the tests was performed with a probe provided with two pore pressure sensors (Danziger et al., 1997).

Results of PZ23, Figure 5, are typical of the site, with corrected cone resistance \( q_c \) and pore pressure \( u \) measured at cone face in the range 100-350 kPa.

Results at PZ11, Figure 6, show clearly a sand lens about 0.5m thick, which was, however, not found in other soundings. The normalized pore pressure ratio \( N_p \) is shown in Figure 7 for PZ23 and
Below the peat crust values of \( B_s \) are, in the range 0.5-1.0 for PZ3. These are typical site values, which are also close to values obtained for Saupui and Sergipe clays (Sandroni et al, 1997). With exception of the sand layer, \( B_s \) values for PZ11 are higher than for PZ3. Values of \( q_t \) and \( B_s \) suggest less clearly the existence of three layers noticed in water content data.

3. CLAY COMPRESSIBILITY DATA

Samples used in laboratory tests were obtained in two field programmes using respectively 100 mm and 125 mm diameter piston samplers. The diameters of the oedometer tests are shown in Table 1. All specimens were 20 mm high. In each vertical specimen were taken in three different depths with the purpose of settlement calculations.

<table>
<thead>
<tr>
<th>Sampler</th>
<th>Oedometer No.</th>
<th>Specimen</th>
<th>Quality (*)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>50</td>
<td>10</td>
<td>A</td>
</tr>
<tr>
<td>125</td>
<td>71</td>
<td>7</td>
<td>B</td>
</tr>
</tbody>
</table>

(*) A = very good to excellent; B = good to fair; C = poor

Quality of the specimens can be assessed based on the ratio \( \Delta \varepsilon = \varepsilon_s \) equal to the change in voids ratio to reach the in situ stress \( \sigma_{v0} \) divided by the voids ratio at \( \sigma_{v0} \), following the sample disturbance criteria proposed by Lame et al (1997). The assessment of the sample disturbance shown in Table 1 indicates that 52% of specimens are of very good to excellent quality, and 30% are good to fair. Oedometer test data of a good quality specimen, Figure 8, shows clearly the high compressibility of this clay.

4. CLAY CONSOLIDATION DATA

4.1 Measured coefficient of consolidation

Piezocene dissipation tests interpreted with Houbby and Teh (1983)'s solution produced coefficients of horizontal consolidation \( c_h \) for the normally consolidated condition as shown in Figure 10. These are about three times higher than corresponding values measured in oedometer tests, assuming a ratio \( c_h/c_v \) between horizontal and vertical coefficients of consolidation equal to 1.5. Values of \( c_v \) from piezocene tests were adopted in settlement calculations, following accumulated local experience (Almeida and Ferreira, 1992).

Figure 6 - Piezocene data for PZ11

Figure 7 - Pore pressure ratio for PZ3 and PZ11
4.2 In situ coefficient of consolidation

A settlement analysis using Aseoka’s method has been performed to obtain the in situ horizontal coefficient of consolidation \( c_h \) and the degree of consolidation \( U \). The analysis is similar to that performed by Almeida and Ferreira (1992) and takes into account drain smear and the combined radial and vertical drainage (assuming a ratio \( c_v/c_h = 1.5 \)).

A summary of \( c_h \) and \( U \) results for four settlement plates is shown in Table 2. A reasonable agreement is noticed between \( c_h \) from CPTU data (Figure 10) and those obtained from settlement analysis (Table 2).

### Table 2: \( c_h \) and \( U \) from settlement analysis

<table>
<thead>
<tr>
<th>Settlement plate</th>
<th>( c_h ) ( \times 10^3 ) m²/s</th>
<th>( U ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP3</td>
<td>11.8</td>
<td>91</td>
</tr>
<tr>
<td>SP7</td>
<td>9.8</td>
<td>88</td>
</tr>
<tr>
<td>SP10</td>
<td>11.9</td>
<td>88</td>
</tr>
<tr>
<td>SP19</td>
<td>15.4</td>
<td>94</td>
</tr>
</tbody>
</table>

5. CLAY STRENGTH

5.1 In situ measured strength

Undrained strengths \( S_u \) were measured in different locations using mainly a vane borger equipment. Results in SP2 presented in Figure 11 show the expected trend of \( S_u \) increasing with depth. \( S_u \) values measured in UU triaxial are slightly smaller than vane strengths. Subsequent \( S_u \) measurements (Souza et al., 1997) using an electric vane borger developed at COPPE with a tension cell close to the vane blades confirmed \( S_u \) vane strength shown here.
Sensitivity $S_v = 5.0$ measured in vane tests is in the upper range of Brazilian soft marine clays. Clay friction angle $\phi$ measured in CIU triaxial tests for the normally consolidated condition is in the range $40^\circ - 45^\circ$, which is consistent with the organic nature of this clay.

5.2 Field mobilized strength

The vane correction proposed by Azzaouz et al. (1983) was applied to measured vane strength. The design strength profile shown by a dashed line in Figure 11 is the corrected vane strength. These are in close agreement with critical state strength values and vary from 5 to 15 kPa. Stability analyses showed to be necessary the use of berms and geotextile to assure a factor of safety $F_s = 1.3$. During construction undue accumulation of fill at one particular site location resulted in a local failure. Analysis of the failure yielded $F_s = 1.03$, thus suggesting that a realistic $S_v$ profile was adopted in design.

6. CORRELATIONS WITH PIEZOCONE DATA

Empirical cone factors computed using uncorrected $S_v$ measured in vane tests are shown in Figure 12 and show a wide range of data. The average value for $N_1$ is about 9. The typical $N_1$ value for Brazilian clays (Almeida, 1990) is about 13, but the present clay is more compressible than the other clays.

The ratio between preconsolidation stresses $\sigma_{pc}'$ measured in oedometer tests and pore pressures measured with piezocone tests is given in Figure 13 for close boreholes. It is noticed that the Mayne and Holtz (1989) proposal is in the upper range of measured values. A similar observation can be drawn between the ratio $\sigma_{pc}' / (q_1 - \sigma_u)$ measured here and the Mayne and Holtz (1989) proposal, as shown in Figure 14.
7. CONCLUSIONS

A well planned and detailed site characterization was carried out for the design of an embankment on a very soft organic clay. Piezocone and index tests were quite useful for the definition of the clay layering. Magnitudes of settlements were computed from good quality odometer test data. Piezocone dissipation tests adequately interpreted were used for settlement rate computations, thus defining vertical drain spacing. Settlement measurements confirmed consolidation and compressibility parameters adopted in design. Corrected vane strength was used for stability computation and an economic design was reached as field evidences have indicated.

The specification and supervision of the site investigation by the team in charge of the embankment design has been a key factor for the overall success of the project. The same team is now monitoring the embankment construction. Embankment performance confirms overall hypotheses adopted in design.

8. ACKNOWLEDGEMENTS

The author is indebted to all those at COPPE involved in the site investigation reported here, particularly J. Campinho, S.G. Garcia, S. Iório, E.N. Paiva and H. G. Souza. Prof. W. A. Lacerda provided important advice throughout the project. Special thanks are directed to resident engineers A. S. Rodrigues and I. N. Souza and particularly to F. Bitencourt from SENAC for his overall interest and support.

REFERENCES


Methods of pile design based on CPT and SPT results

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Purdue University, West Lafayette, Ind., USA

ABSTRACT: Pile bearing capacity depends on the same factors as the static penetration resistance measured in a cone penetration test (CPT) and, to a lesser extent, as the dynamic penetration resistance measured in a standard penetration test (SPT). Thus, many methods have been proposed relating pile bearing capacity to CPT tip resistance and sleeve friction, and SPT blow count. These methods, although conceptually quite similar, differ in many important details, such as the definition of ultimate bearing capacity, the way the penetration resistance values are defined for use in calculations, the relative importance of penetration resistance values above and below the pile base for base resistance calculations, and the types of soils and piles for which the methods were developed. In this paper, we review some of the empirical methods proposed to estimate pile bearing capacity from penetration tests.

1 INTRODUCTION

The construction of heavier, taller structures in increasingly marginal sites, and the technological improvements of equipment used to install piles make the economics of deep foundations ever more attractive. This has increased the need for better assessing pile capacity (Salgado, 1995). However, accurate determination of deep bearing capacity is still a challenging problem, as there are many uncertainties related to the models and parameters used in calculations.

When the cone penetrometer test, CPT, was first used, it appeared to offer an effective way to define deep foundations because the test itself was seen as a scaled-down load test on a pile (Salgado, 1995). Thus, several expressions were developed by empirical correlations with CPT data and load tests to define a direct relationship between the ultimate pile capacity $Q_{ut}$ and the CPT cone resistance, $q_c$, and sleeve friction, $f_s$. Similar expressions have been developed for $Q_{ub}$ in terms of SPT blow count.

Most of the empirical methods have been developed for specific pile and soil conditions. It is the intent of this paper to present and summarize some methods that use CPT and SPT results to estimate the ultimate bearing capacity of piles. Most of the expressions are presented within the same framework for easy comparison.

2 $Q_{ub}$ PREDICTION FROM CPT RESULTS

The ultimate bearing capacity of a single pile, $Q_{ub}$, may be predicted by means of empirical equations using CPT cone resistance and sleeve friction. Generally, $Q_{ub}$ is expressed as the sum of ultimate base resistance, $Q_b$, and ultimate shaft resistance, $Q_s$:

$$Q_{ub} = Q_b + Q_s = (q_b A_b) + \Sigma (q_s A_s)$$

where $q_b$ = base resistance; $q_s$ = shaft or skin resistance within a layer of a single soil type, labeled $i$, penetrated by the pile; $A_b$ = area of pile base; $A_s$ = pile shaft area interfacing with layer $i$.

According to several authors (e.g., Aoki & de Alencar Veloso, 1975; Buitendam & Gianessi, 1982; de Ruiter & Beringen, 1979; Lopa & Laprovitera, 1988; Philippinat, 1980; Schmertmann, 1978), shaft and point resistances of a single pile are proportional to cone resistance. Franke (1989), and Jamtlihovski & Lancelotta (1988) proposed expressions only for point resistance. The relationships are defined through conversion factors. Values proposed in the literature for these empirical factors were developed by comparison between load test results and CPT results, so they may vary according to the protocol followed for performing and interpreting the load tests. It is recommended that empirical factors should be applied only under conditions similar to those under which they were determined. Table 1 shows pile types and soil
Table I. Some methods of pile design and their applicability.

<table>
<thead>
<tr>
<th>Method</th>
<th>Soil</th>
<th>Pile type</th>
<th>Cxk criteria</th>
<th>Pn (in-place compressive)</th>
<th>CP1/SPT specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aslak et al. (1977)</td>
<td>Sand, silt &amp; clay</td>
<td>Bored, driven, concrete, bored piles</td>
<td>Van der Veen's method</td>
<td>NA</td>
<td>SPT: E&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;0.72&lt;/sub&gt; (psi weight hammer)</td>
</tr>
<tr>
<td>Bluck &amp; Kuhl (1986)</td>
<td>Cohesive &amp; non-cohesive soils</td>
<td>Driven, driven, driven piles</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Brand &amp; Vardar (1981)</td>
<td>Sand, silt &amp; clay</td>
<td>Bored, driven, concrete, bored piles</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Bucarcet &amp; Gierasch (1982)</td>
<td>Gravel, sand, silt &amp; clay</td>
<td>Bored, driven, concrete, bored piles</td>
<td>NA</td>
<td>E&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;0.5&lt;/sub&gt;, E&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;1.0&lt;/sub&gt;</td>
<td>NA</td>
</tr>
<tr>
<td>Demir &amp; Quinones (1982)</td>
<td>Sand, silt &amp; clay</td>
<td>Driven, driven, concrete, bored piles</td>
<td>Van der Veen's method</td>
<td>F&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;0.4&lt;/sub&gt;, F&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;0.1&lt;/sub&gt;</td>
<td>SPT: E&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;0.72&lt;/sub&gt; (psi weight hammer)</td>
</tr>
<tr>
<td>De Ruyter &amp; Remmers (1979)</td>
<td>Sand &amp; clay</td>
<td>Driven piles</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Fresco (1988)</td>
<td>Sand, silt &amp; clay</td>
<td>Bored piles</td>
<td>V&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;0.05&lt;/sub&gt;</td>
<td>2</td>
<td>NA</td>
</tr>
<tr>
<td>Fukumoto (1990)</td>
<td>Gravel, sand, silt &amp; clay</td>
<td>Bored piles</td>
<td>V&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;0.05&lt;/sub&gt;</td>
<td>2</td>
<td>SPT: E&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;0.62&lt;/sub&gt; to 0.85</td>
</tr>
<tr>
<td>Junek &amp; Lancellotta (1988)</td>
<td>Sand</td>
<td>Bored piles</td>
<td>V&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;0.05&lt;/sub&gt;</td>
<td>2</td>
<td>SPT: E&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;0.72&lt;/sub&gt; (psi weight hammer)</td>
</tr>
<tr>
<td>Lopez (1980)</td>
<td>Sand, silt &amp; clay</td>
<td>Driven piles</td>
<td>Van der Veen's method</td>
<td>NA</td>
<td>SPT: E&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;0.72&lt;/sub&gt; (psi weight hammer)</td>
</tr>
<tr>
<td>Migliavacca (1980)</td>
<td>Gravel, sand, silt &amp; clay</td>
<td>Driven &amp; bored piles</td>
<td>Maximum settlement rate in the 15 320 curve</td>
<td>2</td>
<td>NA</td>
</tr>
<tr>
<td>Prato (1983)</td>
<td>Stiff clay</td>
<td>Small-diameter jack-in, driven &amp; bored piles</td>
<td>V&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;0.05&lt;/sub&gt; for jack-in &amp; bored piles</td>
<td>NA</td>
<td>SPT: penetration rate: 0.7 - 20 mm/s</td>
</tr>
<tr>
<td>Greb &amp; O'Neil (1983)</td>
<td>Sand</td>
<td>Large-diameter bored piles</td>
<td>V&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;0.05&lt;/sub&gt;</td>
<td>2</td>
<td>SPT: E&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;0.55&lt;/sub&gt;</td>
</tr>
<tr>
<td>Schuttert (1978)</td>
<td>Sand &amp; clay</td>
<td>Driven in-place piles</td>
<td>NA</td>
<td>F&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;2.25&lt;/sub&gt; with plastic tip</td>
<td>SPT: Fugro and Beekhoven friction-cone penetrometer</td>
</tr>
<tr>
<td>Tanaka &amp; Orihata (1980, 1985)</td>
<td>Gravel, sand &amp; silt</td>
<td>Large-diameter bored piles</td>
<td>V&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;0.05&lt;/sub&gt;</td>
<td>2</td>
<td>SPT: E&lt;sub&gt;n&lt;/sub&gt;&lt;sub&gt;0.72&lt;/sub&gt; (psi weight hammer)</td>
</tr>
</tbody>
</table>

NA: Not available.

1 Some energy ratios/SPT equipment information from Clayton (1990).
conditions for which each method applies, \( Q_{\text{eq}} \) criteria, the recommended safety factors for each method, and some of the CPT and SPT specifications used in the acquisition of the data on which the correlations are based. \( Q_{\text{eq}} \) criteria are based on the load \( Q \) versus settlement curve obtained from load testing. The general form of these prediction methods for estimating \( q_b \) and \( q_u \) is:

\[
q_b = c_b q_*
\]

\[
q_u = \sum c_q q_i
\]

where \( c_b \) = factor to convert from \( q_* \) to base resistance; \( c_q \) = factor to convert from \( q_* \) to shaft resistance for layer \( i \); \( q_i \) = representative cone resistance at pile base level; \( q_j \) = representative cone resistance for layer \( j \).

When the soil is fairly homogeneous, the expression for \( q_* \) may be reduced to:

\[
q_* = c_q f_s
\]

where \( c_q \) = factor to convert from \( q_* \) to shaft resistance; \( f_s \) = representative cone resistance value over the full length of the pile.

Price & Wardle (1982) and Schmertmann (1978) proposed expressions for relating shaft resistance, \( q_* \), to cone sleeve friction, \( f_s \), as follows:

\[
q_* = c f_s
\]

where \( c \) = factor to convert from \( f_s \) to shaft resistance.

In order to write equations in non-dimensional form, a reference stress is defined as \( p_r = 100 \text{ kPa} = 0.1 \text{ MPa} \) \( \times 1 \text{ kgf/cm}^2 = 1 \text{ tsf} \). Likewise, a reference length is defined as \( L_r = 1 \text{ m} = 100 \text{ cm} = 1000 \text{ mm} = 40 \text{ in} = 3.28 \text{ ft} \).

### 2.1 Aoki & de Alencar Velloso's method

Based on load tests and both CPT and SPT results, as well as their own \( N_q-q_* \) correlations, Aoki & de Alencar Velloso (1975) defined the \( c_b \) and \( c_q \) resistance factors as follows:

\[
c_b = \frac{1}{F_1}
\]

\[
c_q = \frac{F_2}{F_1}
\]

where \( F_1, F_2, \) and \( c_b \) are empirical factors given in Tables 2 and 3; \( F_1 \) and \( F_2 \) are functions of pile type; and \( c_b \) depends on soil type. Aoki & de Alencar Velloso (1975) calculate the allowable bearing capacity, \( Q_{\text{eq}} \), using a global factor of safety, \( p_{r_k} \). The \( K, n_b, \) and \( n_q \) factors appearing in Table 2 are discussed in section 3.

### Table 2. Factors \( K, c_b, n_b, n_q, c_q \) for determining \( q_b \) and \( q_u \)

<table>
<thead>
<tr>
<th>Soil type</th>
<th>( K_{p_b} )</th>
<th>( n_b )</th>
<th>( K_{p_q} )</th>
<th>( n_q )</th>
<th>( c_b )</th>
<th>( n_q )</th>
<th>( c_q )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>6</td>
<td>0.35</td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>110</td>
<td>1.4</td>
<td>60</td>
<td>1.4</td>
<td>4.0</td>
<td>0.65</td>
<td>4.0</td>
</tr>
<tr>
<td>Silty sand</td>
<td>80</td>
<td>2.0</td>
<td>53</td>
<td>1.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Silty sand with clay</td>
<td>70</td>
<td>2.4</td>
<td>53</td>
<td>2.4</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Clayey sand with silt</td>
<td>50</td>
<td>2.8</td>
<td>53</td>
<td>2.8</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Clayey sand</td>
<td>-</td>
<td>6.0</td>
<td>3.0</td>
<td>3.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Sandy silt</td>
<td>5.5</td>
<td>3.2</td>
<td>4.8</td>
<td>3.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Sandy silt with clay</td>
<td>4.5</td>
<td>2.8</td>
<td>3.8</td>
<td>3.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Silty sand</td>
<td>6.0</td>
<td>3.0</td>
<td>4.8</td>
<td>3.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Clayey silt with sand</td>
<td>2.5</td>
<td>3.0</td>
<td>3.8</td>
<td>3.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Clayey silt with silt</td>
<td>3.4</td>
<td>3.4</td>
<td>3.0</td>
<td>3.4</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Sandy silt with clay</td>
<td>3.5</td>
<td>2.4</td>
<td>4.8</td>
<td>4.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Sandy silt with silt</td>
<td>3.0</td>
<td>2.8</td>
<td>3.0</td>
<td>4.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Silty silt with sand</td>
<td>3.3</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Silty silt with silt</td>
<td>2.2</td>
<td>4.0</td>
<td>2.5</td>
<td>5.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Clay</td>
<td>2.0</td>
<td>4.0</td>
<td>2.5</td>
<td>4.5</td>
<td>1.2</td>
<td>6.1</td>
<td>1.2</td>
</tr>
</tbody>
</table>

(1) Aoki & de Alencar Velloso (1975); (2) Lopes & Lopes Neto (1983); (3) Pfeiffer (1982); (4) Huerta (1990); (5) Sudo (1980).

### Table 3. Factors \( F_1 \) and \( F_2 \) for determining \( q_b \) and \( q_u \)

<table>
<thead>
<tr>
<th>Pile type</th>
<th>( F_1 )</th>
<th>( F_2 )</th>
<th>( F_3 )</th>
<th>( F_4 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aoki &amp; de Alencar Velloso (1975)</td>
<td>1.35</td>
<td>1.35</td>
<td>1.35</td>
<td>1.35</td>
</tr>
<tr>
<td>Lopes &amp; Lopes Neto (1983)</td>
<td>2.5</td>
<td>7.0</td>
<td>6.1</td>
<td>5.2</td>
</tr>
<tr>
<td>Friction</td>
<td>2.5</td>
<td>5.0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Steel</td>
<td>1.75</td>
<td>3.5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Precast concrete</td>
<td>1.75</td>
<td>3.5</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

### 2.2 Bustamante & Gianessi’s method (LCPC method)

After numerous CPT tests compared with load tests on several pile types (Table 1), Bustamante & Gianessi (1982) published their design method using factors depending on both soil and pile types (Tables 4 and 5).
The empirical factor \( c_d \) is expressed as:

\[
\frac{c_d}{\alpha_2} = 1
\]

(8)

where \( \alpha_2 \) is given in Table 4. Table 5 contains the factor \( \alpha_2 \) to avoid the use of unconservative shaft resistance values, these authors recommend upper limits for \( q_s \) (Table 4). The piles are classified as:

1. Category IA: bored piles (dry and slurry methods); CFA piles; barrettes; and micropiles installed with low injection pressures;
2. Category IB: bored piles with steel casing;
3. Category IIA: driven or jacked precast and prestressed concrete piles;
4. Category III: driven or jacked steel piles;
5. Category IIIA: driven grouted or rammed piles;
6. Category IIIB: small- and large-diameter injected piles (high injection pressures);
7. Group I: bored piles with casing and piles in Category IA;
8. Group II: driven cast-in-place piles and piles in Categories IIA, IIIB, IIIA, and IIIB.

The \( q_{su} \) used to calculate \( q_s \) according to Bustamante & Gianessi (1982), should be the equivalent cone resistance \( q_{su} \) calculated in four steps, as follows:

1. Smooth the \( q_s \) versus depth curve by eliminating sharp peaks and valleys;
2. Calculate \( q_{su} \) as the average of the smoothed curve between a distance 1.5B above and 1.5B below the pile base, where B = shaft diameter;
3. Eliminate values of \( q_s \) higher than 1.3\(q_{su} \) both above and below the pile base, and lower than 0.7\(q_{su} \) above the pile base from the smoothed curve;
4. Calculate \( q_{su} \) as the average of \( q_s \) values between a distance 1.5B above and 1.5B below the pile base.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Factor ( c_d )</th>
<th>Limit value of ( q_{su} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>IA</td>
<td>IIA</td>
</tr>
<tr>
<td>Soft clay and mud</td>
<td>0.30</td>
<td>0.35</td>
</tr>
<tr>
<td>Moderately compact clay</td>
<td>10 to 50</td>
<td>0.35</td>
</tr>
<tr>
<td>Silty and loose sand</td>
<td>50</td>
<td>0.35</td>
</tr>
<tr>
<td>Compact to stiff clay and</td>
<td>50</td>
<td>0.35</td>
</tr>
<tr>
<td>compact silt</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil chalk</td>
<td>50</td>
<td>0.35</td>
</tr>
<tr>
<td>Moderately compact sand</td>
<td>50</td>
<td>0.35</td>
</tr>
<tr>
<td>and gravel</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weathered to fragmented</td>
<td>50</td>
<td>0.35</td>
</tr>
<tr>
<td>chalk</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compact to very compact</td>
<td>50</td>
<td>0.35</td>
</tr>
<tr>
<td>sand and gravel</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5. Factor \( c_d \) for determining \( q_s \) according to Bustamante & Gianessi (1982).

<table>
<thead>
<tr>
<th>Soil type</th>
<th>( q_{su} )</th>
<th>( \alpha_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Group I</td>
<td>Group II</td>
</tr>
<tr>
<td>Soft clay and mud</td>
<td>0.65</td>
<td>0.65</td>
</tr>
<tr>
<td>Moderately compact clay</td>
<td>0.55</td>
<td>0.55</td>
</tr>
<tr>
<td>Silt and loose sand</td>
<td>0.55</td>
<td>0.55</td>
</tr>
<tr>
<td>Compact to stiff clay and</td>
<td>0.55</td>
<td>0.55</td>
</tr>
<tr>
<td>compact silt</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil chalk</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>Moderately compact sand</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>and gravel</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>Weathered to fragmented</td>
<td>0.40</td>
<td>0.40</td>
</tr>
<tr>
<td>chalk</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compact to very compact</td>
<td>0.40</td>
<td>0.40</td>
</tr>
<tr>
<td>sand and gravel</td>
<td>0.40</td>
<td>0.40</td>
</tr>
</tbody>
</table>

2.3 De Ruiter & Beringen's method

According to de Ruiter & Beringen (1979), \( c_d = 1/3\sigma_3 \) for compression and 1/4000 for tension piles in sand. In \( q_s \) calculations in sand, \( c_d = 1 \) and \( q_s \) is the average cone resistance determined following the procedure proposed by Schwertmann (1978). For overconsolidated cohesionless soils, \( c_d \) is reduced to account for the strength reduction that these soils might experience during driving. Thus, for very gravelly coarse sands and sands with overconsolidation ratio OCR = 2 - 4, \( c_d = 0.68 \), for fine gravels and sands with OCR = 6 - 10, \( c_d = 0.5 \). In both cases, the limit value of \( q_{su} \) is 150\(P_B\).

For clays, \( c_d \) is given by the following expression:
\[ c_s = \frac{c_s^*}{N_s} \]  

where \( N_s \) = cone factor that assumes values in the 10-20 range; \( \alpha_1 = 0.5 \) for overconsolidated clays and 1.0 for normally consolidated clays. The value of the cone factor \( N_s \) should be determined locally for the soil in question. De Ruiter & Beringen (1979) recommended checking \( q_s/N_s \) against the results of undrained shear strength of the clay obtained from laboratory tests. The factor \( c_s \) for clays is given by:

\[ c_s = \frac{q_s}{N_s} \]  

According to de Ruiter & Beringen (1979), the final design value of \( q_s \) is the lowest of the following three values: the local sleeve friction, \( f_s \); the \( q_s \) estimated using \( q_{s1} \); and a limit value of 1.2\( f_s \). Similarly, the design \( q_s \) value is the lowest of the \( q_s \) estimated using \( q_{s1} \) and the \( q_s \) corrected according to OCR, and a limit value of 150\( f_s \).

2.4 Jamiolkowski & Lancellotta’s and Franke’s base resistance factors

For bored piles installed under well controlled conditions in dense and very dense sand, Jamiolkowski & Lancellotta (1978) proposed the \( c_s \) factor given by Table 6 for \( s/B > 0.05 \) and embedment ratio \( D/B > 8 \), where \( D \) = pile length embedded into a uniform cohesionless layer; \( B \) = base diameter. Franke (1989) recommended \( c_s = 0.2 \) for \( s/B = 0.10 \) in bored piles.

Table 6. Factor \( c_s \) for determining \( q_s \) of bored piles according to Jamiolkowski & Lancellotta (1988).

<table>
<thead>
<tr>
<th>( D/B )</th>
<th>( c_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>0.20</td>
</tr>
<tr>
<td>0.05</td>
<td>0.17</td>
</tr>
<tr>
<td>0.10</td>
<td>0.14</td>
</tr>
<tr>
<td>0.15</td>
<td>0.12</td>
</tr>
<tr>
<td>0.20</td>
<td>0.11</td>
</tr>
<tr>
<td>0.25</td>
<td>0.09</td>
</tr>
<tr>
<td>0.30</td>
<td>0.08</td>
</tr>
<tr>
<td>0.35</td>
<td>0.07</td>
</tr>
</tbody>
</table>

2.5 Lopes & Laprovittola’s factors

After examination of cases in the literature and their own load test-SPT correlations, Lopes & Laprovittola (1988) proposed modified values of \( F_1 \), \( F_2 \), and \( \alpha_1 \) for bored piles for use Aoki & de Alencar Velloso’s method. These factors are given in Table 2. Although only SPT results were available for their study, Lopes & Laprovittola state that the proposed factors can be used with CPT results.

2.6 Meyerhof’s method

Meyerhof (1983) introduces the critical depth factor, \( D/D_c \). \( D_c \) = critical depth. \( D_c \) is defined as the embedment depth below which the unit bearing capacity would not increase further with depth. \( D_c \) is a function of soil density. For short piles with \( B/D_c \leq 0.5 \) driven into fairly homogeneous sand, the ultimate base resistance is taken as \( q_{sb} \) and the factor \( c_s \) is given by:

\[ c_s = \frac{D}{D_c} \leq 1 \]  

For long piles driven through a weak layer and embedded in a firm sand deposit with \( H/B \geq 20 \), where \( H = \) thickness of deposit, \( q_{sb} \) is given by:

\[ q_{sb} = q_{s2} + \left( q_{s3} - q_{s2} \right) \frac{D}{H} \leq q_{s2} \]  

where \( q_{s1} \) and \( q_{s2} \) = limit base resistances in the upper weak layer and the lower bearing soil, respectively. The limit base resistance \( q_{s3} \) in a layer may be taken as the cone resistance \( q_c \) provided \( D = D_c = B \). The factor \( c \) varies from 8 for loose sand to 12 for dense sand, although it has been usually taken as an average value of 10. When \( H/B \geq 20 \) and the bearing layer overlies a weak deposit, Meyerhof (1983) proposed the following expression for \( q_{sb} \):

\[ q_{sb} = q_{s3} + \left( q_{s3} - q_{s2} \right) \frac{H}{B} \leq q_{s3} \]  

where \( q_{s1} \) and \( q_{s3} \) = limit base resistances in the upper bearing layer and the lower weak soil, respectively. \( H = \) distance between the pile base and the top of the lower weak soil layer.

For piles with \( 0.5 < B/B_c < 2 \), \( q_s \) is reduced using a factor \( n \) expressed as follows:

\[ n = \frac{B + 0.5B_c}{2B} \leq 1 \]  

where \( n = 1, 2, \) or \( 3 \), for loose, medium dense, or dense sand, respectively.
2.7 Philippouneat’s method

The factor \( c_p \) proposed by Philippouneat (1980) is given in Table 2. For heterogeneous soils, \( q_s \) is determined as:

\[
q_s = \frac{q_{o(A)} + q_{o(B)}}{2}
\]

(15)

where \( q_{o(A)} \) and \( q_{o(B)} \) are the average cone resistances within a distance 3B above and 3B below the pile base, respectively. When the \( q_s \) plot is too irregular, Philippouneat (1980) recommended eliminating extreme peaks as well as imposing the constraint \( q_{o(A)} \leq q_{o(B)} \) on \( q_{o(A)} \). The factor \( c_p \) is given by:

\[
c_p = \frac{c_{pa}}{F_p}
\]

(16)

where \( F_p \) and \( c_{pa} \) (Table 7 and 8) are factors that depend on pile and soil types, respectively. The upper limit of \( q_s \) is shown in Table 8 as \( q_s_{lim} \).

Table 7. Factor \( F_p \) for determining \( c_p \) according to Philippouneat (1980).

<table>
<thead>
<tr>
<th>Soil type</th>
<th>( F_p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay and calcareous clay</td>
<td>50</td>
</tr>
<tr>
<td>Silty sandy clay, and clayey sand</td>
<td>60</td>
</tr>
<tr>
<td>Loose sand</td>
<td>100</td>
</tr>
<tr>
<td>Medium dense sand</td>
<td>150</td>
</tr>
<tr>
<td>Dense sand and gravel</td>
<td>200</td>
</tr>
</tbody>
</table>

Table 8. Factor \( c_{pa} \) for determining \( c_p \) according to Philippouneat (1980).

<table>
<thead>
<tr>
<th>Contact pile-soil</th>
<th>( c_{pa} )</th>
<th>( q_s_{lim} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Precast, cast-in-place with driven/vibrated casing, and injected piles</td>
<td>1.25</td>
<td>1.2</td>
</tr>
<tr>
<td>Bored piles ((B/B_p &lt; 1.5))</td>
<td>0.85</td>
<td>1.0</td>
</tr>
<tr>
<td>Bored piles ((B/B_p &gt; 1.5))</td>
<td>0.75</td>
<td>0.8</td>
</tr>
<tr>
<td>H-section piles</td>
<td>1.10</td>
<td>1.2</td>
</tr>
<tr>
<td>L-section and steel box piles</td>
<td>0.60</td>
<td>0.5</td>
</tr>
<tr>
<td>Steel</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bored piles with casing</td>
<td>0.30</td>
<td>0.25</td>
</tr>
<tr>
<td>Concrete or steel</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Piles installed with a water or slurry jet</td>
<td>Neglect ( q_s )</td>
<td></td>
</tr>
</tbody>
</table>

2.8 Price & Wardle’s method

Based on comparisons made between load tests and eight CPT tests in a layer of stiff clay (London clay), Price and Wardle (1982) proposed the following values of \( c_v \) and \( c_u \) as a function of pile type:

- \( c_v = 0.30 \) for jacked-in piles \((c_o = 0.56 \text{ when locked-in loads are ignored})\);
- \( c_v = 0.35 \) for driven piles;
- \( c_u = 0.62 \) for jacked piles;
- \( c_u = 0.53 \) for driven piles;
- \( c_u = 0.49 \) for bored piles;
- no \( c_v \) values proposed for bored piles.

2.9 Schmertmann’s method

Schmertmann (1978) calculates \( q_s \) with \( c_p = 1 \) and \( q_s \) as an average cone resistance within an influence zone extending from 8B above to 0.7B below the pile base. When a mechanical penetrometer is used in clay, the factor \( c_p = 0.60 \).

The recommended upper limit for \( q_s \) is 150\( \text{Pa} \) in sand and 100\( \text{Pa} \) in very silty sand. The factor \( c_{sv} \) for sand varies with pile type as follows:

- \( c_{sv} = 0.608 \) for open-end steel tube piles;
- \( c_{sv} = 0.612 \) for precast concrete and steel displacement piles;
- \( c_{sv} = 0.018 \) for vibro and cast-in-place displacement piles with steel driving tube removed, and timber piles.

In both \( q_s \) and \( q_o \) calculations, an upper limit for \( q_s \) of 300\( \text{Pa} \) is recommended. For cohesive soils, \( q_s \) is estimated using \( f_s \). The factor \( c_{pa} \) for displacement piles is obtained from Table 9. In this case, \( f_s \) is taken as the average undrained sleeve friction in the clay.

Table 9. Factor \( c_{us} \) for determining \( q_s \) according to Schmertmann (1978).

<table>
<thead>
<tr>
<th>Pile type</th>
<th>( c_{us} )</th>
<th>( q_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td></td>
<td>0.97</td>
</tr>
<tr>
<td>Concrete &amp; timber</td>
<td></td>
<td>0.97</td>
</tr>
<tr>
<td>Precast, cast-in-place with driven/vibrated casing, and injected piles</td>
<td>0.75</td>
<td>0.97</td>
</tr>
<tr>
<td>Bored piles ((B/B_p &lt; 1.5))</td>
<td>0.75</td>
<td>0.97</td>
</tr>
<tr>
<td>Bored piles ((B/B_p &gt; 1.5))</td>
<td>0.75</td>
<td>0.97</td>
</tr>
<tr>
<td>H-section piles</td>
<td>1.00</td>
<td>0.97</td>
</tr>
<tr>
<td>L-section and steel box piles</td>
<td>1.00</td>
<td>0.97</td>
</tr>
<tr>
<td>Concrete or steel</td>
<td></td>
<td>0.26</td>
</tr>
<tr>
<td>Piles installed with a water or slurry jet</td>
<td>Neglect ( q_s )</td>
<td></td>
</tr>
</tbody>
</table>

2.10 Tejahman & Gwizdoz’s method

Tejahman & Gwizdoz (1988) developed a method that expresses the base resistance of large diameter piles (0.5 \( B/B_p \leq 2 \)) in terms of a hyperbolic curve. For \( B/B_p = 0.10 \), \( q_s \) in sandy soils is expressed as follows:
\( q_b = \frac{P_A}{\alpha_s \cdot B_R} \) \( \left( \frac{B_R}{1000B_A} + \frac{1}{\alpha_s \cdot Q_{lim}} \right) \) \( \alpha_s = \left[ 28 - \left( \frac{0.4 Q_{lim}}{P_A} \right) \right] \) \( \alpha_s = 3 - \frac{B}{B_R} \) \( q_{lim} \) = limit base resistance. Eq. (18) is valid only within the range \( 20P_A < q_{lim} < 50P_A \). The \( q_{lim} \) is given by:

\[ q_{lim} = \frac{1}{50} \cdot B \cdot F_{base} \cdot q_{pen} \cdot h \]  

where \( q_{pen} \) = cone resistance as function of penetration depth, \( h \); \( L \) = pile length; and the factor \( F_{base} \) is obtained from Table 10. The authors appear to limit \( q_b \) values used in (20) to 200 \( P_A \). For large diameter piles and \( x/B < 0.10 \), \( q_b \) is given by the expression:

\[ q_b = \frac{1}{L} \int_0^L q_{pen} \cdot dh \]  

where \( F_{base} \) is obtained from Table 10.

\( q_b = \frac{n_b N_b}{n_i N_i} \) \( q_{lim} = \Sigma n_i N_i \)

where \( n_b \) = factor to convert SPT blow count to base resistance; \( n_q \) = factor to convert SPT blow count to shaft resistance for layer \( i \); \( N_b = \) representative \( N_{SPT} \) value at the pile base; \( N_q = \) representative \( N_{SPT} \) value along the pile shaft in layer \( i \). When the soil is fairly homogeneous, the expression for \( q_b \) may be reduced to:

\[ q_b = n_i N_i \]

where \( n_i \) = shaft resistance factor using SPT results; \( N_i = \) representative \( N_{SPT} \) value along the pile shaft.

Other expressions for \( q_b \) and \( q_{lim} \) as function of SPT results in forms different from (22) and (23) are also found in literature (e.g., Brand & Tucker, 1984; Shioti & Fukai, 1985).

3.1 Aoki & de Alencar Velloso’s method

Aoki & de Alencar Velloso (1975) proposed \( n_b \) and \( n_i \) values as follows:

\[ n_b = \frac{K}{F_1} \]  

\[ n_i = \frac{\alpha_i K}{F_2} \]

where \( K = \) empirical factor depending on soil type (Table 2). Factors \( F_1, F_2 \), and \( \alpha_i \) are the same as those used in the CPT version of the method (Section 2.1). When factors \( n_b \) and \( n_i \) given by (25) and (26) are used, \( N_b \) = average of the three \( N_{SPT} \) values closest to the pile base; and \( N_q = \) average \( N_{SPT} \) value along the shaft in layer \( i \), excluding those used to calculate \( N_b \).

3.2 Bazaraa & Kurkur’s method

From Egyptian experience, Bazaraa and Kurkur (1986) found correlations for \( q_b \) and \( q_{lim} \) according to pile type and quality of installation. The proposed pile categories and factors are given in Tables 11 and 12. For Raymond piles in cohesionless soil, \( q_b \) is calculated by the following equations:

\[ q_b = (0.4 + 0.01N_i)p_A \]  

for \( B/B_A < 0.5 \).
\[ q_v = (0.8 + 0.02N_p) \frac{B_h}{B_h} \quad \text{for} \quad B_h/B_p > 0.5 \]  

(28)

\[ N_p \] should be calculated as the average \( N_{ERP} \) value within 1B below and 3.75B above the pile base, and \( N_4 \) as the average \( N_{ERP} \) value along the pile. The upper limit of \( N_4 \) is 50 for cohesionless soils.

Table 11. Pile categories for selection of \( n_a \) and \( n_b \) according to Bazaram & Kurkur (1986).

<table>
<thead>
<tr>
<th>Pile category</th>
<th>Pile description</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Preparent piles using high-pressure mortar injection</td>
</tr>
<tr>
<td>II</td>
<td>Driven piles, Bauer piles with careful execution and Preparent piles with low injection pressure</td>
</tr>
<tr>
<td>III</td>
<td>Bored piles with careful execution and Bauer piles with some defects in execution</td>
</tr>
<tr>
<td>IV</td>
<td>Bored piles with some defects in execution</td>
</tr>
</tbody>
</table>

Table 12. Factors \( n_a \) and \( n_b \) according to Bazaram & Kurkur (1986).

<table>
<thead>
<tr>
<th>Pile categories</th>
<th>Cohesionless soil</th>
<th>Cohesive soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>I and II</td>
<td>( n_a = 0.025 \frac{P_h}{P_0} ), ( n_b = 0.025 \frac{P_h}{P_0} )</td>
<td>( n_a = 0.035 \frac{P_h}{P_0} ), ( n_b = 0.035 \frac{P_h}{P_0} )</td>
</tr>
</tbody>
</table>
| For \( B_h/B_p \) < 0.5                            
| For \( B_h/B_p \) > 0.5 |
| III and IV   | \( n_a = 0.005 \frac{P_h}{P_0} \), \( n_b = 0.15 \frac{P_h}{P_0} \) | \( n_a = 0.005 \frac{P_h}{P_0} \), \( n_b = 0.15 \frac{P_h}{P_0} \) |
| For \( B_h/B_p \) < 0.5                            
| For \( B_h/B_p \) > 0.5 |

3.3 Briaul & Tucker’s method

Briaul & Tucker (1984) developed a method for determining \( q_v \) and \( q_a \) as a function of pile settlement using SPT results and the elastic modulus, \( E_p \), and dimensions of the pile. This method models both \( q_v \) versus \( s \) and \( q_a \) versus \( s \) curves as hyperbolas and considers the effect of residual stresses. Residual stresses result from rebound movements of the pile after each blow during driving. The ultimate pile capacity is the sum of the residual loads and the load measured in a load test with initial zeroed instrumentation. Applying the proposed equations for \( s/B = 0.10 \), \( q_v \) and \( q_a \) are given by:

\[ q_v = \frac{1}{K_a} \left( \frac{0.1}{q_{a_{max}} - q_{a_{min}}} + q_{a_{min}} \right) \]  

(29)

where:

\[ K_a = 18.684 \left( \frac{N_s}{100} \right)^{0.85} \frac{P_h}{B_p} \]  

(30)

\[ K_v = 200 \left( \frac{N_s}{100} \right)^{0.87} \frac{P_h}{B_p} \]  

(31)

\[ q_{a_{max}} = 19.75 \left( \frac{N_s}{100} \right)^{0.87} \frac{P_h}{B_p} \]  

(32)

\[ q_{a_{min}} = 0.224 \left( \frac{N_s}{100} \right)^{0.87} \frac{P_h}{B_p} \]  

(33)

\[ q_{a_{min}} = 5.57 \Omega \frac{P_h}{B_p} \]  

(34)

\[ q_{a_{min}} = q_{a_{min}} A \frac{A_4}{A_4} \]  

(35)

\[ \Omega = \frac{K_p \rho}{A_4} \]  

(36)

\[ \Omega = \frac{K_p \rho}{A_4} \]  

(37)

In (31) - (37), \( N_s = \) average \( N_{ERP} \) value within a zone extending from 4B above to 4B below the pile base; \( N_s = \) average \( N_{ERP} \) along the shaft length, \( L \); \( A_4 \) and \( A_4 \) = pile base and shaft areas, respectively; \( A_4 \) = cross-sectional area (\( A_4 = A_4 \) for piles with uniform cross-section); \( \rho = \) density; and \( p = \) shaft perimeter.

3.4 Decourt & Quaresma’s method

Decourt & Quaresma (Decourt, 1982) proposed the following expression for \( q_v \):

\[ q_v = \left( \frac{N_s}{30} + 0.1 \right) P_h \]  

(38)

where \( N_s \) is taken as no less than 3 and no greater than 15 for precast, Franki, and Strauss piles and no greater than 50 for bored piles. These limits were based on the observation that shaft resistance was bounded from below for soils with low blow counts and from above for soils with high blow counts. Table 2 contains values of \( n_b \). The method provides values for clayey and sandy soils based on experience with residual soils.

3.5 Hirayama’s method

Based on Japanese experience with the SPT, Hirayama (1990) recommended coefficients for preliminary design of bored piles, as given in Table 2. For applying those coefficients, \( N_s \) is calculated as the average \( N_{ERP} \)
value within a zone of influence extending from 1Bb above to 1Bb below the pile base. When there is a weak
layer below this influence zone, the range should be extended up to 2Bb below the pile base. The limit
value of \( q_b \) is 2PA for sand and 1.5PA for clay.

3.6 Lopes & Lavoristerra’s factors

Lopes & Lavoristerra (1988) recommended modified values of \( F_1, F_2, \alpha_1, \) and \( K \) for bored piles for use in
Aoki & de Alencar Velloso’s method. Those factors are given in Table 2.

3.7 Meyerhof’s method

For short piles driven into fairly homogeneous sand, Meyerhof (1983) proposed a factor \( n_b \) given by:

\[
n_b = 4P_a \left( \frac{D}{B} \right) \leq 4P_a
\]

(39)

\( n_b \) is defined by this author as the average \( N_{ufr} \) value near the pile base. For pile diameters within the
range \( 0.5 < B/B_b < 2 \), \( q_b \) is reduced using the factor \( n_b \) given in section 2.6. The value of \( n_b \) for silts is taken as
two thirds of \( n_b \) value for sands. The factor \( n_b \) is taken as 0.02P\( \alpha \) and \( N_b \) is defined as the average \( N_{ufr} \) value
along the embedded pile length. For small-displacement piles, such as H-piles, \( n_b \) = 0.01P\( \alpha \) (Meyerhof, 1976).
The upper limit of \( q_b \) is between 0.02P\( \alpha \) and 0.04P\( \alpha \) for bored piles.

3.8 Reese & O’Neill’s base resistance factors

According to Reese & O’Neill (1989), \( n_b = 0.60P_a \) for bored piles in sands with diameters in the range
\( 0.52 < B/B_b < 1.2 \) and SPT blow count in the range \( 5 < N_b < 60 \). The upper limit of \( q_b \) is 45P\( \alpha \), and \( N_b \) is
calculated as the average \( N_{ufr} \) value within a distance 2Bb below the pile base. For bored piles with
\( B/B_b > 1.27 \), \( n_b \) is given by:

\[
n_b = 0.762P\alpha \left( \frac{B_b}{B} \right)
\]

(40)

Reese & O’Neill (1989) estimate \( q_b \) for bored piles in sands using the \( \beta \)-method.

3.9 Shioi & Fukui’s method

Shioi & Fukui (1982) recommended use of \( n_b = P_a \) for bored piles in sand and \( q_b = 0.6S_a \) for cast-in-place piles in
clayey soil, where \( S_a \) is the undrained shear strength of the soil. For driven piles, \( n_b \) is a function of the
embedded length in the bearing layer, \( D \), and the pile diameter, according to the expression:

\[
n_b = \left(1 + 0.4 \frac{D}{B} \right) P_a \leq 3P_a
\]

(41)

which takes the form:

\[
n_b = 0.6P\alpha \left( \frac{D}{B} \right) \leq 3P_a
\]

(42)

for open-end steel tube piles. For soils with cohesive and cohesionless layers, \( q_b \) is determined using the expression:

\[
q_b = n_{ba} N_{ba} A_{ba} + n_{bc} N_{bc} A_{bc}
\]

(43)

where \( n_{ba} \) and \( n_{bc} \) factors to convert SPT blow count to shaft resistance; \( N_{ba} \) and \( N_{bc} \) average \( N_{ufr} \) values
along the pile shaft, and \( A_{ba} \) and \( A_{bc} \) - pile shaft areas embedded in sand and clay layers, respectively.

The factors \( n_{ba} \) and \( n_{bc} \) have the following values:

- \( n_{ba} = 0.01P_a \) for bored piles in sandy soil;
- \( n_{bc} = 0.05P_a \) for cast-in-place piles in clayey soil;
- \( n_{ba} = 0.02P_a \) for driven piles;
- \( n_{bc} = 0.05P_a \) for bored piles in sandy soil;
- \( n_{bc} = 0.1P_a \) for cast-in-place piles in clayey soil, and for driven piles;

4 CONCLUSIONS AND RECOMMENDATIONS

Penetration tests are well suited for calculations of pile bearing capacity because penetration resistance is
influenced by the same factors as pile capacity, particularly in the case of the CPT. A number of
methods were reviewed for relating pile base and shaft resistances to \( q_b, \) \( f_b, \) and \( N_{ufr} \). These methods are all
empirical, relying to a great extent on pile load testing experience. Additionally, the methods differ in some
important ways: (1) the definition adopted for \( q_{sho}, \) (2) the type of equipment used to obtain \( N_{ufr}, q_b, \) and \( f_b, \)
(3) the selection of \( N_{ufr} \) or \( q_b \) values to use in the empirical equations; (4) the relative importance of \( N_{ufr} \)
or \( q_b \) values above and below the pile base for \( q_b \) estimates; (5) the importance of embedded length and
Critical depth; (6) soil types and conditions for which the methods are applicable. These factors are not
alwasy specifically provided in the references, but are important to consider in selecting a method for designing piles for axial loads. There appear to be opportunities for research in assessing the importance of such factors in pile capacity calculations.

5 REFERENCES


Evaluation of liquefaction potential and lateral deformations using CPT and field case histories

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ABSTRACT - In this study, a series of field case histories are collected from literature. The sites are selected such that they liquefied during earthquakes and also CPT's have been performed in those sites. These data are sorted to see effect of confining pressure on the liquefaction potential and now charts are developed to assess liquefaction potential for different fine content. A chart for evaluation of large deformation is also proposed based the data available to the authors.

1 INTRODUCTION

For many years, most laboratory and field liquefaction assessment studies focused on the mechanisms of pore-water pressure build-up due to cyclic shear loading, and useful engineering correlations were developed to predict excess pore pressure and liquefaction triggering (Yousif and Perkins 1978, Seed 1979, Dobry et al. 1982, Seed et al. 1983). More recently, the focus has switched to the evaluation of ground deformation and flow sliding. These efforts include: (1) development of laboratory- based techniques to determine the in-situ undrained steady- state or residual shear strength of the soil, for use in analysis of flow failure and ground deformation (Castro et al. 1983, Castro 1987, Dobry et al. 1991, Dobry and Baziar 1992; Ishihara 1993); (2) correlation of residual shear strength backcalculated from available case histories to field penetration resistance (Seed 1987, Davis et al. 1988; Seed and Harder 1990, Robertson et al. 1992, Stark and Mesri 1992; Ishihara 1993); and (3) development of empirical correlations to predict the permanent lateral ground deformation (Yousif and Perkins 1987, Burtlett and Yousif 1992, Baziar and Dobry, 1995).

Nearly all field work performed as a part of the studies referenced above, and those others referred to within these studies, entailed the use of standard penetration test (SPT). Recently, it has been recognized that CPT offers certain advantages over SPT for use in liquefaction assessment studies as well as prediction of lateral ground deformation, including: a) CPT provides a continuous penetration resistance record which allows identification and delineation of thin (approximately 10 cm or more in thickness) liquefiable sand or silt layers. b) CPT is more standardized and its results are less operator dependent than SPT. c) CPT results are more reproducible than SPT, d) CPT allows a more comprehensive subsurface investigation in a similar span of time, thus, offering a more economical alternative as compared with SPT.

As pointed out by others, the primary reasons why cone penetration test (CPT) has not been used extensively for prediction of liquefaction assessment and lateral ground deformation are: (1) The lack of a sample for visual soil classification and/or other routine tests, and (2) limited data base of CPT results involving case histories of liquefaction and lateral ground deformation. The number of field case histories with available CPT data has significantly increased during past 15 years.

Specifically, liquefaction case histories with available CPT data have been presented by Seed et al. (1983), Yousif and Bonetti (1983), Robertson and Campaanita (1985), Shibata and Toparka (1983), Robertson (1990), Mitchell and Tseng (1990), Mahmodian et al. (1991), Mitchell et al. (1992), Stark and Mesri (1993), among others. On the other hand, Castro et al. (1982), Seed (1987), Castro (1987), Yousif and Perkins (1987) Davis et al. (1988), Seed and Harder (1990), Dobry and Baziar (1992), Robertson et al. (1992), Stark and Mesri (1992), Ishihara (1990, 1993), Burtlett and Youd (1992, 1993), and Baziar and Dobry (1995), among others, have reported case histories of lateral spread and/or large ground deformations as well as correlation of residual shear strength backcalculated from available case histories and field penetration resistances.

The present paper proposes new charts to determine liquefaction assessment of clean sand, silty sand and sandy silt based on initial effective stress (e35) and corrected CPT tip resistance (qC). Also, a new chart for evaluation of large deformation potential is presented based on CPT results and case histories.
EVALUATION OF LIQUEFACTION POTENTIAL BY CPT

Seed et al. (1985) proposed a procedure to estimate the liquefaction potential of sandy soils based on equivalent cyclic stress ratio (CSR) and corrected blow count of SPT, \(\text{N}_{100}\).

The penetration resistance from the CPT, like SPT, is influenced by soil density, soil structure, concentration, ageing, stress state, and stress history and thus, can be used to evaluate the liquefaction potential of soils (Robertson and Campanella 1985, Ishihara 1993, Stark 1995). SPT, \(\text{N}\) - values must be corrected for effective overburden stress, hammer type, release system, sampler configuration, and drill rod length (Seed et al. 1985). However based on CPT advantages, many researchers have been preferred to use CPT results in liquefaction assessment (Robertson and Campanella 1985, Shibata and Teparkska 1988, Robertson 1990, Mitchell and Tang 1990, Mahanoodravedan et al. 1991, Mitchell et al. 1994, Ishihara 1990, 1993).

Recently Stark and Olson (1995) collected reliable case histories in which CPT data is recorded in liquefied and nonliquefied sites for many earthquakes. Also, they presented a procedure to evaluate liquefaction potential based on corrected CPT tip resistance \(\text{q}_{\text{c}}\) and seismic shear - stress ratio (S.S.R), induced by earthquakes. They proposed charts for liquefaction evaluation potential for clean sand \((0.25 < D < 0.5\, \text{mm}, F_{C} < 5\%\), silty sand \((0.1 < D < 0.5\, \text{mm}) < 0.2\%\, 5\% < F_{C} < 15\%\), sandy silt \((D < 0.1\, \text{mm}), F_{C} < 35\%\).

In each chart, liquefaction zone is separated by a boundary from non liquefaction zone (Fig 1).

In this paper, the authors have used CPT case histories that were collected by Stark and Olson (1995) and have presented new charts for liquefaction assessment based on CPT tip resistance, \(\text{q}_{\text{c}}\), and effective overburden pressure \(\sigma_{\text{vo}}\).

LIQUEFACTION POTENTIAL OF CLEAN SAND

Fig. 3 presents a chart for evaluation of liquefaction potential on clean sand based on \(\text{q}_{\text{c}}\) versus effective overburden pressure, \(\sigma_{\text{vo}}\). From the field data, a boundary line is drawn between liquefied and nonliquefied sites. This boundary defines a relationship between the effective overburden pressure and CPT, \(\text{q}_{\text{c}}\) values for clean sand. This boundary represents a reasonable lower bound of the liquefied sites.

Fig. 3 also indicates that the proposed liquefaction potential relationship for clean sand is in good agreement with the field case history data except for two cases. These two cases are from the 1976 Tangshan Earthquake (Shibata and Teparkska 1988) at Sounding Y - 14 and Y - 15. It should be noted that the fine content of
there are another two cases which must be studied more. The
two nonliquefied case histories, plotted adjacent to the
proposed line on the side of liquefied zone, are from the
1989 Loma Prieta Earthquake (Kwon et al. 1992
Mitchell et al. 1994) at readings Poet-1 and Poet-7.

Nevertheless, presence of non liquefied sites in the
liquefied zones indicates that the proposed boundary may
be a little conservative.

![Fig. 3. Relationship between Effective Overburden Pressure and q(L)-value for Liquefaction Triggering in Silt with Clean Sand.](image)

**4 LIQUEFACTION POTENTIAL OF SILTY SAND**

Fig. 4 presents a chart for evaluation of liquefaction
potential of silty sand. From the field data, reported by
Stark and Olson, a boundary separating liquefied sites
from non liquefied sites is established. Similar to the case
for clean sand, this boundary defines a relationship
between the effective overburden pressure and CPT, q(L)
-values for silty sand.

It is seen that several nonliquefied case histories are
plotted under proposed boundary. Two of them are from
the 1976 Tangshan Earthquake (Shihata and Tepa-antaka
1984) at readings T29 and T36. For these cases, the
fine contents are not available and only median grain size
diameters were used to determine the appropriate soil
category. In other words, the fine contents in these case
histories may be more than 35%. The other six cases are
from 1989 Sanguenay Earthquake (Tuttle et al. 1990).

It should be noticed that in all these cases, median grain
diameters are equal to 0.1 mm. It is speculated that the
diameter of grains may have influenced the CPT results.

However, the proposed boundary for silty sand can be
considered as a conservative boundary.

![Fig. 4. Relationship between Effective Overburden Pressure and q(L)-value for Liquefaction in Silt with Fine Sand.](image)

**5 LIQUEFACTION POTENTIAL OF SILTY SAND TO SANDY SILT**

Fig. 5 presents a chart for evaluation of liquefaction
potential on silty sand to sandy silt. From the field data, reported by Stark and Olson, a boundary separating
liquefied sites from nonliquefied sites is established. This
boundary defines a relationship between the effective
overburden pressure and CPT, q(L)-values for silty sand
to sandy silt.

![Fig. 5. Relationship between Effective Overburden Pressure and q(L)-value for Triggering Liquefaction in Silt with Silty Sand to Sandy Silt (q(L)=35).](image)

There are five non liquefied cases that are plotted under the
proposed boundary. Four of them are in soils with fine
content of 50% or greater and another one with 49% fine
content. It is believed that the large value of fine content
causes an undrained or partially drained condition during
performing CPT, which probably resulted in an
underestimation of the q(L)-value (Stark and Olson 1995).

Therefore, it is reasonable to screen all cases which their
fine content is 50% or more. The silty sand to sandy silt
cases which have fine content between 35 to 50% are
compiled in Fig (6).

It is seen that proposed boundary is now more reasonable
with field case histories and suitable for expressing the
relationship between effective overburden pressure and q(L)-value. It should be noticed that the only case
which is out of boundary has fine content more than 50% and hence no need to be included in these data.

Fig. (7) presents a chart for evaluation of liquefaction potential for silty sand to sandy silt with fine content 50% or more. This chart shows that three non liquefied cases are still plotted under proposed boundary. One of these cases, is the Middle School Site during 1971 Haicheng Earthquake (Ambraseys et al. 1986). In this case, the soil layer that was reported to have liquefied, had a clay size fraction of more than 20%. This large value of clay size fraction probably accounts for the low $q_\text{p}$ value.

Fig. 6. Relationship between Effective Overburden Pressure and $q_{\text{p}}$ value for Triggering Liquefaction inSites with Silty Sand to Sandy Silt ($35 < C < 50$).

6 EVALUATION OF LARGE GROUND DEFORMATION FROM CPT RESULT

Effects of liquefaction of a loose saturated sandy site or slope during future earthquake is an important and complex engineering problem. Liquefaction effects include foundation bearing capacity failures, flow slides of dams, and lateral spread in widely sloping deposits including permanent horizontal ground deformations from a few inches to more than 30 ft.

More recently many researchers have switched to the evaluation of permanent ground deformation and potential of flow sliding using field penetration resistance (Seed et al. 1988, Seed and Harder 1990, Robertson et al. 1997, Sum and Moni 1992, Ishihara 1993, Bazant and Dobry 1995). Due to importance of effective overburden stress on residual shear strength of liquefiable soil, it seems that ground deformation potential is mainly dependent on effective overburden pressure. Thus, it is reasonable to find a relationship between overburden pressure $\sigma_\text{v}'$ and CPT tip resistance $q_{\text{p}}$ for cases of sandy soils which were subjected to large deformation.

Table 1 presents case histories of ground deformation and lateral spread (DH > 4 ft) in liquefied sites where CPT data and effective overburden pressure are available to the authors. The values of $q_{\text{p}}$ were also directly measured in the field.

Table 1. Case Histories for Large Deformation Induced by Liquefaction and CPT Data.

Two other cases which are plotted under the proposed boundary, are from the 1971 San Fernando Earthquake in Arcadia Hall California at Saundintg 18 - C. These two cases had a fine content of more than 50%. However, at present time, there is insufficient data for sandy silt with fine content greater than 50%. Therefore, this chart will be verified or modified by other researchers in the light of more field data available in the future.
reported in Table 1. From the field data, a boundary is drawn to limit the value of $q_{sf}$ which causes large deformation.

In other words, the points on the left side of the curve are the sites which liquefied during earthquake and also showed lateral displacement more than four feet.

It is well understood that to validate this chart, it is necessary to have more data for the field that liquefied but did not show ground deformation. Therefore, there is no doubt that with more data field available in the future, this chart will be valid or modified.

7 CONCLUSIONS:

CPT is more suitable than SPT to evaluate liquefaction potential and to predict lateral ground deformation.

In this paper, based on laboratory investigation, the authors have special emphasis on the effect of the initial effective overburden stress on the assessment of liquefaction potential.

As a result, new charts are proposed to determine liquefaction potential of clean sand ($0.25 < D_{50} (mm) < 2.0$ and $F_{C} (%) < 5$), silty sand ($0.1 < D_{50} (mm) < 0.25$ and $5 < F_{C} (%) < 35$) and silty sand to sandy silt ($D_{50} (mm) < 0.10$ and $55 < F_{C} (%) < 50$) based on $q_{sf}$ and corrected CPT tip resistance $q_{ct}$.

A new chart is also presented to evaluate large ground deformation potential (lateral spread greater than 4 ft and flow failures) using CPT results and case histories.

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Cone penetration testing in the Mid-Mississippi River Valley

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ABSTRACT: The Cone Penetration Test (CPT) offers several advantages over the Standard Penetration Test (SPT) including better precision and repeatability, lower cost, and a continuous record of penetration resistance with depth. These advantages are especially important in alluvial deposits where stratigraphy can vary significantly in both the vertical and horizontal direction over small distances. Cone Penetration Tests were conducted in conjunction with Standard Penetration Tests and laboratory testing at 13 sites in the Mid-Mississippi River Valley (MMRV) as part of geotechnical site investigations. All CPT data was obtained using standard (10 cm² tip area) 10 ton electric cones at a penetration rate of 2 cm/sec.

This paper compares CPTU (CPT with pore-water pressure readings) and SPT field test results from soundings and borings that were conducted adjacent to one another (within approximately 3 meters). The results allowed comparison of soils in the MMRV with published CPT correlations for soil classification, CPT-SPT conversion, and the cone factor Nc.

1. INTRODUCTION

The general geology of the Mid-Mississippi River Valley consists of cretaceous and tertiary deposits of stratified sand, silt, and clay. On bluffs outside of the local erosional features of the river, pleistocene age loessial deposits are encountered. Only 2 of the 13 sites contained loess. Surficial deposits along the river consist of Holocene age alluvium.

The locations of the 13 sites are shown in Figure 1.

CPTU data, boring logs, and laboratory test results from the sites were obtained from project files. The data used in this paper was originally obtained as part of geotechnical site investigations. Therefore, not all sites provided adequate data for use in the comparisons. All data was used as appropriate. For instance, at one site, the pore-water pressure element of the cone was ineffective because of smearing of a very soft, high plastic clay deposit at the ground surface. Therefore pore-water pressure data was not available for the soundings at this site. The CPT data could then only be plotted on the soil classification charts which did not use tip resistance corrected for excess pore-water pressure during penetration (qt).

CPTU soundings were conducted in accordance with ASTM D-3441. The pore-water pressure element of the cones used were located directly behind the cone tip. When appropriate, CPTU tip resistance was corrected for effective overburden stress and for excess pore-water pressure during penetration.

SPT borings were conducted in accordance with ASTM D-1586. In all cases, mud rotary drilling techniques were used below the water table to provide valid blowcount results. When appropriate, blowcounts was corrected for effective overburden
stress and for energy delivered to the drillstem. In some cases, direct energy measurements were obtained.

2. SOIL CLASSIFICATION

One of the disadvantages of the use of the CPT is the lack of soil samples. Therefore, a soil classification chart is required to estimate soil type from CPTU results. The soil classifications estimated from classification charts recommended by Robertson and Campanella (1983), Olsen (1984), Campanella and Robertson (1988), and Robertson (1990) were compared with visual classifications made on samples retrieved during SPT and undisturbed sampling. This comparison was limited to soil classification charts typically used by the authors and is not intended to be all encompassing.

Figure 2 presents a part of the CPTU data plotted on the respective charts. In order to make the plots legible, data from only four of the sites are shown. The CPTU results were averaged over the same depth interval as the SPT or undisturbed sample. Data points at contacts of different geological conditions (clay/sand contacts) were discarded. The percentages of CPTU data being classified correctly using data from all of the sites were:

<table>
<thead>
<tr>
<th>Source</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Robertson and Campanella</td>
<td>63.1%</td>
</tr>
<tr>
<td>Olsen (1984)</td>
<td>67.7%</td>
</tr>
<tr>
<td>Campanella and Robertson</td>
<td>79.9%</td>
</tr>
<tr>
<td>Robertson (1990)</td>
<td>77.7%</td>
</tr>
</tbody>
</table>

The majority of points that were misclassified by the classification charts were in the transition zones. In some cases, clays and silty clays plotted in zones that were indicated to be for silts and silty sands. It should be noted that several data points that consisted of sand with thin clay stringers and clay with thin sand stringers did not plot correctly on all four of the classification charts. This is probably attributed to the averaging of the CPTU results over the depth interval of the sample used to make the comparison.

Campanella and Robertson (1988) and Robertson (1990) both account for excess pore-water pressure during penetration in the classification chart by using $q_c$. The charts using $q_c$ appear to improve the classification at these sites. Given the fact that the clay and silty clay data plotted in the transition zones, the area of the classification charts indicating sandy silts and silt may need to include silty clays for better classification of MMRV soils. The two plots using $q_c$ performed better in the transition zones, but that remained the location of most of the discrepancies. It should be noted that the loosest deposits in this area are borderline silty clay to clayey silt material. Therefore, some of the apparent misclassifications in the transition zones may have actually provided reasonable classification of these borderline soils. Despite this, it appears that soil classification charts from Campanella and Robertson (1988) and Robertson (1990) provide reasonably good classification of MMRV soils.

3. CPT-SPT CONVERSION

Foundation design is often based on empirical relationships utilizing SPT blowcounts. However, the advantages of the CPT over the SPT make CPT testing an increasingly popular technique to characterize sites. Therefore, in order to utilize the large experience base for the SPT, it is necessary to relate CPT tip resistance to SPT blowcount. The CPT-SPT conversions (based upon median grain size, $D_{50}$) from Robertson and Campanella (1983, 1984, 1986), Andrus and Youd (1989), Kulfwv and Crow (1990), and Stark and Olson (1995) are compared with ratios of CPT tip resistance to SPT blowcount obtained at the 13 sites in the MMRV.

The comparison is presented in Figure 3. Unfortunately, the data were more limited than originally anticipated. A full sieve test was required to accurately obtain $D_{50}$, and much of the laboratory test results included only fines content (percent passing the #200 sieve). The 28 data points from our records are shown with data presented in the literature. The darkest best fit line is based only upon our data and not those from others the best fit line is not intended to be a new conversion curve. Despite the scatter in the data, the correlations proposed by Robertson and Campanella (1985) and Stark and Olson (1995) provide a reasonable conversion for the MMRV soils.

4. CONE FACTOR $N_c$

The CPT is often utilized in soft cohesive soils because of the inability of the SPT to accurately characterize these soils. The cone factor $N_c$ is used to estimate the undrained shear strength, $s_u$, of cohesive soils from CPT tip resistance as follows: $s_u = (q_t - c_u)N_c$. However, a wide range of $N_c$ values have been reported in literature to estimate $s_u$. Results from laboratory unconfined compression, triaxial tests (UU and CU), field remold tests, pocket penetrometers, and torque data were used to calibrate $N_c$ at the MMRV sites.

Data from the pocket penetrometers and torvanes proved to be too variable and unreliable for use in calibration ($N_c$ ranged from 0 to 106). The field remold test, a field unconfined compression test, as well as the laboratory unconfined compression test also proved to be highly variable. Based on nine
FIGURE 2
CPT COMPARISON OF SOIL CLASSIFICATION

1 SENSITIVE FINE GRAINED
2 ORGANIC MATERIAL
3 CLAY
4 SILTY CLAY TO CLAY
5 CLAYEY SILT TO SILTY CLAY
6 SANDY SILT CLAYEY SILT
7 SILTY SAND TO SANDY SILT
8 SAND TO SILTY SAND
9 SAND
10 GRAVELLY SAND TO SAND
11 VERY STIFF FINE GRAINED (+)
12 SAND TO CLAYEY SAND (+)
(+) OVERCONSOLIDATED OR CEMENTED

LEGEND
△ CLAY (CL & CH)
● SAND (SP & SM)
■ SILT (ML)
1 SENSITIVE FINE GRAINED
2 ORGANIC SOIL — PEATS
3 CLAYS — CLAY TO SILTY CLAY
4 CLAYEY SILT TO SILTY CLAY
5 SILTY SAND TO SANDY SILT
6 CLEAN SAND TO SILTY SAND
7 GRAVELLY SAND TO SAND
8 VERY STIFF SAND TO CLAYEY SAND (+)
9 VERY STIFF, FINE GRAINED (+)
(+) OVERCONSOLIDATED OR CEMENTED
triaxial test data indicated a range of $N_k$ from 12 to 26. The majority of our test results indicate that a cone factor in the range of 17 to 20 is appropriate for MMRV soils. Site-specific testing is still recommended for determining $N_k$.

5. CONCLUSIONS

The large number of CPTU correlations available in the literature can lead to uncertainty in their use. Based upon the data currently available, it is apparent that additional data will be required to confirm the preliminary correlations presented herein. The use of the CPTU as a standard tool in this area is still relatively new but is becoming increasingly popular. As future explorations are performed, the data will be added to our initial database. The correlations of soil classification presented by Campanella and Robertson (1985) and Robertson (1990), CPT-SPT conversion presented by Robertson and Campanella (1995) and Stark and Olson (1995), and an $N_k$ factor of 17 to 20 provide reasonable results for utilization and interpretation of CPTU data in MMRV soils. This paper is intended to aid geotechnical investigators in the Mid-Mississippi River Valley by providing guidance on the selection of CPT correlations for use in future investigations, and not meant to replace site-specific testing and specifications.

6. REFERENCES


Horizontal cone penetration testing

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ABSTRACT: In order to find the relationship between the cone resistances measured in horizontally and vertically aligned cone penetration tests, a test series has been performed in a 2 m. diameter rigid wall calibration chamber using a 30 mm. cone. This calibration chamber contains an unsaturated uniform sand, which can be prepared at different densities. It is found that the horizontal cone resistance is higher than the vertical cone resistance at a given point, whilst the side friction is lower horizontally than vertically. A simple cavity expansion model is used to explain the ratio of horizontal over vertical cone resistance.

1 INTRODUCTION

The cone penetration test (CPT) has been used extensively over the last decades to measure in situ soil properties, especially in delta areas where the upper layers consist of soft sediments. Measurements are traditionally taken from ground level in the vertical direction, to gain information about stratification and soil properties. With the introduction of mechanized tunnel boring in the Netherlands a need for soil data along the alignment of the tunnel has arisen. As most of these tunnels are built in heavily stratified soils, with strong variation and local irregularities, an extensive soil survey would be needed to gain sufficient information from vertical measurements only. Measurements from ground level are further complicated in urbanized areas where buildings are present over the tunnel alignment. To overcome these problems it has been proposed to perform cone penetration tests from the tunnel boring machine in a horizontal direction. In that way continuous information about the soil directly before the tunnel boring machine can be gained, which can complement the information gained from vertical soil surveys.

Although the equipment used to perform a vertical cone penetration test (VCPT) can easily be converted to allow its use in a horizontal cone penetration test (HCPT), the measurements obtained cannot be interpreted as easily. In a VCPT the horizontal effective stress $\sigma_h$ acts around the body of the cone, whilst the vertical effective stress $\sigma_v$ acts in the direction of penetration. This stress state around the cone is used implicitly in most theoretical models, e.g. Vesic (1972), Carter (1986), Salgado (1997), and empirical models, e.g. Schmertmann (1975), Hoekby (1988). The initial stress state around the cone in a HCPT differs radically however and is further complicated when one takes into account that most soils have been deposited in a layerwise manner. So it is to be expected that the measurements obtained by a HCPT differ from those obtained in a VCPT.

In order to find the relationship between the measurements from HCPT and VCPT or the relationship between measurements from HCPT and soil properties, a test series has been set up in a calibration chamber in which both a horizontal and a vertical CPT could be performed in the same sand sample. In addition to these tests a simple cavity expansion model is used to explain the differences between horizontal and vertical CPT.

2 TEST SETUP

The test series has been executed in a large diameter rigid wall calibration chamber with a diameter of 1.9 m. and a height of 3 m. In the wall of this chamber two holes have been made, at 2.01 m. and 2.23 m. from the top of the tank. Both openings are sealed with ball valves of 37 mm. internal diameter. This allows a standard 36 mm. cone to penetrate the sand, without sand spilling through the opening along the push rods. A filter bed at the bottom of the tank, in combination with several vibratory units along the tank wall, allows the sand in the tank to be fluidized and compacted,
in order to prepare samples at different densities. This results in a sand level between 91 and 103 cm, from the top of the tank, corresponding to relative densities between $R_d = 0.172$ and 0.749. For a schematic layout of the calibration chamber see Figure 1. The sand used in this tank is a uniformly distributed Oosterscheldesand with a $d_{50} = 180 \mu m$, for a detailed distribution see the sieve curve in Figure 2.

Measurements were carried out using a standard 36 mm. cone fitted with a friction sleeve, at the standard speed of 20 mm/s. A push ram was fitted horizontally at one of the wall openings, another could slide over the tank to obtain three vertical measurements in each sample, spaced 0.4 m, from each other and 0.56 m, from the rigid wall in order to minimize the boundary influence for this chamber according to preliminary tests performed in this tank. The vertical penetrations were made in such a way that there was a distance of 5 cm between the straight path of the horizontal and vertical cones. In this way three points were obtained in each test with both horizontal and vertical cone resistances and side friction known. A total of 20 samples has been prepared, in ten of those samples the horizontal measurements were obtained at the 2.01 m. level, in the remaining 16 samples the horizontal measurements were taken at the 2.23 m. level.

3 RESULTS

Some typical horizontal cone resistances for different densities at the 2.01 m. level are shown in Figure 3. For each of these densities one of the vertical measurements is shown in Figure 4. It is clear that the HCPT has less variation over its length, as could be expected as the entire penetration takes place at the same stress level, but that the curves show some influence of the rigid boundaries of the tank. The side friction measurements for these penetration tests are not plotted here, but show a common trend with the cone resistance. It should be stated that the horizontal CPTs have been made with different orientation (rotation) of the cone and that this orientation has shown no influence on the measurements.
3.1 Horizontal cone resistance

In total 78 combinations of horizontal and vertical cone resistance have been obtained. To combine the results from the tests at the 2.01 m and 2.23 m level, all cone resistances have been divided by the effective vertical stress $\sigma' v$. These values have been plotted in Figure 5. The error in the measurements falls within the size of the plot symbol used. The scatter present in the data is accounted to variations in (local) density in the sand.

At first glance there is a linear relation between the horizontal and vertical cone resistances. When the ratio of horizontal over vertical cone resistance $q_{HV}/q_v$ is plotted against the relative density of the sand however (Figure 6), it shows that the relationship between $q_{HV}$ and $q_v$ is not simply linear. For intermediate densities the mean of horizontal cone resistance is approximately 20% higher than the vertical cone resistance. The authors posit that this is the effect of the differences in the initial stress state of the plane perpendicular to the cone. A more detailed explanation will be given in Section 4.

For low and high relative densities the ratio $q_{HV}/q_v$ approaches one. This value of $q_{HV}/q_v$ at these limits can be deduced from simple considerations without going into detail about the influence of the stress state. For low densities Schmertmann (1975) has shown that there is almost no dependence of the cone resistance on the stress level. If this also holds for horizontal CPT, it can be expected that for low densities the cone resistance is only a function of density, and therefore that $q_{HV}/q_v$ approaches 1.

High densities on the other hand are realised in this calibration chamber by prolonged vibrating. This not only densifies the sand, but also increases the effective horizontal stress $\sigma'_v$ and thereby the value of $K = \sigma'_v/\sigma'_v$. This overconsolidation effect has been measured using an earth pressure gauge. These measurements show that for relative densities between 0.6 and 0.8 the value of $K$ increases from 0.5 to 1. Therefore at high densities, where $K = 1$, there is no difference between the stress states around the horizontal and vertical cone. In that case we expect the cone resistance to be independent of orientation, and therefore the $q_{HV}/q_v = 1$.

Given the scatter in the data, no function adequately describes the relation between $q_{HV}/q_v$ vs. $R_d$. The mean of these measurements however can be described by the function

$$ q_{HV}/q_v = 1 + 0.20e^{-0.2(R_d-0.45)} $$

which expression has been plotted as the dashed line in Figure 6.

![Figure 5: Horizontal vs. vertical cone resistance](image)

![Figure 6: Horizontal over vertical cone resistance vs. relative density](image)

3.2 Horizontal side friction

As the cone used had been fitted with a friction sleeve the side friction has been obtained along with the cone resistance. For all measurements the side friction has been divided by the cone resistance to obtain the friction number $R_f$. This friction number is frequently used to get an indication of soil stratification from a CPT. As Beggemann (1969) has shown there is a relation between the amount of fines and the friction number, where a friction number around 1% indicates clean sand. In that light it is of interest whether the horizontal friction number $R_f$ is equal to the vertical friction number $R_f$, and the established relation between friction number and soil classification also holds for horizontal CPT.

The obtained horizontal friction number has been plotted against the vertical friction number in Figure 7. It can be seen that the horizontal and vertical friction number show the same amount of scatter and that the vertical friction number is equally distributed around 1.00%. It is also clear that the horizontal friction number is lower than would be expected. This effect becomes even more apparent when the ratio of horizontal over
vertical friction number $R_{HV}$ is plotted in Figure 8 against relative density. The mean of all the horizontal over vertical friction numbers plotted here lies at $R_{HV} = 0.76$. There is no clear dependency of this ratio on the density, nor on the ratio of cone resistances. If the effect of lower horizontal side friction occurs in other soils too, the classification charts based on friction number will probably have to be recalibrated for HCPT.

4 CAVITY EXPANSION MODEL

One of the methods to describe a vertical cone penetration by use of a cavity expansion model, as proposed by Voege (1972) and adapted by numerous authors. Measurements by Honish and Hitchman (1988) have shown that the effective horizontal stress $\sigma_{H}$ is of controlling influence on $q_c$. This correlation between $\sigma_{H}$ and $q_c$ leads a.o. Salgado et al. (1997) to propose a cylindrical cavity expansion as the basis for a model of cone penetration. Using the assumption that plane strain conditions hold for this problem, the cavity expansion can entirely be described as the expansion of a circular cavity in a plane perpendicular to the penetration direction. For a vertical CPT the initial stress state in this plane is a uniform radial stress equal to $\sigma_0$, as sketched in Figure 9. Although such a model cannot describe the deformations in front of and around the cone in full detail, it shows the correct correlation between $\sigma_0$ and $q_c$

To describe a horizontal CPT using a similar model we again propose a circular cavity in a plane perpendicular to the penetration direction. The difference lies in the more complicated stress state in this plane. The initial stress varies between $\sigma_0$ and the generally higher $\sigma_0$ see Figure 9. This non-uniform stress state somewhat complicates the model. The initial stress state in the vertical case is simply given by

$$\sigma_{TV} = -\sigma_0, \quad (2)$$

while the horizontal stress state, dependant on the angle $\theta$ with the vertical, can be approximated by

$$\sigma_{TH} = \sigma_0 \cos \theta - \sigma_0 \sin \theta \cos 2\theta, \quad (3)$$

where in both cases $\sigma_0$ is the radial stress component. Combining these stress states with a uniform radial displacement at the cavity boundary, the expansion in a hyperelastic medium can be solved straightforward using the general solution method described by Blumenstein (1954). In this way we obtain the stress and deformation states around the cavity. These cannot be compared directly with each other, due to the angle dependency of the horizontal solution and as such give little information on the relation between horizontal and vertical CPT. For both cases however we can calculate the total work done in expanding the cavity, integrated over all angles $\theta$. The work W in these cases is defined by

$$W = \int_{\theta}^{2\pi} \int_{\theta}^{2\pi} \sigma(r) \rho dr d\theta \quad (4)$$

with $\sigma(r)$ the stress state around the cone as function of the strain $\epsilon_0$, $\epsilon_0$ and $\epsilon_f$ the initial and final strains, and $r$ the radius of the cavity. The radial strain, equivalent to a displacement from $r = 0$ to
of course be incorporated into the model. The relatively simple model presented here is only intended to illustrate the underlying mechanism which causes the measured – higher – horizontal cone resistances.

5 CONCLUSIONS

It has been shown that the measurements from horizontal cone penetration testing in sand differ somewhat from those obtained in vertical CPT. For normally consolidated soils at intermediate relative densities the horizontal cone resistance is averagely 20% higher than the vertically measured value. This difference can be attributed to the difference in initial stress state of the soil in a plane perpendicular to the cone penetration direction. The side friction, expressed as the friction number, is lower when measured in a HCPPT than in a VCPT. If this effect occurs in other soils too, the soil classification charts based on friction numbers should be slightly modified for HCPPT. To pass a final judgment on this matter a number of tests has to be executed in other types of soil.

Besides the direct use in interpreting the measurements from HCPPT, the observed influence of the initial stress state on the cone resistances can be used in order to improve the models used to describe cone penetration testing. Even though it is inconvenient for the interpretation of field tests that the stress state perpendicular to the penetration direction is of major influence on \( q_h \), this effect should be taken into account when interpreting CPT.

6 ACKNOWLEDGEMENTS

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NOTATION

\( d_0 \) diameter at which 50% passes sieve

\( K \) earth pressure coefficient

\( m \) alternative elastic modulus

\( r \) radial distance from the centre of the cavity

\( r' \) ultimate radius of the cavity, radius of the cone tip

\( \rho_1 \) relative density

\( R_f \) friction number

\( R_{HV} \) friction number from a horizontal measurement

\( R_{AV} \) friction number from a vertical measurement

\( R_{HV} \) ratio of horizontal over vertical cone resistance

\( q_c \) cone resistance

\( q_{HV} \) cone resistance from a horizontal measurement

\( q_{AV} \) cone resistance from a vertical measurement

\( q_{HV} \) ratio of horizontal over vertical cone resistance

\( W \) work done in expanding the cavity

\( W_H \) work in case of horizontal CPT

\( W_V \) work in case of vertical CPT

\( W_{HV} \) ratio of horizontal over vertical work

\( \varepsilon \) strain

\( \varepsilon_0 \) initial strain, eq. to \( r = 0 \)

\( \varepsilon' \) final strain, eq. to \( r = r' \)

\( \lambda \) first Lamé constant

\( \mu \) second Lamé constant

\( \theta \) angle with the vertical direction

\( \sigma \) stress

\( \sigma_h \) horizontal effective stress

\( \sigma_v \) vertical effective stress

\( \sigma_r \) radial stress

\( \sigma_{HR} \) radial stress in case of horizontal CPT

\( \sigma_{AV} \) radial stress in case of vertical CPT
Site characterization of soil deposits using recent advances in piezocone technology

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Mining Group, Klohn-Crippen, Richmond, B.C., Canada

ABSTRACT: Proper site characterization of soil deposits involves assessing the stratigraphic, geotechnical, hydrogeological and geochemical nature of the site in a manner which best represents actual in-situ conditions. In-situ testing methods can offer the best available and most economical technology to achieve this representative characterization for many site conditions. The key in-situ tools available for this characterization are the piezocone, resistivity piezocone and the BAT technology water sampling system.

1 INTRODUCTION

A complete site characterization requires a full 3-D representation of stratigraphy (including variability), estimates of geotechnical parameters and hydrogeological conditions and properties. Environmental concerns add the need for also determining geochemical conditions. Non-intrusive surface geophysical methods work effectively to guide and supplement data from penetration in-situ test tools like the resistivity/seismic piezocone and BAT technology specific depth pore water sampling systems. These tools can also accurately and economically meet the requirements for environmental site characterization as set out by the US Environmental Protection Agency (USEPA, 1989) which includes stratigraphy, water level data, hydraulic conductivity and chemical distribution and sources/receptors for potential and existing contaminants.

This paper presents a brief review of selected penetration methods for site characterization of soil deposits, and recent developments and experiences in the UBC in-situ testing group. These methods include the piezocone penetration test, the resistivity piezocone and the BAT water sampling and in-situ hydraulic conductivity measuring systems. The seismic cone is not covered here but a recent ASTM symposium on field methods for dynamic geotechnical testing covers the topic in great detail (Campanella, 1994).

2 IN-SITU TESTING

2.1 Piezocone - The piezometer core penetration test (CPTU) involves pushing into the ground a 60° apex cone of typically 35.7 mm diameter (10 cm² area) as shown in Fig. 1. Pushing at a constant 2 cm/s (~1 m/min) is achieved by hydraulic force supplied typically by either a drill-rig or a specially outfitted cone pushing vehicle. Davies and Campanella (1995) list typical pushing capabilities for clay and sand soils.

The UBC piezocone measures tip resistance (qₜ), friction sleeve stress (qₛ), and pore pressure response at up to three locations on the cone tip face, immediately behind the cone tip and immediately behind the friction sleeve (referred to as U1, U2, and U3, respectively). The U3 location has a more sensitive pore pressure transducer to measure accurate small dissipations and equilibrium pressures compared to U2 or U1. Most correlations and direct calculations assume measurement at the standard U2 location. Temperature (T) and inclination (I) are also measured simultaneously as the piezocone is advanced into the ground. All channels are continuously monitored and typically digitized at 25 or 50 mm intervals. Campanella and Robertson (1988) outline the piezocone's main advantages, limitations, and standard testing and recommended interpretation procedures.

2.2 Resistivity Piezocone - The resistivity piezocone (RCPTU) provides the ability to measure the
electrical resistance to current flow in the ground on a continuous basis. This ability is extremely valuable due to the large effects that dissolved and free product constituents have on soil resistivity (conductivity). The RCPTU consists of a resistivity module which is added behind a standard piezocene. Davies and Campanella (1995) give an overview summary of the RCPTU and its perceived application areas.

Figure 1. UBC Resistivity Piezocene (RCPTU)

Measurements of bulk resistivity trends indicate whether some form(s) of dissolved or free product constituent exist below or above background values. Background values are usually established from on-site testing. The areas where readings are very different from background values are then further evaluated with appropriate groundwater sampling at discrete depths for detailed chemical analysis. Of considerable practical value is the fact that the measured resistivity in saturated soil is almost totally governed by the pore fluid chemistry and soil mineralogy, porosity, and particle size have a limited effect in most circumstances.

The UBC resistivity module of electrode rings is attached behind the standard piezocene, is electrically isolated from cone electronics and has no current leakage and gives linear calibrations of resistivity to very high values of 5,000 ohm-m. The smallest electrode spacing (15 mm) is useful for detection of thin layers of contrasting bulk resistivity, whereas the largest electrode spacing (150 mm) is used for AC current excitation and measures an average resistivity over a larger depth. See Campanella and Weemees (1990) for the R & D of the resistivity module.

2.3 BAT discrete-depth water sampling system - A modification of the commercially available BAT System (named after the inventor, Bengt Arne Torstensen, 1984) is recommended for obtaining in-situ pore fluid samples. The original system consists of a sampling tip that is accessed through sterile evacuated glass sample tubes and a double-ended hypodermic needle set-up pushed through septum seals. The tube sampler is lowered either by cable or electrical wire depending upon whether a pore fluid sample is taken with or without a pressure test being carried out. Figure 2 shows the modifications made at UBC (Wilson and Campanella, 1997) which include using a stainless steel or Lexan sampling carrier, a modified probe to push down a previous cone hole and replacing the hypodermic needle system with Swagelock quick-connect push-on valve fittings. This latter modification allows much more accurate and feasible sampling in higher TDS environments as experienced, for example, during water sampling in metallic mine tailings. The BAT is hydraulically pushed with the same equipment used for cone penetration testing.

Figure 2. UBC Modified BAT Groundwater Tool

The US-EPA and other high conformance requirement groups have adopted BAT technology as appropriate and preferred for many environmental characterization applications. The attraction of no drill cuttings and the repeatability of the data are cited as the key reasons for this preference. BAT technology has been scrutinized by many
investigators and has met with widespread acceptance (e.g., Zeno et al., 1992).

After BAT water samples are retrieved to the ground surface, preliminary chemical tests should be conducted on-site and then the sample can be stored for further laboratory analyses. Field measurements should, at a minimum, include conductivity, temperature, and pH. Once enough sampling is carried out at a specific depth, the BAT probe is then pushed to the next depth and the procedure repeated. There is no limit to the number of samples that can be taken at one location.

2.4 BAT hydraulic conductivity ($K$), measuring system - Recent studies at the University of British Columbia (UBC) (Wilson and Campanella, 1997) have made use of the UBC-modified to perform out-flow hydraulic conductivity, $K$, tests. The analytical solution was verified in comparison testing where the BAT tip is made to function as an out-flow slug test. Not only were the results identical but laboratory tests in 5 m high water columns showed that the current limiting highest $K$ of the measuring system with 50 mm long filter section and 3/8 inch valves was 0.0001 m/s (or a medium sand) as opposed to the original use of hypodermic needles of 0.000001 m/s. An important finding in this study showed clearly that an in-flow $K$ test in the field often gave incorrect and misleading $K$ values which were usually more than an order of magnitude too low due to fines migrating and plugging the filter. Thus, water sampling (in-flow) could not be used to also give $K$ of the soil.

2.5 Piezocene hydraulic conductivity and gradient measuring system - The measurement of high speed pore pressure dissipation when CPTU penetration is stopped in sandy deposits is also used to estimate the time for 50% dissipation, $t_{50}$, which can be 5 sec. or less, from which the hydraulic conductivity can be calculated. However, the equation constant needed to calculate, $K$, is directly calibrated using the out-flow $K$-BAT determination at the same locations. Only a few $K$-BAT determinations are usually needed at a given study site. Figure 3 shows a typical example of rapid dissipation of excess pore pressure to equilibrium at two depths and interpretation of results.

3 USE OF IN-SITU TESTING IN ENVIRONMENTAL SITE CHARACTERIZATION

As summarized by Davies and Campanella (1995), the resistivity piezocene can be used to evaluate the following environmental/geotechnical parameters:
1. soil stratigraphy, 2. soil density, 3. undrained shear strength parameters, 4. hydraulic conductivity, 5. in-situ hydraulic gradients, and 6. relative geochemical nature of pore water. The geochemical nature comes from evaluation of the continuous bulk resistivity signature from the resistivity piezocene compared with chemical analyses on samples obtained from the BAT sampling system. Table 1 presents a small sampling of typical RCPTU bulk soil resistivity measurement values and corresponding measurements of pore fluid resistivity. Note the wide range of values for different pore water chemical constituents.

<table>
<thead>
<tr>
<th>Material type</th>
<th>Bulk Resistivity $\rho_b$, $\Omega\cdot m$</th>
<th>Fluid Resistivity $\rho_f$, $\Omega\cdot m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dolous sands with saltwater intrusion</td>
<td>2</td>
<td>0.5</td>
</tr>
<tr>
<td>Drinking water from sand</td>
<td>&gt;50</td>
<td>&gt;15</td>
</tr>
<tr>
<td>Typical sand/di leachate</td>
<td>1-30</td>
<td>5-10</td>
</tr>
<tr>
<td>Mine tailings (base metal) &amp; oxidized sulphide leachate</td>
<td>0.01-20</td>
<td>0.0-5-15</td>
</tr>
<tr>
<td>Mine tailings (base metal) no oxidized sulphide leachate</td>
<td>20-100</td>
<td>15-50</td>
</tr>
<tr>
<td>Arsenic contaminated sand and gravel</td>
<td>1-10</td>
<td>5.4</td>
</tr>
<tr>
<td>Industry site: inorganic contaminants in sand</td>
<td>0.5-1.5</td>
<td>0.3-0.5</td>
</tr>
<tr>
<td>Industrial site: creosote contaminated sills and sands</td>
<td>200-1000</td>
<td>75-450</td>
</tr>
<tr>
<td>Industrial site: wood waste in clayey sills</td>
<td>300-600</td>
<td>80-200</td>
</tr>
</tbody>
</table>

Note. Conductivity($)[S/cm] = 10,000/[$\rho$ Resistivity] [Ohm-m]

In a recent compilation study of factors affecting bulk soil resistivity, Daniels, 1997, showed that it was possible to estimate porosity, $n$, and degree of saturation, $S_s$, from RCPTU tests. Lab calibration of two soils yielded the following result:

- $F = \rho_b/\rho_f = 1.84 \times 10^{-11}$ (sulphide tailings)
- $F = \rho_b/\rho_f = 6.60 \times 10^{-1.5}$ (quartz rock flour)

where $F$ = formation factor, $\rho_b$ = bulk, $\rho_f$ = fluid and $\rho$ = resistivity.
4 MINE TAILINGS: CASE HISTORY

The site, which is relatively flat and consisting of tailings from a sulphide ore-body, had several geotechnical, hydrogeological and geochemical concerns. The resistivity piezocene and BAT sampling technology were selected for characterizing the site for the specific concerns.

Because of the very large extent of the site, a portable surface geophysical tool called a ground conductivity meter (GEONICS® EM31) was used to obtain a preliminary estimate of the location of high ionic groundwaters and plumes. A single person walks the site with the meter collecting digital data every 2 m in a fixed grid spacing. Figure 4 shows the effective conductivity to a depth of about 5 m in an area 3 km by 3.5 km, which was walked in one day. The higher the apparent conductivity, the higher the ion concentration in the groundwater and the lower the bulk resistivity. In this case the existing observation wells could not identify the plume or its direction of movement in a buried channel.

Figure 5 shows a typical resistivity piezocene sounding from the site. From the continuous record of tip stress and penetration pore pressure, the strength and drainage characteristics of tailings could be accurately assessed. With the additional information from the sleeve friction, and hence friction ratio (a percentage value of sleeve friction to tip stress), the tailings were shown to be largely contractant, small-sized and possessing a high susceptibility to static or dynamic triggered liquefaction. This strength characterization work was used to optimize remedial works (berms) that were deemed necessary.

Geochemically, the UBC-BAT sampling program provided site specific relationships between bulk resistivity piezocene values and chemical testing of water samples. The relationship between total dissolved solids (TDS) in pore water and bulk.
conductivity in saturated soil is linear. Specific ion correlations with RCPTU bulk resistivity values are most commonly site-specific in nature although salbate anions and divalent iron have shown remarkable global correlation in our experience to date in mine tailings as shown in Figure 6 for sulphate.

Figure 5. Typical resistivity piezoecone sounding profile from sulphide ore mine tailings

Figure 6. Sulphide ore tailings pore water chemistry

With the aid of the EM31 data, the resistivity piezoecone was used to delineate ionic rich plumes whose sampled characteristics included pH values as low as 1 and TDS concentrations to 60,000 mg/l (ppm). The delineation from the resistivity piezoecone allowed the future optimal spatial placement of regulatory required monitoring wells; both in plan and to accurately locate discrete well screens with depth and a cut-off catchment for the acid drainage.

5 CONCLUSIONS

Recent environmental characterization projects by UBC using the tools described include:
1. DNAPL and creosote contaminated site in B.C.;
2. sulphide mine tailings leachpools, Ontario;
3. mine tailings geotechnical and groundwater evaluations at several mines in B.C.;
4. arsenic contaminated sands and gravels in B.C.;
5. salt-water intrusion to a fresh-water aquifer in B.C.; (Figure 7 shows a 500 m long profile of piezoecone stratigraphy and estuary salt intrusion indicated by bulk resistivity values less than 5 ohm-m)
and
6. regional hydrogeological evaluation in major aquifers in B.C. Other recent activities include combining the presented in-situ tools with surface geophysical techniques to provide enhanced site

Commercially available resistivity piezoecone work is readily available in Canada for roughly $25 to $30 Cdn per metre in most instances. Many environmental characterization projects are carried out each year in materials well-suited to the technology presented in this paper.

This paper has briefly summarized the main in-situ tools available for geo-environmental site characterization. The piezoecone is used as a screening tool to determine stratigraphy, estimate strength and stability parameters, and to identify and indicate the hydro recharge parameters (equilibrium water pressure, gradients and K). The resistivity module when added to the piezoecone also evaluates chemical characteristics (contaminant plume delineation) of the groundwater, particularly in course soils where contaminants are mobile and water sampling is fastest. The BAT is used to take water samples, measure K and therefore to validate correlations. The resistivity piezoecone and BAT sampling technology are establishing themselves as the premier tools where ground conditions are appropriate.

6 ACKNOWLEDGMENTS

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Figure 7. Resistivity Piezocene (RCPTU) profile showing estuary salt water intruding into sand aquifer.

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7 REFERENCES


1000
Geotechnical site characterization of Recife soft clays


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ABSTRACT: This paper presents the database with geotechnical information from the Recife soft and medium clays, including some statistical correlations, research and practical applications of the database. At present there are two research sites, where there has been performed laboratory (characterization, oedometer, triaxial) and in situ tests, such as, SPT, CPTU, DMT, and PMT. The geotechnical database shows to be a very good tool in research and practical projects. All laboratory and in situ tests were well performed and the results were satisfactory and, in general, they compared very well.

1 INTRODUCTION

In about fifty percent of the metropolitan lowland area of the city of Recife, located in the northeast of Brazil, we find deposits of organic or inorganic soil clays, which can occur in the surface or in a depth below a sandy layer. Due to its high compressibility and low strength, soft clays usually present serious problems to foundation and geotechnical engineering.

The Recife soft clay deposits have been studied since 1980 by the Geotechnical Group of the Federal University of Pernambuco (UFPE). At present, there is a database with geotechnical information from about fifty places, including two research sites.

This paper presents the structure of the database, empirical correlations, use of the database for preliminary projects, and examples of the results from laboratory and in situ tests taken from the research sites.

2 SITE DESCRIPTION AND PRESENTATION OF DATABASE

Recife is situated on the Northeastern coast of Brazil (Figure 1) and presents a plain formed in the Quaternary Period with the influence of salt and fresh water. Soft clay and organic soil deposits are found in about fifty percent of the lowland area, formed in the Holocene Period, having a maximum age of about 10,000 years. The land level is close to sea level, thus the soft soil deposits, in general, are almost totally below the water table level.

Figure 1. Location of Recife - Pernambuco / Brazil

Figure 2 shows the map of Recife with the location of the investigated sites included in the database. The geotechnical information is usually obtained from laboratory and in situ tests carried out by the University for research and practical projects of foundation engineering and embankments on soft soils. The standard laboratory tests program consists of characterization, oedometer incremental loading, and triaxial UU-C and CU-C. In the practical projects, the SPT profile and other information are provided by a private design firm. The laboratory information of the database is:
Identification (address, location, project, SPT / geotechnical profile)

- characterization (grain size distribution, water content, limits of Atterberg, organic content, unit weight, degree of saturation)
- compressibility and consolidation (compressive index, swelling index, initial void ratio, preconsolidation stress, overconsolidation ratio, sample quality, and coefficient of consolidation)
- shear strength (undrained strength, strain at failure, Young's modulus)
- classification of soils (UCS, SPT, consistency)

Table 1 shows a summary of information that is classified by report and district, with one example of typical results of three investigated sites.

In the research sites more detailed investigation has been performed, including in situ tests, such as, SPT, Piezocone, Marchetti Dilatometer, and Ménard Pressuremeter.

2.1 Tools for research

The database has diverse ways of research and tools to facilitate the analysis of the collected data. The main tools are: filtering, crossting, research, and the creation of subgroups.

The filtering is a research tool that groups together data that coincide in the filtered information. For example, information to be filtered: district; name of the chosen district: Boa Viagem. The filtering is capable of utilizing at least two criteria of research; also, it is capable of even using criterion of comparison. 1. E. information: void ratio; criterion 1 \( e_v < 3.0 \); criterion 2 \( e_v > 1.5 \).

The crossed research tool permits us to filter the data, utilizing various information at the same time. The result of the research of this is a list of data that coincides in the information, making the researched group more specific. 1. E. 1st information: soil type, criterion 1: organic clay, criterion 2: peat; 2nd information: consistency, criterion 1: very soft; criterion 2: soft.

The creation of subgroups possibilizes the making of new files in the database, which stored the results of any finished research, permitting the subdivision of the database for future analysis. 1. E.: district, soil type, range of natural water content.

2.2 Type of soils

In this paper the soil clays presented were divided in three big groups: sandy, silty, and organic. The last one being the most common. The database information of organic soil/pit will not be presented in this paper.

Figure 3 presents the plasticity chart with results of laboratory tests of the Recife soft clays identified by the type of clay. In the chart, there are included ranges and facts of organic clays (Perrin, 1974) and inorganic clays (Nagaray & Jaydev, 1983) as presented by Coutinho & Lacerda (1987).

### Table 1. Summary of database information - geotechnical parameters (typical results)

<table>
<thead>
<tr>
<th>Report</th>
<th>District</th>
<th>Depth (m)</th>
<th>( W_i ) (%)</th>
<th>( W_o ) (%)</th>
<th>( Y ) (kN/m²)</th>
<th>( \sigma'_{cm} ) (kPa)</th>
<th>OCR</th>
<th>( C_v )</th>
<th>( C_s )</th>
<th>( S_{ph} ) (kPa)</th>
<th>( S_{ph} / S_o )</th>
<th>( S_o / \sigma'_{cm} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>250/95</td>
<td>Ibenha</td>
<td>8.1 / 1</td>
<td>195</td>
<td>70</td>
<td>149</td>
<td>13.0</td>
<td>35</td>
<td>1.11</td>
<td>2.55</td>
<td>0.25</td>
<td>13.5</td>
<td>1111.11</td>
</tr>
<tr>
<td>350/95</td>
<td>Ibenha</td>
<td>15.4 / 2</td>
<td>90</td>
<td>36</td>
<td>80</td>
<td>14.9</td>
<td>74</td>
<td>0.96</td>
<td>1.33</td>
<td>0.14</td>
<td>27.3</td>
<td>208.79</td>
</tr>
<tr>
<td>224/99</td>
<td>Madalena</td>
<td>9.4 / 1</td>
<td>118</td>
<td>42</td>
<td>97</td>
<td>15.7</td>
<td>100</td>
<td>1.25</td>
<td>1.64</td>
<td>0.20</td>
<td>59.0</td>
<td>293.25</td>
</tr>
<tr>
<td>224/99</td>
<td>Madalena</td>
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<td>69</td>
<td>30</td>
<td>55</td>
<td>16.5</td>
<td>150</td>
<td>1.00</td>
<td>0.73</td>
<td>0.14</td>
<td>51.0</td>
<td>191.74</td>
</tr>
<tr>
<td>227/99</td>
<td>Boa Viagem</td>
<td>18 / 7</td>
<td>78</td>
<td>18</td>
<td>54</td>
<td>18.3</td>
<td>118</td>
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<td>28.0</td>
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<td>29</td>
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<td>0.64</td>
<td>0.12</td>
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<td>134.48</td>
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</table>
3 STATISTICAL CORRELATIONS

Statistical correlations have been developed to help research and practical projects. Examples of them are $C_s$ vs. $e_0$, $C_s$ vs. $W$(%), and $e_0$ vs. $W$(%) for all soft clays and for specific groups: sandy clays, silty clays, organic clays, and organic clays. The results are presented on Table 2. Figures 4 and 5 show correlation results considering all soft clays in the analysis. The equations between $e_0$ and $W$ have the highest correlation coefficients ($r^2$). It can also be observed that organic clays presented greater $r^2$ for all correlations. The general equation between $C_s$ and $W$ obtained for Recife clays is very similar to the equation presented by Bowles (1979) for organic soils and clays ($C_s = 0.0115W$), and it is also very similar to those presented by Djoenolid (1985) (see Kulhawy & Mayne, 1990).

The water content was chosen for correlation, because this parameter can be easily obtained from SPT tests. Figure 6 shows that with a standard procedure the results from SPT are very close to laboratory results from Shelby samplers. The site presented in this figure is one of the research sites of Recife (SES-Bura). The influence of sensitivity and in situ void ratio on compression index was reported by Lerouvel et al. (1983). The correlation of the Recife clays presented in this paper ($C_s$ vs. $e_0$) is situated around of the line correspondent to 8 of sensitivity in this chart, which is in the same order of the first sensitivity results (4-10) obtained from this deposit. The result is also very similar to those presented by Djoenolid, 1985 (see Kulhawy & Mayne, 1990). Other correlations are in development in the research project.

4 PRACTICAL APPLICATIONS

Some of the principal applications of the information of the database to practical and research works are: a) Initial estimate, for preliminary project, of parameters from statistical correlations; b) Evaluation of the parameters obtained from the laboratory - comparison between estimate versus obtained geotechnical parameter, to check or make feed-back.
analysis with this results; c) evaluation / calibration of empirical correlation from literature of in situ tests (CPTU, DMT, PMT), to be used in the Recife soft clays / Brazil; d) contribution for spreading the use of other in situ tests (different of SPT) in Brazil, especially in the Northeastern area.

In Brazil, particularly in Recife, the practical engineering normally uses SPT soundings for the basic investigation. The proposal of this research is that natural water content be obtained using a standard procedure. These results will improve the identification or separation of different clayey layers, with more reliability than SPT soundings conventionally. Figure 6 shows this fact, the separation of the deposit in two soft layers from results of water content and that it was not detected by SPT results. The water content determination also permits the utilization of the proposal correlations to

Figure 5. Statistical correlation between $C_C$ vs. $c_0$

Figure 7. Results of characterization tests Research Site Internacional Club (Costinho & Oliveira, 1997) be used in a preliminary project or evaluation of the laboratory results.

5 RESEARCH SITES

At present there are two research sites (located in Recife) studied by the Geotechnical Group of the Federal University of Pernambuco (Figure 2). In these sites there has been performed laboratory (characterization, oedometer, triaxial) and in situ tests, such as, SPT, CPTU, DMT, and PMT. The geoelectrical and dilatometer tests performed represents a joint effort of the Geotechnical Group of the UFPE, COPPE / UFRJ and Geomecnicas Ltd -Rio de Janeiro. The MiniLab penetrometer (PMT) tests represent a joint effort between UPPE and UFPB - Campina Grande.

Figures 7 and 8 present results of the characterization tests (water content, Atterberg limits, organic content) and SPT profile from the two research sites: 1 - Internacional Club and 2 - Sesi-Recife.

The determination of the organic matter content was done by two methods: a) potassium dichromate (EMBRAPA, 1979); b) ignition loss by heating at 400°C during 12 hours a sample dried at 105°C (see Al-khataggi & Andersland, 1981).

The plasticity index in the Site 1 presents values in the range of 70.4 ± 12.4 in the first layer (6.16m), while in the second layer (16.26m) the values are in the range of 33.0 ± 5.7. In the Site 2, PI presents values in the range of 97.5 ± 13.6 to the first layer (4.11.5m) and 53.1 ± 5.9 to the second layer (11.5-21m).

Figure 9 shows results of compression index ($C_C$), initial void ratio ($e_0$), swell index ($C_S$), and
preconsolidation pressure ($\sigma^\prime_{ps}$) obtained from the research Site 2, using oedometer incremental loading tests. The samples were obtained with a sampler stationary piston. The parameters of the first layer are in the order of twice of the values in the second layer (e.g. $C_{DL} \approx 2.4$, $C_{DL} \approx 1.2$). The $\sigma^\prime_{ps}$ results seem to show that this site is still consolidating under the fill's stress (25 years old). In general, it has been observed that the Recife soil clays are slightly overconsolidated (OCR $\leq 2.5$) or close to a normally consolidated (OCR $< 1.5$).

The results of the dilatometer index parameters for the three tests performed in the Site 1 are shown in Figure 10. Typical results and parameters indexes of piezocone tests, from the Site 2, are presented in Figure 11. These results (CPTU and DMT) presented a very good repeatability and identification of soil type and classification.

Figure 12 presents a summary of parameters ($K_s$, $M$, $S_0$) obtained from in situ tests together with results of the laboratory tests (Site 1) (Coutinho & Oliveira, 1997). In this paper, results of undrained shear strength from P-2MT tests were included, using empirical correlations proposal by Powell (1990) (Cavalene, 1997).

The undrained shear strength results from all in situ tests, in general, compare very well with the laboratory results. The results of $K_s$ shows that the DMT results were very close to the “laboratory” and that $K_s$ values from piezocone are very dependent on the correlation used. The results of constrained modulus ($M$) show a very reasonable agreement in

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**Figure 8. Results of the characterization tests - Research Site 2 - SESH-Ibam**

**Figure 9. Results of oedometer tests - Research Site 2 - SESH-Ibam**

**Figure 10. Results of the dilatometer index parameters - Research Site 1 (Coutinho & Oliveira, 1997)**

**Figure 11. Typical results and parameters from piezocone tests - Research Site 2**
the soft layer 2. In the soft layer 1, the DMT values were slightly higher than the oedometer results. More details from Site 1 can be seen in Coutinho and Oliveira (1997).

6 CONCLUSIONS

1. The database of the Recife soft / medium clays, with information from about fifty places, including two research sites, seems to be a very good tool in research and practice projects.

2. Statistical correlations have been developed to help research and practical projects such as $C_v$ vs. $C_t$, $C_v$ vs. $W(\%)$, and $C_r$ vs. $W(\%)$. Other correlations are in development in the research.

3. It is proposed the determination of the natural water content during SPT sounding to improve the capability of identifying different soft layers. In Recife, this information can be used together with the database results, to get an estimation of geotechnical parameters, for a preliminary project or evaluation of laboratory results.

4. In situ tests (CPTU, DMT) presented a very good repeatability and usually very good prediction of the geotechnical parameters for the Recife soft clays.

ACKNOWLEDGMENTS

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REFERENCES


Role of piezocene tests in heavy lift foundation design

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ABSTRACT: A very large mobile crane was used to lift 3,150 kN steam generators through the roof of the containment building of a nuclear power plant. Soil conditions were stiff clay underlain by softer clay. Bearing capacity and settlement of the crane foundations were critical. This paper describes the detailed piezocene and field vane testing program and the results of the tests used to derive the undrained shear strength values of the clay soils. These values formed the basis of the bearing capacity analysis and helped determine that no virgin consolidation would occur beneath the crane foundations.

1 INTRODUCTION

Steam generator replacement (SGR) at Rochester Gas and Electric Company's R.E. Ginna Nuclear Plant on Lake Ontario involved lifting two 3,150 kN steam generators through the roof of the containment building (approximately 35 m high) and replacing them with two new generators. The maximum load on the crane tracks was almost 20 MN at a working radius of 75 m, giving an average pressure of about 575 kPa per track. Adequate crane foundations were obviously essential, since not only would foundation bearing failure during the steam generator lift have been catastrophic, but differential settlement of as little as 20 mm between the tracks could have caused tipping, with equally catastrophic results.

Borings made in the crane operating area indicated a 3- to 4-m-thick layer of stiff clay underlain by a 4- to 5-m layer of soft to medium stiff clay, and then hard silt and bedrock. Given the extent of the soft clay, driven or drilled piles were considered for crane foundation support. However, since the crane consisted of two crawler units separated by almost 30 m (the rear unit being the counterweight component), and plans called for extensive maneuvering of the fully-loaded crane during rigging of the steam generators, the estimated number of piles was substantial, as was the cost (to the order of $500,000). Detailed piezocene and field vane investigations were conducted to explore alternative solutions to pile foundations. The main object of the investigations was to determine if the crane could operate on a soil-supported mat foundation.

This paper describes the piezocene and field vane programs and how the results provided a high degree of confidence in the estimation of thickness and shear strength parameters of the clay layers. This allowed the ultimate bearing capacity and rate of settlement of the crane mat foundation to be established with the needed degree of certainty, enabling a soil-supported mat foundation for the crane to be successfully designed, constructed, and used in the steam generator replacement.

2 SUBSURFACE CONDITIONS

Three sample borings made in the crane operating area for the SGR indicated the subsurface conditions in Figure 1. The locations of these borings are shown in Figure 2, which also delineates the limits of travel of the front and rear crawlers of the crane.

The main plant structures are founded on the sandstone of the Quentin formation or on the thin layer of overlying hard till (Layer 3). The layers of interest for crane support were the Layer I mortled brown and gray stiff to very stiff silt clay and the Layer 2 gray soft to stiff silty clay. These clays are glacial and lakebed deposits (Rochester Gas and Electric Corporation 1999). The Layer 1 clay is a till deposit related to a minor glacial readvancement that occurred about 12,000 years ago. The underlying softer Layer 2 clay is a lakebed material believed to have been deposited in the bed of a former glacial lake, Lake Iroquois. Groundwater is typically at about mid level in Layer 2.
Although the Layer 1 clay showed a considerable variation in SPT N-value (Figure 1), it was obviously a significantly overconsolidated deposit, with average moisture content close to the plastic limit. Based on the N-values, it was initially assigned an undrained shear strength ($S_u$) of 145 kPa in the top 2-m thick crust, decreasing to 95 kPa at the bottom of the layer. Various tests were made on the Layer 2 clay to estimate its $S_u$ value, including unconsolidated undrained triaxial, unconfined compression, torvane, and pocket penetrometer tests on the SGR boring samples, and $S_u$ versus N-value correlations. The results, shown on Figure 3, also include data from two original adjacent plant boorings. As might be expected, there was considerable variation in the strength estimates, with no significant trend with depth. The data on Figure 3 suggest that Layer 2 was also somewhat
overconsolidated. The average liquid limit of 27,
average plastic limit of 14, and average moisture
content of 21 tended to confirm this. However, a
consolidation tests performed on a Layer 2 sample
gave an overconsolidation ratio (OCR) of about 1.
Bearing capacity estimates for the fully-loaded
crane on a 1.15-m thick heavily reinforced concrete
mat that extended approximately 2 m beyond the
echo of the crawler track (Davie et al. 1998)
indicated that a Layer 1 Sa value of 145 kPa reducing
to a Layer 2 Sa value of about 50 kPa would provide
an average factor of safety (FS) against bearing
capacity failure of over 3. It was apparent from the
Figure 3 results that the actual undrained shear
strength of the Layer 2 clay could be 50 kPa or more.
However, given the scatter of the results, this could not
be assumed without appreciably more, and
better, testing. Considering the high cost of an
alternative deep foundation system, the decision was
made to conduct a detailed piezcone and field shear
vane program to confirm (or not) that there was
adequate shear strength available to support the mat
foundation with an FS > 3.

3 PIEZOCONE SOUNDINGS AND FIELD VANE
TESTS
3.1 Piezcone soundings
 Fifteen soundings, located approximately 12 m to 15
m apart as shown in Figure 2, were performed for
Bechtel by ConeTec, Inc., of Warren, New Jersey,
using an integrated electronic piezcone
manufactured for Con Tec by Adara Systems of Vancouver. The piezocene dimensions and operating procedures were in accordance with ASTM D 3441.

The piezocene used was a compression model with a 10 cm² tip and a 150 cm² friction sleeve designed with an equal end area friction sleeve and a tip end area ratio of 0.85. The cone was a soft soil model with a 22.25 kN load cell consisting of four sets of strain gauges with a full scale output of 7.5 V dc. The voltage signal was carried from the cone through a cable in the cone rods and digitized by the data acquisition system. In the piezocene portion of the cone, located immediately behind the tip, 6-mm-thick pore pressure filter elements of porous plastic were saturated with glycerine under a vacuum. The cone penetrometer was also used to measure the shear wave velocity of the clay, with a geophone located 200 mm behind the cone tip. The apparatus was also capable of measuring temperature and instrument slope.

Before each sounding was made, a complete set of baseline readings was taken using a multi-meter plugged into the cable. The readings were compared with the digitized value on the computer screen, thus providing a check on the analog to digital conversion board. The baseline shift from sounding to sounding was so small—typically less than 0.1 percent of full scale—that no corrections were applied to the data. The cone was then attached to steel rods and advanced at about 50 mm/sec. using a drill rig. The cone tip resistance, sleeve friction, and dynamic pore pressure were recorded onto magnetic media and simultaneously printed every 50 mm as the cone was advanced into the ground. To obtain the corrected cone tip resistance, $Q_c$, during penetration, the data acquisition system subtracted the baseline value from the voltage measured by the load cell, multiplied the result by the calibration factor to obtain the applied load, divided this by the cone tip area, and corrected the result for pore pressure effects.

A porewater pressure dissipation test and seismic cone penetrometer tests were also performed in the Layer 2 clay. The results of these tests are not presented here.

3.2 Field vane tests

The field vane tests were performed by Con Tec at five of the piezocene locations, as shown in Figure 2. The test locations were chosen by Boechtel based on the piezocene test results, at representative locations and at locations where lower values of tip resistance had been recorded. The purpose of the vane tests was to establish a site-specific correlation between the undrained shear strength of the clay and the piezocene tip resistance.

The Nilcon vane had four blades made from high strength (1725 MPa) tempered chrome-nickel steel; outside dimensions of the vane were 65 mm diameter by 130 mm long. The vane had small relative vane volume displacement and extremely thin blade edges and used a calibrated vane torque recording head. The torque head was both a loading and recording instrument, using a crank-operated, two-speed loading device and a waxed paper disc on which the permanent test record was scribed by a sharp steel pointer. A very low friction slip coupling between the vane and the steel rod connecting the vane to the torque recorder allowed 15 degrees of free slip so that, during the test, the rod rotated first and the torque for that rotation was recorded. This torque was subtracted from the total torque required to rotate the rod and vane.

Field vane tests were performed at an average of three depths at each location. The borehole was advanced to about 1 m above the planned field vane depth, and then the vane was pushed about 1 m into the clay.

4 PIEZOCONE SOUNDINGS AND FIELD VANE TEST RESULTS

4.1 Piezocene soundings

The results of the 15 piezocene soundings showed consistent tip resistance, sleeve friction, and pore pressure profiles in the Layer 2 clay across the crane foundation area. Figure 4 shows the results from a typical sounding C-17. The Layer 1 clays were usually too strong for the cone apparatus in the top 1 to 2 m and had to be predrilled and cased. Once tip resistance, $Q_c$, dropped below about 3000 kPa, it fell rapidly to around 950 to 1150 kPa and then held almost constant with depth, except for occasional spikes that indicated thin sand layers, confirmed by low pore pressures. Layer 1 clays were defined as having $Q_c$ values of 3000 kPa or more, while Layer 2 clays had $Q_c$ values in the 950 to 1150 kPa range. As noted earlier, the $Q_c$ values plotted in Figure 4 were already adjusted for dynamic pore pressure effects, i.e.:

$$Q_c = Q_{cu} + (1 - u)u$$  \[(1)\]

where $Q_{cu} =$ measured cone tip resistance, $u =$ tip end area ratio = 0.85, $u =$ measured dynamic pore pressure.
From the 15 piezocone soundings, depths to the top of Layer 2 agreed reasonably with the depths interpreted from the borings in Figure 1, although the piezocone results did not show the marked increase in depth to the top of Layer 2 from west to east indicated in Figure 1. The median depth to the top of Layer 2 estimated from Q was 4.0 m. There was a transition zone from the bottom of Layer 1 (Q > 3000 kPa) to the top of Layer 2 that had a median thickness of about 0.75 m. The median thickness of Layer 2 was harder to judge from the piezocone results since at least one of the soundings refused on a sand layer. Excluding that sounding, estimated Layer 2 thickness ranged from 3 m to 6.6 m, with a median thickness of 4.25 m.

4.2 Field vane tests

Table 1 shows the results of the 17 field vane tests. The undrained shear strength, $S_u$, was calculated from the standard expression for vanes with a length-to-diameter ratio of 2, i.e.:

$$S_u = (G/100)^{1/3}$$  \hspace{1cm} (7)
### Table 1. Field vane test results.

<table>
<thead>
<tr>
<th>Location</th>
<th>Test</th>
<th>Layer</th>
<th>Depth, m</th>
<th>$S_u$ kPa</th>
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<td>2</td>
<td>5.4</td>
<td>46</td>
</tr>
<tr>
<td></td>
<td>FVT-2</td>
<td>2</td>
<td>6.4</td>
<td>54</td>
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<tr>
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<td>3.0</td>
<td>&gt;170</td>
</tr>
<tr>
<td></td>
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<td>47</td>
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<td></td>
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<td>2</td>
<td>6.6</td>
<td>46</td>
</tr>
<tr>
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<td>2.5</td>
<td>&gt;80</td>
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<tr>
<td></td>
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<td>*</td>
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<tr>
<td></td>
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<td>96</td>
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</tr>
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</table>

*No clear failure peak was evident.

Where $T = $ applied torque, $D = $ diameter of vane.

Four of the tests were in Layer 1 clay; three of the four had readings higher than the maximum equipment limits, and the fourth showed an undrained shear strength of 135 kPa. The 13 remaining tests were in the Layer 2 clay. Comparison of the test depths with the corresponding piezocene plots indicated that all of the tests were in the clay and not in any of the thin sand seams, although some of the results suggested the readings might have been elevated by sand presence. Two of the tests showed no clear failure peak. Of the remaining 11 tests, 5 gave undrained shear strengths of more than 80 kPa, one gave an undrained shear strength of 67 kPa, and the remaining 5 were closely clustered in the 46 to 54 kPa range.

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5 DERIVATION OF SOIL PARAMETERS

Of primary importance was the derivation of a lower bound undrained shear strength for the Layer 1 and Layer 2 clays to confirm the adequacy of the bearing capacity of a shallow mat foundation. Estimation of total and differential settlement and rate of settlement was also considered necessary for the safety of the loaded crane.

5.1 Undrained shear strength

The undrained shear strength, $S_u$, of a clay is not a unique parameter as it depends significantly on the type of test used, the rate of strain, and the orientation of the failure planes. Figure 3 indicated a large variation in the estimates of $S_u$ for Layer 2 based on the original testing from the SOR borings. The largest variations were in the estimates from SPT N-value correlation, torque, and pocket penetrometer—not surprising given the very approximate nature of such estimates even under optimum conditions. The $S_u$ results from the three unconsolidated undrained tests were quite consistent—ranging from about 38 to 41 kPa. It is generally accepted that values of $S_u$ from laboratory tests on "undisturbed" samples will be somewhat lower than in-situ tests on the same soils because of the inherent disturbance involved in collecting, extruding, and preparing the laboratory samples. Thus, the unconsolidated undrained triaxial test results agree quite well with the five lowest field shear vane $S_u$ results, which ranged from 46 to 54 kPa.

Although the piezocene results showed there were occasional sand seams within the Layer 2 clay that could have affected the rate of settlement of the foundation, they would not significantly affect the amount of settlement or the bearing capacity of the foundation. The most significant observation was that the $Q_v$ value never dropped below about 950 kPa in any of the 15 soundings, i.e., although there was variation in the layer thickness and there were occasional thin sand layers within the clay, there were no layers within the clay that had $Q_v$ values lower than 950 kPa. Estimates of $S_u$ from the piezocene results were made using the following equation (Robertson and Campanella 1988):

$$S_u = (Q_v \cdot \Delta s_w) / N_{cu} \quad (3)$$

Where $\Delta s_w = $ total vertical stress, $N_{cu} = $ empirical cone factor.

Values of $N_{cu}$ for normally consolidated marine clays range from 11 to 19, with an average of 15 (Robertson and Campanella 1988). For nonfailure overconsolidated clays, the average $N_{cu}$ value is 17. The average depth to the center of Layer 2 was about 6.1 m, which gave $\Delta s_w = 124$ kPa, using a measured unit weight of 20.4 kN/m$^2$ for both Layers 1 and 2. Using $Q_v = 950$ kPa and $\Delta s_w = 124$ kPa, $S_u = 55$ kPa for $N_{cu} = 15$, and $S_u = 49$ kPa for $N_{cu} = 17$.

Increases in OCR are generally reflected in increases in $N_{cu}$ (Lamme et al. 1985). Thus, the Layer 1 $N_{cu}$ should be higher than the Layer 2 value. $N_{cu} = 20$ was used in the estimation of $S_u$ for Layer 1. The lower bound value of Layer 1 $Q_v$ in the piezocene soundings was earlier determined as 3000 kPa. The average depth to the center of Layer 1 was about 2 m, giving $\Delta s_w = 40$ kPa. For $Q_v = 3000$ kPa, $\Delta s_w = 40$ kPa, and $N_{cu} = 20$, the resulting $S_u = 148$ kPa.

To summarize, for the Layer 2 clays, the lowest field vane tests gave $S_u = 40$ to 54 kPa. The lowest
piezocene Qb value gave $S_h = 49$ to 55 kPa, using average $N_s$ values for normally consolidated and nonfissured overconsolidated clays. Thus, for conservative bearing capacity design purposes, it was reasonable to assume $S_h = 50$ kPa for Layer 2. $S_h = 145$ kPa was assumed for Layer 1, with linearly decreasing values in the transition zone between the bottom of Layer 1 and the top of Layer 2. As noted earlier, these undrained shear strength values result in the required FS > 3 for the fully-loaded crane on a heavily reinforced concrete mat foundation.

5.2 Elastic and consolidation properties

The two most important questions about the potential crane settlement were: (a) would any virgin consolidation take place in Layer 27 and (b) if such consolidation occurred, would it be significant given the relatively short time the crane would be in its critical lift position? Regardless of the consolidation issue, elastic settlement would be a factor. The modulus of elasticity, $E$, values of the Layer 1 and Layer 2 clays were estimated using the empirical relationship $E = 600 \times S_h$ (Davie and Lewis 1988), giving $E = 87$ MPa for Layer 1 and $E = 30$ MPa for Layer 2. Using these static $E$ values gave an average computed elastic settlement under maximum crane loading of about 12 mm.

The values of compression ratio and recompression ratio computed from Layer 2 consolidation tests were 0.143 and 0.0062, respectively. These values were also used for Layer 1. Assuming that only recompression occurred in both layers gave an average computed settlement beneath the mat to the maximum loaded crane of 17.5 mm. With only recompression of Layer 1, but assuming Layer 2 with an OCR = 1, gave a corresponding computed settlement of about 115 mm (Davie et al. 1998). Thus it was important to determine what amount of virgin consolidation (if any) could be anticipated in Layer 2.

Skempton's equation (Skempton 1957) was used as an approximate means of estimating maximum past pressure. For normally consolidated clays:

$$S_h/(\gamma_{w} \cdot \Delta z) = 0.11 + 0.0037 \frac{\Delta z}{L_o}$$

where $\Delta z =$ effective overburden pressure, $L_o =$ plasticity index.

From Equation 4, using $S_h = 50$ kPa and $L_o = 13$ (median $L_o$ of 15 samples, with a standard deviation of 2.9) gives a corresponding $\Delta z = 316$ kPa, i.e., for a normally consolidated clay with $S_h = 50$ kPa and $L_o = 13$, the effective overburden pressure would be 316 kPa. This can be regarded as an approximation of the maximum past pressure on the clay, assuming groundwater at about mid level in Layer 2, $s_w$, at the top and bottom of Layer 2 is about 82 and 147 kPa, respectively. Maximum vertical pressure at the top and bottom of Layer 2 from the crane at maximum load was 82 and 58 kPa, respectively (Davie et al. 1998). Thus, total maximum applied pressure plus effective overburden pressure at the top and bottom of Layer 2 was 164 and 205 kPa, respectively. Since the maximum applied pressure plus effective overburden pressure was less than the assumed maximum past pressure, there would be no virgin consolidation of the Layer 2 clay under maximum load.

Application of the SHANSEP method (Ladd and Foott 1974) led to the same conclusion. Using the SHANSEP relationship for a low plasticity clay (Boston blue clay):

$$S_h/(\gamma_{w} \cdot \Delta z) = 0.2(OCR)^{1/2}$$

Using the $\gamma_{w}$ values at the top and bottom of Layer 2 (i.e., 82 and 147 kPa) gives OCR values of 4 and 2, respectively. If the maximum applied pressure is added to $\gamma_{w}$ (to give 164 and 205 kPa), then the corresponding OCR values are 1.7 and 1.25, respectively, i.e., no virgin consolidation.

The vertical coefficient of consolidation, $C_z$, from consolidation tests ranged from about 19 to 32 mm²/sec for the maximum load. The maximum estimated time for full loading, i.e., the maximum time the crane would be suspended at maximum working radius, was 12 hours. Assuming a drainage length of half the Layer 2 thickness and $C_z = 32$ mm²/sec, results in about 7 percent of total consolidation, i.e., just over 1 mm of recompression settlement.

6 CONCLUSIONS

The results of the piezocene soundings and field vane tests provided a high degree of confidence in the thickness and strength parameters of the clays, allowing the ultimate bearing capacity of the soil-supported crane mat foundation to be established with the needed degree of certainty. Analyses showed that consolidation due to the fully-loaded crane would be limited to recompression. Estimated settlement and rate of settlement of the foundation gave total settlement values much smaller than the allowable differential values, indicating no potential problems from settlement.

REFERENCES


An integrated CPT approach for a major housing development project

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ABSTRACT: Extensive site investigations and office studies have been performed for the subsoil characterization and geoenvironmental evaluations of a major housing development project with 500,000 square meters floor area in İzmir, Turkey. The performed investigations, covering detailed and complete studies from preliminary investigations at the initial stage of construction to final design and construction performance are presented. The investigations consist of two parts, the first part being the subsoil characterization and foundation design of the housing project. The second part of this integrated study is the contamination assessment and remediation studies performed for the closed municipal solid waste disposal site of the city, which is located near the extension area of the housing development. CPT has been performed both in the housing development site for subsoil characterization and in the solid waste disposal site for determination of the extent of contamination. Water and gas samples have been taken from various depths with a special device connected to the CPT penetration device. The results of the performed field and laboratory tests have been evaluated in the development of the remedial design project.

1. INTRODUCTION

Extensive site investigations and office studies have been performed for the subsoil characterization and geoenvironmental evaluations of a large scale housing development with a total of three thousand units exceeding 500,000 square meters floor area. The site is situated in İzmir, Turkey 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 development due to increasing value of land. Deep alluvial deposits govern the subsoil conditions. Soil borings and CPT testings have been performed for the determination of subsoil conditions.

The investigations consist of two parts, the first part being the subsoil characterization and foundation design of the housing project. Due to the presence of soft clay deposits to large depths and considerably large structural loads of 22 storey high buildings, 65 cm diameter cast in-situ driven piles with lengths between 35.0 and 40.0 m are installed. At early stages, 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 criterion which was developed as a result of wave propagation analysis.

The second part of this integrated investigation is the contamination assessment and remediation studies performed for the closed municipal solid waste disposal site of the metropolitan city of İzmir, which is located near the extension area of the housing development. The disposal site is located within one of the main extensions of the city towards north and partially occupies the extension zone of the housing development project, locally known as Mavişehir. The site has been used over the years for waste disposal by the municipality of İzmir. No precautions such as impermeable base liner and cover have been taken to isolate the waste and contamination from the environment. Although the site is closed to waste disposal, the storage area is a source of contamination and presents major environmental problems.

Now with the development of the city towards north, near the site, remedial measures have to be taken to prevent the major problems related with contamination. Integrated field and laboratory studies have been conducted to enable the solution of the existing environmental problem with determination and assessment of geometrical geotechnical and contamination parameters of the disposal site.
Cone Penetration Testing has been performed in the solid waste disposal site and the depth of contamination has been determined with the obtained conductivity measurements. Water and gas samples have been taken from various specific depths with a special device connected to the tip of the penetration device. The degree of contamination has been systematically assessed with the laboratory testing of the retrieved samples. The results of the performed field and laboratory tests have been evaluated in the development of the remedial design project and it is expected that the geoenvironmental effects of the solid waste disposal site will be eliminated upon the implementation of the proposed design.

2. FIELD AND LABORATORY INVESTIGATIONS

2.1. Housing development site

Total of thirty nine borings were performed at the initial stage of investigations of the housing development area. 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 40 CPT’s up to bearing strata have been performed to determine subsoil conditions and estimate pile capacity. The subsoil stratification present in the site with consequent foundation behavior is outlined below (ZETAS, 1994).

• A recent fill of 3.0 m in present which was constructed in order to establish working platform and reclaim land from the sea.
• 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 the recent 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 depth which contribute to most of the pile capacity.

2.2. Solid waste disposal site

The extent and geometry of the irregularly stored solid waste have been determined with topographical investigations performed at the initial stage of the studies. CPT has been conducted at 17 locations within the storage area, in order to determine the depth of the waste, subsoil stratification, corresponding geotechnical properties and parameters. The volume of the waste has been estimated to be 817,000 m³.

![Typical geotechnical cross section](image)

Figure 1. Typical geotechnical cross section
A typical cross section of the waste and the subsoil is shown in Figure 1. The subsoil formation of the disposal site is similar to the subsoil of housing development area in terms of general characteristics. The average thickness of the waste is determined to be 3.0 meters, in some regions reaching the maximum thickness of 5.0 meters. Silty clay is located at the top on a continuous sublayer of silty sand. Below these a considerably thick clay layer and sandy clay overlying the bearing layer of clayey gravel located at a depth of 27.0-30.0 meters from surface is present, as seen in the cross section. Erratically distributed sand and gravel lenses are present within the subsoil, creating vertical and horizontal connection between soil layers and increasing the rate of seepage within the subsoil. Such a geological formation is typical of the area, i.e., alluvial deposits of Gediz River.

In addition to conventional measurement of cone resistance and sleeve friction, conductivity of the soil has been measured with the CPT device to determine the extent of contamination within the subsoil. The presence of ions in the contaminated soil increases the electrical conductivity of the soil. Therefore the increase in the electrical conductivity in such areas is a measure of the extent of contamination within the subsoil.

A typical conductivity measurement at a CPT test is given in Figure 2. It is seen from the conductivity measurement that the first eight meters of the soil exhibits a relatively high electrical conductivity indicating the zone of contamination which extends to the clay strata within the subsoil.

Water and gas samples have been taken at test locations from various specific depths with a special sampling equipment connected to the cone penetration device (Durgunoglu and Tugrul, 1995). The sampler is driven into the soil with the penetrometer, the filter is opened with and with the aid of a peristaltic pump located at the ground surface the waste or gas (whichever is present at that location) is transferred into proper storage units.

The advantage of such sampling procedure over the methods associated with drilling is that the water or gas sample is characteristic of the certain depth at which the sampler is located. Therefore with the aid of such an equipment, the variation of contamination with depth can be determined with only one penetration. The degree of contamination within the subsoil and groundwater has been systematically assessed with the laboratory testing of the samples.

Chemical analyses have been performed on 12 water samples taken from various locations and depths in the area. The results of the analyses have been summarized in Table 1. Some criteria are also given for comparative purposes.

Concentrations measured in water samples are within the range and above the values given in column (a) which shows the concentration ranges measured in leachate from solid waste disposal sites in Wisconsin (Avec et al., 1994). Especially, the chloride concentration is very high and reaches to 34000 mg/l in average. Such a concentration is an important measure of contamination in the subsoil and groundwater.

Chemical oxygen demand (COD) has been measured to be 3450 mg/l in average. This value is far above the value 600 mg/l given in column (a) as waste water standard and is within the range given in column (b) as an example of leachate water.

It is seen from the comparison of the listed values of certain chemicals that groundwater has been contaminated with the leachate generation. Such a contamination makes it impossible to provide the drinking and domestic water from groundwater sources in the area which is planned to be developed for housing purposes. The high concentration of sulfate in the groundwater makes it necessary to take precautions against corrosive effects of the groundwater for the safety of underground concrete
Table 1. Analyses of Water Samples and Comparison with Available Criteria

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Various Criteria</th>
<th>Average of Samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>pH</td>
<td>6.5-10</td>
<td>7.93</td>
</tr>
<tr>
<td>Chloride mg/l</td>
<td>180-2651</td>
<td>13600</td>
</tr>
<tr>
<td>Total Phosphorus mg/l</td>
<td>0.1-1.17</td>
<td>1.10</td>
</tr>
<tr>
<td>Calcium mg/l</td>
<td>0.07-0.01</td>
<td>0.495</td>
</tr>
<tr>
<td>Total Chromium mg/l</td>
<td>0.05-0.31</td>
<td>0.253</td>
</tr>
<tr>
<td>Zinc mg/l</td>
<td>0.2-0.5</td>
<td>0.283</td>
</tr>
<tr>
<td>Chemical Oxygen Demand mg/l</td>
<td>1120-3459</td>
<td>3450</td>
</tr>
<tr>
<td>Total Nitrogen mg/l</td>
<td>4.7-14.70</td>
<td>7.2</td>
</tr>
<tr>
<td>Sulfate mg/l</td>
<td>8.4-590</td>
<td>2410</td>
</tr>
<tr>
<td>Lead mg/l</td>
<td>10-1.11</td>
<td>0.41</td>
</tr>
<tr>
<td>Oil mg/l</td>
<td>50</td>
<td>1280</td>
</tr>
<tr>
<td>Total Precipitating Solids mg/l</td>
<td>27-3</td>
<td>75</td>
</tr>
<tr>
<td>Total Solids in Suspension mg/l</td>
<td>2180-25833</td>
<td>751</td>
</tr>
<tr>
<td>Total Dissolved Solids mg/l</td>
<td>28-2833</td>
<td>105450</td>
</tr>
</tbody>
</table>

* a: below measurement limit
b: Content range of chemicals present in solid waste disposal sites in Wisconsin, USA, (Avci et al., 1994)
c: Standards for drinking water in Turkey

Structures (foundations, piles, culverts) planned to be built near the area. Observation wells have been installed at CPT locations to monitor the groundwater. These may also be used for future sampling purposes.

The content of H2S, CO2 and CH4 have been investigated in 11 gas samples taken from various locations of the site, especially from the waste itself. Carbon dioxide (CO2) is present in all samples. Except for 4 samples, the ratio of CO2 is determined to be approximately 100%. Methane (CH4) has been measured in 6 samples. The concentrations measured exceeded 80% in 4 of the samples tested.

In addition, the organic content of the waste has been determined to be 23-37%, indicating that although decomposed to a great extent, the waste is still a possible source of gas generation in anaerobic conditions. Therefore, a gas collection and removal system has to be constructed for the removal of generated gas to eliminate possible problems on nearby housing and units.

3. DESIGN CONSIDERATIONS

Vertical pile capacity is estimated utilizing CPT soundings and final pile capacities are determined from evaluation of the results based on pile load tests. Subsoil conditions at the site are improved by means of preloading. The settlement under the 3.0m 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.

A remedial project has been designed with the evaluation of the geotechnical model and contamination data to eliminate the present and future complications related to waste storage. The solid waste distributed over a large region within the storage space is planned to be compacted and stored in a limited area. A new storage geometry has been proposed to minimize the area of storage.

3.1. Pile design and load tests

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 cast in-situ driven piles. The final length of the piles are determined at the site during construction with the comparison of measured driving resistance and given criteria.

Cast in-situ driven 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.
Table 2. Summary of Pile Capacities from CPT

<table>
<thead>
<tr>
<th>Zone</th>
<th>Skew Length Tip Depth (m)</th>
<th>Tip Depth (m)</th>
<th>Skin Frict. Allowable Capacity (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2720 1040 740 36.5 12.0 604 293</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>3400 2370 1030 35.0 13.4 680 738</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>3500 2600 1040 36.0 6.0 328 1278</td>
<td></td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>3380 2220 1110 35.5 5.3 400 1029</td>
<td></td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>3720 2670 1050 36.0 2.9 61 1604</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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 is driven after this depth until the criteria for 25 cm penetration is achieved (Duranglah et al., 1993).

Test piles are produced at different locations in the construction area at the design phase of the project. Pile load tests are performed up to failure on test piles to evaluate the design assumptions. 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 as outlined in Table 2.

The load-settlement behavior of the piles are monitored during construction stage with pile load tests performed on a certain number of piles randomly selected among the constructed ones (1 out of 100). 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 3. The evaluations of the test are given below.

- The tested piles safely carry the applied load which is 1.5 times the design capacity.
- The settlements of the piles under the applied maximum load is 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.

3.2. Solid waste disposal site

With the evaluation of the geotechnical model and contamination data, an appropriate remedial project has been designed which will eliminate the present and future complications related to waste disposal.

The solid waste distributed over a large region within the storage area and approximated to be 817,000 m³ in volume, is planned to be compacted and stored in a smaller area. The solid waste disposal site partially occupying the extension zone of Marisbar Housing Development will be limited in extent and the remaining portions will be opened to housing development gaining extensive amount of land.

Figure 3. Pile load tests for quality control
The gas generation in solid waste disposal sites is divided into four stages. In the first stage named 'anaerobic media' O₂ decreases to zero from 20% and N₂ starts to decrease from 80%, whereas the amount of CO₂ starts to increase. In the second stage taking place in anaerobic media the amount of CO₂ rises to 70-75% and H₂ rises to 20%, whereas N₂ drops to 10%. In the following stage (anaerobic media - formation of instable methane) CO₂ decreases to 45% and CH₄ rises from zero to 55%, whereas N₂ and H₂ diminish to zero. In the last stage (Anaerobic media - formation of stable methane) the CO₂ and CH₄ balance in 45% and 55% while no other primary components are present except these.

High amount of methane (CH₄) and carbondioxide (CO₂) has been measured in the solid waste disposal site. This condition corresponds to the fourth stage mentioned above as 'anaerobic media - formation of stable methane'. A proper leachate and gas collection system has been developed to eliminate the problems associated with the leachate and gas generation.

Stability and settlement analyses have been performed to eliminate the problems foreseen to be associated with the compacted waste disposal site. The geometrical configuration and height for the waste are optimized with the evaluation of these analyses.

4. SUMMARY AND CONCLUSIONS

Integrated studies including CPT and pile load tests have been performed in design of the foundations for a major housing development in Izmir, Turkey.

Cast in-situ driven 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 housing development 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.

CPT has also been utilized in the investigations performed for the assessment of the degree and extent of contamination of a solid waste disposal site which partially occupies the extension of the housing development. Water and gas samples have been taken from the waste and subsoil with a special device connected to the cone penetrometer. Laboratory tests have been performed on the samples and the extent of contamination has been assessed.

A remedial project has been designed with the evaluation of the geotechnical model and contamination data to eliminate the present and future complications related to waste storage. The solid waste distributed over a large region within the storage space is planned to be compacted and stored in a limited area. A new storage geometry has been proposed to minimize the area of storage and the proposed design has been checked in terms of stability and settlement. The final storage scheme has been optimized with the evaluation of these analyses. A monitoring system consisting of settlement columns and piezometers has been proposed in the design to monitor the settlements and pore water pressures expected to occur during the construction of the compacted waste and are critical in terms of stability.

ACKNOWLEDGEMENT

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Undrained shear strength from cone penetration test

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ABSTRACT: Unconfined compression and cone penetration test results were used to develop a relationship between undrained shear strength and cone tip resistance for the soft to stiff, saturated glacial clays of downtown Chicago, Illinois. The cone factor relating the unconfined compression strength to the cone tip resistance, termed \( N_{cu} \), has an average value of 15.5 for Chicago soft to stiff, saturated clays. Additional values of \( N_{cu} \) were compiled from the literature for clays with similar consistency. The unconfined compression strength for soft to stiff clays with plasticity indices of 15 to 50 can be estimated using the cone penetration test results and an \( N_{cu} \) value of 16. For saturated clays in this plasticity range, the unconfined compression strength provides a practical estimate of the mobilized undrained shear strength for foundation design, and embankment and excavation stability analyses.

1 INTRODUCTION

In Chicago, the unconfined compression test (UC) has been frequently used to estimate the undrained shear strength of the glacial clays since construction started on the Chicago Subway in 1939. Clays with very soft to medium consistencies are limited to a zone extending five miles or less from the shore of Lake Michigan and have a thickness between 3 and 15 meters. Stiff to hard clays are usually encountered before bedrock is reached. The cone penetration test (CPT) provides a quick insight into soil stratigraphy and is frequently used during initial site investigations to design an efficient boring and sampling program. An empirical correlation between cone penetration tip resistance and undrained compressive strength is developed to facilitate usage of the CPT in design and construction activities in the Chicago area.

2 EXISTING CORRELATIONS

CPT results are used to estimate the undrained shear strength of clays through empirical correlations and/or theoretical solutions (Bazile 1980). The most commonly used formula is based on the bearing capacity theory proposed by Terzaghi (1943) and rewritten as:

\[
q_c = N_k \cdot f_c + c' \rho
\]

where \( q_c \) is the cone tip resistance, \( N_k \) is the empirical cone factor, \( S_u \) is the undrained shear strength, and \( c' \rho \) is the total vertical stress at the depth of penetration. The wide scatter in the empirical and theoretical \( N_k \) values presented in the literature shows that no single value of \( N_k \) covers all types of clays, penetrometers, and test conditions (Amar et al. 1975; Schmertmann 1975). However, for a given clay deposit, penetrometer, and test condition, it seems likely that there is a unique relationship between cone tip resistance and undrained shear strength, e.g., Luu and et al. (1976), Koutsolias and Fischer (1976), and Stark and Delashaw (1990).

Data collected by Luu and Kleven (1981) and Janiokowski et al. (1982) show that for very soft to medium clays the cone factor based on \( S_u \) measured using a field vane shear test \( (N_{kv}) \) decreases with increasing plasticity index and ranges from 9 to 26. Bjerrum (1972) reviewed sixteen well-documented embankment, footings, and excavation failures through cohesive soils and developed the field vane correction factor, \( \chi_{kv} \). The correction factor reflects the influence of soil disturbance, progressive yielding, mode of shear, and strain rate on the difference between the undrained shear strength measured using the field vane test, \( S_u (FV) \), and the undrained shear strength mobilized at full-scale instability. If the vane shear strength values are corrected using Bjerrum’s correction factor, the resulting corrected cone factor
(N^*_{UV}) appears to be independent of plasticity index. As shown in Figure 1, N^*_{UV} values are between 8 and 24, with an average of approximately 15.

Figure 1. Previously published corrected field vane cone factors (after Lumes and Klevin 1981, and McElhiney 1987)

Stark and Delashaw (1990) developed a correlation between undrained shear strength from unconsolidated-undrained triaxial tests, S_u (UU), and cone tip resistance. They studied unconsolidated normally to lightly overconsolidated clays from twenty one different sites with an emphasis on San Diego, California. Unconsolidated-undrained (UU) triaxial compression test results were obtained using 38-mm-diameter specimens. The cone factor relating UU triaxial strength to the cone tip resistance, termed N_{tria}, ranges from 8.5 to 16.5, with an average value of approximately 12 (Figure 2).

Figure 2. Variation of UU triaxial cone factor with plasticity index (after Stark and Delashaw 1999)

The values of N_{tria} show less scatter than N^*_{UV} values. The reduction in scatter is probably due to the use of tip resistance values measured using only a standard electrical cone and the repeatability and simple interpretation of UU triaxial tests.

Terzaghi et al. (1996) showed that values of cone factor for stiff fissured clays based on UU compression triaxial tests on 100-mm-diameter specimens range from 11 to 30. The more fissures included in the specimen and the more local softening due to a slower strain rate during the laboratory undrained shearing, the lower the measured laboratory undrained shear strength compared to the cone resistance. Both effects should increase with the plasticity of the clay. Therefore, the value of the cone factor for stiff fissured clays depends on fissure spacing and plasticity.

3 NEW EMPIRICAL CONE FACTOR

A number of different techniques for measuring the undrained shear strength (field vane, unconfined compression, consolidated-undrained triaxial, and unconsolidated-undrained triaxial) were considered during this study. Despite the limitations of the unconfined compression test, the undrained shear strength obtained from this test is widely used in the geotechnical profession and in particular in Chicago. The popularity of the unconfined compression test is due to the ease in performing the test and interpreting the results compared to the vane shear test and other laboratory undrained shear strength tests.

Figure 3 compares the undrained shear strength from the unconfined compression test, S_u (UC), to the average mobilized strength along the surface of sliding of failed embankments S_{m} (mob). The S_m (UC) data for the cases summarized in Figure 3 correspond to tube samples of D to B quality (Terzaghi et al. 1996). The data in Figure 3 show that specimen disturbance in the unconfined compression test leads to S_m (UC) values that are smaller than S_u (FV). However, in the plasticity index range of 20 to 60%, S_m is close to unity. Therefore, for saturated clays in this plasticity range, the unconfined compression strength from tube samples of D to B quality provide a practical estimate of the mobilized undrained shear strength for foundation design, and embankment and excavation stability analyses (Terzaghi et al. 1996).

Owing to the popularity of the UC test and the direct correlation between S_m (UC) and the mobilized undrained shear strength in the field, a new cone factor that relates the unconfined compressive strength to the cone tip resistance (q_s) is presented herein and referred to as N_{tria}. The undrained shear strength data was obtained using 35 and 50 mm-diameter Shelby tube samples. Only cone soundings using a standard electrical cone advanced at approximately 2 cm/sec and in accordance with ASTM D3441 (ASTM 1990) were used in the correlation. The electric cones have a apex angle of 60 degrees and a projected area of 10 cm².
3.1 Chicago, Illinois sites

To facilitate use of the CPT in the Chicago area, a research program was initiated to develop a cone factor ($N_{uc}$) for Chicago soil deposits. Nine sites in downtown Chicago were studied to develop the cone factor. These sites/projects are the Chicago central library, Evanston tunnel, McCormick Place 3, Museum of Contemporary Art, Navy Pier, Northwestern University geotechnical test site, Canal & Harrison mall facility, Rush Presbyterian hospital, and University of Illinois engineering research facility.

The subsoil of the Chicago area consists of a series of glacial clays, each somewhat stiffer than the one above. Beneath the downtown districts of Evanston and Chicago, the clays have very soft to medium consistencies for thicknesses up to 15 meters. Hard clays are usually encountered before bedrock is reached, but many deposits of waterbearing sands and gravel are present near the rock (Peck and Reed 1954). Cone penetration tests at depths up to 21 meters were used to investigate the very soft to stiff clay layers. Stiffer clays were not considered because of the uncertainty in interpreting cone measurements in them. The average liquid limit, plastic limit, and clay-size fraction (% by weight < 0.002 mm) of Chicago clays are 32%, 17%, and 13%, respectively.

Figure 4 shows the net cone resistance ($q_n$) and the corresponding undrained shear strength from unconfined compression tests at the nine Chicago sites. It can be seen that the average value of $N_{uc}$ for Chicago soft to stiff, saturated clays is 15.5. Some of the scatter around the trend line is probably due to the difference in the disturbance of the undrained compression specimens.
3.2 Variation of the cone factor with plasticity index

Additional data were compiled from the literature and values of $N_{NC}$ were calculated for clays with similar consistency to investigate the accuracy of the $N_{NC}$ values for Chicago clays and the variation of the cone factor with plasticity index (PI). Only test areas with unconfined compressive strengths measured on test specimens having a degree of saturation at or near 100 percent were selected. In addition, only cone soundings, using a standard electric cone advanced at approximately 2 cm/sec and in accordance with ASTM D3441, were used. The test areas and source of the data are shown in Figure 5.

![Variation of UC cone factor with plasticity index](image)

Figure 5. Variation of UC cone factor with plasticity index

Figure 5 shows the variation of $N_{NC}$ with the plasticity index. Each data symbol represents the average value of PI and $N_{NC}$ calculated at each testing area, while the lines surrounding each point illustrate the range of PI and $N_{NC}$. It can be seen that the values of $N_{NC}$ range from 8 to 25 for all of the sites, with an average value of approximately 16.

To facilitate determination of the undrained shear strength, the data were plotted in terms of net cone resistance and undrained shear strength for each site in Figure 6. It can be seen that the majority of the data plot along a straight line corresponding to a value of $N_{NC}$ equal to 16. This average value of $N_{NC}$ is comparable to the corrected field vane factor $N_{CV}$ (Figure 1). In addition, the correction factor $u_{NC}$ shown in Figure 3 is equal to approximately unity in the plasticity index range over which the average $N_{NC}$ was approximated to be 16 (Figure 5). As a result, for sites with soft to stiff clays with a plasticity index ranging from 15 to 50 percent, the mobilized field undrained shear strength can be estimated using $N_{NC}$ approximately equal to 16. Therefore, the resulting undrained shear strength values do not have to be corrected, as suggested by Bjerrum (1973) for the field vane test results, to estimate the mobilized undrained shear strength.

4 CONCLUSIONS

The main objective of this research was to develop an empirical cone factor for the soft to stiff, saturated glacial clays of downtown Chicago, using the tip resistance from electrical cone penetration tests conducted in accordance with ASTM Standard D3441 and values of undrained shear strength measured in unconfined compression tests. The following conclusions are based on studying nine different sites in downtown Chicago and additional sites with clays of similar consistency.

1. The cone factor, $N_{NC}$, relating the unconfined compressive strength to the cone tip resistance has an average value of 15.5 for Chicago soft to stiff, saturated clays.

2. The unconfined compression strength for soft to stiff clays with plasticity indices of 15 to 50 percent for many sites including Chicago can be estimated using cone penetration test results and an $N_{NC}$ value of 16.

3. For soft to stiff, saturated clays with plasticity index ranges from 15 to 50 percent, the unconfined compressive strength calculated using the cone resistance and $N_{NC}$ equal to 16 provides a practical estimate of the mobilized undrained shear strength for foundation design, and embankment and excavation stability analyses.

ACKNOWLEDGEMENTS

The writers acknowledge Robert G. Lukas of Ground Eng Consultants, Inc., Tony A. Kiefer of STS Consultants Ltd., Michael H. Wymacek, and Nelson Kawamura of the University of Illinois at Urbana-Champaign for their help in collecting data for the Chicago area. The help of Gordon P. Boutwell of Soil Testing Engineers, Inc. in compiling data for the Louisiana and Alabama sites is also acknowledged.
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Cone penetration resistance of sand from seismic tests

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ABSTRACT: This paper presents a procedure to estimate geotechnical properties of cohesionless soils from seismic test results. A large published data base derived from three different series of CPT calibration chamber and resonant column tests on sands with different stress history and compressibility are reviewed. A relationship is suggested to estimate cone penetration resistance from small strain shear modulus measured in seismic methods, so that data from seismic tests can be used in existing CPT based design correlations. In particular, the proposed method can be applied to nondestructive/intrusive seismic methods which require no borehole for site characterization. It is shown that the proposed correlation is influenced very little by soil compressibility. The influence of stress history on the correlation is also investigated and discussed. The method can be applied to unconfined unaged cohesionless soils. Field evidences are used to illustrate and evaluate the proposed procedures.

1 INTRODUCTION

Site characterization has been a major part of geotechnical engineering practice and various drilling, sampling and in-situ testing techniques are now well established. A major concern in soil characterization is whether the soil is tested in its natural physical, chemical and biological environment. Common causes of disturbances to the sample are mechanical disturbance of the soil structure during drilling, sampling, transportation, storage and handling in the lab as well as change in water content and stress conditions and possible chemical changes and mixing and segregation of the soil constituents. In particular, where cohesionless soils are encountered, reliable soil characteristics can not be obtained using conventional sampling and laboratory testing, due to unavoidable disturbance to the samples and difficulties associated with sampling. Drilling techniques generally produce considerable disturbance to the materials surrounding the drill hole, which can have a significant effect on subsequent sample quality.

The most rapidly developing site characterization techniques for geotechnical purposes involve direct push technology, i.e. penetration tests. The direct push devices produce little overall disturbance. The most popular test for geotechnical investigations in soil is the Cone Penetration Test (CPT).

Seismic testing are among the few in-situ tests that produce little or no soil disturbance. Such techniques induce very small strain in the ground that preserve its natural feature. Seismic methods measure small strain response of a large volume of the ground, whereas the penetration of the cone measures large strain response of the ground since average stress levels around the cone approximately equals failure in the soil. During seismic tests, the shear wave velocity \( V_s \) can be measured. Based on elastic theory, the small strain shear modulus \( G_s \) can be determined from the seismic shear wave velocity \( V_s \) using:

\[
G_s = \rho (V_s)^2
\]  

where, \( \rho \) is soil mass density.

The Standard Penetration Test (SPT) results in gravelly soils become unreliable and often too high, due to the large particle size relative to the sampler diameter. The SPT also has very poor resolution in soft fine grained soils, such as silts and sandy silt. The penetration of SPT in most dense soil formations, particularly in gravel and cobble deposits is relatively slow and sometimes impractical.

The CPT is ideal for loose soils since the pushing force is small. Application of the CPT in very stiff (hard) soils can require large push forces. The CPT is difficult in cemented soils, dense sand layers and gravely soils and boulders because they restrict the penetration and deflect CPT rods and damage cones.

In coarse-grained soils Non-destructive Seismic
techniques have found useful applications. To make use of world-wide foundation performance data base currently available for the CPT, there is a need to have reliable \( q_c \)-\( G_s \) correlations.

Methods employing wave propagation principles in determining elastic properties of ground can be intrusive or nonintrusive. Intrusive methods require physical penetration of the ground or borehole drilling to determine compressional and shear wave velocities. Nonintrusive methods are used to determine velocity profiles from the ground surface, such as surface reflection, surface refraction, steady-state vibration and spectral analysis of surface waves (SASW). Surface reflection and refraction are generally more suited to geophysical exploration work and can not provide accurate profiles of shear wave velocities for geotechnical purposes. To a varying extent, the first two nonintrusive methods use surface wave dispersion to indirectly determine shear wave velocities. (Addo and Robertson, 1992)

Spectral analysis of surface waves (SASW) employs the phenomenon of dispersion to determine the body-wave velocities of the medium of propagation. It shares all the advantages of the steady-state method and improves upon it by the deployment of a polychromatic source and an inversion scheme based on elastodynamic theory. Field testing is therefore much quicker, and resulting shear-wave velocity profiles from the inversion process are potentially more accurate.

The objective of this paper is to present a procedure to estimate geotechnical properties of cohesionless soils from seismic test results. The method is based on the correlation between cone penetration resistance and small strain shear modulus.

2 PROPOSED METHOD OF ESTIMATION \( q_c \) FROM \( G_s \)

Geotechnical engineers often use penetration resistance from the CPT to estimate the small strain shear modulus, \( G_s \). Several correlations have been developed, mainly based on calibration chamber test and resonant column test results, to relate \( G_s \) with CPT tip resistance \( q_c \) for sands where the grain materials are predominantly quartz (e.g. Robertson and Campanella 1983). However, with the advent of the seismic CPT (SCPT) in which both the cone resistance and small strain shear modulus are measured during the same sounding in the same soil, the application of such correlations is not justified in practice. The available correlations between \( G_s \) and \( q_c \) mostly require a trial and error to obtain \( q_c \) from \( G_s \) (e.g. Robertson and Campanella 1983 and Esfahami and Robertson 1996).

For a direct estimation of \( q_c \) from \( G_s \), an attempt has been made to correlate cone penetration resistance \( q_c \) with small strain shear modulus \( G_s \) and vertical

![Figure 1. Correlation between \( q_c \) and \( G_s \) for uncedemented unaged sands](image)

1026
Figure 2. $q_c - G_o$ correlation for uncremented unaged normally consolidated sands

Figure 3. $q_c - G_o$ correlation for overconsolidated sands (1<OCR<15)
Figure 4. $q_c - G_o$ correlation for highly overconsolidated sands ($7 < OCR < 15$)

Figure 5. Proposed $q_c - G_o$ correlations
effective stress $\sigma_{se}'$ for unaged unconsolidated sands with variable compressibility. A regression analysis based on data from a large number (210 tests) of published calibration chamber test results (Baldi et al. 1986, Fiorevante et al. 1991 and Almeida et al. 1991) on unconsolidated normally consolidated and overconsolidated Ticino sand, Toyoura sand and Quiou sand gives the following correlation:

$$ \frac{q_u}{P_s} = \left( \frac{G_s}{205.2P_s} \right)^{2.03} $$  \hspace{1cm} (2)

where,

$$ G_s = \frac{G_k}{P_s} \left( \frac{P_s}{\sigma_{se}'} \right)^{2.3} \hspace{1cm} (3)$$

in which, $G_k$ is normalized small strain shear modulus, and $P_s$ is atmospheric pressure in same units as $G_k$, $q_u$, and $\sigma_{se}'$.

Figure 1 presents a summary of the calibration chamber test results in the form of Eq. 2. The correlation factor for the above regression analysis is $R^2 = 0.84$. In general, $\sigma_{se}' / P_s$ varies between 0.5 to 2.0, and $G_k / P_s$ ranges from 270 to 1800. Hence, $G_s$ varies between 300 to 1500.

Figure 1 also illustrates that the proposed correlation is influenced little by soil compressibility since highly compressible Quiou sand plot within the same band as low compressible Ticino and Toyoura sands. Hence, the proposed correlation between $G_s$ and $q_u$ is not highly influenced by sand mineralogy. Overconsolidated sands, however, appear to fall to the right side of the band. This indicates a limited influence of stress history on the proposed correlation.

A large portion of the above tests (139 tests) were carried out in normally consolidated sands. This can dominate Eq. 2 to provide a better result in normally consolidated sands than in overconsolidated sands. Hence, to avoid such effects, separate relationships for sands with various overconsolidation ratios are required. However, when stress history of sand is unknown Eq. 2 can be used to estimate cone penetration resistance with reasonable accuracy. Practitioners should be cautious that $q_u$ can be overestimated using Eq. 2 for highly overconsolidated sands.

The general form of the $q_u - G_s$ correlation can be written as follow:

$$ \frac{q_u}{P_s} = \left( \frac{G_s}{A} \right)^{2.0} $$ \hspace{1cm} (4)

Table 1 summarizes the results of various regression analyses based on calibration chamber test results on unconsolidated normally consolidated sands, overconsolidated sands and highly overconsolidated sands of the same data base.

<table>
<thead>
<tr>
<th>Sand</th>
<th>No. of samples</th>
<th>A</th>
<th>B</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC+OC</td>
<td>210</td>
<td>205.3</td>
<td>3.53</td>
<td>0.84</td>
</tr>
<tr>
<td>NC</td>
<td>139</td>
<td>207.5</td>
<td>3.60</td>
<td>0.86</td>
</tr>
<tr>
<td>OC</td>
<td>71</td>
<td>253.7</td>
<td>4.00</td>
<td>0.80</td>
</tr>
<tr>
<td>1&lt;OCR&lt;15</td>
<td>16</td>
<td>315.2</td>
<td>4.60</td>
<td>0.91</td>
</tr>
</tbody>
</table>

Figures 2, 3 and 4 illustrate the proposed correlations based on regression analyses for NC sands, OC sands and highly overconsolidated sands, respectively.

Figure 5 summarizes the correlations proposed for both normally consolidated sands (OCR = 1) and highly overconsolidated sands (OCR > 10). This chart can be used to estimate cone penetration resistance, for unaged unaged sands with various compressibility and different level of stress history. Also shown in Figure 5 is the field data from Kidd, Massey and Alaska sites (Robertson et al., 1995).

The Kidd and Massey sites are located near Vancouver, B.C. Both sites contain natural alluvial sediments as part of the Fraser River delta. At both sites, SCPT data are within a 20 to 30 m thick complex of distributary channel sands that underlies most of the delta plain (Monahan et al., 1995). Fraser River sand is a young, unconsolidated predominantly quartz sand with some mica and feldspar, has a $D_{10} = 0.30$ mm and contains on average about 5% fines. Measured in-situ void ratio has been reported as 0.70 to 1.06 at Kidd site, and 0.68 to 1.06 at Massey site. Seismic CPT’s have been performed at various depths and locations at these sites. Both sites have been identified as normally consolidated sands having a coefficient of earth pressure of about $K_0 = 0.4$ based on self-boring pressuremeter test results. Both sites were part of the CANLEX project (Robertson et al., 1995).

Alaska sand is a tailings deposit which has been deposited into the sea and has a 30% fines content. The fines have a high crushed shell content and, hence, Alaska sand has high compressibility.

Figure 5 illustrates that there is a good agreement between the proposed correlation for normally
consolidated sands and field data of Kidd, Messey and Alaska sands. Unfortunately, there is limited SCPT data available from overconsolidated sands to fully evaluate the proposed method.

3 CONCLUSIONS

A procedure has been presented to estimate cone penetration resistance from small strain shear modulus. The proposed method can be used in unstressed unsaturated sands. The proposed method can be applied in cases where the CPT may not be possible due to difficult access, hard ground conditions, gravel layers, cobbles and boulders, etc. The resulting estimated values of qc can be used for subsequent geotechnical design. The proposed correlations are not highly influenced by sand mineralogy. The limited influence of overconsolidation on the correlations have been evaluated and discussed. Seismic CPT data of normally consolidated sands of various compressibility from 3 sites have been used to evaluate the proposed correlations. The estimated values of cone resistance have been found in good agreement with those measured at field.

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Cone penetration testing in Irish soils
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ABSTRACT: This paper examines the application of cone penetration testing to typical Irish soil conditions in a programme of tests specifically aimed at addressing the scepticism amongst practitioners of its viability for routine site investigations. In all, twenty soils were tested at sites where comprehensive borehole data existed. Internationally accepted correlations are seen, in general, to provide a good basis for soil classification and site characterisation.

1. INTRODUCTION

The use of the CPT or piezocone test (CPTU) in Ireland has been relatively limited up to recently. Practitioners have preferred other in-situ tests such as the SPT or field vane, quoting reasons for such preferences as the high intensity of cobbles in Ireland's glacial tills or the extremely weak nature of much of the alluvial deposits/peat. As a consequence of this scepticism, CPT/CPTU tests were conducted by Trinity College Dublin (TCID) and three other contractors in a total of twenty soil types between January and July 1997. The primary aim of this testing programme was to demonstrate the viability and usefulness of cone testing as a means of site characterisation in typical Irish ground conditions.

2. GEOLOGY

Most subsols found in Ireland were derived from the complex series of deposits that were laid down over two successive Quaternary period glaciations which terminated about 10,000 years ago. Post-glacial deposits include alluvium in the floodplains of Ireland's rivers and lacustrine accumulations in glacial lakes and estuarine deposits. Poor drainage and a wet climate have been such that a relatively thin mantle of peat covers one seventh of the area of Ireland. The depth to bedrock (which is limestone over much of the country) is typically not greater than 20m.

Three of the most commonly encountered and economically important soils in Ireland are described in the following; typical CPT logs at sites including these soils are shown on Figure 2.

2.1 Dublin boulder clay

Over one third of Ireland's population live in the Dublin area, most of which is underlain by a material known as Dublin boulder clay. A slightly weathered brown boulder clay layer with a thickness less than 3m generally overlies black boulder clay. The latter is a hard, heavily overconsolidated, well graded, low plasticity clay (fracture) till, with a lower water content and a higher undrained shear strength than the brown boulder clay. The high stone content and exceptional strength of the overconsolidated till often makes good quality sampling extremely difficult and impractical. Estimates of the in-situ undrained shear strength are generally made from SPT N values and range from 300 kPa to 700 kPa (Farrell et al., 1988).

2.2 Belfast estuarine silt

A post-glacial estuarine deposit known locally as "bleach", which is a grey soil sensitive silty clay is of particular interest as it underlies a considerable portion of Belfast and its surrounding area (see Figure 1). The layer has been considered unsuitable as a bearing stratum because of its high compressibility and low strength and loads from heavy structures are normally carried by piles to competent underlying materials. However, major reconstruction and increased construction activity in the densely developed city centre has necessitated detailed studies of the geotechnical properties of the deposit.
2.3 Organic soils of the midlands

The organic soils found in Cavan (see Figure 1) are typical of the soft organic soils existing throughout the country. These soils are extremely soft and compressible (vane shear strengths are often less than 5kPa) and are frequently interbedded with layers of peat. The problems presented by these deposits are of considerable importance to many Irish towns.

1. Slowing the rate of cone installation from the standard rate of 2cm/s to 0.5cm/s in some glacial tills. This reduction in rate, while having a relatively small influence on the cone end resistance (Meigh 1987), gave the operator sufficient time to react and halt the cone whenever a cobble obstruction was encountered. As a consequence, overestimating of the cone’s load cells was prevented.

2. Pre-boring using a mechanical cone through the upper fill layers at some sites.

3. Careful selection of load cell ‘zeros’ in soft soils in addition to care over casting arrangements to facilitate access.

4. SOIL TYPE CLASSIFICATION

Soil type was identified and categorised with the aid of the classification charts proposed by Robertson (1990) which are based on normalised CPT and CPTU records. The data recorded at each site are summarised on these charts on Figure 3. Points of note include:

1. In all cases, the interpreted stratigraphy was broadly in line with that encountered in the boreholes. In addition, the tests indicated the presence of sand and clay lenses which were not recorded on the borehole logs.

2. Friction ratios in excess of the 10% maximum indicated on the chart were measured in the organic soils found in Cavan and Athlone. These ratios also exhibited a high degree of variability, ranging from 5% to 90%; such apparent variability is attributed primarily to inadequate accuracy of resolution in the measurements of $q_4$ and $f_4$ (both of which are very small).

3. The $q_4$ and $f_4$ data for black (unweathered) Dublin boulder clay suggest the material is a very stiff sand to clayey sand (i.e. soil type 8). Such a description does not reflect the very low in-situ permeability of the material seen in practice ($k=10^{-7}$ m/s). Pore pressure data (i.e. from a CPTU test) in addition to $q_4$ and $f_4$ thus lead to a much improved characterisation of this material.

4. The high $B_c$ value for peat ($C_b$) distinguishes it from the other organic clays in the data base of soils investigated. Notably, the friction ratio chart classifies the peat as soil type 2 and the $B_c$ chart classifies it as soil type 1.

5. Pore pressures measured in the Irish clay tills were negative and therefore their accuracy cannot be relied upon. This is a typical response in heavily overconsolidated clays and should ideally be accommodated in the $B_c$ classification chart.
A feature of the test results which has particular relevance to geotechnical design in Ireland was the tendency for the q<sub>t</sub> profiles to indicate that the consistency of a given site was significantly less variable than suggested by SPT N values. It follows that the high sensitivity of SPT N values to drilling disturbance leads to an over-prediction of the degree of variability of given site.

5. Interpreted soil parameters

5.1 Cone factor, N<sub>c</sub>

In-situ vane tests are normally conducted to assess the in-situ undrained shear strength (c<sub>u</sub>) of the soil alluvial organic soils in Ireland. Cone factors, N<sub>c</sub> = (q<sub>t</sub> - m<sub>c</sub>) / (3c<sub>u</sub>), based on vane strengths, were

---

**Figure 2.** Typical CPT and other profiles at Clonee, Belfast and Cavan
Figure 3. Soil type classification charts.

Table 1. Soil type designation.

<table>
<thead>
<tr>
<th>Site</th>
<th>Soil encountered</th>
<th>Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>Abalone</td>
<td>Mad</td>
<td>Aa</td>
</tr>
<tr>
<td></td>
<td>Grey Organic Clay</td>
<td>Aa</td>
</tr>
<tr>
<td></td>
<td>Brown Laminated Clay</td>
<td>Aa</td>
</tr>
<tr>
<td></td>
<td>Glacial Till</td>
<td>Aa</td>
</tr>
<tr>
<td>Belfast</td>
<td>&quot;Sleach&quot; Estuarine Silt</td>
<td>Bb</td>
</tr>
<tr>
<td></td>
<td>Sand</td>
<td>Bb</td>
</tr>
<tr>
<td>Cavan</td>
<td>Sandy Clayey Silt</td>
<td>C1</td>
</tr>
<tr>
<td></td>
<td>Peat</td>
<td>C1</td>
</tr>
<tr>
<td></td>
<td>brown silty Clay</td>
<td>Cb</td>
</tr>
<tr>
<td></td>
<td>Gravel</td>
<td>Cb</td>
</tr>
<tr>
<td>Clone</td>
<td>Brown Boulder Clay</td>
<td>D1</td>
</tr>
<tr>
<td></td>
<td>Black Boulder Clay</td>
<td>D1</td>
</tr>
<tr>
<td>Dublin Port</td>
<td>Hydraulic Fill</td>
<td>D1</td>
</tr>
<tr>
<td></td>
<td>Estuarine silty Clay</td>
<td>D1</td>
</tr>
<tr>
<td></td>
<td>Gravel</td>
<td>En</td>
</tr>
<tr>
<td>Kildare</td>
<td>soft sandy gravelly Clay</td>
<td>E1</td>
</tr>
<tr>
<td></td>
<td>firm sandy gravelly Clay</td>
<td>E1</td>
</tr>
<tr>
<td>Killarney</td>
<td>Glacial Till</td>
<td>G1</td>
</tr>
<tr>
<td>Portlaise</td>
<td>Glacial Till</td>
<td>H1</td>
</tr>
<tr>
<td>Ringend</td>
<td>laminated port Clay</td>
<td>H1</td>
</tr>
</tbody>
</table>

backfigured at each of the sites with these materials and are summarised on Table 2. It is apparent that N<sub>n</sub> depends significantly on the soil type, e.g. N<sub>n</sub> =18 in Dublin estuarine silty clay, =14 in the Belfast silt and =10 in the peat at Cavan. The standard deviations of the N<sub>n</sub> values (also given in Table 2) show a wide range, presumably again associated with measurement inaccuracies at very low cone resistances. Cone testing should therefore be used as an exclusive replacement for the assessment of in-situ undrained strength.

In the much stiffer glacial clay tilts, because of the previously mentioned sampling problems, it is common practice to assume that the undrained shear strength in triaxial compression (C<sub>u</sub>) is (in kPa units) approximately six times the SPT N value (Stroud, 1974). Application of this correlation suggests N<sub>n</sub> values in excess of 35 and well above the commonly encountered range of 12 to 25 (e.g. see Lurie et al., 1985). It follows that Stroud's relationship potentially under-predicts the in-situ strength of Dublin boulder clay by a least 30%.
5.2 Overconsolidation ratio

The overconsolidation ratio (OCR) in clay soils was estimated using the simple hybrid theory developed by Mayne (1993), based on spherical cavity expansion and Modified Cam Clay. The theory relates the in-situ OCR to the normalised piezocene parameter \((q_{s0}/\gamma_s)\) and effective friction angle \(\phi'\) of the clay.

The OCR values measured in oedometer tests on samples of the organic soft clays in Cavan and Athlone and the estuarine deposits in Belfast and Dublin Port are in good agreement with Mayne’s theory. However, the same theory predicts OCRs in the glacial till sites of well in excess of 100. Oedometer tests, which involved loading of clay till samples to a maximum stress of 10 MPa, indicated OCRs for the till from below 3m depth of between 10 and 20.

It is noteworthy that the CPT data showed the till from Kildare which is thought to be exoglacial in origin to have a significantly lower OCRs and strength than the three lodgement tills at Clonee, Kiltteel & Portlaoise (see Figure 4).

5.3 Relationship with SPT N

The relationship between cone resistance \(q_c\) and SPT N value for each cohesionless soil is investigated on Figure 5 which reproduces Burland and Burbidge’s (1985) correlation between \(q_c/N\) and average particle size \(D_10\). It is evident that all the results fall within the proposed zone. The scatter within this band, however, suggests that use of \(q_c\) directly in empirical correlations should be carried out rather than backfiguring SPT N values.

5.4 Relative density (DR)

Relative density estimates were made for the cohesionless soils using Lonne and Christoffersen’s (1983) correlation with \(q_c\) and Skempton’s (1962) correlation with the ‘corrected’ SPT N value. Good agreement was found between both approaches in the soil underlying the Belfast alluvial silt and the gravel at Dublin Port. The SPT N data in the hydraulic fill at

<table>
<thead>
<tr>
<th>Site</th>
<th>Soil Type</th>
<th>(N_{50}) (in-situ value)</th>
<th>Standard Deviation</th>
<th>(N_{50}) (SPT N value)</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Athlone</td>
<td>Marl</td>
<td>15</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>grey organic Clay</td>
<td>14</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>brown laminated Clay</td>
<td>16</td>
<td>9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Belfast</td>
<td>“Sleek” estuarine Silt</td>
<td>14</td>
<td>7</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>sandy clayey Silt</td>
<td>6</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Peat</td>
<td>4</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>brown silty Clay</td>
<td>5</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clonee</td>
<td>brown boulder Clay</td>
<td>54</td>
<td>54</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>black boulder Clay</td>
<td>36</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dublin Port</td>
<td>estuarine silty Clay</td>
<td>18</td>
<td>9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kildare</td>
<td>grey sandy gravelly Clay</td>
<td>100</td>
<td>27</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Dublin Port, however, indicated a D3 value of 40% compared to value of 68% interpreted from the qc data. This discrepancy may be partly associated with high hydraulic gradients at the base or boreholes during execution of the SPT’s.

6. CONCLUSIONS

A programme of cone penetration testing was conducted in Ireland with a primary objective being to inspire confidence amongst Irish practitioners in the specification of the CPT for routine geotechnical projects. The findings of this paper provide conclusive proof of its viability and its usefulness as a means of site characterisation. Classification from cone testing will inevitably improve as CPT usage increases and cone parameters particular to Irish soils become better established.

ACKNOWLEDGEMENTS

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ABSTRACT: The paper summarizes the results of CPT’s, DMT’s and PLT’s performed in Calibration Chambers in saturated carbonatic Quiou sand. Such results, in combination with those of triaxial and resonant column tests, permitted a preliminary assessment of correlations between in situ tests results and stiffness of the test sand.

1. INTRODUCTION

The bioelastic carbonate sands are mostly, but not exclusively, encountered offshore and onshore of the South Mediterranean Sea, East Indian Ocean, Persian Gulf, Red Sea, Western Australia, and Brazilian coasts. Such deposits composed primarily of calcium carbonate (CaCO₃) exhibit a complex stress-strain-time behaviour and have recalled the attention of geotechnical engineers in relation with the design and construction of offshore structures of the oil industry.

The mechanical behaviour of carbonate sands is different from the siliceous sands behaviour because of the following main features:

- the occurrence in nature at much higher void ratio (e);
- the angular and subangular grains consisting of fragments of marine animals and plants, faecal pellets, detrital elements [Fookes (1988)] having low mechanical strength;
- the random and generally weak to moderate cementation due to chemical precipitation of calcite.

The first two characteristics confers to carbonate sands a relatively high volumetric compressibility, a prevalent contractive behaviour in shear, even at low confining stress, and a low stiffness. More comprehensive information on the mechanical behaviour of carbonate sands can be found in the works by Semple (1988), Poulos (1989), Randolph et al. (1993), Fabey (1993), and Jawell (1993).

In the light of the above, the Hydraulic and Structural Research Center of the Italian National Electricity Board (ENEL CRIS) of Milano, the ISMES of Bergamo and the Department of Structural Engineering of the Technical University of Torino have undertaken a research aimed at the interpretation of the most common in situ tests in carbonate sands.

The research involves:
- Static Cone Penetration Test (CPT)
- Multichannel Flat Dilatometer Test (DMT)
- Borehole Pile Loading Test (PLT)

performed in carbonate Quiou sand (QS) [Golightly (1988)] in Calibration Chambers (CC).

In addition, laboratory tests have been carried out aimed at the stress-strain characterisation of the test sand.

2. EXPERIMENTAL WORK

The present chapter summarizes the experimental work performed in the writers laboratories on uncemented gravelsly deposited QS.

2.1 Calibration Chamber

Two CC’s have been used whose characteristics are given in Table 1.

<table>
<thead>
<tr>
<th>CC no</th>
<th>H₀ mm</th>
<th>D₀ mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1500</td>
<td>1200</td>
</tr>
<tr>
<td>2</td>
<td>1600</td>
<td>600</td>
</tr>
</tbody>
</table>

H₀ = specimen height
D₀ = specimen diameter
In both chambers it is possible to perform tests using in situ devices under strictly controlled boundary conditions (BC) imposed on the boundary stresses and strains.

All CC tests have been carried out according to the following stages:

a) Preparation of the specimen by pluvial deposition.

b) Saturation of the specimen with desired water. This operation has been checked by means of Skempton's pore pressure coefficient $B = 0.93$.

c) Consolidation of the specimen to the desired stress level and overconsolidation ratio (OCR). During this stage BC3 has been imposed on the specimen, corresponding to controlled vertical stress ($\sigma'_v$) and zero horizontal strain ($\varepsilon'_h$).

d) Execution of "in situ" tests by penetrating the CC with CPT or DMT and by loading incrementally the plate. This latter device was inserted in the specimen during its preparation.

Details concerning CC's features and the related tests can be found in works by Bellotti et al. (1982, 1988), Ghionna and Jamiolowska (1991), Bellotti and Pedroni (1991) and Ghionna et al. (1994).

2.2 In Situ Devices

Electrical cones manufactured by ISMES have been used, whose dimensions are summarized in Table 2.

<table>
<thead>
<tr>
<th>$d_c$ (mm)</th>
<th>$A_c$ (mm$^2$)</th>
<th>$A_s$ (mm$^2$)</th>
<th>$D_0 (1200)$</th>
<th>$D_0 (60)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>35.7</td>
<td>1000</td>
<td>15000</td>
<td>33.6</td>
<td>16.9</td>
</tr>
<tr>
<td>20.0</td>
<td>314</td>
<td>none</td>
<td>60.0</td>
<td>30.0</td>
</tr>
</tbody>
</table>

$d_c = $ cone diameter  
$A_c = $ cone area  
$A_s = $ sleeve friction area

As a limited number of cone penetration tests have been performed using a cone tip allowing the measurement of the pore pressure (CPTU) during penetration [Almeida et al. (1991)], the obtained results showed that the penetration process in QS is occurring in practically drained conditions.

The dilatometer used in the CC tests was a standard device conforming the ASTM (1986) and the description reported by Marchetti (1980, 1997).

The steel plate used for the borehole loading tests was 104 mm in diameter (D) and 10 mm in thickness and can be considered as infinitely rigid in relation to the stiffness of the QS. The overall arrangement adopted for the PLT can be inferred from Figure 1.

All CC with the above in situ devices have been performed under:

- BC1, i.e. $\sigma'_v = $ constant; and $\varepsilon'_h = $ constant
- BC3, i.e. $\varepsilon'_h = $ constant; and $\varepsilon'_h = 0$.

2.3 Test sand

The QS is a skeletal carbonate sand of biogenic origin dug-out from a borrow pit close to the village of Plouneve in Bretagne (France). The mineralogical and morphological characteristics of the QS and its grading are shown in Table 3 and Fig.2 respectively. Its specific
Table 3 - Mineralogical and morphological analyses of Quiou sand.

<table>
<thead>
<tr>
<th>Mineralogical composition</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shell fragments</td>
<td>73.5%</td>
</tr>
<tr>
<td>Calcium carbonate aggregates</td>
<td>14.5%</td>
</tr>
<tr>
<td>Quartz</td>
<td>11.8%</td>
</tr>
<tr>
<td>Rock fragments</td>
<td>0.2%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Grain shape</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very angular</td>
<td>1.5%</td>
</tr>
<tr>
<td>Angular</td>
<td>18.2%</td>
</tr>
<tr>
<td>Subangular</td>
<td>50.0%</td>
</tr>
<tr>
<td>Subrounded</td>
<td>28.8%</td>
</tr>
<tr>
<td>Rounded</td>
<td>1.5%</td>
</tr>
</tbody>
</table>

3. TEST RESULTS

In this chapter after a brief resume of the QS stiffness as obtained from laboratory tests, the results of CPT’s, DMT’s and PLT’s performed in the CC are reported.

3.1 Stiffness

The stiffness of QS sand has been evaluated by means of isotropically (CID) and anisotropically consolidated (CAD) drained triaxial compression and torsional shear tests. In addition, the small strain initial stiffness has been measured during a number of resonant column tests performed on isotropically consolidated specimens. All laboratory tests have been performed on saturated specimens of pluvially deposited sand. In these tests involving specimens whose volume does not exceed 540000 mm³ only virgin sand has been employed discarding the material after the experiment. As to the initial pseudo-elastic stiffness evaluated at strain less than linear threshold strain, the following empirical relationships between deformation modulus soil state and stress history have been used to fit the experimental data:

\[ E_0 = C_u F_0 (\sigma_{v0} / \sigma_0)^{k \cdot \text{OCR}} \]

where:
- \( E_0 \) = pseudo-elastic initial Young’s modulus
- \( C_u \) = anisotropic nondimensional material constant referred to \( E_0 \)
- \( F_0 \) = void ratio function = \( e^x \)
- \( \sigma_{v0} \) = effective mean consolidation stress
- \( \sigma_0 \) = reference stress = 1 bar = 100 kPa
- \( n_0 \) = modulus exponent
- \( k \) = OCR exponent

Fitting the results of a large number of triaxial compression tests the following parameters of eq.(1) have been determined: \( C_u = 2562, x = 1.08, n_0 = 0.525 \) and \( k = 0.199 \).
\[ g_\alpha = C_\alpha f(e) \left( \sigma_{\max} \right)^n \left( \rho_s \right)^m \times (OCR)^k \]  

(2)

where:
\[ G_i \] = pseudo-elastic initial shear modulus
\[ C'_c \] = anisotropic [in this case \( G_c = G_{cV} \)] shear modulus on vertical plane
\[ n \] = nondimensional material constant referred to \( G_c \)
\[ F(e) = e^a \]

Fitting the results of numerous resonant column tests, the following parameters of eq (2) have been inferred:
\[ C'_c = 933.2 \times 1.30, n = 0.612, k = 0.313 \]

The above formulae holds for \( e < e_{\text{v}} = (1 + 2) \times 10^{-4} \), in case of QS.

As soil stiffness beyond \( e_{\text{v}} \), QS exhibits a highly pronounced non-linearity as shown in Fig.3 giving examples of degradation of the normalized Young's modulus \( E/E_{\text{v}} \) as function of the mobilized deviator stress \( q/\sigma_V \), being \( q \) and \( \sigma_V \) applied and failure deviator stresses respectively. The decay of the QS secant stiffness with deviator stress level can be fit by means of quasi-linear models proposed by Tatsuoka and Shibuya (1992) and Burland and Parkin (1990).

Details of the above models and the related parameters describing the stress-strain curves of QS can be found in the work by Lo Presti et al. (1997).

In addition, it is worth mentioning here that QS exhibits a pronounced creep when subjected to the prolonged action of isotropic or anisotropic effective stresses. For example, the growth of QS observed in laboratory, under constant consolidation stress, ranging between 100 and 400 kPa, results between 5 to 6 percent for tenfold increase in time.

During triaxial compression tests, both local \( e \) and \( v \) were measured, it was possible to evaluate Poisson ratio \( v \). At small strain \( v \rightarrow v \) resulted to range between 0.10 and 0.15. As \( e \) increases the \( v \) also increases reaching values around 0.30 in correspondence of axial strain \( 15\% \).

### 3.2 CPT's

CPT's results performed on saturated QS in small and large CC's are summarized in Tables 4 and 5 respectively.

Because of the test sand high compressibility [Bellotti et al. (1991b)], the values of cone resistance \( q_c \) resulted from both series of tests, are comparable, although the values of \( D/D_c \) range between 17 and 60. This trend of QS to exhibit a modest chamber size effect [Parkin and

<p>| Table 4 - Summary of CPT's performed in small calibration chamber. |
|---|---|---|---|---|</p>
<table>
<thead>
<tr>
<th>test n.</th>
<th>BC</th>
<th>( \sigma_{\text{v}} )</th>
<th>OCR</th>
<th>( q_c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>B1</td>
<td>0.738</td>
<td>0.62</td>
<td>1</td>
</tr>
<tr>
<td>7</td>
<td>B1</td>
<td>0.759</td>
<td>0.65</td>
<td>1</td>
</tr>
<tr>
<td>8</td>
<td>B1</td>
<td>0.749</td>
<td>0.62</td>
<td>1</td>
</tr>
<tr>
<td>21</td>
<td>B1</td>
<td>0.753</td>
<td>1.24</td>
<td>1</td>
</tr>
<tr>
<td>22</td>
<td>B1</td>
<td>0.781</td>
<td>0.90</td>
<td>1</td>
</tr>
<tr>
<td>23</td>
<td>B1</td>
<td>0.731</td>
<td>2.38</td>
<td>4.2</td>
</tr>
<tr>
<td>24</td>
<td>B1</td>
<td>0.727</td>
<td>0.75</td>
<td>3.9</td>
</tr>
<tr>
<td>25</td>
<td>B1</td>
<td>0.766</td>
<td>0.45</td>
<td>2.0</td>
</tr>
<tr>
<td>26</td>
<td>B1</td>
<td>0.719</td>
<td>0.74</td>
<td>2.0</td>
</tr>
<tr>
<td>27</td>
<td>B1</td>
<td>0.755</td>
<td>0.36</td>
<td>2.0</td>
</tr>
<tr>
<td>28</td>
<td>B1</td>
<td>0.717</td>
<td>0.40</td>
<td>5.8</td>
</tr>
<tr>
<td>29</td>
<td>B1</td>
<td>0.757</td>
<td>1.83</td>
<td>1</td>
</tr>
<tr>
<td>30</td>
<td>B1</td>
<td>0.737</td>
<td>0.93</td>
<td>1</td>
</tr>
<tr>
<td>31</td>
<td>B1</td>
<td>0.742</td>
<td>1.04</td>
<td>1</td>
</tr>
<tr>
<td>32</td>
<td>B1</td>
<td>0.852</td>
<td>0.86</td>
<td>3.9</td>
</tr>
<tr>
<td>33</td>
<td>B1</td>
<td>0.871</td>
<td>0.78</td>
<td>2.1</td>
</tr>
<tr>
<td>34</td>
<td>B1</td>
<td>0.859</td>
<td>0.73</td>
<td>1</td>
</tr>
<tr>
<td>35</td>
<td>B1</td>
<td>0.842</td>
<td>0.95</td>
<td>4.9</td>
</tr>
<tr>
<td>26</td>
<td>B1</td>
<td>0.839</td>
<td>0.82</td>
<td>3.0</td>
</tr>
<tr>
<td>27</td>
<td>B1</td>
<td>0.848</td>
<td>0.86</td>
<td>6.0</td>
</tr>
<tr>
<td>38</td>
<td>B1</td>
<td>0.904</td>
<td>0.72</td>
<td>1</td>
</tr>
<tr>
<td>39</td>
<td>B1</td>
<td>0.885</td>
<td>1.17</td>
<td>1</td>
</tr>
<tr>
<td>41</td>
<td>B1</td>
<td>0.914</td>
<td>0.45</td>
<td>1</td>
</tr>
</tbody>
</table>

Fig. 3 - Non-linearity of carbonate Queso sand during triaxial compression tests.
Table 5: Summary of CPT's performed in large calibration chamber.

<table>
<thead>
<tr>
<th>test n.</th>
<th>BC</th>
<th>e</th>
<th>σ_m</th>
<th>OCR</th>
<th>q_c</th>
<th>q_o</th>
<th>σ_v</th>
</tr>
</thead>
<tbody>
<tr>
<td>358</td>
<td>B1</td>
<td>0.883</td>
<td>0.66</td>
<td>80.4</td>
<td>871</td>
<td></td>
<td></td>
</tr>
<tr>
<td>359</td>
<td>B1</td>
<td>1.050</td>
<td>0.70</td>
<td>40.5</td>
<td>721</td>
<td></td>
<td></td>
</tr>
<tr>
<td>365</td>
<td>B2</td>
<td>0.792</td>
<td>0.71</td>
<td>109.8</td>
<td>104</td>
<td></td>
<td></td>
</tr>
<tr>
<td>366</td>
<td>B1</td>
<td>0.816</td>
<td>0.64</td>
<td>88.2</td>
<td>946</td>
<td></td>
<td></td>
</tr>
<tr>
<td>367</td>
<td>B3</td>
<td>0.792</td>
<td>0.67</td>
<td>101.1</td>
<td>1007</td>
<td></td>
<td></td>
</tr>
<tr>
<td>373</td>
<td>B1</td>
<td>0.760</td>
<td>0.71</td>
<td>103.2</td>
<td>1108</td>
<td></td>
<td></td>
</tr>
<tr>
<td>380</td>
<td>B1</td>
<td>0.919</td>
<td>0.69</td>
<td>48.0</td>
<td>828</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

σ_m = mean effective consolidation stress

OCR = void ratio after consolidation.

Lunne (1982), Schmued and Honshby (1990), Salgado (1993) on q_c is confirmed by the negligible influence, which the boundary conditions imposed on the CC specimen has on q_c, see Fig. 4. (Because such tests have been performed in dry QS, have not been mentioned in Tables 4 and 5).

The dependence of q_c on e, σ_m, and OCR can be fitted by the following formula:

\[ q_c = 59.8 \left( e^{-1.33} \right) \frac{\sigma_m}{p_s} (OCR)^{0.19} \]  

(3)

Although beyond the scope of the present work, it might be worth mentioning that, during the CC tests performed in saturated QS, with the standard CPT tip, very low values of the local sleeve friction f_s ranging between 3 and 10 kPa, have been measured.

3.3 Plate loading tests

Table 6 summarizes the characteristics of the CC specimens in which plate loading tests have been performed. Fig. 5 shows two PLT’s results performed in very dense QS, for comparison, the results of a plate loading test performed under similar consolidation stresses, on medium dense siliceous Ticino sand, are also reported.

Table 6 shows also the values of the ultimate bearing pressure q_u, obtained from the PLT’s. According to Poulos and Chau (1985), Gilingly and Naurey (1990), Randolph et al. (1993), the bearing capacity of the plate has been taken as the average pressure causing its relative displacement (u/D) equal to 10%, being u plate settlement. Fig. 6 reports the values of q_u as function of the e, confirming a plot of similar data shown by Randolph et al. (1993). The values of q_u reported in the above figure indicate that the bearing pressure in highly compressible contractive carbonate sand is quite marginally influenced by σ_m and OCR.

In Table 6, values of G_u, E_u, and q_u corresponding to the state of the specimen (e, σ_m, OCR) at each PLT has been performed are also reported. Such values have been computed by means of eqs. (1) through (3) respectively.

![Diagram](image_url)

**Fig. 4** Influence of boundary conditions on penetration resistance of dry carbonate Quia sand (tests performed in large CC).

3.4 DMT’s

The results of a limited number of DMT’s performed on NC saturated QS consolidated in K_s conditions are summarized in Table 7. The table reports also the values of G_s and E_s computed by means of formulae (1) and (2) making reference to the state (e, σ_m) of the CC specimens at which the DMT’s have been performed.
4. CORRELATIONS WITH STIFFNESS

Geotechnical engineers have long [De Beer (1948)] been attempting to correlate the stiffness of granular soils against the results of penetration resistance. With time and with a better understanding of the complexity of the soil stress-strain behavior, it became obvious that despite all attempts to rationalize the correlations, they remain of an essentially empirical nature. This is mostly originated by the following circumstances:

i) Any penetration process is basically a bearing capacity test and, as such, though depending also on compressibility-deformability properties, is mostly controlled by the shear strength of soil.

ii) Because of the non-linearity of soil stiffness and of its dependence on the current effective stress state, the correlation between moduli and penetration resistance, even in the same soil, cannot be unique.

With the above in mind, it is necessary to recognize that, in general, a ratio of stiffness to penetration resistance, in a given granular soil, is a complex function of the following factors (listed in a decreasing impor-

\[
\text{Table 6 - Plate loading tests performed in large calibration chamber.}
\]

<table>
<thead>
<tr>
<th>test</th>
<th>n</th>
<th>BC</th>
<th>e</th>
<th>(\sigma_m)</th>
<th>OCR</th>
<th>(Q_0)</th>
<th>(F_0)</th>
<th>(q_u)</th>
<th>(q_c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>401</td>
<td>B1</td>
<td>0.922</td>
<td>0.72</td>
<td>1</td>
<td>863</td>
<td>2347</td>
<td>12.75</td>
<td>60.7</td>
<td></td>
</tr>
<tr>
<td>402</td>
<td>B1</td>
<td>0.876</td>
<td>0.63</td>
<td>1</td>
<td>856</td>
<td>2326</td>
<td>9.70</td>
<td>70.9</td>
<td></td>
</tr>
<tr>
<td>403</td>
<td>B3</td>
<td>0.833</td>
<td>0.60</td>
<td>1</td>
<td>819</td>
<td>2240</td>
<td>9.18</td>
<td>67.9</td>
<td></td>
</tr>
<tr>
<td>404</td>
<td>B3</td>
<td>1.033</td>
<td>0.65</td>
<td>1</td>
<td>729</td>
<td>2036</td>
<td>5.49</td>
<td>46.8</td>
<td></td>
</tr>
<tr>
<td>405</td>
<td>B3</td>
<td>1.033</td>
<td>0.76</td>
<td>1</td>
<td>770</td>
<td>2137</td>
<td>6.51</td>
<td>44.6</td>
<td></td>
</tr>
<tr>
<td>406</td>
<td>B3</td>
<td>0.950</td>
<td>0.74</td>
<td>2.91</td>
<td>1180</td>
<td>2859</td>
<td>13.01</td>
<td>80.6</td>
<td></td>
</tr>
<tr>
<td>407</td>
<td>B3</td>
<td>0.953</td>
<td>1.82</td>
<td>1</td>
<td>1466</td>
<td>3699</td>
<td>8.78</td>
<td>75.2</td>
<td></td>
</tr>
<tr>
<td>408</td>
<td>B3</td>
<td>0.950</td>
<td>0.77</td>
<td>2.83</td>
<td>1205</td>
<td>2912</td>
<td>9.94</td>
<td>73.9</td>
<td></td>
</tr>
<tr>
<td>410</td>
<td>B3</td>
<td>0.944</td>
<td>0.71</td>
<td>2.03</td>
<td>1045</td>
<td>2572</td>
<td>9.18</td>
<td>63.9</td>
<td></td>
</tr>
<tr>
<td>411</td>
<td>B3</td>
<td>0.944</td>
<td>1.81</td>
<td>1</td>
<td>1447</td>
<td>3223</td>
<td>7.35</td>
<td>77.4</td>
<td></td>
</tr>
<tr>
<td>412</td>
<td>B3</td>
<td>0.829</td>
<td>0.70</td>
<td>2.98</td>
<td>1281</td>
<td>3243</td>
<td>14.50</td>
<td>113.5</td>
<td></td>
</tr>
<tr>
<td>413</td>
<td>B3</td>
<td>0.769</td>
<td>1.82</td>
<td>1</td>
<td>1853</td>
<td>4495</td>
<td>19.45</td>
<td>136.2</td>
<td></td>
</tr>
<tr>
<td>414</td>
<td>B3</td>
<td>0.776</td>
<td>0.71</td>
<td>1.95</td>
<td>1324</td>
<td>3214</td>
<td>15.20</td>
<td>128.8</td>
<td></td>
</tr>
<tr>
<td>415</td>
<td>B3</td>
<td>0.783</td>
<td>0.64</td>
<td>1</td>
<td>990</td>
<td>2625</td>
<td>20.04</td>
<td>102.2</td>
<td></td>
</tr>
<tr>
<td>416</td>
<td>B3</td>
<td>0.789</td>
<td>0.62</td>
<td>1</td>
<td>963</td>
<td>2564</td>
<td>19.42</td>
<td>99.0</td>
<td></td>
</tr>
<tr>
<td>417</td>
<td>B3</td>
<td>0.863</td>
<td>0.63</td>
<td>1</td>
<td>873</td>
<td>2364</td>
<td>14.60</td>
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<tr>
<td>418</td>
<td>B3</td>
<td>0.777</td>
<td>0.36</td>
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<tr>
<td>419</td>
<td>B3</td>
<td>0.773</td>
<td>0.38</td>
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<td>2289</td>
<td>17.00</td>
<td>91.8</td>
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<tr>
<td>420</td>
<td>B3</td>
<td>0.737</td>
<td>1.19</td>
<td>1</td>
<td>1579</td>
<td>5909</td>
<td>30.80</td>
<td>152.3</td>
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\[
\text{Table 7 - Summary of ISMT's performed in large calibration chamber.}
\]

<table>
<thead>
<tr>
<th>test</th>
<th>n</th>
<th>e</th>
<th>(\sigma_m)</th>
<th>OCR</th>
<th>(P_0)</th>
<th>(\nu_1)</th>
<th>(k_D)</th>
<th>(E_D)</th>
<th>(M_D)</th>
<th>(M_0)</th>
<th>(Q_0)</th>
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<td>378</td>
<td>0.734</td>
<td>1.08</td>
<td>0.50</td>
<td>1</td>
<td>3.48</td>
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<td>1.38</td>
<td>11.13</td>
<td>2.00</td>
<td>1.40</td>
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<td>268</td>
<td>113</td>
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<tr>
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<td>3.01</td>
<td>1.50</td>
<td>1</td>
<td>3.76</td>
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<td>4.59</td>
<td>1.25</td>
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<td>201</td>
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<tr>
<td>388</td>
<td>0.773</td>
<td>1.04</td>
<td>0.54</td>
<td>1</td>
<td>1.35</td>
<td>19.95</td>
<td>3.80</td>
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<td>628</td>
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<td>389</td>
<td>0.731</td>
<td>1.90</td>
<td>0.99</td>
<td>1</td>
<td>5.89</td>
<td>24.25</td>
<td>3.12</td>
<td>3.10</td>
<td>637</td>
<td>944</td>
<td>308</td>
</tr>
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</table>

\(P_0\) = effective lift-off pressure  
\(\sigma_m\) = dilatometer modulus  
\(M_D\) = constrained modulus = \(f(\varepsilon_n)\)  
\(k_D\) = lateral stress index  
\(\nu_1\) = density index
A remarkable exception with this respect is represented by the initial pseudo-elastic stiffness ($G_e$, $E'_e$) at $e_1$. But, like any penetration resistance, are two different functions of the same variables ($e$, $\phi$, $\sigma_0$, structure) to which, in case of carbonate sands, one should also add OCR. This has corroborated, in the last two decades, [Jamiołkowski and Robertson (1988), Rix and Stokoe (1991), Lucre et al. (1997)] the opinion that the correlations between $G_e$, or $E'$, and $q_s$, or $N_{s,oc}$, are more reliable than those linking the penetration resistance to moduli at $e > e_1^*$.

As far as this kind of correlation is concerned, Fig. 7 shows the $G_s/q_s$ ratio in function of $e$, obtained on the basis of the data reported in [4] and 5. Analogously to what has been observed for siliceous sands [Jamiołkowski and Robertson (1988), Rix and Stokoe (1991)] the $G_s/q_s$ ratio increases with increasing $e$, i.e. with decreasing $D_e$. This ratio assumes values of 9.2±1.2 and 15.6±0.5 for very dense and loose QS respectively. As it results from the data in Fig. 7 the $G_s/q_s$ ratio does not seem to be influenced by OCR.

The collected experimental data allows also to evaluate the correlation between constrained tangent modulus $M_s$ measured during the consolidation stage of CC tests and $q_s$. On average, the $M_s/q_s$ ratio in QS results equal to 1.0 and 13.5 for NC and OC specimens respectively. The coefficient of variation referred to the above ratios results around 0.2.

The borehole PLT offers the opportunity to compare directly the values of shear modulus $G$ from these tests against $q_s$. For such computation, the $q_s$ has been computed by means of eq. (3) for conditions ($\epsilon$, $\phi$, OCR) at which any single PLT has been carried out in the CC. The tangent shear modulus from plate loading tests has been computed by means of equation from the theory of elasticity:

$$G = \frac{\pi D_s}{8} \left(1 - \nu^2\right) f(\frac{Z}{D_s})$$

where:
- $A_t$ = an increment of pressure applied on the plate
- $h$ = plate thickness = 104 mm
- $D$ = plate diameter
- $\nu$ = Poisson's coefficient assumed equal to 0.30

The correction factor taking into account the relative embedment of the plate, taken equal to 0.65 according to Donald et al. (1980).

In Figures 8 and 9 the values of $G$ inferred from CC PLT's normalized with respect to $G_e$ and plotted as function of the acting fraction of $q_s$, i.e. $q_s/\sigma_0$, and of the relative settlement ($s/D$), are reported. Such figures evidenced the highly pronounced non-linearity of the QS stiffness as inferred from the PLT's, similar to that resulting from the laboratory tests, see Fig.3. Figure 10 shows the values of $G_s/q_s$ ratio computed for four different values of $s/D$ considered representative for the relative settlement of properly designed foundations in carbonate sands.

Therefore, the values of $G$ which can be deduced from this figure should be considered as average operational stiffness to be used in the preliminary evaluation of foundations settlement in uncremented sands having similar conditions.
characteristics to the QS used in this research. The data shown in Fig. 10 confirm the already mentioned non-linearity of QS stiffness as well as the trend of G/sq to increase with increasing void ratio, even if such trend is remarkably minimized in comparison with that observed for G/sq ratio. The values of G from the CC PLT’s have been obtained from tests in which each load increment was maintained for 15 to 30 minutes. Therefore, in view of the previously mentioned creep susceptibility of QS, these moduli do not incorporate the long term secondary deformations far from being negligible in carbonated sands.

The occurrence of creep in QS is evidenced by the data exposed in Table 8 giving the values of:
- the coefficient of secondary consolidation C_s measured during one-dimensional compression in CC tests;
- the coefficient b quantifying the rate of creep settlement of the plate:

\[ s/D = (s/D)_0 + b \log (t/t_0) \]  

as observed during the same CC tests, being (s/D)_0 - relative settlement of the plate at t_0 < t.
Evaluating $C_q$ and $b$, a period equal to 30 minutes has been conventionally taken as the end of primary compression.

As far as the DMT is concerned, the number of the tests available does not permit to work out any specific correlation. From the data exposed in Table 7 the following can be observed:

- The ratio of $G_0/E_0$, being $E_0$ = dilatometer modulus, varies between 2.2 and 2.5 for low and high void ratio respectively.
- The measured values of $M_0$ are smaller than those inferred from DMT’s. This might indicate that the calculation procedure $M_0 = f(E_{100}, K_0, l_1)$ worked out by Marchetti (1980) for siliceous sands is not applicable directly to crushable carbonate materials.

Table 8 - Coefficient of secondary compression $C_q$ and creep parameters from CC-PLT n° 604*

<table>
<thead>
<tr>
<th>Consolidation stage</th>
<th>Plate loading tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_v$</td>
<td>$C_{eq}$</td>
</tr>
<tr>
<td>bar</td>
<td>bar</td>
</tr>
<tr>
<td>0.39</td>
<td>$2.5 \times 10^4$</td>
</tr>
<tr>
<td>0.57</td>
<td>$8.8 \times 10^4$</td>
</tr>
<tr>
<td>0.77</td>
<td>$1.0 \times 10^5$</td>
</tr>
<tr>
<td>1.00</td>
<td>$1.2 \times 10^5$</td>
</tr>
<tr>
<td>(*) $\varepsilon = 1.005$</td>
<td>4.35</td>
</tr>
<tr>
<td>$\sigma_v$</td>
<td>0.99 bar</td>
</tr>
<tr>
<td>$\sigma_h$</td>
<td>0.48 bar</td>
</tr>
<tr>
<td>(**) $C_{eq}$ referred to vertical strain</td>
<td></td>
</tr>
</tbody>
</table>

5. FINAL REMARKS

A series of "in situ tests" have been performed on pluvially deposited saturated crushable carbonate Quio sand in calibration chambers. Their analysis, summarized in the previous sections of this paper, allows the following conclusions:

1. The test sand exhibits a creep susceptibility comparable to that of the low plasticity clays.
2. The ultimate bearing pressure $q_u$ from PLT’s taken as $q$ which causes s/D = 0.1 results mostly controlled by the current density of the specimens. The $q_u$ is only marginally influenced by the applied confining stress and imposed OCR.
3. The initial pseudo-elastic stiffness $G_q$ or $E_q$ measured at strain less than linear threshold strain $\varepsilon_0$ (1 to 2) $10^3$ can be correlated to $q_u$ quite reliably.

4. The correlation between stiffness at $e > e_0$ and $q_u$ are of more empirical nature, hence both $G$ and $E_q$ represent a complex elasto-plastic moduli depending on many more factors than $G_0$ or $E_0$. Figure 10 attempts to present a correlation of $G_k$ vs $s/D$ for relative displacement of practical interest making reference to $G$ from CC PLT’s.

5. As for siliceous sands any ratio of stiffness to $q_u$ tends to decrease as the void ratio decreases (Baldis et al. (1991)). This reflects the very different influence that sand density has on deformation moduli ($G$, $E$) and strength ($q_u$). Such trends seem to attenuate with increasing strain and the relative deformation levels at which the reference stiffness is considered.

For sake of clarity, the readers of this paper should keep in mind that all the experimental results exposed have been obtained on freshly deposited sand. The behavior of such material can differ from that of the same sand in situ, that has been subject to aging, light cementation or other post-sedimentation allochthonous processes, especially important in carbonate deposits. With this respect, an optimistic hint is represented by the results of DMT’s performed by Marchetti (1995) in the natural deposit of medium dense very weakly cemented QS at Plouine site, as summarized in Fig. 11. Such figure shows that the results of DMTs performed in CC are comparable to those obtained in situ.

Fig. 11 - Dilatometer tests in carbonate Quio sand, field versus calibration chamber tests results.
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CPT data for the bearing capacity of piles with variable section calculation

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Scientific-Research Institute on Civil Engineering, Ufa, Bash, Nilstroi, Russia

ABSTRACT: Results of elaborating an engineering method of tapered piles calculation in clayey soils from CPT data are given. CPT data of in-situ tapered piles equipped with strain-gauges are reviewed as well as data of CPT in place of test pile manufacturing. By means of mathematical processing of experimental data the empirical dependences for determination of transfer coefficient from a probe to a pile as a function of relative depth of deposited layer are obtained. Calculation scheme is worked out and formulas for tapered pile bearing capacity evaluation are obtained, in which design soil parameters are defined from CPT data.

1 INTRODUCTION

Soil test with CPT has become one of the main methods of soil field in-place investigation in conditions of its natural bedding. The method is mostly wide used for pile foundations bearing capacity evaluation. For the usual pragmatic driven piles methods of bearing capacity calculation from CPT data are worked out and successfully used. But for piles with the variable lateral section along their length (tapered piles) being widely used in Russia and some other countries such method is lacking, as there are no corresponding theoretical and experimental investigations till now. The most effective are tapered piles concreted in situ in holes that are formed without soil excavation, i.e. with stamping-out. Such piles have some definite advantages compared with cast-in-place piles with constant cross-section by length. So, as the hole is formed with stamping-out (without soil excavation), the concreted in it pile "wants" in consolidated soil and hence has higher bearing capacity. The tapered form of the pile promotes greater soil consolidation around the pile that provides also the pile soil base resistance increase. At the same time, the stressed-strained state of cast-in-place tapered pile soil base, vertically loaded, as well as the regularities of achieving the tapered pile soil base limit states have their own peculiarities. That's why for design scheme working out and for solving the problem of such pile bearing capacity evaluation from CPT data one should obtain the experimental data on contact stresses along the lateral pile surface and under its toe and compare it with CPT parameters. The solving of all these problems is shown in given article.

2 CPT PECULIARITIES AND ITS APPLICATION FOR THE TAPERED PILES

Nowadays there are several widely used probe designs and methods of CPT in world practice. In Russia a probe of type II with cross-sectional area of 10 cm² equipped with the "friction sleeve" and with lateral surface area of 350 cm² found its application. Such a probe allows to measure separately soil resistance under the probe tip q₁ and soil resistance along the friction sleeve lateral surface f₁ at any point of depth. These parameters measurement is carried out with the probe penetration velocity of 1-2 m/min. It is well known that the parameters of soil resistance q₁ and f₁ measured in the process of a probe penetration into the soil are essentially higher by value than that of soil resistance for vertically loaded piles measured in the limit state of a system "pile-foundation". One of the main reasons of this is the sufficient difference between the physical phenomena in soil in the process of a probe penetration and pile behaviour in soil under the action of the vertical pressing-in load. That's why in order to increase the accuracy of pile design the procedure of CPT is worked out.
that helps to take probe recordings in its equilibrium when the probe variables approach 6. This procedure was called CPT "with stabilization". Figure 1 shows the typical diagrams of soil resistance $q_s$ obtained by standard method and with the CPT "with stabilization". By standard method the diagram has the form of a continuous broken line (fig.1a), with CPT "with stabilization" at the moment of a probe stop and its transition into the equilibrium state the meaning of the parameter decreases up to some value and then stabilizes. This is clearly shown on the diagram (fig.1b). The diagrams consist of separate sections, each of which includes: a) a section of a curve CB that characterizes the change of value $q_s$ in the process of uniform probe loading; b) a section of a curve that characterizes the stabilization during time of the resistance $q_s$ at the point of a probe stop (section BEDF) when the pressing-in forces acting at the probe and soil reaction become equal. The section CF is taken as the finite meaning of $q_s$ and from this data the finite diagram $q_s$ can be plotted (see fig.1c).

In order to calculate the bearing capacity of the usual driven prismatic piles methods of calculation from CPT data are worked out and successfully used including calculation methods "with stabilization".

The specific feature of tapered piles behaviour in soil is that along the pile lateral surface together with soil friction resistance there is a normal soil resistance component. That's why the use of soil resistance value under the probe tip $q_s$ is important not only at the depth of a pile tip location (as for prismatic piles) but at the whole depth within the pile length in soil as well. Taking into account that CPT gives us such an opportunity one can plot a design scheme in which the resistance of a pile toe with inclined edges is based on three components, i.e. soil resistance under the pile toe, soil friction resistance along the pile lateral surface and normal soil resistance along the pile lateral surface.

3 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 site No.2 at the depth of 1.5-2 m. In place of test piles manufacturing CPT was carried out with the help of the unit S-832, equipped with the standard tapered probe of 10 cm² cross-sectional area with

![Figure 1. Diagrams of soil resistance at CPT: a - by standard method, b and c - by CPT "with stabilization".](image)

The Russian codes permit their application equally with the standard method. However, as the piles with the variable section along their length found a wide application in Russia and some other countries it is advisable to work out the method of such piles bearing capacity calculation using CPT data.

friction sleeve for measuring the soil friction forces along the pile lateral surface. CPT was carried out by standard method and by the method "with stabilization" worked out in BashNILstroi.
<table>
<thead>
<tr>
<th>Site No.</th>
<th>Sampling depth, m</th>
<th>Natural humidity, W</th>
<th>Plasticity index, Pi</th>
<th>Consistency index, I</th>
<th>Volume mass, t/m³ of soil with natural moisture, γ</th>
<th>Porosity coefficient, η</th>
<th>Angle of internal friction, φ, degree</th>
<th>Cohesion, c, MPa</th>
<th>Moisture degree, G</th>
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<tbody>
<tr>
<td>1</td>
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<td>0.24</td>
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<td>1.52</td>
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<td>18</td>
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<tr>
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<tr>
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<tr>
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<td>0.8</td>
<td>14</td>
<td>0.05</td>
<td>1.03</td>
</tr>
</tbody>
</table>

At site No.1 a hole was stamped-out of 60 x60/20 x20 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 sm section and 4.2 m length with strain-gauges along its lateral surface was manufactured.

In order to transfer all the load through the pile lateral surface a metal plate was placed 3 sm above the hole bottom up to the place of concreting, i.e. the pile toe had no contact with the soil.

Figure 2. Dependences Diagrams.

a - at site No.1; b - at site No.2; 1 - "load-settlement" of a driven pile; 2 - idem of cast-in-place pile; 3 - idem at pile extracting; 4 - lateral surface resistance because of friction due to pile settlement; 5 - idem because of normal pressure due to pile settlement; 6 - bottom end resistance due to pile settlement; 7 and 8 - resistance of driven pile lateral surface owing to friction and standard soil resistance, respectively, due to pile loading; 9 and 10 - idem of cast-in-place pile; 11 - resistance of the bottom end due to pile load.
At both sites piles were penetration and extraction load tested and recordings were made at all stages of loading. At place 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 the vertical penetration load tested, at this moment the pile toe began to work. In the process of the hole stamping-out at both sites a soil uplift zone was measured by levelling.

4 ANALYSIS OF TESTS RESULTS

Figure 2 shows the results of static test piles with vertical load, figure 3 shows the diagrams of soil friction resistance along the pile lateral surface and the soil resistance under the pile point at both test sites in condition of limit load action.

Diagrams of standard soil resistance along the pile lateral surface are also obtained. However, they are not shown in the given paper because of its limited volume. Soil CPT data are also shown here for comparison. In figure 2 curves 1, 2, and 3 are obtained from static tests data, curves 4-11 - from results of soil contact stresses measurement along the pile lateral surface and under its toe.

When pile testing at site No.1 at the initial stage of loading friction along the lateral surface of the upper pile part is significantly greater than that in its bottom part (figure 3 a). With the load increase the friction naturally increases in the bottom pile part and becomes greater in the direction to its tip. At the pile load of 250 kN the friction of upper soil layers reaches its limit value and is constant up to the limit load. The soil under the pile toe begins to work at the load about 200 kN and its resistance reaches the limit value (75 kN) at the pile load of 600 kN and the settlement of 15 mm. 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 the pile load increases, friction and normal soil pressure increase simultaneously. It must be noted that the soil resistance along the tapered pile lateral surface due to friction is continuously increased with the settlement increase and reaches its maximum at the load limit. It is taken into account that the tapered piles are in conditions of constantly increasing normal soil pressure with the load increase along the pile lateral surface. This causes friction forces increase even at great pile settlements when its shear relative to soil occurs.

Figure 3 a,b shows that values of friction along the pile lateral surface at the limit load are less according to strain-gauges recordings than that measured with CPT, this discrepancy decreases with depth and becomes insignificant near the pile tip. The decrease of friction and soil pressure along the pile lateral surface 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 mm respectively and then decreased up to 0 at the distances 1.8 and 2 m from the stomp. The volumes of the uplifting soils at these sites were 0.36 and 0.32 m³ respectively, that was 24% of the driven stomp 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

![Fig.3. Diagrams of specific friction $\tau$ along the pile lateral surface at the limit load and CPT data: a - at site No.1; b - at site No.2; 1 - along the lateral cast-in-place pile surface; 2 - CPT with stabilization; 3 - CPT by standard method; 4 - along the lateral driven pile surface.](image-url)
loosening when the stamp (pile) driving lead to resistance decrease along the pile lateral surface. That's why the use of CPT data for a tapered pile calculation is possible only whith the correction coefficients $\beta$ intoing. Figure 4 a,b shows these coefficients dependences on the relative depth of the soil layers bedding $z/z_0$ for both sites as the relation of specific friction $\tau$ and normal soil pressure $K$ along the pile lateral surface at the limit load to the corresponding resistances $f$ and $q$ according to CPT data obtained with the unit S-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 $\beta$ were determined from measurements results for a driven pile. The coefficients $\beta$ obtained from the measurements results when the cast-in-place pile testing at site No.2 can be approximately applied to the tapered piles with the sharp end, as the additive pressure of the lateral pile surface bottom part looks at the settlement of such piles.

5 TAPERED PILE CALCULATION FROM CPT DATA

Approximating data of figure 3 with the least square method, we obtain the empirical dependences for the transfer factors $\beta_1$ and $\beta_2$

\[
\beta_1 = K_1 f L + K_2; \quad \beta_2 = K_1 f + K_3,
\]

Table 2. Coefficients

<table>
<thead>
<tr>
<th>Type of soil CPT</th>
<th>$K_1$</th>
<th>$K_2$</th>
<th>$K_3$</th>
<th>$K_4$</th>
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<tr>
<td>With the blunt end stabilization</td>
<td>0.73</td>
<td>0.26</td>
<td>0.3</td>
<td>0.18</td>
</tr>
<tr>
<td>By standard</td>
<td>0.44</td>
<td>0.12</td>
<td>0.38</td>
<td>0.1</td>
</tr>
<tr>
<td>With the sharp end stabilization</td>
<td>0.7</td>
<td>0.25</td>
<td>0.72</td>
<td>0.18</td>
</tr>
<tr>
<td>By standard</td>
<td>0.35</td>
<td>0.13</td>
<td>0.36</td>
<td>0.12</td>
</tr>
</tbody>
</table>

where $\beta_1$ and $\beta_2$ - dimensionless transfer factors from a probe to a pile for the specific and the soil resistance under the probe tip, respectively. $K_1$, $K_2$, $K_3$, and $K_4$ are the empirical coefficients, the values of which for the given soils are shown in table 2.

The experimental data shown allow to present the design scheme of a tapered pile as a vertically loaded stiff rod deepened into the soil with the reactive soil resistance along the lateral surface as friction $f$ and normal pressure $K$, and with the soil resistance under the toe $K_1$. Let's divide the soil in the limits of a pile length into "n"layers and take the soil friction resistance and penetration resistance in the limits of separate soil layers constant and the law of the pile section change $d_2$ take by length as:

\[
d_2 = d_0 \left(1 - \frac{\Delta}{d_0} \right),
\]

where:

\[\Delta = d_0 - d_1\]
Having projected all the forces onto the vertical axis, we obtain the tapered pile vertical load resistance.

\[ F = U_t A_t + \frac{1}{2} \left( \frac{U_s}{U_t} A_s \cdot 2B_t \cdot \frac{B_s}{B_t} + A_sR_s \right) \]  

where \( U_t \) - pile section perimeter at the soil surface level;
\( R_s \) - soil resistance under the pile toe;
\( A_t \) - the area of the pile cross-section.

\[ A_t = \sum_{i=1}^{N} f_i (Z_i - Z_s) \]

\[ B_t = \sum_{i=1}^{N} f_i (Z_i^2 - Z_s^2) \]

\[ A_s = \sum_{i=1}^{N} R_i (Z_i - Z_s) \]

\[ B_s = \sum_{i=1}^{N} R_i (Z_i^2 - Z_s^2) \]

Using CPT data, soil design characteristics are defined as:

\[ \bar{R} = \bar{R}_s \beta_1 \]

\[ \bar{R} = \bar{R}_s \beta_2 \]

where \( \bar{R}_s \) - soil resistance along the probe lateral side; \( \bar{R}_s \) - soil resistance under the probe tip; \( \beta_1 \) and \( \beta_2 \) are found by formula (1) for each layer.

### Table 3

<table>
<thead>
<tr>
<th>Section, sm</th>
<th>Pile bearing capacity, kN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>By static test</td>
</tr>
<tr>
<td></td>
<td>with stabilization</td>
</tr>
<tr>
<td>60x60 / 20x20</td>
<td>6</td>
</tr>
<tr>
<td>60x60 / 20x20</td>
<td>5.5</td>
</tr>
<tr>
<td>60x60 / 20x20</td>
<td>5.5</td>
</tr>
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<td>60x60 / 20x20</td>
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</tr>
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<td>60x60 / 20x20</td>
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</tr>
<tr>
<td>80x80 / 30x30</td>
<td>3.6</td>
</tr>
<tr>
<td>60x60 / 20x20</td>
<td>6</td>
</tr>
<tr>
<td>80x80 / 30x30</td>
<td>4.5</td>
</tr>
</tbody>
</table>

2. The empirical dependences are obtained for determination the coefficients of transfer from CPT data to soil resistance along the lateral tapered pile surface 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 CPT data.

### 6 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 the cohesive soils. The calculations were carried out by formula (3) using the coefficients \( \beta \). The calculation from CPT data without stabilization using coefficients \( \bar{R} \) by formula (1) gives the most reliable data (error does not exceed 17%), while the errors obtained "by standard" are more than 19%.

### 7 CONCLUSIONS

1. The experimental investigations of in-situ piles in clayey soils allow 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.
CPT for the bases deformability evaluation

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ABSTRACT: Methods of evaluation of the foundation deformability with the CPT unit S-832 M that is designed in the scientific-research Institute BashNIIstroi are given in the article. The "equilibrium probe" is used that allows to fulfill CPT with the uniform probe penetration as well as at the equilibrium probe state, actually at its stop, when the resistance under the probe tip is balanced with the excess pressure in the hydrosystem.

Based on the consideration of the system "a probe in the soil", "a pressure of compressed air in a hydraulic system", equilibrium, decisions for the factors of bases stiffness evaluation are obtained. The method suggested allows to avoid the sufficient errors that occur while evaluation of the bases stiffness factor as a function of deformations modulus and a coefficient of lateral tension calculated from CPT data. The method is recommended for heavily compressible soils when monoliths sampling and laboratory investigations are difficult to carry out.

I INTRODUCTION

When considering a problem of foundation calculation on a compressible base the complications appear mostly while bases deformability evaluation. The most researchers suppose the bases for such foundations can be modelled using "the model of the variable coefficient of subgrade reaction". The coefficient of subgrade reaction of the foundation on the natural base is estimated as a function of modulus of deformation. The pile foundation coefficient of subgrade reaction is characterized with the coefficient of pile stiffness. The traditional method of soil modulus of deformation evaluation from CPT data supposes correlation with the specific resistivity under the probe. The correlation relationship \( E = f(q) \) is determined not only from CPT data but also from the results of modulus of deformation in situ and laboratory determination. Thus, the correlation relationship is determined with the double error, this decreases to a great extent the accuracy of modulus of deformation calculation from CPT data and correspondingly influences the accuracy of foundation stiffness factor evaluation.

Pile stiffness factor that characterizes pile foundation compressibility is usually determined from pile static tests results. Pile stiffness factor from CPT data is calculated as pile load to settlement ratio. In addition, pile settlement is determined using soil characteristics calculated from CPT data. Thus, the above peculiarity of modulus of deformation determination is used as well for pile stiffness factor determination.

In the scientific-research Institute BashNIIstroi a unit S-832M for CPT is developed that besides the usual CPT with the constant velocity \( V = 2m/min \) provides the CPT with the probe stop. Such mode of CPT is called CPT with the "equilibrium probe" or CPT"with the probe stabilization".

The practice of CPT "with the probe stabilization" in different soil conditions showed that some CPT parameters such as time of a probe stabilization and a probe settlement during stabilization depend, to a great extent, upon soil deformability. These parameters increase with the soil compressibility increase. These practical investigations results made it possible to suppose the relation between soil deformability and parameters that characterize the process of a probe "stabilization".
In order to evaluate the type of this relation the physical regularities of a probe "stabilization" process are studied and the deformational characteristics of a base used in practical calculations as the base stiffness factor are obtained.

Thus, there appeared a possibility of a base stiffness factor evaluation not with the modulus of deformation, but directly in the process of CPT. As the deformational characteristics obtained differ from the traditional characteristics (E, M) used when foundations calculation, the ways of their use in design practice of both foundations on a natural bed and pile foundations are considered.

2 MATHEMATICAL MODELLING OF THE PROCESS OF A PROBE PENETRATION AND "STABILIZATION"

The regime of a probe "stabilization" obtained with the unit 3-832M is achieved due to the peculiarities of this unit's hydraulic system. Unlike the other well known CPT units the construction of the above unit includes "the damping device" that represents a cylinder filled with the air and connected with the main hydropneumatics. This hydraulic scheme of CPT is shown in figure 1. Taking into account the scheme in figure 1 let's watch the process of the penetration and "stabilization" of a probe in the soil. The penetration of a probe is due to oil pressure in hydropneumatics \( P_p \). With the transition into the stabilization regime the probe is stopped, the pressure in the system sharply decreases, however due to excess pressure of compressed air in the damper \( P_d \) the probe moves for some time till the pressure in the system is compensated with the soil resistance. This state can be considered as the probe "stabilization" and the pressure at which this state is achieved is \( P_c \).

When the probe penetration and at the moment of its stabilization the hydraulic system is continuously in equilibrium, the oil pressure in hydropneumatics is balanced with the pressure of compressed air in the damper. That's why the scheme in figure 1 can be simplified by means of changing the compressed air in the damper and the probe in the soil with the elastic springs with the stiffnesses \( K_a \) and \( K_s \) (figure 2). The equilibrium condition of the system "the probe in soil - air damper" can be written as follows:

\[
\Delta y = K_a - S_a = K_s \tag{1}
\]

where \( \Delta y \) - the change of oil uplift height in the damper during stabilization; \( S_a \) - the probe displacement from the moment of its stop up to its stabilization.

Fig.1. Hydraulic scheme of CPT "with stabilization": 1-hydropneumatics; 2-platons of hydropneumatics; 3-damper; 4-probe.

Fig.2. The conditional scheme of CPT "with stabilization": 1-the elastic spring substituting the air in the damper with the stiffness \( K_a \); 2-the elastic spring substituting the probe in soil with the stiffness \( K_s \).

The coefficient \( K_a \) is evaluated by analogy with the stiffness factor of any spring as the load at which the height of the air column in the damper increases (or decreases) by 1 sm, i.e. the air volume in the damper changes by the value
\[ \Delta V = S \times 1 \text{ sm} \] (where S is the section area of the damper). The air state in the damper at the different stages of the stabilization process can be described as follows:

\[ P_V = \text{const}; \]

\[ P_V = P_t + \Delta P_t \]

\[ P_t = \frac{P_r V_t}{V_t + \Delta V_t} = \frac{P_r h_t}{h_t + (\text{sm})}; \]

\[ \Delta P_t = P_t - P_0 = (1 - \frac{h_t}{h_t + (\text{sm})}); \]

\[ K_s = \frac{\Delta P \cdot F}{P_t}; \]

\[ S = \frac{V_t}{h_t}; \]

\[ P_0 = P_t \cdot V_t \]

\[ V_t = \frac{P_r F}{P_r}; \]

\[ \Delta y = \frac{h_t (P_t - P_0)}{P_r} \]

where \( P_r, V_t \) - air pressure and volume in the damper at the moment of a probe stop;

\( P_t, \Delta V_t \) - air pressure and volume in the damper at the change of the air column height by 1 sm;

\( h_t \) - height of the air column in the damper at the moment of a probe stop;

\( P_0 \) - air pressure and volume in the damper at the moment of stabilization;

\( F \) - the area of hydrocylinder piston cross-section.

Substituting (2) and (3) into the equation (1), we obtain:

\[ K_s = \frac{(P_r - P_0) F \cdot A}{P_r S \cdot 0.001} \]

Without changing the physical sense the decision (4) is reduced to a form that lets to obtain the stiffness factor with the traditional dimension MPa/m²:

\[ K_s = \frac{(P_r - P_0) F \cdot A}{P_r S \cdot 0.01} \]

where \( P_r, P_0 \) - pressure in the hydraulic system, MPa;

\( S, \Delta y \) - a probe settlement, sm;

\( F, S \) - a probe area, sm².

\[ A \cdot \text{the coefficient depending on the CPT regime,}
\]

\[ \text{is changed in the range of 0.3-1.} \]

At CPT with the unit S832M with the probe diameter \( \varnothing = 3.6 \text{ sm} \) and area \( F = 10 \text{ sm}^2 \) the expression (5) can be simplified:

\[ K_s = \frac{(P_r - P_0) F \cdot 7.7 \cdot A}{P_r - 0.001 \cdot S}; \]

As a result, the probe stiffness factor in soil \( (K_s) \) that characterizes the foundation deformability is obtained.

According to its physical sense this index corresponds to the foundation stiffness factor, that by Winkler hypothesis is evaluated with the ratio of the average pressure under the foundation bed \( (P) \) to the settlement \( (W) \). Proceeding from the decisions of the elasticity theory, the settlement of the stiff square stamp will be defined with the formula:

\[ W = 0.88 \left[ \frac{(1 - \mu^2)}{E_0} \right] \frac{P}{a} \]

where \( E, \mu \) - modulus of deformation and the soil Poisson's ratio;

\( P \) - the average pressure;

\( a \) - the stamp side.

Using the dependence (7), the stiffness factor is defined:

\[ K_s = 1.14 \left[ E_0 \left(1 - \mu^2 \right) \cdot a \right] \]

Thus, the value of the foundation stiffness factor depends on the foundation dimension and outline, while according to Winkler hypothesis this value should depend only on the kind of soil.

The method of the foundation stiffness factor evaluation from CPT data (4) has no such a contradiction. The value of the stiffness factor at CPT with one and the same unit depends only upon the kind of the soil.

3 EXPERIMENTAL INVESTIGATION RESULTS

The experimental control of the decision obtained was done at the experimental site composed of argillous clays with fine-grained sandstone partings. CPT was made with the unit S-832M at the depth up to 10 m with rate of 2 m/min and with the probe stop in each meter. In the process of CPT the following parameters were measured:
specific resistances under the probe tip;
specific resistances along the lateral probe side ($P_x$ and $P_y$);
pressure in the hydraulic system;
probe settlement in course of "stabilization" ($S_s$).

In the following table the results of pressures in the hydraulic system and the probe settlement measurements and $K_s$ calculation results are shown.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>No. of measurements</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_x$, Mpa</td>
<td>1.2 1.3 1.4 0.3 0.9 0.7 1.1 1.1</td>
</tr>
<tr>
<td>$P_y$, Mpa</td>
<td>0.5 0.5 0.3 0.3 0.4 0.4 0.4 0.4</td>
</tr>
<tr>
<td>$S_s$, mm</td>
<td>0.5 0.4 0.4 0.4 0.3 0.5 0.3 0.3</td>
</tr>
<tr>
<td>$K_s$, Mpa</td>
<td>840 130 157 843 1285 177 1684 1384</td>
</tr>
</tbody>
</table>

According to data in the table a diagram of the design value $K_s$ dependence on the ratios of the characteristics ($P_x$, $P_y$) and $S_s$ (fig.3) measured during CPT is plotted.

$$W = \frac{P^2(1-\mu^2)}{8R}$$

where $R$ is the distance from the point of force application to the point of the settlement evaluation.

Having changed $\frac{P}{W_0}$ to the coefficient $C$, called "the coefficient of the elastic half space", and having expressed the dependence (10) relative to $\frac{P}{W_0}$, obtain:

$$\frac{P}{W_0} = CR$$

Thus, taking into account that the both characteristics $K_s$ and coefficient $C$ characterize the deformability and making comparison between the experimentally obtained dependence (9) and the theoretical one (10), one can make a conclusion about their identity. Hence, the value obtained from CPT data can be considered as the base deformability and correspondingly used in decisions of the theory of elastic half space for different kinds of foundations.

4 THE EVALUATION OF THE BASE STIFFNESS FACTORS

As an experimental and practical experience show, the bases stiffness factors used for foundations calculated as constructions on the elastic base, depend not only on the bedding soil elasticity but on some other factors as well. The most important of them are: foundation type, foundation dimensions and form, stresses in foundation base. Taking all these factors into account and using the base deformability from the results of CPT a method of stiffness factor evaluation for the calculation of foundations on both natural bed and pile foundations is worked out.

4.1 Stiffness factor evaluation for the calculation of foundations on natural bed

For foundations on the natural bed the calculation of which is done just as for construction on the elastic base, it is recommended to evaluate the stiffness factor by formula (Titovitchen 1963):

$$K = K_s \left[1 + \frac{2\pi h b}{\Delta F} \right] \frac{F}{F_t}$$

1000
where $K_s = K_e$ - constant of a base elasticity not depending on the foundation dimensions;  
$\varphi$ - dimensions of a foundation bed;  
$F$ - specific pressure transferred onto the foundation base;  
$q$ - specific resistance under the probe tip;  
$\Delta$ - elasticity constant, in practical calculations can be taken equal to 1 m$^4$.  
Thus, the method of stiffness factor calculation involves the following steps:  
- soil CPT is performed with a probe penetration velocity $V=2$m/min without a probe stop, resistances under a probe bottom end and along its lateral side ($q, l$) are measured, the character of soil bedding is evaluated and according to the experimental dependence $E=\psi(q)$ the modulus of deformation is approximately evaluated;  
- based on results of step 1 the preliminary foundation overall dimensions are estimated, the depth of its embedding is fixed, the compressed air thickness $H$ is calculated, based on the analysis of soil bedding character the site is divided into sections;  
- in each section soil CPT is done not less than at 5 points up to a depth not less than $H$, the probe penetration velocity is $2$m/min with the probe stop (“with stabilization”) in each meter of penetration, in the process of CPT parameters $P_c, P_r, S_r, q$ are defined;  
- constant of the base elasticity ($K_e$) is defined for each point of CPT at different depths of the probe penetration by formula (13);  
- the mean value of the base elasticity constant $K_e$ within the height of the compressed zone $H$ is estimated for each point of CPT:

$$K_e = \frac{1}{H} \sum K_e$$  

(13)  
- the stiffness factor $K_s$ is defined for each point of CPT by formula (13);  
- for each section the mean value of $K_s$ is defined:

$$K_s = \frac{1}{H} \sum K_s$$  

(14)  
where $n$ - number of CPT points in a section.  

4.2 The stiffness factor evaluation for pile foundations calculation

Strain measurements done by the author for the "floating" piles with the section $30x30$ cm in clayey soils (N. Gotman 1995) show that the elastic deformation of a pile in soil is characterized with a curve part “settlement-load” from zero to a point corresponding to a moment of full realization of friction forces along the lateral pile surface and to the load transfer onto the probe toe (fig.4). This state is characterized as a limit state, and the ratio of a pile load in the limit state ($F_{l}$) to its settlement ($S_l$) is taken as the deformational characteristic of a pile in soil.  
If the process of CPT is considered as a uniform probe tip penetration into the soil, it can be expressed as a set of limit states when the resistance along the probe lateral side is fully realized, the load is transferred to the tip and the penetration is due to the external load exceeding soil resistance. The probe in the “equilibrium” state that is reached when transferring to CPT “with stabilization”, models the pile limit state.  
The condition of a pile simulation with a probe allows to take an assumption that the relation between the geometrical dimensions and settlements of a probe and a pile meet the condition:

$$\frac{S_i}{S} = \frac{U_i}{U}$$  

(15)  
where $S_i$, $S$ - settlements of a probe and a pile in limit equilibrium;  
$U_i$, $U$ - a probe and a pile perimeter.  
The relation between a load and a settlement of a pile and a probe is defined as follows:

$$F_{l} = K_s S; \quad F_{l} = K_s S$$  

(16)
where $F$, $F_p$, $S$, $S_0$ the load transmitted to a probe and a pile, settlement of a probe and a pile under this load; $K_1$ and $K$-stiffness factors of a probe and a pile. If $F_p$ is the limit pile resistance from CPT data "with a probe stabilization" with the use of parameters $q$ and $f$, then $F_s$ is the limit probe resistance equal to the load transmitted to the probe at the moment of "stabilization":

$$F_s = P_x F$$  \hspace{1cm} (17)

Substituting (16), (4) and (17) into the formula (15), obtain:

$$K = \frac{F_p S - F_s P_s A}{P_s S_0 + 0.01 B}$$  \hspace{1cm} (18)

where $F_p$ - the limit pile resistance from CPT "with the probe stabilization" results;

$B = \frac{U}{U'}$.

Thus, the method of probe stiffness factor calculation includes the following steps:

- soil CPT is done with a probe penetration velocity $V=2$ m/min without a probe stop, resistances under a probe bottom end and along its lateral side ($q$ and $f$) are measured, with the use of standard methods the character of soil bedding is evaluated, the pile bearing capacity is calculated, the depth of pile penetration is fixed.

- According to the depth of pile penetration chosen, the site is divided into sections;

- in each section soil CPT is done not less than at 5 points up to a depth not less than $1.4d$ (d-pile dimension), parameters $P_p$, $P_s$, $S$, $q$, $f$, are defined in each meter "with stabilization";

- pile stiffness factors are evaluated by formula (18) for each point of CPT with the chosen depth of pile penetration;

- the mean value $K$ is evaluated for each section:

$$K = \frac{\sum K_i}{n}$$  \hspace{1cm} (19)

where $n$ - number of CPT points in the section.

5 CONCLUSIONS

1. A decision for base deformability evaluation by means of mathematical simulation of a probe penetration process from the moment of its stop and up to the "equilibrium" state is suggested.
Geometry and scale effects in CPT and pile design

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ABSTRACT: A study of geometry, particle size and stress effects in the cone penetration test (CPT) is presented. The influence of the penetration distance to mobilise full resistance is considered in relation to some CPT penetrations and model anchor extractions in centrifuge tests. Some implications for the use of CPT results in the design of long and short piled foundations are then discussed.

1 INTRODUCTION

The cone penetration test (CPT) is often chosen to characterise cohesionless material. Values are generally taken to apply to soil conditions at a point, or a local zone, through some empirical correlation obtained from calibration tests. The nature of calibration chambers makes it difficult to discriminate between geometrical, size and stress-level effects. The application of CPTs in geotechnical and foundation design therefore relies on empirical factors which are not fully understood.

Commenting on the failures of the Herleik beam, Been and Crooks (1988) suggested that some of the correlations derived from calibration chamber tests could be erroneous. Erbrich (1994) who modeled the foundations of an offshore structure on a sand seabed, pointed out that it is impossible to reproduce the 20 m deep field tip resistance of 60 MPa in a calibration chamber. These seem to question the direct applicability of calibration chamber data.

The launch of the ‘Seacont’ system by Fugro U.K. Ltd which adopts a 10 mm cone, as compared to the conventional 38 mm cone, has further encouraged the study of geometry effects in the CPT.

2 INTERPRETATION OF RESULTS

A detailed description of CPTs in the centrifuge has been presented elsewhere (Gui, 1995) and will not be repeated here. Dimensional analysis has also been recommended (Bolton et al, 1993) and used to interpret the results. In general, the tip resistance \( q_t \) is normalized with respect to overburden pressure \( \sigma_v' \), and the penetration depth \( z \) is normalized with respect to cone diameter \( B \). Normalized tip resistance \( Q \) and normalized penetration depth \( Z \) are given as:

\[
Q = \frac{q_t - \sigma_v'}{\sigma_v'} \quad (1)
\]

\[
Z = \frac{z}{B} \quad (2)
\]

where \( \sigma_v' \) and \( \sigma_v'' \) are the total and effective stresses respectively.

3 GEOMETRY AND SCALE EFFECTS

3.1 Initial penetration \( \Delta z/B \) effect

It has been observed that a CPT will not be able to register the absolute tip resistance at the instant when it penetrates into a new soil layer. Fig 1 shows two tip resistance profiles, the observed and the ideal profiles. After some distance, the observed tip resistance profile starts to deviate from the ideal profile before entering the hard soil layer because the cone is capable of detecting the hard boundary at a few cone diameters away. Once it enters the hard soil,
"development" takes place prior to registering the full resistance of the hard soil. Thereafter, the observed profile falls drastically before it enters the soft soil lying beneath it.

An approximate analysis based on a modified Boussinesq's solution (Yuen-denhil et al 1994) also demonstrates the effect. This has a significant impact if the resolution of a thin soil layer is important to the design. It is therefore important, at least qualitatively, to study the effect of the penetration depth (z) required to develop the resistance of a new soil layer.

A set of unusual cone uplift tests (Gui, 1995) sheds further light on the displacement z as required for development of the full penetration resistance. Fig 3 clearly shows the slow mobilization of resistance as a buried cone develops resistance in the sand ahead of itself. Comparative CPT data fell just above the envelope created by the uplift tests. There is no layering in this case. It simply takes about 5 cone diameters of displacement to "develop" a uniformly graded sand ahead of an advancing probe, whichever direction it is travelling in. Well-grained materials have been observed to require smaller development distances, perhaps as small as 1 or 2 diameters of penetration. A displacement pile will develop the soil ahead of its tip by crushing particles as the tip approaches. A wider grading is produced which leads to voids reduction, permitting the pile to advance. The crushing strength of broken fragments always exceeds that of the original particles; this is proposed by Bolton and McDowell (1997) as the origin for "plastic hardening" during "normal consolidation" of soils.

A driven pile may therefore be capable of carrying almost the complete CPT resistance of the formation, but only if the greater "development" distance of pile is allowed for in the competent soil layer, and if no incompetent layer is in the zone of influence beneath the tip. A cast-in-place pile or anchor will not benefit from any prior development, and will suffer excessive displacements if it is asked to carry more than a small fraction (e.g. 10 to 20%) of the CPT resistance.

3.2 Grain size B/d50 effect

The effect of the ratio of cone diameter to mean grain size (B/d50) was studied for Leighton Buzzard sand by Lee (1990). For fine sand at a single relative density, normalized tip resistance Q is plotted against normalised depth Z.

---

**Fig 1:** Effects of development on CPT profiles.

**Fig 2:** Test set-up for upward-pointing cone tests.

**Fig 3:** Mobilization of upward pointing cones.
normalized depth \( Z \) for cones of different diameter in Fig 4(a). Tests have been carried out in different gravity fields (21g, 40g and 60g) because it is necessary to preserve a constant stress level \( \sigma'_r \) for the different cones, in relation to the constant aggregate crushing strength \( p_c \). Now

\[
\sigma'_r = \rho_{dy} \cdot g \cdot Z \cdot N
\]

which can also be written as

\[
\sigma'_r = \rho_{dy} \cdot g \cdot \frac{Z}{B} \cdot N \cdot B
\]

Hence, we must keep "\( N \cdot B \)" constant in order to preserve a constant \( \sigma'_r \) for each value of \( Z \). Each test therefore models a single prototype cone, of 0.4 m diameter in this case.

Fig 4(a) shows that the data from this modelling-of-models trial superimposed nicely until each cone approached the base of the test container. This proves that the soil particle size does not affect the result for the ratio \( B/d_{so} \) in the range of 85 to 28.

Fig 4(b) repeats the same plot for medium and coarse Leighton Buzzard sand. Treating each soil separately, the plots for the medium sand merge reasonably well for \( B/d_{so}=48 \) and 25, but there is a suggestion of a small amount of extra resistance at \( B/d_{so}=16 \). For coarse sand, all the data are somewhat higher and while there is insufficient evidence of distortion in reducing \( B/d_{so} \) from 21 to 11, it can be seen that a further reduction to 7 does raise resistance especially at shallow depths.

<table>
<thead>
<tr>
<th>No.</th>
<th>( B ) (cm)</th>
<th>( d_{so} ) (mm)</th>
<th>( B' )</th>
<th>( Q_{min} )</th>
<th>( Q_{min} ) (( B'/B ) ))</th>
<th>( Q_{min} ) (( B'/B ) ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>T40</td>
<td>10.0</td>
<td>0.9</td>
<td>10.9</td>
<td>395</td>
<td>320</td>
<td>1.19</td>
</tr>
<tr>
<td>T41</td>
<td>6.35</td>
<td>0.9</td>
<td>7.25</td>
<td>434</td>
<td>524</td>
<td>1.30</td>
</tr>
<tr>
<td>T49</td>
<td>6.35</td>
<td>0.4</td>
<td>7.25</td>
<td>304</td>
<td>278</td>
<td>1.13</td>
</tr>
</tbody>
</table>

Effective diameter \( B' \) reduces as the increased stress level induces crushing.

Fig 5: Particle characteristic: degree of freedom.

There may be more than one effect causing the differences in Fig 4. Particle angularity or roughness (which always vary with sand sizes) may be significant dimensional parameters in addition to relative density. However, particle size does also seem to be involved explicitly. Fig 5 demonstrates an empirical definition of an effective diameter of the cone \( B' \) increased by one particle diameter (\( B'=B+d_{so} \)). The extra resistance of initially large soil
grains can be perceived in terms of the increase in the ratio \( \frac{Q_{\text{pen}}}{Q_{\text{pen},\text{final}}} \), where \( Q_{\text{pen}} \) is the peak normalized tip resistance and \( Q_{\text{pen},\text{final}} \) is the final normalized tip resistance before the base boundary effect is detected, by which stage particle crushing would have transformed the native soil. Table 1 shows that normalized tip resistance ratio \( Q_{\text{pen}} \) is almost exactly equal to the area ratio \( \left( \frac{A'}{B'} \right)^2 \), which shows that \( B' \) can eliminate the peak effect in Fig 4 if other effects can explain the rest.

4 EFFECTS IN PILE DESIGN

The application of CPT in pile design has been very common among practising engineers on the grounds that the cone penetrometer can be treated as a model pile. Meyerhof (1976) suggested that when the pile point is above some critical depth in the bearing stratum, the unit tip resistance of a shallow pile \( q_{\text{pen}} \) should be reduced below the tip resistance \( q_t \), in proportion to the embedment ratio \( \frac{z}{B} \) in this stratum:

\[
q_{\text{pen}} = q_t \left( \frac{z}{B} \right)^{1/4}
\]

(5)

where \( z \) is the embedded depth of the pile. Presumably the factor \( \frac{z}{B} \) in eqn. (5) is to take care of the geometry effects and also the stress history effect.

Fig 6: Geometry effects between a penetrometer and a pile.

Fig 7: Tip resistance vs vertical stress.

Jamiolkowski et al (1985) demonstrated in calibration chamber tests that \( q_t \) is proportional to \( (\sigma_s')^{1.5} \). This should correspond to piles deeper than their critical depth. To illustrate the relationship between \( q_t \), (\( \sigma_s' \))\(^{1.5} \) and \( (z/B) \) for all depths, centrifuge results obtained by Kokkina (1993) are plotted in Fig 8. Prototype diameters of his piles were 452, 791, 800, 1400, and 1412 mm. All the results were plotted in the fashion of \( (q_t/\sigma_s') \) versus normalized depth \( (z/B) \). For a particular relative density and regardless of the diameter of the piles, all the results seem to adopt a unique form similar to that in Fig 9.

For \( (z/B)_{\text{crit}} \geq (z/B)_{\text{crit}} \), Fig 9, it is reasonable to assume the tip resistance of a pile to be:

\[
q_{\text{pen}} = q_t
\]

(6)

For \( (z/B)_{\text{crit}} < (z/B)_{\text{crit}} \), Fig 9, the tip resistance of a pile is taken to be:

\[
q_{\text{pen}} = \alpha \left( \frac{z}{B} \right)^{1/4}
\]

(7)
Taking $\alpha' = \gamma' z$, and knowing that

$$q_c = \alpha' \sqrt{\frac{1}{N_{e,1}} - \left(\frac{z}{B}\right)^m}$$

We obtain, $(\sigma/B)_{\text{min}} < (\sigma/B)_{\text{crit}}$:

$$\frac{q_{\text{piv}}}{q_c} = \left(\frac{z}{z_c}\right)^{1.5}$$

Fig 8: Plot of $\frac{q_{\text{piv}}}{q_c}$ versus normalized depth $Z$.

Depending on the relative density, $(\sigma/B)_{\text{min}}$ may lie between 5 to 20. Compared to dense sand, loose sand will have a smaller value of $(\sigma/B)_{\text{min}}$. Thus the equation proposed by Meyehof (1976) might well underestimate the tip resistance of a pile in loose sand. On the other hand, it would overestimate the tip resistance of a pile in dense sand, if Fig 8 is considered relevant.

5 DISCUSSION

It is clear that there are two phases of behaviour depending on the critical penetration depth ratio $Z_{\text{crit}}$. Shallower than some critical ratio $(\sigma/B)_{\text{crit}} < 20$ in Fig 4(a) the coefficient $Q$ increases with depth ratio in the fashion of shallow foundations. At depths greater than this critical depth, the coefficient seems to hold steady, or to fall slightly, characteristic of deep foundations. The dichotomy was observed by Meyehof (1983) in his 1g tests on model piles. This effect is highly significant in model tests with CPT probes which behave more like prototype driven piles, and less like field CPTs.

Fig 9: Idealized plot of $\frac{q_{\text{piv}}}{\sqrt{\gamma' z}}$ versus normalized depth $Z$.

A shallow mechanism is associated with surface heave while a deep mechanism is associated with local penetration. When the probe penetrates into the soil, cavity expansion occurs around the cone. As the probe penetrates further downwards, it will generate a succession of cavities, Fig 10(a) and (b) at increasing pressure. This phenomenon is introduced and defined here as the development of stress history. In a calibration chamber, the specimen is covered by a rigid plate which allows the vertical stress to be applied, Fig 10(a). Since heaving of soil around the probe is not possible, the shallow mechanism can not be developed as it is in the case of centrifuge or field tests, Fig 10(b).

Fig 10: Penetration mechanism in (a) calibration chamber, and (b) field/centrifuge.
Also, in calibration chamber tests, the shallow mechanism is omitted since most of the data points are recorded at the mid depth of the sample, much deeper than the critical depth. This means that calibration chamber correlations are not suitable for direct use in shallow foundation design.

6 CONCLUSION

A CPT can be regarded as a scaled-down pile, and its tip resistance offers the best starting point for design of full-scale driven piles. However, the bearing capacity of short piles must be reduced in relation to the critical depth of the formation, which can readily be explored in simple centrifuge tests.

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Delineating geostatigraphy by cluster analysis of piezocone data

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ABSTRACT: Cluster analysis is a statistical method for grouping similar mathematical data sets. Herein, cluster analysis is used to evaluate large numbers of piezocone data to delineate soil layering and subsurface stratigraphy in vertical profiles. In contrast with available statistical procedures using autocorrelation coefficients and varigrams, cluster analysis is a superior and objective approach. Clustering can accommodate multiple variables, such as the three readings taken at each depth (q, f, and uw) during piezocone penetration. Its application to defining stratigraphic interfaces is illustrated with two case studies with layered profiles and criteria based on normalized piezocone parameters, Q = (q - qc) / qc and U = (uw - uw) / uw.

1. INTRODUCTION

Stratigraphic profiling is a necessary first step in geotechnical site characterization and involves the delineation of vertical boundaries, number and types of soil layers, and their presence and locations of seams, lenses, and other features (e.g., cemented zones, voids, inclusions). Routine and conventional rotary drilling, discrete sampling, and soil testing methods are slow and expensive. Consequently, these are now being supplemented or supplanted by newer in-situ testing and sampling methods, such as the piezocone penetration test (PCPT). The piezocone is popular because it provides three independent measurements (q = cone tip resistance, f = sleeve friction, and uw = penetration porewater pressure) and screens the soil in the vertical direction at short intervals of between 1 cm and 5 cm (Lanne, et al. 1997). Notably, however, piezocone data are functions of both soil type and soil behavior, and thus the interpretation of soil layering and subsurface stratigraphy can necessitate uncertainty in the logging of results.

In current practice, piezocone data are interpreted to provide site stratigraphy by one or more of three approaches: (1) visual examination, which depends on the level of experience and expertise of the engineer; (2) using available empirical classification charts to separate like soil types; and (3) univariate and multivariate statistical methods (e.g., Wicks, 1989; Zhang, 1994). The latter category includes the intraclass correlation coefficient and generalized distance methods, however, neither of these is able to consistently and objectively define soil layer boundaries (Hegazy et al., 1997).

As an alternative procedure, a statistical method termed “cluster analysis” is proposed for analyzing piezocone data. The method is able to detect the inherent correlation between the piezocone data and give an objective assessment of layering and association with the stratigraphy. Example cluster results are presented and verified using backup laboratory and in-situ data for two case studies.

2.0 CLUSTER ANALYSIS

Cluster analysis is defined as a grouping of data which have similar mathematical descriptions (Everitt, 1974). The cluster analysis can be applied to data belonging to the same group in order to divide them into sections based on a predetermined criterion or set of properties (Romelfanger, 1984). Hartigan (1990) suggested the term “classification” interchangeably with the term “cluster” because it is familiar in different science fields such as medicine, biology, and engineering. Everitt (1974) suggested several possible uses of cluster analysis, including:

- Determining true typology or systematic classification of types having common characteristics;
- Model fitting:
• Estimation based on groups;
• Hypothesis generation and testing; and
• Data reduction and exploration.

In this study, cluster analysis is applied to the last category, namely piezocene data reduction and soil exploration.

A cluster analysis is introduced to: (1) objectively define similar groups in the soil profile, (2) delineate different layer boundaries, and (3) allocate the lenses and outliers within a sublayer. Different clustering techniques are reviewed and the hierarchical distinctions are established for the purposes of this study. The cluster analysis must be tailored for the particular application. In this case, measurements taken in natural geosystems, and thus associated with some scatter, are analyzed to determine the number of layers. Hegazy (1998) recommended the use of a "score" procedure for normalizing the data and a "cosine" measurement to indicate the similarity of the data. The statistical definitions of these parameters depend upon the mean and standard deviation (Stdev) of a cone variable X

\[ E(X) = \text{Mean}(X) = \frac{\sum x_i}{n} \]

\[ \text{Stdev}(X) = \sqrt{\frac{\sum (x_i - E(x_i))^2}{n-1}} \]

\[ \text{score} = s_i = \frac{x_i - E(X)}{\text{Stdev}(X)} \]

\[ \cos \theta = d_i = \frac{\sum x_i s_j}{\sqrt{(\sum x_i)^2 (\sum s_j)^2}} \]

where \( d_i \) is the similarity between two vectors, \( x_i \) and \( x_j \), of two cone readings (e.g., \( q_1 \) at different depths. Sibson (1972) and Milligan and Cooper (1988) recommended the use of single cluster methods when satisfy mathematical conditions such as continuity and minimum distortion.

If desired, a cluster grouping can provide as many clusters as there are data sets, therefore, a criterion must be set to evaluate the fewest number of clusters for consideration. Hegazy (1998) conducted extensive cluster analyses at many sites for a guide and the evaluations ranged from a cluster number \( N_c = 2 \) to \( N_c = 100 \). A simple criterion was developed to choose the appropriate cluster number based on a correlation coefficient (\( \rho \)) calculated between the data ranks of each consecutive pair of clusters as follows:

\[ \rho = \frac{\sum_{i=1}^{n} [(x_{i,j} - E(x_{j})) (x_{i,j+1} - E(x_{j+1}))]}{\sqrt{\sum_{i=1}^{n} (x_{i,j} - E(x_{j}))^2} \sqrt{\sum_{i=1}^{n} (x_{i,j+1} - E(x_{j+1}))^2}} \]

where -1 ≤ \( \rho \) ≤ +1, \( x_i \) is a cluster rank of a data vector (i), and j is the cluster number. For instance, if \( \rho \) between the two clusters j and j+1 is equal to 1, then the cluster j+1 does not add any more divisions of the data set than cluster j and the analysis can be stopped at cluster j. However, if \( \rho \) is equal to 0.5, then that might indicate significant differences between the assigned cluster ranks for the data. This will suggest that the cluster analysis needs to be performed again until \( \rho \) approaches a value = 1. Although \( \rho \) becomes close to 1, it will never equal 1 because there will be at least one difference of one point rank between two successive clusters. Cluster results will be examined at the peaks of \( \rho \), or in other words, after a significant change of data groups occur.

3.0. INTERPRETATION OF CLUSTER RESULTS

The cluster analysis provides the following results: related and unrelated soil layers, seams, lenses, anomalies, and transition zones between different layers. The anomalies can include natural soil inclusions, such as cemented layers or voids, or systematic errors related to measurement difficulties, including electrical noise, rod breaks, and random events.

Definitions are needed for a minimum layer thickness, a transition zone, and outliers due to soil lenses or anomalies. First, for the minimum layer thickness, Trethewell (1975) and Schmettmann (1978) found that the measured cone penetration resistance (\( q_s \)) is influenced by the interface ahead and behind the tip. The full response of \( q_s \) in calibration chamber tests
having softer to stiffer soil can be obtained after an interface zone equal to 10 to 20 times the cone diameter. However, in the case of a moving cone going from softer to stiffer soils, the full response of q, can be obtained in a distance smaller than 10 times the cone. Visirirat (1978) suggested that a soil layer should contain at least 20 points (almost equivalent to a layer thickness between 0.5 m and 1 m) of the piezocone data to be statistically significant.

Based on the above discussion, a minimum layer thickness t = 0.5 meters has been chosen. Two layer definitions are given as follows: A primary layer (designated "A") has t ≥ 1 m, while a secondary layer (designated "a") has 0.5 m ≤ t < 1 m. Soil mixtures and transitions are denoted by a, and A which indicate that there is no continuous group of data with t ≥ 0.5 m or t ≥ 1 m within layers "a" and "A", respectively. Note that a, and A have 0.5 m ≤ t < 1 and t ≥ 1 m, respectively. A transition zone (t < 0.5 m) is defined between soil layers or where at least three consecutive cone measurements have ascending or descending order of cluster numbers within a soil mixture.

The appropriate cluster is chosen to represent a certain soil stratigraphy at a minimum Nc where no primary layer with t ≥ 1 m is separated at larger cluster numbers. The chosen cluster can occur at a certain peak of q, or between two consecutive peaks. Therefore, if the criterion is satisfied at a certain peak of q, (say l) at Nc, the cluster results should be examined between the two peaks i and l-1. A summary of the cluster analysis procedure is shown in Fig. 1.

4.0 APPLICATION TO PIEZOCONE DATA

Comparative experimental studies on the reliability and accuracy of piezocone measurements (q, f, and v) using different commercial cone penetrometers have shown that tip (q) and pore pressure (v) readings at the same depth are relatively repeatable (Lunno, et al. 1986; Tanaka, 1995). However, these studies indicated a large scatter of the sleeve resistances (f) and a significant dependency on the manufacturer type of penetrometer. Therefore, f measurements have not been included in the cluster analysis considered herein. If improved cone production results in reliable f measurements at later dates, however, cluster analyses can easily accommodate this third reading, as well as additional measurements such as downhole resistivity, dielectric readings, and/or other independent data.

Wright (1984 and 1988) recommended the use of the normalized cone tip resistance, Q = (q - qo)/qo, and normalized porewater pressure, Bp = Δwp(q - qo), for interpreting piezocone data. Also, Robertson (1991) proposed a soil classification method based on Q and Bp. Consequently, Q and Bp are used as inputs to the cluster analysis in this study. The method is applied to evaluate soil stratigraphy at two case studies having layered vertical profiles.
4.1. Cluster Analysis at Amherst, Massachusetts

One of 15 similar piezocene soundings (PCPT-1) conducted by the authors in a deposit of varved clay is shown in Fig. 2. The clay site is located at the National Geotechnical Experimentation Site in Amherst, Massachusetts (Lally, 1993; Lutzenegger, 1995). The soil stratigraphy consists of a 2-m silty clay fill underlain by a 2-m desiccated sandy silt to sandy clay crust that overlies a soft brown to gray clay to a depth of 5 m underlain by a nearly normally-consolidated gray varved silty clay until the end of the sounding at a depth of 14.5 m.

A visual examination of the $q_s$ measurements suggests four separate layers, with boundaries given at approximate depths of 1.8 m, 4.0 m, and 5.2 m. The visual boundaries of the $q_s$ measurements and borehole soundings are in general agreement. With the poro-water pressure data ($u_2 = u_3$), only one apparent boundary at $z = 4$ m is evident. This may be attributed to penetration in the vadose zone above the water table which may have caused partial desaturation of the porous element. As a consequence, this resulted in a delay in full response of the measured penetration pore pressure at lower depths.

The modified Robertson chart (1991) was applied to classify the soil types at the site, indicating sandy to silty mixtures between depths of 0 to 4 m. At depths between 4 and 14.5 m, the soil categorized as clay to silty clay with indications that the overconsolidation ratio was decreasing and sensitivity was increasing with depth.

A single cosine-z score method (Hegazy, 1998) was applied to the normalized parameters $Q$ and $B_s$. The autocorrelation between consecutive clusters was studied for cluster numbers of up to $N_c = 100$ and the cluster results at these peaks of $p_c$ varied up to $N_c = 29$. At $N_c = 2$, two major clusters appeared with a boundary at 5.8 m. An additional cluster was added at regular depth intervals, indicating the locations where successive cone rods were added during the sounding and thus corresponding to a test procedure. The fact that this procedure caused an error in recorded measurements is important since the cluster results are unaffected! Consequently, the cluster analysis can effectively separate out systematic errors (as well as random data or outliers) from the natural geological data. In further clusters, a primary cluster level ("A") with $t > 1$ m separated from the upper cluster at $N_c = 3$. At $N_c = 8$, a new primary cluster separated at depths of between 2.2 m and 3.9 m. For cluster numbers $N_c > 8$, no new primary layers of type "A" appeared, although transitional regions and lenses continued throughout the profile. In the final tally, a cluster number 8 was chosen as the minimum grouping that sufficiently demarcated the stratigraphy at the Amherst site. The results are fully-supported by water content data given by Lutzenegger (1995), as shown on
Fig. 2. Additional data (consolidation, unit weights) support the layering and boundary depths.

4.2. Cluster Analysis at Troll Site, North Sea

Cluster analysis was performed on piezocone data for an offshore deposit at the Troll site in the North Sea (Amundsen et al., 1985). The soil stratification from below the mud line and down to 45 m consists of firm and very stiff clay. A representative piezocone sounding is depicted in Figure 2. A visual look at the q<sub>v</sub> and u<sub>2</sub> profiles shows a single uniform layer at the Troll site, interrupted by an intermediate silty or sandy zone between depths of 17 to 20 m.

Using Robertson's chart (1991) and normalized parameters Q and B<sub>4</sub>, indicates the presence of a thick clay to silty clay layer from 0.75 m to 44 meters depth. The small "bip" at 17 to 20 meters also categorized as a silty to clayey material. No distinction was evident between the upper (0 to 15 m) and lower (20 to 45 m) layers of silty clays.

A single-core-Z-score cluster analysis was applied to the Q and B<sub>4</sub> piezocone data and fully-visualized for large numbers between N<sub>L</sub> = 2 and 100. Cluster peaks of p<sub>L</sub> were observed up N<sub>L</sub> = 41. At early as cluster number N<sub>L</sub> = 2, two primary clusters were separated and indicated a major difference in the soil types and/or properties above 17 m and below 20 m. In the latter intermediate zones, the cluster number

Fig. 3. Cluster Analysis and Stratification from Piezocone Data at Troll Site, North Sea.
(In-situ test data from Amundsen et al., 1985; Analysis by Hagley, 1998.)

The separate clay layers discovered by the cluster results are confirmed by laboratory water contents and plasticity measurements obtained at Troll, also shown in Fig. 3. The data are divided into two groups with a boundary at 18 m. The liquid limits average 59 percent in the upper clay stratum (0 to 18 meters) and 37 percent in the lower clay unit (20 to 45 m depths). Similarly, the water contents average about 65% in the upper unit and only 24% in the lower.

Clay sensitivity measurements also support the layer boundaries from cluster results (Fig. 3) and show that the upper unit is more sensitive (S<sub>L</sub> = 6) than the lower stratum (S<sub>L</sub> = 2).

5. CONCLUSIONS

A type of cluster analysis is formulated based on a single-core-Z-score assessment using normalized piezocone parameters, Q and B<sub>4</sub>, and used to delineate soil stratigraphy. The applicability of clustering the data is illustrated with results from two case studies, both having layered strata. The method is able to detect the depth of soil boundaries, lenses, seams, transitions and number of separate soil types. Cluster analysis using the single-core-Z-score method is shown independent of systematic errors, random
outliers, and surprisingly even able to detect subtle changes within the stratigraphy not evidenced by a visual examination of the raw data.

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Interpretation of signals from an acoustic cone penetrometer

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ABSTRACT: An Acoustic Cone Penetrometer has been developed using standard components to modify a CPT, and tested in sands under various combinations of stresses and density a large calibration chamber. The acoustic signal is measured by a standard audio microphone, and is characterised by using an Autoregressive model. The coefficients from this model, together with the data obtained from the Cone Penetrometer Test, are used to identify the type, density and stress level of a sand by means of neural networks. It is demonstrated that the analysis of the acoustic signal can improve the characterisation of soils at relatively low cost. The acoustic data is shown to be a reliable indicator of soil type. It can only be used, however, to identify relative density or stress conditions in soils that are similar to those in the data set used to train the neural network. The project serves as an example of application of modern signal processing methods to time-varying signals. The value of such methods is demonstrated, in particular the Autoregressive model is found to have advantages over the FFT to characterise the acoustic signal.

1 INTRODUCTION

The purpose of this research was partly to explore the applicability of modern signal-processing methods to data obtained from sensors used in civil engineering, and hence achieve a transfer of expertise between the two disciplines. Civil Engineers could gain by learning new methods that can be applied to their data; Information Technologists could gain from the experience of exposure of general methods to specific practical problems. This objective was met by applying the methods to interpretation of a variant the Cone Penetrometer (CPT) in which the acoustic signal emitted by the soil during penetration is also measured.

The CPT is commonly used in site investigation. It consists of a 60° conical tip at the end of a steel rod that can be pushed into the ground. The cone is usually driven from a purpose-built truck, which contains a hydraulic pushing device and houses all the necessary instrumentation and data acquisition systems. The cone typically has a cross-section of 15cm², and is pushed into the ground at 20mm/s. It is instrumented to measure (by a strain-gauged load cell) the resistance on the tip of the cone and the friction on a sleeve immediately behind the cone shoulder. Further instrumentation is often added. A variety of techniques have been developed to interpret the measured results to determine the mechanical properties of the soil. These are based on experience with field tests (e.g., Robertson et al., 1986), calibration in the laboratory (e.g., Houslsby and Hitchman, 1985) or theoretical analysis.

As the CPT penetrates the soil it makes an audible noise, and the technicians who operate the device come to recognise the noise associated with particular soil types. This has led to the development of the Acoustic Cone (ACPT), in which a microphone at the cone tip is used to pick up the signal during penetration. Villet (1981), Tringale (1983), Triangale and Mitchell (1982) and Mastarech (1986) developed such devices, and made qualitative observations about relationships between soil type and the recorded frequency and amplitude. In the research reported here, the applicability of this approach has been carried much further, including exploration of the possibility of quantitative interpretation to determine, for instance, soil density.

The research proceeded in four main phases:
- construction of equipment,
- testing to obtain a database of results in different soil types,
• development of methods to characterise the acoustic signal,
• development of methods to relate the acoustic signal and other measured quantities to known soil properties.

These phases are described in the following sections.

2 EQUIPMENT

2.1 Development of an Acoustic Cone Penetrometer

The ACPT was intended to be entirely compatible with existing equipment used by the industry. A standard 15cm³ CPT was loaned to Oxford University by Fugro McClelland Ltd for the duration of the project. The tip of the CPT, which is usually solid, was modified so that a standard Sennheiser audio microphone could be mounted within it (Figure 1). The microphone was powered from the same supply that was used for the load cell, and the signal channelled to the data acquisition unit using spare channels on the standard CPT system.

Data acquisition was achieved through two systems, both PC based. One operated at low frequency, and was used to measure the cone penetration, tip resistance and sleeve friction. The other system provided analogue-to-digital conversion at 100kHz, and was used to log the acoustic signal. The use of standard equipment means that the capital cost of converting a CPT to an ACPT is low (of the order of £300 for equipment, plus some technician time).

2.2 Calibration Chamber

The tests used to provide a database of ACPT results were carried out in a large calibration chamber developed at Oxford University for previous research (Schmidt and Housby, 1992). The chamber allows a cylindrical sample of sand, 1.0m diameter by 1.5m high, to be prepared at a known density.

Fixed vertical and horizontal stresses can then be applied to the sample by pressure bags (see Figure 2). The ACPT can be driven into the sample at the same rate as is used in field testing. Some minor modification to the calibration chamber was necessary for this research.

3 TEST PROGRAMME

Some 44 tests were completed in the calibration chamber. The tests covered:
• three different sand types,
• a range of densities for each sand,
• a range of stress states for each sand.

The sands used were: (a) Leighton Buzzard 14/25 sand, which is a hard-grained quartz sand, (b) Dega Bay Sand, which is a very weak-grained carbonate sand and (c) Hokksund Sand, which is a fine feldspathic sand with a significant micro content. The three sands have very different properties, and all have been much used in previous geotechnical research, including calibration chamber tests, so that their properties are quite well documented.

The density of a sand has an important influence on its engineering behaviour, especially the friction angle, and it was controlled by changing the rate of deposition for the specimen, which were prepared by pluviation from a hopper. Samples were prepared in "loose", "medium" and "dense" states, and characterised by their Relative Density $R_D$. 
Figure 3: Stress states used in tests

An important feature of the calibration chamber is the control that can be achieved of both horizontal and vertical stresses, so that the separate influence of these on the test results can be examined. As in previous research (Hodgson and Hitchman, 1988), a series of standard stress points were chosen (see Figure 3). These allow tests to be compared on the basis of different values of the mean normal stress \( \sigma' = (\sigma_v' + 2\sigma_h')/3 \), or of the stress ratio \( K = \sigma_h'/\sigma_v' \).

Table 1 gives the combinations of stress states and densities used in the 44 calibration chamber tests of the ACPT.

For each test a record of the data during the entire penetration of the ACPT is available, but near the tip and bottom of the chamber the results are affected by the boundary conditions. All the analysis is carried out on data from a 200mm section of the test near the centre of the sample. The data recorded for this section are:
- the sand type and density,
- the applied vertical and horizontal stresses,
- the conventional CPT measurements of tip resistance \( q_t \) and sleeve friction \( f_s \),
- the acoustic signal, digitised at 100kHz.

Each 200mm section was in turn divided into 10 slices, and averaged data obtained for each slice, so that a total of 440 sets of data were available from 44 tests.

4 TIME-SERIES ANALYSIS

The frequency content of the acoustic signal was first characterised by using the Fast Fourier Transform (FFT). The signal was represented by 512 frequency components, and a typical power spectrum is given in Figure 4(a). The power spectrum divides into three broad frequency ranges:
1. from DC to about 3kHz there is a response which is thought to be mainly associated with mechanical noise in the driving system etc.
2. from about 3kHz to about 13kHz is a series of rather well-defined peaks. The frequencies at which these peaks occur do not change from test to test. Since it is difficult to envisage such well-defined frequencies being associated with the soil, it is thought that these correspond to resonances within the ACPT itself.

![Figure 4: Comparison of FFT and AR representation of spectrum for Test A on loose Leighton Buzzard Sand](image-url)
3. from about 17kHz to 25kHz is a broad peak which is thought to be principally related to the soil properties. Trimble (1983) also detected a response in this range of frequencies.

A major disadvantage of the PFT is that the spectrum is defined by a large number of parameters (typically 512). This makes subsequent data-handling and analysis complex, and a more compact means of characterizing the signal is desirable. An Autoregressive (AR) model was found to be a suitable alternative. In this approach the magnitude of the signal \( s_n \) at any sampling point \( n \) is estimated as a weighted average of the previous \( p \) samples:

\[
 s_n = \sum_{i=1}^{p} a_i s_{n-i} 
\]

(1)

The weighting coefficients \( a_i \) are obtained by least-squares fitting, and are related to the dominant frequency content in the signal. The power spectrum can be reconstructed from the coefficients, and Figure 4(b) shows the spectrum for the same test as in Figure 4(a), using \( p = 15 \). Although the detail of the individual peaks in the 3kHz to 15kHz range is lost, the overall character of the spectrum is retained, even though many fewer coefficients are used (15 rather than 512 for the PFT).

By filtering the signal digitally to obtain just the response in approximately the 16kHz to 25kHz range, it was found that the signal could be satisfactorily described by as few as five coefficients in the AR model \( (p = 5) \).

5 INTERPRETATION USING NEURAL NETWORKS

Identification of soil properties from CPT tests is usually carried out using a series of ad hoc procedures and classification charts (see for example Robertson et al., 1989). Such an approach is difficult to extend to interpretation of more than a very few (say 2 or 3) measured quantities, and so a more systematic approach was taken to interpretation of the ACPT. An automated means of identifying soil characteristics from the data obtained in each test was developed using a Neural Network.

A neural network consists of number of interconnected “neurons”. Three layers are of neurons are used here: an input layer, a hidden layer and an output layer. The input layer neurons are assigned values depending on measured quantities. For instance, for the conventional CPT test, the input variables were the vertical stress (on the assumption that this can usually be estimated quite well), the tip resistance and the friction ratio. The values associated with the other two layers are obtained from weighted averages of the values in the previous layer. Figure 5 shows a single network with 3 inputs, a hidden layer (also of 3 neurons) and 3 outputs. In this case the outputs represent a classification as one of the three types of sand tested. Using methods that are well developed in the artificial intelligence area, the network is first trained by presenting sets of known data. During this phase the weighting factors that define the function of the network are determined. The trained network is then tested against new data, and its performance is assessed.

Table 2 shows the results of trials on the ability of a network to identify soil type. In Trial A, half the data from each of the 44 tests is used to train the network, and the other half is used for testing. When only the conventional CPT data is used, the soil is correctly identified in 99.35% of the tests. When the acoustic data is used (5 AR coefficients and a measure of the acoustic energy), a 100% identification rate is achieved. Each result presented in fact represents the average performance of 10 independently trained networks.

Trial B is a harder test in which the results of 14 (randomly chosen) tests are used to train the network, and the remaining 30 used to check the performance. The network based on the CPT data does not perform as well as the one using the acoustic data. Finally Trial C is an even more challenging test in which only 5 tests are taken as input and the remaining 39 are used for testing. The network trained on the CPT data achieves only a

<table>
<thead>
<tr>
<th>Trial</th>
<th>CPT data</th>
<th>Acoustic data</th>
<th>CPT and acoustic data</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>99.55</td>
<td>100.00</td>
<td>100.00</td>
</tr>
<tr>
<td></td>
<td>(44 input 4 output)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>86.57</td>
<td>99.93</td>
<td>100.00</td>
</tr>
<tr>
<td></td>
<td>(14 input, 30 output)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>41.33</td>
<td>100.00</td>
<td>93.23</td>
</tr>
<tr>
<td></td>
<td>(5 input, 39 output)</td>
<td></td>
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</tr>
</tbody>
</table>
41% success rate (scarcely better than the performance that would be expected from a random choice) whilst that using the acoustic data still achieves 100% recognition. The message is clear: for the purpose of identifying soil types (at least in sands) the acoustic signal provides much more reliable identification than the conventional data. Even when the network has been trained on only a tiny amount of acoustic data on a soil, a successful identification is achieved.

The counter-intuitive result that the performance in Trial C when both CPT and acoustic data is used is marginally poorer than when only the acoustic data is used is explained as follows. The number of weighting factors that have to be determined increases as the number of input variables is increased, and a correspondingly larger amount of training data is needed. For a fixed amount of training data, the performance of a network may therefore be degraded by introducing additional input variables.

The next stage was to use a neural network to identify quantitative measurements of the soil properties. The two variables chosen were the Relative Density and in situ horizontal stress, since these are important quantities often needed in engineering design, but difficult to determine by sampling methods (Houlshby and Wreath, 1989). In the equivalent of Trial A above, i.e. using half the data from each test for training and half for testing, the following results were achieved. When the CPT data are used the standard deviation of \( R_D(\text{measured}) - R_D(\text{calculated}) \) is 8.9%, indicating that the relative density can be determined with reasonable confidence. When the acoustic data is used alone the standard deviation is 12.0%, and when the CPT and acoustic data are combined it is 7.8%. This indicates that the acoustic data itself is not a particularly good indicator of density, but when used as a supplement to the CPT data it improves the estimation. Figure 6 shows the calculated and measured Relative Density for Trial A, with both the conventional CPT and acoustic data used as inputs to the network.

Disappointingly, when the equivalent of Trials B and C are carried out, the estimation of Relative Density becomes much poorer (the standard deviation as used above increases significantly). This indicates that the identification of \( R_D \) using the neural network is effective for cases where the network is presented with materials similar to those encountered before (i.e. interpolation), but performs much less well for cases dissimilar to those in the training set (i.e. extrapolation).

The conclusions for the identification of horizontal stress are broadly similar. For the equivalent of Trial A the standard deviation of \( \sigma(\text{measured}) / \sigma(\text{calculated}) \) is 27%, indicating that horizontal stress can typically be estimated to within 27% of the correct value (this is in fact remarkably close for a simple in situ test). The equivalents of Trials B and C showed, however, too much variability for the method to be a useful indicator of horizontal stress.

The above observations confirm the finding of Houlshby and Hitchman (1988) that the results of CPT tests depend on both horizontal stress and density, but in such a way that they cannot be determined individually without further independent testing. The acoustic signal was expected to provide independent data since the frequency content was expected to depend on the soil modulus, but the dependence is insufficiently strong to use it to establish horizontal stress accurately.

The next stage of development of the ACPT is seen as a programme of field trials to accumulate a database of acoustic measurements in a wider variety of materials.

6 CONCLUSIONS

The conclusions fall into two groups: general conclusions on the experience with applying information processing techniques in civil engineering, and specific conclusions about the ACPT.
6.1 General conclusions

1. The Autoregressive model was found to be a highly efficient way of describing the main features of a time-varying signal, with it being preferred to the Fourier Transform representation because many fewer parameters are required.

2. Neural networks were found to be simple and convenient tools to establish correlations between a variety of measured quantities (CPT and acoustic data) and the desired engineering parameters (soil characteristics).

6.2 Specific conclusions

1. The part of the acoustic signal obtained from an ACPT test associated most strongly with soil characteristics appears to be that in the 13kHz to 23kHz range.

2. The acoustic signal provides excellent characterisation of materials (at least for sands), even when only a very small amount of data is available.

3. Data from the ACPT provides a broad indication of density of a sand.

4. The acoustic signal from the ACPT is not strongly dependent on stresses, and so cannot be used to determine stress reliably except in soils similar to those used for training the neural network.

7 ACKNOWLEDGEMENTS

This research was carried out with the support of the Engineering and Physical Science Research Council of the U.K., in co-operation with Prof. L. Tarassenko. The CPT was loaned to Oxford University by Pagro McClelland Ltd.

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In-situ soil characterization using Vision Cone Penetrometer (VisCPT)

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ABSTRACT: The cone penetration test (CPT) has gained wide acceptance for in-situ measurement of mechanical soil properties and for stratigraphic characterization. However, the determination of soil type can only be made through empirical correlations with CPT sounding data or by separately retrieving field specimens. This drawback led to the development of a new “vision cone penetrometer” (VisCPT) which permits direct, continuous, real-time visual observation and recording of the subsurface. The cone is pushed into the ground at the standard CPT advance rate of 20 mm/sec. High and low magnification miniature CCD camera housed within the VisCPT having fields of view of 2.3 and 18.0 mm respectively provide the continuous stream of images. A field video recording system allows the video sequences from each camera to be recorded in real time on two super VHS (S-VHS) format video recorders. Subsequently, this set of images are digitized and computer vision techniques based on textural image analysis are utilized to interpret the images for characterization. Results of field tests are presented.

1 INTRODUCTION

Proponents of the CPT cite its many advantages over the standard penetration test (SPT) for site characterization and determination of engineering properties of soil. However, unlike the SPT, the CPT does not furnish a soil sample for direct observation and classification. Instead, the soil type is inferred through empirical correlation with the cone tip resistance and sleeve friction. A piezo-cone does help by measuring the development and dissipation of excess pore pressures in the vicinity of the cone tip; however, the soil type is still characterized empirically and indirectly. Furthermore, piezo-cone results are sensitive to the degree of soil saturation and interpretation is only reliable at full saturation. CPT results are often ambiguous and inconclusive, particularly in thinly stratified soils where the readings reflect the composite response of adjacent layers.

A new vision cone penetrometer (VisCPT) (Rueschke and Hryciw 1997a) described herein removes the major existing limitation of the cone penetration test: that is, the inability to directly observe the soil. Changes in stratigraphy, precise location of interfaces between soil layers, and small-scale subsurface anomalies such as clay seams, sand lenses, etc., which may be too small for detection by the standard CPT can thus be identified.

2 VISION CONE PENETROMETER (VisCPT)

The VisCPT consists of a housing unit assembly and two camera/lens/lighting systems. Figure 1 shows the assembled camera housing units, housing unit connector, housing unit to penetrometer connector and housing unit to push rod connector. The lower housing unit contains the low-magnification camera while the upper housing unit contains the high-magnification camera. Each CCD camera consists of a remote miniature head unit and a camera control unit that are connected via a video/power cable. The camera’s remote head contains a 13 mm CCD imager with a resolution of 768 x 494V picture elements. All of the camera’s other electronic components (e.g. shutter speed controller, gain setting, etc.) are enclosed in a separate camera control unit above ground. An optical system is mounted on the remote camera head, consisting of an objective lens and an extension tube. The level of magnification is achieved by selecting the appropriate focal length distance of...
the lens and the corresponding length of extension tube. The camera system is mounted vertically downward in the camera housing unit using a special assembly. Each camera has its own custom lighting system, each consisting of a different geometrical arrangement of miniature red LED lights for illumination. Two opposite faces of each camera housing unit are machined flat across a 25 mm wide strip for placement of a synthetic sapphire viewing window as shown in Figure 1. The housing units and connectors share a common thread diameter and pitch to make the system modular. Therefore it is possible to use as many camera units for subsurface exploration as desired.

An in-field video system is used for recording and viewing the subsurface images. The images are recorded in real time (30 image frames/sec) as the probe is pushed into the subsurface at the standard CPT advance rate of 20 mm/sec. The high and low magnification cameras provide image fields of view of 2.3 and 18.0 mm respectively (measured diagonally). Digitized images have spatial resolutions of approximately 0.0008 and 0.022 mm/pix, which allows for particle-level visualization of soils with grain sizes ranging from coarse sand to silt. Live video images from each camera can be observed on monitors as the VisCPT probe is pushed into the ground. Two S-VHS video recorders are also used to record the video image sequence for archival purposes and for subsequent digitization. In the digitizing process, each image is converted into a digital gray scale matrix of integer values ranging from 0 to 255. Computer image analysis based on statistical textural analysis is employed to characterize the soil images as will be outlined in the next section.

3 COMPUTER IMAGE ANALYSIS

Imaging techniques are proving to be a viable alternative to mechanical sieving for determination of soil grain size distribution. While grain size distributions are easily obtained when the soil grains are non-contacting (Raschke and Hryciw, 1997b); interpretation of in-situ images of contacting grains (assemblages) is considerably more difficult. Several different approaches have been developed based on textural analysis by Fourier transforms (Hryciw and Raschke 1990); edge detection and completion by Hough transforms and active contouring; and pixel edge density analysis (Hryciw et al., 1997).

In this paper, a more versatile approach based on statistical textural analysis is investigated. Statistical textural analysis techniques have been extensively utilized in industrial applications and in the health sciences. Industrial inspection and assessment of quality control were investigated by Siow et al., 1988; Terrance classification from satellite and aural imagery photographs was first studied by Weszka et al., (1976). Computer classification of reservoir sandstones was investigated by Haralick et al., (1973). Many other applications can also be cited.

Statistical Texture Analysis (Spatial Gray Level Dependence Method)

Texture is essentially the uniformity or spatial distribution of gray scale intensities in an image. It can be described and quantified by the overall or “average” spatial relationships of gray levels in the image. In VisCPT images, variations in texture arise from several combined effects including; 1) higher
illuminated of particles at the face of the sapphire window, 2) dark zones along particle edges due to shadowing and 3) natural variations in particle color and translucence. Since VisCPT images are continuous and at relatively high magnification, only the overall image texture needs to be considered while variations of texture within individual images are ignored.

To begin quantifying texture, we let \(I(x,y)\) be the digital image matrix under consideration, whose cell values belong in the gray scale spectrum with integer values ranging from 0 to \(N_0\) where \(N_0\) is the number of gray levels in the image (typically \(N_0=255\)). From \(I(x,y)\) a co-occurrence matrix \(P(i,j;d)\) is created. In this new matrix, both \(i\) and \(j\) are the gray scale values from 0 to \(N_0\) and \(d\) is a chosen spatial distance typically taken as 1 pixel unit. The co-occurrence matrix \(P(i,j;d)\) is then the normalized probability density function of having two image pixels of gray level values of \(i\) and \(j\), separated by a spatial distance \(d\). In other words, the matrix \(I\) is searched for all pixels having gray scale values \(i\) and \(j\) separated by distance \(d\) along the eight 45° fan rays originating from the first pixel. In mathematical form, \(P(i,j;d)\) can be given as follows:

\[
P(i,j;d) = \frac{1}{R} \left\{ \begin{array}{c} f(\{k,l\};m,n) \text{ such that } |k-m| = d, |l-n| = 0 + f(\{k,l\};m,n) \text{ such that } |k-m| = 0, |l-n| = 0 + f(\{k,l\};m,n) \text{ such that } |k-m| = 0, |l-n| = d + f(\{k,l\};m,n) \end{array} \right\}
\]

where \(f\) is the accumulation frequency operator, \(\{k,l\}\) and \(\{m,n\}\) are the locations of pixels \(i\) and \(j\) respectively in the image matrix \(I\), \(R\) is the normalizing factor and equal to the number of entries utilized to accumulate the \(P\) matrix. The summation of the entries in the \(P\) matrix is unity:

\[
1.0 = \sum \{ P(i,j;d) \}
\]

The \(P\) matrix is obviously camera magnification (i.e. pixel scale) dependent. A magnified image of sand and air-photos of boulders can yield similar \(P\) matrices if the pixel to sand grain and pixel to boulder size ratios are equivalent. Conversely, by varying the image magnification of a sand, widely varying \(P\) matrices can be generated. In the present study, only images taken by the low magnification camera at a pixel scale of 0.022 mm/pixel were utilized. At this scale, coarse sand grain would be 30 to 100 pixels in diameter, fine sand grain would be 3 to 10 pixels and silt grain would be 0.1 to 3.0 pixels across.

If the soil is coarse relative to a pixel unit, and \(d\) is set small compared to the average grain size, then the likelihood of having a pair of pixels at separation \(d\) at similar gray levels would be high. This means that in coarse grained sands, high values in the matrix \(P\) should be concentrated on or near its main diagonal. Conversely, for a fine textured sand this likelihood would be smaller and high matrix entries would not concentrate along the diagonal. For fine sands, the higher frequency of particles and interfaces will generate more off-diagonal entries in the co-occurrence matrix, \(P\).

However, this trend reverses itself as particle sizes fall below the size of pixels. In this case, the pixel entries in \(I\) reflect the average gray scale of the particles, edges and voids within the pixel boundaries. The ultimate result is a very uniform (single valued) texture which yields a \(P\) matrix with a single dominant peak situated on the main diagonal.

Hurlburt (1973) proposed a set of measures or "indices" that can be used to extract useful information from the co-occurrence matrix \(P\). The four indices used in this study are:

1) Energy: \(E = \sum \{ P(i,j;d) \}^2 \) \hspace{1cm} (3)

This is a measure of the homogeneity of an image. As mentioned earlier, in a homogeneous image, such as of silt or clay, there are very few dominant gray-level transitions. Hence the \(P\) matrix will have a few large magnitude entries along the diagonal and therefore a large \(E\).

2) Entropy: \(ENT = -\sum \{ P(i,j;d) \log P(i,j;d) \} \) \hspace{1cm} (4)

Entropy is a measure of image complexity. It is largest for uniform distribution of small \(P(i,j;d)\) entries, and small when they are very unequal and sparsely distributed. The log function in Eq. (4) diminishes the effect of peak values within the \(P\) matrix as typically generated by uniform gray scale images. On the other hand, values of smaller magnitude (but more uniformly distributed) will endure substantially lower reduction in magnitude.

3) Contrast: \(CON = \sum \{ (i-j)^2 P(i,j;d) \} / N_0 \) \hspace{1cm} (5)

This is essentially the moment of inertia of the matrix entries around its main diagonal and a measure of the amount of local variation present in the image. In other words, if there are frequent changes in the gray level of an image over short distances, more off-
diagonal entries in the P matrix will be generated. This will ultimately lead to an increase in the contrast ‘ maurice’. Clayey and silty soils with no pronounced variations in gray scale lead to low contrast.

4) Local Homogeneity:
\[ LH = \sum \left( p_{i,j} \cdot d_{i,j} \cdot (1 + \mu^2) \right) \]  

(6)

This measures the degree to which similar gray levels tend to be neighbors. This parameter is basically the opposite of contrast.

4 EXPERIMENTAL RESULTS

In this study, a 15 cm diameter hole was drilled to a depth of approximately 1.0 meter. The hole was carefully filled with four layers of selected soils. A typical image of each soil type captured by the low magnification camera is shown in Figure 2. The soil profile and soil types are shown in Figure 3. Soils of different average grain size and uniformity were used in this study for the purpose of establishing relationships between these traditional geotechnical indices and the textural indices discussed earlier.

The test was performed by pushing the ViaCPT through the center of the hole, penetrating through all of the soil layers and into a natural post-glacial till below. A water table was found at approximately 0.90 m below the ground surface. For this study, 1916 subsurface soil images of the topmost 1.30 m was digitized. To preserve image quality and to reduce computational time, only the middle 400x300 sub-image was considered for analysis. The surrounding border was slightly blurry and was not included in the analysis. In order to minimize the adverse effect of noise introduced in the tape recording process and image compression during digitization, a 3x3 mean average filter was utilized to smooth out the image. This process was found vital in minimizing the scatter in the results. Results from the textural analysis are plotted versus depth in Figure 3.

As predicted earlier, the energy shown in Figure 3a increases abruptly as the natural fine-grained soil is reached (layer #5). As observed in Figure 2, a typical image of layer #5 has no discrete grain edges, very narrow gray scale bandwidth, and little contrast in gray scale values between adjacent pixels. This results in high energy (E), low contrast (CONT), low entropy (ENT), and high local homogeneity (LH). It is important to point out that any of these parameters would be adequate in identifying silt and clay soils.

The insert in Figure 3a is a magnified energy profile of the first meter of depth. It is observed that for soils of uniform gradation, E is generally higher for coarser grained than finer grained sands. The presence of the water table is identified by the gradual increase in E as water content increases.

Figure 2. Typical soil images of different soil layers
Figure 3. Log profile of textural analysis test results. Image Scale = 0.022 mm/pix.
The presence of moisture tends to make the image more uniform with less contrast, as it diminishes the presence of grain edges and narrows the gray scale bandwidth of the image. This behavior is noticed in all four graphs shown in Figure 3.

Figure 3.b is the Entropy or complexity. It may also be a good index for categorizing silty-clay soils. The TNT shows less scatter in the data compared to E. However, the entropy does not seem to be a good indicator of average grain size in sand.

The contrast profile shown in Figure 3.c seems to be an excellent indicator of the uniformity of coarse soil grains. Layer #1 (Ottawa Sand 20-30), a fairly uniform soil, exhibited the lowest average value for contrast, while the well graded soil of layer #2 exhibited the highest. The CONTs of the other two layers were intermediate. It is interesting to mention that the degree of scatter in the contrast profile is proportional to the average grain size in the image. This fact can be attributed to the increase in the variation in images as grain size increases. As mentioned earlier, the presence of moisture in the soil tends to obscure the presence of grain edges in the image. This results in an appearance of uniformity which in turn leads to lower contrast levels as observed beneath the ground water table.

The local homogeneity (LH) shows excellent correlation with the average grain size for uniform soils as shown in Figure 3.d. The consistent values of LH in each layer, narrow scatter of data and sharp transitions at the exact locations of layer interfaces is clearly indicated.

5 CONCLUSIONS

It is shown that statistical textural analysis holds great promise for soil characterization and classification. This technique is more computationally economical than other methods such as edge detection and spectral analysis. The spatial gray level dependence method does not require any image transformation, which is an essential element in the spectral analysis. Likewise, it does not require computing the computationally expensive gradient matrices necessary to locate particle edges needed by edge detection methods.

In this study, four textural indices were evaluated for field captured images of different types of soils. Energy and Entropy were shown to be good indices of the general soil type. Contrast was additionally shown to be a good index of soil uniformity for sands. Local Homogeneity shows the greatest promise for in-situ soil characterization as it not only identifies the general soil type but appears to resolve grain sizes in coarse grained material.

6 ACKNOWLEDGEMENTS

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Uncertainty propagation in CPT-based liquefaction resistance evaluation

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ABSTRACT: Existing CPT-based methods for evaluation of liquefaction resistance or potential are mostly empirical or semi-empirical. Uncertainties generally exist in the soil parameters used in the analysis as well as in the correlations (or models) with which the analysis is performed. This paper examines the propagation of these uncertainties in the liquefaction evaluation and the effect of these uncertainties on the results of the evaluation. The results clearly demonstrate the significance of the parameter and model uncertainties in the calculated liquefaction resistance.

1. INTRODUCTION

Because of the discrepancy in the equipment and the practice of the SPT, consistent repeatability of the test cannot be assured and the reliability of the SPT-based liquefaction evaluation methods might be challenged (e.g., Robertson and Campanella, 1985). Thus, use of CPT-based methods for liquefaction evaluation is generally preferred.

Earlier CPT-based methods were essentially a conversion from the SPT-based methods, using SPT-CPT conversion (Robertson and Campanella, 1985). In recent years, significant progress on the CPT-based methods has been made, including ever-increasing field performance data from sites with CPT data (Stark and Olson, 1995; Suzuki et al., 1995; Olsen and Koester, 1995). These recent field performance data show that the correlation between the cyclic resistance ratio (CRR) and the normalized cone tip resistance \( q_{u,c} \) established by Robertson and Campanella (1985) provides a good estimate of CRR for clean sands. For sands with significant fines content, an integrated approach has recently been proposed by Robertson and Wride (1997) which, among several other features, has a built-in step for correcting the measured cone tip resistance to a clean sand equivalent value. The CRR-\( q_{u,c} \) correlation curves established by Robertson and Wride (1997) for sands with different fines content, based on this integrated approach, compared favorably with Stark and Olson’s (1995) correlations developed based on field performance data. The correction of \( q_{u,c} \) to account for fines content proposed by Robertson and Wride (1997) also compared favorably with Olsen and Koester (1995) and Suzuki et al. (1995).

While Robertson and Wride’s (1997) integrated approach (Figure 1) is a comprehensive method for liquefaction evaluation, its extensive use of the empirical correlations/models inevitably introduces much uncertainty into the solution. Thus, it should be of interest to examine the propagation of all the parameter and model uncertainties in the evaluation process and its effect on the calculated liquefaction resistance.

2 METHODOLOGY

To begin with, the method by Robertson and Wride (1997), called the RW method hereinafter, is used to evaluate historical cases compiled by Stark and Olson (1995). Because the sleeve friction \( f_s \) is not available in Stark and Olson’s data set, only those cases that are listed with fines content (FC) are evaluated. In those cases, FC is treated as an input data, which essentially “by-passes” the steps of calculating FC from \( f_s \) and \( q_{u,c} \). In addition, a few cases that were considered questionable by Stark and Olson (1995) are not included. A total of 82 cases
are evaluated and the result is shown in Table 1. In this table, the "correct" prediction refers to the cases where the predicted outcome (liquefaction or not) agrees with field observation. Note that the number of correct predictions is conservatively counted, as in some cases, the count can go either way because the CRR is approximately equal to the cyclic stress ratio (CSR). As shown in Table 1, the RW method is fairly reliable in "predicting" historic liquefaction performance data.

Table 1. Evaluation of field performance cases

<table>
<thead>
<tr>
<th>Method</th>
<th>Number of cases examined</th>
<th>Number of correct prediction</th>
<th>Percent of correct prediction</th>
</tr>
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<tbody>
<tr>
<td>RW</td>
<td>85</td>
<td>64</td>
<td>75%</td>
</tr>
<tr>
<td>Stank</td>
<td>85</td>
<td>69</td>
<td>81%</td>
</tr>
</tbody>
</table>

The uncertainties associated with the four models/correlations are briefly discussed in the following. The correction of $q_s$ for overburden pressure in the calculation of $q_{ps}$, referred to as model $C_{ps}$, is considered as an uncertain variable, as some discrepancy exists among the existing empirical models for $C_{ps}$. The term $L_k$ is a function of $q_s$ and $f_c$, no uncertainty is assessed; rather, the uncertainty is reflected in the model that is used to calculate $FC$ (referred to as the model FC). The correction of $q_{ps}$ for its clean sand equivalence is another source of uncertainty, and it is reflected in the model $K_{eq}$. The last model uncertainty considered is the CRR. It is well recognized that there is currently no "true limiting state" and the model for calculating the CRR generally involves uncertainty. In the present study, the propagation and the effect of the parameter and model uncertainties are examined.

Uncertain parameter is generally modeled with a random variable that is specified to assume a particular distribution. The normal distribution, the log-normal distribution, and the beta distribution are frequent choices for soil parameter and model uncertainties. Use of these probability distributions implies knowing the uncertainty a lot more than our limited knowledge and data can justify. While the choice of distribution does not affect the methodology presented herein, a simple triangular distribution is used in the present study. The triangular distribution, shown in Figure 2, requires only an additional estimate of the range of the
uncertain variable, in terms of a certain percentage of the mean value. For example, a ±30% uncertainty on the depth of groundwater table \(z_w\) means that the groundwater table could be anywhere between 0.7 \(z_w\) to 1.3 \(z_w\), although it is most likely to be at the depth of \(z_w\). This concept is also applied to the model uncertainty. Thus, for example, a uncertainty of ±20% for the CRR model means that the "true" CRR may be in between 0.8 to 1.2 times the calculated CRR, assuming the calculated value is the "best estimate" according to this model.

![Figure 2. A triangular distribution](image)

To investigate the effect of the uncertainties in the parameters and the models, a spreadsheet, shown in Figure 3, is set up with Excel to implement the RW method (other spreadsheet programs may be used). In addition, an add-on software called @Risk is used. This software is a Excel macro which allows the user to specify any cell as a random variable with a prescribed distribution. The four uncertain soil parameters and the four uncertain models are specified as the triangularly distributed random variables. The software @Risk uses a simulation technique to handle the distribution. In the present study, an efficient sampling technique called the Latin Hypercube, rather than the Monte Carlo sampling technique, is used. The solution begins with sampling of each of all the uncertain parameters/models. Then, it proceeds to calculation using the RW method until a factor of safety (FS), defined as the ratio of CRR to CSR, is obtained. The process is repeated and iterated to a specified number of times, or until the convergence of the key results (including the mean value and the standard deviation of FS, and the cumulative distribution curve of FS, which is defined by a set of percentile values) is secured. In the present study, the maximum number of iterations is set at 10,000, and the convergence criterion is set as a relative error of 1%. In all cases studied, the convergence is reached before reaching the maximum number of iterations.

3. NUMERICAL EXAMPLES

The site from which the CPT data were obtained is in Lüli, Taipei, Taiwan. The soil profile obtained from bored holes consists of (from top to bottom) a layer of clayey silt of about 5 m, a layer of silty sand/sandy silt of about 15 m, and a deep layer of clayey silt and clay mixture below the depth of 15 m. The CPT data at 5 depth levels are used as an example to illustrate the methodology outlined above.

A total of 15 cases involving different combinations of uncertain parameters/models is analyzed to investigate the effect of uncertainty. These cases are grouped into three categories: 1) uncertainty in the input parameters only, 2) uncertainty in the models only, and 3) uncertainty in both parameters and models. Table 3 shows a summary of these cases.

Note that the probability of failure \(P_f\), defined as the probability that FS is less than 1, and the FS reported in Table 2, are for the soil at the depth of 10 m. The results for other depth levels are not shown to save space. Obviously, the FS and \(P_f\) values depend not only on the resistance (CRR) but also on the load (CSR). In this study, the major contributing factors of CSR, namely, \(a_{\infty}\) and M, are set to constant \((M=7\) and \(a_{\infty}=0.14\), as the focus in this study is on the effect of uncertainties on the calculated CRR (and thus, the FS and \(P_f\). In Case 1, both the parameter uncertainty and the model uncertainty are considered in the @Risk analysis.

The level of uncertainty in each of the uncertain parameters and models is assumed based on judgment and may be adjusted as needed. However, it does serve the present objective of investigating the propagation of the uncertainty. Note that on a 160 MHz Pentium-based PC, it took only 25 seconds of runtime to complete the @Risk analysis which involved about 1100 iterations in this case. Table 3 gives results of the analysis for Case 1 in greater detail. As shown in Table 3, the extremes of the FS values can be very much different from the mean values. This may explain why some historical non-liquefaction cases locate above the limiting state.
Liquefaction Analysis based on Robertson & Wride’s Method (1997)

Cyclic stress ratio (CSR) caused by a design earthquake:

\[ \text{CSR} = 0.65 \times MWF \times A \times SVR \times RD \]

- \( M = 7.0 \)
- \( W_r = 1.9 \) gm/cm\(^3\) (soil density)
- \( A = 0.137 \)
- \( W_w = 1 \) gm/cm\(^3\) (water density)

Normalized cone tip resistance for overburden pressure (q)eN:

\[ \text{q}eN = \frac{(\text{qc} - \text{P}_w)}{C} \]

Soil type index (Io) and fines content:

- \[ \text{Io} = \left( (3.47 - \log Q)^2 + (\log F + 1.21)^2 \right) \times 0.5 \]
- \( \text{FC} = 1.75 \times (\text{Io} - 3.25) + 3.7 \)

Normalized cone tip resistance corrected to an equivalent clean sand

\[ \text{q}eN_{\text{eq}} = \frac{\text{q}eN}{(1 - K_{\text{cept}})} \]

Cyclic resistance ratio (CRR) for clean sands under level ground

(\( M = 7.5 \) & No. of cycles=15)

\[ \text{CRR} = 95 \times \left( \frac{(\text{q}eN_{\text{eq}} \times 1000)}{t} \right)^{0.08} \]

<table>
<thead>
<tr>
<th>( z ) (m)</th>
<th>q_c (bar)</th>
<th>( f_s ) (bar)</th>
<th>\text{sig vo} (bar)</th>
<th>\text{sig vo} (bar)</th>
<th>C_q</th>
<th>\text{Io}</th>
<th>FC (%)</th>
<th>CRR \times 1000</th>
<th>K_{\text{cept}}</th>
<th>q_{eN} (bar)</th>
<th>CSR</th>
<th>CRR</th>
<th>F_s</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.20</td>
<td>24.44</td>
<td>0.16</td>
<td>0.79</td>
<td>0.97</td>
<td>1.12</td>
<td>2.21</td>
<td>19.48</td>
<td>27.47</td>
<td>0.29</td>
<td>44.78</td>
<td>0.10</td>
<td>0.09</td>
<td>0.92</td>
</tr>
<tr>
<td>10.00</td>
<td>47.63</td>
<td>0.16</td>
<td>1.22</td>
<td>1.85</td>
<td>0.91</td>
<td>2.26</td>
<td>21.03</td>
<td>42.23</td>
<td>0.43</td>
<td>75.54</td>
<td>0.11</td>
<td>0.12</td>
<td>1.05</td>
</tr>
<tr>
<td>15.00</td>
<td>62.51</td>
<td>0.37</td>
<td>1.66</td>
<td>2.79</td>
<td>0.78</td>
<td>2.15</td>
<td>17.49</td>
<td>48.57</td>
<td>0.23</td>
<td>72.88</td>
<td>0.11</td>
<td>0.12</td>
<td>1.08</td>
</tr>
<tr>
<td>28.45</td>
<td>26.31</td>
<td>0.41</td>
<td>2.84</td>
<td>5.30</td>
<td>0.59</td>
<td>2.65</td>
<td>37.72</td>
<td>15.61</td>
<td>0.80</td>
<td>78.03</td>
<td>0.08</td>
<td>0.12</td>
<td>1.57</td>
</tr>
<tr>
<td>38.40</td>
<td>82.53</td>
<td>1.75</td>
<td>3.81</td>
<td>7.34</td>
<td>0.51</td>
<td>2.21</td>
<td>19.45</td>
<td>42.59</td>
<td>0.39</td>
<td>68.85</td>
<td>0.07</td>
<td>0.11</td>
<td>1.63</td>
</tr>
</tbody>
</table>

Parameter and Model Uncertainty

(In terms of percent variation)

- \( W_r = 0.05 \)
- \( Z_r = 0.20 \)
- \( \text{qc} = 0.15 \)
- \( f_s = 0.30 \)
- \( C_q = 0.10 \)
- \( FC = 0.40 \)
- \( K_{\text{cept}} = 0.25 \)

Factor of safety \( F_s = \frac{\text{CRR}}{\text{CSR}} \)

Figure 3. Spreadsheet implementing the RW method
Table 2. Summary of cases analyzed

<table>
<thead>
<tr>
<th>Case No.</th>
<th>Uncertain Variables</th>
<th>FS-CRR/CSR</th>
<th>Pr</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W, = ±5%</td>
<td>1.14</td>
<td>0.41</td>
</tr>
<tr>
<td></td>
<td>Z, = ±35%</td>
<td>0.05</td>
<td>0.10</td>
</tr>
<tr>
<td></td>
<td>q, = ±15%</td>
<td>1.05</td>
<td>0.04</td>
</tr>
<tr>
<td></td>
<td>fs = ±50%</td>
<td>0.04</td>
<td>0.10</td>
</tr>
<tr>
<td></td>
<td>C, = ±10%</td>
<td>0.05</td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>FC = ±40%</td>
<td>1.05</td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>Krev = ±25%</td>
<td>0.05</td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>CRR = ±20%</td>
<td>0.05</td>
<td>0.01</td>
</tr>
</tbody>
</table>

Table 3. Results of the analysis of case 1

<table>
<thead>
<tr>
<th>z</th>
<th>5.2 m</th>
<th>10 m</th>
<th>15 m</th>
<th>28 m</th>
<th>39 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>min</td>
<td>0.64</td>
<td>0.67</td>
<td>0.77</td>
<td>0.85</td>
<td>1.15</td>
</tr>
<tr>
<td>max</td>
<td>2.99</td>
<td>4.45</td>
<td>3.95</td>
<td>2.09</td>
<td>5.11</td>
</tr>
<tr>
<td>mean</td>
<td>0.94</td>
<td>1.14</td>
<td>1.16</td>
<td>1.37</td>
<td>1.71</td>
</tr>
<tr>
<td>std</td>
<td>0.13</td>
<td>0.37</td>
<td>0.26</td>
<td>0.25</td>
<td>0.39</td>
</tr>
<tr>
<td>Pr</td>
<td>0.75</td>
<td>0.41</td>
<td>0.34</td>
<td>0.06</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Figure 4. Frequency distribution of calculated FS

To determine which uncertain parameter/model possesses the greatest effect in the calculated FS is rather difficult in this case, as these uncertain variables interact with each other. However, @Risk can be used to calculate the correlation coefficient between each uncertain variable and the resulting FS. This information, as shown in Table 4, gives some indication on the effect of uncertainty. As may be seen here, CRR model consistently ranks highest in correlating with the calculated FS, and FC model and Krev model consistently rank second and third. The influence of the CRR model (i.e., the "limiting state") on the calculated FS value is quite obvious. For a "fixed" CRR model, however, the FC model and the Krev model would have the greatest effect on the calculated FS. This is as expected, and caution should be exercised to verify the FC value when using the RW method. It is also observed that the effect of groundwater table could be very significant at some depths.

To further investigate the effect of uncertainty, 14 other cases, as shown in Table 2, were analyzed. Cases 2 through 6 were used to investigate the parameter uncertainty. With uncertainty in the groundwater table alone (Cases 2 and 3), the
standard deviation (std) of the calculated FS is negligible for the case of ±10% variation, and the \( p_i \) is almost equal to \( q_i \) when the uncertainty increases to ±30%, however, the \( p_i \) increases to 0.1. This indicates that the uncertainty in the location of groundwater table could be significant to the prediction of liquefaction potential. The effect of the uncertainty in the CPT data (combining \( q_i \) and \( f_i \), as in Cases 4 and 5) is seen to be slightly greater than the effect of the uncertainty in the location of groundwater table. The combined effect of the uncertainty in all four parameters (Case 6) leads to a slightly higher standard deviation and the \( p_i \) although the mean of the calculated FS slightly increases. Note that the even with a mean FS of 1.07, there is still significant chance for the occurrence of liquefaction, as indicated by the \( p_i \) value of 0.22 (Case 6).

Table 4. Correlation coefficient with calculated FS

<table>
<thead>
<tr>
<th>( F_1 ) at z = 5.2m</th>
<th>( F_1 ) at z = 10m</th>
<th>( F_1 ) at z = 15m</th>
<th>( F_1 ) at z = 20m</th>
<th>( F_1 ) at z = 25m</th>
</tr>
</thead>
<tbody>
<tr>
<td>CRR 0.88</td>
<td>0.98</td>
<td>0.98</td>
<td>0.99</td>
<td>0.99</td>
</tr>
<tr>
<td>( K_{crr} ) 0.51</td>
<td>0.83</td>
<td>0.75</td>
<td>0.81</td>
<td>0.74</td>
</tr>
<tr>
<td>FC 0.48</td>
<td>0.74</td>
<td>0.69</td>
<td>0.34</td>
<td>0.67</td>
</tr>
<tr>
<td>( f_i ) 0.11</td>
<td>0.09</td>
<td>0.12</td>
<td>0.09</td>
<td>0.10</td>
</tr>
<tr>
<td>( q_i ) 0.45</td>
<td>0.21</td>
<td>0.10</td>
<td>0.02</td>
<td>0.06</td>
</tr>
<tr>
<td>( W_1 ) 0.04</td>
<td>0.19</td>
<td>0.25</td>
<td>0.17</td>
<td>0.19</td>
</tr>
<tr>
<td>( W inverse ) 0.20</td>
<td>-0.18</td>
<td>-0.01</td>
<td>-0.07</td>
<td></td>
</tr>
</tbody>
</table>

The model uncertainty is examined with Cases 7 through 15. The effect of the uncertainty in the CPT model alone is approximately the same as that of \( z_i \) (Case 2). The uncertainty results in the FC model, \( F_{crr} \), and CRR model (the correlations that are used to estimate FC, \( K_{crr} \), and CRR in the RW method, respectively) is seen in Table 2 as very significant. In the possible range of variation, the \( p_i \) can go up to 0.36. The effect of the combined model uncertainties (Case 15) leads to a standard deviation of 0.35 for a mean value of 1.14. Note that the mean FS is halved slightly to 1.14 from a typical range of 1.05 to 1.07. Two possible reasons are: 1) the interaction among the 8 uncertain variables is highly non-linear, and 2) some upper bounds are set in the correlations used in the RW method. The almost identical results of Case 15 and Case 1 also supports the earlier observation that the effect of model uncertainty is greater than that of the parameter uncertainty. Another point worth mentioning is that the \( p_i \) can go as high as about 0.40 (2 in 5 occurrence rate) even the mean FS is as high as 1.14. In all cases examined in this study, the calculated mean FS has to be at least 1.7 in order to reduce the \( p_i \) to near 0.

4. CONCLUDING REMARKS

In this study the RW method has been used as an example to investigate the propagation of the parameter and model uncertainties in a CPT-based liquefaction evaluation method. The RW method is quite reliable in predicting the "mean" liquefaction resistance. However, the effect of the uncertainties on the calculated FS against liquefaction could be very significant. The \( p_i \) can go as high as 0.40 (2 in 5 occurrence rate) even the mean FS is as high as 1.14. For the cases studied, the calculated mean FS has to be at least 1.7 in order to reduce the \( p_i \) to near 0. The study also found that the model uncertainty is far more significant than the parameter uncertainty on the calculated FS and \( p_i \).

5. REFERENCES


Effect of negative pore pressure on the result of soundings and in situ tests in silt

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Swedish Geotechnical Institute, Linköping, Sweden

ABSTRACT: Negative in situ pore pressures often have a large impact on the test results in silty soils. In weight sounding tests, the resistance to penetration is then often very high and the soil often becomes classified as too coarse and stiff. In CPT-tests and dilatometer tests, the in situ pore pressure and the effective vertical stress are used in the interpretation of the test results and the soil classification will be affected and yield coarser soils than is the actual case. Also the estimation of shear strength and stiffness may be significantly affected.

The deformation characteristics and bearing capacity may also be determined by pressuremeter tests and plate loading tests. In both cases, the results are significantly affected by existing negative pore pressures. Unless it can be ascertained that the negative pore pressures are prevailing, the bearing capacity and the stiffness at higher loads may be seriously overestimated.

1 INTRODUCTION

Thick deposits of silty soils are common in large parts of Sweden, like in many other parts of the world. In Sweden, silt is normally a sedimentary soil which has been deposited in slowly streaming water. Most of the thick deposits have been deposited on top of coarser soil at a certain distance from the re-depositing ice front following the last glaciation. Other deposits have been and are still being deposited in river deltas or as lateral fluvial sediments and wave-washed sediments. Because of seasonal and climatic variations, the silt deposits are often varved and intermixed with layers of both coarser and finer soil.

After the deposition, many of the deposits have been elevated far above the sea level as a result of the subsequent land heave. Rivers and streams have eroded the relatively easily eroded silt layers, often all the way through to coarser soil in the bottom. During this process, the groundwater head has been lowered and a matrix suction has been created in the soil above the free ground water level. The matrix suction can be considerable due to the high water retention capacity (or capillarity) of the silt. Very high and steep slopes have often been created, the stability of which is highly dependent on this suction. Because of the higher permeability in the coarser soil layers below, the lowering of the free groundwater and the matrix suction in the silt layers may extend over very long distances from the ravines. Areas with thick silt deposits are usually fairly flat but broken through by a pattern of erosion ravines, often frequent enough to make the matrix suction prevail under all of the higher flat ground.

In other areas, the land heave has created conditions where the water in the coarser bottom layers is connected to water supplies in higher ground and the silty soil above acts as a semi-tight lid causing a condition with artesian water pressures. Sometimes, both matrix suction and artesian pressures may act in the same profile.

The stability of the steep slopes in silt depends heavily on the matrix suction and this has to be taken into account in order to explain their existence. At the same time, silt is highly susceptible to frost heave and smaller slides and surface erosion are common during the periods of snow melting and thawing and, in slopes without vegetation, also during periods of heavy rains.

Methods for measuring matrix suction in silt deposits have been developed, its importance for the stability in silt slopes has been illustrated and monitoring of its seasonal variations has also been performed at Chalmers University of Technology, (Olberg 1997). Parallel to this, an investigation of suitable field and laboratory test methods for investi-
2 GROUND WATER CONDITIONS.

The ground water conditions in a location with a low lying free ground water level may be divided into two main zones, the saturated and the unsaturated zones. The saturated zone consists of the ground water zone below the free ground water table and the capillary zone above the free ground water table in which the soil is fully saturated because of capillary forces. The location of the free ground water table may vary seasonally and the capillary zone with it. The unsaturated zone consists of an intermediate vadose zone where the soil is only partly saturated with a moderate seasonal variation and a soil water zone closest to the ground surface where the water conditions are heavily dependent on climatic conditions. In the soil water zone, the soil may thus become fully saturated during periods of thawing, snow melting and heavy precipitation and almost completely dry during periods of drought.

The pore water pressures below the free ground water level are positive and often more or less hydrostatic and in the capillary zone they are negative and inversely hydrostatic. In the intermediate zone, the pore water pressures are negative and the matrix suction is higher than the capillarity of the soil. The size of the matrix suction depends on the degree of water saturation and the water retention curve for the particular soil. In the soil water zone, the matrix suction may vary seasonally from its maximum value for the particular soil to zero depending on the climatic conditions. Evaporation and surface run off as well as the vegetation and its root system play important roles for how much the surface layers will soften up and how much of the surface water will infiltrate down to the ground water table. The loss in matrix suction during wet periods, in Sweden particularly during thawing and snow melting in the spring, may extend down to considerable depths, i.e. include the soil volumes affected by normal shallow foundations. On the other hand, the matrix suction may still remain intact enough to provide stability for the natural slopes, particularly with respect to deep potential slip surfaces.

In locations with the free ground water level closer to the ground surface, the conditions may vary very rapidly. The soil above the free ground water level is then almost fully saturated because of the capillarity. As a result, the location of the free ground water table will vary strongly and rapidly at precipitation and evaporation because of the very limited amounts of water required to be added or removed for this. Variations in several metres can thus occur between seasons and also during short periods of heavy rain or drought within a season.

3 EFFECT OF PORE PRESSURES AND MATRIX SUCTION ON THE SHEAR STRENGTH

The shear strength, $\tau$, of a saturated soil is normally expressed as

$$\tau = c' + (\sigma - u_s) \tan \phi'$$

(1)

where $c'$ = effective cohesion; $\sigma$ = total normal stress; $u_s$ = pore water pressure and $\phi'$ = effective friction angle.

This expression is thus valid for the soil in the saturated zone, i.e. the ground water zone below the free ground water level and in the capillary zone above it.

In the unsaturated zone, the pore water pressure acts over a reduced part of the potential shear surfaces, which has to be taken into account. This can be made in different ways. For example, Bishop (1955) proposed that the shear strength in unsaturated soils could be expressed as

$$\tau = c' + (\sigma - u_s - \gamma \alpha (u_s - u_w)) \tan \phi'$$

(2)

where $u_s$ = pore air pressure; $u_w$ = pore water pressure; and $\gamma$ = a parameter which varies with degree of saturation between 0 and 1 in a pattern depending on the soil type.

Fredlund et al. (1978) proposed the expression

$$\tau = c' + (\sigma - u_s) \tan \phi' - \gamma \alpha (u_s - u_w) \tan \phi'$$

(3)

where $\phi'$ = reduced friction angle in the strength component caused by the matrix suction ($u_s - u_w$).

The proposed expressions above require elaborate testing to determine the parameters $\gamma$ or $\phi'$. For rougher estimates, the formulae

$$\tau = c' + (\sigma - u_s) \tan \phi'$$

when $u_s \geq 0$ and

$$\tau = c' + \frac{u_w}{2} \tan \phi'$$

when $u_s < 0$$

(4)

were proposed in the Swedish guidelines for slope stability investigations (Commission on Slope Stability 1995). This roughly corresponds both to the values of $\phi'$ in relation to $\phi$ reported in literature and to the empirical correlation for the "false or apparent cohesion" caused by the matrix suction, ($= \gamma \alpha (u_s - u_w) \tan \phi'$), reported by Mjöslund (in Heleneck 1965), Fig. 1.
where $d$, $x$, $i$, $g$, and $b$ are correction factors for depth of embedment, shape of foundation, inclined load, inclined ground surface and inclined base respectively.

The effect of negative pore pressures, (matrix suction), on the bearing capacity can be very great. According to Fredlund and Rahardjo (1993), the effect of the matrix suction can be converted into an "apparent cohesion" which can be added to the cohesion component in the general bearing equation. However, Fredlund and Rahardjo also stress that it must be ascertained that the matrix suction taken into account will prevail throughout the life-time of the construction.

5 EFFECT OF MATRIX SUCTION ON RESULTS OF SOUNDINGS AND IN SITU TESTS

5.1 Soundings

The increased bearing capacity because of the matrix suction results in an increased resistance during penetration in sounding tests, both in terms of tip resistance and rod friction. This has been shown to be the case in particular for weight sounding tests, but also all other tests are affected. Results from dynamic probing tests have previously been shown to be affected by excessive rod friction due to negative pore pressures developed at driving of the probe, and the generated pore pressures during driving probably largely mask the effects of possible matrix suction in the surrounding soil. Furthermore, the results from dynamic probing tests should be corrected for the rod friction estimated by rotation of the rod system at regular depth intervals.

Also the results of CPT tests are affected by a matrix suction. In this case, the in situ pore pressure is a parameter which is normally used extensively in the interpretation of the test, either by itself or for calculation of the in situ effective vertical stress. It is thus possible to correct for the matrix suction, provided that it has been measured.

As an example, the results from comparative investigations in a profile with alternating layers of medium stiff silty clay and sandy silt/silty sand are shown in Fig. 2. In this profile, the free ground water level was located at 12 m depth. Above this level, the soil was fully saturated up to the approximately 1 m thick crust and the pore pressures were negative and inversely hydrostatic.

The results from weight sounding tests indicated a very stiff soil without any greater variations and the soundings were terminated at 15 m depth after the number of half-turns required for 0.2 m penetration had remained between 200 and 400 from 5 metres depth. A preliminary examination of remoulded
samples taken by a screw auger indicated that the soil consisted mainly of silt.

Dynamic probing tests yielded a very different picture of the stiffness of the soil. According to the results of these tests, the soil, when assumed to be silt, is very loose down to 6 m depth, loose between 7 and 8 m depth, dense between 8 and 10 m depth, and very dense between 10 and 11 m depth. The soil is then mainly medium dense down to great depths.

CPT-tests indicated that the soil in the profile consists of medium stiff clay down to 6.5 m depth. There is then a layer of medium dense sand about 1.5 m thick, followed by about 1 metre of medium stiff clay, 1 metre of medium dense sand and 1 metre of alternating layers of sand and clay. Thereafter, the CPT tests registered sand, which at 11 m depth becomes so dense that the test could not be advanced further with the equipment employed. Continuous sampling has shown that the classification based on the CPT tests in principle is correct but has also provided more detailed information indicating, for example, that the clayey parts consist of clay and silty clay, that the sand is a silty fine sand and that there is sandy silt in the transitions between the main layers.

The good classification from the CPT tests was obtained only when the matrix suction was taken into account. Unless this was made, the entire soil profile above 8 m depth would have been classified as alternating layers of silt and sand, the overestimation of the coarseness of the soil increasing towards the ground surface. The employed classification system mainly uses net cone resistance normalised versus effective vertical stress but also other classification systems using, for example, pore pressure parameters would have yielded similar results. In clays, the classification is often made using the generated excess pore pressure as a parameter. Unless the matrix suction is considered, this parameter becomes too small and the results become erroneous. The omission of taking the matrix suction into account in the evaluation may also bring errors in the estimation of a number of parameters in the soil, such as overestimation of friction angles in silty fine sands.

5.2 In situ tests

In dilatometer tests, the parameters material index, I_O, and horizontal stress index, K_H, evaluated from the test results are both sensitive to the in situ pore pressure. A neglect of the matrix suction results in an overestimation of the coarseness of the soil since I_O becomes underestimated. The overconsolidation ratio OCR, the coefficient of earth pressure K_0, and the moduli, E or M, also become underestimated because K_0 becomes overestimated. Depending on the evaluation method employed, also the shear strength parameters may be affected.

The results of pressuremeter tests are also affected. The results interpreted according to the Menard procedure are mainly affected in that way that the limit pressure corresponding to a state of failure in the soil becomes higher the higher the matrix suction is. This in turn entails a higher calculated bearing capacity and may also affect the rheological parameters used in the estimation of settlements.

![Weight sounding test](image1.png)

**Fig. 2. Results from soundings in a profile with matrix suction in the soil.**

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In other types of interpretation of pressuremeter test results, particularly the stress-strain relations at higher pressures where the effect of the matrix suction is taken into account are affected. A precondition for the results to be relevant for estimation of the behaviour of constructions is that the matrix suction in the soil at the time for the test will prevail throughout the lifetime of the construction. Otherwise, the results would have to be corrected for the influence of the matrix suction, for which there is no established procedure.

Similarly, the results of all plate loading tests and screw plate tests are affected by the matrix suction existing during the tests and the relevance of the results for constructions depends on the durability of the matrix suction. In this case, the matrix suction can be measured or otherwise estimated and the test results in terms of measured bearing capacities may be corrected accordingly.

6 EXAMPLE OF RESULTS FROM FIELD INVESTIGATIONS AND LOAD TESTS IN A SILTY SOIL.

As an example, the results from a series of investigations and load tests at Vyhamn is in central Sweden can be studied. The soil profile in the area consists of layered medium dense silt to a depth of about 25 m, where coarse and hard bottom layers are encountered. The area is flat but a small river meanders through in a deep ravine. The closest distance to the river from the test area is about 350 m and the water level in the river is there about 20 m below the ground surface. In the test area, the free ground water level is found at about 18 m below the ground surface. Above this level, the pore pressures are negative and approximately inversely hydrostatic in a 4 to 5 m thick water saturated zone and the matrix suction above is fairly constant between 40 and 50 kPa.

Weight sounding tests indicated a very stiff soil. Both CPT and diltometer tests provided good classifications provided the matrix suction was taken into account. Otherwise, the soil in the upper part of the profile was classified as coarser than the actual in both types of tests. Pressuremeter tests were performed according to the Menard procedure and evaluated both according to Menard (Baguelin et al. 1978) and a new procedure proposed by Briaud (1995).

A series of plate load tests was then performed. The estimated bearing capacities using different methods varied much, and the calculations using the general bearing capacity equation showed that the matrix suction played a very great role for the estimated bearing capacity. In the load tests, it was not possible to obtain a real failure for any of the plates, but the results indicate that the calculations using the general bearing capacity equation and full matrix suction provided results relevant for the conditions during the tests. As an example, the calculations for the smallest plate measuring 0.5 x 0.5 m are shown in Fig. 3. The load test was in this case terminated at a load of 450 kN when the plate had settled about 125 mm and started to tilt. The calculated failure load using the general bearing capacity equation ranged from about 60 kN at zero matrix suction to about 600 kN at full matrix suction. The very large effect of the matrix suction is partly due to the small dimensions of the plate and a small depth of embedment, but also in more ordinary cases the effects of a matrix suction are large. The bearing capacity calculated on the basis of pressuremeter tests according to the Menard procedure was about 300 kN. In addition, two new methods of estimating the curved load-settlement relation together with the failure criteria of settlement/plate width = 0.1 were employed; Briaud (1995) using pressuremeter test results and Larsson (1997) using modulus evaluated from diltometer tests or alternatively CPT or dynamic probing tests. These methods also yielded bearing capacities of about 300 kN.

Fig. 3. Calculated bearing capacity for the 0.5x0.5 m plate at Vyhamn as a function of matrix suction.

In order to investigate the possible variations in the matrix suction, a study of the effect of soaking the ground surface was then performed. In the study, the surface layer was scraped off over a fairly large area leaving a shallow but extensive pit, only a few decimetres deep. The area was then sprinkled with water with the intention of simulating a very heavy precipitation. This resulted in the pit being filled up, whereupon the sprinkling was stopped. The reaction
in the pore pressure in the ground was measured in piezometers installed at various depths below the ground surface. During the first day, the pore pressure reacted down to a depth of 3 m. During the following two to three days, the effects spread downwards resulting in a total loss of matrix suction down to between 2 and 3 metres and then gradually diminished to become negligible below 6 m depth. The matrix suction then gradually returned and had fully recovered to the normal values after about 2 weeks, Fig 4.

![Pore water pressure graph](image)

**Fig. 4. Variation of pore pressures during the soaking test at Vathhammer**

How representative this soaking test is for the worst case of natural soaking is difficult to judge. A large but still limited amount of water was used. The test area was fairly large and the surface run off was very limited. Nevertheless, it may be assumed that part of the limitation in affected depth is due to a horizontal water flow in the layered soil which would have been less pronounced if a larger area had been involved. A conclusion is that, for foundations in this area, it would be prudent to disregard any effects of matrix suction in the soil unless it is ascertained that reliable measures have been taken to make the negative pore pressures prevail.

7 CONCLUSIONS

The pore pressure conditions in silt deposits are often complex and have to be examined carefully. At low lying free ground water tables, there is often a considerable matrix suction in the soil above, which greatly affects the results of both soundings and in situ tests and also the actual bearing capacity of shallow foundations. The pore pressure conditions have to be investigated in detail and a possible matrix suction at the time for the site investigations has to be measured. The relevance of the pore pressure conditions measured at this time for long term conditions then has to be evaluated from a prognosis of possible variations during the life time of the structure in consideration.

In the site investigations, methods should be selected in which the influence of the matrix suction on the test results can be accounted for.

In design, great care should be taken not to utilise properties of the soil which cannot be relied upon in the long term.

8 REFERENCES


Estimations of geotechnical properties from piezocone penetration tests in Korea

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Department of Civil Engineering, College of Engineering, Seoul National University, Korea

ABSTRACT: From 41-CPTu in South Korea, the piezocone factors ($N_{fp}$, $N_{dp}$, and $N_{pp}$) to estimate the undrained shear strength of clayey soil are determined. From pore pressure dissipation tests coefficients of consolidation $e_0$ are calculated using the analytical methods and compared with the results obtained by laboratory tests. Based on Unified Soil Classification System, a new soil classification chart for South Korea using $B_f$ and $B_p$ is proposed.

1 INTRODUCTION

Site characterization is one of the most important steps in projects solving geotechnical problems. Piezocone penetration test (CPTu) is a fast, economic and accurate method of site characterization. It is now being used most popularly all over the world to evaluate soil profiles and geotechnical properties of soils.

Many theoretical methods of analyzing the CPTu results have been proposed. However, it is very difficult to analyze the behavior of soils adjacent to the penetrating cone due to the complex stress and pore pressure variation during cone penetration. In order to overcome these difficulties many researchers proposed empirical correlations between CPTu results and geotechnical properties with local confidence. Unfortunately, there has not been such correlations to be used for geotechnical projects in South Korea.

CPTu using a piezocone whose porous element is located behind the tip are carried out, and compared with some in situ tests (e.g. field vane test and standard penetration test) as well as laboratory tests (e.g. triaxial compression test and oedometer test). From the results of these tests piezocone factors ($N_{fp}$, $N_{dp}$, and $N_{pp}$) to estimate undrained strength of clayey soil are obtained. Performing pore pressure dissipation tests, coefficients of consolidation in the horizontal direction are calculated using some analytical methods and compared with one another regarding laboratory results as a reference value. Based on Unified Soil Classification System, a new soil classification chart using friction ratio $B_f$ and pore pressure parameter $B_p$ is developed.

2 TEST SITES

41-CPTu works are carried out in 12 sites in South Korea. The locations of each site are shown in Figure 1.

Many sites are deposited with normally consolidated or slightly overconsolidated clay from the ground surface. Pyeongtaek and Youngsan are exceptionally deposited with overconsolidated clay of mean OCR being 3.6 and 2.9, respectively. Ilsan, Changwon and Gangneung consist of sandy silt to silty sand or sand.

Figure 1. Location of CPTu works

3 PIEZOCONE FACTORS

$B_f(1)$ shown below is commonly used to estimate the undrained strength of clayey soil from CPTu (Lunne et al., 1985).

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where $s_u$ = undrained strength; $q_T$ = corrected cone tip resistance; $\sigma_{ov}$ = total overburden stress; and $N_{FT}$ = piezocene factor. Besides $q_T$, pore water pressure $u_w$ is also used to estimate the undrained strength of clayey soil with the following Eq.(2)(Vesic, 1972, 1975) and Eq.(3) (Canpanella et al., 1982) shown below.

$$s_u = \frac{q_T - \sigma_{ov}}{N_{FT}}$$  \hspace{1cm} (1)  \\

$$s_u = \frac{u_w - \sigma_{ov}}{N_{SW}}$$  \hspace{1cm} (2)  \\

$$s_u = \frac{q_T - u_w}{N_{gw}}$$  \hspace{1cm} (3)

where $u_w$ = pore water pressure measured behind the cone tip; $\sigma_{ov}$ = hydrostatic water pressure; $N_{SW}$ and $N_{gw}$ = piezocene factor.

The representative CPTu results and $s_u$ profiles by field vane tests and triaxial compression tests are shown in Figure 2. To determine piezocene factors for different OCR and $q_T$ sites are divided into three. One is Pyeongtak site which has a mean OCR = 1.6 and a mean $q_T = 22\%$, another is Youngam which has a mean OCR and $q_T$ of 2.9 and 31\%, respectively. The others (e.g., Yangan, Hudong, Kwangyang, Seochon, Incheon, Asan, Yongsan, Silcheong, Gangneung) have mean OCR = 1–2 and $q_T = 20$–26\%.

Assuming the distribution of CPTu results vs. undrained shear strength to be normal distribution, the mean piezocene factors and the ranges of 99% reliability are shown in Table 1.

As shown in Table 1, the value of $N_{FT}$ and $N_{gw}$ increases and $N_{SW}$ decreases as OCR increases. And $N_{SW}$, $N_{Sw}$, and $N_{gw}$ have no special relations with $q_T$. Comparing with those of other regions, $N_{FT}$ is similar to Jamiolkowski et al., 1982; Doolib, 1988; Jones, 1995; Tonpala, 1995; Wang, 1995; etc., but $N_{SW}$ is about 0.5 times smaller and $N_{gw}$ is 2–3 times greater than the values by Kourad et al., 1985; Ohtsuka, 1993; Chen & Mayne, 1993.

Many researchers analyzed the stress and pore pressure distribution adjacent to penetrating cone. $N_{FT}$ and $N_{SW}$ are mainly function of rigidity index which is utilized in the derivation of the bearing capacity and cavity expansion theory. And $N_{gw}$ is a function of effective friction angle and plastic volumetric strain ratio which are utilized in the cavity expansion theory combined with modified Corn Clay model (Chen & Mayne, 1993). In Table 2, piezocene factors determined from theoretical approaches are shown. The rigidity index in each site is calculated from stress-strain curves of triaxial compression tests assuming $E_T = E_{T} = E_{T} = E_{T} = E_{T}$, where $E_{T}$ = tangential modulus and $E_{T}$ = secant modulus at 50% of peak strength. The rigidity indices are obtained to be 100–150 in Pyeongtak, 120–175 in Youngam, and 40–100 in the other sites. For Teb & Houlsby’s solution in Table 2, it is assumed that shear stress at the cone face($\gamma$) and at the friction sleeve($\alpha$) is equal to $\gamma_u$ and $\alpha_u = 1$, which results in $\gamma = \alpha_u - 0.87$ and $\Delta = (\sigma_{ov} - \gamma_{ov})/2\gamma_{ov} = 0$.

Figure 3 shows the comparison of empirically determined piezocene factors with theoretically determined factors. Theoretical values of $N_{FT}$ are similar to empirical values of $N_{FT}$ determined in Youngam and in the other sites except that by Baligh (1975). Theoretical values of $N_{FT}$ determined in Pyeongtak

![Table 1: Piezocene factors in each site](image)

![Figure 2: CPTu results and $s_u$ profiles](image)

![Figure 2: CPTu results and $s_u$ profiles (continued)](image)
### Table 2: Pleocrocic factors determined by theoretical approaches

<table>
<thead>
<tr>
<th>Pleocrocic factor</th>
<th>Pyongtack</th>
<th>Yangsan</th>
<th>The Others</th>
<th>Reference</th>
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<tr>
<td>$N_{el}$</td>
<td></td>
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<tr>
<td>7.00-9.94</td>
<td>7.00-9.94</td>
<td></td>
<td></td>
<td>Terzaghi(1943), Czerniak &amp; Kierke(1956), Meyerhoff(1951), De Brey(1977)</td>
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<td>$\frac{1}{\sqrt[3]{1 + \frac{K_{cl}}{K_{pl}}}}$</td>
<td>8.5-9.0</td>
<td>8.0-8.8</td>
<td>7.3-8.5</td>
<td>Meyerhoff(1951)</td>
</tr>
<tr>
<td>$\frac{1}{\sqrt[3]{1 + \frac{K_{cl}}{K_{pl}}}}$</td>
<td>8.5-9.0</td>
<td>8.0-8.8</td>
<td>7.3-8.5</td>
<td>Skempston(1951)</td>
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<td>8.7-9.5</td>
<td>8.0-9.2</td>
<td>Gibson(1955)</td>
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<tr>
<td>$\frac{1}{\sqrt[3]{1 + \frac{K_{cl}}{K_{pl}}}}$</td>
<td>9.0-7.7</td>
<td>8.7-9.5</td>
<td>8.0-9.2</td>
<td>Gibson(1955)</td>
</tr>
<tr>
<td>$\frac{1}{\sqrt[3]{1 + \frac{K_{cl}}{K_{pl}}}}$</td>
<td>7.5-8.0</td>
<td>7.0-7.8</td>
<td>6.3-7.5</td>
<td>Vesic(1972)</td>
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<td>$\frac{1}{\sqrt[3]{1 + \frac{K_{cl}}{K_{pl}}}}$</td>
<td>10.1-10.6</td>
<td>9.6-10.4</td>
<td>8.9-10.1</td>
<td>Vesic(1975)</td>
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<td>$\frac{1}{\sqrt[3]{1 + \frac{K_{cl}}{K_{pl}}}}$</td>
<td>16.6-17.0</td>
<td>16.2-16.8</td>
<td>15.7-16.6</td>
<td>Baligh(1975)</td>
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<tr>
<td>$\frac{1}{\sqrt[3]{1 + \frac{K_{cl}}{K_{pl}}}}$</td>
<td>11.3-11.9</td>
<td>10.7-11.7</td>
<td>9.8-11.3</td>
<td>Teh &amp; Hoehfly(1991)</td>
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<td>$N_{w}$</td>
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<tr>
<td>5.0-5.5</td>
<td>6.0-6.8</td>
<td>7.0-7.5</td>
<td></td>
<td>Vesic(1972, 1975)</td>
</tr>
<tr>
<td>$N_{w}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.0-5.5</td>
<td>5.8-6.5</td>
<td>6.1-6.8</td>
<td></td>
<td>Chen &amp; Moynier(1999)</td>
</tr>
<tr>
<td>$\frac{3}{10} \times 2.9^*$</td>
<td>6.6</td>
<td>6.5-6.8</td>
<td>6.6-6.9</td>
<td>Chen &amp; Moynier(1999)</td>
</tr>
</tbody>
</table>

* $H = 6 \sin \phi / (1 - \sin \phi), w = (3 - \sin \phi) / (6 - 3 \sin \phi), \delta = \sin \phi \cos \phi / H^2, A = 1 - 2 \delta$ 

![Graph](image)

**Figure 2:** Comparisons of empirical pleocrocic factors with those theoretically determined.
are 1/3–1/2 times smaller than empirical value of $N_{p}$. Theoretical values of $N_{m}$ and $N_{m}$ (using typical value of $A=0.75$ as suggested by Chen & Mayne, 1993) are totally different from empirical values.

4 COEFFICIENT OF CONSOLIDATION

By stopping cone penetration and monitoring the decay of excess pore pressure with time, approximate coefficient of consolidation in the horizontal direction $c_{c,HP}$ can be obtained.

To compare $c_{c,HP}$ calculated by analytical method (Torsenstam, 1977; Baligh & Levadoux, 1980; Levadoux & Baligh, 1986; Teh & Houlsby, 1991) with $c_{c,HP}$, pore pressure dissipation tests are carried out in Seocheon, Youngnam and Kwangyang where relatively homogeneous thick clayey soils are deposited. Reference values of $c_{c,HP}$ are determined from standard oedometer tests.

![Diagram of Seocheon I and II](image)

![Diagram of Kwangyang and Youngnam](image)

Figure 4. Comparisons of the values of $c_{c,HP}$ at a degree of consolidation 40%, 50%, and 60% with the value of $c_{c,HP}$ estimated from oedometer tests ($c_{c,HP}$ is $cm^2/sec < 10^5$).

Generally, the value of $c_{c}$ is greater than the value of $c_{c}$, because of a soil anisotropy, but their quantitative difference is not exactly known. The quantitative difference of the coefficient of permeability between in the horizontal direction ($k_{h}$) and in the vertical direction ($k_{v}$) is investigated from various insitu and laboratory permeability tests by Lee and others (1977). They reported that the value of $k_{h}$ is 2–3 times greater than the value of $k_{v}$ when void ratio is constant. Therefore, $c_{c}/k_{c}$ ratio is assumed to have a value of $2–3$ regarding the soil compressibility to be isotropic (Parry & Wroth, 1977). This leads to a mean value of $c_{c,HP}$ to $3.8–5.6 cm^2/sec < 10^5$ in Seocheon I site, $0.7–2.5 cm^2/sec < 10^5$ in Seocheon II site, $0.7–1.1 cm^2/sec < 10^5$ in Kwangyang site, and $0.7–1.1 cm^2/sec < 10^5$ in Youngnam site.

The comparisons of the values of $c_{c,HP}$ at the degree of consolidation, 40%, 50%, and 60% with the value of $c_{c,HP}$ estimated from oedometer tests are shown in Figure 4.

As shown in Figure 4, the value of $c_{c,HP}$ (stitched area) estimated from oedometer test lies between A (Torsenstam’s solution-spherical cavity expansion) and B (Torsenstam’s solution-cylindrical cavity expansion) or C (Teh & Houlsby’s solution-strain path method) except in Seocheon I site. So, it is conceived that the value of $c_{c,HP}$ from Torsenstam’s solution is most useful among the analytical solutions for South Korea.

5 SOIL CLASSIFICATION

Several charts exist for soil stratigraphic profiling using friction ratio (Schuetzmann, 1978; Douglas & Olsen, 1981; Robertson & Campanella, 1983; Campanella & Robertson, 1988) and pore pressure parameter (Jones & Rust, 1982; Sennett & Jamiolkowski, 1985; Campanella & Robertson, 1988). However, those charts should be used with local confidence and only as a guide for soil classification.

Figure 5 and Figure 6 show the distribution of friction ratio $R_f = (g_0/g_1 < 100%)$ vs. $A_f$ and pore pressure parameter $R_p = (g_0-g_1)/(g_1-g_0)$ vs. $A_f$, respectively, based on Unified Soil Classification System (USCS).

In clay (Figure 5(ab), b) and Figure 6(ab), b) with $1<OCR<4$, $q_r$ has the value less than $30 kN/m^2$ and $R_p$ decreases (this is contrary to existing classification charts) and $R_p$ increases as OCR decreases. And $CH$ has lower value of $R_f$ and higher value of $R_p$ than $CL$. Particularly, it is possible for $R_p$ to have a value less than zero. In silts (Figure 5(c) and Figure 6(c)), the portion of silt is almost overlapped by clay (CL). And $R_p$ decreases and $R_p$ increases in $I_p$ decreases. In sand (Figure 5(d), (e), (f)) and Figure 6(d), (e), (f)) there is no outstanding difference between SM and SP. SW has slightly higher values of $q_r$ than SM and SP. And $q_r$ has a higher value as $D_p$ increases and as fines contents decreases. Particularly, $R_p > 0$ in case of $q_r > 50 kN/m^2$.

Figure 7 shows the preliminary classification chart based on USCS symbol. $A$ is a region of clay (CH, CL) of OCR<4, in which $R_p$ decreases and $q_r$ increases as OCR increases. $B$ is predominantly a region of CL and MH, and ML partly exists. C is a...
Figure 5. Distribution of $q_f$ vs. $R_f$ of soil based on USCS

Figure 6. Distribution of $q_f$ vs. $R_f$ of soil based on USCS

Figure 7. Preliminary classification chart based on USCS symbol using $R_f$ and $R_i$
region of various soils (e.g. CL (3<OCR<4), ML, SM, and SP). Much care should be taken in region C and additional investigation for soil classification is needed in this region. D and E are regions of sand (SM, SP predominant in D and SW predominant in E). At fines contents decreases and OCR increases, qT increases, but βT and βE slightly decrease in these region. Figure 8 shows Dongbu soil classification chart which is newly proposed in this study with local confidence.

6 CONCLUSIONS

The piezocone factors N<sub>f</sub>, N<sub>30</sub>, and N<sub>p</sub> to estimate the undrained shear strength of clayey soil are determined in South Korea. The values of N<sub>f</sub> from some theoretical approaches agree well with the empirical value of N<sub>f</sub> for clay of OCR<3. Theoretically determined values of N<sub>30</sub> and N<sub>p</sub> show some discrepancy with the empirical results.

The horizontal coefficient of consolidation estimated from pore pressure dissipation tests agrees best in the case of Torsvendson’s solution with the horizontal coefficients of consolidation estimated from laboratory consolidation tests.

A soil classification chart using friction ratio and pore pressure parameter is developed and proposed for the use of it in South Korea with local confidence.

ACKNOWLEDGMENT

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Direct shear tests between diatomaceous mudstone and friction sleeve materials with different surface roughness

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T. Igarashi
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ABSTRACT: Peck cone penetration tests (CPTs) were conducted in a diatomaceous mudstone on the Noto Peninsula, Japan, where a series of static load tests (SLTs) of instrumented piles have been carried out, to explore a method for estimating the pile shaft friction, $f_s$, from CPT data, especially from the sleeve friction, $f_{sl}$. In the CPTs, various surface roughness of the friction sleeve ranging from 2 $\mu$m to 50 $\mu$m was used to investigate the influence of the surface roughness on the sleeve friction, $f_{sl}$ (Takesue, Matsumoto & Sasso 1998).

In this paper, a series of direct shear tests (DSTs) between the diatomaceous mudstone and steel plates having different surface roughness are conducted to provide the interface properties which will be a help to investigate correlation between the pile skin friction and the CPT sleeve friction.

1 INTRODUCTION

A new highway route is under construction on the Noto Peninsula, Japan. Several highway bridges have been constructed for the highway route. Among them, the bearing strata of foundation piles for Noetsu bridges No. 3 and No. 4 are a diatomaceous mudstone. A series of load tests on open ended steel pipe piles were carried out in 1990, in association with pile design (Matsumoto, Michi & Hirano 1995). In this test piling, load tests as well as laboratory soil tests such as unconfined compression tests, oedometer tests and triaxial tests, and various in-situ tests such as Standard penetration tests (SPTs), borehole lateral load tests (LLTs) and cone penetration tests (CPTs) were conducted.

Additionally, extra CPTs were conducted in 1996 and 1997 at the test site. In the CPTs, various cone penetration rate and surface roughness of the friction sleeve were used so that the cone penetration rate covered the pile penetration rates in the static load test (SLTs) from a very low rate to a high plunging rate, and the surface roughness covered the usual roughness of the sleeve friction of 2 $\mu$m to the roughness of the test piles of some 10 $\mu$m. It was observed that the sleeve friction, $f_{sl}$, tended to increase with increase in surface roughness and level off at a given surface roughness (Takesue, Matsumoto & Sasso 1998).

In this paper, a series of direct shear tests (DSTs) between the diatomaceous mudstone and steel plates having different surface roughness are conducted to provide the interface properties which would be a help for interpretation of CPT data to estimate the pile shaft friction.

2 RELATION BETWEEN CPT SLEEVE FRICTION AND PILE SHAFT FRICTION

2.1 Test site and test pile

Three test open-ended steel pipe piles, designated as $T_1$, $T_2$ and $T_3$, were driven in a limited area of $9m \times 9m$ (Fig.1) with a diesel hammer having a rated energy of 108 kN·m. Geometrical and mechanical properties of the test piles are listed in Table 1. Foil strain gages were mounted on the inner surface of each test pile at a ratio of $\frac{1}{5}$ levels. Steel channels welded inside the piles increased the net cross-sectional area, $A$, of the piles to 0.041 m².
Four electronic pore-pressure transducers, designated as A, B, C and D, were embedded in boreholes B1 through B4 at a depth of 4.5 m from the top level of the diatomaceous mudstone (Fig. 2) prior to pile driving to measure pore pressure responses around pile T1 during and after pile driving.

The test site is featured as a thick deposit of the diatomaceous mudstone below the top soft clay of 1.5 m thick (Fig. 1). Notice that the top soft clay was removed prior to pile driving so that the test piles were embedded in the diatomaceous mudstone only.

Figure 3 shows the blow count, N, from SPTs and the CPT results such as the tip resistance q_t, the sleeve friction f_s and pore pressure u. The CPT was conducted just near the location of pile T1. The piezometer is illustrated in Fig. 4. The maximum surface height (Japanese Standards Association 1982), R_max, of the friction sleeve was 2 mm. This friction sleeve is called "standard sleeve" hereafter. The piezometer was pushed into the ground at a penetration rate of 20 mm/s without interruption of penetration by means of a double jacking reaction device developed by Kajima Tech. Research Institute.

It is seen from Fig. 5 that the diatomaceous mudstone in the test site is relatively uniform to a depth of 14 m. Natural water content, w_n, wet density, \rho, and unconfined compression strength, q_u, also indicated the uniformity of the test site: w_n = 113 - 140 \%, \rho = 1.27 - 1.40 \text{t/m}^3 and q_u = 0.6 to 1.0 MN/m^2.

### 2.2 Pile shaft friction vs. CPT sleeve friction

Static load tests on piles T1 and T2 were conducted.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
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<td>Length</td>
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<td>Outer diameter ( \bar{D}_o )</td>
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<tr>
<td>Inner diameter ( \bar{D}_i )</td>
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<tr>
<td>Wall thickness ( t_w )</td>
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<tr>
<td>Cross-sectional area ( A )</td>
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</tr>
<tr>
<td>Young's modulus ( E )</td>
<td>2.06 \times 10^3</td>
</tr>
</tbody>
</table>

* including cross-sectional area of steel channels

---

Figure 3 shows the results of standard penetration tests and cone penetration test.
29 days and 11 days after pile driving. The soil inside pile $T_2$ was excavated to 0.5 m below the pile toe prior to the load test so that the contributions of the inner shaft friction and the toe resistance were reduced.

The load-displacement curves of pile $T_1$ and $T_2$ are shown in Fig. 5. Both curves are identical until the load, $P$, reaches 3 MN, implying that only the outer shaft friction was mobilized in pile $T_1$ until this load.

The axial force distributions of pile $T_1$ and $T_2$ at ultimate loads are shown in Fig. 6. The shaft resistance to a depth of 4 m from the ground level (7 m from the pile head) is smaller than that at shallower depths. This may be caused by lateral movements of upper portions of the piles by shear deformations due to inevitable eccentric impacts during pile driving. In reality, the gap between the pile and the soil was observed at the ground level, which may reduce the outer shaft resistance.

Decay of the axial forces with depth of pile $T_1$ and $T_2$ are similar, indicating that the contribution of the inner shaft resistance in pile $T_1$ is limited to a short pile section just above the pile toe.

Hence, the outer shaft resistance, $f_{ox}$ of pile $T_1$ and $T_2$ was estimated from the axial forces of the pile section along $x = 7$ m to 10 m, and is compared with the CPT sleeve friction, $f_s$, in Fig. 7. The pile shaft friction, $f_{ox}$, is approximated as $f_{ox} = 3f_s$

Excess pore pressures, $\Delta u$, measured at points B, C and D during pile driving of pile $T_1$ are shown in Fig. 8. The pore pressures sharply increased when the pile toe reached the depth of the piezometers, and then decreased with the progress of pile penetration. Negative pore pressures were generated at the end of pile driving at point C and D, which were located relatively far from the pile. At point B, positive pore pressure was generated at the end of pile driving and dissipated gradually to a hydrostatic pressure with time after the end of driving.

This pore pressure behavior suggests that the effective lateral stress on the pile surface at the static load test increased compared to that during pile driving. It is also inferred that the effective lateral stress on the pile surface at the static load is much larger than that on the CPT friction sleeve, since relatively large pore pressure is generated around the piezometer (see Fig. 4). This is thought to be one of reasons for $f_{ox} = 3f_s$.

Another possible reason for $f_{ox} = 3f_s$ is different surface roughness of the piles and the CPT friction sleeve. Takeda et al. (1998) conducted two series
of the CPTs using special friction sleeves with lathed around their surfaces having different maximum height, $R_{max}$. Figure 9 is an example of measured surface roughness of a friction sleeve with $R_{max} = 30 \mu m$.

Figure 10 shows the variation of sleeve friction, $f_s$, and $f/t_s$ with $R_{max}$ of the friction sleeve. Here, the subscript 'a' indicates that measurement was obtained by the special friction sleeve. The $f_s$ is the sleeve friction measured using the standard sleeve with $R_{max} = 2 \mu m$. It is seen that $f_s$ and $f/t_s$ increases with increasing $R_{max}$ until $R_{max}$ reaches about 15 mm and then tends to level off for larger $R_{max}$. $R_{max} \sim 15 \mu m$ seems to correspond to the "critical roughness" proposed by Tsuchida et al. (1993), where the interface slip mode changes to the soil shear failure mode.

A disadvantage of the CPT is that lateral stress on the friction sleeve is not measurable. The influence of the surface roughness of the friction sleeve seems to be affected by the interface frictional coefficient as well as the effective stress at the soil-sleeve interface.

3 DIRECT SHEAR TESTS BETWEEN DIATOMACEOUS MUDSTONE AND STEEL PLATES

In order to investigate the interface properties between the diatomaceous mudstone and steel (friction sleeve material and pile material) in detail, direct shear tests (DSTs) between the mudstone and steel plates were conducted.

3.1 Test apparatus

Figure 11 is the schematic view of direct shear test apparatus. Rigid specimen ring has a diameter of 60 mm. The steel plate is placed on the ring with the clearance of 0.005 mm between them. The steel plate is reacted by the reaction beam through ball-rollers to minimize the friction between the steel plate and the reaction beam. Normal stress, $c$, is applied to the soil specimen through the piston by the air cylinder.
The pore pressure transducer is embedded in the steel plate to measure pore pressure generated at the soil-steel interface. The direct shear device is put in the pressure cell. The pressure cell is filled with water and the air pressure, which is converted to water pressure, is used to apply back-pressure to the soil specimen. The horizontal force is applied to the steel plate by displacement-controlled reaction device.

Measurements were made for $\sigma$, shear stress, $\tau$, excess pore pressure, $\Delta p$, at the interface, vertical deformation, $D_v$, of the soil specimen, and horizontal displacement, $D_h$, of the steel plate.

3.2 Properties of diatomaceous mudstone and steel plates

The properties of the diatomaceous mudstone used in the DSTs are listed in Table 2. The cylindrical soil specimen, 60 mm in diameter and 20 mm in thickness, were put in vacuum water for more than 3 days prior to the tests.

Three different steel plates were prepared. Two plates were made of steel and their surfaces were lathed to have different surface roughness. The other plate was made from steel pile material whose surface were covered with a thin layer of mill scale.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Degree of saturation</td>
<td>$S = 100%$</td>
</tr>
<tr>
<td>Water content</td>
<td>$w = 42%$</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>$G = 2.19$</td>
</tr>
<tr>
<td>Wet density</td>
<td>$\rho = 1.29$ ton/m$^3$</td>
</tr>
<tr>
<td>Void ratio</td>
<td>$e = 3.13$</td>
</tr>
</tbody>
</table>

Figure 12 shows the measured surface roughness of the steel plates and maximum heights, $R_{max}$.

3.3 Test results

The soil specimens were consolidated under the normal stresses, $\sigma_n$, of 100, 300, and 500 kN/m$^2$ that
were less than the yield stress of 1200 kN/m². After the completion of consolidation, cyclic shearing was performed at a shearing (horizontal displacement, $D_h$) rate of 0.5 mm/s. This shearing rate was employed so that excess pore pressure generated at the soil-plate interface was negligibly small. A back-pressure of 100 kN/m² was applied throughout the tests.

Figure 13 is the test results of plate No.1 having $R_{pm}=5$ m with $\alpha=500$ kN/m². A total of 10 cycles of forward and backward shearing were conducted. In the 1st loading, the failure shear stress, $\tau_f$, is mobilized by $D_h$ of only 0.2 mm.

The increment of vertical deformation, $\Delta D_h$, of the soil specimen during the shearing process of 10 cycles is 0.1 mm that corresponds to 0.3 % axial strain.

Figures 14 and 15 are the test results of plates No.2 and No.3, respectively, with $\alpha=500$ kN/m². In these tests, $\Delta D_h$ were around 0.5 %, in the test of plate No.2, post-peak reduction of $\tau$ is observed clearly in the 1st loading (Fig.14). The post-peak reduction of $\tau$ tended to diminish with increasing number of shearing cycles. On the other hand, post-peak reduction of $r$ is not so evident in the test of plate No.3 that has $R_{pm}$ similar to plate No.2. It is inferred that not only $R_{pm}$ but also the surface configuration and the surface material affect the interface properties. Note that the surface material of plate No.3 is mild steel, as mentioned earlier.

Figure 16 is the change in the interface friction coefficient, $\mu = \frac{\tau_f}{\sigma_n}$, with increasing shearing cycles, $N$. Although increase in $\mu$ in the 2nd shearing cycle is evident, $\mu$ tends to converge after several shearing cycles.

Figures 17 and 18 are $\tau_f$ vs. $\sigma_n$ measured in the 1st loading and after several loading cycles where stationary $\mu$ is observed. It is seen that there is no cohesion and that the failure at each interface is governed by the effective normal stress.

The interface friction coefficients, $\mu$, obtained in the 1st loading are shown in Fig. 19. The $\mu$ of plate No.2 is about 15 % higher than plate No.1, suggesting that $R_{pm}$ is one of the major influential factors on the interface properties.

4 CONCLUSIONS

In attempts to explore methods for estimating the shaft resistance of steel pipe piles in the diatomaceous mudstone from the CPT data, direct shear tests between the diatomaceous mudstone and steel plates
were conducted. Although further experiments are required, the findings from this study are as follows:

1. Failure at the soil-steel interface is governed by the effective normal stress.
2. Surface material, surface configuration and cyclic shearing as well as $R_{nmax}$ are major influential factors on the distomaceous mudstone-steel interface properties.

REFERENCES

Clay stress history evaluated from seismic piezocone tests

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ABSTRACT: The preconsolidation stress \( \sigma'_{cp} \) of clays can be conveniently and expeditiously evaluated from the net cone resistance \( q_c - \sigma_m \) using the relationship: \( \sigma'_{cp} = k_p (q_c - \sigma_m) \), where a mean value of 0.3 is commonly adopted. In fact, the parameter \( k_p \) varies from 0.1 to 0.5 and decreases with the plasticity index (PI) of clay, or alternatively, with the in-situ void ratio (e). During cone penetration, neither the PI or e, are known beforehand. However, if a seismic piezocone is utilized, the independent measurement of shear wave velocity \( (V_s) \) can provide an estimate of \( k_p \), thus improving the assessment of \( \sigma'_{cp} \) immediately and while in the field.

1. INTRODUCTION

A direct link between the effective preconsolidation pressure or vertical yield stress \( \sigma'_{cp} \) and the net cone tip resistance \( q_c - \sigma_m \) has been well-established for a variety of soft to firm to stiff intact clays (e.g., Mayne & Holtz, 1988). The general expression is of the form:

\[
\sigma'_{cp} = k_p (q_c - \sigma_m) \quad (1)
\]

where the parameter \( k_p \) is a site-specific value for a particular clay. Commonly, a value \( k_p = 0.3 \) is used in practice. In all cases, the measured cone tip resistance \( q_c \) must be corrected for porewater pressure effects acting in unequal areas of the tip geometry to obtain \( q_t \). This requires porewater pressures be obtained at the cone shoulder, designated as \( u_t \) readings (Lunne, et al. 1986; Campanella & Robertson, 1988).

An alternate form uses normalized parameters, specifically stress history in terms of the overconsolidation ratio, OCR = \( \sigma'_{cp} / \sigma'_m \) and normalized cone tip resistance, \( Q = (q_t - \sigma_m) / \sigma'_m \) (Wroth, 1988; Robertson, 1990) which gives:

\[
OCR = k_p Q \quad (2)
\]

Other normalization schemes are possible (e.g., Crooks, et al. 1988), whereby the OCR and/or Q are defined in terms of horizontal stresses, or mean stress (1). However, these are not readily amenable to practical utilization because of the difficulty in assessing

<table>
<thead>
<tr>
<th>Type of Clay</th>
<th>( k_p )</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay till, UK</td>
<td>0.20</td>
<td>Powell et al. (1988)</td>
</tr>
<tr>
<td>Sensitive Eastern</td>
<td>0.28</td>
<td>Demers et al. (1994)</td>
</tr>
<tr>
<td>Canadian clays</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Swedish clays</td>
<td>0.29</td>
<td>Larsson &amp; Molabadić (1991)</td>
</tr>
<tr>
<td>Soft clay, UK</td>
<td>0.30</td>
<td>Powell et al. (1988)</td>
</tr>
<tr>
<td>U.S. clays</td>
<td>0.33</td>
<td>Mayne &amp; Kulhawy (1995)</td>
</tr>
<tr>
<td>Norwegian clays</td>
<td>0.34</td>
<td>Lunne et al. (1992)</td>
</tr>
<tr>
<td>Polish clays</td>
<td>0.30</td>
<td>Borowczyk &amp; Szymański (1995)</td>
</tr>
</tbody>
</table>

- ing the in-situ lateral stress coefficient \( k_p \).

Reference profiles of yield stresses in clays are obtained from their reconstructed geologic history and series of one-dimensional oedometer tests on samples taken from conventional soil borings. Data from adjacent but separate piezocone tests permit a site-specific calibration of \( k_p \) for each clay deposit. Table 1 lists some representative values for selected clays of differing geologic and geographic origin.

A fairly extensive collection of worldwide sources from 205 clay sites (Chen & Mayne, 1996) did indeed
confirm a first-order best-fit line:

\[ \sigma'_{y} = 0.305 (\gamma_s - \sigma_{w}) \]  \hspace{1cm} (3)

where the number of data \( n = 1256 \) and the coefficient of determination \( r^2 = 0.820 \).

Further studies have shown that while \( k_r = 0.3 \) provides a reasonable mean relationship, an improved accuracy is obtained if index properties are also known, specifically the liquid limit (Larsson & Malmqvist, 1991) or the plasticity index of the clay (Chen & Mayne, 1996). That is, the coefficient \( k_r \) appears to decrease with increasing LL and PI.

During cone penetration testing, neither the liquid limit nor plasticity index is known a priori. However, by use of a seismic piezocone (Robertson et al. 1986; Campauella 1994), the independent measurement of shear wave velocity \( (V_s) \) relates to the in-situ void ratio and plasticity of the soil (Mayne & Rix, 1995), thus serving to provide a finer detailing of the interrelationship between stress history and measured cone resistances while still on-site.

2. SEISMIC CONE DATABASE

To investigate an improved means of assessing \( \sigma'_{y} \) directly from CPT, a seismic piezocone database was compiled from sources involved with soft to firm clay sediments. Data from structured clays, cemented soils, and fissured materials were not included in this study, as these are important facets that should be evaluated by separate studies. Thus, the trends reported herein relate to soft normally- to lightly-overconsolidated intact clays \((1 < OCR < 2.5)\) without cementation and with shear wave velocities \( V_s < 400 \text{ m/s} \).

For brevity of presentation, the sources of data are grouped by country, including the United States (Sweeney & Kraemer 1992; Hegazy 1997), Canada (Roberts et al. 1986; Suly 1991), Sweden (Larsson & Malmqvist 1993); Norway (NGI 1985, 1987, 1988, 1989); Great Britain (Nash et al. 1992); Italy (Jamiolkowski et al. 1995), and Japan (Tanaka, et al. 1995). All results were obtained from piezocones with shoulder porewater pressure measurements (designated \( u_w \)) so that the proper corrected cone tip resistances \( (q_c) \) were analyzed. Procedures for conducting the piezocone tests are given in Lunne, Robertson, & Powell (1997).

A total of 262 data points from 26 different intact clays were contained in the database. The full range in plasticity index was 8 \( \leq PI \leq 152 \) percent and the in-place void ratios ranged 0.49 \( < e_i \leq 5.64 \). Laboratory consolidation tests gave reference values of OCRs between 1 and 2.38. The shear wave velocity profiles were obtained in a downhole manner using seismic cones, with measurements in the range 26 \( \leq V_s \leq 330 \) meters/sec. In the database spreadsheet, each entry has a matched set of values of preconsolidation stress, void ratio, plasticity index, corrected cone tip resistance, and shear wave velocity, thus providing a consistent set of results for comparative studies.

An initial assessment of the data provides the overall trend between \( \sigma'_{y} \) and corrected cone tip resistance \( (q_c - \sigma_{w}) \) as obtained from arithmetic regression analyses \((n = 262); r^2 = 0.868\):

\[ \sigma'_{y} = 0.28 (q_c - \sigma_{w}) \]  \hspace{1cm} (4)

This mean relationship is presented in Figure 1 and shows on logarithmic scales to detail the large variations in preconsolidation stress which vary from 20 \( \leq \sigma'_{y} \leq 800 \text{ kPa} \) for all sites. It may be concluded that the observed trend for the seismic cone database is consistent with prior statistical results, and therefore, the compilation represents a good population sample.

The quality of the database is difficult to assess since the results come from a variety of different sources. Of particular interest are the Japanese series (4 sites) from Tanaka et al. (1995) and Swedish series (9 sites) from Larsson & Malmqvist (1995), since they were obtained using consistent equipment and personnel and each series was reported as a single research program. All sources of data were professional firms with high standards of practice, yet
differing equipment was used for laboratory and field testing. Penetrometers from different manufacturers provide similar yet not exactly the same measurements of tip resistance and porewater pressures (Lumme et al. 1997). In particular, the sleeve friction readings can be significantly different amongst different penetrometers (Lumme et al. 1986; Tanaka, 1995).

Uncertainties exist in the reference values of the preconsolidation stresses and OCRs and these should be recognized since sample disturbance, natural soil variability, and test methods affect the results. The preconsolidation stresses of these clays were obtained by both incremental-load (IL) oedometers and/or by constant-rate-of-strain (CRS) consolidation tests. These do give differing results for the same clay, however, only global trends were under investigation here and therefore the values should be of relatively comparable magnitudes. In general, the preconsolidation stresses from Scandinavia and Japanese sites were obtained by testing in CRS apparatuses, the Rotations data from the U.K. obtained from both CRS and IL devices, while the remaining sites utilized IL oedometers for the most part.

The effect of plasticity on the parameter \( K_p \) is shown in Figure 2, with \( K_p \) decreasing from 0.5 to 0.1 as the PI increases from about 10 to 150. Specifically, two data sets offer a closer examination of this trend: (a) four Japanese sites \((n = 49; r^2 = 0.466)\) and (b) nine Swedish sites \((n = 113; r^2 = 0.713)\). Within each series, the sites have similar geologic origins and have been tested with the same equipment.

A similar trend of \( K_p \) decreasing with increase void ratio \( \varepsilon \) is also evident (Figure 3). Again, the Japanese series \((n = 49; r^2 = 0.489)\) and Swedish data \((n = 113; r^2 = 0.713)\) provide sufficient quantities to define specific trends for each country. The separate trends may occur because of the mineralogical differences, notably that Japanese clays contain a high percentage of "rock flour" and Swedish clays commonly contain a significant organics fraction.

3. STRESS HISTORY FROM SEISMIC CONES

The seismic piezocene is a valuable and versatile tool for purposes of site characterization, stratigraphic profiling, and the evaluation of soil properties. In addition to providing readings taken at failure-strain levels (i.e., \( q_u, \sigma_c, \) and \( \varepsilon \)), the seismic cone test (SCT) obtains nondestructive measurements of shear wave velocity \( V_s \), corresponding to the small-strain elastic behavior region (Roberts, et al. 1986).

During cone penetration testing, the profiles of PI and/or \( K_p \) are not normally known beforehand. Thus, while the trends from Figures 2 or 3 might be useful for obtaining a more accurate assessment of \( K_p \) for profiling the in-situ OCR of a particular clay deposit, the trends are not truly amenable to practical implementation, especially while in the field.

The shear wave velocity of soils is highly dependent on the in-situ void ratio and effective confining stress level (Jamieson and LoPresti, 1994). Consequently, the direct measurement of \( V_s \) by SCPT could serve as a surrogate measurement for \( K_p \) (Burns and Mayne, 1996) and therefore used to provide improved estimates of \( K_p \) during field operations.

![Figure 2. Observed trend of \( K_p \) with PI.](image1)

![Figure 3. Observed trend of \( K_p \) with void ratio.](image2)
For the SCPT database, Figure 4 shows the trend of $k_p$ increasing with $V_s$. Extensive field and laboratory studies on clays have shown, however, that $V_s$ is also strongly influenced by the effective confining stress level, as well as the direction of wave propagation and wave polarization (e.g., Jamiolkowski, et al., 1995). Thus, a more appropriate parameter is the normalized shear wave velocity, $V_{ns}$, which is defined:

$$V_{ns} = \frac{V_s}{(\sigma_{uo}'/\rho_{pm})^{0.25}}$$

where $\rho_{pm} =$ reference stress equal to one atmosphere (Note: 1 atm. = 100 kPa) and $\sigma_{uo}'$ is in same units as $\rho_{pm}$

Figure 5. Coefficient $k_p$ versus Normalized $V_{ns}$.

A better defined relationship occurs between $k_p$ and $V_{ns}$, as evidenced by Figure 5. As before, separate regression trends are observed for the Japanese series ($r^2 = 0.462$) and Swedish series ($r^2 = 0.452$). The determination of $V_{ns}$ depends on a proper evaluation of the effective overburden stress ($\sigma_{uo}'$). This requires prior knowledge of the variation of unit weight with depth, as well as hydrostatic conditions. Unit weights for the clays considered herein ranged considerably, from 11.9 to 20.6 kN/m$^3$. A guestimated value of unit weight could therefore increase the uncertainty in the evaluation of $V_{ns}$. Therefore, it is noted that procedures for estimating the unit weight from piezocene data can be found in Lunne, Robertson, & Powell (1997).

Statistical relationships presented by Mayne & Rix (1995) showed that the cone tip resistance ($q_c$) can substitute for a direct evaluation of $\sigma_{uo}'$, or perhaps more fundamentally, $I_{1}' = \frac{16}{3} \alpha_{uo}'(1+2K_0)$. Moreover, a correlation between $V_s$ and $q_c$, $\alpha_{uo}'$ is observed such that interdependence exists. The relationship is considerably strengthened if $\sigma_{uo}'$ is also considered.

Figure 6. Yield stress versus shear wave velocity.

A possible relationship directly linking $\sigma_{uo}'$ to $V_s$ was therefore explored with the results shown in Figure 6. The regression analysis indicated a fairly strong correlation ($n = 262$, $r^2 = 0.823$):

$$\sigma_{uo}' = (V_s/4.59)^{1.47}$$

with units of kPa for stress and m/s for velocity. Thus, this correlation could be of use in assessing the degree of preconsolidation of clays during geotechnical site
investigations where only geophysical data from crosstool, downhole, or surface wave measurements are available (Campanella, 1994).

The aforementioned relations lead to a final step involving multiple regression analyses which indicate 
\[ n = 262 \text{ and } r^2 = 0.917 \]
\[ \sigma'_n = (q_n - \sigma_{vo})^{0.70} \left( \frac{V}{64} \right)^{0.76} \]  
(7)

where stress terms are in kPa and \( V \), in m/sec. The higher value of \( r^2 \) compared with that of eqn.(4) verifies an improved accuracy in the approach. The summary interrelationship between effective preconsolidation stress with the net cone resistance and shear wave velocity is presented in Figure 7.

Additional studies are warranted to investigate and quantify the influences of clay mineralogy, fabric, colloid content, sand fraction, cementation, and the effects of fissuring on the derived relationships, since these too are important factors that will affect the correlational trends in soils.

4. CONCLUSIONS

A statistical study of 26 well-documented soft to firm intact and uncemented clay sites tested by seismic piezocone has been conducted for improved stress history evaluations. Direct profiling of effective preconsolidation stress (\( \sigma'_n \)) by net cone tip resistance (\( q_n - \sigma_{vo} \)) takes the general form: \[ \sigma'_n = \sigma_0 (q_n - \sigma_{vo}) \]
where \( \sigma_0 \) is shown to decrease from 0.5 to 0.1 as both the plasticity index and void ratio increase. The independent measurement of shear wave velocity (\( V_n \)) during SCPTs is shown to provide an alternative and immediate assessment of \( \sigma_0 \) for profiling OCR in clays.

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REFERENCES


Acoustic emission cone penetration testing (AE–CPT)

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ABSTRACT: Research is done on the use of the Acoustic Emissions (AE) generated when penetrating the soil with a CPT-cone in order to define more detailed soil characteristics. AE, both in low and high frequency range, are generated due to friction between grains, between grains and cone and due to crushing of soil particles. In this research AE in the range of 100 kHz to 1 MHz are recorded and analysed. In this phase of the project, two distinct goals are put forward to be achieved with the AE measurements: soil structure recognition in granular materials and detection of very thin sand seams in clay deposits, which can not be found with any other type of CPT equipment. Tests were done in the laboratory with a small needle sounding apparatus and with a prototype AE-CPT cone. These tests were performed to examine the parameters which influence the AE response of the soil when being penetrated with a CPT cone. This paper focuses on the equipment used and on the results obtained with the prototype AE cone. It is shown that clear distinction can be made between granular material and non granular material. From the needle penetration tests we know that also sample preparation and ageing, next to other parameters, influence the signal.

1 INTRODUCTION

When penetrating a soil with a CPT-cone or an other type of sounding device, the soil particles are displaced and crushed. These mechanical actions generate elastic waves in the soil with a frequency range from the audible up to the ultrasonic frequency-range. These elastic waves are called acoustic emissions. Due to the origin of AE, one can expect AE contain information on detailed soil characteristics such as grain size, mineralogy, soil skeleton structure and parameters affecting the ageing behaviour of the soil.

For this research it was chosen to use only the AE in the ultrasonic frequency range. Although much more energy is available in the audible frequency range, it is believed that more detailed and point to point information can be found from higher frequencies due to the strong damping of higher frequency waves in the soils’ skeleton structure. The signals measured should thus be generated close to the penetrating cone tip.

2 PREVIOUS RESEARCH

The use of sound with CPT testing is already known for a long time. CPT operators can learn from the sound generated via the CPT rods when course material is met during the penetration test. Feeling the vibrations on the rods by hand even gives a wider range of soil types to be distinguished roughly.

In earlier years rather limited research was performed to use the noise generated when penetrating the soil with a sounding device. With simple microphones the sound was captured and recorded on magnetic tape for analysis (Murunachi et al., 1974; Vilet et al., 1981; Tringale and Mitchell, 1982). The rms-value of the output signal showed to be a more sensitive means to distinguish soil layers than the q-value.

Mastenbroch (1986) was the first to use AE in the ultrasonic frequency domain with CPT testing. He introduced a resonance piezo-electric AE transducer (resonance frequency 210 kHz) with preamplifier in the tip of a CPT cone. Mastenbroch reported that with this technique seams of sand and
silt material with a thickness of a few mm could be detected within a clay deposit with meters of thickness while the seams could not be ‘seen’ with a piezocone.

3 AE MEASUREMENT EQUIPMENT AND SIGNAL ANALYSIS

The AE measurement equipment basically consists of a piezoelectric AE sensor, a preamplifier and a high speed digitising board plugged in a portable personal computer. The sensor has a flat response in the frequency range between 100 kHz and 1 MHz. Frequency analysis could be performed on the measured signals.

The amplified signal is digitised and stored on hard disk for analysis later on. The signal is measured as it is generated at the output of the preamplifier. During the penetration, a continuous signal is generated. The electronic equipment available at the time the research started offered the possibility to register only short ‘shots’ (4096 samples) of the continuously generated AE signal. At a sample rate of 4 MHz, this means a sample time of only 1.024 ms or a penetration depth of 20.48 μm (at a penetration rate of 20 mm/s). Each shot was written to hard disk for analysis later on at the highest possible throughput and than the board was triggered for a new measurement. Up to 10 shots per second could be recorded (figure 1).

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

4 AE NEEDLE SOUNDING APPARATUS

In order to study the parameters affecting the AE signal when penetrating a soil sample, a small laboratory testing device was setup. The parameters tested were: soil type, penetration rate, relative density, sample preparation technique, overburden pressure and aging. This apparatus and the results are discussed in Mengé et al (1995). In figure 2 and 3 the test setup an a typical burst-type AE waveform measured during penetration rate of 5 mm/s of a sample made with Molsand are given.

Figure 1. Measuring of the waveforms

Figure 2. AE needle sounding apparatus
Figure 3. Typical AE waveform generated during a needle penetration test

Following conclusions could be drawn:
1. within the test range of 10 to 50 mm/min, the RMS value per unit depth of penetration is only marginally affected by penetration rate;
2. AE intensity increases with higher \( v_0 \) values;
3. when examining rather similar soil types (various sand types), the frequency distribution (which seems to be influenced solely by soil type) gives most information; this is also a powerful tool to get more information about the mineralogy of the sand; when crustable minerals are present, more energy in the higher frequency range is found;
4. AE generated in samples prepared with the provation method are clearly less intensive than the AE signals generated in dynamically compacted samples;
5. ageing is clearly visible from samples with a relative density \( D = 75 \% \); from these tests it could be concluded the AE parameters are more sensitive to ageing than the mechanical penetration resistance \( q_0 \) is;
6. as \( q_0 \) values, there seems to be a 'critical depth' from which the depth or stress condition does not influence the intensity of the AE signal any longer.

Several of these observations can be explained as one considers the source mechanisms producing the AE. From the AE signal plot (figure 3) it is clear that the signal mainly consists of two parts: a general noise type pattern with bursts or events exceeding the noise level. These bursts most certainly are generated in the contact points between grains and between grains and penetration device when sliding or crushing occurs. Higher densities, higher penetration rates and soil structures with a higher number of contact points show more AE activity. On the other hand, soil type and soil particle characteristics also have an imported role on the number of contact points between grains and thus on the AE intensity, it also reflects in the frequency spectrum which seems to be some kind of signature for a certain soil type.

5 AE CPT TESTING

A prototype AE CPT cone was produced for testing in a laboratory test container. The test container consisted of a concrete cylinder 1.00 m in diameter and 1.00 m in height. The container has rigid boundary \( b_n = 0 \) and the vertical stress is applied through a top plate with membrane and air pressure. Side friction could be neglected. The overburden pressure to be applied was limited to 50 kPa and no vertical deformations were measured during pressurising. The sample preparation was done using a raining device with constant falling height for each layer \( t \) 100 mm thickness) or by means of tamping. A general overview of the container and measurement setup is given in figure 4.

Details of the AE CPT cone is given in figure 5. As suggested by Massarsch (1986), a needle tip was used. This design is more sensitive and better suited to detect thin seams. Figure 6 shows a typical waveform obtained with the AE CPT device (penetration rate 20 mm/s). It is clear that a strongly different signal is measured compared to figure 2. This is mainly due to the penetration speed, individual events can no longer be
Figure 4. Setup for AE CPT testing in the laboratory test container

Figure 5. AE CPT cone with needle tip

Figure 6. Typical AE CPT waveform

Figure 7. AE signal RMS values versus $q_c$

Figure 8. AE signal RMS values versus $I_D$

Figure 9. AE signal amplitude distributions
distinguished. The scatter in RMS values and other characteristics from waveform to waveform is less significant than with the needle sounding device. Figures 7 and 8 give the RMS value in function of \( q_s \) and in function of \( I_p \) respectively. From both graphs a clear influence from relative density and soil strength on the AE RMS value can be seen. It is also clear that samples prepared in different ways (rained and tamped) show a clearly different behaviour.

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

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

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

6 CONCLUSIONS

The application of the AE technique with penetration testing was examined using two laboratory test setups. Various influencing
parameters were examined. The material type and the relative density define the order of magnitude of the AE signals’ energy. Also the influence of sample preparation technique and ageing was tested and could be recognised clearly.

This illustrates that the soil skeleton structure, the orientation and the number of contact points between the grains are parameters influencing the source mechanisms of AE.

The sandtype clearly influences the pattern of the power spectrum.

It is shown that very high differences in energy level of the signal occur when penetrating different soil types, especially when going from coarse granular material to more cohesive material and vice versa. Thanks to the high sensitivity and sharp differences in AE response, very thin sand layers in clay deposits can be detected.

At this stage the equipment is not ready for use in real practice. Recording and real time analysis of the continuous signal generated while penetrating the soil should be possible nowadays with special dedicated hardware and software. Further research will be focused in this direction.

ACKNOWLEDGEMENTS

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Investigation methods for soft tailings deposits

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ABSTRACT: The majority of gold tailings produced in Western Australia (WA) are deposited hydraulically in large storages. The impoundment walls are periodically raised using upstream construction to increase storage capacity. Much of the gold comes from highly weathered rock, resulting in tailings with significant clay contents. Consolidation of this material can be very slow, particularly if occurring under the effects of self-weight alone. Therefore, management strategies rely on allowing successive layers of tailings to dry sufficiently before the next layer is deposited. To obtain better data, it is necessary to develop a method of gaining access to the tailings storage at all stages following deposition. A small hovercraft has been modified to enable in situ tests and sampling to be conducted in positions that would otherwise be inaccessible. Results gained from a number of investigations on active tailings storages have enabled full characterisation of these sites to be accomplished.

1 INTRODUCTION

In Western Australia (WA) much of the gold produced comes from low grade, near surface ore bodies. This leads to large volumes of fine grained slurred waste (tailings) that must be disposed of in an environmentally acceptable manner. In most operations, these materials are hydraulically deposited in large tailings storage facilities (TSF) that consist of an above ground containment area surrounded by 'ring like' walls, generally constructed of oxidised waste material or imported fill. A significant proportion of the gold comes from highly weathered rock, which results in tailings with significant clay contents. Consolidation of these materials can be very slow, particularly if occurring under the effects of self-weight alone. Therefore, tailings storage management strategies rely on allowing successive layers of tailings to dry to a high density before the next layer is deposited.

In order to ensure a final stable surface crust of sufficient strength to allow subsequent access for rehabilitation, constant monitoring is required to prevent the buildup of thick layers of poorly consolidated material. Information on the state of the tailings is also required before construction of additional wall lifts using the upstream method, since the additional lifts are supported partly on the tailings. Determining the overall stability of the walls of the storage also requires knowledge of the strength profiles and the degree of consolidation of the tailings adjacent to the walls, and of the location of the piezometric surface in this area.

Gaining access to all areas on an active TSF to conduct the required in situ testing can be difficult in many cases. This may prove impossible even with small truck mounted drilling rigs unless an access ramp is constructed across the tailings surface to the positions of interest. This process inevitably disturbs the material beneath the ramp, and in some cases may prove extremely difficult due to insufficient bearing capacity of the tailings surface. The consequence is that investigation is rarely carried out on active tailings storages, and then only to a very limited extent.

The work described in this paper forms part of a project funded by MERIWA (Minerals and Energy Research Institute of Western Australia) Project No. M241 entitled 'Research into Saline Tailings Disposal and Decommissioning', which is being conducted by the Geomechanics Group at the University of Western Australia. The overall aim of the research is to develop a better understanding of the mechanisms controlling the consolidation and evaporation behaviour of gold tailings in general, and saline gold tailings in particular. This study forms part of a general thrust by the gold-mining industry in WA aimed at achieving better and more economical methods of operating and rehabilitating TSFs.

To ascertain the current state of a representative sample of existing tailings storages in Western Australia, a site investigation programme has been
carried out. Some of the sites chosen were active tailings storage sites and this required new methodologies to be developed. These are described in the following sections.

2 SITE INVESTIGATION METHODOLOGY

Tailings moisture contents in active storages can vary from ‘as deposited’ moisture contents well in excess of 100% to residual values of only a few percent. The majority of tailings lie between these two extremes, depending upon the effectiveness of the disposal strategies employed on the site. Therefore carrying out investigation at such sites requires gaining access on to surfaces with shear strengths frequently less than 10 kPa. Faced with this situation, it was decided to employ a hovercraft for this work to allow access to all points of the storages and to provide a stable platform for probing and sampling operations. An additional advantage of using a hovercraft is that unlike the majority of other methods employed for this type of work, surface disturbance is reduced to a minimum.

A range of in situ testing devices has been developed or modified for use with the hovercraft. These include a field vane, a stationary piston sampler and a piezoecone fitted with a resistivity module. The main consideration in the manufacture or modification of these devices has been to keep the weight to a minimum, because of the limited carrying capacity of the hovercraft.

2.1 Hovercraft details

The hovercraft used in the work is shown in Figure 1. The basic hovercraft was purchased from Queensland Hovercraft Supplies Pty Ltd, and subsequently modified. The hull is 3.5m long and 2m wide and is constructed of fibreglass, which gives the craft a total weight (unladen) of 220kg. When set down on water, its buoyancy is more than 400 kg. It is powered by a 38kW liquid-cooled Rotax two stroke engine, which provides both thrust (maximum speed 80 km/h) and lift for the vehicle, and the ability to hover at a height of 230mm with a maximum payload of 300kg. The air flow is ducted to supply approximately 30% lift and 70% thrust from the 12-blade fan at the rear.

To allow access to the soil surface when the hovercraft is set down, a 150mm diameter hole was constructed through the central ridge that forms the seat support. This hole is sealed during flight, and opened when the craft is set down on the surface. In situ testing and sampling operations are conducted through this hole. Strong points have been added to the hull to provide rigid anchoring points for a drilling platform. The platform is 2m by 1.5m in area, and is equipped with a manually-operated jacking rig, also shown in Figure 1. A small clack (a 4-jaw lathe clack) is used to grip onto the rods. The clack is mounted on a chain-drive system, moved by two handles rotated by one or two operators. Penetration into firmer materials is limited only by the combined dead weight of the hovercraft, operators and equipment. Cone penetrometer tests (CPT) have been successfully conducted in materials with tip resistances of over 4 MPa.

Figure 2: Bearing capacity v depth for A2 and A4

The craft has been tested on different surfaces to assess liftoff and setdown ability, and maneuverability. The hovercraft can operate well on slurred tailings, water, solid tailings and even on surfaces with large desiccation cracks (1m deep and 0.4m wide), although the cracks tend to let some of the air escape from the skirts and reduce the efficiency of lift.

2.2 Resistivity cone

The resistivity piezoecone (RCPTU) is based on a standard cone penetrometer, with a cone projected...
The sampler is operated by using the jacking system to jack it into the soil with the two rods locked together and the piston protruding from the end of the sampling tube. On reaching the target sampling depth, the inner rod is held stationary while continuing to penetrate the outer rod. The two rods are locked together again for retrieval. With this system, samples can be obtained at exact target depths in tailings of very low shear strengths (less than a few kilopascals).

A field vane has also been developed, again with high strength aluminium rods (to minimise weight), to allow shear strengths to be measured to a depth of 20 m. Two crosstree vane is used (L=150 mm and 100 mm) with 2:1 aspect ratio and tapered triangular end sections. The vane is inserted using the jacking system, such that penetration to 15–20 m have been achieved in soft tailings. Torque is applied using a torque wrench, which registers the peak torque. A slip coupling allows the rod friction correction to be determined before the vane itself is engaged.

3 TYPICAL INVESTIGATION RESULTS

Data from two sites (A and B) are presented in this section. At each site, investigations were carried out at four positions that lay on a transect along the axis of the storage, from the discharge point to the decant pond. At each position piston samples were taken and returned to the laboratory to ascertain in situ salinities, moisture contents, densities and particle size distributions. A proportion of these samples were also used for chemical analysis to determine cyanide concentrations and speciation, and concentrations of heavy metals through the tailings profile. Soundings were taken at each point using the RCPTU, to give bearing capacities, friction ratios, pore pressure distributions and salinities. Vane tests were conducted to find undrained shear strengths.

2.3 Pitson sampler and field vane

A lightweight thin-walled stationary piston sampler has been constructed for taking undisturbed samples of tailings of 50 mm diameter and 750 mm length. The sampler uses a double rod system consisting of outer rods of high-strength aluminium and inner solid rods of stainless steel. The system can be used to retrieve samples at depths of up to 20 m in soft soils. Generally, thin-walled PVC sample tubes, with a machined cutting edge are used. The front face of the piston is conical, to aid penetration.
3.1 Cone Penetrometer Testing (Site A)

Figures 2-5 show RCPTU data from positions on a decommissioned tailings storage (site A). This storage has been inactive for approximately 2 years. Four boreholes (A1-4) were positioned 80m apart along a transect from the storage wall to the decant. Borehole A1 lay nearest the wall (and the discharge point) and A4 nearest the decant.

![Salinity concentration vs depth for A3](image)

Figure 5: Salinity concentration vs depth for A3

Figure 2 shows the cone tip resistance at two positions, A2 and A4. Near the wall, at point A2 (thin line), the tip resistance, $q_t$, varies from 0.1 MPa at the surface to an average of 0.7-0.8 MPa at 12m depth. The profile shows a high degree of variability of tip resistance with depth, which is indicative of an interlayered sequence typical of tailings. The profile for A4 (thick line) has similar characteristics, but shows a softer material varying from <0.65 MPa at the surface to 0.3 MPa at approximately 10m, where the cone encountered a more hard end layer.

Figure 3 shows the magnitude of the friction ratio for both boreholes (A2 and A4). The profile for A2 varies from 1 to 4%, with an average of about 1.5% and that for A4 varies from 15% with an average of 2.5-3%. Both friction ratio profiles are highly variable and combining this information with the measured tip resistances, these materials can be classified as sensitive fine grained materials with lenses of sandy silt, according to the classification of Robertson et al. (1986).

The pore pressure profile for A2 (measured behind the tip) is shown in Figure 4. This has been plotted together with an assumed hydrostatic pore pressure. The majority of the deposit is still saturated, and assuming that there is no significant base drainage, the data indicate sand lenses at a number of depths throughout the tailings profile.

Figure 5 shows salinity (solid line) plotted against depth for A3. The salinity shown here is the concentration of NaCl that would give the measured pore fluid resistivity. The results indicate quite a variable profile of salinity with depth, with an average salinity of approximately 250 g/l total dissolved solids. The salinity concentration ($c_s$) of the deposited tailings on this site are typically 180-200 g/l. As a comparison, the salinity of sea water is approximately 30 g/l. The variability of these measurements is related to the sub-aerial deposition of the tailings in layers. As each layer is left to dry (during rotation of deposition around the storage) evaporation causes the development of a surface salt crust. Over time, high concentrations of salinity accumulate near the surface of each layer, leading to the variable profile shown in the figure. It is interesting to note that $c_s$ of the deposited tailings is much lower than the average value shown. This is due to the aforementioned evaporation process that gradual increases the average concentration. This can have important consequences with regard to seepage through the base of the storage transporting toxic levels to the groundwater.

3.2 Laboratory Results (Site A)

Figure 5 shows the RCPTU profile of salinity along with discrete measurements of salinity from laboratory samples. The laboratory values were found from 1:5 (soil to water) extractions measured using electrical conductivity meters and by filtering and drying. The discrete values compare well with the values found in situ by the cone.

![Moisture content vs depth for A1, 3 and 4](image)

Figure 6: Moisture content vs depth for A1, 3 and 4

Moisture contents for positions A1, A3 and A4 found from piston samples are shown in Figure 6. These values have been corrected for the presence of the high concentrations of salt. The results show a variation of moisture content at the surface from 56% at A4 to 23% at A1. The moisture content for A1 drops fairly quickly to 12% and remains constant to 3.5m. At this point the rig was unable to progress the borehole further. At depth in A3 and A4 the
moisture contents are reasonably similar with an average of approximately 30%, to a depth of 8m. The same samples were passed through a 75μm sieve (usually taken to be the division between silt and sand size) as a rough indication of particle size. The results of the sieving are shown in Figure 7. This suggests that at the surface, A4 has more fines than A3 and A1 (the coarsest). At depth A3 and A4 were similar, with A1 being the coarsest.

![Figure 7: Particle size vs depth for A1, 3 and 4](image)

3.3 Undrained shear strength (Site A)
Comparisons of the vane shear strength, $S_v$, with depth are shown in Figure 8. Values of $S_v$ measured with the field vane have been plotted against values estimated from the RCPTU data, using the corrected cone resistance $q_c$ and a cone factor $N_{s_c}$ of 8. The results appear to correlate quite well, with both data sets showing a similar trend of $S_v$ increasing with depth.

3.4 Cone Penetrometer Testing (Site B)
Figures 9 and 10 show data from RCPTU tests on a different site (B). The bearing capacity with depth is shown in Figure 9 for a position near the wall (B1). This shows almost no strength in the top 5m and then reasonably firm material beneath ($q_c$< 1.2 MPa). Figure 10 shows a salinity profile from the same position. Whilst there is some variability in the profile it is not nearly as marked as in Figure 5. Here the concentration varies from 300 g/l at the surface to approximately 200 g/l at depth.

4 DISCUSSION
Upstream wall construction relies on the segregation of the coarse and fine particles as the tailings flow down the beach (from the wall) towards the centre of the storage facility. The ideal situation is for the majority of the coarse particles to remain near the discharge point (close to the walls) and the fines to be deposited away from the walls. This allows subsequent wall lifts to be constructed with material with a high friction angle. The more compressible, low permeability fines are located well away from the walls of the impoundment. It also ensures that a low phreatic surface can be maintained near the walls, thereby increasing stability and reducing erosion problems.

The data shown for site A indicate that the surface material near the wall is drier, coarser and firmer than that near the decant. The data also suggest that the surface material in much of the storage is clayey, whilst at depth the material appears to be coarser and may in fact be predominantly fresh rock tailings (i.e. rock flour). At all depths the segregation appears to have been adequate and the area around the decant is quite clayey and still retains much of the moisture from the decant pond (this would have been submerged until very recently). However, despite the storage having been left fallow for two years, it would still be difficult to cap this structure for rehabilitation at present.

The storage at site B initially contained a mixture of fresh rock and oxide (clay) tailings. As new ore bodies were processed there was a considerable increase in the clay content of the tailings. Unfortunately the rate of rise of the deposit was also quite high (>3m/year) and the poor state of the storage was the result. The salinity profile shows that some surface evaporation has been occurring.

![Figure 8: Undrained shear strength vs depth for A2](image)

However, very little evaporation seems to have occurred during the filling process itself. Whilst the tailings below 5m depth appear to be quite competent the surface material is very soft. Upstream wall lifts with the material in its current state would be extremely difficult. If the storage was to be decommissioned, a considerable time would need to elapse before capping (for rehabilitation).
would be possible. Comparison of laboratory samples and CPT results also suggest that very little segregation occurred during the filling. In fact, the rate of rise may have been such that the filling was effectively sub-aqueous.

The data from sites A and B, and others in the site investigation programme illustrate how complex the behaviour of the tailings can be. The generally accepted view that sufficient segregation occurs in most storages appears to be a misconception. Poor control of beaching and deposition can affect segregation and even storages filled with fairly benign tailings can quickly become unmanageable.

5 CONCLUSION

The paper has described a method of investigating active tailings storage facilities using a specially modified hovercraft. The craft has been fitted with lightweight equipment to enable field vane tests, undisturbed piston samples, and RCPTU to be conducted on soft slurred tailings. Results gained from a number of site investigations on active tailings storages have been presented. The data have enabled better characterisation of active tailings storages to be accomplished, and have shown this to be a viable approach.

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Commercial CPT profiling in soft rocks and hard soils

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ABSTRACT: The principal interests for Cone Penetration Test (CPT) profiling in soft rocks and hard soils are the detection and characterisation of thin weak zones and, conversely, thin competent zones. Such profiling is close to the limits of current technology. An important question is: (1) Is the currently available equipment adequate to penetrate the expected soft rock and hard soil? This question is addressed by summarising the current thrust capabilities of various CPT systems, including "smart" solutions that profit penetration. A second question relates to the implicit commercial requirement for the use of a single cone penetrometer for a very wide range of ground conditions: (2) Are the results sufficiently meaningful to obtain geotechnical parameters or do secondary effects dominate the CPT measurements? This complex subject is considered by assessment of secondary effects, such as non-uniform penetration rate due to rapid loading/unloading of push rods and transient heat flow through the penetrometer for which no temperature compensation is possible. Examples illustrate the difficulties but also the opportunities for commercial CPT profiling in soft rocks and hard soils.

1 INTRODUCTION

The Cone Penetration Test (CPT) is probably the most efficient tool for detailed profiling of soil (Lunne et al., 1997), but its commercial application in soft rocks and hard soils is close to the limits of current technology.

The principal interests for (piero-cone) CPT profiling in soft rocks and hard soils are the detection and characterisation of thin weak zones and, conversely, thin competent zones. Weak or soft zones are often important for the in-place behaviour of a particular structure. For example, a thin weak zone in otherwise competent ground may control the sliding resistance of a structure. The detection and characterisation of a thin zone of soft rock or hard soil can be important for the constructability of a particular facility. A common example is the feasibility of pile driving as construction method. A fairly recent development is the direct use of CPT parameters for pile design in soft rock and hard soil (Randolph et al., 1994; Randolph et al., 1996; Jardine and Chow, 1996; Zuidberg and Vergegli, 1996). This development increases the requirements for reliable penetration capabilities of cone penetration test apparatus and, to a lesser degree, for accurate CPT parameter values.

This paper discusses operational "question marks" surrounding the confident use of the CPT in soft rock or hard soil. The discussions consider electric cone penetrometers with a base area between 500 mm² and 1500 mm², which is within the range standardised by ASTM (1993), BSI (1990) and NNI (1996).

2 CASHING OR REFUSING: PENETRATION CAPABILITIES

Is the currently available equipment adequate to penetrate the soft rock and hard soil? This question has immediate technical and commercial impact, in particular during the negotiation phase for CPT employment. The following section discusses the principal technical considerations and the section titled "Examples From Practice" illustrates where real-world CPT penetration in soft rock and hard soil was successful, abortive and/or subject to significant loss of equipment.

The penetration capabilities of a CPT system depend on the weakest link in the major
components: the cone penetrometer, the push rods and the thrust machine. The cone penetrometer is the most delicate part of the CPT apparatus that is in direct contact with harsh ground conditions. It is the subject of separate sections of this paper.

An important consideration for soft rocks and hard soils is push rod buckling stability between the thrust machine and the ground surface, and below the ground surface in case of soft soils. Design of high-strength tubular push rods and push-rod connectors is currently close to the technical limits, for the standardised geometrical constraints. Appropriate use of additional guidance casing is generally effective; in fact it is often essential.

Surface-based and downhole thrust machines are available. The surface-based systems date from the 1930s. The early systems were suitable for soft soils only, but systems with 100 kN thrust capacity were already available in the late 1940s. Current surface-based systems usually consist of a ballasted vehicle equipped with hydraulic jacking facilities or a (seabed-based) platform equipped with hydraulic jacking or wheeldrive thrust. The common jacking and wheeldrive systems allow application of a centric thrust force of up to about 200 kN, if properly maintained.

Development of downhole systems was offshore-driven, with prototypes emerging in the early 1970s. A downhole system derives reaction from a borehole drill string that is fixed at ground surface by a clamp attached to a ballasted or anchored frame. Operation of a downhole system is by wireline, with or without a so-called umbilical cable for data transmission. The available thrust force for a downhole system is typically about 100 kN for a stroke length of 1.5 m or 3 m.

Commercial considerations will generally favour the use of a robust surface-based system for cone penetration testing. The use of a smart complementary tool or a downhole system may be necessary if project requirements dictate a specific penetration depth as termination criterion. Table 1 lists some tools for enhancing CPT penetration.

<table>
<thead>
<tr>
<th>Penetration Profi Tool</th>
<th>Characteristics</th>
<th>Commercial Availability</th>
<th>Additional Operating Cost</th>
<th>Reference(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>friction reducer</td>
<td>Effective in ground with a high friction, but ineffective or possibly adverse in very dense cohesionless soil and convoluted soil or weak (coarse) rock</td>
<td>High</td>
<td>Negligible</td>
<td>Versaud (1948), Van de Graaf and Schenk (1983), NNI (1996)</td>
</tr>
<tr>
<td>cone penetrometer with diameter larger than push rod diameter</td>
<td>As for friction reducer</td>
<td>High</td>
<td>Negligible</td>
<td>Van de Graaf and Schenk (1988)</td>
</tr>
<tr>
<td>guide casing</td>
<td>Indirectly effective in low-strength non-surface soils to reduce the risk of push rod buckling at high thrust forces</td>
<td>High</td>
<td>Some additional operational activities</td>
<td>Versaud (1948)</td>
</tr>
<tr>
<td>drill-bit and use of push rod support casing</td>
<td>Effective in most soft rocks and hard soils but progress in ground with frequent, thin layers (e.g., gravel, cobble particles) giving high cone resistance may be slow</td>
<td>High</td>
<td>Additional drilling equipment and casing, additional drilling time and additional time for retraction and re-injection of push rods for drill-out lengths</td>
<td>NNI (1996)</td>
</tr>
<tr>
<td>dynamic penetration</td>
<td>Increased penetration in soft rocks and hard (gravelly) soils in comparison with conventional CPT systems</td>
<td>Regional</td>
<td>Special apparatus including impact hammer equipment and (impact) cone penetrometer</td>
<td>Sanglent (1979), Sanglent et al. (1995)</td>
</tr>
<tr>
<td>mud injection</td>
<td>Effective in ground with high friction only</td>
<td>Limited</td>
<td>Additional apparatus, including mud pump and mud mixing; additional handling of mud tubing</td>
<td>Jeffries and Panzani (1983), Van Sluveren (1995)</td>
</tr>
<tr>
<td>downhole thrust system</td>
<td>Effective in most soft rocks and hard soils, but with single push stroke limited to typically 1.5 m or 3 m</td>
<td>Limited</td>
<td>High-tech downhole apparatus, in combination with 100 mm ID borehole drilling, sampling and coring</td>
<td>Zundelberg (1974), Peterson and Johnson (1983), Powner and Greis (1994).</td>
</tr>
</tbody>
</table>
Most of the common "penetration profit" tools enhance the penetration capabilities for particular ground conditions only. However, no penetration limits apply to the drill-out system and the downhole thrust systems, because interruption of CPT penetration is possible at any depth. Further borehole drilling, sampling, rock coring and in-situ testing at the same test location is then feasible to suit geotechnical or environmental characterisation. Progress for deep drill-out CPT's can be slow because of the need for retraction and re-insertion of the push rods for each CPT stroke and switching between drilling mode and push mode. The operational speed of the faster wireline downhole thrust system is typically equivalent to or better than a conventional borehole with sampling or coring.

3 COMMERCIAL IDEOLOGY: 
MULTI-RANGE CONE PENETROMETER

Soft or weak soils often overly soft rock or hard soil, but the reverse situation is also present. Theoretically, the use of two or more penetrometers per test location would appear to resolve this matter, with the measuring ranges suited to the ground conditions. Piezoe-cone development work reported by Zuidberg (1988) suggests that only limited benefits of such practice can be expected. Obviously, the use of two or more penetrometers per test location does not work for characterisation of a thin weak zone in otherwise competent ground. Similarly, it would not be commercially attractive in case of a site with a hard soil top layer overlaying peat or normally consolidated clay. A single penetrometer applicable to a very wide range of ground conditions is necessary to remain in business.

Experience shows that the use of the CPT in soft rock or hard soil requires a cone penetrometer with a cone resistance range of at least 100 MPa to allow survival. The calibrated range may be less for the purpose of precision and accuracy of lower cone resistance values. Similarly, the requirements for the minimum ranges for sleeve friction and pore water pressures are typically about 5 MPa and 10 MPa respectively. Higher limits will be necessary for deep-water offshore operations. The following sections include comments on the implications of the "survival criteria" for accuracy of CPT parameter values.

Subtraction-type penetrometers dominate the commercial CPT market for conventional soils. Typical features of a subtraction-type (piezo-cone) penetrometer include (1) 100 kN load sensors placed in series for cone resistance and sleeve friction and (2) absolute (rather than differential) pressure sensor for water pressure measurements (typically 2.5 MPa to 15 MPa burst pressure). The more "robust" versions of subtraction-type penetrometers are generally adequate for soft rock and hard soil (e.g. Sternfelt and Sorensen, 1995; Puschel et al. 1996). Robustness implies features such as a) survival under high loads and pressures, b) resistance to fouling and c) resistance to cross-talk between load sensors for the cone and the friction sleeve. It does not necessarily mean accuracy. For example, a measuring range (not necessarily the calibrated range) of 100 kN corresponds with a maximum range for cone resistance of 0 to 100 MPa. Measurement of sleeve friction is by a second 100 kN load sensor that measures the sum of the load on the cone tip and the friction sleeve. In practice, sleeve friction values of up to about 3 MPa are permissible. Practical (laboratory) accuracy of available large-range load sensors is typically linear at 0.1% of full range (equivalent to 100 kPa contribution to cone resistance uncertainty). This accuracy applies to a wide measuring range but can be handled by conventional Analogue to Digital (A/D) converters.

4 MEANINGFUL OR MEANINGLESS 
RESULTS: ABOUT ACCURACY

4.1 General

The use of a single cone penetrometer raises a second question (2): Are the results sufficiently meaningful to obtain geotechnical parameters or do secondary effects dominate the CPT measurements? This paper considers the influence of secondary effects, including: (a) non-uniform penetration rate due to rapid loading/unloading of push rods, (b) transient heat flow through the penetrometer for which no temperature compensation is possible (c) the geometry of the friction sleeve with respect to that of the cone and (d) compression/decompression of the piezo-cone filter. These factors illustrate the difficulties but also the opportunities for reliable and commercial CPT profiling in soft rocks and hard soils.

Before discussing secondary effects, it is necessary to define the metrological meaning of accuracy (ISO, 1992). Confusion about this term
appears common among geotechnical professionals and it should be no surprise that the European Technical Committee on standardisation of laboratory tests dedicates a chapter to this subject. Figure 1 presents a graphical explanation of some metrological terms.

4.3 Transient heat flow

Post and Neerbek (1995) present results of field measurements and laboratory simulation for quantification of the accuracy (or measurement uncertainty) of a Fugro CPT system with a conventional subtraction-type cone penetrometer. The observations of the thermal gradient within a cone penetrometer as a result of soil-steel friction are of particular interest. Post and Neerbek report a temporary shift in cone resistance output of about 130 kPa for penetration of a sand layer with a cone resistance of about 25 MPa. They recommend adjustment of operational procedures for critical ground strata. For example, a penetration interruption before penetration of soft ground allows temperature stabilisation in the sensors of the cone penetrometer.

4.4 Cone penetrometer wear

Jetel (1988) and Schaap and Zuidberg (1982) report studies of the mechanical wear of cone penetrometers and implications for robust cone penetrometer design. Mechanical wear is of minor importance for the accuracy of cone resistance, in particular because of the possibility for correction of base area reduction. The measurement of sleeve friction $f_s$ is particularly sensitive to wear. The Dutch standard on electric cone penetration tests recognises this uncertainty about geometrical tolerances and considers, for example, Class 3 (common good practice) accuracy requirements for $f_s$ of 50 kPa and 20% of the measured value, whichever is higher. These requirements appear "relaxed", but consideration should be given to the correct definition of accuracy of $f_s$ as a measured parameter. Important factors for measurement of sleeve friction in soft rocks and hard soils are a) "end bearing resistance" of the friction sleeve because a (permited) larger diameter than that of the cone and b) conical wear of the friction sleeve and c) smoothness of the friction sleeve.

Significant reductions in sleeve friction and, therefore, friction ratio may take place within about 100 m of penetration for soft steel types. The use of abrasion resistant steel and the adherence to test procedures for monitoring of the (standardised) geometry of the cone penetrometer are essential for
achieving a base-line accuracy of CPT results. It is of interest to note that implementation of an accreditation scheme (Peschken et al., 1995) for the cone penetration test industry in the Netherlands showed (unpublished commercial) "surprises" about the rate of wear of the friction sleeve immediately above the cone.

4.5 Pore pressure measurements

Comparison of piezo-cone and friction-cone penetration test results indicates that conventional friction-cone tests are generally adequate for characterisation of soft rock and hard soil. The main benefit of the more delicate piezo-cone tests is characterisation of soft strata penetrated as part of a test in soft rock and hard soil. The principal commercial consideration for piezo-cone penetration tests in soft rock or hard soil is the survival of the water pressure sensor in case of very high excess pore pressures. In addition, the piezo-cone filter is important, as one of its purposes is to protect the pressure sensor against mechanical damage arising from direct soil and sensor contact.

Conventional practice by Fugro is to use (ductile) HDPE cylindrical filters in the cylindrical extension of the cone (u filter). A ductile HDPE filter maintains its geometry reasonably well in soil, but well inevitably compress and deform upon penetration of soft rock or hard soil. Trials with relatively brittle ceramic filters showed frequent cracking upon extraction of the cone penetrometer or loss in the ground. Recent trials with radial button filters appear to show a tendency for development of lower pore pressures than those recorded by conventional cylindrical filters.

Peschken et al. (1996) discuss aspects of HDPE filter wear and filter compression for tests in residual soil and weathered rock. Replacement of filter elements was necessary after each test, but pore pressure response was according to expectations.

5 THE REAL WORLD: EXAMPLES FROM PRACTICE

Steenfelt and Sorensen (1995) describe experiences with more than 4000 m of CPT profiling in complex heterogeneous till deposits, marl and limestone. The glacial tills showed undrained shear strengths ranging typically from 250 kPa to well over 1000 kPa. In addition, the tills included gravel, cobbles and boulders as well as lenses and layers of sand and gravel. The selected 200 kN penetration apparatus was (scabbed) surface-based. Pioneering activities by Fugro were not without commercial implications as cone penetrometer damage or loss was initially about 1.6 (i.e. one damaged or lost cone penetrometer for every 6 tests). This led to development of a set of termination criteria, as follows: (1) maximum surface-based thrust of 200 kN, (2) maximum cone resistance of 80 MPa, (3) maximum sleeve friction of 0.5 MPa and (4) cone penetrometer inclination of maximum 20° or 3° sudden increase in combination with a high cone resistance. The use of the above termination criteria also removed the potential risk of invalid data because of slightly damaged or malfunctioning penetrometers. This has commercial benefits, in particular for offshore activities where the thrust machine may remain submerged for two or more tests. Steenfelt and Sorensen also report difficulties with the interpretation of pore pressures in glacial till, in particular where pore pressures approached zero or became negative. The reason attributed to these apparent anomalies is the presence of gravel-sized (or larger) particles.

Peschken et al. (1995) document three case histories comprising nearly 900 (piezo-) cone penetration tests in tropical residual soils, including soft sedimentary, metamorphic and igneous rock types. The commercial operations included conventional 200 kN surface-based thrust machines, 1000 mm² cone penetrometers (subtraction type), a friction reducer at 1 m above the cone and the use of a short guide casing. Table 2 summarises penetration depths and applied CPT termination criteria for the test sites. A duricrust layer at one of the sites presented penetration difficulties. Furthermore, anisotropy and heterogeneity of the parent rock led to significant differences in penetration depth and, in some cases, resulted in inclined penetration. The authors conclude that CPT penetration capabilities depend on parent rock type as well as weathering grade. Penetration capability estimates for the employed test apparatus and procedures indicate that CPT penetration into sandy residual soil is feasible for completely weathered rock (Grade V according to BSI, 1981). Similarly, CPT penetration into clayey residual soil of weathering Grades III and IV (highly and moderately weathered) is feasible.

Figure 2 presents CPT data acquired for high-capacity piles in the Middle East. The ground profile includes clays with cone resistance values of greater than 10 MPa and very dense soils with cone resistance values of up to about 120 MPa. Practically
Table 2. CPT penetration depth and termination criteria after Pechen et al. (1996).

<table>
<thead>
<tr>
<th>Parent Rock Type</th>
<th>Number of PCPT Tests</th>
<th>Average Termination Depth Below Ground Surface (m)</th>
<th>Average Depth in Residual Soft and Soft Rock (m)</th>
<th>Cone and Friction Load Cell Overload (180 kN) (%)</th>
<th>Cone Load Cell Overload (100 kN) (%)</th>
<th>Maximum Inclinations (15°) (%)</th>
<th>Maximum Thrust (200 kN) (%)</th>
<th>Combination of Factors (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>mudstone, rhyolite and sandstone</td>
<td>163</td>
<td>22</td>
<td>6</td>
<td>51</td>
<td>21</td>
<td>11</td>
<td>2</td>
<td>13</td>
</tr>
<tr>
<td>granite</td>
<td>662</td>
<td>22</td>
<td>14</td>
<td>19</td>
<td>16</td>
<td>9</td>
<td>21</td>
<td>35</td>
</tr>
<tr>
<td>schist</td>
<td>65</td>
<td>21</td>
<td>16</td>
<td>15</td>
<td>20</td>
<td>19</td>
<td>17</td>
<td></td>
</tr>
</tbody>
</table>

All SPT results for adjacent boreholes showed refusal conditions (N-value of more than 50) for the full profile. Similarly, Figure 3 illustrates the use of a downhole thrust system for very dense North Sea sands in the depth range 50 m to 80 m below seabed. The maximum available stroke of the push rod was 1.5 m.

Many more examples from practice are available to illustrate the successful and not so successful use of CPTs in soft rocks and hard soils. Fugro’s experience includes very hard tertiary “clays” (essentially soft clays) in Kazakhstan, Georgia and Russia, chalk in the UK, coral and soft carbonate rock in India, Philippines, Middle East and Australia, very dense glacial sands of Northern Europe and permafrost in Siberia.

6. MAIN POINTS

1. Technical optimisation and associated commercial benefits are likely to maintain a drive for increased use of the CPT in soft rocks and hard soils

2. Well-designed CPT systems and controlled operations are essential for successful application of the cone penetration test in soft rocks and hard soils.

3. Experience with cone penetration tests in soft rocks and hard soils is increasing and so does the confidence to predict the feasibility of obtaining commercial benefits from relevant and reliable CPT results for a particular project site.

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Cone penetration testing in cemented soils: Comparisons between field and laboratory chamber test results

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ABSTRACT: This paper presents a field test program conducted on a loose cemented site near Vicksburg, Mississippi. Several friction cone penetrometer tests were conducted as a part of the testing program. In addition, laboratory strength tests were conducted on block samples taken from the same site. These tests results are used to evaluate the correlations and conclusions developed from chamber tests conducted on cemented sands. Cementation has more influence on field cone test results than the confinement primarily due to low friction angles of silts. The evaluation of Janbu and Seamaske's bearing capacity theory shows that the correlation developed from chamber studies appears to provide reasonable results. In addition to cementation, the partial saturation also influences the cohesion properties. More field data and calibration chamber tests on cemented soils at different saturation levels will provide further insight into the influence of cementation effects on cone test results.

1 INTRODUCTION

The effect of cementation on the strength and deformation behavior is very important in the analysis and design of geotechnical engineering systems constructed on or in cemented deposits. Cementation bonds which produce a cohesion intercept are neglected in the analysis and design for practical purposes. However, neglecting the effect of cementation levels results in an underestimation of the strength of soil deposits as well as underestimation of the stability of soil slopes (Frydman et al. 1980; Rad and Clough, 1982). Several slope failures on cemented deposits due to earthquake or heavy rains were reported in the literature (Rad and Clough 1980; Rad and Turnay, 1986; Puppala, 1993). In addition, the behavior of cement stabilized subgrade soils is also important in the design of pavements and airport runways. Considering all these, it can be stated that it is important to identify natural and artificial cementation levels in subsoils prior to any earth structure design.

A laboratory calibration chamber study was conducted to calibrate a friction cone penetrometer to identify the cementation effects on sand (Puppala, 1993). In this study, cemented sand specimens prepared at three relative density ranges were tested at three levels of cementation pressures. This large scale laboratory study provided several conclusions on the effects of cementation on cone resistances in sands. One of the conclusions from this study is that the cementation effect is more significant on cone tip resistance at shallow depths than at larger depths. Test results are also used in the development of correlations and in the assessments of theoretical bearing capacity and cavity expansion models. The conclusions and correlations from laboratory study will only be valid by evaluating them with test results conducted on a natural cemented deposit. Hence, a field study was planned and conducted on a cemented soil site and the study results are used in the evaluation of conclusions drawn from chamber study (Arslan, 1993).

This paper summarizes results from cone penetration tests and shear strength tests conducted on a cemented loose deposit near Vicksburg, Mississippi. The paper then compares field test results with chamber test results. Observations noted from these comparisons are explained. Future research directions in this area are also provided for further understanding of cementation effects on cone test results.

2 CEMENTED SOIL SITE

Cemented soils consisting of sands and silts are found in many parts of the world. They cover large areas in
the U.S., Asia, Africa, and Europe. A majority of the cemented deposits consist of sandy particles. In some sites like the one near Vicksburg, Mississippi, the major soil type in the cemented deposits is silt. Though chamber studies were conducted on cemented sands, the authors have selected the Vicksburg site with silt soil for field investigation. This is because the site has cemented deposits at shallow depths which makes block sampling less arduous. In addition, costs associated with the transportation of a test vehicle to known cemented shallow sand deposits in California and other states from Louisiana (where the chamber study was conducted) can be quite high. The site near Vicksburg is quite close to Louisiana and thus selected for field investigation.

The cemented soil deposits in the lower Mississippi Valley are known to be loess deposits. Krinitzky (1950) mentioned that these deposits are unstratified, calcareous, slightly plastic, porous loam with an average grain size diameter ranging between 0.05 and 0.01 mm. The cementation in these soils is attributed to the precipitation of calcium carbonates from plant vegetation or change in pH of the groundwater or a combination of both (Krinitzky, 1930). Figure 1 shows the schematic of calcareous loess deposits in the Mississippi Valley.

Figure 1: Calcereous Loess deposits in Mississippi

3 TESTING PROGRAM

This section provides a description of the test vehicle used for field tests, field testing program, and block sampling and laboratory tests conducted in the study.

3.1 Research Vehicle for Geotechnical In Situ Testing

The Research Vehicle for Geotechnical In Situ Testing and Support (REVETS) of Louisiana Transportation Research Center (LTRC) was used for performing CPT tests. The 20 tonne all wheel-drive vehicle is well-equipped for in-situ subsurface soil exploration (Tumay, 1994). The cone penetration testing system is placed in a van body mounted vehicle. This vehicle also includes hydraulic leveling and CPT operation with a 1 m stroke clutching system. The vehicle penetration thrust system has two double acting hydraulic cylinders. Maximum drive load is 200 kN and pulling load is 260 kN. With the clamping device, penetration rods of 35.6 mm and 55 mm in diameter can be penetrated and extracted.

A reference cone penetrometer with a nominal diameter of 35.7 mm was used for cone penetration. The cross-sectional area of the cone is 10 cm², and the friction sleeve area is 150 cm². The apex angle of the cone is 60 degrees. A displacement transducer manufactured by Fugro McClelland is used for depth monitoring purposes.

An on-site data acquisition system placed inside the vehicle provides real-time data monitoring of the test. The data acquisition system consists of a signal conditioning unit, a Compaq portable III micro computer, and a data translation board. A data acquisition software coded in Pascal is used for storing the data, analyzing the data, and displaying the results in the graphical format.

3.2 Field Testing

Several cone penetration tests were conducted on the Loess bluff located near Waterways Experiment Station in Vicksburg, Mississippi. The ground water table is located 25 m below the ground level. The research vehicle was used for conducting these tests. The research vehicle was taken to the top of the slope and located 10 m away from the edge of the slope. Figure 2 depicts a schematic of the testing in the field.

Four cone penetration tests were performed on the Loess bluff using three different penetration rates: 0.25 cm/sec, 1 cm/sec, and 2 cm/sec (2 tests at the standard speed of 2 cm/sec). The spacing between test locations was varied between 1 and 1.5 m. Cone test
soundings were conducted for depths ranging from 10 m to 20 m. Figure 3 presents one of the typical cone test results conducted with a standard speed of 2 cm/sec.

3.3 Block Sampling

Approximately 0.03 to 0.04 m³ block samples of the soil were hand carved from the bluff. Block samples were taken at two different elevations as shown in theFigure 2. Samples taken at depths of 7 to 7.5 m are termed as WES 1 samples. Those taken at 0 to 0.5 m are known as WES 2. A total of thirty-five rectangular block samples (plan dimensions are approximately 25 cm by 15 cm) are collected and carefully placed in plastic bags. These bags are stored in plastic thermoses and then transported to the laboratory for strength tests. In the laboratory, the thermoses are kept in a humidity room in order to preserve their natural moisture contents.

Cylindrical samples of sizes close to 71 mm are carefully trimmed from the block samples. Several samples were lost during this process. Both unconfined and drained triaxial tests were conducted on these samples. These results are used in the evaluation of developed correlations from the chamber studies.

4 ANALYSIS OF TEST RESULTS

4.1 Basic Soil Properties

The loess deposit is a uniformly graded soil with a mean diameter size of 0.018 mm. The specific gravity of the soil is 2.70 and the initial degree of saturation is 50%. Block samples from both depths contain 75-80% of silt and 20-25% of clay.

![Figure 2: Schematic of Cone Testing Setup in the Field](image)

![Figure 3: Cone Penetration Test Results](image)
4.2 Strength Properties

Samples from both depth locations are subjected to consolidated drained triaxial tests. The following table provides a summary of peak and residual strength parameters.

Consolidated Drained triaxial (CD) test results showed that the WES2 samples exhibit dilation behavior at low confining pressures. This indicates that these samples are over consolidated at shallow depths closer to ground surface. On the other hand, the WES1 samples displayed contraction behavior at all confining pressure levels.

The CD tests were conducted on fully saturated samples. Full saturation is generally known to soften the cementation bonds. Hence, these tests may not provide true cohesion parameters. Therefore, unconfined compression strength (UCS) tests are conducted on the samples at natural moisture content conditions. Both test results on WES1 and WES2 samples indicated that the unconfined compression strength of both samples are the same and 125 kPa. Additional tests were conducted to ensure repeatability which reconfirmed these strength values. According to geological classification as reported by Rad (1984) based on UCS values, the loess deposits can be termed as weakly cemented. The variation between cohesion values from triaxial and UCS tests indicate the influence of partial saturation effects which is known to induce apparent cohesion intercept in soils.

4.3 Cone Test Results

Four cone tests conducted with three different penetration rates (presented in Table 2) showed that the change in penetration rate has not affected the tip and sleeve friction values. Based on this observation, it can be assumed that drained conditions prevail during cone penetration in these deposits. Otherwise, the tip resistance varies and depends on penetration rate.

5 COMPARISONS BETWEEN FIELD AND LABORATORY CHAMBER TEST RESULTS

A laboratory calibration chamber study was conducted to calibrate a friction cone penetrometer to identify the cementation effects on sand (Pappala et al. 1995). In this study, cemented sand specimens prepared at three ranges of relative densities (D_r) 65 - 55%, 65 - 75%, and 85 - 90% were tested at three levels of confining pressures such as 100, 200, and 300 kPa. A miniature friction cone penetrometer was used in order to reduce boundary effects. This large scale laboratory study provided several conclusions on the effects of cementation on cone resistances in sands. These conclusions need to be validated by the present field test results.

It should be noted that there are several differences between laboratory chamber test conditions and field test conditions. In spite of these differences, it is still planned to conduct comparison studies between laboratory and field test results in an attempt to investigate whether the correlations and conclusions derived from chamber studies on sands are valid for field study results on loess deposits.

5.1 Cementation and Confinement Effects

One of the conclusions from the chamber tests is that the effect of cementation on cone tip resistances is more significant at shallow depths than at larger depths. In other words, the cementation effects dominate the confinement effects on the cone tip
resistance values at shallow depths. This observation also agrees with the field test results.

The field cone test results showed that, for the first layer of 0-7 m, an average cone tip resistance of 50 bars is obtained. This value remains constant with respect to depth up to 7 m showing no effects of confinement at shallow depths. In the second layer of 7-15 m, cone tip resistance values of 15-20 bars are recorded. The effect of confinement on tip resistance at these depths is not noticed until the depth of 11 m. Beyond 11 m, the confinement increases the tip resistance. All four cone tests showed the same trend.

From Table 1, the cohesion intercept and the friction angle at WES1 (deep) are smaller and larger in magnitudes than the same at WES2 (shallow) depths. The small friction angles and high cohesion intercepts mean that the tip resistances depend more on the cementation than on the confinement in these deposits. This observation agrees with the chamber study conclusion. More field studies are still necessary for further investigating the chamber study conclusion.

5.2 Evaluation of Theoretical Models

A bearing capacity theoretical model developed by Janbu and Senneset (1974) was evaluated in chamber studies in the interpretation of cone tip resistance. This model is re-evaluated here for the interpretation of field study results.

The J & S theory uses a bearing capacity parameter, $N_q$, which depends on plastification angle, $\beta$. The plastification angle which represents the angle to the zone with which the failure plane fans out from the horizontal, depends on the compressibility and density of soils. At present, this angle is selected arbitrarily without any formalism. Puppala (1993) derived the following relationship between this angle and the dilation angle ($\psi$) measured from triaxial tests using chamber test results.

$$\beta = 1.43 + 1.38$$

The validity of the above relationship is evaluated by comparing field predictions with interpreted relationships. Triaxial test results (Table 1) and dilatational angles measured from the triaxial data are used to determine tip resistances at different depths. The loess deposits at larger depths exhibited a contraction behavior. An assumption is made that the above equation is identical on contraction side and then the same relationship can be used to estimate plastification angle by changing the sign convention of the final calculated angle. The calculated tip resistance by using this approach is plotted against measured values in Figure 4.

Figure 4 shows that the predicted tip resistance is lower than the measured tip resistance within the first layer of 0-7 m. Then, it matches well with the results from the second layer, 7-15 m. Several explanations can be given for the variation in the resistances for the top layer:

1. Strength parameters for the top 7 m were obtained by testing field samples at 100, 200, and 300 kPa. These pressures are quite high compared to the overburden pressures generated at these depths. Therefore, tests at these pressures may not provide realistic strength properties.

2. The partial saturation effects on the cohesion intercept are not accounted for since triaxial tests were conducted on fully saturated samples.

3. Full saturation may soften the cementation bonds, and hence the cohesion measured from triaxial tests can be quite small in magnitude.
Figure 5: Comparisons Between J & S Theory Predicted and Measured Cone Tip Resistances Using Modified Strength Properties

4. Though the block sampling technique is used, the sample can be still exposed to disturbance effects from sampling, trimming, and other handling related disturbances.

Prefabricated loess samples tested at low confining pressures along with previously tested triaxial data have yielded a modified cohesion intercept of approximately 150 kPa. Using this value, the theoretical model has provided a better agreement with the measured results for the top layer (Figure 5). This analysis clearly indicates the importance of including partial saturation effects in the cemented soils in the determination of strength properties. Otherwise, strength and cone test parameters will be underestimated.

Another major observation is on the inability of friction cone tests to identify various cementation levels in soils. This is not evaluated in the field study since all cone tests are conducted on the same cemented loess deposit (with similar cementation levels).

The present field tests, though not many, nevertheless showed the importance of including field test results in the development of interpretation methods for cemented soils. Further field tests on other cemented sites will evaluate theoretical models and correlations from the chamber study in their evaluation of strength properties of cemented soils. They can also complement the laboratory test results in developing correlations.

6 SUMMARY AND CONCLUSIONS

This paper presents cone test results from a field study conducted on loess cemented soil near Vicksburg, Mississippi. Few conclusions are noted from this study. These are:

1. The cementation effects on cone test results are more significant at shallow depths than at larger depths.
2. All four field cone penetration tests with different penetration rates showed the same trends and tip resistance magnitudes. This can be attributed to the drainage conditions prevailing around the cone peacimeter.
3. Block sampling still provided samples with significant amount of disturbances in weakly cemented soils.
4. The J & S theory based on triaxial tests conducted on fully saturated field samples provided tip resistance interpretations which are different from measured cone test values at shallow depths. The predictions are close to measured values when modified strength parameters by accounting for partial saturation effects are used in the interpretations. This conclusion signifies the need to either include or differentiate the partial saturation effects along with cementation effects on cone test results.

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Some theoretical and practical problems of CPT use

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ABSTRACT: Some results of theoretical researches which essentially influenced the development of a CPT method in Bashkortostan Republic (Russia) are generalized in the given paper. The suggested theoretical dependence connects the CPT results with the standard properties of the soil considered as elasto-plastic medium. This dependence explains a variety and some discrepancy of the existing empirical dependences. The principles of combined use of CPT with other methods of soil research are developed. They are based on the application of the mathematical statistics regularities and Bayes rule. It is shown that the CPT effectiveness is in the greatest measure exhibited when carrying out a great number of tests at a site with this method.

1 INTRODUCTION

In opinion of many researchers CPT is purely empirical method. CPT data is used for solution of quite different problems including the evaluation of shear strength, deformability and discernment of lithological soil types. In this case the physical sense of the empirical dependences used remains not enough clear. There is a problem also, whether the low accuracy of soil performance evaluation from CPT data can be compensated with its speed and cheapness. Results of these problems study with reference to the most typical cases of mass construction are given below.

2 INTERPRETATION OF CPT DATA AND THEIR USE WHEN FOUNDATIONS CALCULATION

In paper (Pushkova 1973) approximated solution of a problem about a penetrometer pushing into elasto-plastic medium with the modulus of deformation \( E \) and shear parameters \( \phi \) and \( c \) was obtained. It was assumed that the influence of pushing of the penetrometer tip into the above medium is equivalent to expansion of a cylindrical hole from a radius \( r_0 \) up to \( r_e \) (\( r_e \) is a penetrometer radius). The dependence obtained was represented as:

\[
q = \beta \psi r_e^2 \left( \frac{E}{\gamma} \right)^{1-\c} \times \\
\nu = \left( \frac{E}{\gamma} \right)^{1-\c} \times \\
\left[ \frac{(l + c)c}{2\sqrt{c(l + 2c)^2 - c^2}} \right]^{-\c} - 1 \right] \frac{1}{1g\psi}, \quad (1a)
\]

\[
\beta = \frac{t_\phi}{1g\sigma_t} + \phi, \quad (1b)
\]

\[
\tau = \sigma_t 1g\psi + c, \quad (1c)
\]

\[
\zeta = \frac{1g(\frac{E}{2} - \frac{\phi}{2})}, \quad (1d)
\]

where \( \phi \) and \( c \) - angle of internal friction and specific cohesion respectively; \( \sigma_t \) - in situ pressure (pressure in the soil at a considered depth before the penetrometer pushing); \( E \) - modulus of deformation; \( \alpha_t \) - half an angle of a cone point.

Graphically the dependence \( \psi \) on \( \phi \) and \( E/\gamma \)
Fig. 1. Dependence of $\psi$ on $\varphi$ and $E/\varphi$

can be represented with a set of curves shown in
figure 1. It was accepted $\psi' = \psi/\varphi = \psi'$, that
according to paper (Panascoa 1973) does not lead to
large errors. The Poisson’s ratio $\nu$ because of the
relative simplicity of its prediction was taken as a
constant. The static resistance coefficient (the ratio
of the vertical pressures in the soil to horizontal
pressures before the penetrometer pushing) was
taken equal to 1.

The similar model of an elasto-plastic medium
was used by A.S. Vesic (Vesic 1965) who obtained
the relationship similar to (1) considering the expan-
sion of a cylindrical hole. A.S. Vesic expressed it a
little differently (with a binomial formula) and used
not $E/\varphi$, but $G/\varphi$ ($G$ is the modulus of shear
deformation) that was principally not significant.

The value $\beta$ in formula (1) characterizes the
transition from the stresses on the walls of the
imaginary cylindrical hole to the cone resistance.
The above value reflects the soil behaviour in a zone
of the greatest normal stresses (at a contact with a
cone surface). In this zone the normal stresses by an
order of their value are close to soil resistances $q$
obtained. With such stresses clayes become an ideally
cohesive medium where $\varphi = 0$, that according to
(1a) corresponds to $\beta = 1$. Such an assumption is
unfair for sands. The deviations from the Coulomb
law are found out in sands at very high normal
stresses (usually more than 10 MPa). With a stan-
dard angle of a cone point $60^\circ$ ($\alpha_i = 30^\circ$) the values
of $\beta$ in sands and gravel at $\varphi = 26, 30, 35, 40^\circ$
will be $\beta = 2.5, 3.0, 4.0; 4.8$ respectively.

According to formula (1) the specific resistance $q$
characterizes simultaneously both shear strength
and deformability of the investigated medium. The
numerous combinations $\varphi, c, E$ can correspond to
one and the same value $q$. However, the real soil per-
formances ($\varphi, c, E$) are not independent accidental
values. They are combined with a mutual correla-
tion, the form of which is defined with a lithological
type and a soil genesis. It is confirmed, in fact, with
a long-term effective use of various tables reflecting
the correlation between soil performances. Such tab-
bles are presented, for example, in Russian codes on
foundations design CHInTeI 2.02.01-83* where from a
porosity coefficient $\varphi$ and a flow index $E$, of clayey
soils, values of $\varphi, c$ and $E$ are found. In codes
CHInTeI 1.02.07-87 there are tables reflecting the em-
pirical combination between CPT resistance $q$ and
performances $\varphi, c, E$ of the most widely spread
soils. For clays (except glacial and lake-glacial) this
connection is shown in table 1 and for sandy soils
- in table 2. The sandy soil performances correspond
to a depth of 5 m. Modulus of deformation $E$
in brackets corresponds to alluvial and fluvo-glacial
soils, the values out of brackets correspond to all
other types of soils. The tables show that to each $E$ value
there corresponds its own $\varphi, c$ as well as to each $c$
value correspond some definite values of $\varphi$ and $E$, etc.
Approximately the same relations of $\varphi, c, E$ are re-
ceived when averaging the above tables data of
CHInTeI 2.02.01-83*, where the empirical relations
are more differentiated (clays and loams of different
origin are considered separately).

| Table 1 |
|-----------------|-----------------|-----------------|-----------------|-----------------|
| Empirical relation | Theoretical calculation by formula (1) |
| $q$ MPa | $E_i$ MPa | $c_0$ MPa | $\varphi_0$ | $q_0$ MPa |
| 1 | 7 | 0.024 | 17 | 1.1 |
| 2 | 14 | 0.036 | 19 | 1.9 |
| 3 | 21 | 0.047 | 22 | 2.8 |
| 4 | 28 | 0.058 | 24 | 3.5 |
| 5 | 35 | 0.070 | 26 | 4.4 |
| 6 | 42 | 0.082 | 28 | 5.2 |

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Results of CPT resistances $q$ calculation by formula (1) for given relations of performances $\varphi$, $c$, $E$ are shown in tables 1 and 2. As one can see from the above tables the coherence between the theoretical and empirical values $q$ is quite satisfactory.

On the bases of the relations between the performances of the definite type of soil and formula (1) one can receive for such soil the approximated dependences (formulas and tables) relating the resistance $q$ with each of the mechanical performances, i.e. $E = f(q)$, $\varphi = f(q)$, $c = f(q)$. Depending on the above relations (that can be approximately characterized with the value $a = E/c\varphi$) such dependences can differ from the dependences in tables 1 and 2. For example, the dependences (1) can be transformed proceeding from the assumption that the value $a = E/c\varphi$ is known:

$$q = \beta \psi \alpha - \beta \psi (E/c)$$

Then

$$E = \alpha (\beta \psi) E - kE \quad (2)$$

If the most typical values $\alpha$ and $\psi$ for the definite soil type are substituted into the approximate dependence (2), the simplified formulas conforming to the known empirical dependences are obtained. So, for sandy soil the formula (2) will be $E = (2.5\ldots 3)q$, for clays and alluvial loams $E = 5.6\ldots 7.4q$, for glacial loams $E = (6.5\ldots 8.5)q$. Obviously similarly it is possible to deduce from the formula (1) the simplified dependences for $\varphi$ and $c$.

One more transformation of the formula (1) seems to be useful:

$$q/\psi = \beta \psi$$

Instead of $\tau \alpha$ it is possible to use $\psi \alpha$ - local side friction of a penetrometer that is close to $\tau \alpha$ by an order of its values. The influence of radial stresses, exceeding the in situ pressure $\alpha \varphi$, on the friction sleeve is compensated with the imperfections of the friction sleeve contact with the soil and the soil structure disturbance. A scattering diagram of the values $\psi \alpha$ and $\tau \alpha$ in alluvial and fluvio-glacial clays and sands is given in figure 2.

Fig.2. Interconnection between $\tau \alpha$ and $\psi \alpha$

The figure confirms that $\psi \alpha$ is close by its value to $\tau \alpha$, in fact in sands it can be significantly less. Thus:

$$q/\psi = \beta \psi \quad (3)$$

The product $\beta \psi$ in sands and gravel should be 45...300, in clays and loams 15...25, i.e. significantly less. This explains the empirical fact that "the friction index" $I_f = q/\psi$, can be used for differentiation of sandy and clayey soils. In practice in Bashkortostan it is most often taken that a condition $I_f > 50$ corresponds to sandy soils, and $I_f \leq 50$ corresponds to clays and loams.

3 CONDITIONS OF CPT EFFECTIVE USE

The evaluation of any soil performance (both from CPT data and any other method data) is connected with the errors of two types. The errors of particular values of this performance because of the used for-
CPT USE IN COMBINATION WITH OTHER METHODS OF SOIL STUDY

In complex engineering-geological investigations the most complicated step is the estimation of design values of the soil unknown characteristics. The results of each of applied methods of soil investigation differ from the results of other methods both by value and by their reliability. In conditions of a real soil heterogeneity some separate tests characterize different site points with the soil performances, characteristic of just these points. In practice, in most such cases a problem of design characteristics choice is solved with simplification. The most reliable method is preferred, all other methods to which CPT often belongs becomes a sort of insurance of safety. This leads to not sufficient use of the information obtained. The more effective seems to be the other approach based on use of rules of probabilities theory. The most attention deserves the use of Bayes rule. This rule is intended for the overestimation of the probabilities of various hypotheses \( H_i \) after the occurrence of the event \( \alpha \) with the known conditional probability \( p(\alpha | H_i) \). i.e. probability, provided that one of \( \alpha \) considered hypotheses \( H_i \) is true:

\[
p(\alpha | H_i) = \frac{p(\alpha) p(H_i | \alpha)}{\sum_{i} p(H_i) p(\alpha | H_i)}
\]

(4)

where \( p(H_i) \), \( p(\alpha | H_i) \) - probabilities of \( i \)-th hypothesis before and after the events \( \alpha \) achievement, respectively, named a priori and a posteriori.

The approach developed in Balsilnitriov is based on considering the possible errors of a mean value of the unknown soil performance as a priori hypotheses, and actually obtained errors of some definite tests at the site as the events \( \alpha \). The probabilities of possible errors (in kind of the approximate values to accurate ones ratio), corresponding to \( p(H_i) \) and \( p(\alpha | H_i) \) are evaluated from their relative frequencies by means of the archival data analysis. They are represented as discrete distributions. The points of CPT are placed more or less uniformly along the whole site and in the most typical places called “key sections” the accurate tests are carried out. The result of the exact definition of the unknown soil property index (for instance, \( \varphi \), \( c \), \( E \)) or pile bearing capacity \( (F) \) is compared with the approximate one (\( \varphi' \), \( c' \), \( E' \) or \( F' \)) defined from CPT at the same point. By histogram of conditional prob-

mulus and tables inaccuracy and in some extent because of the instrument errors refer to the first type. The errors of transition from the particular values to the design ones that reflect the soil heterogeneity influence refer to the second type. The latter is due to the limit number of the measurements (tests). The both types of errors are capable to essentially distort the finite result. That’s why the only increase of the accuracy of the unknown performance particular values evaluation without increase a number of such evaluations doesn’t solve a problem of foundations design reliability and efficiency increase.

In paper (Pavonno 1987) a problem of separate tests reliability and their number at different soil heterogeneity was theoretically considered. Each test was considered from a position of information theory as a means of decrease of an uncertainty (“entropy”) of spatial distribution of the unknown performance particular values within the site and as a means of this performance mean value errors decrease. The errors of mean value showed simultaneously the influence of separate tests accuracy and soil heterogeneity. The results obtained in both cases showed the sufficient number of the approximate evaluations to be capable by its informativity to exceed not numerous exact tests. This number was the less, the exact were the evaluations themselves and the more was heterogeneous the soil. Within the accepted model, for example when evaluation of modulus of deformation at site with an area of several thousand m² 1-2 exact tests with a stamp was equivalent in soils of medium homogeneity approximately to 10..15 points of CPT. With great heterogeneity the informativity of “exact” tests decreased so that such number of “exact” tests gave as little information as 5-8 CPT points. Though the above quantitative relations characterize the idealized conditions, they show some important circumstances from the practical point of view. For instance, they show that advantages of CPT (as any other express-tests) are exhibited in full measure only with its wide use within the site. The result of such test taken separately, usually is not very exact, but the set of a large number of such results can characterize the geotechnical properties of the site even better than not numerous “exact” tests. At the same time, the expensive “exact” tests sufficiently decrease their information value if they are performed not in complex with such express-tests as CPT.
abilities of the error values the conditional probability of an error (for example, $E/r$) at the "key section", i.e. an event $\alpha$ is evaluated. By means of calculations by Bayes rule the new (a posteriori) probabilities of the mean errors (for example, $E/r$) are evaluated. In case of several "key sections" the calculation is done successively for each of them. In this case a posteriori probabilities evaluated at the previous key section are taken as a priori probabilities for the following one. After the calculations at all key sections one or another fiducial probability (usually 0.95) is taken and the choice of the error corresponding to this probability is done. It will be a correction factor to the mean value of performance obtained from CPT data ($E$ or $F$). As an example the correction factors $\gamma$ calculated with the above method for the modulus of deformation and cast-in-place pile bearing capacity are given in Table 3. It is taken that the approximate values of modulus of deformation ($E$) and pile bearing capacity ($F$) are evaluated in each point of CPT from tables and formulas according to Russian codes. The correction is done by data of one accurate test: while evaluation $E$ it is a soil test with a stamp, while evaluation $F$ it is in situ pile dead load. At the key section the relation $K=E/E$ or $K=F/F$ is estimated and with its help the factor $\gamma$ is defined. Then it is supposed to divide the mean approximate value $E$ or $F$ by this factor.

The approach given in the paper doesn’t make high demands of the formulas exactness for soil parameters evaluation. It is only important to relate them to values taken as exact ones. The use of Bayes rule may be useful in more complicated case when not only CPT is considered as an approximated method but the whole complex of tests on a definite site as well (Ryzhkov 1995).

The practice has shown the above approach proves to be especially effective when pile bearing capacity evaluation when static pile tests serve as a reference. According to researches conducted in BashNIIstroj the above approach allows to decrease pile foundations cost in average by 10% without the reliability decrease compared to the method accepted in Russian codes CНИИ 2.62.0.3-87. The Russian codes in most cases are guided by the "worst" result of static pile tests.

5. Conclusions

1. The investigations have shown the CPT should not be considered as purely empirical method. The interconnection of its results with the standard soil property indices has the theoretical explanation. Taking this explanation into account one can increase the efficiency of practical work.

2. The existing empirical dependences are in great measure private cases of more common regularity resulting from the solving of a problem on soil penetration into an elasto-plastic medium. The dependences can be only approximated as they reflect the influence of one or another part of factors defining the soil resistance to CPT. However, the inaccuracy of formulas or tables used is compensated with a large number of points of site investigation, with more accurate account of soil performances variability within the site.

3. The errors of a systematic character can be eliminated with the correction of results from not numerous "accurate" tests data. Such a correction can be based on the probability theory rules and can provide the most efficient and reliable decisions.

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<table>
<thead>
<tr>
<th>Tab. 3</th>
<th>The value estimated.</th>
<th>Factor $\gamma$ with $K$ equal to</th>
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<td></td>
<td>0.60 0.80 1.00 1.20 1.40</td>
</tr>
<tr>
<td>E</td>
<td>0.78 0.96 1.14 1.23 1.30</td>
<td></td>
</tr>
<tr>
<td>r</td>
<td>0.84 1.03 1.20 1.31 1.34</td>
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</tr>
</tbody>
</table>
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Comparison between CPTU data, mechanical and consolidation behaviour of soils – Statistical and mathematical analysis

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1. Introduction

The present work is based on geotechnical surveying carried out principally all over the Eastern Po Valley in Emilia Romagna, in the context of realizing the geological map of the plains of Emilia Romagna (mostly those between Bologna and Ravenna) and near Venice in order to achieve geotechnical data for landfill sites. The area of main concentration of CPTU tests and bore-holes is shown below.

The samples taken from bore-holes drilled in the same site of some CPTU tests were examined with conventional laboratory tests, mostly classification and oedometeric tests. Only a few CD triaxial tests were run on a site to the South of Venice.

2. Equipment

CPTU tests were run with the following equipment:
- three static piezometers having 250 to 300 kN of nominal thrust with 2 cm/s sounding velocity;
- standard piezcone (according to ISMFIE specification) with inclinometers allowing correction of the depth according to the deviation of rods while sounding;
- acquisition system with higher A/D conversion, with particular methods for applying calibration curves to all sampled values. Since the beginning of the work (May '95) it has been necessary to unify CPTU data obtained from different kinds of cones. The acquisition system (basically data loggers) has therefore been programmed to acquire and memorize pure electrical values (due to strain-gauges, pressure and inclinometer sensors).

These values have been constrained via software to follow the exact calibration curve (previously obtained in the laboratory) before being shown on the computer screen. For this reason, since the coefficients derived from the calibration curve have been applied during each test, we can state that all the different data conform. CPTU tests reached depths from 20 to 34 m from ground surface (on average). Before each test a 1m-deep pre-hole saturated with water was drilled and the filter (in systersized metal of constant permeability and saturation behaviour) was changed. For a significant number of CPTU tests “very-long-litting” dissipation tests (minimum two or three hours) were run in different kinds of soils, as well as in soil with high expected OCR (over consolidation ratio). In some tests U detection was very frequent, occurring every meter of depth; this was in order to determine the shape of the dissipation curve in the first 60 seconds in clayey soils as well as the aquifer groundwater pressure for calibrating the U reference line.

they are located in the North-East of Italy between the Apennines to the South and the Adriatic Sea to the East.

Over 450 CPTU tests were performed from May '95 to December '96 in characteristic sites, some of which were in the proximity of bore holes where some undisturbed samples were taken with thin-wall samplers (Osterberg). At present (July '97) the work is still in progress with more piezocone and dissipation tests around Ravenna and, on the other side of the Po River, around Venice, which will total over 1000 CPTU tests.

The CPTU tests in Emilia Romagna have been conducted in order to determine the stratigraphy of the first 55 m along with some geotechnical and hydrogeological characteristics like permeability and soil trend of the first aquifer.
3. In-situ tests—description and comments

In the geologically similar sites the repeatability of CPTU tests for all characteristic values (Q, P, U) was basically verified in two ways:
- by comparing all the average values concerning the same layer in different CPTUs.
- by verifying, when possible, the reliability of soil classification obtained with penetrometer data compared to that of continuous-core drilling.
The repeatability of Q average values was found to be very reliable, given that all the graphs were similar and perfectly superimposable within the same site for the same layer.
The typical stratigraphy of the studied soils is represented by clay and silty clay, generally overconsolidated everywhere. In the coastal areas, on the contrary, below a shallow sand layer (2-12 m of thickness), there is a sequence of normal-consolidated clay layers alternating with sandy layers which are frequently not at hydrostatic pressure (mostly under-pressured). Furthermore, in the case of overconsolidated clays above the gravel, even if the Qc cone resistance was relatively low (between 0.07 to 1.5 MPa) the values of U were close to zero or slightly negative (even -0.08 to -0.11 MPa). The “risk” of having zero or negative ΔU values during sounding due to loss of silicon-oil saturation of the filter was avoided by drilling deeper pre-holes (always filled with water), and in many cases through to the groundwater saturation level (normally from -1.5 m to -3 m in these sites). It becomes more difficult to analyze the reliability of U detection in the above-mentioned layers of OC clays; the U variation during penetration is in fact sometimes very low and ranging between 0.1 and 0.2 MPa close to zero. In these cases the analysis must be performed with a statistically valid number of tests, neglecting those differing from others done on the same site. In OC clays or silty clays the “traditional” approach of relating soil classification and permeability and/or compressibility parameters to combination of penetrometric data is not very easy to apply, due to the fact that a great number of lab-tests in these soils does not normally exist for comparison. In addition, low or negative U values during penetration may sometimes lead the operator mistake it for a malfunctioning of the equipment.

4. Comparison with lab-tests and correlation in NC soils

Since we had at our disposal a wide range of data in NC cohesive soils, including laboratory oedometer tests at the same depth of dissipation, and once we had verified that all the data concern comparable soils, a statistical analysis was run searching for correlation between CPTU and dissipation data and compressibility/permeability parameters.
First of all it is important to define the parameters that describe a dissipation curve, as shown in the following figure:

\[ \Delta U = U_{	ext{max}} - U_{	ext{ref}} \]
\[ V_p = \text{velocity of } U \text{ variation starting from 75\% of normalised dissipation to 50\%} \]
\[ V_v = \text{velocity of } U \text{ variation starting from 100\% of normalised dissipation to 75\%} \]
\[ V_m = \text{velocity of } U \text{ variation starting from 100\% of normalised dissipation to } U_{	ext{ref}} \]

for “velocity” considering \( \Delta U/\Delta t \) in normalised dissipation curve.

By considering NC (or slightly consolidated) cohesive soils, with a comparison between 81 indisturbed samples lab tests with as many dissipation tests a significant result was obtained plotting \( \Delta U/\Delta t \) in dissipation tests vs. \( Q_p/\rho_i \) in a logarithmic graph with:

- \( n = \) hydrostatic pressure at the average depth of the layer
- \( U_{	ext{ref}} = U_{	ext{max}} - U \) during dissipation test
- \( Q_p = \) average point resistance in that layer
- \( \rho_i = \) preconsolidation pressure (from oedometer tests)

Since the \( \Delta U \) in NC soils is normally limited, the connection between parameters can be expressed with the following logarithmic correlation:

\[ Q_p/\rho_i = 0.736 \log(\Delta U/\Delta t) + 9.137 \]

Plotting \( \Delta U/\Delta t \) vs. OCR in the same NC soils the following logarithmic graph is obtained.
having row the correlation:

\[ \text{OCR} = 0.203 \ln(\Delta U/V_{100}) + 1.7337 \quad (R^2=0.74) \]

Once an average compressibility coefficient \( m \), \( m = (m_1 + m_2) / 2 \), has been defined with \( m_1 \) and \( m_2 \) coefficient calculated in the first part of oedometer e-log(p) curve (loads< \( p_1 \) ), and \( m_2 \) coefficient calculated in the second part (loads> \( p_2 \) ), a probable correlation has been found with \( V_{100} \) in dissipation tests (as defined above), as shown below:

Thus the correlation between an average value of \( m \), \( [\text{MPa}] \) and \( V_{100} \), [s] (as calculated in normalised \( U \) vs.time dissipation) can be expressed with:

\[ m = 0.0011 (V_{100})^{0.71} \quad (R^2=0.71) \]

The coefficients (0.0011 and 0.47) can vary slightly according to the number of correlated tests. Furthermore, the results confirm that there is a good correlation between \( T_{90} \) and permeability \( K \) (as calculated in oedometer tests), even if the range of permeability of the tested cohesive soils \( (10^{-12} \text{ to } 10^{-9} \text{ cm/s}) \) was too narrow to be properly correlated to the dissipation data: we can only state that the higher the \( T_{90} \) the lower is permeability.

5. - analysis in OC soils

On the other hand, when analysing the OC soil dissipation curves, it is necessary to distinguish the first part of the disscurve (in which \( U \) is rising from a start value to the maximum value) from the second part of the curve. For this reason all the dissipation curves in OC clays were replotted, splitting the first part from the second to allow analysis of the second part by conventional means. Consequently the starting time of the new diss test was shifted to \( T_{90} \), corresponding to the time the curve starts to drop.

The superimposition of normalised \( U \) graphs in the descending part of the disscurve shows that only a low percentage of curves reach the flex and \( T_{90} \). This means that, even if the dissipation tests were long-lasting (up to 6 hours), a complete definition of a "conventionally"-shaped curve would have required a longer time to develop. In these cases, for example, the known correlation of permeability with \( T_{90} \) for NC soils can probably be applied, but it seems more correct to subtract the time from the \( T_{90} \) value the time required to reach its maximum value.

On analysing a group (n=35) of dissipation tests run in OC clays in which dynamic U was negative and with relatively low \( Q \) values (as described in §3) some significant results are obtained.

A correlation has been found plotting \( \Delta U/V_{100} \) in dissipation tests vs. \( Q/Q_0 \), where:

\( Q \): hydrostatic pressure at the average depth of the layer

\( Q_0 \): effective pressure at that depth (\( p_s \cdot 1.9 \text{ g/cm}^3 \));

In the same dissipation tests a comparison has been made between the velocity of \( U \) variation in the first 60 seconds (\( V_{100} \)) and the velocity with which \( U \) reaches its maximum value starting from 100 (\( V_{100} \)).

A direct correlation was found, which implies that, generally and in such kind of soils a \( U \) vs.time measurement can be useful even in the first 60s, for predicting \( U_{max} \) as shown in this graph:
6. Numerical approach

The numerical approach to the problem was first used to validate the experimental data (in situ and in laboratory) and therefore to explore some theoretical dissipation curves with the variation of typical soil parameters, both in NC (above all for validation) and in OC clays (for the analysis).

6.1 Governing equations

It is assumed that a pore fluid pressure \( p \) causes only a uniform volumetric strain by compressing the grains and that the major deformations of the porous skeleton is governed by the effective stress \( \sigma' \)

\[
\sigma' = \sigma - n_p \sigma_p = [1 \ 1 \ 0 \ 0 \ 0]
\]

where \( \sigma' \) is the total stress. The constitutive equation relating the effective stress to the strains of the skeleton is now independent of the pore pressure \( p \) and for a general non-linear material can be written in a tangential form, thus allowing plasticity to be incorporated. Hence, neglecting creep strains,

\[
d\sigma' = \mathbf{b}(\mathbf{d} - \mathbf{d}_0 - \mathbf{d}_c)
\]

where \( \mathbf{d} \) represents the total strain of the skeleton and

\[
\mathbf{d}_c = \mathbf{m}(\mathbf{d}_p / \mathbf{K}_c)
\]

represents the overall volumetric strains caused by uniform compression of the particles by the pressure of the pore fluid, with \( \mathbf{K}_c \) being the bulk modulus of the solid phase; finally, \( \mathbf{d}_0 \) indicates the "outogeneous" strains and \( \mathbf{D} \) is the tangent matrix depending on the considered constitutive model.

The general equilibrium equation can be written, starting from the principle of virtual work, as

\[
\int \sigma' \delta \mathbf{d} - \int \mathbf{b} \delta \mathbf{d}\delta t - \int \mathbf{f} \delta \mathbf{u}\delta t = 0
\]

\[(4)\]

where \( \sigma' \) is the total stress, \( \delta \) are the virtual forces, \( \mathbf{D} \) the boundary tractions, and \( \mathbf{b} \) the displacements. Considering this expression in incremental form and taking into account the constitutive relationship (2), we obtain

\[
\dot{\mathbf{d}} = \mathbf{m}(\mathbf{d}_p / \mathbf{K}_c) + \mathbf{b}(\mathbf{d} - \mathbf{d}_0 - \mathbf{d}_c) / \mathbf{D}
\]

\[(5)\]

6.2 Elasto-plastic model

The theory relates stress and strain increments for a general elastoplastic material in which strain hardening, as a function of volumetric plastic strain, is allowed for. Here the Mohr-Coulomb yield surface is considered.

Mohr's theory of failure involves the construction of an envelope to all possible circles of stress that can be drawn for a particular problem. Coulomb failure criterion states that there is a linear relationship between the shear stress \( \tau \) at failure and the normal stress \( \sigma_n \)

\[
\tau = c + \phi \sigma_n
\]

where \( c \) is the apparent cohesion and \( \phi \) the angle of internal friction.

6.3 Fluid phase behaviour

The fictitious seepage velocity (Darcy velocity) is defined as

\[
\mathbf{u} = -\frac{1}{k} \nabla p
\]

\[(7)\]

where \( k \) is the absolute permeability matrix of the medium, \( \mu \) the dynamic viscosity of the fluid, \( p \) the fluid pressure, \( \rho \) the density, \( g \) the gravitational acceleration, and \( h \) is the head above some arbitrary datum.

The case of only water flowing at saturated conditions is considered here; the continuity equation of water becomes

\[
\nabla \cdot \begin{pmatrix} \mathbf{u} \\ \frac{1}{\phi} \end{pmatrix} = -\frac{1}{k} \nabla p
\]

\[(8)\]

where the index \( w \) refers to water and \( \phi \) is the porosity.

6.4 Coupled solution

The fully coupled solution of the one-phase flow equation in an elastoplastic porous medium is treated. The continuity equation (8), together with the equilibrium equation (5.a), represents the governing equations for soil mechanics problems according to of Biot's self-consistent theory.

6.5 Boundary value solution

The equilibrium equations (5.a) have the boundary conditions already incorporated. As regards the continuity equation, the boundary conditions satisfy:

(a) the continuity of flow across the boundary

\[
\Pi_n = -n \cdot \nabla p - \rho_g \dot{q} = 0
\]

\[(9)\]

where \( n \) is the unit normal vector and \( \dot{q} \) is the outflow rate per unit area of the boundary surface; and

(b) prescribed pore pressures \( p_{\text{psw}} \).

The condition that the continuity equation (8) applies throughout the continuum and the condition (9) applies on the boundary requires that

\[
\int_{\Omega} \mathbf{b} \cdot \mathbf{d} \delta t + \int_{\Gamma} \mathbf{u} \cdot \mathbf{n} \delta t = 0
\]

\[(10)\]

where \( \mathbf{a} \) and \( \mathbf{b} \) are a set of arbitrary functions since \( \mathbf{X} \) and \( \mathbf{W} \) are identically satisfied throughout their respective domains.
6.6 Finite element solution

The finite element method is applied to equations (5a) and (10) in terms of displacements and pore pressures.

The final set of equations becomes

\[
\begin{align*}
\frac{\partial^2 \delta F}{\partial t^2} + 1.5 \frac{\partial \delta F}{\partial t} - f &= 0 \\
\frac{\partial \delta F}{\partial t} + \frac{\delta F}{2} + U \frac{\partial \delta F}{\partial t} - f &= 0
\end{align*}
\]

(11)

The integration of these equations usually requires the use of numerical techniques, and a standard method is that of Gaussian quadrature. Since the discretization in space has been carried out, eqs. (11) now represent a set of ordinary differential equations in time.

6.7 Numerical examples and simulations

Axially symmetric sections of some NC clay layers with the interaction of penetrometer are considered. The penetrometer is supposed to be fixed in the layer exciting a variable pressure according to the considered experimental data. The various OCRs are obtained by modifying in a proper way the cohesion and friction angle of the elasto-plastic law. A typical set of material data is shown in Table 1.

<table>
<thead>
<tr>
<th>Elastic modulus</th>
<th>4.5 MPa</th>
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<tr>
<td>Poisson ratio</td>
<td>0.25</td>
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<tr>
<td>Cohesion</td>
<td>0.63 + 0.005MPa</td>
</tr>
<tr>
<td>Friction angle</td>
<td>0.25</td>
</tr>
<tr>
<td>Permeability</td>
<td>$10^{-5}$ cm/s</td>
</tr>
</tbody>
</table>

*From bibliography

**From lab-tests

Fixed values of pore pressure (experimentally obtained for t = 0) are applied as initial condition to two nodes of the discretization corresponding to the penetrometer’s filter and then released to develop in time. The curves of pore pressure for each case are shown in the following graphs (pore pressure vs. time and o. vs. horizontal distance).

The first graph shows that the calculated dissipation curves fit with the "in situ" curve for the tested NC clay. The second graph shows the pore pressure U vs. radial space D calculation; the maximum value of U is within a distance of 10 cm from the cone; the minimum is over 100 cm. The U-D curve is orthogonal to the dissipation one; the same calculation in OC clays confirms this. The same analysis performed in OC clays with given known values of geotechnical parameters (obtained from correlation with Qc, Fy) gives the same confirmation of superimposability between in situ and math.curves, as shown below.

In the same case (OC clays) a different (qualitative, not comparative) analysis has been performed by varying both E and K in a wide range (E from 0.5 to 50 MPa and K from $10^{-9}$ to $10^{-7}$ cm/s) obtaining the following two graphs; these ones are not to evaluate K or E but only to show what the shape of the curve should be, according to the variation of the above mentioned parameters.

The evidence is that, the higher K is (and/or the less is E), the more the curve shifts to left, and viceversa, a wide decreasing of K produces the same effect of increasing the E modulus.
7.- Conclusions

In case of IC (OCR<1.6) clays the comparison between dissipation and oedometer data gives the following conclusions:

a) there is a correlation between the normalized U variation in the first step of the dissipation curve $\Delta U / U_0$ and overconsolidation ratio, both directly ($\Delta U / U_0$ vs OCR) and in relation to the cone resistance ($\Delta U / U_0$ vs $Q_p$).

b) a relation was found between $V_0$ as above defined and the coefficient of permeability $m_v$.

On the other hand, the mathematical simulation shows that:

c) referring to a given known layer where CPTU, dissipation and lab tests are available the envelope of the "in situ" dissipation curve perfectly fits with the theoretical curve, obtained with the input of known soil parameters (C, $\Theta$, $E$ as obtained from oedometer analysis);

d) $U$ vs time analysis with the parameters of clays with OCR<1.6 shows that in all cases the rising of U occurs in the period immediately after the cone has stopped (a few seconds). In theory only for clay with OCR<1 the dissipation curve should drop from the output ($t=0$).

This demonstrates that the U rising (even in soft soils) is not due to bad saturation; sometimes it may be that the short time from the push-stopping of rods and the start of acquisition for the dissipation curve corresponds to the rising time (few seconds).

In most of 95% of the observed cases the maximum value $U$ max reached during dissipations is higher than dynamic U (i.e. during pushing). This fits with the simulation because for every given starting value of $U$, it is not the minimum one reached during diss.test, even with a given combination of low cohesion values and very high $U$ values (as they were detected in very soft cohesive soils).

In the case of OC clays, analysing the dissipation tests run in all clayey soils marked by relatively low $Q_p$ values and very low or negative $U$ values during penetration, there is a correlation between $\Delta U / U_0$ vs $U_0$ and $Q_p$ ($\sigma$: effective overburden pressure). Furthermore, the velocity of diss. curve in the first minute was found to correlate to that from $t=0$ to the time corresponding to the maximum value of U. This makes it possible to obtain some data from a short dissipation, since $\Delta U$ normally takes hours to dissipate in OC clays.

The mathematical simulation also gives the following preliminary indications:

e) in an elastoplastic soil, while the cone is moving, the dissipation curve is orthogonal to that obtained considering the U variation in the space (at a given moment $t$);

f) by considering different values of horizontal permeability $K_v$ with given values of Q, F, and U, some parametric curves can be obtained. The superimposition of a field curve to mathematical one can give a range of $K_v$.

g) likewise, by varying the $m=U/E$ at fixed range of $K_v$ in the mathematical curves other parametric curves can be obtained, on which the field ones can be superimposed to obtain a value of $m$.

A comparison (superimposition) between "in situ" curves and theoretical ones can thus be made to evaluate soil properties.

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Aspects of cone penetration in natural weakly-cemented deposits

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ABSTRACT: Aspects of interpretation of cone penetration tests in naturally cemented weathered profiles are presented in this paper. Two sites exhibiting intergranular bonding are considered, a decomposed coarse-grained granite and a decomposed fine-grained claystone. The study provide insight into the understanding of drainage conditions, soil classification and shear strength from cone penetration on cemented materials.

1 INTRODUCTION

Considerable improvement has been recently made on existing in situ testing techniques both on instrumentation capability and theoretical approaches. A rational basis for interpretation of tests carried out in undrained clays or drained sands is now available, which can account for the complex stress field developed around a penetration tool (e.g. Baligh, 1986; Teh & Hoek, 1988; Salgado, 1997). These approaches have not yet been extended to non-standard materials such as weathered cemented deposits, unsaturated profiles, waste fills; analysis is complicated by factors such as drainage conditions, anisotropy, non-uniform strain fields, disturbance and structure breakdown. In particular, the possible effects of cementation loads on the stress-strain behaviour of soils should not be neglected (e.g. Lerouiel and Vaughan, 1990) as should not be their influence on field test results.

The need to use in situ testing techniques is prompted by the difficulties in sampling naturally cemented weathered deposits. The cone test (CPT) is recognised as a profiling device that gives a continuous record of the soil allowing the determination of thickness and macroscopic of different layers. It is therefore a powerful tool for the characterisation of highly variable and erratic strength weathered profiles. However, identification of soil type and prediction of soil properties from cone penetration measurements can be distinctive from standard interpretation procedures adopted for sedimentary deposits. This paper attempts to highlight some errors that might arise from interpretation. Suggestions of interpretation procedures are given and results of penetration tests in weathered profiles from southern Brazil are used to illustrate their applicability.

2. CPT PROBES

Electrical penetrometers 10 cm² and 15cm² cross section area measuring tip cone resistance and shaft friction have been used. The load sensors for measuring cone and shaft resistance have typical maximum ranges of 50 KN and 7,5 KN, respectively. Calibration of sensors is typically linear given a resolution of 0.01% full range.

A porous element for pore pressure measurements is located on the shoulder immediately above the cone face. However, the benefits of a pore pressure sensor is rarely considered since the water table is often at significant depth. Additional measuring sensors can be incorporated and the need for seismic downhole measurements for determination of the small strain shear modulus is addressed.

3 WEATHERED PROFILES

The characteristics of a completely decomposed lateritic claystone and a decomposed saprolitic granite were studied using laboratory and field tests. The two sites have been selected due to the
Figure 1. Typical profile of a decomposed saprolitic granite.

Figure 2. Typical profile of a decomposed lateritic claystone.

different soil matrix. Site 1 is a predominantly coarse-grained soil and Site 2 is a fine-grained soil. Superimposed data from cone measurements and a brief description of both profiles are given in Figures 1 and 2. Both sites possessed intergranular bonding, discontinuities and are unsaturated in the first several meters. The water table is below 10 m at Site 1 and at about 4 m at Site 2.

The behaviour of the fine-grained site has been investigated by a comprehensive laboratory testing programme. Drained triaxial test results carried out in saturated undisturbed specimens are presented in Figure 3. For the applied range of stresses, the peak strength envelope is linear with a slope of 26° and an intercept of 17 kPa. The tasks of sampling, testing and predicting soil properties for the coarse material are challenging which enhance the need for in situ testing investigation. The peak strength envelope in direct shear tests performed in soaked undisturbed samples is presented in Figure 4. The
best fit envelope has a slope of 32° and an intercept of 10 kPa. Shear strength envelopes for both Site 1 and Site 2 have characteristic cohesive frictional behaviour which has a pronounced effect on soil parameters.

![Shear strength envelope](image)

**Figure 3.** Triaxial stress-strain response of weathered claystone.

### 4 ASPECTS OF INTERPRETATION

#### 4.1 Conditions of drainage

The most limiting factor affecting cone interpretation relates to the drainage field adopted for tests carried out at a fixed rate of penetration. In many practical applications it is assumed that undrained conditions apply (e.g., Perbeck et al., 1996) probably due to the fine grain size distribution of lateritic layers. However, the coefficient of hydraulic conductivity of weathered soils is not only a function of the soil matrix, it is determined by the presence of more permeable discontinuities (e.g., Haberfield, 1997) and the high porosity attributed to the structure (Leroueil and Vaughan, 1990).

The question of drained versus undrained response cannot always be resolved with pore pressure data due to the unsaturated nature of deposits. Experience in cemented soils has shown that many of them appear to be unsaturated, and such soils demonstrate a high shear strength due to their bonded structure and suction. In unsaturated conditions an increase in strength occurs due to the pore water action caused by surface tension effects. Both profiles presented in Figures 1 and 2 exhibit characteristic unsaturated behaviour. Under this conditions, pore air pressure is zero and pore water pressure is negative; the negative water pressure due to capillarity will act on the large void space at the back of the cone which implies on q_s = q_u. It does not imply however that the principles of effective stresses are satisfied, since the behaviour of unsaturated soils requires two independent stress parameters for complete description (e.g. Fredlund & Morgenstern, 1977; Alonso et al, 1990). These parameters may be taken as the net stress, σ_u, and the suction u_s, where σ is the total stress, u_p is the pore air pressure and u_s is the pore water pressure. These two variables replace the single effective stress variable σ_u_s that is assumed to control the behaviour of saturated soils. Parameters adopted for the interpretation of piezometers in saturated soils should be re-defined and in unsaturated soil conditions factors such as κ and B_u will depend on both the overconsolidation ratio and soil suction.

Below water table, it is in principle possible to predict shear strength parameters from the correct measured cone resistance q_s if pore pressures are recorded. The need to convert the measured values of q_s into correct cone resistance q_u is recognised. In intermediate drainage conditions interpretation of penetration data can be made adopting effective stress parameters, as proposed by Janbu & Snesrud (1974) and illustrated by Bugno & McNeilan (1984) Hight et al (1994) and Puchen et al (1996).

![Shear strength envelope](image)

**Figure 4.** Shear strength envelope of weathered granite.

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4.2 Soil Characterisation

On the absence of pore pressure measurements, soil classification from CPT data is based on the correlation between tip cone resistance \( q_t \) and friction ratio \( R_f \). Despite of the detailed provided by \( q_t \) and \( R_f \) plots, data from the two sites suggest that \( R_f \) is not appropriate to give identification of soil type and cement content, since penetration through layers of different degrees of cementation is marked by large variations in \( q_t \), but it is insensitive to \( R_f \). (Akli et al., 1988; Pappas et al., 1995; Schnaid & Consoli, 1995). It is therefore necessary to record additional measurements in the penetration process to enhance the information obtained from the cone. The combination of seismic shear wave velocities with tip cone resistance permits a more comprehensive soil characterisation since both strength and stiffness are inherently considered (Lunne et al., 1995; Schnaid, 1997). An example of the potential of combining \( G_s \) and \( q_t \) measurements is illustrated in Figure 2, in which the variation of \( G_s/\rho_q \) with depth at Site 2 is presented. The ratio \( G_s/\rho_q \) appears to decrease with increasing cementation in the lateritic claystone soil layer.

4.3 Shear strength

The compressive and tensile strength of cemented soils have been studied in the past by several investigators such as Clough et al. (1981) and Consoli et al. (1995). From the interpretation of results from laboratory tests it appears that \( q'c \) can be taken to be independent of degree of cementation whereas the magnitude of \( c' \) is largely affected by variations on cement content. It is therefore suggested that it is relatively simple to predict a single value of \( q'c \) representative of the soil matrix from triaxial or direct shear laboratory tests; this may then be assumed to be a suitable value for all degrees of cementation. On the other hand, the behaviour of naturally occurring cemented soils is largely affected by soil structure produced from cementation and so is the magnitude of the cohesion intercept. Predictions of \( c' \) values can be made in situ from combined field measurements of tip resistance and laboratory measured friction angle; changes in the values of \( c' \) would reflect possible changes in cementation with depth. A similar assumption has been made by Schnaid & Montaras (1998), in a companion paper, on the interpretation of pressuremeter tests in cohesive frictional materials.

The framework for interpretation depends largely on semi-empirical approaches. Bearing capacity theories, cavity expansion theory, state parameter concepts can all be explored. The bearing capacities theories proposed by Terzaghi & Mitchell (1973), Janbu & Sorensen (1976) have been selected in this paper as a framework for discussion due to their simplicity in analysis and wide use in practice. The cohesion intercept was calculated using the two formulations. The calculated values of \( c' \) are compared with the measured laboratory values in Figure 5, for the lateritic fine-grained unsaturated soil of Site 1. At present there is no laboratory data for comparison below 3.0 m depth. In general the theoretical predictions provide an acceptable correlation with measured values from both theories, but negative values or unrealistic high values of \( c' \) are often reported. As a general conclusion it is indicated that attempts to interpret strength from cone penetration in cemented deposits would require other field or laboratory results.

![Figure 5. Values of cohesion intercept predicted from cone penetration and measured in triaxial tests](image)

5. CONCLUSIONS

Testing of weathered profiles in southern Brazil has provided insights for interpretation of cone penetration tests, which does not necessarily follows conventional interpretation for sedimentary
deposits. In summary, the conclusions are as follows:

- CPT is ideally suited for use in site investigation and profiling of natural weathered lightly structured deposits. A major limitation is that soil type and degree of cementation is not directly inferred from standard classification charts.
- Drainage conditions are determined not only by hydraulic conductivity of the soil matrix but also by the presence of more permeable discontinuities and as a result undrained CPT behaviour not always apply. At present, the cone does not characterise shear strength from intertalic and saprolitic deposits; complementary laboratory or field data are necessary to support interpretation.
- The need for incorporating seismic sensors is stressed not only for a direct measurement of the small strain shear modulus but also for a promising approach of assessing soil type from the ratio of G/G0.

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Correlation of CPT and modulus of subgrade reaction in silty sand

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J.K. Azhor
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ABSTRACT: This paper presents the results of an investigation using controlled field tests to determine the relation between the static cone resistance (q_c) and the modulus of subgrade reaction (k) in silty sand soil. CPT and plate load tests were made using two meter cube water tight test pits. The test pits were filled with soil under different degrees of compaction and saturation. The degree of compaction (DC) was between 87.9% and 98.5%, and the degree of saturation (DS) was between 35% and 98%. A linear relation, using statistical regression analysis, was determined between k and q_c. The ratio of k to q_c was 3.25; prediction of k was influenced when DC and DS were considered.

1 INTRODUCTION

Jeddah, Saudi Arabia, is a rapidly growing city on the Red Sea. The groundwater level rises at an annual rate of approximately 500 mm reaching the ground surface in many areas, which causes gradual reduction in the bearing capacity of the subgrade under the flexible streets pavements. This, in addition to the ever increasing axle loads caused premature failure in some of the heavily trafficked streets. As the problem exists, rigid and composite pavements may be used in some parts of the affected areas as a viable alternative.

The modulus of subgrade reaction (k) is one of the main parameters in the analysis and design of rigid pavements. It is usually determined by running in situ plate load tests which are cumbersome, expensive and time consuming. Also, pavement design is sensitive to lower values of k and hence additional tests would be necessary for better estimation of the field k values. Static cone penetration tests (CPT) have been used successfully in the assessment of strength and density of shallow soils for pavement construction. CPT is relatively simple to run and could be automated which helps in running numerous tests over relatively short period of time (Chedid et al 1979). CPT can provide reliable information on soil strength indexes and on soil characterization (Olsen 1988). Correlations were determined between the static cone resistance q_c and the elastic modulus of the soil E for different types of soils. Tests conducted on flexible pavements located throughout the State of Florida (Habu-Twenehwa et al 1988) showed that CPT provided detailed stratification of the test sections and helped to identify possible zones of weakness in the pavement and underlying subgrade soils. The tests also indicated the potential for CPT prediction of the modulus of the layered pavement system and subgrade soil. Allowable bearing capacity values of a silty soil with few patches of clay and sand (Dayal 1982) obtained from CPT are in agreement, but they are half the values computed from plate load tests. Also, the values of the elastic modulus predicted from CPT compared well but are appreciably lower than these determined from in-situ plate load tests.

Linear correlation was determined between q_c and E for clay, clayey sand, and coarse sand (Marcu & Popescu 1982) where E, the deformation modulus, was determined in the proportionality domain of the relationship between pressure and settlement as determined by the plate test. Another correlation was determined between the Young's modulus of fine silty sand and q_c where a parabolic relation was found to exist between E and q_c (Denver 1982).

2 EXPERIMENTAL DETAILS

Controlled semi-field tests were conducted in eight water-tight pits of masonry brick wall constructed below the ground surface. The layout of the test pits is shown in Fig. 1. The pits were filled with a silty sand soil, a predominant soil type in Jeddah. Two variables were considered in preparing each
test pit, namely degree of compaction and the depth of water level below the surface of the soil in each pit. For this purpose, each pit was divided into four layers, 500 mm thick each. The scheme of compaction and the depth of water level in the different pits is given in Table 1.

![Diagram of test pit layout](image)

**Fig. 1 Layout of the test pits**

2.1 Preparation of test pits

Each test pit was 2.0 ms cube. The bottom of the pit was built of a 10.0 cm thick layer of plain portland cement concrete and the four sides were built with masonry brick. The pits were made water tight by plastering the bottom and the inner four sides with portland cement mortar. Providing water and controlling its level was achieved with special piping arrangement shown in Fig. 2. Perforated PVC tubes were laid down horizontally on the bottom of the pit. These tubes were connected to a vertical (reservoir) PVC tube extending above the top of the pit. Water was supplied continuously to the pit throughout the test duration. The level of water in the pit was similar to the level of water in the vertical tube which was controlled using a float. The pits were filled with light brown silty sand soil classified as A-3-4 (0), (ASTM D3562-91). Particle size distribution of the soil is shown in Fig. 3. The maximum dry density was 2190 kg/m³ (ASTM D1557-91) at an optimum water content of 7.8%.

The soil in each pit was placed and compacted in eight equal layers. Compaction was achieved using a vibratory compactor with a frequency of 500-600 cpm and a compacting plate 280 x 340 mm in size. In order to obtain the required degree

<table>
<thead>
<tr>
<th>Pit No.</th>
<th>Layer No.</th>
<th>D.W.L.</th>
<th>DC %</th>
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<td>100</td>
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<tr>
<td></td>
<td>3 &amp; 4</td>
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<td>3</td>
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<tr>
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<td>3 &amp; 4</td>
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</table>

DC = Degree of compaction
D.W.L. = Depth to water level

![Diagram of piping arrangement](image)

**Fig. 2 Details of piping arrangement in the test pits**
load on the plate was applied against a reaction steel beam which was fixed to the central dead weight concrete block 4.0 x 4.0 x 5.0 cm in size. The load was applied with a 200 kN hydraulic jack, and settlements were measured at three points on the top surface of the bottom plate. Measured settlement was less than 0.4 mm in all of the tests.

2.3 Static cone penetration test

Three cone penetration tests were run for each plate load test. The plate load test was conducted first followed by the CPTs. Location of the CPT relative to the plate test is shown in Fig. 5. The CPT was run using a cone of 10 cm base area and a 60° apex angle fixed to a multiple part penetration rod. Each part, 200 mm long, was added whenever the rod was completely pushed into the soil. The rod was pushed using a 200 kN hydraulic jack with a 200 mm stroke length. The load was measured using a 200 kN electric load cell. A truncated detachable cone was used to eliminate disturbance of the soil when pulling out the penetration rod.

of the layers in the pit, a relation was established between the degree of compaction and the number of passes of the compactor (Fig. 4). The actual unit weight of the soil layers was determined later after running the plate load test and the CPT using the sand cone method (ASTM D1556-90).

2.4 Determination of unit weight

The unit weight of the top layer was determined at mid-depth of the layer after running the plate load test and the cone penetration tests. Additional soil samples were taken in order to determine the water content. The degree of compaction, DC, was calculated as the ratio of the dry unit weight to the maximum dry unit weight of the soil. The degree of saturation, DS, was calculated using the determined

![Fig. 3 Grain size distribution](image)

![Fig. 4 Relation between number of passes and degree of compaction](image)

![Fig. 5 Location of plate test and CPT](image)

3 RESULTS AND DISCUSSION

The unit weight of the top layer was determined at mid-depth of the layer after running the plate load test and the cone penetration tests. Additional soil samples were taken in order to determine the water content. The degree of compaction, DC, was calculated as the ratio of the dry unit weight to the maximum dry unit weight of the soil. The degree of saturation, DS, was calculated using the determined
water content. Values of DC and DS for the different tests are given in Table 2. Values of DC were in the range between 87.9% and 98% and values of the degree of saturation varied between 35% and 98%. Distribution of the water content with depth is shown in Fig. 6.

### Table 2 Summary of tests results

<table>
<thead>
<tr>
<th>Test #</th>
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<th>DS %</th>
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The modulus of subgrade reaction was calculated as the ratio of a stress value of 10 psi (69 kPa) to the average settlement of the plate corresponding to that stress value. A representative pressure vs. settlement curve is shown in Fig. 7 which also shows the effects of both the degree of compaction and the degree of saturation. It is seen that the settlement increases as the depth water level decreases and it decreases as the degree of compaction increases.

The cone resistance values are the average of three tests. Distribution of the cone resistance q, with depth is shown in Fig. 8 for pits number 1 to 4 and in Fig. 9 for pits number 5 to 8. The soil in the test pits of the first group has a higher degree of compaction. The general observation is that instantly all tests, the cone resistance exhibited a highest value at a penetration depth of approximately 200 mm below the surface of the top layer. It is this maximum value that was used to determine the correlation of the cone resistance and the modulus of subgrade reaction of the soil. Comparing figures 8 and 9 shows that the degree of compaction has a significant influence on the cone resistance which dropped
significant when a lower degree of compaction was used. Values of the modulus of subgrade reaction and the average cone resistance are given in Table 2.
To enhance the prediction of $k$, additional parameters such as the degree of compaction and the degree of saturation of the top layer were considered. The following model has the highest coefficient of correlation:

$$k = 22.095 + 0.41779 q_e (DC/DS)^2$$  

(SEB (4.105) (0.078))

$R^2 = 0.63$, $F = 28.8$, $n = 19$, $p = 0.00005$

Addition of the new factor $(DC/DS)^2$ in equation number 3 increased the values of $R^2$ and $F$ by 37% and 160% respectively over the values of equation number 1. As would be expected, the value of $k$ increases with higher degree of compaction and with lower degree of saturation. The relation between $k$ and the new factor $q_e (DC/DS)^2$ is shown in Fig. 11.

![Graph showing relation between modulus of subgrade reaction and cone resistance](image)

**Fig. 11** Relation between modulus of subgrade reaction and cone resistance, $DC$ and $DS$

5 CONCLUSIONS

From the results of this study, the following conclusions may be made:

1. The strength of the silty sand soil in terms of both the modulus of subgrade reaction $k$ and the cone resistance $q_e$ is adversely affected by the increase in the degree of saturation and the decrease in the degree of compaction.

2. A linear relationship exists between the modulus of subgrade reaction $k$ and the cone resistance $q_e$.

3. Prediction of $k$ is enhanced when the degree of compaction and the degree of saturation of the top layer were considered.

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Experience from site investigations in glacial soils of the Alpine region

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ABSTRACT: Glacial soils are quite heterogeneous and their composition and properties vary substantially over short distances. As coarse grained soils may be present within fine-grained soils the application of standard equipment for the investigation of uniform fine-grained soils may be limited. The deposition environment and the subsequent stress history of the soil has a significant effect on soil properties. In glacial terrain within a mountain range the stress history is highly variable. Thus a reliable methodology of characterization of the subsoil is necessary. Experience from two case histories, small to medium size projects from Switzerland, is presented. Different types of in-situ investigations, like cone penetration tests (CPTU), Marchetti Dilatometer tests (DMT), pressure meter tests (PMT), standard penetration tests (SPT) have been used. Also laboratory tests were carried out and their results compared to the in-situ tests. A combination of different techniques proved to be useful for a reliable site characterization.

1 INTRODUCTION

Experience from site investigations in glacial soils in Switzerland is described and compared. Switzerland is very densely populated and poorer ground is used for construction of buildings, road works and infrastructure. In one case a 140 m long tunnel for separation of a grade crossing between a commuter railway and a major road had to be constructed in glacial sands and silts and postglacial sensitive clay, in Schönbühl, in the northern suburbs of Bern.

The second case is a building for a Health Center in Lugano, Switzerland, an addition to the General Hospital (Ospedale Civico) built in 1969.

In-situ investigations have been applied for other projects (Steiner et al., 1992, Steiner, 1994) where stability and deformation problems had to be solved. There in-situ investigations with DMT and CPTU proved extremely valuable in characterizing soil properties and subsequent analyses. This experience will be also used in the evaluation of site characterization methods.

2 CASE HISTORY SCHOENBUEHL

This site is located some 10 kilometers north of Bern in an area where, during the ice-ages two glaciers had conflized. During their retreat the glaciers had left sand and silt deposits overlying ground moraine.

During the postglacial phase sensitive clays and peat were deposited in the remaining lake. During earlier centuries when building roads, railways and villages this unfavourable ground was avoided by following the margins of the swamp land. With the increasing population density property with good foundation conditions became scarce. In the 1960's an underpass was constructed under the main east-west railway, substituting two grade crossings. The access road had to be built on a thick, close to 10 meters, layer of sensitive clay. For acceleration of the consolidation of an embankment of 2 - 3 meters height on clay, sand drains were required. Under an additional temporary surcharge of 3 meters 1 to 1.8 meters of total settlement were monitored.

At the beginning of the 20th century a narrow gauge railway was constructed which over time mutated to a commuter railway. During the construction of this railway embankments experienced large scale instabilities and settlements. In the 1960's the construction of the motorway N 1 led to a temporary reduction of transit road traffic. However, a large shopping center was built in area of swamp land, on a large pile foundation, thus traffic increased again. It became necessary to separate a grade crossing of the commuter railway with the state road. Initially, for geotechnical reasons an underpass should be avoided and overpasses were considered. However, their appearance was judged to be detrimental to the town and bridges were rejected. Finally, the conclusion was reached that a 340 meter long cut-and-cover tunnel was the best solution both technically and environmentally. Subsoil investigations already existed, which were, however, mostly executed to insufficient depth. New ground investigations for geotechnics and hydrogeology were executed in 1990. The 340 meter long cut-and-cover tunnel has slurry walls reaching to a depth of approximately 17
meters from ground surface and might form a hydrogeologic barrier, resulting in secondary settlement effects. This site investigation included 11 drill holes in soil between 15 and 30 meters depth. Also 12 cone penetration tests with pore pressure measurements (CPTU) were carried out to depths from 12 to 30 meters depth. The cone penetration tests had to be terminated due to excessive tip resistance, once they had reached the ground moraine. The project was delayed several years. In 1994 five more borings were drilled also with in-situ tests. The main scope was hydrogeologic monitoring: filter tips were placed in sealed zones. These could be equipped with electric pore pressure transducers at a "docking station", thus giving a closed system for measuring pore pressure allowing for automatic registration.

2.1 Main results from site investigation

One third of the tunnel lies in sensitive postglacial clay and the remaining two-thirds were in ice-marginal sands and silts. The groundwater table was close to the ground surface, only in 1 to 3 meters depth. In deeper zones, 15 to 20 meters depth, the pressure was slightly higher. The combination of drill holes and cone penetration tests allowed a comparison of in-situ measurements with laboratory tests. As glacial deposits are also heterogeneous within the same stratigraphic unit the individual strata will vary quite erratically. With CPTU these variations of the layers in the range of decimeters was easily to determine. The contact between the sensitive clay and the ice-marginal deposits was rather steep. The sands were deposited by currents in a lake (delta deposits) leaving a rather irregular surface of the sand. In the immediate vicinity of the slurry walls the sensitive clay had a thickness of 14 meters, yet only 10 meters further to the north the sensitive clay reached to a depth of 19 meters. From the construction of the new town center, 1986-1988, to the north of the eastern end of the tunnel, it was known that the depth of the sensitive clay might reach to 25 meters depth. The piles, driven to the ground moraine with tip bearing, had introduced a disturbance into the sensitive clay reducing the undrained shear strength.

2.2 Characterization of sands and silts.

Cone penetration test CPTU-6 was carried near the center of the tunnel, with predominantly sandy to silty soil as was confirmed by a parallel boring. In Fig. 1 the strength properties of the subsoil based on the interpretation of normalized tip resistance and normalized pore pressure ratio indicated less cohesive sections with lower undrained shear strength.

This experience illustrates the problems associated with silty soils which behave as partly draining during cone penetration. In any case the large undrained shear strength obtained are judged to be non-reliable values. Silt in glacial area is mostly rock flour or rock flour, a granular material. The friction angles computed in Fig. 1 with a range from 32° to 43°, however, appear reasonable.

With the DMT the compressibility of the sands was estimated and compared favorably to values back calculated from earlier settlement observations in the area of the underpass constructed in the 1930's.

2.3 Characterization of postglacial clays

Cone penetration test CPTU 10 was mostly located in postglacial sensitive clay. A parallel boring with DMT measurements was also carried out and in 1994 undisturbed samples were taken from an additional boring. The interpretation of the undrained shear strength for \( N_c = 14 \) with the \( Q_d \) versus \( F_a \) classification is shown on Fig. 2. Also shown are the undrained shear strength determined in the boring by the flat dilatometer (DMT).

The undrained shear strength appears rather constant with depth and shows a minimum in the center of the layer at 12 meters depth. In contrast two peaks with \( c_u = 40 \) kPa are present at 10 and 15 meters depth. For further interpretation one has to take
into consideration construction activity in the vicinity of the site. Piles had been driven several (3 to 4) years prior to carrying out the core penetration tests.

The reduction of the shear strength is most likely the effect of disturbance caused by the pile driving; otherwise one would expect an increase in undrained shear strength with depth. The peaks may have been caused by zones of larger permeability resulting in more rapid consolidation.

The results of the undrained shear strength with a mean $C_{u} = 30$ kPa and a standard deviation of 5 kPa analyst were used for a probabilistic analysis of the stability of slurry trenches of varying width (Steiner and Rieder, 1997) which proved very useful. Moduli determined from DMT were compared with moduli from oedometer tests (Fig. 3) and good agreement was found.

3. CASE HISTORY HEALTH CENTER

During 1996 site investigations were carried out for an addition to the General Hospital at Lugano, southern Switzerland. The Health Center has a footprint of 50 by 50 meters. From the construction of the General Hospital the results of the site investigation were available. The site is located on a valley slope and the existing buildings are partly founded on bedrock (mica-schist) and the remaining sections are founded on large diameter bored piles in order to achieve homogeneous foundation behaviour.

Four core borings with continuous sampling and a core diameter d = 145 mm were executed. Localized in-situ tests were carried out, namely 24 SPT tests, 13 DMT (Flat Dilatometer tests), 5 PMT (pressure meter tests) in these borings and were supplemented by a single core penetration test (CPT-U) in an area where a track had access. The CPTU had to be executed in two phases, since from 4 to 8 meters depth dense gravelly ground moraine was present which gave too much resistance to the penetrometer. Also the DMT device could only push ahead a limited distance from the bottom of the borehole permitting only two readings. This was most likely caused by two factors: (1) the dense nature of the overconsolidated soils and (2) the limited thrust of the light-weight drill rig which had to be used due to the limited accessibility.

The short period available for the site investigation did not permit to take undisturbed samples and carry out a detailed laboratory investigation. However, data from the site investigation (50 mm lab test from 65 mm push-in tube sample) from 1996 were available and comparisons were drawn as well as laboratory results carried out on 70 mm samples carved from the 145 mm diameter core in 1996.

3.1 Stratiography

The stratigraphy can be summarized as:

(1) Overburden with a maximum thickness of 4 meters, fill of various origin, which will be mostly removed for the construction.

(2) Glacial and glaciolacustrine deposits (Wirm) consisting of gravel, often rather coarse embedded in a matrix of silty or sandy clay extremely variable with boundaries changing erratically. Included are also sandy silts and clayey silts often with varves. This layer is the weakest link of the soil and was investigated in detail.

(3) Bedrock, Mica schist, 10 to 15 meters below the future foundation level.

The hydrogeologic conditions can be characterized that no continuous water table is present. Water percolates on impervious layers like the surface of bedrock and interbedded clay layers.

3.2 Interpretation of in-situ tests

The interpretation will focus on the properties of the
layer of glacial and glaciolacustrine deposits and the comparison of in-situ measurements of soil compressibility and shear strength with laboratory results (Tab. 1 and 2).

Compressibility (Oedometer moduli M) from tab samples are shown on Fig. 4 and moduli obtained from different procedures are shown on Fig. 5. With regard to the compressibility of these soils the following conclusions can be drawn:

1. No distinctions can be made for different types of sampling procedures whether the crude 1969 core samples or the larger 1996 "undisturbed" core samples.

(2) The modulus of compressibility M from oedometer tests at overburden stress are below the true values. The samples from overconsolidated sandy clay with gravel inclusions probably always suffer extensive sample disturbance, such that "undisturbed" samples are of little value.

(3) The maximum overconsolidation ratio estimated from the 1996 laboratory test is OCR = 5.5. The deformation properties obtained by DMT suffer in quality from the difficulty of penetration of the blade. It may also be possible that the drilling introduced a major disturbance ahead of the core bit. The moduli estimated from DMT are only slightly above those estimated from CPTU with the original formula by Kulhawy & Mayne (1990) assuming non-cohesive soil and OCR = 1.

(4) In Fig. 6 modified formulae have been applied for PMT, CPTU and SPT in order to compare
moduli. Assuming the modulus $M$ from DMT as reference, the following relations were deduced:

- $M$ (PMT) = 33 $p_c$
- $M$ (CPTU) = $q_u 10^{0.6(M-0.1)}$ (Dr for OCR = 5)
- $M$ (SPT) = 1.8 $N$

### 3.3 Shear strength of soil

The results of shear strength data obtained from different methods are shown on Figs 7 and 8.

### 3.4 Classification of soils

The comparison of classified soils on the soil chart (Fig 9) shows that different techniques are necessary to select the proper classification procedure. Here the classification with CPTU ($Q_t$-FR correlation) give obviously better agreement with the DMT classification for these overconsolidated partially saturated than the $Q_t$-BR. This classification is also in agreement with the laboratory tests (Tabs 1 and 2).
3. Site characterization in heterogeneous glacial soils cannot be based on a single type of in-situ investigation. CPT has to be calibrated on DMT or PMT measurements. (2) In overconsolidated soils sample disturbance may be substantial.

The in-situ methods provide profiles of soil properties and the variability is shown. These data may be used as input to probabilistic methods of design.

REFERENCES


Correlation between ultimate pile skin friction and CPT data

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ABSTRACT: Static loading tests on six piles and cone penetration tests (CPTs) were performed at four sites to determine the relation between ultimate pile skin friction and CPT data. This work was part of a research program on methods of evaluating pile bearing capacity using CPT. The main findings were as follows: (1) There is notable correlation between CPT excess pore pressure \( \Delta u \) and the ratio \( f_p/f_s \) of ultimate pile skin friction \( f_p \) to CPT sleeve friction \( f_s \). (2) This correlation can be applied to accurately predict the ultimate pile skin friction using CPT. (3) Some reasons for this correlation can be identified based on the results of basic tests on the generation behavior of \( f_s \).

1 INTRODUCTION

Static loading tests on six piles and cone penetration tests (CPTs) were performed at four sites to determine the relation between ultimate pile skin friction and CPT data. This work was part of a research program on methods of evaluating pile bearing capacity using CPT. Several methods have been proposed for using CPT data to directly predict ultimate pile bearing capacity. These methods can be roughly classified into two types: those which use tip resistance \( q_t \) alone (e.g., Buzastante and Gianselli, 1982), and those which use both tip resistance \( q_t \) and sleeve friction \( f_s \) (e.g., Schmertmann, 1978; de Raiter and Beiringen, 1979). Few methods directly use pore pressure \( \Delta u \), though it is becoming increasingly common to use CPT to measure \( q_t, f_s \), and \( \Delta u \) simultaneously. By comparing various methods using CPT data, Robertson et al. (1988) and Briaud (1988) have concluded that the LCPC method (Buzastante and Gianselli, 1982) which directly uses \( q_t \) alone is best. However, since the mechanism for generating \( f_s \) appears closer to that for generating pile skin friction than that for generating \( q_t, f_s \) can be considered a suitable index for skin friction evaluation. Thus, there seems to be room for further study on methods of using \( f_s \), especially with \( \Delta u \), which is rarely used in conventional methods.

Thus, the focus is on \( f_s \) and \( \Delta u \). The findings which focus on the relation between ultimate pile skin friction \( f_p \) and \( f_s \) are presented in the former part of this paper. The results of basic tests on the generation mechanism of \( f_s \) and the discussion on the findings presented in the former part based on the results are presented in the latter part. To examine the generation behavior of \( f_s \), CPTs were conducted at six sites, including those where the pile loading tests were performed, using special procedures. In the CPTs, the penetration rate and surface roughness of the friction sleeve were adopted as experimental parameters.

2 RELATION BETWEEN ULTIMATE PILE SKIN FRICTION AND CPT DATA

2.1 Outline of pile loading test and CPT

Table 1 summarizes the test piles and the soil types. Three bored cast-in-situ concrete piles and three driven steel pipe piles were included in the tests. Strain gauges were installed in all the test piles at the soil layer boundaries to evaluate the pile skin friction in each layer. Ultimate pile skin friction \( f_p \) was determined in 17 of the ranges in which pile skin friction was measured, from the relation between pile displacement \( \delta \) and skin friction \( f_s \) according to the criteria described below.

1. If \( f_s \) had an obvious peak in the range \( \delta < 30 \text{ mm} \), the peak value was adopted as \( f_s \).
2. If \( f_s \) had no obvious peak in the range
Table 1. Summary of test piles and soil types.

<table>
<thead>
<tr>
<th>Site</th>
<th>Pile type</th>
<th>Diameter (m)</th>
<th>Length (m)</th>
<th>Soil type (soil description)</th>
<th>Soil type (soil description)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>P1</td>
<td>1.0</td>
<td>62.4</td>
<td>Sand (Fine sand)</td>
<td>Inter. soil (Silty shale)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Inter. soil (Silty shale)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Clay (Silty clay)</td>
</tr>
<tr>
<td>S2</td>
<td>P2</td>
<td>2.5</td>
<td>38.9</td>
<td>Sand (Fine to medium sand)</td>
<td>Inter. soil (Silty sand)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Inter. soil (Sandy silt and silt)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Clay (Silty clay)</td>
<td></td>
</tr>
<tr>
<td>P3</td>
<td>P4</td>
<td>1.5</td>
<td>40.0</td>
<td>Sand (Fine to medium sand)</td>
<td>Inter. soil (Silty sand)</td>
</tr>
<tr>
<td>P5</td>
<td>P6</td>
<td></td>
<td></td>
<td>Inter. soil (Silty sand)</td>
<td>Inter. soil (Sandy silt and silt)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Clay (Silty clay)</td>
<td></td>
</tr>
<tr>
<td>S3 P4</td>
<td>D3</td>
<td>0.8</td>
<td>8.3</td>
<td>Clay (Dolomitic mudstone)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S4 P5</td>
<td>D4</td>
<td>0.9</td>
<td>63.5</td>
<td>Sand (Fine sand)</td>
<td>Inter. soil (Silty sand)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Clay (Silty clay)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Inter. soil (Sandy silt)</td>
<td></td>
</tr>
</tbody>
</table>

Legend: D = Pile diameter
L = Embedment length
BC = Bored cast in-situ concrete pile
DS = Drilled shaft pipe pile

Table 2. Criteria for soil type classification.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Sand</th>
<th>Clay</th>
<th>Intermediate soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand friction (S)</td>
<td>&gt; 50</td>
<td>&lt; 20</td>
<td>20-30</td>
</tr>
<tr>
<td>Plasticity index, Ip</td>
<td>NP &gt; 25</td>
<td>NP &lt; 25</td>
<td></td>
</tr>
</tbody>
</table>

Legend: NP = Non-plastic.

δ < 30 mm when the maximum value of δ was greater than 30 mm, the value of f for δ = 30 mm was adopted as f30.
3. If f had no obvious peak when the maximum value of δ was smaller than 30 mm and if abnormality was observed in the relationship between δ and f, no value was adopted as f30.

At sites S2 and S3, loading tests were performed on two piles in identical soils. The ranges for which f30 was determined include 12 soil types, including special layers such as shirasu layers (site S1, Takeo et al., 1995) and a dolomitic mudstone layer (site S3, Matsumoto et al., 1995). Soils were classified into three types, sand, clay, and intermediate soil, according to the criteria in Table 2 (JSSMFE, 1992). The pile loading tests were performed in conformity with JSSMFE standard (1993) more than one month after the piles were placed. The CPT data presented in this chapter, obtained from the standard procedures (JSSMFE, 1989), indicate the soil characteristics at the test sites before the piles were placed. The standard cone penetrometer was used, i.e., with a cone tip of 10 cm3 base area, a friction sleeve of 150 cm3 surface area, and a porous filter for pore pressure measurement.

Figure 1. Relation between fp and f30.

CPT sleeve friction, f30 (kPa)

Figure 1 shows the relation between ultimate pile skin friction f and CPT sleeve friction f30. Data by Shibata et al. (1987) are included for reference. Though the relations for all soils tend to be widely scattered, the average relations for clay and for sand and intermediate soil can be roughly described as f = 3f30 and f = f30, respectively. Pile type has little influence on these relations.

The relatively large scatter in the relation between fp and f30 shown in Figure 1 makes it inappropriate to apply it directly to pile design. The tendency for the ratio of fp to f30 (fp/f30), termed pile-CPT friction ratio, to vary with soil type can be seen in Figure 1. Thus, the investigation was carried out to determine whether the ratio fp/f30 can be estimated using CPT indexes which correlate with soil type.

Figure 2 shows the relation between fp/f30 and excess pore pressure Au (Au = u - u0, u0 = equilibrium pore pressure). The index Au had the best correlation with fp/f30 of the various indexes adopted in this study, although the full results are not indicated here on account of limited space. The solid line in Figure 2 indicates an average

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relation between $f_{psf}$ and $\Delta u$. The ratio $f_{psf}$ tends to increase with $\Delta u$ irrespective of pile type. The increase tends to be steeper in the range where $\Delta u$ is large.

2.3 Prediction of ultimate pile skin friction

Ultimate pile skin friction $f_p$ can be predicted from CPT sleeve friction $f_s$ and excess pore pressure $\Delta u$ using the relation indicated by the solid line in Figure 2. Figure 3 compares the predicted ultimate pile skin friction $f_{psf}$ by this method with the measured $f_p$. The predicted values agreed well with the measured values. This method, which uses two pieces of CPT data $f_s$ and $u$, is effective for predicting ultimate pile skin friction.

3 DISCUSSION BASED ON GENERATION BEHAVIOR OF SLEEVE FRICTION

The previous section showed that it is possible to predict ultimate pile skin friction $f_p$ using the correlation between pile-CPT friction ratio $f_{psf}$ and excess pore pressure $\Delta u$. It is important to determine the reasons for this correlation to estimate its applicability.

Particular CPTs were conducted to examine the generation behavior of $f_s$. In these CPTs, penetration rate and surface roughness of the friction sleeve were adopted as experimental parameters. The reasons for the correlation were estimated from the results.

Table 3 presents the soil properties at sites for particular CPTs.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Properties</th>
<th>Site</th>
<th>Range No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>Ip = 70, OCR = 3</td>
<td>S2</td>
<td>7, 11</td>
</tr>
<tr>
<td>Clay</td>
<td>Ip = 70, OCR = 3</td>
<td>S3</td>
<td>12, 13</td>
</tr>
<tr>
<td>Clay</td>
<td>Ip = 35 - 50, OCR = 2</td>
<td>S4</td>
<td>16</td>
</tr>
<tr>
<td>Clay</td>
<td>Ip = 35 - 50, OCR = 1</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Clay</td>
<td>Ip = 44, OCR = 2</td>
<td>S5</td>
<td>-</td>
</tr>
<tr>
<td>Sand</td>
<td>FC = 34 - 44%</td>
<td>S1</td>
<td>2</td>
</tr>
<tr>
<td>Sand</td>
<td>Dsu = 0.09 - 0.14mm</td>
<td>2</td>
<td>-</td>
</tr>
<tr>
<td>Sand</td>
<td>FC = 91%</td>
<td>S1</td>
<td>2</td>
</tr>
<tr>
<td>Sand</td>
<td>OCR = 2</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>Dsu = 0.19 - 0.51mm</td>
<td>2</td>
<td>4, 8</td>
</tr>
<tr>
<td>Sand</td>
<td>FC = 7 - 10%</td>
<td>S6</td>
<td>-</td>
</tr>
<tr>
<td>Sand</td>
<td>Dsu = 0.13 - 0.32mm</td>
<td>2</td>
<td>-</td>
</tr>
</tbody>
</table>

Legend: FC = Fines content, Dsu = Mean grain size, $f_p$ = Friction index, OCR = Overconsolidation ratio.
3.1 Influence of penetration rate

Figure 4 shows the variation of excessive pore pressure $\Delta u$, sleeve friction $f_s$, and sleeve friction variation ratio $f_s/f_s'\Delta u$, with penetration rate, $R_c$. Here, the subscript "a" indicates that the measurement was obtained under arbitrary conditions; $f_s$ is sleeve friction at the standard penetration rate of 20 mm/s. The standard cone penetrometer described previously was used in the tests.

The tendencies read from Figure 4 are as follows:

1. In clay, as $R_c$ slows down, $\Delta u$ decreases while $f_s$ and $f_s/f_s'\Delta u$ increase continuously.

2. In sand, as $R_c$ slows down, $\Delta u$ increases to 0 and $f_s$ and $f_s/f_s'\Delta u$ also increases in the range of $R_c$ between 2 and 20 mm/s; $\Delta u$ reaches 0 and $f_s$ and $f_s/f_s'\Delta u$ become almost constant in the range where $R_c$ is slower than 2 mm/s.

3. The variation in intermediate soil is analogous to the variation in clay or sand, depending on whether the properties are more like those of clay or sand.

The reason $f_s$ increases as $R_c$ slows in clay and intermediate soil is presumably that the effective lateral earth pressure on the friction sleeve increases when $\Delta u$ decreases due to reduction in $R_c$. In sand, $\Delta u$, and $f_s$ increase simultaneously as $R_c$ slows. The reason for this is not evident. However, it could be that in sand some mechanism, depending on the dilatancy characteristics of sand, which raises total lateral earth pressure, is generated when $R_c$ slows.

Figure 5 compares the relation between pile-CPT friction ratio $f_p/f_s$ and pile displacement rate $R_p$ (broken lines) with the relation between sleeve friction variation ratio $f_s/f_s'\Delta u$ and cone penetration rate $R_c$ (solid lines). Since the pile loading tests were performed using load-controlled methods, the pile displacement rate is not constant. Figure 5 shows the range of $R_p$ variation at the loading stage when $f_p$ was generated. In Figure 5, the $f_s/f_s'\Delta u$-$R_c$ relation tends to be in the vicinity of the extension of the corresponding $f_p/f_s$-$R_c$ relation. This tendency is more pronounced for clay. It indicates that the generation mechanisms of ultimate pile skin friction $f_p$ is similar to that of CPT sleeve friction $f_s$. However, the wide difference between pile displacement rate when $f_p$ was generated and standard CPT penetration rate suggests that $f_p$ and $f_s$ were obtained under different drainage conditions.

Figure 2 shows the tendency for the gradient of $f_p/f_s$ to $\Delta u$ to be steep when $\Delta u$ is large. From the results shown in Figure 4 and 5, one reason may be that in soil with large $\Delta u$, the difference in drainage conditions caused by the difference between $R_p$ and $R_c$ has a great influence on $f_p/f_s$ while in soil with small $\Delta u$ the difference in drainage conditions caused by the difference between $R_p$ and $R_c$ has a small influence on $f_p/f_s$.
influence is slight.

Figure 2 shows the tendency for \( f_s / f_s' \) to be greater than 1 when \( \Delta u \) is large, and to be 1 or less when \( \Delta u \) is small. The difference between \( R_p \) and \( R_e \) is considered as a major reason for the tendency for \( f_s / f_s' \) to be greater than 1 when \( \Delta u \) is large. This is because this tendency corresponds to that indicated in Figure 4 where \( f_s / f_s' \) is greater than 1 when \( R_e \) is low in soil with large \( \Delta u \).

However, there must be some other major reason for the tendency for \( f_s / f_s' \) to be 1 or less when \( \Delta u \) is small, other than the difference between \( R_p \) and \( R_e \). This is because Figure 4 indicates that \( f_s / f_s' \) is greater than 1 when \( R_e \) is low even in soil with small \( \Delta u \).

3.2 Influence of Surface Roughness

To evaluate the influence of surface roughness of the friction sleeve, CPTs were conducted using special friction sleeves with grooves lathed around their surfaces in addition to a standard friction sleeve with a smooth surface. Figure 6 shows an example of the surface roughness of one of these special friction sleeves. The special sleeves have regular indentations on their surfaces. Their surface roughness was quantified as maximum height \( R_{max} \) (Japanese Standards Association, 1982). A gauge length of 4 mm was employed, over which surface roughness was measured. To determine the effect of surface roughness of piles, different special friction sleeves with \( R_{max} \) up to about 100 \( \mu m \) were used. The CPTs were conducted at a standard penetration rate of 20 \( \text{mm/s} \).

Figure 7 shows the variation of sleeve friction \( f_s \) and sleeve friction variation ratio \( f_s / f_s' \) with sleeve surface roughness \( R_{max} \). Here, the subscript "a" indicates that the measurement was obtained under arbitrary conditions; \( f_s' \) is sleeve friction when the sleeve with standard surface roughness \( (R_{max} = 2 \mu m) \) was used.

Tendencies read from Figure 7 are as follows:

1. In clay, \( f_s \) increases with \( R_{max} \) when \( R_{max} \) is smaller than a certain level, but is constant when \( R_{max} \) is larger than that level.

2. In sand, sleeve friction \( f_s \) increases continuously with surface roughness when \( R_{max} \) is smaller than 100 \( \mu m \).

The reason \( f_s \) becomes constant when surface roughness is greater than a certain level in clay is presumably that the mode of slip between the friction sleeve and the soil shifts from a mode in which slip occurs at the surface of the sleeve to a mode in which slip occurs in the soil near the sleeve and finally reaches the limit corresponding to the shear strength of the soil.

Since the level of roughness defining the boundary is considered to correspond with the critical roughness determined in the laboratory friction test (Tanbakihara et al. 1993), it is termed critical roughness here as well. The critical roughness in clay ranges from 5 to 17 \( \mu m \) while that in sand is not reached when \( R_{max} \) is less than about 100 \( \mu m \).

The surface roughness of piles, which apparently depends on the soil type and the installation method, is assumed to be equivalent to an \( R_{max} \) of 5 to 20 \( \mu m \) for driven steel pipe piles and over 100 \( \mu m \) for bored cast-in-situ concrete piles. Provided the surface roughness of the friction sleeve is equivalent to that of the piles, the sleeve friction variation ratio \( f_s / f_s' \) quantified from the relation between \( f_s / f_s' \) and \( R_{max} \) in Figure 7 is about 1 to 2 for the surface roughness of a driven steel pipe pile in both clay and sand, about 1.3 to 2.
for the surface roughness of a bored cast-in-situ concrete pile in clay, and about 4 or greater for that in sand.

Figure 2 indicates that f_sfs is 2 or greater for both types of piles in clay, around 1 for bored cast-in-situ concrete piles in sand, and smaller than 1 for a driven steel pipe pile in sand. That f_sfs is 2 or greater in clay concurs with the above variation ratio. Thus, the difference in surface roughness between the pile and the friction sleeve can be considered to be the major reason for f_sfs to be 2 or greater in clay. However, that f_sfs is about 1 or smaller than 1 in sand, does not concur with this variation ratio, suggesting that there should be some major causal factor other than the difference in surface roughness between pile and friction sleeve.

3.3 Overall discussion

The difference between pile displacement rate and CPT penetration rate as well as the difference in surface roughness between pile and friction sleeve are considered to be a major reason for f_sfs being 1 or less when w is large, as shown in the correlation indicated in Figure 2. However, some other major reason is suggested for f_sfs being 1 or less when w is small.

The soils with small w include sand and intermediate soil, which have no cohesive strength that they tend to lose during pile installation. Thus, loosening of soils during installation can be a major reason for f_sfs to be 1 or less when w is small.

CPT sleeve friction depends on penetration rate and the surface roughness of the friction sleeve. It should be noted that the relation between the ultimate pile skin friction and CPT data described in the previous section is based on the data obtained by the standard CPT method.

4 CONCLUSION

The conclusions reached are summarized as follows:

1. There is notable correlation between CPT excess pore pressure w and pile-CPT friction ratio f_sfs.
2. This correlation can be applied to accurately predict the ultimate pile friction f using CPT.
3. The difference between pile displacement rate and CPT penetration rate and the difference in surface roughness between pile and friction sleeve are considered as major reasons for f_sfs being greater than 1 when w is large in the f_sfs-w correlation.
4. Loosening of the soil during pile installation work is considered as a major reason for f_sfs being 1 or less when w is small in the f_sfs-w correlation.

5 REFERENCES


A continuous intrusion electronic miniature cone penetration test system for site characterization

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ABSTRACT: This paper describes the implementation of a continuous intrusion miniature cone penetration test system (CIMCPT) for rapid characterization of subgrade soils, quality control assessment, construction control of embankments, and assessment of ground improvement effectiveness. Novel features of this new in-situ testing device are the caterpillar-type continuous chucking device for advancing the cone penetrometer which greatly increases productivity and serviceability. The coiled thrust rod eliminates threaded connections and simplifies cabling and water proofing. The device is mounted in a small 4 wheel drive all terrain vehicle equipped with a Global Positioning System. A new state-of-the-art data acquisition system has been implemented to acquire and display data on a computer screen in real time. The implementation of the miniature cone penetrometer is tested and verified by comparing the penetration profiles with those obtained using a standard 10 cm² cross-sectional area cone penetrometer.

1 INTRODUCTION

Among the various in situ test methods currently available, the cone penetration test (CPT) is becoming increasingly popular especially because it is reliable, fast and economical and gives continuous detailed soil profiles. CPT essentially consists of pushing into the soil an electronic probe known as the cone penetrometer at a rate of 2 cm/min. The cylindrical probe has a conical tip of apex angle equal to 60 degrees. The device is equipped with a load cell at the tip to measure the cone or tip resistance (q) which is the force offered by the soil to the tip during intrusion divided by the projected cone area. The projected cone area of the standard cone penetrometer is 10 cm². It is also equipped with a friction sleeve (150 cm² surface area), and a load cell to measure the sleeve friction (f) which is the local friction between the surrounding soil and the shaft of the probe. The term friction ratio (RF) often used in CPT data interpretation is the ratio of the sleeve friction to the cone resistance expressed as a percentage. The CPT data (i.e., the tip resistance and friction ratio) are used to determine the soil type for classification and for profiling subsurface soil stratigraphy. Coarse grained soils such as sands are characterized by high cone resistance and low friction ratios whereas fine grained soils such as clays are characterized by low cone resistance and high friction ratios. The CPT data is also used to estimate various engineering soil properties needed for analysis, design and construction.

2 THE CIMCPT SYSTEM

The Research Vehicle for Geotechnical In situ Testing and Support (REVIGITS) is a 20-ton all wheel drive vehicle that represents state-of-the-art equipment in the area of site exploration (Tumay 1994). REVIGITS is capable of performing standard cone penetrometer tests (CPT), piezocene penetration tests (PCPT) using 10 cm² and 15 cm² cones, seismic cone penetration test (SCPT), conductivity cone penetration test (CCPT), self boring pressuremeter tests (SBPT), and dilatometer tests (DMT). The cone penetrometer was miniaturized and a prototype system was implemented for highway design and construction control (Tumay and Kurup 1997).

The authors have recently developed and implemented a field ruggedized continuous intrusion
miniature cone penetration test system (CIMCPT) for shallow (c. 15 m) subsurface exploration. This system is mounted in a 4 wheel drive, one ton, all terrain vehicle (figure 1). A novel feature of this new in-situ testing vehicle is the chain driven caterpillar-type continuous push device (figure 2) powered by a hydraulic motor for advancing the cone penetrometer which greatly increases productivity and serviceability. Hydraulic power is provided from the vehicle’s transmission. Penetration rate is controlled by a pressure compensated flow control valve. The reversible hydraulic motor is capable of both inserting and/or retracting continuously the penetration rod which consists of a single continuous coiled tube. The cone rod is a 12.7 mm diameter stainless steel tube. It has a cone penetrometer attached to one end and a connector at the other. Coiling the thrust rod is one unique feature of this miniature cone system. Coiling eliminates threaded connections and simplifies cabling and waterproofing. The coiling mechanism also straightens the rod prior to insertion into the soil.

Two miniature cone penetrometers (Cone no. X01 and Cone no. X02) with projected cone area of 2 cm², friction sleeve area of 40 cm² and a apex angle of 60° were developed. They both are of the suction type (i.e. they measure cone resistance; and combined cone resistance and local sleeve friction resistance).

The penetration depth is measured by a displacement transducer that works via an optical incremental shaft encoder which is friction coupled to the rod. A Global Positioning System (GPS) installed in the vehicle outputs test locations directly to the computer via an RS-232 port.

A pentium notebook computer running at 100 MHz with 16 MB RAM and 810 MB hard drive capacity is used for data acquisition, processing and analysis. The data acquisition system consists of interface modules that convert analog input signals to engineering units and transmit, in ASCII, to any host with a serial port. This modular approach to data acquisition is extremely flexible, easy to use and cost effective since data is acquired on a per channel basis and one can add as many channels as required. Three modules are used: one each for acquiring cone resistance, sleeve friction, and depth. A data acquisition, processing and display software has been developed in Turbo C++ to acquire and display data on screen in real time. A printer is also available to obtain hard copy output and plots of the cone penetration profiles.

The CIMCPT provides a reliable, economical and time saving tool for site characterization compared to the conventional boring, sampling and laboratory test methods. The novel continuous feed device implemented in the CIMCPT system minimizes the

Figure 1. Continuous Intrusion Miniature Cone Penetration Test System (CIMCPT)
3 CALIBRATION OF THE C1MCPPT SYSTEM

The implementation of the miniature cone penetrometer is tested and verified by comparing the penetration profiles with those obtained using a standard 10 cm³ cross-sectional area reference cone penetrometer developed by Fugro. The 10 cm² electronic cone penetrometer has a friction sleeve area of 150 cm² and a 60° cone apex angle. It is essential to conduct tests at well-documented sites with homogeneous soil deposits to minimize the effect of soil variability on the measured data. The miniature cone is capable of detecting thin layers compared to the large size cones and this feature has to be taken into account while interpreting C1MCPPT data.

The C1MCPPT was field tested and calibrated near the intersection of Highland Road and Interstate 10 (LA SR-42) in Baton Rouge, Louisiana. The soil at the test site is overconsolidated, desiccated silty clayey silt formed during the Pleistocene period and deposited in a deltaic environment. The soil is of stiff consistency, low moisture content and fissured with slickensides and occasional sand pockets (Armman and McManis 1977). The ground water table was located at a depth of 4.5 m. Detailed piezoecone penetration tests, soil sampling and laboratory tests have been performed by Chen and Mayne 1994 to a depth of 34 meters. In the top 8 meters the liquid limit ranges from 52 to 76% and the plasticity index ranges from 26 to 40%. The soil is classified as CH material in the Unified Soil Classification System (USCS). The natural water content varies from 30 to 42%. The OCR decreases from 15.6 at a depth of 5.5 meters to 11.9 at 7.9 meters. The compression index (Cₜ) varies from 0.47 to 0.62 and the swelling index (Cₛ) ranges from 0.14 to 0.22. Isotropically consolidated triaxial compression tests (CIUC) show that the undrained shear strengths range from 60 kN/m² to 120 kN/m².

3.1 In situ testing and evaluation

Five C1MCPPT’s (MCPT1, MCPT2, MCPT3, MCPT4 and MCPT5) were performed at the corners of two equilateral triangular grids of 2.22 m length each side (figure 3). Two 10 cm³ reference cone penetration tests (CPT1 and CPT2) were conducted at the centroid of each triangles. The five C1MCPPT’s were conducted first before conducting the two reference CPT’s to minimize interaction and influence of soil disturbance on the tests results. At this site it was possible to

Figure 2. Continuous push device

normal stress release and consolidation effects on standard cone penetration data often observed during intermittent pushings. The C1MCPPT system can be operated by one person. The miniature cone penetrometer gives finer stratigraphic details compared to the standard size cone penetrometer making it more attractive for pavement subgrade characterization, construction control and assessment for ground improvement effectiveness. The soil data obtained can also be used in the design of short piles, piers and abutments supporting bridges. The system may be used to test beneath existing pavement via a 1°-diameter access hole and is hence less disruptive compared to larger diameter borings. The system can be mounted in small all-terrain vehicles due to smaller reaction forces needed to push the miniature cone as compared to large size cones. Installation in an easily manageable smaller vehicle provides greater mobility and site accessibility. The problem of ground water seeping in through joints (as in the conventional 1 meter long weighted push rods) and damaging the electronics is minimized since the push rod is one single continuous coiled piece.
conduct CIMCPT's to maximum depths ranging from 7.75 m to 8.75 m. The evaluation and analysis for this paper will be based on a maximum penetration depth of 7.75 m for consistency.

Figure 3. Test plan for the in situ calibration

Figures 4 shows CPT profiles compared with those of MCPT1, MCPT2 and MCPT3. Figures 5 shows CPT2 profiles compared with those of MCPT3, MCPT4 and MCPT5. When conducting in situ calibration it is essential to choose a site with uniform soil properties for sufficient depths. The soil variability in the lateral extent should also be minimum. Zones that indicate high variability such as thin soil layers and lenses of various soil types should be excluded from the analysis. The top 1.25 m consists of a stiff, desiccated deposit showing high variability in cone resistance. The soil deposit between 1.25 m and 4.5 m is heavily overconsolidated, desiccated clayey silt with stiff consistency, low moisture content and fissured with slickensides and occasional sand pockets. The presence of these sand pockets and lenses can be seen as increase in tip resistance between 4.0 m and 4.3 m in MCPT1; between 3.5 m and 4.25 m in MCPT2; at 2.8 m, and between 4.0 m and 4.5 m in MCPT4; and between 3.7 m and 4.0 m in MCPT5. The zones containing these sand pockets/lenses are excluded from the analysis and calibration procedure.

It is can be seen in figure 4 and 5 that the profiles of tip resistance, sleeve friction and friction ratio are uniform between 4.5 m and 7.75 m. Hence interpretation and analysis for calibration will be based on test data obtained between 4.5 m and 7.75 m. Table 1, shows the average values of cone resistance, sleeve friction and friction ratio (along with their respective standard deviations) between 4.5 m and 7.75 m.

It can be seen that the average MCPT cone resistance is higher than the average CPT cone resistance. The mean value of the MCPT cone resistance (q_c = 1.665 MPa) obtained from the average MCPT profiles is 10% higher than the mean value of the CPT cone resistance (q_c = 1.604 MPa). The mean value of the CPT sleeve friction (f_s = 86.15 kPa) is 12% higher than the mean value of the MCPT sleeve friction (f_s = 77.04 kPa). Hence the CPT friction ratio (RF = 0.50%) is 23% higher than the MCPT friction ratio (RF = 0.47%). Previous studies (de Lima 1990, de Lima and Timouy 1991, Timouy and de Lima 1992) have indicated a 15% higher cone resistance for a 1.27 cm³ miniature cone penetrometer compared to that of a 10 cm³ cone penetrometer. These results show that "scale effects" (due to difference in size and strain rate effects) do exist between CPT and MCPT data and should be taken into account while interpreting MCPT data for soil classification and for estimating soil properties. Detailed in situ tests to calibrate the CIMCPT system and to further study "scale effects" will be performed at National Geotechnical Experimentation Sites (NGES) in the south and north eastern United States. A more rigorous analytical study will then be performed with data representing a wide range of soil and in situ conditions.

Figure 4. Comparison of penetration profiles of MCPT1, MCPT2, MCPT3 and CPT1

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3.2 Undrained Shear Strength

The undrained shear strength, $s_u$, is an important soil property required for the analysis, design and construction of geotechnical systems. The miniature cone penetrometer may be used for rapid $s_u$ determination. It can also be used for the assessment of strength improvement after ground stabilization by preloading, compaction or by other means.

One of the commonly used semi-empirical methods to estimate $s_u$ is the one suggested by Lunne et al. (1985):

$$ s_u = \frac{q_c - \sigma_{mv}}{N_{AV}} $$

where $\sigma_{mv}$ is the total vertical stress that may be estimated from the depth of penetration and density of the soil deposit. Even for a given design of the cone penetrometer, various in situ state and soil properties are known to affect the empirical cone factor $N_{AV}$. The empirical cone factor could vary widely and values between 4 and 30 have been reported in actual practice. Several factors such as the lateral stress coefficient, plasticity, stress history, stiffness, sensitivity, micro and macro fabric of the deposit are known to be the cause for such wide variations. The authors believe that $N_{AV}$ also depends on the cone penetrometer design and also on the penetration speed. The miniature cone penetrometer is calibrated using the reference cone penetrometer at the same site to determine the influence of cone size on $N_{AV}$ values. Since $N_{AV}$ values are site specific it is essential to calibrate them for a particular site using reference undrained shear strength determined from laboratory triaxial tests or other laboratory in situ test methods. In this paper the reference shear strength is obtained from consolidated isotropic undrained compression (CIUC) tests. It has been reported (Chen and Mayne 1994) that the undrained shear strength at this site may be underestimated because the applied consolidation stress during CIUC tests might not have been adequate in closing the swelling cracks caused by high preconsolidation pressures and stress release. The consequence of underestimating the undrained shear strength is to overpredict the empirical cone factor during calibration. The penetration profiles of tip resistance are fairly uniform between depths of 4.5 m and 7.75 m and their mean values are used for determining empirical cone factors to estimate the undrained shear strength. The total overburden stress varies from 94 kPa at 4.5 m to 144 kPa at 7.75 m (114 kPa ± 30 kPa over a thickness of 3.25 m). An average total overburden stress value of 114 kPa may be assumed at mid depth. This simplified interpretation is justified at this particular site because the variation in overburden stress for the thickness considered is small compared to the magnitude of the cone resistance and the high preconsolidation stress (1.04 to 1.4 MPa). In fact the very high OCR decreasing from 15.9 at 5.5 m to 11.9 at 7.9 m is one of the reasons why the cone resistance is also fairly uniform (instead of increasing...
6 REFERENCES


Obtaining material properties for slope stability analysis of gold tailings dams in South Africa

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ABSTRACT: Material parameters for the slope stability analysis of gold tailings dams in South Africa are usually obtained from laboratory tests. Due to the predominantly fine sandy and silty nature of gold tailings it is difficult to obtain high quality samples, especially below the water table. It would therefore be useful to confirm laboratory parameters with in situ measurements. This paper describes and compares the results obtained from various in situ and laboratory test methods conducted as part of the investigation into the failure of a gold tailings dam in South Africa. It is becoming common practice in South Africa to determine the in situ pore pressure regime in tailings dams with piezoecone testing. A correlation between piezoecone results and other test results is described.

1. INTRODUCTION

The grading and relative density of slurry deposited on tailings dams may vary from time to time due to variations in the rock type mined, the reduction process and the water content of the slurry. This, together with the method of tailings dam construction (deposition strategy) results in the intense layering observed in the gold tailings dams in South Africa. The tailings generally comprise a large silt size fraction with varying fine sand and clay fraction. The material is essentially non-cohesive and therefore it is difficult to obtain high quality samples for laboratory testing. While it is possible to take samples of the more homogeneous layers of unsaturated material, it is virtually impossible to obtain quality samples below the water table. Laboratory tests are normally conducted on reconstituted samples. In situ tests can be performed on tailings dams, but it is often difficult to judge the quality of the results. Obtaining material properties for slope stability analyses of these structures is therefore difficult.

As part of assessing the general condition of a tailings dam, it is common practice in South Africa to conduct piezoecone tests. Currently the results are mainly used to determine the seepage regime in the dam for stability assessments and to obtain an indication of the presence of weak layers. Rust et al. (1993) described methods to determine the pore pressure gradient in a tailings dam from incomplete dissipation tests, as well as the anisotropy in permeability from the slope of the phreatic surface and the pore pressure gradient, thereby solving the complete seepage regime.

It is normally not possible to assign strength parameters using the piezoecone results. With the data base of properties available after the inquest into the Merriespruit tailings dam failure it was possible to draw correlations between various material properties and piezoecone results. This paper describes some of the tests conducted and their results. The results are compared and an attempt is made to correlate the piezoecone results with other test results.

2. BACKGROUND

On the night of 24 February 1994 a 31m high tailings dam failed on the Freestate Goldfields in South Africa after a late afternoon thunderstorm, resulting in 600 000 m³ of tailings flowing through the town of Merriespruit, killing 17 people and causing widespread devastation.

An extensive investigation, comprising a variety of laboratory and in situ tests, was conducted as part of an inquest to determine the cause of the failure (Wagener et al. 1997). The investigation included various test methods that are normally used to assess the condition of a tailings dam. The in situ tests included piezoecone and vane shear tests. Triaxial tests were conducted on both "undisturbed" and
reconstituted samples. Indicator tests on all tailings samples included grading analysis, Atterberg limits and moisture content tests.

The tailings dam, showing the failure and the test positions referred to in the paper, is shown in Figure 1.

Figure 1 View of Merriespruit tailings dam.

3. GRADING OF GOLD TAILINGS

A large number of gold tailings samples were subjected to indicator testing (grading analysis and Atterberg limits). The results of all the grading analyses are summarized in Table 1. The average grading and upper and lower bounds of the envelope, representing a 95% confidence interval, are shown.

Table 1. Summary of grading analyses.

<table>
<thead>
<tr>
<th>Fraction</th>
<th>Coarse sand fraction</th>
<th>Fine sand fraction</th>
<th>Silt fraction</th>
<th>Clay fraction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper limit</td>
<td>0%</td>
<td>5%</td>
<td>79%</td>
<td>16%</td>
</tr>
<tr>
<td>Lower limit</td>
<td>5%</td>
<td>50%</td>
<td>39%</td>
<td>6%</td>
</tr>
<tr>
<td>Average</td>
<td>2%</td>
<td>28%</td>
<td>59%</td>
<td>11%</td>
</tr>
</tbody>
</table>

The grading analyses show that the gold tailings is essentially a silt with varying fine sand and clay fractions. In some cases the fine sand fraction may however be dominant. The variation of the fine sand and clay fractions generally depends on the position of the sample relative to the delivery points on the dam. The tailings tend to be more sandy close to the delivery point, while the material furthest from the delivery tends to have a larger clay fraction. Grading analyses on samples of the slurry, obtained before deposition, show that the tailings has a fine fraction (silt and clay) of approximately 70%. The average for the gold mining industry of South Africa is 80% (Billight et al. 1979) and the Merriespruit tailings are therefore slightly coarser than the average for the industry.

During the life of the tailings dam the deposition strategy, for example the positions of delivery points, may change. Furthermore, a thin layer (1mm to 5mm) of silt and clay may be formed on the top of each deposited layer due to the fact that these fine particles settle only when moist water has evaporated. These factors result in the intensely layered structure observed in tailings dams. It is however clear that in spite of the observed layering, which may appear sandy and clayey, the material generally consists of a silt matrix with varying fine sand and clay fractions.

4. SHEAR STRENGTH

Gold tailings dams generally comprise of an unsaturated zone where high negative pore pressures may exist, above a fully saturated zone. Due to the lack of a generally accepted framework for describing the material behaviour in the unsaturated zone, discussion of this zone is limited in the paper. This paper essentially focuses on the determination of material parameters for the saturated zone.

4.1 Triaxial test results

Consolidated undrained triaxial tests with pore pressure measurement were conducted on a number of undisturbed and reconstituted samples obtained from the following sources:

1. Slurry samples obtained in boreholes PESA and PESB. One specimen at each depth was tested.
2. Two block samples obtained from the sides of the breach. Four specimens were tested from each of the block samples. These two samples were obtained to represent the finer tailings which were expected to have the lowest shear strength.
3. A reconstituted sample comprising material obtained from the breach. Four specimens were tested.

The results of the triaxial tests were used as follows:

- Firstly it was used to obtain the effective shear strength parameters of the tailings.
- Secondly an attempt was made to obtain a general framework for describing the material behaviour. If such a model existed it could be used to describe the undrained shear strength behaviour of the tailings.

The effective shear strength parameters obtained from the test results are summarized in Table 2. The shear strength parameters obtained from samples of various origins are remarkably consistent. It is evident that the tailings is essentially cohesionless, with all the samples except B1 showing a zero cohesion intercept. The internal friction angle varies between 18° and 30° degrees except for the
Table 2. Summary of shear strength parameters.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Cohesion (kPa)</th>
<th>Internal friction angle (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1 (Block sample)</td>
<td>8</td>
<td>38°</td>
</tr>
<tr>
<td>B2 (Block sample)</td>
<td>0</td>
<td>39°</td>
</tr>
<tr>
<td>B7 (reconstituted)</td>
<td>0</td>
<td>35°</td>
</tr>
<tr>
<td>PESA ( Shelby samples)</td>
<td>0</td>
<td>39°</td>
</tr>
<tr>
<td>PESA ( Shelby samples)</td>
<td>0</td>
<td>38°</td>
</tr>
</tbody>
</table>

reconstituted sample which shows a value of 35°.

It is the authors' opinion that this narrow range in the shear strength parameters results from the fact that the material behaviour is largely governed by the predominantly silt matrix of the material and the fact that the material particles, irrespective of size, are generally of the same mineralogy, a quartzitic rock mechanically broken down. This suggests that it may be possible to normalise the results to obtain a simplistic framework for describing the material behaviour.

The results can be normalised with respect to the normal consolidation line or the critical state line. Since no consolidation tests were available the results were normalised with respect to the critical state line. Most of the triaxial tests were conducted to axial strain exceeding 20% and most reached critical state. The specific volume, \( v \), was plotted against the mean effective stress at critical state, \( p'_{c} \), for the samples which had reached critical state. This plot is shown in Figure 2.

The plot shows that a critical state line (solid line) can be fitted with a fair amount of confidence. The exceptions are two of the results obtained from block sample B1. A possible critical state line for sample B1 (dashed line) is also shown. It should be noted that the majority of the tests conducted on samples B7 (reconstituted) and PESA did not reach critical state and are therefore not shown. The equation of the critical state line is:

\[
v = 3.56 - 0.695 \log p'_{c}
\]

where \( p'_{c} = p_{c}^{*} \).

The stress paths were then normalised with the mean effective stress at critical state, given the specific volume during the test. The results are shown in Figures 3 and 4. In Figure 3 the stress paths of the tests used to obtain the critical state line are shown. In Figure 4 all the stress paths are shown.

From the figures it is evident that all the normalised stress paths do not converge to the same critical state. As expected the stress paths in Figure 3 converge to the same critical state with the exception of two of the results from sample B1. In Figure 2 it is shown that sample B1 may have a different critical state line. From Figure 4 it is clear that the majority of the samples which did not reach critical state, does not converge to an unique critical state.

There is not sufficient data available to explain the observed behaviour, but it may be explained by the
following mechanism. Close examination of the grading of all the samples tested shows a variation in fines fraction of between 35% and 95%, a significant variation. In general samples B1, B2, B7 and PES1 have a larger fines fraction than sample PESA. Sample B2 is the finest with a 95% fines fraction. The samples dry of critical state show a small amount of contracting behaviour, which is followed by dilution. Due to strain localisation some of these samples did not reach the critical state. In three of the tests the dilution stage is followed by contracting behaviour. This is believed to be due to crushing of material at particle contacts. The samples wet of critical state show significant contracting behaviour which is followed by a phase transfer and dilution. The exception is sample B2 which does not dilute. A general trend of an increase in the extent of the dilution phase with a decrease in fines fraction, is observed. With an increase in coarser grain particles more energy is required to reach critical state. The critical state is therefore reached at a lower mean effective stress for samples with a smaller fines fraction, for example sample B2. Assuming the critical state of the fine grained tailings to be the critical state for all the samples tested is conservative. Based on this assumption the critical state shown in Figure 3 is taken as being representative of Merriepiut gold tailings in general. The following relationship can then be obtained from Figure 3:

\[ \frac{\sigma}{e_e} = 0.6 \]

(2)

or for undrained conditions:

\[ e_u = 0.6p_e \]

(3)

Using this relationship a conservative estimate of the undrained shear strength can be obtained if the void ratio of the gold tailings is known. A correlation between void ratio and penetration resistance is presented in Section 5.

4.2 Vane shear tests

As part of the Merriepiut site investigation, vane shear tests were carried out at approximately 1.5m depth intervals in boreholes PESA and PES1. The vane shear tests were conducted using a Norwegian type, 110mm long with a 55mm diameter.

The vane shear device was developed to measure the undrained shear strength of clayey soils. In attempting to measure undrained shear strength with the vane shear device, caution must be taken of the fact that conditions may not be undrained around the vane. In the analysis it was however assumed that conditions below the phreatic surface were undrained. Some doubt does, however, exist about the validity of this assumption due to short dissipation times recorded by the piezcone after interruption of penetration.

The shear strengths obtained with the vane shear are shown in Figure 6.

4.3 Piezcone results

Piezcone tests were conducted at numerous positions on the tailings dam, including 2 tests in close proximity to boreholes PESA and PES1. The piezcone results were used to determine the pore pressure regime in the tailings dam. It was also used to estimate the undrained shear strength of the tailings. A typical piezcone test result is shown in Figure 5. In the figure the penetration resistance, dynamic pore pressures and ambient pore pressure profile, determined from pore pressure dissipation data, are shown. During penetration some layers behaved normally consolidated, giving rise to positive pore pressures, while others appeared to be overconsolidated with negative excess pore pressures observed.

![Piezcone profile PESA](image)

Figure 5 Piezcone profile PESA.

In calculating the undrained shear strength of the material from the penetration resistance profile, it was assumed that cone penetration could be modelled as a bearing capacity problem obeying the modified Terzaghi bearing capacity equation:

\[ q_u = \lambda_s N_c \]

(4)

where \( q_u \) = cone penetration resistance; \( \lambda_s \) = shape factor; \( N_c \) = Terzaghi's bearing capacity factor for cohesion for general shear; \( c_u \) = undrained shear strength.

The piezcone results were used to calculate the
undrained shear strength ($u_c$) profile at the positions of boreholes PESA and PESB. The calculated undrained shear strengths are shown and compared to the triaxial and vane shear values in Figure 6.

4.4 Comparison of shear strength results

The undrained shear strengths obtained with the various test methods are plotted in Figure 6. Only results obtained below the water table are shown.

![Figure 6: Undrained shear strength determined from vane shear, piezocene and triaxial results.](image)

The piezocene and vane shear values significantly underestimate undrained shear strengths estimated from the triaxial test results. This can be explained as follows. In the tailings dam majority of the gold tailings in the zone comprising the tailings dam wall is believed to be over-consolidated and dry of critical state. Upon shearing of this material, the tendency is to dilate which results in pore pressure reductions if conditions are undrained. In the triaxial apparatus conditions are truly undrained and the result is that the critical state is reached at constant specific volume. In situ, drainage does occur during testing. Consequently the material dilates, hence increasing the specific volume so that the critical state is reached at a lower mean effective stress. The shear strength at failure mobilised in situ is therefore lower than that measured in the triaxial tests. The second reason for the discrepancy is that the location of the failure plane in the triaxial test is not reproduced. With the in situ tests the failure plane is however determined largely by the method of testing. The difference in failure modes will influence the shear strength mobilised at failure. These two factors may also be the reason for the discrepancy between the shear strengths determined from the vane shear and piezocene results. The significance of these effects is expected to be less for material wet of critical. In conclusion the over-prediction of the in situ "undrained" shear strength of the tailings from the triaxial test results is important to note. Even under the relatively quick in situ testing conditions, there seems to be some drainage. This should be kept in mind when conducting undrained stability analysis in gold tailings.

The undrained shear strength values determined from the vane shear and piezocene results correlate reasonably well. The shear strength calculated from the piezocene results generally underestimated the vane shear values. The correlation between undrained shear strength determined from piezocene results and vane shear tests was further investigated by attempting to obtain an empirical classification of the material type and consistency at the various test depths. The many layers were stratified into zones by means of a technique of trigonometric polynomial curve fitting, applied to the penetration resistance data, as described by Vermeulen et al. (1995). This produced representative penetration resistances and excess pore pressures for each zone. Using the representative penetration resistances and excess pore pressures, the material was classified according to the Janous & Rust soils identification chart (Jones et al. 1982). The material generally classified as loose sands, medium dense sands, loose silty sands or stiff silts. The material types and consistencies are shown in Figure 6.

It was generally found that as the consistency of the material increased (and the likelihood of being dry of critical state) the under-prediction of the shear strength determined with the piezocene became more significant. In other words, the correlation for material wet of critical state was better. The reasons given previously for the discrepancy between the shear strength values, viz. rate or testing and amount of drainage occurring, as well as the significance of the induced failure plane, would be more important for material dry of critical. The test results therefore seem to confirm the hypothesised behaviour.

5. CORRELATION BETWEEN PENETRATION RESISTANCE, SATURATED DENSITY AND VOID RATIO

During the investigation into the tailings dam failure, Shelby tube samples were taken at the depth of each vane shear test to determine the density profile of the material. The correlation between saturated density and void ratio with penetration resistance was investigated. Figure 7 shows a plot of saturated density and void ratio against penetration resistance.
Despite some scatter in the results it is apparent that a relationship between saturated density and cone resistance exists. If it is assumed that cone resistance is a function of density and shear strength, this indicates that the contribution of shear strength to cone resistance for this material is small. Hence it implies that a relationship between density and shear strength exists. This is further confirmation of the material behaviour discussed in Section 4.1.

Using this relationship, an indication of the void ratio and saturated density of the tailings can be obtained from piezocene results. This can then be used in the stability analysis, as well as to obtain an indication of the range of undrained shear strengths.

![Figure 7 Saturated density and void ratio versus penetration resistance.](image)

6. DISCUSSION AND CONCLUSIONS

Gold tailings is essentially a non-cohesive, fine sandy silt. The difficulty of obtaining high quality samples for laboratory testing, aimed at selecting material properties for stability analysis, is discussed. As part of the inquest into the failure of the Merriespruit tailings dam, various in situ and laboratory tests were conducted to determine the properties of the gold tailings. An overview of the results of the testing programme is presented.

The triaxial test results show that the effective shear strength parameters fall into a narrow band. This is due to the fact that the mineralogy of the material is essentially the same, irrespective of the grain size. The influence of variations in the fine sand and clay size fractions of the material is not significant. Further research is required to investigate obtaining effective strength parameters from in situ test results.

Normalisation of the triaxial test results with regard to a critical state line show that the critical state of the gold tailings is influenced by the fine sand fraction of the material. This has implications for the undrained shear strength determined from triaxial test results. Assuming that neglecting the influence of this variation is conservative, a relationship between specific volume and undrained shear strength is proposed.

Undrained shear strengths obtained from vane shear tests and piezocene results correlate reasonably well. Comparison of these shear strengths with undrained shear strengths obtained from triaxial test results show that in situ measured values are much lower than triaxial values. This indicates that conditions in the field are not undrained and also that the difference in modes of failure may influence the mobilised shear strength. Selecting parameters for undrained stability analysis should therefore be done with care. An evaluation of the degree of undrained behaviour should be made and parameters selected accordingly. It is recommended that values obtained from triaxial tests should not be used without careful consideration, as they may grossly overestimate the in situ mobilised shear strength.

Finally it is shown that there is a relationship between cone resistance and void ratio exists. This confirms that there is a relationship between density and shear strength for the material. The piezocene result can therefore be used to evaluate the in situ void ratio and the range of undrained shear strengths, firstly using the relationship between cone resistance and shear strength and secondly by using the void ratio and the relationship determined from the triaxial test results. These relationships should however be compared with test data from other tailings dams to confirm its validity for gold tailings in general.

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Correct use of cone penetrometer sensors to predict subsurface conditions

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ABSTRACT: When cone penetrometer testing (CPT) technology is used with in-situ sensors and probes to characterize subsurface conditions in environmental investigations, each sensor must be calibrated with high-quality, site-specific data to establish essential interpretation criteria. Mechanical, geophysical, and chemical sensor data collected for a site in South Carolina without such controls were misleading. Core logs obtained subsequently had major lithologic discrepancies with the soil classification based on the CPT sensor data. In addition, detailed core sampling and laboratory analysis showed that the sensor data on chemical contaminants included false-positive and false-negative results. In contrast, for a site in Nebraska, CPT data calibrated with high-quality site controls provided a detailed interpretation of subsurface conditions relevant to contaminant fate and transport. On the basis of the work in Nebraska, Argonne scientists are continuing to develop criteria to improve the interpretation of complex subsurface stratigraphy.

1 INTRODUCTION
Since the early 1970s, CPT technology has gained wide acceptance for use in environmental investigations. New and improved sensors for collecting in-situ geologic, hydrogeologic, and chemical data have made the CPT highly useful as an investigative and monitoring tool. The ability to collect subsurface data faster, cheaper, and more safely has fueled research to further advance CPT technologies.

Although most professionals warn that interpretation of CPT data must be based on and integrated with existing site controls (such as core holes, laboratory samples, and hydrogeologic testing), CPT is too frequently the only site investigative tool. Because the soil classification systems currently in use are empirical and because knowledge about the in-situ behavior of many environmental contaminants is limited, caution must be exercised in interpreting data collected with CPT technology. This paper discusses two cases where CPT data were used for site characterization. In the first case, the entire site was characterized geologically and for petroleum hydrocarbon contamination by using only CPT technology. A later study at this site determined that the interpretation of the CPT data was flawed. Had site controls been used, much time and money would have been saved. In the second case, existing geologic information and site controls (such as geologic core holes, surface and downhole geophysics, and sampling and analysis) were integrated with the CPT data. In this case, use of CPT technology to supplement and augment the existing data proved to be extremely valuable and highly cost-effective.

2 FIRST CASE HISTORY: SOUTH CAROLINA
A small land area (24.6 m by 18.0 m [75 ft by 55 ft]) owned by the U.S. Department of Energy (DOE) historically housed two aboveground diesel fuel tanks. Because of suspected fuel contamination, the subsurface was characterized in August 1995 by using CPT and the Rapid Optical Screening Tool (ROST™), a laser fluorescence technology developed as an in-situ method of investigating sites for petroleum contamination. Laser-induced fluorescence (LIF) has been used for a number of years in the laboratory to detect and analyze petroleum products. In LIF, a fiber-optic spectroscopic sensor measures the fluorescence generated when petroleum hydrocarbons are excited with ultraviolet laser energy. The screening-level data produced are expected to be supported by traditional sampling and analytical methods. At the DOE site, tip, sleeve, pore pressure, and ROST™ readings were collected at 14 locations (Figure 1) in and around the storage tank area. The CPT operator
Figure 1. Locations of CPT sampling and sensor data collection and of soil core holes at the DOE site.

stressed, in the cover letter accompanying the final report (Fugro 1995), that because the soil behavior chart was empirical, the soil identification should be verified locally. Nevertheless, the only additional work performed was the collection of 23 soil samples for laboratory analysis for total petroleum hydrocarbon (TPHs) (Burbage et al. 1996). Soil sample locations were selected on the basis of peak signal responses by the ROST™ sensor. Figure 2 shows 4 of the 14 data logs produced for the CPT-ROST™ sensors and the results of chemical analysis of the soil samples collected at those three locations.

In June 1996, Argonne National Laboratory, under contract with DOE, used the same area for an evaluation of LIF sensor technology (Argonne 1996). The site was selected for Argonne’s work because it was believed to be fully characterized for fuel contamination and to contain the contaminant concentrations needed for the evaluation. However, Argonne’s review of the previous CPT, ROST™, and soil sampling data revealed that the results of the soil sample collection did not support the ROST™ sensor data, which had produced both false-negative and false-positive readings (Figure 2). Therefore, during Argonne’s evaluation, two core holes were drilled and logged in detail by a geologist, then sampled continuously for petroleum hydrocarbon analyses (PAHs [polynuclear aromatic hydrocarbons] and BTEX [benzene-toluene-xylene]) to establish site control. After samples were collected for chemical analysis, the remaining core was sent to a geotechnical laboratory for grain analysis with a microscope. Each core hole was placed immediately adjacent to a CPT-ROST™ data collection location from the 1995 study in order to confirm earlier results (with core hole L01 adjacent to LT3B and core hole L02 adjacent to LT02).

Argonne began by logging the core. Figure 3 compares the original CPT log as interpreted by the soil classification software program on the truck with the core hole log described by the geologist and the laboratory after microscopic examination. Some major discrepancies are apparent. At location L02, the soil boring log was described by the geologist as being dominated by silt and clay in the upper 4.9 m (15 ft) and by sand at 4.9-9.5 m (15-31 ft). The CPT computer-generated soil classification log at this location identified the material at 9.8 m (32 ft) as clay, with sand and silt layers at 6.9-8.2 m (21-25 ft).

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The geologist’s log at location LT05B/L01 indicates that the upper 4.9 m (15 ft) was dominated by silt and sand, while the interval at 4.9-8.9 m (15-27 ft) was predominantly clay and silt. The CPT log was interpreted as clay with sand and silt layers at 3.3-4.6 m, 7.5 m, and 8.5-9.2 m (10-14 ft, 23 ft, and 26-28 ft).

In addition, during the June 1996 field work, a CPT truck equipped with an LIF sensor was pushed at two locations around core hole L01. The major objective was to evaluate the LIF sensor's ability to detect PAHs accurately in the soil. Figure 4 shows the results for one of the LIF sensors placed around core hole L01, along with analytical results for soil samples collected. Cluster 1 data were collected from either side of core hole L01. Additional data collected throughout the site (but not shown here because of space limitations) produced similar results. Little or no correlation between the LIF peaks and the TPH analytical concentrations is apparent. Virtually all of the contamination found at the site during the June 1996 effort was confined to the upper 4.9 m (15 ft) of material. The concentrations of the TPHs may not have been high enough to be detected by the LIF, or chemical complexing interactions with the clay material at the site may have caused erroneous readings from the sensor. The in-situ behavior of most contaminants is not well understood; clay is known to attenuate the signal produce by the LIF, but the reasons for the discrepancies in this study are not known.

This study clearly shows the importance of establishing good site control, both stratigraphic and chemical, when CPT sensor technology is used. In-situ sensor readings can rapidly identify problem areas without producing wastes. However, the interpretation of data generated can be meaningless without a thorough understanding of the site and the contaminants of concern.

3. SECOND CASE HISTORY: HUMPHREY, NEBRASKA

Humphrey is a small farming community (population 725) in eastern Nebraska. The town's drinking water supply is contaminated with carbon tetrachloride. Figure 5 shows the town and the study area. In June of 1995, Argonne began a site characterization at Humphrey to define the geologic and hydrogeologic regime and to determine the nature and extent of the carbon tetrachloride plume. During this investigation, CPT technology was successfully integrated with traditional technologies (drilling, sampling, geophysics, monitoring well installation) to meet these objectives.
The work at Humphrey began with construction of a conceptual model of the subsurface from data in existing borehole geologic logs. Test holes drilled and logged by the Nebraska Geological Survey were used to develop the geologic framework for the region. Local and on-site private and public wells logged by the drillers were matched, where possible, with the regional geologic logs to develop the preliminary conceptual model of the site's subsurface. The initial field work was designed to test and support the model.

In the field, the CPT sensor response was calibrated with a continuously cored drill hole installed to test the conceptual model and to establish the lithologic units in the stratigraphic sequence. The drill hole was logged for moisture content, and the saturated zones were identified. Downhole and surface geophysics were also used to map some of the major subsurface features; the geophysical data were integrated with the core hole geology and CPT sensor profiles.

Figure 5 shows the location of Area 1, where the initial calibration was performed, as well as the three areas where additional subsurface stratigraphy and hydrostratigraphy were required. Area 2 is believed to be the source area for the carbon tetrachloride contamination. Area 3 is immediately east of the contaminated public wells. Area 4 is southeast of the source, along the regional flow direction (E&E/FIT 1988).

Figure 6 shows the responses of the CPT sensors and the major lithologies described from the core at SB01. The CPT track was unable to penetrate into the main aquifer (a gravel with some boulders) at this location. Therefore, the CPT application was limited to investigation of the upper formations and the associated waters above the main aquifer. The water-bearing zones were identified as (1) a zone associated with the paleosol at 542.6-538.7 m (1,784-1,632 ft) above mean sea level (AMSL), (2) a sandy layer at 330.2-527.2 m (1,084-1,727 ft) AMSL, and (3) the top of the main aquifer at 510.2 m (1,670 ft) AMSL.

The CPT sensor data collected at Area 2 correlate closely with the geology described at Area 1. Although some variations were noted, the main lithologic changes and the major groundwater zones were identified. Chemical and isotopic analyses of samples from the groundwater zone at 530.8-532.2 m (1,741-1,756 ft) AMSL at Area 2 proved that this groundwater is the same as that present at similar depth at Area 1. The CPT track penetrated the main aquifer at this location at approximately 509.5 m (1,670 ft) AMSL and continued to 563.6 m (1,849 ft) AMSL before refusal.
Because of the excellent correlation between the CPT logs and the core hole logs at Areas 1 and 2, the CPT was used to collect data at Areas 3 and 4. The CPT logs (Figures 7 and 8, respectively) indicate a significant change in the subsurface stratigraphy at Areas 3 and 4. Results of the geophysical survey were used in an effort to understand these changes. However, surface seismic data did not reveal any major change in the profile at Area 3 (Figure 7). Therefore, minor conductivity responses produced by the CPT sensor at 528.2–528.6 m (1,610–1,610 ft) AMSL were interpreted as groundwater saturation. However, groundwater samples could not be collected at this elevation. A gravel unit was indicated by the CPT log at 507.9–505.2 m (1,514–1,514 ft) AMSL, and a groundwater sample was collected at this elevation. The material underlying this gravel unit was extremely compact, and the CPT truck was unable to penetrate below 495.4 m (1,590 ft) AMSL. Similarly, at Area 4 (Figure 8), a minor conductivity response at 529.9–528.2 m (1,615–1,610 ft) AMSL was interpreted as a saturated unit; however, no groundwater was encountered. Even after a four-hour period, no water was present in the CPT hole at this elevation. The CPT truck met refusal at 512.4 m (1,680 ft) AMSL, short of the main aquifer.

To better understand these variables, two additional core holes were drilled, one at each location. The core was logged in detail. Figures 7 and 8 show the correlation between the core and the CPT logs. At Area 3 (Figure 7), the core log shows the palaeosol to be partially saturated, as at Areas 1 and 2. The sandy zone seen at Areas 1 and 2 at approximately 528.2 m (1,610 ft) AMSL is absent at Area 3. The upper 10 ft of the boulder-gravel main aquifer was indicated by the CPT sensor data. However, the deeper portion of the main aquifer, as logged at Area 1, was not logged at Area 3. A compact, firm glacial till occurred under the boulder-gravel unit, overlying the bedrock.

The core log at Area 4 (Figure 8) shows saturation in the palaeosol, locally forming a perched aquifer. The groundwater zone at 528.2 m (1,610 ft) AMSL was a clayey silt with little or no sand that was nevertheless saturated. The main aquifer here was a mixed bed of coarse gravel to boulders, again underlain by a glacial till.

The differences from Areas 1 and 2 to Areas 3 and 4 show that changes in geology can occur over small distances. Without geologic confirmation, CPT logs can easily be interpreted erroneously. Integration and interpretation of the drilling, CPT, and surface geophysics data during the initial stage of the field investigation established the controls required to calibrate the CPT sensor responses with known geology and hydrostratigraphy. The calibrated responses successfully identified the main subsurface features affecting contaminant migration. Optimal sampling sites for determining the horizontal and vertical distribution of contaminants were selected by considering both the core and CPT sensor logs.

4 CONCLUSIONS

Using CPT sensor data without the proper site calibration and control can lead to misinterpretations. At times, even with geologic control and initial site calibration of the responses, the data can be misleading or difficult to interpret, leading to improper conclusions about the site, the contaminants, the potential for migration, and the remedial design. When CPT electronic logs are used correctly, no CPT computer-generated soil classification system is needed. The geologist should interpret the site’s CPT logs by using controls identified in the characterization program, including soil core logs, grain analysis, soil moisture data, and hydrogeologic data. Even after the initial site control and calibration are established, additional control
points may be required as the subsurface conditions change across the site. Conceivably, use of CPT technology can save time and costs in many site characterization and monitoring programs. Argonne's approach integrates core hole geology and laboratory sampling with resistivity, tip, sleeve, pore pressure, and analytical data produced by CPT to generate high-quality results.

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Vane test used for very soft soil-like materials characterization

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ABSTRACT: Modified vane was used to help in solving two particular geoenvironmental problems. The first was the research carried on re-suspension of river sediments and the second was the problem of reclamation of low-level radioactive tailing pond of MAPE Uranium Ore-Preparation Plant. Undrained shear strength of young river sediments in the Elbe river and in the tailing pond was determined with the use of enlarged vane and other methods. The aim of tests and their evaluation are described and discussed in the article.

1 AIM OF VANE TESTING

The research on re-suspension of river sediments was a part of an International Research Program called Elbe River Project and was carried out in the Water Research Institute of T. G. Masaryk in Prague. Re-suspension of fine-grained sediments due to variations of velocity of river stream was studied with the use of physical and numeric modelling. The role of the team from the Department of Geotechnics of CTU was to determine input data of the river sediment, to design the way of undisturbed sampling for physical modelling and to check the quality of samples installed in the hydraulic-flow model. Within input data set the undrained shear strength dominates. Because of the need of testing on the site and inside the hydraulic-flow model the laboratory and enlarged field vane were used, Zálešek et al (1997).

After "MAPE Uranium Ore-Preparation Plant" Mydlovary wound up its operation, Czech Republic faces the important problem of maintenance of large tailing ponds. The waste from the processed ore was, after its neutralization by lime milk, deposited in layer of thickness up to 25 m to exhausted opencast coal quarry in form of a thixotropic mass. There is a large set of geotechnical problems, one of them is to determine bearing capacity of tailings to carry the load from capping structure. Numeric modelling of the process of capping of tailing pond needs determination of variations of shear strength and deformation modulus in horizontal and vertical directions. Both was investigated with the use of standard vane and plate loading tests. Based on consulting of problems of capping of tailing pond with CTU and on the experience with enlarged vane the enlarged vane was used for completion and approval of set of tests.

2 VANE USED FOR TESTING OF RIVER SEDIMENTS

Testing of undrained shear strength during the laboratory hydraulic-flow modelling of re-suspension of fine-grained sediments was necessary from two points of view:

a) this was the main factor of resistance against its re-suspension and basic input data for numeric modelling,

b) screening the undisturbance and quality of sediments samples installed in hydraulic model before physical modelling.

For determination of undrained shear strength during the hydraulic-flow modelling was chosen laboratory vane with conventional torque spring Wykeham Farrance. It was necessary to prove conformity of results of laboratory vane and cone tests on one side and laboratory and field vane test on the other side. It was decided to perform laboratory vane test at first and to prove its feasibility because of material which had very high moisture content and porosity and differed from a soil.
Figure 1. Comparison of laboratory vane and cone tests

Figure 2. Laboratory vane and cone test compared with field vane tests
2.1 Tests of conformity and results

All the tests were carried out on the river sediment having at least 75% of fines (clay and silt particles) and they were classified as clay with extremely high plasticity.

Test were carried out in several steps:
1. The first tests were carried out on undisturbed samples in 50 mm sampling tubes. The aim was to approve feasibility of tests on very soft and sticky material; one of the set of tests is labeled “vane prelim.” in figure 1. Undrained shear strength was determined successfully with laboratory vane using simple formula for calculation:

\[ c_u = \frac{M_t}{K_s} \]  

(1)

where \( M_t \) = measured torque moment (kNm) and \( K_s \) = the relation describing geometry of used vane (m\(^3\)). Because of extremely soft sediment the biggest vane 25.4 \times 25.4 mm was used with the softest torque spring.

2. The second set of undisturbed samples was used for tests of conformity of the laboratory methods. Values obtained by the vane were compared with the results of laboratory cone test using the cone with angle of 60° and varying weight from 60 to 230 g depending on sediment parameters to obtain cone penetration about 10 mm. Vertical displacement of cone was measured by LVDT. Karlsson and Eichler formulas were used for undrained shear strength calculation:

\[ c_u = k \cdot \frac{P}{h^2}, \]  

(Karlsson)  

\[ c_u = \frac{P}{2} \cdot \frac{1}{(1+\phi)} \cdot \frac{1}{h} \cdot \frac{\tan \phi}{2}, \]  

(Eichler)  

(2)

where \( c_u \) = undrained cohesion (kPa), \( k = 0.025 \) - \( 0.035 \) (recommended range) coefficient by Karlsson \( \phi \) = vertical load \( P \), \( h \) = penetration of cone (mm) and \( \phi \) = cone angle (°).

The results are presented in Figure 1, twice for Karlsson’s formula (upper and lower limits of \( k \)).

3. On the basis of these results (undrained cohesion from 1 to 10 kPa) the size of field vane 180 mm both height and diameter has been designed. Several bars were made to be able to achieve up to 9 m under the river water-level. The Fannell field vane apparatus was fixed on the ship and used for measurement in different depths from water-level. On the same place the undisturbed sampling was carried out to obtain samples in 50 mm dia tubes for parallel laboratory tests, laboratory vane and cone test. The results with good conformity are presented in figure 2.

4. Finally, it was possible to use the field vane in the place where undisturbed sampling for hydraulic-flow modelling was carried out. The results were taken as the base value for quality testing of the sample being installed in hydraulics-flow model. Undisturbed sampling for hydraulic-flow model was carried out by diver with the use of special thin steel boxes of dimensions 500 x 300 x 80 mm. The boxes were used for transport of the samples and they were placed directly to the hydraulic-flow model. After placing the boxes undrained shear strength was checked with laboratory vane, Figure 3. Because of dimensions of the field vane, the undrained cohesion was measured in the layer having bigger thickness than in the hydraulic-flow model.

Four experiments were carried out in the hydraulic-flow model using two undisturbed samples for each test. The samples were placed to 1,0 m long cavity in the model and the undrained cohesion was tested with laboratory vane having a long shaft to reach the samples under the water.

2.2 Summary

The main aim of our part of the research - to determine the input data for physical and numeric modelling - was achieved. The possibility of testing of re-suspension of river sediments in the hydraulic-flow model was approved from the geotechnical point of view with the use of tests of conformity of different means of testing and of different vanes and taking into account special undisturbed sampling.

Comparing the final results being presented in the Figure 3, we can summarize, that the laboratory vane test, field vane and cone tests were used with good conformity with the exception of Karlson’s formula with high limit of coefficient \( k \).

3 VANE USED FOR TESTING OF TAILING

General reclamation of tailing ponds by means of classic covering with soil turned out to be technically very difficult. Mostly, the heavy layer of soil sinks in muddy subgrade. Using of auxiliary stabilizing elements (geotextiles, geo-drains etc.) increases costs and their application is technically difficult in given conditions.

Presently, solidified waste from burning of solid fuels (fly ash, slag and by-product of desulfurization) is used for reclamation. This waste has necessary geomechanical properties. Its volume weight is 8 -11 kN/m\(^3\), angle of internal friction is greater than 26° when saturated and could reach 36°. After depositing in layers the
Figure 3. Field vane tests and tests of installed samples in hydraulic-flow model with laboratory vane.

Figure 4. Plate load test of tailing.

Table 1. Vane test results overview

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Vane Type</th>
<th>Vane Edge under Water Level (m)</th>
<th>Water Level above Flap Surface (m)</th>
<th>Vane Shear Strength (kPa)</th>
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<td>HOLE 15 C</td>
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<td>3,5</td>
<td>2,1</td>
<td>9,34</td>
</tr>
<tr>
<td>HOLE 15 D</td>
<td>big</td>
<td>3,1</td>
<td>2,1</td>
<td>13,09</td>
</tr>
<tr>
<td>HOLE 16 A</td>
<td>small</td>
<td>3,2</td>
<td>0,7</td>
<td>10,84</td>
</tr>
<tr>
<td>HOLE 16 B</td>
<td>big</td>
<td>3,6</td>
<td>1,2</td>
<td>5,30</td>
</tr>
<tr>
<td>HOLE 16 C</td>
<td>big</td>
<td>3,5</td>
<td>1,2</td>
<td>6,01</td>
</tr>
<tr>
<td>HOLE 16 D</td>
<td>big</td>
<td>3,5</td>
<td>1,2</td>
<td>15,56</td>
</tr>
<tr>
<td>HOLE 16 E</td>
<td>small</td>
<td>0,1</td>
<td>1,2</td>
<td>2,78</td>
</tr>
</tbody>
</table>
material is rigid enough for safe travelling of heavy trucks. The material has also relatively small permeability (1.10^-8 to 1.10^-5 m/sec).

From geotechnical point of view, the most problematic thing is the response (embankment edge stability, subgrade bearing capacity and its deformation) of slurry radioactive pulp after rising of the solidificate layer on the projected level (up to 7 m in the center of the tailing pond). To assess pulp geotechnical parameters, many field geotechnical tests were performed on Tailing Pond III, which is already mostly covered by first so-called construction maintenance layer (about 2.5 m thick) of solidificate. Static penetration tests by means of GOUDA HOLLAND mobile set, small and large static load plate test and mainly field vane tests were performed on surface and under surface of the radioactive pulp deposited in the tailing pond. Purpose of the field tests was to assess the interval of total pulp cohesion and divide the tailing pond into geotechnical quasi-homogeneous units. These units will create the basis for design of modifications to the technology of solidificate depositing on radioactive pulp surface.

3.1 Laboratory Tests

The radioactive pulp granularity is mostly a fine silty sand with variable clay fraction (up to 20% of volume). Volume weight of moist pulp was between 1380 - 2080 kg/m³, dry volume weight was between 620 - 1770 kg/m³. Specific weight of preparation plant mud was assessed to be between 2670 and 2760 kg/m³. Total porosity was between 34 and 77%. The lowest porosity values were recorded by samples with higher percentage of sand. Other laboratory tests revealed that the pulp has high salinity with pH between 6.3 and 7.5 and high concentration of Mn, Be, NH₄⁺, Mg etc.).

3.2 Set of Field Tests

Vane tests were performed by a vane with diameter 0.0765 m (height h = 0.155 m) and by a vane with diameter 0.18 m (height 0.18 m). Friction between the soil and the rod was assessed by rotating the rod without vane and was taken into account when calculating undrained cohesion. Rectangular vanes were used in both cases. Altogether 16 vane tests of pulp were
performed at five places of the tailing pond. Table 1 shows results of all performed vane tests.

The shear strength revealed by tests ranges in a wide interval from very small values of tens of kPa - up to 15.6 kPa. In the case of 11 vane tests, undrained pulp cohesion was determined higher than 5 kPa (up to mentioned 15.58 kPa). Results given by the small and the big vane in similar conditions (the same testing place) were well comparable. No relation was found between results of tests performed on various places (positional dependence) and depths of the tailing pond. In some cases, a vane test result revealed that a deeper layer has lower total cohesion than the layer above it.

This indicates a chaotic depositing of pulp into the tailing pond. It results partly in differently granular environment in the tailing pond with locally or very different porosity, and partly in various "consolidations" on individual places by chemical additions from the preparation process.

In evaluation of radioactive pulp geotechnical data within the area of tailing pond, results of cone penetration testing (CPTU), Figure 4 (performed out of the vane tests area) as well as small static load plate tests on and below the pulp surface (close to vane tests area), were taken into account, as well as small static load plate tests on and below the pulp surface (close to vane tests area). Moreover, in terms of maintained Tailing Pond III - Oleník field monitoring, a field loading test of a solidficate layer edge (ash embankment of about 4 m height) was performed. At the same time, the substrata was partially displaced (displacement of the embankment into the tailing pond). The regressive stability analysis confirmed undrained pulp cohesion 8 kPa in the place of the great loading test. 3.3 Summary

Successful maintenance of tailing ponds around MAPE Mydlovary, which contains great amount of radioactive pulp from the preparation process, also requires determination of its shear parameter. The vane tests proved that pulp was deposited (deposited) into the tailing pond very chaotically and with different granularity.

Undrained pulp cohesion varies from 0.3 to 15.6 kPa. No regularity of mentioned variable change with depth under pulp surface was found. Also in the area of the monitored mudsetting pit, the cohesion value changes randomly.

Mentioned pulp undrained cohesion values well correspond with results of loading tests and cone penetration tests (CPTU) on the mud-setting pit. The undrained pulp cohesion characteristic value is very hard to determine and its value will be probably stated with the help of numeric modelling of stability of earping.

4 CONCLUSIONS

The very simple vane test was successfully used for very soft soil-like materials characterization. Enlarged vane was for very soft material testing after several tests of conformity. Due to very fine grained material the agreement of results obtained by standard laboratory and enlarged field vane was very good.

By the tests carried out in the river stream the thickness of young river sediments being very sensitive to erosion and re-suspension was determined as well as undrained cohesion. Sets of vane tests were good tool for quality checking of undisturbed samples for physical modelling. Test of radioactive tailings were carried out according to national health and safety regulations.

In this case the vane test is a very important mean of strength and variability of pulp testing, because there is no need to transport samples out of the lagoon. It is quite easy to clean vane and rods. Together with modified plate load test the above mentioned methods are very important in this case of site investigation. The exposition of humans could be significantly decreased and there is no danger of tailing handling and transport out of lagoon.

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Advanced interpretation for strength and deformation
Application of in-situ testing on cohesive soils

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ABSTRACT: The last twenty years have been characterized by significant developments in the area of in-situ testing. The most relevant feature of this period is a better understanding of the relationships between the results of in-situ tests and basic soil behavior. This last fact has contributed to a remarkable rationalization of the interpretation of different kinds of in-situ tests and of the use of their results in design. The use of in-situ testing results in geotechnical design may be split into two distinct approaches; the direct approach which gives the opportunity to pass directly from in-situ measurements to the performance of foundation without the need to evaluate any intermediate soil parameters, and the indirect approach which leads to interpretation method that allow evaluation of the parameters describing the stress, strain, strength and consolidation behavior of soils. This paper presents interpretation of in-situ tests conducted in very soft to clay soil. Emphasis is placed on the cone penetration test (CPT), standard penetration test (SPT), vane shear test (VST) and unconfined compression test (UCT). Empirical relationships were developed to forecast shear strength from penetration resistance. The empirical correlation were compared with previously published data.

1 INTRODUCTION

The cone penetration test (CPT) has been used for decades to investigate the properties of soil in-situ. It has been increasingly used because of important advantages it offers, such as simplicity, speed and continuous profiling. Originally, the penetrometer was used to measure only tip resistance. Over the years, sensors have been incorporated into the cone to measure the friction along a sleeve, the arrival of seismic shear wave, pore pressure and other quantities. This versatility has further promoted the use of CPT, but the underutilized advantage of CPT, is that the penetration process is amenable to theoretical modeling. In-situ tests have complicated boundary conditions and involved significant stress and strain variations within the soil and uncontrollable drainage conditions. Therefore, interpretation of in-situ test results is difficult and requires varying degree of empiricism in estimating soil parameters for design purposes.

On the other side, laboratory tests generally provide well-defined, controllable boundary and drainage conditions in addition to uniform stresses or strains within soil specimens, thereby enabling easy interpretation of test results. The major disadvantages of laboratory testing are sampling disturbance and uncertainties associated with estimating the in-situ spatial variation of soil parameters from the very small volume of soil normally tested.

Many investigators have analyzed cone penetration as a bearing capacity problem (Duncan & Mitchell 1975), (Jain & Suresh 1974). These theories assume different experimental verified failure mechanisms, which are then used to calculate ultimate bearing capacities with the limit equilibrium approach. Cone penetration in soils of low permeability may lead to excess pore pressure $\Delta u$. The effective stress interpretation requires a bearing capacity formula in incorporation with $\Delta u$. The appropriate bearing capacity formula in terms of effective stress is given by Senneset & Jumna (1985) and Senneset et al. (1989).

A theory based on cavity expansion and stress rotation analysis was developed for computing the cone penetration resistance of sand (Bolgado et al. 1995).

Ko and Law (1987) presented a combined cone resistance and pore pressure method to evaluate the preconsolidation pressure in normally consolidated to lightly overconsolidated clay.

Sully et al. (1988) related the OCR of the soil to a normalized pore pressure difference parameter (PPD) defined as,
\[
\left[ (u_t - u_0) / u_r \right] \quad (1)
\]

where,

- \( u_t \) penetration pore pressure measured on face of the cone,
- \( u_0 \) pore pressure measured behind the cone tip,
- \( u_r \) equilibrium in-situ pore pressure.

Many of the derived soil properties are often dependent on empirical correlation ideally based on some theoretical framework (Wroth, 1984). Sully & Campanella (1991) considered the pore pressure distribution around a penetrating cone in cohesive soils, and correlated the measured values to lateral stress condition.

The development of penetrometers with multiple piezometers has supported the idea that the pore water pressure measured at different locations can be related to changes in stress history. Several relationships have been proposed between excess pore water pressure (\( \Delta u \)) and \( S_u \) based on theoretical or semi-theoretical approaches using cavity expansion theory (Campanella & Robertson 1985).

The normalized form of pore water pressure \( (u_t - u_0) / u_r \) was correlated to lateral stress condition \( S_u \) (Sully & Campanella 1991).

In case of piezometers, Robertson & Campanella (1983) suggested that, based on cavity expansion theory, the undrained shear strength of normally consolidated clays can be estimated as:

\[ 3 < \left( \Delta u' / S_u \right) < 5 \quad (2) \]

Where,

- \( \Delta u' \) measured - \( u \) equilibrium.

Interpretation of soil strength from CPTU is dependent on drainage condition. Generally, undrained shear strength \( S_u \) is determined from undrained penetration. Comprehensive reviews of \( S_u \) evaluation from CPT and CPTU data have been presented by Baligh et al. (1980), Lusue & Kleven (1981), Jamiołkowski et al. (1982) and Robertson et al. (1986). The evaluation is complicated by the fact that \( S_u \) is not a unique parameter and depends on type of test, rate of strain and orientation of failure planes, among other parameters (Wroth 1984).

Campanella & Robertson (1988), suggested that the reference \( S_u \) should be taken as the field vane (VST) value.

Three distinct approaches are used for interpreting soil shear strength \( S_u \) from cone resistance \( q_c \). In the first approach, \( q_c \) is corrected for penetrating pore water pressure and \( q_c \) is calculated, in the second, \( q_c \) is used directly without any corrections, and in the third approach, the effective cone resistance \( q_c' \) is obtained and used instead of \( q_c \).

Campanella & Robertson (1988) reported that

\[ S_u = \left( q_c - \sigma_3' \right) / N_{ST} \quad (3) \]

Where,

- \( q_c \) cone resistance corrected for pore pressure effects, \( q_c = q_r + u \Delta u \)
- \( \sigma_{3'} \) Total overburden pressure
- \( N_{ST} \) cone factor,
- \( A_c \) total area of cone tip, \( A_T \) area of lower section of friction sleeve.

The cone factor \( N_{ST} \) varies between 4 and 30 depending on some factors, such as sensitivity, stress history, stiffness, macrofabric and definition of \( S_u \) (Campanella & Robertson 1988).

The undrained shear strength of the clay \( S_u \) has been estimated from the measured cone bearing \( q_r \) using the following bearing capacity equation:

\[ S_u = \left( q_r - \sigma_3' \right) / N_i \quad (4) \]

From a study of highly uniform deposits of soft clay, Parkin (1988), reported that, Lusue et al. (1978) calculated cone factor \( N_i \) varied from 7 to 27, with a clear correlation with clay plasticity (Aas et al. 1980). When Bjerrum correction factor was applied to the vane shear strength, a value of 17 for cone factor was reported.

Senneset et al. (1982) have suggested the use of effective cone resistance \( q_r \) to determine \( S_u \), where \( q_r \) is defined as,

\[ q_r = q_r - u \quad (5) \]

In which,

- \( u \) total penetration pore water pressure measured immediately behind the cone tip.

But, in soft normally consolidated clays, the total pore pressure \( u \), generated immediately behind the tip is often approximately 90 % or more of the measured cone resistance \( q_r \), the difference between \( q_r \) and \( u \) is often very small. The \( u \) is often an extremely small quantity and is therefore sensitive for small errors in \( q_c \) measurements.

Peak and residual cohesion intercepts are used to develop semi-empirical relationships for tip resistance, sleeve friction and friction ratio in weakly cemented sand, unconfined compressive strength of less 60 kPa (Puppala & Senneset 1993), (Puppala et al. 1995).

Relationships between cone penetration tip resistance and the liquefaction potential of sandy soils are developed to facilitate the use of cone penetration test CPT in liquefaction assessments (Stark & Olson 1995).
An extensive study has been made by Stroud (1974) concerning the standard penetration test (SPT) in insensitive stiff clays and soft rocks. The study indicated that the SPT could be used to estimate the properties of a variety of stiff clays and weak rocks. Empirical relationship has been presented by Stroud which gives the mass shear strength \( S_{h0} \) as a function of the standard penetration resistance as

\[
S_{h0} = f_1 \cdot N \quad \text{in kN/m}^2 \tag{6}
\]

Where,

\( f_1 \) : Parameter varies with the plasticity index of clay,
\( f_1 \) has values varies from 5 to 4 as the plasticity index varies from 20 to 70.

Stroud stated that the mass shear strength of fissured London clay may be only one quarter to one half of the shear strength of the intact material \( S_{h0} \); consequently, the shear strength of intact clay may be expressed as:

\[
S_{h0} = (0.25 - 0.18) N \quad \text{in kN/m}^2 \tag{7}
\]

The available data in literature indicated that, very few research works are dealing with standard penetration testing in clays, and the subject needs more research works to be carried out to correlate the measured \( N \) - values with the properties of clays. Also, the research works available indicated that the cone factor depends on plasticity index, among other factors, these factors need to be investigated.

2 FIELD TESTS

Mechanical cone penetration tests were conducted at three locations in a project site at the eastern side of Suez canal in Sinai, Egypt (figures 1, 2, 3). The test consists of pushing a penetrometer with a standard geometry, cylindrical with a diameter of 35.7 mm and a conical point with an apex angle of 60° into the soil at a rate of 20 mm/sec, while measuring the tip resistance and the local friction. Continuous records up to a depth of 50m were obtained. The CPT results reflect that the soil at the site is homogeneous. The cone penetration tests were carried out by soil mechanics laboratory, Cairo University.

In saturated clay soils having low permeability, penetration occurs in an essentially undrained mode, so in this case the results of penetration tests used for evaluation of shear strength in terms of total stresses are reflected in the undrained strength \( S_u \).

Ilahig & Levadoux (1980) and Ilahig (1985) indicated that 50% of the excess pore water pressure (\( A_w \)) caused by CPT penetration requires more than 2 to 5 minutes to dissipate, so the test is run virtually in undrained conditions. Also, Elsworth (1993) reported that the pore water pressure at a piezocene tip driven in clay becomes steady within approximately 1 sec. Following driving initiation at a standard rate of 20 mm/sec. Shallow pressures within 10 radii of the tip equilibrate within 10 sec. Recovered samples from four boreholes conducted at the project site, revealed that the soil at the site consists of very soft / soft clay, as shown in figure (4). The clay is of alluvial deposit, normally
Figure 3. Cumulative friction ratio vs depth

Figure 5. Standard penetration test results

Figure 4. Soil profile

Figure 6. Variation of liquid limit, plastic limit and water content with depth

deposit using the Geonor Push-in type vane boxes. Obtained results versus depth are presented in figure (7).

Undisturbed samples recovered from boreholes using Shelby tube, were tested in unconfined compression apparatus. The unconfined compressive strength of the undisturbed samples was drawn with depth, figure (8). The figure indicates scatter in the obtained results due to sampling disturbance.
3 INTERPRETATION OF RESULTS

Procedures to evaluate $S_u$ in saturated cohesive deposits from the results CPT's can be grouped as either theoretical or empirical. The former refers to bearing capacity theories, based on different failure mechanisms (Vesic 1972), (Doligh 1986). According to this theory, the approximate $S_u$ is expressed as:

$S_u = \frac{(q_0 - \sigma_vd)}{N_{HT}}$  \hspace{1cm} (8)

Where,

$S_u$ undrained shear strength,
$q_0$ total cone resistance,
$\sigma_vd$ initial oedometer total stress at the depth where $q_0$ has been measured,
$N_{HT}$ point resistance penetration.

The theories of expanding cavities in linear-elastic perfectly plastic soil can, also, be used to predict $S_u$ on the basis of the penetration pore pressure measured during the CPTU (Campbell & Robertson 1988).

The empirical approach for assessing $S_u$ is based on:

$S_u = \frac{(q_0 - \sigma_v)}{N_h}$  \hspace{1cm} (9)

Where:

$N_h$ nondimensional empirical cone factor.

It has been assumed that there is a unique property of a given clay termed the undrained shear strength ($S_u$). It is well recognized that this property does not have a single value when determined experimentally for two reasons (Wright 1988); the strength is affected by the condition and size of the sample tested, and the strength depends on the type of test by which it is determined.

Subject to the limitation of loading ratio, calibration becomes a matter of relating observed cone resistance to direct measurements of shear strength of clay ($S_u$), from which a cone factor may be evaluated. The choice of a reference standard will normally be between the in-situ vane test ($S_{vun}$) and laboratory unconfined compressive strength ($S_{cu}$) of undisturbed samples.

The field vane strength ($S_{vun}$) has been generally the reference standard, not because it is not without limitation, but because it is the most convenient method (Ladd et al. 1977). The particular limitations of the vane test as well as the CPT have been discussed by (Schmertmann 1975). Some apply equally to the unconfined compressive strength, as for example the dependence of $S_u$ on strain rate, and possible anisotropy, uncertainties about the shape of the failure surface and the effect of progressive failure.

Figure (9) illustrates the variation of cone factor $N_h = \frac{(q_0 - \sigma_v)}{S_{cu}}$ with the depth of the clay deposits. The figure indicates that the cone factor $N_h$ is dependent upon the total overburden pressure ($\sigma_v$) at depth where $q_0$ has been measured and a unique value cannot be obtained. A factor $\zeta$ was introduced to the cone factor in a way that the values ($q_0 - \sigma_v$) / $S_{cu}$ were shown against the depth of
where, $S_{uw}$ unconfined compressive strength of the clay. The figure confirm that $(q_u - \sigma_{la}) / S_{uw}$ has a unique value of 19.76. The correction factor $\zeta$ was expressed as:

$$\zeta = \frac{\sigma_{uw}'}{75} , \sigma_{uw}' \leq 75 \text{ kN/ m}^2$$ (14)

and

$$\zeta = 75/ \sigma_{uw}' , \sigma_{uw}' \geq 75 \text{ kN/ m}^2$$ (15)

Figure 9. Variation of cone factor ($N_k$) and modified cone factor ($N_{m}=\zeta$) with depth.

The local friction factor ($\alpha$) was related to local friction $f_l$ measured by friction sleeve, and the vane shear strength $S_{uv}$ via the equation:

$$\alpha = \frac{f_l}{S_{uv}}$$ (16)

Figure (11), illustrates that the local friction factor $\alpha$ is dependent upon the overburden pressure and has not a unique value. A correction factor $\zeta_{uv}$ was introduced in a way that, the local friction factor is modified as: $f_l = \zeta_{uv} \cdot \alpha$ . The modified local friction factor $\alpha_{uv}$ has a unique value of 1.22.

The correction factor $\zeta_{uv}$ is expressed as:

$$\zeta_{uv} = \frac{\sigma_{uv}}{100} , \sigma_{uv} \leq 100 \text{ kN/ m}^2$$ (17)

and

$$\zeta_{uv} = \frac{100}{\sigma_{uv}'} , \sigma_{uv}' \geq 100 \text{ kN/ m}^2$$ (18)

The local friction factor ($\alpha$) was related to local friction $f_l$ measured by friction sleeve, and the vane shear strength $S_{uv}$ via the equation:

$$\alpha = \frac{f_l}{S_{uv}}$$ (16)

Figure (11), illustrates that the local friction factor $\alpha$ is dependent upon the overburden pressure and has not a unique value. A correction factor $\zeta_{uv}$ was introduced in a way that, the local friction factor is modified as: $f_l = \zeta_{uv} \cdot \alpha$ . The modified local friction factor $\alpha_{uv}$ has a unique value of 1.22.

The correction factor $\zeta_{uv}$ is expressed as:

$$\zeta_{uv} = \frac{\sigma_{uv}}{100} , \sigma_{uv} \leq 100 \text{ kN/ m}^2$$ (17)

and

$$\zeta_{uv} = \frac{100}{\sigma_{uv}'} , \sigma_{uv}' \geq 100 \text{ kN/ m}^2$$ (18)

1214
The unconfined compressive strength of clay was used for assessing the local friction factor \( \alpha \) (\( \alpha = \frac{\sigma}{S_{uv}} \)), figure (12). The figure indicated that a modified value \( \zeta \) should be introduced to obtain a unique value of 1.32. The modified local friction factor \( \alpha_{mod} \) is expressed as:

\[
\alpha_{mod} = \zeta \frac{\sigma}{S_{uv}}
\]  \tag{19}

where,

\[
\zeta = \frac{\alpha_{mod}}{75}, \sigma_{mod} \leq 75 \text{ kN/m}^2
\]  \tag{20}

and

\[
\zeta = \frac{75}{\sigma_{mod}}, \sigma_{mod} > 75 \text{ kN/m}^2
\]  \tag{21}

Figure (13) illustrates a correlation between the measured \( N \)-values and the vane shear strength \( S_{uv} \), \( S_{um} \). From the figure the following correlation can be drawn:

\[
\frac{N}{S_{uv}} = 0.26
\]  \tag{22}

Where,

\( S_{uv} \) undrained shear strength obtained from field vane test, kN/m².

In case of unconfined compressive strength of clay, the correlation may be expressed as;
\[
\frac{[N/\sigma_{\text{v}}]}{S_{\text{un}}} = 0.0039
\]

(23)

Where,

\( \sigma_{\text{v}} \) effective overburden pressure at depth where \( N \) has been measured, kN/m²

\( S_{\text{un}} \) undrained shear strength obtained from unconfined compression test, kN/m²

Figure 14 illustrates the variation of \([N/\sigma_{\text{v}}]/S_{\text{un}}\) with depth.

![Figure 14. Variation of \([N/\sigma_{\text{v}}]/S_{\text{un}}\) with depth](image)

4 CONCLUSIONS

1. The cone factor \( N_4 \) back calculated from shear strength of very soft/soft clay measured by borer vane and unconfined compression apparatus depend upon the effective stress in the ground at the horizon of the test.

2. The local skin friction \( f_s \) measured with friction sleeve was related to direct measurements of shear strength of very soft/soft clay.

3. The SPT results were related to direct measurements of shear strength of very soft/soft clay.

5 REFERENCES


Site investigation and parameter evaluation for analysis of a landslide along the Göta River, Sweden

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ABSTRACT:

Slides occur quite frequently in the Swedish soft clays and a lot can be learned from comprehensive site investigations and back analyses of such slides. In this paper certain aspects of one study is reported. The shear strength determined by cone penetration tests, vane borings, fall-cone tests and direct simple shear tests are compared. The potential and great benefit of empiricism is emphasized. The stability analysis shows good agreement with the existing sliding activity.

1. BACKGROUND

Parts of Sweden consist of large areas covered with deposits of soft high-plastic clays. Streams and rivers flowing through the areas cause gradual erosion, resulting in landslide activities which in many respects constitute a major problem. The Göta River Valley, north of Göteborg, is one such area well known among geotechnical engineers for large slides like those at Surte and Göta. These were both devastating slides, causing several deaths and destroying several buildings. Such major slides only occur a few times per century, but minor slides occur much more often, in spite of all remedial work carried out.

One such minor slide occurred in a rural area in the spring of 1996, involving an area of approximately 110 x 60 m², where about 30 000 m³ of clay slid out into the river. It was considered important to investigate the slide and the lessons to be learned. After this slide at Vätternlanda, personal from SGI (Swedish Geotechnical Institute) and Chalmers University of Technology made a joint effort to document the geotechnical parameters and the geometry of the slide and analyse the cause of the slide.

The site investigations, which were quite comprehensive, included undisturbed sampling, CPT tests, field vane tests and pore pressures measurements. Numerous tests were then made in the laboratory.

See Fig. 1.1 for an overview, and Fig 1.2 for a photo of the slide.

2. SCOPE AND OBJECT OF THE PRESENT PAPER.

A proper site characterisation is very important in a project like this, but also for all stability analyses of other slopes in soft clay areas. Many different site and laboratory test methods are available and the cost for the geotechnical investigation can easily become very high. Nevertheless knowledge about the interrelation between different parameters as interpreted from different tests is often overlooked or at least not fully appreciated. Of particular interest

Figure 1.1 Overview of slide area.
is, which is the object of this paper, the correlation between the results from laboratory tests on undisturbed samples and results from in situ tests like CPT tests and vane borings. Equally important is to evaluate the value and validity of empiricism, i.e. knowledge available a priori. Especially the value of empiricism is believed to be underestimated, and not used to the extent that it should be.

3. GEOLOGY, TOPOGRAPHY AND BASIC GEOTECHNICAL PARAMETERS

The area is situated some 50 km north of Göteborg on the western banks of the Göta River. The river is there typically 100 to 150 m wide and has a depth greater than 7 m, often more than 10 m. The banks are 6-9 m high with slopes of typically 1:2. The valley is surrounded by hills and there is a gentle upward slope of the ground surfaces towards the sides of the valley. The soil profile consists of a thin dry crust of about 1 m, followed by post glacial and glacial clay down to depths often exceeding 40 m. The groundwater table is usually situated 0.5 to 1.5 m below the ground surface and the pore water pressures increase slightly more than hydrostatic pressure with depth because of artesian water pressure in the bottom layers. The clay is very homogeneous and slightly over consolidated with a clay content of anywhere between 40 and 80% and a water content varying between 60 and 80%. The shear strength is typically around 15 kPa in the upper 5-8 m and then increases with depth with about 1.5-2 kPa/m.

The slide occurred the 16th of April 1996 in a rural area and covered about 7000 m². The detailed geometry and extent of the slide is given in Fig. 4.1. No eyewitness recorded the event. It was reported by a captain on a ship passing the site at 6.40 p.m. on the 16th of April, and it is not known whether it started with a small slide or occurred as one major event.

4. SITE INVESTIGATIONS

A rather comprehensive site investigation program was planned and carried out. All together it included

Figure 4.1 Location of site investigations.
undisturbed sampling down to 20 m at two points, vane borings down to about 20 m at five points and CPT tests with pore pressure measurements at five points on land and three points in the river. Ten piezometers were also installed. The locations of the different tests are given in Fig. 4.1, where it also can be seen that some of the tests were made within the slide area and some tests were made in the river from a barge.

The pore pressure regime was analysed with the help of a number of piezometers placed mainly behind the back scarp of the slide.

The laboratory tests included routine testing and numerous oedometer tests, and also a number of direct shear tests and triaxial tests.

4.1 Quality of investigations

These soft clays are rather sensitive to disturbance during sampling and handling and also for deviation from the standard procedures for the field tests. But the field and laboratory data were critically analysed and no differences between the two field teams or laboratories could be detected, leading to the conclusion that all the data obtained were of prime quality.

4.2 Shear strength

The undrained shear strength was evaluated from field vane tests, CPT tests, fall-cone tests, direct shear tests and triaxial test. A correction of the vane and fall-cone strength was made as a function of liquid limit according to Swedish practice.

\[ \tau_{ud} = \left( \frac{0.43}{w_L} \right)^{1.60} \tau_{uL} \]  

(4.1)

where

- \( \tau_{ud} \) = undrained shear strength
- \( \tau_{uL} \) = shear strength determined by vane test
- \( \tau_{uF} \) = shear strength determined by fall-cone test
- \( w_L \) = liquid limit

The undrained shear strength was evaluated from CPT tests using the following equation

\[ \tau_{uF} = \frac{q_F - \sigma_{yy}}{13.4 + 6.65w_L} \]  

(4.2)

where

- \( q_F \) = total tip resistance
- \( \sigma_{yy} \) = over burden pressure

When sufficient reference strength measurements are available, as in this particular case, a locally calibrated evaluation of the CPT test can be applied.

The shear strength profile varied somewhat depending on local topography and pore pressure profile. In general, the strength profile corresponded to the expected result of the load history, giving somewhat lower values below the river where the clay was unloaded by the erosion process.

When comparing the strength from field vane, CPT and fall cone tests, very good agreement was obtained for depths down to 10-15 m, see Fig. 4.2. For greater depths the fall cone test gave lower values, which is in line with earlier observations and the general opinion that the fall cone is fairly reliable down to depths of 10-15 m. It should here be pointed out that the evaluation of the undrained shear strength from CPT tests is sensitive to the quality (test class) of the sounding itself. In this case, the high requirements in the Swedish test class 3 for soft clays were applied. For greater depths there was a tendency for slightly higher values for the vane tests compared to the CPT tests and a locally

![Figure 4.2 Undrained shear strength results from vane, CPT and cone tests.](image)
calibrated evaluation could be applied for these results.

The direct shear tests gave values in the same range as the vane and the CPT tests, see Fig 4.2. The results from the triaxial tests are discussed in section 4.3.

4.3 Preconsolidation pressure

In Sweden oedometer tests are routinely carried out as constant rate of strain tests (CRS) and the preconsolidation pressure is evaluated according to Fig 4.3.

Results from two bore holes taken behind the slide are given in Fig 4.4, and it is obvious that the clay behind the river banks is lightly overconsolidated by about 25%. Results from $K_C$-consolidated undrained triaxial tests confirm these results.

It should be pointed out that the plastic Swedish clays are anisotropic and show a yield and failure behaviour of the type predicted by an anisotropic critical state model, see Fig. 4.5.

Figure 4.3 Evaluation of the preconsolidation pressure from constant rate of strain tests.

Figure 4.4 Preconsolidation pressures for the area just behind the slide.

Figure 4.5 Stress-paths and yield surface for an anisotropic soft clay.

The preconsolidation pressure can thus be evaluated from a consolidated triaxial undrained test if the soil sample is consolidated to stresses high enough, so that the stress path reaches the yield surface before the failure line, illustrated in Fig. 4.5 by stress path A. In cases with overconsolidated clays, the stress path reaches the failure line first, stress path B in Fig. 4.5. The sample should then tend to dilate and fail, somewhere between points C and D. However, many sensitive Swedish clays fail directly upon arrival at point C.

4.4. Empirical correlations

Empiricism is widely used in soil mechanics but often not given the appropriate attention or status that it deserves.

The quasi preconsolidation pressure was early recognised (Leonards 1964; Bjerrum, 1967) and a systematic study (Larsson and Stulffors, 1995) has shown that the Swedish marine clays, usually assumed to be called normally consolidated, are overconsolidated by about 25%. For some clays the value is substantially higher. The results given in Fig. 4.4 are in line with this observation. In a research oriented project like this, it is natural and possible to run a large number of oedometer tests. In an ordinary consulting project another approach should be adopted. A priori it is known that most clays of the type at hand at this site is
overconsolidated by about 25% or more. The object should then be to verify that so is the case, or slightly adjust that assumption due to the results of the site investigation.

Once the preconsolidation pressures are known, the undrained shear strength can be estimated using empirical correlation for the \( \tau_d' / c' \)-ratios. The interpretation of the results from the in situ tests should also be made in the light of that knowledge. This opens up for an indirect quality control. Great effort should be given to investigate and verify deviations from the a priori assumptions. In many cases the deviations will be found to be caused by lack of quality rather than anomalies in nature.

So in conclusion, for this site, the vast number of tests made, to at great extent confirmed the advantage of this approach.

5. STABILITY ANALYSIS

No site investigations were performed before the slide in this rural area. However the investigations showed very consistent strength profiles for the area around the slide and also for the clay under the river. As was pointed out in the previous section, the strength and consolidation properties were also very much in line with the well documented empiricism. Therefore the strength in the soil involved in the slide can also be considered to be well known. The pore pressure, often an equally important parameter, was measured after the slide and can have been affected by the slide activity. Artesian pressure was noted during the early stage of the site investigations, but was less significant during the later measurements. Therefore calculations had to allow for certain variations in the pore pressure regime. Some uncertainty also exists regarding the geometry before the slide. Recent topographical maps and interpolation from existing geometry of

![Figure 5.1 Results from the stability analysis.](image-url)
the landscape determined through levelling has formed the base for the section used for the backanalysis.

In Fig. 5.1 the geometry, strength profile and pore water pressure used in the calculations are given. Factors of safety around 1.0 were obtained. The different assumptions used rendered factors of safety in the range 0.98-1.1.

According to Swedish recommendations and current practice a combined analysis was used, where the lowest value of the drained and undrained strength was used for each slice, see Fig. 5.2. Undrained or drained analysis result in moderately higher factors of safety.

![Diagram showing strength properties](image)

**Figure 5.2** Strength properties used in a combined analysis.

The critical shear surface obtained for the combined analysis is given in Fig. 5.1, and it is in fairly good agreement with the observations made at the site in situ.

6. CONCLUSIONS

The undrained shear strength of the clay determined by CPT, vane, fall-cone and direct shear tests gave similar strength results and corresponded well with empirical relations between the undrained strength and the preconsolidation pressure. Also the preconsolidation pressure as determined in the laboratory rendered results that agreed well with empirical experience in the region, showing that the clay was overconsolidated by about 25%.

Stability analysis of the slope showed good correlation with the slide geometry, and the combined analysis, where the drained and undrained parameters of the clay are accounted for in the same calculation, gave the most reasonable results. It was also concluded that the value of a priori knowledge of strength and deformation properties is of great value when planning the site investigation as well as when interpreting the results.

7. REFERENCES


Performance of in situ testing methods in predicting deep foundation capacity

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Schnabel Engineering Associates, Gainesville, Ga., USA

ABSTRACT: In situ testing methods are frequently used to predict deep foundation capacities. Some in-situ testing methods include Standard Penetration Testing (SPT), Cone Penetration Testing (CPT), Flat Dilatometer (DMT), and Pressuremeter (PMT). This paper demonstrates the performance of these in-situ testing methods in predicting deep foundation capacities. Three case studies from different geological settings are presented that illustrate how in-situ testing methods can be used to evaluate soil properties used in deep foundation design. The predicted behavior is then compared with the results of instrumented pile load test programs for each study.

1 INTRODUCTION
The purpose of this study is to evaluate the predictive capabilities of conventional geotechnical design methods in estimating deep foundation capacity based on in-situ test data. The predicted pile capacities were compared with the observed behavior from the load test programs. Three case studies are presented using in-situ test methods.

2 SAVANNAH, GEORGIA

2.1 Site Characterization
The first case study involves a site on Hutchinson Island in Savannah, Georgia. The site geology is characterized by recent alluvial deposits overlying ancient coastal plain soils. The proposed construction consists of a large convention center and hotel. The structures are designed for deep foundation support. SPT’s, CPT’s and DMT’s were used to characterize the site.

2.2 Pile Load Test
Two pile load tests were performed on 16-inch diameter auger cast-in-place piles. Each pile was instrumented with vibrating wire strain gauges that yielded information about load transfer into the surrounding soils.

Figure 1. Soil Test Boring Data for the Savannah, Georgia site.

Figure 2. Cone Penetration Test Data for the Savannah, Georgia site.
2.3 Comparison of Results

The predicted pile capacities and the measured loads are summarized in the table below.

Table 1. Ultimate Load Capacities for Savannah Site

<table>
<thead>
<tr>
<th>Method</th>
<th>Qs (kN)</th>
<th>Qr (kN)</th>
<th>Qt (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPT (Meyerhoff 1976)</td>
<td>419</td>
<td>454</td>
<td>873</td>
</tr>
<tr>
<td>CPT</td>
<td>1121</td>
<td>932</td>
<td>*</td>
</tr>
<tr>
<td>(R=qA1+qF3A3D)</td>
<td></td>
<td></td>
<td>2053</td>
</tr>
<tr>
<td>DMT</td>
<td>2795</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(f=(P+Pm)sin tanθ)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pile Load Test</td>
<td>1780</td>
<td>579</td>
<td>2359</td>
</tr>
<tr>
<td>(Davisson Method)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>*Tip resistance in sand zone</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Qs – Skin friction</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Qr – Tip resistance</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Qt – Total resistance</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3 SHANGHAI, CHINA SITE

3.1 Site Characterization

The second case study involves a highrise office and residential complex in downtown Shanghai, China. The geologic setting is very deep soft clays associated with the Yangtze River Delta. The complex consists of a 50-story office tower, two 20- to 30-story residential buildings, and a 9-story retail podium. Over a thousand concrete bored piles were used to support the complex. Bored piles of 850 mm in diameter with 76 meters in length and of 700 mm in diameter with 56 meters in length were used to support the main tower and other structures, respectively. PMT's, along with other in-situ and laboratory testing were performed to characterize the subsurface conditions. Figure 4 shows a typical subsurface profile for the site.

PMT data obtained near test pile 4 is presented in Figure 5.

3.2 Pile Load Tests

Eleven pile load tests were performed to verify the design capacities of the bored piles and to evaluate the contractor's installation procedures. Six of the test piles were instrumented with strain gages to measure skin friction along the piles.

3.3 Comparison of Results

The PMT data was used to predict ultimate vertical load capacity (Birnid, 1986). Table 2 summarizes
the predicted and measured pile capacities. Test pile 4 was selected because of the proximity of the test pile to the PMT location. Four common methods of determining ultimate load from a pile load test were used (Fell, 1980). The result presented in Table 2 represents an average of the results based on the four methods. Other pile load tests indicated similar results and are not presented here due to the limitation on the length of the paper.

Table 2. Ultimate Load Capacities for Shanghai Site

<table>
<thead>
<tr>
<th>Method</th>
<th>Qv (kN)</th>
<th>Qf (kN)</th>
<th>Qt (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PMT (Briaud, 1986)</td>
<td>9893</td>
<td>1545</td>
<td>11438</td>
</tr>
<tr>
<td>Pile Load Test</td>
<td>9701</td>
<td>1352</td>
<td>11053</td>
</tr>
</tbody>
</table>

4.2 Pile Load Tests

Ten pile load tests were performed, including 5 axial compression tests, 2 axial tension tests, and 3 lateral tests. Each test pile was instrumented to allow estimation of load distribution with tell-tale. The results presented in Table 3 are based on in-situ soil testing and one pile load test performed in a specific area of the site.

4.3 Comparison of Results

SPT, CPT, and DMT data were used to predict vertical pile load capacity. Because DMT

4 ATLANTA, GEORGIA

4.1 Site Characterization

The third case study is the Atlanta Olympic Stadium. The stadium site is underlain by man-made fill, residual soil, partially weathered rock and rock. The subsurface profile is generally erratic with the depth to bedrock varying from 13 to 28 meters. The 80,000-seat stadium is supported on augered cast-in-place piles up to 25 meters deep. SPT's, CPT's, and DMT testing, along with other field and laboratory testing were performed. SPT, CPT, and DMT data obtained at the site were used to estimate pile capacities.
Figure 8. DMT Data for the Olympic Stadium Site.

sounding were not able to penetrate to the pile bearing strata, this data was used only to calculate skin friction.

5 CONCLUSIONS

This paper presented three case studies from different geologic settings. It demonstrated how in-situ tests can be used to predict deep foundation capacities. The case studies indicate that in-situ test data used with established prediction methods is reasonably accurate in predicting pile load capacity. However, some tests are better than the others for certain soil type.

In one case history (Shanghai site), CPT data predicted pile load capacity very well. CPT data also predicted pile load capacity fairly well in the other two case studies. The correlations between tip resistance and DMT data are not available. To predict skin friction, friction angles obtained from DMT data are typically used; however, friction angles from DMT depend greatly on the thrust measured during DMT soundings. Experience indicates that the thrust measurements may not always be reliable. In the two case studies, DMT data over predicted skin friction.

It is also important to use engineering judgment to develop soil profile and associated parameters to be used in the prediction method.

<table>
<thead>
<tr>
<th>Method</th>
<th>Q_s (bars)</th>
<th>Q_p (bars)</th>
<th>Q_f (bars)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPT (Meyerhoff 1976)</td>
<td>1580</td>
<td>685</td>
<td>2265</td>
</tr>
<tr>
<td>CPT</td>
<td>3007</td>
<td>472</td>
<td>3479</td>
</tr>
<tr>
<td>(R=σ₀A₀+σ₂A₂D)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DMT</td>
<td>3016</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(ρ_c(ρ₂-ρ)=m tanθ)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pile Load Test</td>
<td>2206</td>
<td>552</td>
<td>2758</td>
</tr>
<tr>
<td>(Davisson Method)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

6 REFERENCES


Comparison of strength and stiffness parameters for a Piedmont residual soil

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ABSTRACT: A geotechnical research site in the Piedmont residual soils of the southeastern United States has been established as a part of Auburn University's foundation research activity. The test site near Spring Villa, Alabama has been the subject of a wide array of laboratory and in-situ measurements, and provides a source of comparison of geotechnical parameters in a residual soil derived from the metamorphic rocks of the Piedmont Plateau. A relatively small area of the site has been the subject of a fairly intensive geotechnical site investigation, and the results of this investigation provide an opportunity for cross comparisons of different in-situ techniques in this micaceous silt residual soil. Comparative geotechnical data are provided from standard penetration tests (SPT), cone penetration soundings (CPT), Memad and push-in pressuremeter (PMT), dilatometer (DMT), seismic wave propagation measurements from seismic cone, seismic dilatometer, and crosshole tests, and laboratory triaxial testing on undisturbed samples. The results of this testing program are compared in terms of soil moduli and strength parameters.

1 INTRODUCTION

A geotechnical research site has been established near Spring Villa, Alabama, in a residual soil of the Piedmont Plateau. The Piedmont covers a wide area of the southeastern United States between the Atlantic Coastal Plains and the Blue Ridge Mountains extending from Alabama to Pennsylvania. These soils are derived from metamorphic rocks, predominantly gneissies and schists of early Paleozoic Age or older (Sowers, 1985), and are composed of micaceous sandy silts. Commonly referred to as "saprolite", these residual soils retain the foliation and structural features of the parent rock, but have the texture and appearance of soil.

The purpose of this paper is to provide a summary of geotechnical data for ongoing and future foundation engineering research and to provide a comparison of strength and stiffness measurements in Piedmont saprolite using different in-situ and laboratory measurements.

2 GENERAL CHARACTER OF SITE

The geotechnical investigation has been conducted within a relatively small area of a very large site. This area was selected after a program of cone penetration soundings (CPT) because the area was fairly level and appeared to be relatively uniform within the upper 13m.

The Site Plan shown on Figure 1 provides locations of the borings and soundings within the area of discussion. Groundwater was present at a depth of 3m. The soil is a micaceous sandy or clayey silt, typically classified as ML-SM, with seams of sand which are remnants of igneous quartz seams into the parent metamorphic rocks. The results of over 40 grain size analyses, water content and classification tests in the upper 13m are summarized as follows:

<table>
<thead>
<tr>
<th>Test</th>
<th>No. Tests</th>
<th>Avg</th>
<th>Std Dev</th>
</tr>
</thead>
<tbody>
<tr>
<td>water content</td>
<td>64</td>
<td>34%</td>
<td>7.5%</td>
</tr>
<tr>
<td>% sand</td>
<td>48</td>
<td>47%</td>
<td>17%</td>
</tr>
<tr>
<td>% silt</td>
<td>22</td>
<td>33%</td>
<td>8%</td>
</tr>
<tr>
<td>% clay</td>
<td>22</td>
<td>10%</td>
<td>6%</td>
</tr>
<tr>
<td>LL'</td>
<td>22</td>
<td>46</td>
<td>10</td>
</tr>
<tr>
<td>PI</td>
<td>22</td>
<td>8</td>
<td>6</td>
</tr>
<tr>
<td>unit wt. (kN/m³)</td>
<td>35</td>
<td>18.2</td>
<td>0.5</td>
</tr>
</tbody>
</table>

* Liquid Limit (LL) and Plasticity Index (PI) data do not include 20 tests which were reported as "very plastic."
Standard penetration tests were made in four
boreholes using a CME automatic hammer and these
indicated SPT-N values which typically increase from
approximately 8 to 14 blows/30cm over the depth
range of 2m to 15m (std deviation ± 2b/30cm). The
mean SPT-N value was 12 blows/30cm (std
deviation ± 3b/30cm).

3 TESTING PROGRAM

The site plan of Figure 1 indicates the variety of in-situ
tests performed. Triaxial tests were performed on
undisturbed samples from the bores. An overview
of the results of this testing program follows.

3.1 Laboratory Triaxial Tests

Good quality undisturbed samples of these silty soils
were relatively easy to obtain using thin walled tubes,
and trimming of the samples for testing was easily
performed. Samples could be back saturated,
consolidated, and sheared relatively quickly due to the
silty nature of the soil and high hydraulic conductivity
relative to most cohesive soils. However, the spatial
variability of the material is significant and leads to a
substantial amount of scatter in the test results, even
though this site is a relatively homogeneous site within
the broad context of Piedmont residual soils.

Presented in Figure 2 are the results of 23
effective stress strength tests (mostly from
isotropically consolidated undrained triaxial
compression tests, CIUC) at maximum principal stress
ratio (q/σ3) where q = (σ1 - σ3)/2 and p = (σ1 + σ2)/2.
Considerable scatter in these data is evident, and
suggests that interpretation of Mohr-Coulomb
strength

parameters from only a relatively few tests could be
very misleading for Piedmont residual soils.

Such variability has been noted for these soils
(Lambe and Hasted, 1988) and multistage tests
proposed as a potentially attractive option to single
tests on different samples. Presented on Figure 3 are
data from four 2-stage CIUC tests in which a second
consolidation and shearing stage is performed at an
effective confining stress (σ3) approximately equal to
twice the σ3 of the first test stage on the sample.
Mohr-Coulomb strength parameters interpreted from
any single such staged test would appear to provide a
reasonable estimate of the overall average strength
parameters given the inherent variability evident in
these materials. These multistage tests may tend to
overestimate ϕ slightly; perhaps the stage one test
produces increased densification during the stage two
consolidation.

Unconsolidated undrained (UU) triaxial
compression tests were performed primarily within the

Figure 1 Site Plan

Figure 2 Triaxial Data
3.3 Cone Penetration Testing (CPT)

Presented on Figure 5 are the logs from six cone soundings in the site area shown on Figure 1, along with average plots of cone tip resistance ($q_t$), sleeve friction ($q_s$), and friction ratio. Using conventional interpretations of cone test data, these average values shown would tend to suggest that this was a clay soil of low plasticity. Although cone tip resistance values were not particularly high relative to the capacity of the rig to pull, cone soundings were observed to tend to drift from vertical somewhat during penetration. This trend is likely due to settlement in the residual soil layers which are remnants of the parent rock.

Based on the average $S_u$ values from laboratory tests, an $Q_u$ value for these data (where $S_u = N_q Q_u$) is backcalculated to be in the range of 35 to 40 rather than the 15 to 20 range typically used for cohesive soils.

Results of several correlations of $Q_u$ with $q_u$ are provided on Figure 6. These correlations have generally been developed for sands and are seen to somewhat overestimate $Q_u$ as determined by CIUC triaxial tests.

3.3 Shear Wave Velocity Measurements

Crosshole measurements of shear wave velocity (CHT) were made using interval measurement between two downhole geophones and a downhole hammer to induce an SV wave. Each set of three crosshole access holes were made using the exploration boreholes with PVC casing grouted into place using a cement-bentonite mixture. Inclinometer profiling of these holes was used to establish distance between holes. Seismic cone (SCPT) measurements were made using a horizontal hit upon the cone truck lift pad to induce a downward travelling SH wave. The seismic dilatometer (SDMT) measurements were made similarly.

Data from different types of wave velocity profiling methods are shown on Figure 7 in summary form. Although pseudo interval measurements can be interpreted from the downhole (SCPT and SDMT) measurements, the data suggest a velocity vs depth profile which is very near constant with depth over the upper 15m of the site. The crosshole measurements of velocity appear slightly higher, but are generally within a quite similar range of around 200 m/s.

---

Figure 3 Multistage Triaxial Data

Figure 4 Undrained Shear Strength from Lab Tests

Upper 3m for determination of undrained shear strength, $S_u$. Presented on Figure 4 are the $S_u$ values plotted as a function of depth, including $S_u$ from CIUC tests consolidated to the effective overburden pressure ($q_se$). Considerable scatter in these data are present, particularly within the upper 3m which is above the present groundwater level. Much of this scatter is likely the result of variability in clay content within the zone where negative pore water pressures would have an effect.

Initial tangent moduli ($E_t$) were estimated from the triaxial test results at axial strain values typically around 0.5%. Considerable scatter were evident in these results, but average values were $E_t = 200S_u$ for the UU tests in the upper 3m and $E_t = 200S_u$ for the CIUC tests in the 3m to 15m depth range.
2.4 Pressuremeter (PMT) and Dilatometer (DMT) Tests

Menard-type pressuremeter tests (PMT) were performed at various depths using a TEXAM pressuremeter and performing the hole by pushing in a thin walled tube sampler. This procedure appeared to work very well, with a good quality preformed hole resulting. Push-in pressuremeter tests (CPMT) were performed using a 10 cm² cone with a pressuremeter built into the cone rod above the tip. Two long days of testing completed 16 PMT's; a total of 29 CPMT's were completed in 6 hours of work one morning. Pressuremeter moduli were determined from each test and are illustrated on Figure 8. As might be expected, the CPMT moduli were slightly higher than the prebored PMT moduli, but on average these values were fairly close. An undrained load cycle was
Figure 8 Pressuremeter Moduli

Figure 9 Dilatometer Moduli

performed on each CPMT and the moduli from these cycles were around an order of magnitude larger than initial CPMT modulus values (data shown later in this paper). Limit pressure values are less well defined from the data at this site.

Dilatometer tests (DMT) were performed at three locations to depths of up to 8m, with the dilatometer pushed in using conventional soil drilling equipment (rather than a cone truck). Elastic modulus values from the DMT results are illustrated on Figure 9.

Because of the relatively fast consolidation times in the triaxial samples, the authors would suggest that these PMT and DMT tests are likely to represent more of a drained test condition with little effect due to transient pore pressures.

4 COMPARISON OF ELASTIC MODULI

The data presented on Figure 10 provide a comparison of shear modulus (G) vs depth derived from all of the tests described above. Poisson’s ratio of 0.3 has been used in all conversions of other elastic constants to G. These data clearly illustrate the effect of strain level on measured values of G. All of the measurements relating to shear wave propagation correspond to very small strain amplitudes and reflect values closer to G<sub>max</sub>. While physical tests such as PMT, DMT, and triaxial tests are performed at larger strains and characteristically exhibit much lower measured G values, it is of interest to note, however, that G derived from the unload/reload portion of the CPMT curve yields a measured shear modulus which is fairly close to G<sub>max</sub> as indicated by wave velocity measurements.

Values of G are plotted on Figure 10 on a semilog scale to better illustrate differences between G from the initial loading PMT and CPMT as well as the DMT and triaxial data. The G values from both types of pressuremeters are seen to be somewhat lower than those of the DMT or initial tangent moduli of the CIUC triaxial tests, but do not appear to indicate a great deal of difference relating to the method of installation between pressuremeters. The substantially higher moduli from DMT and CIUC triaxial tests (on the order of 2 to 5 times that of either type of pressuremeter) may be related to strain levels, relatively smaller installation effects, or some combination of these factors and others. In any event, the large range of soil modulus values which may be obtained using different test methods suggest that there remains a compelling need for engineering judgment, experience, and empirical correlations in the use of these parameters for foundation engineering.

Also shown on that figure are some simple correlations of G with q<sub>C</sub> from the CPT soundings (Senneset, 1988) and of G with SPT-N values (Kalhatovy and Mycne, 1990). These two (of many) correlations were intended for silty soils and appear to suggest a modulus which is slightly larger than that derived from PMT tests but lower than that of the DMT or CIUC triaxial.
5 CONCLUSIONS
Strength and stiffness data from a wide variety of in-situ and laboratory tests are compared for a site composed of Piedmont residual soil. Although the site is a relatively homogeneous deposit relative to most areas of the Piedmont, this soil is characterized by a large degree of spatial variability. This spatial variability is a concern for geotechnical engineers who need to determine representative strength and stiffness parameters across a large area for engineering design purposes and thus need to rely upon correlations with simple and rapid tests which can be used across a large area (such as cone soundings and SPT results).

CPT data are characterized by relatively high friction ratios in these soils. Backcalculated $N$ values are substantially higher than those normally used for cohesive soils. Several correlations of strength parameters with routine CPT tests are shown to somewhat overestimate soil friction for these silty soils, not particularly surprising in light of the fact that most such correlations are developed for granular soils rather than silty soils.

Elastic moduli determined using a variety of in-situ test methods indicate a wide range in values, much of which may be related to strain levels associated with the particular type of test. Highest values are naturally determined using shear wave propagation measurements, and values determined using cone-hole vs downhole techniques agree closely. Unload/reload moduli from push-in cone pressuremeter results appear to agree fairly closely with seismic wave propagation measurements. Dilatometer modulus values were significantly higher than either type of pressuremeter, and agreed fairly closely with initial tangent moduli from CIUC triaxial test specimens. Two correlations of modulus with SPT and CPT data for silty soils are seen to estimate a modulus somewhat higher than pressuremeter values but lower than dilatometer or CIUC triaxial measurements.

ACKNOWLEDGEMENTS
The authors are grateful to the Morris-Shen Bridge Company and Williams and Associates for assistance with CPT soundings, to Alabama DOT for soil borings, classification tests and financial assistance, to Dr. Paul Mayne and Dr. Glenn Rix of Gia. Tech for assistance with shear wave measurements and DMT, to Dr. Frank Townsend of the University of Florida for assistance with CPMT, to Law Engineering for assistance with PMT, and to the Alabama University Highway Research Center for financial assistance and for support of the development of the site as a long term Geotechnical Research facility.

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Uncertainty in undrained shear strength assessment using cone penetration test

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ABSTRACT: In-situ tests for soils have become more relevant for site characterization in recent years. However, difficulties that exist in the testing process is the uncertainty in the interpretation of test results. A database consisting of results from a variety of clays from sites distributed globally has been assembled for the purpose of examining the uncertainties and variabilities associated with prediction of undrained shear strength using the cone penetration test. Results of the cases analysed show that guidance on (i) variability in natural soil variation and (ii) uncertainty in test conversion may be obtained if uncertainty levels are categorised according to the soil description. The reliability of the conversion model for undrained shear strength is influenced significantly by the type of penetrator and careful attention is required for the calibration process. Statistical data for use in probabilistic approaches for assessment of uncertainty involved in such determination are developed.

1 INTRODUCTION

A greater need for determining soil properties in-situ has led to further developments of in-situ test methods. This may be attributed to the increased appreciation of the fact that soil characteristics can be better assessed in the natural environment where the difficulties arising from sampling processes required for conventional laboratory testing can be avoided.

However, the capabilities of in-situ tests continue to be questioned by some because of the complex combination of uncertainties in the natural inherent variability of soils, the interpretation of test measurements, and the effects of equipment and testing procedures. In practice, for the interpretation of soil properties, results from the widely used in-situ soil tests are commonly interpreted using conversion factors determined from some alternative means of measurement. For clay soils, for example, undrained shear strength is one of the more commonly deduced properties, and, for the cone penetration test, a cone factor, \( N_s \), is applied for evaluation of undrained shear strength. Unfortunately, large variations may be observed in the cone factor.

In view of these facts, it may be valuable to make use of a simple probabilistic approach for assessment of uncertainties. The probabilistic treatment of uncertainty involved in the process of soil property determination is first reviewed and then assessments are made of two major uncertainty components, the natural inherent soil variability and the correlation model error, associated with the use of various forms of cone tests for undrained shear strength measurement.

2 TREATMENT OF VARIOUS COMPONENTS OF VARIABILITY AND UNCERTAINTY

Several researchers have addressed fundamental levels of uncertainty involved in soil exploration and foundation design problems previously (e.g. Lamb 1971, Vannsdal 1977, Orchart et al. 1986, Kay et al. 1991, Kay 1995). The forms of uncertainty and variability associated with soil property determination can be classified into three groups. The first group is the natural inherent variability of soils; the second is the measurement variability and uncertainty that exists in the testing programme; the last is the uncertainty associated with the conversion of test results to design parameters. In the larger
context of a complete foundation design, there are additional uncertainties that are related to possible variations in loads, inaccuracies in design models and inevitable variations in the construction process but these are not considered here. A quantitative approach to account for the various uncertainty components involved in the process of soil property determination is outlined in the following. A more detailed description of the approach has been given by Pang (1997).

A general mathematical formulation to represent the effects of the three major uncertainty components is as follows:

\[ \xi = F(\xi, s, b, e) \]  

where \( \xi \) is the design soil parameter, \( F \) is a function that represents the conversion model, \( \xi \) is the true mean soil property, \( s \) is the scatter component that includes the natural inherent soil variability and random test effects, \( b \) is the bias component representing the equipment, procedure and/or operator effects, and \( e \) is the model error involved in converting test results to the design parameter.

In geotechnical engineering, for the usual deterministic approach, best estimates of soil parameters are inferred using reference tests performed with specific equipment under specified conditions such as the consolidated isotropically undrained compression (CIUC) test. Frequently, the conversion of a test measurement to a design parameter may simply require a linear model with a constant multiplier \( \psi \) described by the following equation:

\[ \xi = \psi \xi_0 \]  

where \( \xi_0 \) is the measured soil property and \( \psi \) is simply the transformation model, \( P(\xi_0) \), in Equation (1). However, it is not sufficient to determine an average influence on \( \xi \) without considering the uncertainty level. The coefficient of variation, \( V \), a dimensionless statistical parameter defined as the ratio of the standard deviation to the mean, is a simple and consistent measure of uncertainty for assisting along these lines.

With a knowledge of the variability or uncertainty levels in \( s, b \) and \( e \), the uncertainty in the mean estimate of the design parameter, \( P(\xi) \), may be derived through the Taylor series approximation according to:

\[ P^2(\xi) = \frac{P^2(\xi)}{n} + P(\xi) + P(\psi) \]  

where \( n \) is the number of independent tests. It should be noted that the influence of scatter may be reduced by employing a larger number of tests whereas only proper equipment maintenance and calibration will be useful for reducing the bias effects.

To extend the foregoing formulation to account for the complete spatial effects of the natural soil variability, Kay et al. (1991) proposed a block approach in which two levels of variability are considered: (a) the variability of unit of soil at the test specimen level within the large volume of soil that contributes to a single foundation unit and (b) the variability of the individual foundation response in relation to the site as a whole.

The foregoing considerations may be applied specifically to the test result for example, in the case of the cone tip resistance, \( q_c \).

3 EVALUATION OF UNCERTAINTY OF UNDRAINED SHEAR STRENGTH

3.1 Methods of Interpretation

The relationship for prediction of the undrained shear strength of clay, \( c_u \), using \( q_c \) is commonly expressed in the following form:

\[ c_u = \frac{q_c - \sigma_{ov}}{N_c} \]  

where \( \sigma_{ov} \) is the total overburden stress and \( N_c \) is the cone factor. For piezocene tests, the availability of the magnitude of porewater pressure enables a correction of the cone tip resistance to be made for porewater effects acting at the notched section above the cone during the penetration process and a more reliable result is obtained when the corrected cone tip resistance, \( q_{ct} \), is used in place of the measured \( q_c \).

Alternative theoretical evaluations of \( c_u \) using \( q_{ct} \) for interpretation of \( N_c \) have been proposed based on one of the following: (a) the classical plasticity approach to bearing capacity of deep foundations, (b) the theory of the expansion of cavities in an elastoplastic medium, (c) programs based on the strain path method each of which considers the continuous deep penetration process to be a steady state problem or (d) the finite element approach. Sanglier (1972) and Konrad & Low (1987) provided reviews of methods mainly based on the
first two groups and the more recent proposals have been evaluated by Kay & Mynur (1985). However, none of the methods are, as yet, of particular practical value owing to difficulty in determining the required parameters. In view of this difficulty, back-calculated Ns values are examined.

In the light of the facts that field vane results appear to be highly repeatable and its operation is very simple, it is frequently used for the determination of the in-situ s<sub>v</sub> as a reference strength for correlations. Alternatively, it is not uncommon that the cone tip resistance is compared with some other standard measurements such as s<sub>v</sub> obtained from laboratory triaxial compression tests. In fact, several reference tests have been adopted to infer the undrained strength parameter. To address this matter, Wroth (1984) suggested that one standard reference test type such as the CUC triaxial test should be used.

In the light of the popularity of the use of the cone penetration test for evaluating soil properties such as s<sub>v</sub> through an empirical correlation, it was considered useful to examine the relationships for a variety of reference tests. By adopting Equation (4) as the conversion model and the conversion factor S as the inverse of Ns, distributions for Ns were found using conventional statistical methods.

3.2 Purposes of the Study and Formation of a Database

A database consisting of a variety of clays from sites distributed globally has been assembled. The database shown in Table 1 includes data collected from 24 clay sites. Several criteria were laid down in the selection of reference sources in relation to the equipment geometry and testing procedures. To avoid ambiguities in statistical analyses, only cases using 60° cones of 35.7 mm diameter were included. Tests were generally conducted at a standard rate of penetration, i.e., 20 mm/s. In a few instances results from non-standard tests were included (see Table 1). In three cases test rates of 10 mm/s were used; in one case, computations made by the original authors made use of the effective stress instead of total stress for Equation (4). It was considered that these variations were not sufficient to significantly influence the results.

The objective of the establishment of the database was to examine the Ns values and their variations for all of the sites, to categorise them in terms of the type of cone penetration test and reference test and to observe any consistent trends that may exist. This part represents examination of the conversion error. In some instances, the determination of the natural inherent variability of the clay soils as indicated by the variability in test results measured by the various types of cone penetrometers and reference tests was also facilitated. It should be noted, however, that random test effects associated with equipment and testing errors cannot be separated from the natural inherent soil variability results. Consequently, they were considered together.

Attention is first given to the results of the field in-situ natural soil variability and equipment and testing random error component prior to discussing those of the conversion uncertainty.

4 RESULTS AND DISCUSSION

4.1 Natural Soil Variability and Equipment and Test Scatter

Although the results of variability in test measurements obtained from various cone tests and reference tests summarised in Table 1 are different, they do exhibit consistencies in variability levels for most of the cases in relation to soil type. These can be observed when the corresponding pairs of variability values for the normalised cone tip resistance and the reference strength from the table are shown in Figure 1. In this figure, V(s)<sub>v</sub> and V(s)<sub>r</sub> are the coefficients of variation of the normalised cone tip resistance and the reference strength, respectively. An equal variability line is shown for comparison purposes.

As can be seen, the results generally indicate a consistent trend for levels of variability in cone test measurements compared with the variability in the reference tests. Differences in levels of inherent soil variability are indicated for different sites. However, on the basis of comparisons of the variability results with the line showing equal variability values, the trend indicates that the UU test (and the UC test) produces results that are generally more variable than others while the variability exhibited by the mechanical cone is even greater than that of the UU test. The data for CPT (uncorrected for porewater pressure) show some inconsistency. However, it is clear from Figure 1 that, in addition to test type, the geological origin of the soil has a considerable influence on inherent soil variability.
Table 1. Database for cone penetration test and summary of statistical analyses for inferring natural inherent soil variability and equipment and test random error component and for inferring conversion error

<table>
<thead>
<tr>
<th>Location</th>
<th>Site</th>
<th>Soil Type</th>
<th>Core Typea</th>
<th>n</th>
<th>x (cm, cm, cm) or (g, cm)</th>
<th>w (%)</th>
<th>Ns</th>
<th>dPv</th>
<th>d0.15</th>
<th>d0.60</th>
<th>d1.0</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Canada</td>
<td>McDonald Farm</td>
<td>Silty clay with sand</td>
<td>CPTU</td>
<td>42</td>
<td>2.1 0.15</td>
<td>0.7 0.19</td>
<td>10</td>
<td>0.17</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
<tr>
<td></td>
<td>Lower 232nd St</td>
<td>Silty clay</td>
<td>CPTU</td>
<td>61</td>
<td>2.6 0.33</td>
<td>0.3 0.30</td>
<td>9</td>
<td>0.20</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
<tr>
<td>Canada</td>
<td>Vancouver</td>
<td>Stiff clay</td>
<td>CPTU</td>
<td>34</td>
<td>1.5 0.22</td>
<td>0.4 0.21</td>
<td>12</td>
<td>0.16</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
<tr>
<td></td>
<td>Ottawa STP</td>
<td>Stiff clay of moderate to</td>
<td>CPTU</td>
<td>25</td>
<td>13.1 0.18</td>
<td>1.3 0.14</td>
<td>10</td>
<td>0.14</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
<tr>
<td></td>
<td>St. Albion</td>
<td>Very sensitive soft clay</td>
<td>CPTU</td>
<td>22</td>
<td>2.8 0.25</td>
<td>0.2 0.26</td>
<td>18</td>
<td>0.15</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
<tr>
<td>Denmark</td>
<td>Nivelle</td>
<td>Glacial clay</td>
<td>CPT</td>
<td>35</td>
<td>10.7 0.24</td>
<td>1.1 0.32</td>
<td>8</td>
<td>0.25</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
<tr>
<td></td>
<td>Aalborg</td>
<td>Soil silty clay</td>
<td>CPT</td>
<td>40</td>
<td>18.1 0.35</td>
<td>2.6 0.22</td>
<td>7</td>
<td>0.40</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
<tr>
<td>Ireland</td>
<td>Belfast</td>
<td>Soil organo-clay</td>
<td>MCPT</td>
<td>20</td>
<td>14 0.23</td>
<td></td>
<td>20</td>
<td>0.23</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
<tr>
<td></td>
<td>Derry</td>
<td>Soil organo-clay</td>
<td>MCPT</td>
<td>19</td>
<td>19 0.25</td>
<td></td>
<td>19</td>
<td>0.25</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
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<td>Modena</td>
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<td>0.5 0.26</td>
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<td>VST</td>
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<td>Aceh</td>
<td>Silty marine clay</td>
<td>CPTU</td>
<td>23</td>
<td>14 0.16</td>
<td></td>
<td>23</td>
<td>0.16</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
<tr>
<td>Norway</td>
<td>Dramvik gate</td>
<td>Lean clay</td>
<td>CPT</td>
<td>24</td>
<td>15 0.09</td>
<td></td>
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<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
<tr>
<td></td>
<td>Osmoy</td>
<td>Marine clay</td>
<td>CPT</td>
<td>24</td>
<td>16 0.11</td>
<td></td>
<td>24</td>
<td>0.11</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
<tr>
<td>Sweden</td>
<td>Glimming</td>
<td>Blackened silt</td>
<td>CPT</td>
<td>30</td>
<td>4.0 0.13</td>
<td></td>
<td>30</td>
<td>0.13</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
<tr>
<td>Singapore</td>
<td>Upper marine clay</td>
<td>Soil upper and lower marine clay</td>
<td>CPTU</td>
<td>24</td>
<td>15 0.13</td>
<td></td>
<td>24</td>
<td>0.13</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
<tr>
<td></td>
<td>Lower marine clay</td>
<td></td>
<td>CPT</td>
<td>34</td>
<td>11 0.11</td>
<td></td>
<td>34</td>
<td>0.11</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>MCPT</td>
<td>19</td>
<td>13 0.25</td>
<td></td>
<td>19</td>
<td>0.25</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>MCPT</td>
<td>20</td>
<td>16 0.31</td>
<td></td>
<td>20</td>
<td>0.31</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>CPTU</td>
<td>30</td>
<td>29 0.11</td>
<td></td>
<td>30</td>
<td>0.11</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
<tr>
<td>UK</td>
<td>Eastling</td>
<td>Upper glacial till</td>
<td>CPT</td>
<td>58</td>
<td>20.5 0.35</td>
<td>1.1 0.46</td>
<td>58</td>
<td>0.38</td>
<td>VST</td>
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<td>Lee et al. (1982)</td>
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<td></td>
<td>Rochdale</td>
<td>Clayey glacial till</td>
<td>CPT</td>
<td>23</td>
<td>22.3 0.35</td>
<td>1.3 0.29</td>
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<td>VST</td>
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<td>Lee et al. (1982)</td>
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<td>Warsop</td>
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<td>CPT</td>
<td>23</td>
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<td>1.4 0.49</td>
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<td>Lee et al. (1982)</td>
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<td></td>
<td>Hodnet</td>
<td>Clayey glacial till</td>
<td>CPT</td>
<td>21</td>
<td>26.6 0.29</td>
<td>1.2 0.38</td>
<td>21</td>
<td>0.38</td>
<td>VST</td>
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<tr>
<td></td>
<td>Upper 3rd</td>
<td>Clayey glacial till</td>
<td>CPT</td>
<td>10</td>
<td>3.9 0.24</td>
<td>0.4 0.18</td>
<td>10</td>
<td>0.18</td>
<td>VST</td>
<td></td>
<td></td>
<td>Lee et al. (1982)</td>
</tr>
</tbody>
</table>

* CPT, MCPT and CPTU mean electric and mechanical cone penetration tests, and penetration cone penetration test respectively.
** n means number of data points for variability analysis.
*** CVST and VST mean field cone shear tests with and without corrections, and UC mean undrained unconfined compression tests, and CAUC mean consolidated undrained triaxial compression test.
**** Tests were conducted at 10 cm as rate of penetration.
***** Case tip resistances were corrected for effective overburden pressures.

4.2 Conversion Error

Figure 2 shows the results for all three types of cones for a variety of clays in a graph of the coefficient of variation of Ns versus the derived average value for clays. In all of these cases a separate set of tests was done adjacent to the penetration test. Visual inspection of this figure shows that definite patterns of the data can be observed in relation to the cone type and the soil type ("ordinary clays" in this context refers to clays that are predominantly of clay-size particles). The results can be generally divided into two groups in relation to the level of uncertainty of Ns. The first group includes results obtained from cases associated with glacial materials and clays using mechanical cones, while results for ordinary clays with electric cones and piezocores from the second. Tables 2 and 3 present summaries of statistical results for the respective groups.
It may be noted that the level of uncertainty is especially high for cases in the first group. A similar pattern has been noted for the natural soil variability when discussing results associated with tests on glacial materials (electric cones and piezocores) and the use of mechanical cones.

The range of mean $N_r$ values determined for ordinary clays appears to be consistent with the range obtained from the theoretical determination by Houlby & Teh (1988) in which the factor of major influence was considered to be the rigidity index. Field measurements reported by Kay & Mayne (1989) supported the latter work.

5 CONCLUSIONS

A statistical study has been performed for assessment of the influence of the natural soil variability and the correlation model error associated with the use of the cone penetration test for $N_r$ evaluation. It appears that useful guidelines may be established to categorize variability according to the description of the soil type. Regarding the conversion error, it is also apparent that the reliability of the correlation model is affected by the
soil type. The evidence indicates that the piezocene calibrated using the field vane or the laboratory triaxial CAIS test is the most reliable of the methods examined for s_t determination.

While the conclusions reached are generally qualitatively consistent with presently accepted practice, the actual values obtained to represent uncertainty in terms of values of coefficient of variation, \( \gamma \), should be specifically valuable for use in a quantitative approach to planning a soils investigation programme and, in the broader context, for probabilistic design purposes. A user-friendly knowledge-based computer system that facilitates the processing of results for planning a soils investigation programme is described in a companion paper (Fung & Kay 1998).

REFERENCES


Drained and undrained shear strengths of a liquefied gravelly fill from Kobe Port Island

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ABSTRACT: This paper presents a series of laboratory test results for high-quality undisturbed samples of a gravelly fill made of weathered granite (locally called Masado) in order to investigate its physical and mechanical properties. The sampling site was located on Kobe Port Island which was liquefied almost over its whole area during the 1995 Hyogo-ken Nanbu Earthquake.

1 INTRODUCTION

During the Hyogo-ken Nanbu Earthquake on January 17, 1995, the coastal area, especially the reclaimed fill and also the artificial islands throughout the Hanshin area (around Kobe and Osaka) experienced intense liquefaction. Port Island, which is one of the largest artificial islands in the area, liquefied almost over its whole area. The upper soil layer, about 20 m in thickness of the island, is composed of a weathered granite hydraulic fill (locally called Masado) from the Rokko Mountain, underlain by several meters of soft compressible alluvial clay. Unfortunately, until the Hyogo-ken Nanbu Earthquake, the seismic activity throughout Hanshin area was considered to be low, and because the Masado is well graded and has a large portion of gravel, only a few researchers paid attention to the liquefaction characteristics of Masado (Tanimoto et al., 1970 and Nagase et al., 1990). As a result, lack of a reliable data base for the liquefaction strength of in-situ Masado fill existed.

In order to investigate the physical and mechanical properties of the liquefied Masado fill, in-situ freezing sampling method was used to recover the high-quality undisturbed samples for laboratory tests from the Masado layer about five months after the earthquake. As shown in Figure 1, the sampling site was located about 80 m from the downhole strong motion station on Port Island (CEDREX, 1995). Maximum accelerations of 0.35 g (GL=-9) and 0.58 g (GL=-16m) were observed at the station during the 1995 Hyogo-ken Nanbu Earthquake. Based on these large ground motions and the settlement observed throughout a wide area of the island, the entire Masado fill layer was considered, by many investigators, to have liquefied during the earthquake to a depth of about 18.5 m. This estimation is supported by test results shown in the present study and will be discussed later.

2 SOIL PROFILE OF SAMPLING SITE

Figure 2 shows the soil profile of the sampling site on Port Island. The gravelly soil layer which extends from the ground surface to a depth of about 18.5 m is Masado fill. This fill layer in the area around the sampling site was reclaimed at the end of 1969, about 25 years before the 1995 Hyogo-ken Nanbu Earthquake. The standard penetration test (SPT) blow count, N-value, for this gravel layer is only about 5 at the ground water table, and increases gradually to 15 at the lower end of the layer (GL=-18.5m).
The SPT used in this study was performed by the trip hammer ("torbi" in Japanese) method. The depth of the samples tested are also shown in Figure 2.

Fig. 2 Soil profile and SPT N-value of sampling site

3. UNDISTURBED GRAVEL SAMPLING BY IN-SITU FREEZING

In this study, undisturbed samples were recovered from the Masado fill layer using the in-situ freezing sampling method, which is basically the same as that used for obtaining gravel samples as reported by Hatanaka et al. (1988). Figure 3 shows the sampling procedure using the in-situ freezing technique for the recovery of Masado samples. The details are described below.

1. A hole about 132 cm in diameter was drilled to a depth of 3 m, about two diameters (260 cm) above the top of the gravel stratum to be sampled. A steel casing 130 cm in diameter was then placed into the hole to prevent it from collapsing. Five guide pipes fixed by two steel plates at both ends were installed into the bottom of the 132 cm hole in order to determine the exact locations of the freezing pipe (one 164 mm-pipe) and sampling pipes (four 216 mm-pipes).

2. A 76 mm bore hole was then drilled with extreme care to a depth slightly lower than the sampling depth. An outer freezing pipe (78 mm in diameter) with a poly vinyl chloride (PVC) rod at its tip was carefully installed into the 76 mm hole. A series of thermocouples was set on the PVC rod in order to monitor the temperature of the soil during freezing.

3. A 22 mm open-ended inner pipe was then placed into the 73 mm pipe with about a 100 mm clearance from the bottom. Liquid nitrogen was transmitted into the inner pipe to freeze the surrounding subsoil profile. After using about a 4 day supply of liquid nitrogen, a 140 cm diameter (approx.) frozen gravel column was achieved.

4. By using a double-tube core barrel, the undisturbed gravel samples were recovered from an undisturbed area about 36 cm away from the outside edge of the cylindrical surface of the outer pipe. Chilled drilling mud was then used in coring the frozen gravel, in order to prevent the frozen sample from thawing during coring.

5. The frozen gravel column was separated at the lower end of the core barrel, about one m long, by pulling the core barrel using a boring machine.

6. The frozen gravel column was removed from the double tube core barrel after the tube has been lifted up to the ground surface.

Photo 1 shows a frozen Masado column, 15 cm in diameter and about 1 m long, recovered from a depth of 7~8 m below the ground surface. It can be clearly seen that a gravel column was obtained with a perfect smooth surface.

Fig. 3 Procedure of in-situ freezing sampling method

Photo 1 Frozen Masado column recovered
4 PHYSICAL PROPERTIES OF SAMPLES TESTED

Figure 4 indicates grain size distributions for the samples tested. It is clear that the Ds of Masudo fill is almost beyond the range of the liqueifiable soils indicated in Specification for Highway Bridges of Japan9 (HJ) which is shown in Figure 4 as a solid straight line. In addition, the gradation curves of the Masudo samples are partially distributed outside the range, that is classified in a category of "likely to be liquefied", as described in the Design Standard for Port and Harbor Facilities9 (PHF), indicated by chain dotted lines. Figure 4 also shows a grain size distribution curve (a solid broken line) for a particular sample of erupted soil taken from the ground surface just after the earthquake near the sampling site. There is a significant difference in particle size between the in-situ Masudo fill and this particular sample of erupted soil. Fine particles could be considered much more likely to erode from the subsoil by excess pore water pressure.

![Diagram of grain size distribution](image)

5 STATIC DRAINED STRENGTH

5.1 Triaxial Test Apparatus

The triaxial test apparatus used for investigating the static strength of high quality undisturbed gravel samples is basically the same as that used for gravel samples (e.g. Hatmak3 a et al., 1980), except that the sample size is different. Confining stress was applied pneumatically and the deviator load was applied by hydraulic pressure. A load transducer was placed inside the cell in order to determine the axial load accurately. The axial displacement was monitored by a linear variable differential transducer mounted on a piston rod.

5.2 Testing Method

The procedures for the static drained triaxial test performed on the high quality undisturbed Masudo samples were as follows:

1. The frozen gravel column was cut to a length of 30 cm using a special saw for preparing a test specimen.
2. The frozen test specimen was then placed on the pedestal, and covered with a rubber membrane and sealed to the pedestal and top cap using o-rings.
3. The frozen specimen was then allowed to thaw in a drained state under a confining stress of 19.6 kPa (0.2 kg/cm²). The frozen sample actually thawed in about three hours at room temperature.
4. After the specimen was completely thawed, it was saturated with the aid of CO₂ gas, de-aired water and a back pressure of about 196 kPa (2.0 kg/cm²), until the pore water pressure coefficient B-value reached 0.95 or greater. After saturation, the specimen was isotropically consolidated at a specified confining pressure.
5. After consolidation, a deviator stress was applied to the specimen in a drained condition at an axial strain rate of 0.1 % per minute. The stress was applied until the axial strain reached more than 15 %. The initial effective confining stress was 0.5, 1, 2 or 3 times that of the effective overburden stress at the sampling depth. Three specimens were tested to determine the angle of internal friction.

5.3 Test results

The typical test results of Masudo fill were shown in Figures 5 and 6 obtained in drained triaxial compression tests. In Figure 5, the deviator stress did not decrease significantly until the axial strain reached about 15 % in every test. This result indicates that the Masudo fill has a large residual strength because of its large dry density and transformation from the volumetric contraction to volumetric expansion as axial strain increases. The angle of internal friction, determined from Mohr's circles at maximum deviator stress and their envelopes, ranged between 20.5° and 41.8° for KPU, KPM and KPL samples, as shown typically in Figure 6. Figure 7 shows the relationship between φ of the undisturbed Masudo fill and the normalized penetration resistance obtained in the SPT. Nₜ calculated from Eq. (1), which is proposed by Liao and Whitman (1986).

\[
N_t = \left( \frac{20N_s}{\phi} \right)^{1/3}
\]

where, \( \phi \) (in kPa) is the effective overburden pressure at the depth where SPT was performed.

\[
\phi = (20N_t)^{1/3} + 20 \quad (3.5 \leq N_t \leq 30)
\]
Also indicated in Figure 7 is the empirical correlation between $\phi$ and $N$, proposed by Hatanaka and Uchida (1996) for sandy soils (Eq. (2)). As shown in Figure 7, the angle of internal friction of Masado fill is larger than that estimated by Eq. (2) for sandy soils.

![Figure 7](image1.png)

**Fig. 7** Results of CD test for Masado fill (KPU sample)

![Figure 6](image2.png)

**Fig. 6** Mohr's circles and a failure envelope from CD test for Masado fill (KPU sample)

![Figure 7](image3.png)

**Fig. 7** Comparison of $\phi$ between Masado fill and the empirical correlation proposed for sandy soils

6 PERMEABILITY COEFFICIENT

6.1 Permeability testing apparatus and test method

A schematic diagram of the permeability testing apparatus using a triaxial cell is illustrated in Figure 8. In this testing system, a constant-head type permeability test can be carried out for a specimen at an isotropic confining stress. The coefficient of permeability of the porous plate was measured to be larger than $1 \times 10^{-4}$ cm/s in a preliminary test. The frozen specimen was set in the cell to thaw under a small confining stress in a drained condition. The specimen was then saturated with the aid of CO$_2$ gas, de-aired water and a back pressure of about 98 or 196 kPa. After saturation, the specimen was isotropically consolidated at a specified confining pressure, equal to the effective vertical stress at the sampling depth. The hydraulic gradient, $i$, was applied from 0.1 to 0.5 at an interval of 0.2 in the permeability tests.

![Figure 8](image4.png)

**Fig. 8** Permeability test apparatus

![Figure 9](image5.png)

**Fig. 9** Coefficients of permeability for Masado fill (KPU and KPM samples)

6.2 Test results

Figure 9 indicates the test results of the permeability coefficient for undisturbed Masado fill, KPU and KPM samples. The coefficient of permeability for each sample was almost the same value at every hydraulic gradient. These values were approximately as same as those for undisturbed sand samples reported by Hatanaka et al. (1997) using the same type of testing apparatus. The permeability of Masado fill was small because the Masado samples had not only large gravel particles but also fine particles. Based on this result, the cyclic shear strength of Masado fill during earthquake should be determined in an undrained condition.
7 CYCLIC UNDRAINED STRENGTH

7.1 Cyclic Triaxial Test Apparatus and testing method

The cyclic undrained triaxial test was performed on the undisturbed Masado samples (KPU, KPM and KPL). The test apparatus is basically the same as that used in the static drained tests. The procedures for the cyclic undrained test of Masado fill were the same as that for the static drained test from preparing the sample to the consolidation. The initial effective confining stress used in the cyclic undrained test was the effective vertical stress at the depth of sampling. After consolidation, cyclic deviator stresses were applied to the specimen in an undrained condition until the double amplitude axial strain was larger than 5%. The cyclic deviator stresses were applied in uniform sinusoidal cycles at a frequency of 0.1 Hz.

7.2 Test results

Figure 10 (a) indicates typical time histories of cyclic deviator stress, excess pore water pressure and axial displacement during the cyclic undrained triaxial test for a KPU sample. It can be seen that the amplitude of cyclic deviator stress was successfully maintained at a constant level, even when the specimen deformed with more than 5% of double amplitude axial strain.

Figure 10 (b) shows a typical stress-strain relationship during the cyclic shear. It is clearly known that the axial strain progresses gradually on the extension side like other undisturbed sands and gravels. Figure 10 (b) indicates a typical stress-path of undisturbed Masado samples.

The correlations between the cyclic stress ratios required to cause a double amplitude axial strain (DA) of 5% and the cycles of stress application are shown in Figure 11 for the samples KPU, KPM and KPL, respectively. As indicated in Figure 11, the stress ratios for DA=5% in 15 cycles are within a range of 0.15 (KPU samples) to 0.25 (KPM samples). Considering its large density and the small void ratio, the liquefaction strength of Masado fill is very low. The liquefaction strengths of Masado fill were nearly equal to that of Toyoura sand with a relative density of about 70% as shown in Figure 11.

As shown in Figure 12, the liquefaction strength for undisturbed Masado fill obtained in this study was also compared with the empirical correlation based on the high-quality undisturbed sand, as proposed by Yoshimi et al. (1980). It is obvious that there is a fairly good agreement between the test results of Masado samples and the proposed curve in the range of N from 7.2 to 11.0. This result obtained from cyclic triaxial test suggests that the liquefaction strength of the man-made Masado fill may be roughly estimated from the normalized SPT
8 CONCLUSIONS

Based on the laboratory test results for the Masado samples obtained by in-situ freezing method and the damage observed after the Hyogo-ken Nanbu Earthquake, the following can be concluded.

1. The angle of internal friction of Masado fill ranges between 39.5 and 41.8 degrees, these values are almost the same as those of other gravels, but higher than that of sandy soils with the same N value.

2. The permeability coefficient of Masado fill ranges between $2 \times 10^{-5}$ and $10^{-4}$ cm/sec, these values are almost the same as those for the undisturbed sandy soils shown by Hatatuka et al. (1997). As a result, it is reasonable to investigate the cyclic shear strength of Masado fill under undrained condition.

3. In spite of its large dry density and gravel content, the liquefaction strength (the cyclic stress ratio to cause 5 % double amplitude axial strain in 15 cycles of cyclic stress) of the Masado fill is very low, only 0.15 to 0.23. This value is as low as that of Toyoura sand which has a relative density of about 70 %.

4. The liquefaction strength of the Masado fill is almost consistent with that estimated using the normalized SPT N-value, N, from the empirical correlation proposed by Yoshimi et al. (1989) for clean sands. This result suggests that the liquefaction strength of the Masado fill obtained in cyclic triaxial test can be roughly determined using the simplified procedure with N value.

9 ACKNOWLEDGMENT

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10 REFERENCES


Burnaby Lake – A case history of piezocene testing and ground penetrating radar

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ABSTRACT: This case history describes the use of Piezometer Friction Cone Testing (CPTU) and Ground Penetrating Radar (GPR) to characterize a site on the shore of Burnaby Lake. Alluvial fan sands were deposited over the soft clays and silts of the Still Creek Basin deposits, resulting in the squeezing out of the soft soils. GPR was carried out on land and over water. Results are presented to illustrate the detailed stratigraphic profiling possible using the CPTU, the additional information provided by the resistivity module on the CPTU and the use of GPR to interpolate between CPTU soundings.

1 INTRODUCTION

This paper presents the results of a site characterization in which ground penetrating radar (GPR) carried out from the surface was combined with piezo-cone penetration testing and drilling and sampling to provide a detailed picture of the subsurface stratigraphy of a small alluvial fan overlying peats and soft soils deposited in the Burnaby Lake basin. The work was carried out for the Greater Vancouver Water District.

2 SITE DESCRIPTION AND GEOLOGY

The site was located at Burnaby Lake in the Lower Mainland of British Columbia, Canada where a drinking water pipeline crossed the lake and came ashore on the south side of the lake (Figure 1). The study area was bordered on the south side by Highway 1. Burnaby Lake represents a widening of Still Creek which flows at low gradient from west to east and joins the Brunette River which in turn flows into the Fraser River. The lake is shallow and almost filled with thick deposits of peat as well as sand and silt contributed from streams flowing in from the surrounding higher ground.

Geotechnical exploration in the late 1950’s prior to the construction of Highway #1 indicated that soil conditions on the south side of Burnaby Lake comprised silt and sand wash deposits overlying peat, overlying very soft to soft clayey silt over firm to stiff silt over glacial till. The clayey silts were glaciomarine deposits. The various deposits were later-weathered and interbedded. At the end of glaciation, rising ocean levels flooded the isostatically depressed land surface. The soil immediately over the till may include an initial deposit of marine drop tilts formed by melting icebergs dropping soil materials through the water, overlain by extensive subsequent deposits of silts and clays deposited into a marine environment. Where higher energy outwash streams existed, deposits of sand or sand and gravel were formed, particularly along the margins of the basin. These deposits were interbedded with the finer grained deposits of silt and clay.

Over time, the elevation of the land surface increased relative to sea level, resulting in Burnaby Lake being raised above sea level and the area becoming a fresh water lake. Thick sequences of peat formed in the freshwater environment of Burnaby Lake resulting in the accumulation of thicknesses of peat of up to 6 m. As a result of the change to a fresh water system, leaching of the salts in the original marine silt and clay deposits likely occurred. Hoy, et al. (1967) note that the salt contents in the pore water are no greater than for fresh water.

Around the margin of the lake, tributary streams
entering the basin deposited additional sand or sand and gravel to form alluvial fans, some of which extended out into the lake. Where both the peat and the sand and gravel existed, interbedded or inter-tongued deposits of peat and sand and gravel resulted. The proportion of peat would generally be expected to be highest near the lake and to decrease with increasing elevation away from the lake. The study area was on the edge of one such alluvial fan formed by Robert Burnaby Creek. Deposition of sediments on parts of the fan is ongoing with up to 1 m or more of sediment being deposited on the head of the fan during a high flow event several years ago.

3 SITE CHARACTERISATION

The stability of the lake shore in the vicinity of the pipeline required consideration. Evidence that the area might be subject to slide activity included linear surface features having the form of slight linear depressions which were judged to be tension cracks. It appeared possible that the sands and silts overlying the weak lake deposits could have experienced spreading towards the lake. Erosion along the toe of the slope by more concentrated flows in the lake may have been a contributing factor. Of particular interest to the investigators were the thickness of the alluvial fan, identification of the materials underlying the fan, the topography of the till surface, and delineation of the stratigraphy in the vicinity of the pipeline as it entered the lake.

A first phase of field exploration was planned to obtain a general picture of the soil stratigraphy, including depth to the till surface, and to identify locations for more intensive investigation. Seismic refraction geophysics and an initial phase of drilling and piezometer core penetration testing (CPTU) were carried out near the lake shore. Till was encountered at a depth of about 15 metres. A CPTU profile is presented as Figure 2. The cone used for CPTU testing allowed measurement of tip resistance, sleeve friction and induced pore pressure at 0.05 m intervals during penetration at 2 cm/s. The cone used for this
portion of the work incorporated a resistivity module which permitted measurement of the bulk resistivity of the soil adjacent to the electrodes. The resistivity profile is also shown on Figure 2.

The refraction seismic investigations were intended to identify the depth to till and to allow interpolation of the stratigraphy between drilling locations. However, the results of the initial refraction study failed to correlate with the drilling and CPTU testing, likely due to the presence of a softer layer under the surficial stiff layer. Further site characterization used CPTU testing Ground Penetrating Radar (GPR). The CPTU was used to define the stratigraphy and the depth to till. The GPR work was undertaken to assist in the delineation of the subsurface stratigraphy between cone holes, in particular the location of the till surface and interfaces within the soft sediments, but was found to also provide information on the deformation of the sediments.

3.1 Ground Penetrating Radar

GPR surveys are made by timing the arrival of reflections from a radar signal sent into the ground at a series of closely spaced points along a survey line. The signals are reflected at changes in the dielectric properties of the sediments. The travel time to a horizon depends on its depth, the velocity of the GPR signal along the ray path and the antenna separations. The velocity of the GPR signal is primarily dependent on the water content. The results of numerous reflections are plotted on a time-distance profile to produce a section.

All data were acquired with PolamDeko IV equipment with a 1000 volt transmitter. For the land-based GPR, the equipment was deployed on foot using 50 MHz antenna at 1 m separation over the lake. Readings were taken at about 0.25 m stations with survey control complicated by strong winds. At each station, 128 readings were stacked.

The GPR data were interpreted using a constant velocity determined from a Common Mid Point (CMP) survey. A CMP is a survey where the source of the signal is moved a fixed distance from the receiver. This allows for the determination of the velocity of the signal through the sediments.
antenna separation, S. S vs. T will plot as a hyperbola with a curvature dependent upon velocity.

3.2 Drilling and Cone Penetration Testing

Eleven auger (solid and hollow stem) boreholes and 17 CPTU holes were advanced on three north-south oriented GPR section lines, and 6 electronic piezometers were installed. All CPTUs and five boreholes were advanced to the till surface. Soil samples were obtained for visual identification and classification testing by piston sampling, disturbed sampling off the augers and Standard Penetration Testing. No drilling was done over water.

4 RESULTS

Figure 3 shows a down-slope section with the CPTU tip resistance profiles superimposed on the GPR data. This is one of several composite sections which were developed. The composite section shows the following soil conditions:

1. Sand interbedded with peat and some silt: The sand was overlain by thin peat. The thickness of the sand sequence was typically 5 m on the uphill side of the section and it gradually pinched out to the south. The sand was interbedded with amorphous peat.

2. Amorphous Peat: The amorphous peat formed a bed within and at the bottom of the sand sequence up to 2 m thick. There were two relatively thin beds of silt or sand which gradually pinched out or disappeared in an uphill direction. The contact with the underlying silty clay tended to be interbedded.

3. Clayey silt or silty clay (Plasticity Index (PI) ~10 to 40, Liquidity Index (LI) ~0.7 to 3.4): Clayey silt (or silty clay) was present under the sand/peat sequence and overlying the till throughout the entire section. The overall thickness of the clay varied from less than 1 m on the upper end of the section to more than 10 m near the lake. Detailed examination of the cone data and recovered samples suggested that there were at least three zones within the clay: an upper zone which included some of the weakest material; a middle lower zone of material which was slightly stiffer and slightly stronger; and a bottom zone, comprising bedded silts with a laminated or varved structure which were stiffer than the overlying silts. Except for the bottom zone, the clay was weak and remoulded to a very soft material (like toothpaste). The plasticity index decreased with depth. The liquidity index exceeded one.

4. Gravelly Sand Till: The weaker sediments above were underlain by a sloping till surface. The till was dense and much stronger than the overlying sediments. The resistivity was high in the surficial sands and in sand lenses within the peat and decreased to about 20 ohm-m through the clayey silts. It began to increase close to the base of the silt suggesting a decrease in the conductivity, i.e., fresher pore water. This was confirmed by the piezometers which indicated water pressures at the till surface which were higher than hydrostatic indicating upward flow.

The over-water GPR allowed the detection of a region of distorted sediments on the west side of the pipeline as shown on Figure 4. The interpretation of the layering and the inferred location of the pipe are also shown. This observation was consistent with the history of the site. Fill had been placed to allow access for pipe maintenance. There was evidence of displacement of the underlying soft sediments and this helped to explain some of the settlements of the pipeline known to have occurred.

5 DISCUSSION

The combined plots of the GPR data and the cone data were made by plotting the GPR and cone data to a common scale assuming a single radar transmission velocity for each section. However, as the velocity of propagation of the radar signal will vary for different materials, the use of a single velocity will result in internal distortions within the section where different materials and water contents occur. As a result, there will be some discrepancies in the location of reflectors on the sections relative to their actual positions. The most significant distortions will be where the sand pinches out and the majority of the material at the near surface vertical section becomes high moisture content peat and organic silt.

Despite this limitation, the GPR profile on Figure 3 provides considerable detail about the sand and allows interpolation between the CPTU soundings. In particular, distortion of the sand and silt beds near CPTU-11 and in a wide zone uphill of CPTU-8 was
Figure 3. Stratigraphic cross-section based on composite CPTU and GPR section.
interpreted as evidence of slide movements. At both locations, surface breaks and drops and distorted trees were found in locations which correlated with the GPR areas of apparent tensile movement and shearing. Apparently active tension cracks were also found near the uphill end of the GPR section. The interbedding of peats and sands noted on the section suggests that there have been intermittent influxes of sand. This fits our understanding of the geologic environment. Also of significance to the evidence of sliding in this area was the continuity of the clay layer below the sands even though it became thin towards the uphill side.

6 CONCLUSION

For this site, the combination of GPR and CPTU testing proved useful in the delineation of a complex stratigraphy. By allowing detection of distortion in bedding, the GPR improved our ability to interpret surface features suggestive of landsliding. The GPR data were also very useful in extrapolating the discontinuous units between boreholes. For the offshore sections, the GPR in combination with knowledge of the history of construction in the area permitted delineation of the near surface stratigraphy without the need for costly over water drilling.

This was the authors' first experience combining GPR and the CPTU. The results were encouraging but it is considered that the value of GPR could be enhanced by processing the data to provide a depth display which accommodates vertical and horizontal variations in velocity. This would enhance the ability to interpolate between the detailed stratographies obtained at the CPTU locations. The resistivity module on the cone can also be used to delineate changes in conductivity, providing additional information about the groundwater.

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REFERENCES

Site characterization of a dynamically compacted silty fine sand

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ABSTRACT: A reclaimed land of approximately 2400 hectares is under construction as part of an industrial development at Mai Liao Township, on the West Coast of Taiwan. Silty fine sand dredged offshore was placed hydraulically to reach a final grade of 3 m above sea level. The foundation soil consists of similar material that extends to several hundred meters below sea level. The new sand fill and the underlying foundation soil was dynamically compacted to increase the sand density within top 10 m. Earlier attempts to estimate pile shaft frictions using CPT data according to existing methods were 30 to 70% below those measured in pile load tests at Mai Liao. A new method to predict the shaft friction that considers the characteristics of the silty fine sand at Mai Liao and the effects of dynamic compaction is proposed. The new method was developed based on a series of laboratory experiments that include CPT chamber calibration tests. According to comparisons with four pile load tests, the new method provides a reasonable and slightly conservative prediction for the shaft friction of driven PC piles. The average error was -12%.

1 INTRODUCTION

A project that involves the construction of a 2400 hectares of reclaimed land is being carried out (August 1997) at Mai Liao Township, on the West Coast of Taiwan. The new land is being developed into a petrochemical industrial complex. The soil deposit in the construction area consists mostly of silty fine sand with occasional layers of low plastic silty clay. The depth of this sand deposit exceeds several hundreds of meters. Sand with fines contents in excess of 20%, dredged offshore was placed hydraulically to reach a final grade of approximately 3 m above the sea level. The new fill and the foundation soil near the ground surface were loose. Dynamic compaction (DC) was used to increase the density of sand within the top 10 to 13 m.

Approximately 460,000 piles are expected to be installed at the site to support elevated utility lines and other building structures in the new industrial complex. They are mostly precast concrete (PC), circular piles with a diameter of 500 or 600 mm. Nominal depths of these driven piles are 30 m. Due to the lack of a load soil layer at the toe level, the PC pile develops its capacity almost entirely from shaft friction. The piles are expected to experience uplift as well as compressive forces because of the nature of structures to be supported. Thus, a reasonable estimate of the frictional capacity is imperative for the success of such a large number of piles.

Because of the cohesionless nature of the soil at the Mai Liao project site, cone penetration tests (CPT) are used extensively to characterize the site conditions. CPT data provide the basis for DC quality assurance and design of pile foundations, amongst other purposes. Earlier attempts to estimate the PC pile shaft frictions using the CPT data according to the Schmertmann’s method (1978) were 30 to 70% below those measured in pile load tests at Mai Liao. Campanella et al. (1989) evaluated five methods that involve the use of CPT to estimate pile capacities, including the Schmertmann’s method. They concluded that these methods generally agree with each other within 25%. Similar findings were obtained when applying these methods to the piles at Mai Liao.

The importance of mineral content, particle shapes as well as gradation to the behavior of sand has long been recognized (Koerner, 1970; Jonstra and de Gijl, 1982). The CPT related design
methods such as those mentioned above are often restricted to clean, uniform, moderately compressible quartz sand. The silty sand to be referred to in this paper hardly fits that description. In addition, the application of DC practically assures that the silty sand is not normally consolidated, at least within the top 10 to 13 m. The state of overconsolidation further complicates the interpretation of CPT data. Because of these reasons, a series of studies were carried out to establish a proper procedure to characterize this DC improved silty fine sand. One of the main objectives is to develop a CPT interpretation method specifically for this silty sand and then use it to estimate frictional capacities of piles.

A batch of 20 tons of Mai Liao sand (MLS) was taken from the project site to provide specimens for laboratory experiments. These experiments include a series of one-dimensional compression tests, drained triaxial tests with volume change measurements and CPT calibration tests in a chamber. Field axial compression and tension load tests were carried out on PC piles instrumented with strain gauges to determine the distribution of shaft friction along the piles. These piles were tested to failure. A new method to predict shaft friction for PC piles in MLS was developed based on the CPT calibration tests. This new method was validated against the field pile load tests.

This paper describes the characteristics of the Mai Liao silty sand and the new CPT interpretation method. The validity on the use of this new method to estimate pile shaft friction is discussed.

2 CHARACTERISTICS OF THE MAI LIAO SILTY SAND

Most of the soils on the west coast of Taiwan came from the central mountain range. Rainfalls flushed deteriorated sedimentary and metamorphic rocks, such as shale, slate and mudstone, through steep slopes and rapidly flowing streams before settling down on the west plain. The process of transportation ground the fractured rock into sand and silt particles. The batch of MLS taken from the test site is classified as a silty fine sand (SM) with a nominal fines (particles passing #200 sieve) content of 15%. X-ray diffraction analysis on MLS showed significant contents of muscovite and chlorite, in addition to quartz. The MLS has a maximum void ratio, \( e_{max} \) of 1.04 and a minimum void ratio \( e_{min} \) of 0.57.

The compressibility of MLS was measured one-dimensionally in an oedometer originally designed for creep tests on rock specimens. A comparison with quartz sand under similar loading conditions (Yuanamuro et al., 1976) show that MLS is at least 5 times as compressible as quartz sand, depending on the stress level and density. The one-dimensional compression index according to the \( e-\sigma' \) curves at \( \sigma' \) values between 3 and 10 MPa, is approximately 0.29. This compression index value is comparable to that of an inorganic lean clay. All specimens showed an increase of fines content of approximately 3% after the test. Judging from the moderate increase of fines content, it appears that the compression of MLS under one-dimensional loading is mostly a result of rearrangement of soil particles. The compressibility indicates a strong dependence of void ratio on the confining stress and that loose MLS cannot exist under a high confining stress.

A series of isotropically consolidated drained triaxial (CID) tests were performed on MLS with volume change measurements. The main objective of these tests is to reveal the strength and dilatancy characteristics of MLS. The concept of relative dilatancy index, \( I_{D} \) (Bolton, 1986) is used to evaluate the triaxial test data as they relate to strength and dilatancy. Bolton (1986) has suggested that the drained friction angle, \( \phi' \), should be obtained by dropping a tangent from the origin to a single Mohr circle (i.e., a secant friction angle) that represents the maximum deviator stress conditions from a triaxial test. The secant drained friction angle under the critical state (i.e., a condition where shearing continues without volume change), \( \phi'_{c} \), is mainly a function of mineral content of the sand. The \( e_{min} \) of MLS measured from specimens with relative density, Dr, of 50%, varied from 30.8° to 32.4° with an average value of 31.6°.

According to Bolton (1986), \( I_{D} \) combines the effects of sand density and confining stress, empirically

\[
I_{D} = \frac{D_{r}}{100} \left( \frac{Q}{\log \rho} \right)^{2} - 1
\]

where \( Q \) is an empirical constant that increases with the crushing strength of sand grains and \( \rho' \) is the mean effective stress at peak deviator stress.
stress history due to dynamic compaction. The chamber specimen is 535 mm in diameter and 760-815 mm high. The diameter ratio (RD) of the specimen over a standard cone penetrometer (i.e., diameter of 3.65 mm) is 15. Such a diameter ratio would be considered unacceptably small for CPT calibration tests in clean, quartz sand because of the severe boundary effects (Parkin, 1988). To verify the significance of boundary effects in MLS, a comparison was made by performing CPT in the chamber using a standard cone penetrometer and a 1/2 size cone (diameter of 17.8 mm). Both tests were conducted in a specimen with initial Dr of 70% and under the same boundary stress conditions ($\sigma'_{V0} = \sigma'_{H0} \approx 294.3$ kPa). The diameter ratio for the 1/2 size cone is 30. If there is significant boundary effect, the cone tip resistance, $\sigma'_{vt}$, of the 1/2 size cone should be significantly larger than those of the standard cone. Figure 1 shows the $\phi$ profiles from these two tests. The average $\phi$ values taken at depths between 200 and 500 mm, where they have reached a plateau, agree within 4%. The insensitivity of $\phi$ to the change of diameter ratio, comparable to CPT calibration tests in very loose uniform quartz sand (Parkin, 1988), is a direct reflection of the compressible (or lack of dilatancy) nature of MLS. An important additional implication is that the chamber specimen is large enough for a standard cone penetrometer and there should be minimal boundary effects.

Piezometers were installed within the saturated and back-pressured chamber specimen to monitor the development of excess pore pressure at mid-height of the specimen, during cone penetration. For all the calibration tests performed, the excess

3. LABORATORY CPT CALIBRATION IN THE SILTY SAND

Variables applied in the calibration tests include initial Dr of the specimen, effective vertical stress $\sigma'_{V0}$, and the ratio of effective horizontal stress $\sigma'_{H0}$ over $\sigma'_{V0}$ (K). Table 1 summarizes the range of these variables. All tests used stress-controlled vertical and horizontal boundary conditions.

The K values shown in Table 1 were selected to reflect a moderately to highly overconsolidated condition.

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<td>2</td>
</tr>
</tbody>
</table>

Figure 1. Comparison of $\phi_v$ with two different diameter ratios.
pore pressure measured in the piezometers did not exceed 60 kPa, compare to $q_c$ values which are in terms of MPa. Based on these findings, it is reasonable to consider CPT in MLS as a drained test, at least for the tests reported herein. Also, because the pore pressure development was minimal, no pore pressure correction was made to convert $q_c$ to $q_1$.

Figure 2 shows the averaged $q_c$ values and their relationship with $\sigma_3'$. The average $q_c$ is determined based on readings taken in calibration tests at depths from 200 mm to 500 mm where $q_c$ had reached more or less a steady value. The $q_c$ in MLS has a clear and positive relationship with $\sigma_3'$. The relationship between $q_c$ and $\sigma_3'$ is not nearly as obvious. For low compressibility sand, there have been conflicting reports on the relationship between $q_c$ and $\sigma_3'$ (Houlsby and Hitchman, 1988; Huang and Ma, 1994). In practice at least, $q_c$ in sand is rarely determined based on $q_c$. The trend shown in Figure 2 is an indication that for a high compressibility sand such as MLS, there is a potential to determine $\sigma_3'$ using $q_c$.

![Figure 2. Relationship between $q_c$ and $\sigma_3'$.](image)

Because of the high compressibility of MLS, the change of void ratio or Dr after the specimen is subjected to a confining stress is significant. The volume change in all the chamber tests were accounted for in presenting the data. All of the CPT calibration data were then compiled and an empirical equation that relates $q_c$ with Dr are developed. This equation follows the pattern of Fioranelli et al. (1991) that considers the effects of $\sigma_3'$ and $\sigma_3'$ as follows

$$q_c = 230[\sigma_3']^{0.11}[\sigma_3']^{0.11} \exp[0.45Dr]$$

A comparison between Equation (4) and the chamber CPT data shows a coefficient of correlation of 0.966.

4. PREDICTING PILE SHAFT FRICTION USING CPT DATA

The new method simply defines the pile shaft friction, $q_1$, as

$$q_1 = \sigma_3' \tan \delta$$

where $\delta$ is the friction angle between the pile surface and the surrounding soil. The CPT calibration tests showed a strong relationship between $q_c$ and $\sigma_3'$, it would thus seem reasonable to estimate $q_1$, using Equation (4). To use Equation (4), it is necessary to know $q_c$ and Dr. For a given depth, $q_c$ can be estimated with reasonable accuracy with an assumed soil density and known water table. Because of the compressibility of MLS as above described, Dr cannot completely decouple from stress conditions. Thus, according to Equation (4), a good estimate of Dr should result in a reasonable computed value of $q_1$ that is compatible with the stress history in the field. The use of sleeve friction readings from CPT is avoided in this method. The CPT sleeve friction is known to have scale effect (Parkin, 1988) and is potentially a source of error when used to predict shaft friction.

According to a rather comprehensive experimental study by Pauwowski et al. (1995), $\delta$ is a function of the relative roughness of the pile surface and the friction angle of the sand. The MLS is a silty fine sand with an average grain diameter ($D_50$) of 0.15 mm. The fine nature of MLS should make the pile/soil interface behave as a "rough" contact. For a rough pile/soil interface, shearing occurs within the soil and $\delta$ should be identical with the soil friction angle (Pauwowski et al., 1995). Because of the lack of dilatancy, MLS has a relatively narrow range of $\phi_{sdr}$, $\phi_{sdr}$, as above indicated. Also, the dynamic compaction at the Mai Lin project site would increase $\sigma_3'$ in the field that further inhibits soil dilatancy. For these reasons, $\phi_{sdr} = 32.6^\circ$ should represent a lower
bound but reasonable estimate of $\delta$, at least for the piles tested and restricted to MLS.

5. FIELD VERIFICATION OF THE NEW METHOD

To validate the use of Equation (5) to predict $q_e$ for piles, 4 PC pile load tests were conducted at three test sites in Mai Liao project area. The piles were axially loaded by compression to failure. Table 2 describes the specifications of the test piles. The surface of test pile 2C was roughened by carving evenly distributed 10 mm wide and 10 mm deep channels when the pile was casted. The channels had a wavy shape.

<table>
<thead>
<tr>
<th>Site No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Pile No.</td>
<td>IC</td>
<td>2C</td>
<td>3C</td>
</tr>
<tr>
<td>Pile surface</td>
<td>S</td>
<td>R</td>
<td>S</td>
</tr>
<tr>
<td>Pile diameter, mm</td>
<td>600</td>
<td>600</td>
<td>600</td>
</tr>
<tr>
<td>Pile length, m</td>
<td>30</td>
<td>30</td>
<td>30</td>
</tr>
</tbody>
</table>

Table 2. Specifications of the test piles.

The soil profile and the amount of information available at the other test sites are similar to Figure 3. The fines contents at these test sites are slightly higher than those in the chamber CPT calibration tests. To prevent the fines from being blown away by wind, the construction area was covered by a layer of gravel immediately after the sand fill reached the design grade. The high $q_e$ values within 2 m of the ground surface shown in Figure 3 are mostly the result of the gravel layer. To facilitate the use of Equation (4), it is assumed that Dr is 80% from 0 - 8m, 70% from 8 - 15m, and 60% from 15 - 30m. Figure 4 describes the profile of $K$ values ($\sigma'_k/\sigma'_k$) from 2 (below the gravel layer) to 30 m, according to Equation (4) and $q_e$ of Figure 5. The distribution of $K$ as shown in Figure 4, despite of its scatter, reflects a soil profile that has been improved by DC. Soil near the ground surface is close to passive failure. $K$ decreases with depth as the effects of DC diminishes, and gradually approaches a value of normal consolidation. Considering the young age of the sand deposit and new fill placed at site, the results are reasonable.

Results of the 4 pile load tests, in terms of displacement and axial load at the pile head, are depicted in Figure 5. The test curves of load tests are typical for friction piles where the pile displacement increases suddenly after the frictional resistance is fully mobilized. All compression load tests show minimal end bearing. To use Equation (4), the maximum $q_{e,ref}$ obtained from below the 2m depth was used for the soil within the top 2 m. Table 3 shows a comparison between the average $q_e$ measured ($q_{e,meas}$) in pile load tests and those computed ($q_{e,comp}$) using the method as previously described. The error shown in Table 3 is defined as $(q_{e,meas} - q_{e,comp})/q_{e,meas} \times 100%$.

The computed $q_e$ consistently underpredict the measured value by 8 to 17%. The underprediction is in agreement with the use of $\xi_{sep}$ as the soil/pile interface friction angle that represents a lower
bound. The higher fines contents in the field may also contribute to the underprediction as CPT is closer to an undrained condition and that results in lower $q_u$. In any case, the method proposed in this paper provides a reasonable but slightly conservative prediction of the shaft friction for PC piles driven in DC improved silty fine sand. The limited data indicate that roughening the pile surface does not significantly alter the frictional characteristics of the PC pile. This finding is consistent with the laboratory study by Paykowsky et al. (1995), as all the PC piles are “rough” in comparison with the size of silty fine sand.

Table 3. Comparison between the measured and computed $q_u$.

<table>
<thead>
<tr>
<th>Test pile No.</th>
<th>$q_u$ (meas.)</th>
<th>$q_u$ (comp.)</th>
<th>Error, %</th>
<th>Average error, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1C</td>
<td>73</td>
<td>64</td>
<td>-12</td>
<td></td>
</tr>
<tr>
<td>2C</td>
<td>78</td>
<td>64</td>
<td>-17</td>
<td></td>
</tr>
<tr>
<td>3C</td>
<td>62</td>
<td>57</td>
<td>-8</td>
<td></td>
</tr>
<tr>
<td>4C</td>
<td>63</td>
<td>57</td>
<td>-10</td>
<td></td>
</tr>
</tbody>
</table>

Figure 5. Results of axial compression load tests.

CONCLUDING REMARKS

The proposed method of predicting pile shaft friction based on CPT is unquestionably soil specific and may even be site specific. Nevertheless, it was developed because of the inadequacy of the existing methods. An unfortunate disadvantage of the new method is that it is necessary to estimate the profile of Dr. This disadvantage can be partially compensated, however, as the measured Dr has to result in a reasonable profile of the K values.

REFERENCES


Chamber calibration of CPT under simulated field conditions

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ABSTRACT: An important drawback of calibrating CPT in a chamber is its boundary effects. The lateral boundary of the specimen in a conventional calibration chamber is either stress controlled or rigid. Neither of these two boundary conditions can correctly duplicate the field conditions which lie somewhere in between. In order to eliminate the boundary effects, a new calibration chamber system that enables the field conditions be simulated has been developed. The new chamber wall, 800 mm in diameter and 1600 mm high, is made of 20 vertically stacked steel rings. The boundary movement induced by the cone penetration is measured at each ring level by an extensometer. The lateral stress at each ring level is determined by the stress-strain relationship of the specimen. A series of CPT calibration tests has been performed in a uniform, dry silica sand using the new system. Results show that the new simulator is capable of eliminating boundary effects, even at a relative density of 84%. The paper describes the design of the new chamber system and presents available test data.

1 INTRODUCTION

Due to lack of cohesion, it is difficult to obtain undisturbed samples for sand. In-situ tests are often used to determine engineering properties for sands. The cone penetration test (CPT) is a popular in-situ testing method. However, as for many other in-situ testing methods, we rely on empirical rules to interpret CPT data. Since its early development in the late 1960's (Holden, 1991), the calibration chamber has been an important research tool in establishing interpretation procedures for CPT in sand.

A conventional calibration chamber is capable of creating four types of boundary conditions as shown in Table 1.

<table>
<thead>
<tr>
<th>Condition (No.)</th>
<th>Vertical boundary</th>
<th>Lateral boundary</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>stress</td>
<td>strain</td>
</tr>
<tr>
<td>B1</td>
<td>constant</td>
<td>-</td>
</tr>
<tr>
<td>B2</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>B3</td>
<td>constant</td>
<td>-</td>
</tr>
<tr>
<td>B4</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

The most significant advantages of CPT calibration chamber testing are:
1. Repeatability and uniformity of the specimen.
2. Controlled and known boundary conditions as well as stress history.

The major drawback of calibration chamber is the finite dimensions of a chamber specimen. Been et al. (1988) have indicated that boundary conditions on top and bottom of the chamber specimen have little effect on CPT test results. Parkin (1988) stated that of the four boundary conditions, the most significant are B1 and B3. The field condition is expected to be between B1 and B3. Test results (Parkin and Labue, 1982) show that boundary effects are more obvious in dense sand than in loose sand, and the effects decrease with the compressibility of sands.

The diameter ratio (ratio of the chamber specimen diameter over that of the cone penetrometer, R_d) is used to evaluate the relative scale of chamber and cone penetrometer. For dense sand, CPT results are affected by boundary conditions, even for R_d of 60 (Parkin, 1988). For loose sand, test results are independent from...
boundary conditions (Parkin, 1988), even when $R_d$ is as low as 21.

Ideally, $R_d$ is infinite in the field. In order to account for the boundary effects, empirical correction factors have been proposed (Baldy et al., 1982; Mayne and Kullhawy, 1991). The validity of these correction methods has yet to be verified experimentally by performing CPT in sand with known density, stress conditions and under no boundary effects.

Huang and Ma (1994) used distinct element method (DEM) coupled with boundary element method (BEM) to simulate CPT in a granular material with infinite boundary. An assembly of particulates simulated by DEM replaced the physical sand specimen. The sand from DEM boundary to infinity was simulated as a linear elastic continuum using BEM. Results have indicated the effectiveness of eliminating boundary effects using the coupled DEM/BEM simulation.

The new calibration chamber system consists of a stack of twenty rings to house the sand specimen. These rings are lined with an inflatable silicone rubber membrane on the inside. The boundary expansion and stress are measured and individually controlled, respectively at each ring level during CPT. The soil from physical boundary to infinity is simulated using a non-linear cavity expansion curve derived from a lateral compression test on the specimen. A series of CPT of various diameters has been performed in the new simulator to verify the effectiveness of the new design. This paper describes unique features of this field simulator and presents available CPT data performed in the simulator.

2 GENERAL DESCRIPTION OF THE NEW CALIBRATION CHAMBER SYSTEM

The new calibration chamber system consists of a sand raker, the chamber rings, electronic data logging and control unit, a pneumatic system, and a hydraulic system.

A sand raker similar to that described by Rad and Tamay (1987) is used to prepare the specimen. The lateral boundary was set to rigid, simulating $K_s$ conditions, during sand agitation.

The diameter and height for this new chamber are 750 mm and 1600 mm, respectively. Figure 1 shows the cross section of the new chamber. The vertical stress is applied through four air stroke actuators fixed to the reaction frame. To minimize frictional forces between sand and rubber membranes, the ring stack is supported on four air bellows which give a constant supporting force.

The sand specimen is housed in a stack of twenty rings. These rings are lined with an inflatable silicone rubber membrane on the inside to facilitate boundary displacement measurement and stress control. The membrane is made of 2 mm thick, press molded silicone rubber. The membrane expansion measuring system consists of a wax lubricated, heavy duty fishing line wrapped around the membrane. The ends of the fishing line are attached to a piece of delrin chain and then to a spring loaded extensometer. The extensometer, instrumented with full bridged strain gages, tightens the fishing line and senses circumferential displacement of the rubber membrane.

For every ring, the air pressure control system consists of a digital/analog (D/A) converter channel, an electric/pneumatic (E/P) transducer, an air volume booster, and a pressure transducer. During a cone penetration, the boundary displacement at each ring level is monitored and fetched into a computer program. The stress for each ring level is adjusted pneumatically, according to the desired boundary conditions. The lateral boundary can be set as constant stress (B1), rigid (B3), or simulated field conditions referred to as B5.
3. NUMERICAL SIMULATION OF INFINITE BOUNDARY

Vesic (1977) has demonstrated that the soil displacements at a short distance away from the penetrometer tip can be reasonably described as a cylindrical cavity expansion. Unless the soil specimen is unusually small, it is reasonable to model the soil beyond the physical boundary as a non-linear elastic material (Salgado, 1993; Huang and Mu, 1994). The generic stress strain relationship of the soil mass is:

$$
\phi(\sigma) = (\sigma_r - \sigma_z)
$$

where \( \sigma_r \) = strain in radial direction; \( \sigma_r \) = radial stress; \( \sigma_z \) = circumferential stress and \( \sigma_r = -\sigma_z \) in an elastic medium.

The function \( \phi(\sigma) \) is directly measured from a lateral compression test on the physical specimen. In the case of cylindrical cavity expansion, the following equations govern:

$$
\frac{1}{r} \frac{\partial}{\partial r} \left(n \sigma_r - n \sigma_z \right) = 0
$$

where \( r \) = radial distance from the center of cavity. Compatibility of strains in an elastic medium:

$$
\varepsilon_r = \frac{\partial u}{\partial r} \quad \text{and} \quad \varepsilon_z = -\frac{\partial u}{\partial z}
$$

where \( \varepsilon_r \) = strain in circumferential direction; \( u \) = radial displacement.

The boundary conditions:

$$
\phi(0) = \phi(\sigma) = 0
$$

$$
\sigma_r (r = 0) = \sigma_r
$$

where \( \sigma_r \) = initial or field horizontal stress.

The relationship between stress and radial strain on the physical boundary \( (P_{so}, \varepsilon_{so}, \varepsilon_{mo}, \phi) \) can be derived by integrating Eq. 1 from 0 to \( \varepsilon_{so} \):

$$
P_{so} = \frac{\phi(\sigma)}{2\pi r} \int_0^{\varepsilon_{so}} d\varepsilon
$$

The boundary expansion and stresses are measured and individually controlled at each ring level. During cone penetration, the circumferential displacement at the boundary of each ring level, \( \Delta C \), is converted to \( \varepsilon_{so} \) as

$$
\varepsilon_{so} = \frac{\Delta C}{D}
$$

where \( D \) = diameter of the physical specimen.

The corresponding \( P_{so} \) in response to \( \varepsilon_{so} \), under the simulated field conditions is then determined via the \( P_{so} - \varepsilon_{so} \) curve. The \( \Delta C \) measurements and \( P_{so} \) are continuously updated during cone penetration. Figure 2 depicts the \( \phi(\varepsilon) \) obtained from the lateral compression test and the corresponding \( P_{so} - \varepsilon_{so} \) curve.

Figure 2. \( \phi(\varepsilon) \) obtained from the lateral compression test and the \( P_{so} - \varepsilon_{so} \) curve.

4. TEST RESULTS

A uniformly graded, clean silica sand from Da Nang, Vietnam was used in the experiments. Table 2 summarizes the physical properties of the sand. A series of cone penetration tests were conducted to verify the performance of the newly developed simulator. Two types of cones with cross sectional areas \( A_s \) of 10 and 15 cm² were used to obtain a different range of \( K_t (20.1 \text{ and } 18.1 \text{ respectively}) \) which are needed to verify the effectiveness of B5 simulation. If B5 is successfully simulated then \( q_t \) should be practically the same regardless of \( K_t \) values. After verifying the
capability of eliminating boundary effects, a series of CPT were carried out using the new field-simulator under B5. Table 3 shows the key parameters for all the CPT performed. The cone penetration rate was set at 2.0 mm/second in all the tests. The slow penetration rate was necessary to facilitate boundary stress control. The penetration rate is not expected to influence test results (Dayal and Allen, 1975).

Table 2. Physical properties of Da Nang sand

<table>
<thead>
<tr>
<th>Dₚ  mm</th>
<th>Shape</th>
<th>Gₛ</th>
<th>τₑₑₚ  kPa</th>
<th>τₑₑₚₛ  kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>Subangular</td>
<td>2.61</td>
<td>16.87</td>
<td>14.13</td>
</tr>
</tbody>
</table>

Table 3. Summary of the CPT performed

<table>
<thead>
<tr>
<th>Test No. (Boundary Condition)</th>
<th>Dₚ</th>
<th>Rₑₑ</th>
<th>τₑₑ</th>
<th>Initial τₑₑₛ  kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 - 4(B1)</td>
<td>84</td>
<td>22.1</td>
<td>81.9</td>
<td>40.0</td>
</tr>
<tr>
<td>5(B1),6(B3),7 and 8(B5)</td>
<td>74</td>
<td>22.1</td>
<td>43.7</td>
<td>22.0</td>
</tr>
<tr>
<td>9(B1),10(B3)</td>
<td>84</td>
<td>22.1</td>
<td>43.7</td>
<td>22.0</td>
</tr>
<tr>
<td>11 and 12(B5)</td>
<td>74</td>
<td>18.1</td>
<td>43.7</td>
<td>22.0</td>
</tr>
<tr>
<td>13 and 14(B5)</td>
<td>74</td>
<td>18.1</td>
<td>43.7</td>
<td>22.0</td>
</tr>
<tr>
<td>15 and 16(B5)</td>
<td>84</td>
<td>22.1</td>
<td>43.7</td>
<td>0.5-4</td>
</tr>
</tbody>
</table>

4.1 CPT in the simulator

Test Nos. 1-4 of Table 3 are exact duplications to evaluate repeatability of the tests. Analysis shows that the coefficient of variation amongst these four tests is 1.40 % for cone tip resistance, qₑₑ. A comparison of qₑₑ profiles under B1, B3 and B5 conditions are shown in Figure 3. The qₑₑ values continue to increase with depth under B3, as observed in conventional chamber (Parkin and Lunne, 1982; and Parkin, 1988). The qₑₑ under B5 is higher than that of B3, confirming the fact that qₑₑ under B3 is not an upper limit (Parkin, 1988).

The ΔCₑₑ measurements and Rₑₑ applied to the five of the twenty rings are shown in Figure 4. The depth in Figure 4 are in reference with the cone tip level and normalized with respect to the cone diameter Dₑₑₑₑ. The ΔCₑₑ corresponds to pₑₑₑₑ of no more than 10⁻², small enough to justify the non-linear elastic assumption for soil beyond the boundary. The ΔCₑₑ and pₑₑₑₑ values reach a maximum at 3 to 5 Dₑₑₑₑ ahead of the cone tip, and then decrease slightly but consistently, as the cone tip passes.

Figure 5 depicts the qₑₑ profiles under B5 with Rₑₑ values of 18.1 and 22.1, and Dₑₑ of 74% and 84%. The qₑₑ profiles generally reach a plateau after a depth of 800 mm. Comparisons of qₑₑ values are based on those from depths of 800 to 1200 mm. Results show that the qₑₑ from tests of two Rₑₑ values agree within 4.19 % for Dₑₑ of 74%, and 1.23 % for Dₑₑ of 84%. This is rather convincing evidence that the new simulator is capable of duplicating axi-symmetric field conditions for cone penetration tests.
4.2 Effects of initial stress conditions

In order to evaluate the influence of stress conditions on $q_v$ under simulated field conditions, four different types of earth pressure coefficients ($K = 0.5, 1.0, 2.0$ and $4.0$) with corresponding mean effective stresses ($p^\prime$) of $43.7$, $98.1$, and $147.2$ kPa were chosen. The relative density $D_r$ of the specimen was $74\%$. The average $q_v$ value taken at depths from $800$ to $1200$ mm was used for analysis.

Figures 6 and 7 depict the relationships between $q_v$ and vertical and horizontal stress conditions, respectively according to these tests. The results showed no clear relationships between $q_v$ and vertical stress. The relationship between $q_v$ and horizontal stress is not as strong as that indicated by Houslsby and Hinchman (1988).

There appears to be a reasonable correlation between $q_v$ and the mean effective stress as shown in Figure 8. This correlation can be described by a power function as

$$ q_v = 2.52 \times (p^\prime)_{0.43} \quad (8) $$

$q_v$ is in MPa and $p^\prime$ in kPa.

Equation 8 does not deviate from the test data for more than $13.8\%$. Whether this correlation can be used as a basis for interpreting CPT in sand needs further investigation. It is suffice to say, however, that boundary conditions affect the value of $q_v$ as well as the relationship between $q_v$ and stress conditions.
5. CONCLUDING REMARKS

An axisymmetric field simulator designed for calibrating penetration tests has been developed. Available CPT tests in the new simulator with relative densities of 74% and 84% have indicated that the simulator is capable of duplicating field conditions where the lateral boundary extends to infinity. Because of the capability of physically eliminating boundary effects, it would seem unnecessary to use a correction factor for CPT calibration tests in the future.

Boundary displacement measurements taken in the simulator indicate that unloading does occur in the soil mass as the cone tip passes during penetration. If the unloading effect is neglected and cone penetration is treated as a monotonic cavity expansion (Salgado et al., 1997), \( q_u \) is directly related to the limiting cavity expansion pressure. This analytical procedure would then lead to a \( q_u \) that is higher than the field value, according to the experimental findings presented herein.

Available CPT data under B5 conditions seem to indicate a reasonable correlation between \( q_u \) and the mean effective stress. The finding is different from some of the results reported earlier by others, using a conventional calibration chamber. It is sufficient to say that boundary conditions affect the value of \( q_u \) as well as the relationship between \( q_u \) and stress conditions. Consequently, the earlier CPT interpretation methods established based on tests in a conventional calibration chamber should be re-evaluated.

REFERENCES


Evaluation of undrained strengths obtained from three shear tests

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ABSTRACT: This paper presents evaluation of undrained strengths obtained from unconfined compression tests, constant volume shear box tests and field vane shear tests which were carried out in a site. Theoretical expressions of these undrained strengths derived from the elasto-plastic constitutive model are summarized. And discussed are the correction factors that help converting the undrained strengths obtained from tests to ideal undrained strengths comparable to the theoretical strengths. Throughout comparison of the measured strengths and the theoretical predictions, reliable specification of the constitutive parameters for the model from the measured strengths is examined.

1 INTRODUCTION

Soil/water coupled analysis with an elasto-(visco)plastic constitutive model of soils has been widely utilized in various geotechnical engineering practice (e.g. Duncan, 1994). However, unless rationality and reliability in specifying constitutive parameters required for the model be confirmed, such non-linear analysis would not function in practical use. Since such constitutive parameters required for the elasto-plastic constitutive model are, in general, not provided directly by the usual site investigation program, there still remains much difficulty in estimate of the constitutive parameters. The authors has made attempts aiming at establishing the reliable specification procedure of the constitutive parameters required for the elasto-plastic constitutive model proposed by Sekiguchi and Ohta (1977) (e.g. Izuka and Ohta, 1987, Ohta et al., 1991, Ohta et al., 1992, Ohata et al., 1994). In this paper, throughout the evaluation of undrained strengths obtained from tests performed as a part of the site investigation program, reliable specification of the constitutive parameters from information provided by the site investigation is discussed. If all undrained strengths obtained from tests can be well explained by the constitutive model with a single set of constitutive parameters, one would be able to rely on the specification procedure of the set of constitutive parameters.

2 SITE DESCRIPTION

The site described in this paper is located on a reclaimed land at Shibayama in Ishikawa, Japan. The road embankment is under construction and a series of tests was performed relating to the road construction work. The test program contains tests

![Figure 1. Soil properties obtained from tests (○: from boring A, A: from boring B).](image-url)
for physical properties of soils, oedometer tests and three shear tests: unconfined compression shear (UC) tests, shear box test (SBT) under constant volume and field vane shear test. Clay specimens of the SBT were consolidated in the shear box by \( a'_{0c} = 156.9 \text{ kPa} \) before shearing. Fig. 1 summarizes soil properties obtained from the tests. The soft Holocene clay deposit is laid down to 14m deep. The reclaimed work by drainage was carried out for two years from 1963 to 1965 and the water table was lowered from 1.5m above the ground surface to 0.5m under the ground surface. The sandy layer exists beneath the clay deposit. Since the water head in the sandy layer is 2.0m higher than the ground water level, the pore water pressure in the clay deposit does not result in the hydrostatic distribution. Therefore, attention has to be paid to evaluation of the effective overburden stress. Fig. 2 shows the current distribution of pore water pressure in which \( C \) stands for the computed values by 1-D F.E. consolidation analysis. The current distribution of pore water pressure has been confirmed to be already steady state through the F.E. consolidation analysis.

3 THEORETICAL STRENGTH

The elasto-plastic constitutive model proposed by Sekiguchi and Ohta (1977) can be regarded as an extension of Cam clay model for the natural clay deposit. This model can consider the anisotropical characteristics of clays arising from \( K_s \)-normally consolidation. Its yielding function \( f \) is expressed as,

\[
f = MDn + \frac{2n}{p_0} + Dq^+ - e'f = 0, \tag{1}
\]

and \( \eta^+ = \sqrt{\frac{3}{2}} \frac{\lambda_p}{p_0} \left( \frac{\lambda_y}{p_0} \right) \left( \frac{\lambda_y}{p_0} \right) M = 0.6 \sin \theta', \]

in which \( M \) is the critical state parameter, \( D \) is the coefficient of dilatancy introduced by Shibata (1963), \( e' \) is the plastic volumetric strain which has been chosen as the hardening parameter in this model, \( \eta^+ \) is generalized deviatoric stress ratio, \( q^+ \) is the deviatoric stress component, \( p' \) is the mean effective stress, \( \phi' \) is the effective internal friction angle and the subscript 0 denotes the reference value defined at the time of completion of consolidation. Accordingly, by applying associated flow rule, the stress and strain relation is given as,

\[
d\epsilon_f = b \frac{\partial \eta}{\partial \sigma} \quad \text{and} \quad d\epsilon_q = d\epsilon_f + d\epsilon_\sigma', \tag{2}
\]

in which \( b \) is the proportional constant which can be determined from the condition that the stress state stays on the yielding surface, \( e' \) is the effective stress component, \( \sigma_0 \) is the strain component and the superscripts \( \sigma \) and \( p \) denote the elastic and the plastic components, respectively. The failure condition and the undrained condition can be easily derived from Eq.(2) as:

- failure condition: \( \frac{3}{2n} \left( \frac{\lambda_y}{p_0} \right) \left( \frac{\lambda_y}{p_0} \right) M = 0, \tag{3} \)
- undrained condition: \( \frac{3}{2n} \left( \frac{\lambda_y}{p_0} \right) \left( \frac{\lambda_y}{p_0} \right) M = 0, \tag{4} \)

in which \( \Lambda \) is the irreversibility ratio defined as \( \Lambda = 1 - C_1/C_2 \). \( C_1 \) and \( C_2 \) are the compression and the swelling indices, respectively. Undrained strength can be derived by solving Eq.(3) and Eq.(4) simultaneously under a specific stress condition. For instance, the undrained strength under the plane strain condition has been derived as (Ohta et al., 1985),

\[
\sigma_0 = \frac{(1+2K_s)\Delta e_p(1-\Lambda)}{\Delta e_p(1-\Lambda)} \tag{5}
\]

and

\[
\rho = \frac{3n\Delta e_p(1-\Lambda)}{2M} \quad \text{and} \quad \eta_0 = \frac{3(1-K_s)}{1+2K_s} \tag{6}
\]

in which \( K_s \) is the coefficient of earth pressure at rest, \( \sigma_0 \) is the vertical preconsolidation stress and \( \theta \) is the rotation angle of principal axis of stress.

3.1 SBT STRENGTH

According to the plasticity analysis based on the slip line field method (Nishihara, 1986), directions of characteristic curves in the stress field satisfying Eq.(3) under plane strain condition are expressed as,

\[
\frac{\Delta e_\sigma}{\Delta e_p} = \tan \phi' \left( \frac{\lambda_y}{\lambda_p} \right) \quad \text{and} \quad \frac{\Delta e_q}{\Delta e_p} = \sin 2\theta' \left( \frac{\lambda_y}{\lambda_p} \right) \cos 2\theta' \ln \left( \frac{\lambda_p}{\Delta e_p} \right) \tag{6}
\]

On the other hand, since the associated flow rule is employed as in Eq.(2), directions of characteristic curves in the velocity field (the slip lines) correspond to those in the stress field. Then, the undrained
strength along a slip line can be derived from Eq. (5) and Eq. (6) as (Nishihara, 1986),
\[
S_i = c \cos \beta \sin \alpha \sinh \beta \sin \alpha + \cosh \beta \sin \alpha \cos \beta \sin \alpha \ln(1 + 2K_0 \exp(-\Lambda) / 3\sqrt{3}) \]
\[
\sigma_{\alpha\alpha} = 0 + 2K_0M \exp(-\Lambda) / 3\sqrt{3} \]
\[
eq \lambda_0 \]
(7)
and
\[
eq \lambda_0 \]
(8)
in which \( \alpha \) is the angle of slip line measured clockwise from the horizontal direction (x-axis). In the case of shear box test (SBT), since the slip line is fixed to develop in the horizontal direction, the undrained strength of shear box test is obtained by substituting \( \alpha = 0 \) to Eq. (7) as (Nishihara, 1986),
\[
\sigma_{\alpha\alpha} = 0 + 2K_0M \exp(-\Lambda) / 3\sqrt{3} \]
\[
eq \lambda_0 \]
(9)
Eq. (8) gives the theoretical SBT strength for NC clays.

3.2 FIELD VANE SHEAR STRENGTH

The field vane shear test consists of two kinds of shear mechanism. One is the shear in horizontal direction and the other in vertical direction. If undrained condition is satisfied during shearing, the field vane shear test can be modeled by two kinds of direct shear as shown in Fig. 3. Since the theoretical undrained strength for each direct shear has been derived from Eqs. (3) and (4) as shown in Fig. 4 and the contribution of both direct shears to the vane shear strength is usually evaluated as,
\[
S_{\text{vane}} = \frac{1}{1 + \frac{B}{H}} \left( S_0 + \frac{B}{3H} S_1 \right) \]
\[
\sigma_{\alpha\alpha} = 0 + 2K_0M \exp(-\Lambda) / 3\sqrt{3} \]
\[
eq \lambda_0 \]
(9)
thus, the vane shear strength can be obtained as,
\[
S_{\text{vane}} = \frac{1}{1 + \frac{B}{H}} \left( S_0 + \frac{B}{3H} S_1 \right) \]
\[
\sigma_{\alpha\alpha} = 0 + 2K_0M \exp(-\Lambda) / 3\sqrt{3} \]
\[
eq \lambda_0 \]
(10)
in which \( S_0 \) is the shear resistance of vane on the top and bottom surfaces (\( \tau_0 \) in Fig. 4), \( S_1 \) is the shear resistance of vane on the cylindrical surface (\( \tau_1 \) in Fig. 4), \( B \) and \( H \) are the width and the height of vane, respectively. Eq. (10) gives "ideal" vane strength that is possibly obtained under perfectly undrained condition for inviscid clay materials. Herein note that another theoretical expression of vane shear strength has been derived from the same constitutive model and the different interpretation for vane shear mechanism by Ohts et al. (1985).

3.3 UC STRENGTH

Fig. 5 gives the possible effective stress paths experienced by the soil specimen served to the unconfined compression (UC) shear test (Ohts et al., 1989). The effective stress change associated with the aging effect of normally consolidated clay is expected as shown by 1 → 2 in Fig. 5. The processes of sampling (stress release: 2 → 3), handling (disturbance: 3 → 4) and unconfined compression shear at the quick rate (strain rate: 4 → 5) would affect the UC strength as shown in Fig. 5. On the other hand, the effective stress paths of "ideal" sample and "perfect" sample are indicated as in the figure. The UC strength \( q_u \) depends on how much "undisturbed" sample experiences disturbance. But, the amount of effective residual stress surviving in the "undisturbed" sample cannot be determined from the theory. Only the undrained strength of "ideal"
or “perfect” sample can be theoretically derived (Ohta et al., 1989). Namely, by solving Eqs. (3) and (3.4) simultaneously under the axisymmetric stress condition that \( \sigma_1 = \sigma_0 \), \( \sigma_2 = \sigma_0 \), \( \sigma_3 = 0 \), and others are zero, the undrained strength of NC normally consolidated (NC) clays is expressed as,

\[
\theta_{\text{NC}} = \frac{S_{\text{NC}} \cdot \mathcal{K}}{2 \sigma_0} = \frac{1 + 2 \mathcal{K}_c}{6} \cdot 6 \mathcal{K}_c \cdot 6 \mathcal{K}_c \exp \left( -\frac{A + B}{\mathcal{M}} \right). \tag{11}
\]

For the NC aged clay, Eq. (11) can be extended by considering yielding shown by 1 – 2 in Fig. 5 as,

\[
\theta_{\text{NC, aged}} = \frac{\theta_{\text{NC}}}{2 \sigma_0} \left( \frac{\text{OCR} \cdot 1 + 2 \mathcal{K}_c}{1 + 2 \mathcal{K}_c} \right)^{1/4}, \tag{12}
\]

in which \( \mathcal{K}_c \) satisfies,

\[
\mathcal{M} = \frac{\text{OCR} \cdot 1 + 2 \mathcal{K}_c}{1 + 2 \mathcal{K}_c} + \frac{1 - K_0}{1 + 2 \mathcal{K}_c} \cdot \frac{1 - K_0}{1 + 2 \mathcal{K}_c} = 0,
\]

and OCR is the apparent over-consolidation ratio of the NC aged clay. And it should be noted that the undrained strength of “perfect” sample happens to theoretically correspond with that of “ideal” sample.

4 CORRECTION OF MEASURED DATA

The correction of the measured strength data has to be made for the effects that are not considered in the theoretical strength expressions.

First, let us consider the correction factors for the UC strength. As for the disturbance of sample, Ohta et al. (1989) have measured the residual effective stress in “undisturbed” sample. Their obtained result is shown in Fig. 6. The amount of effective residual stress has been expressed as the survival rate by the measured \( \theta_{\text{UC, aged}} \) over the theoretically estimated \( \theta_{\text{UC}} \). Thus they have derived the correction factor \( \mu_{\text{UC}} \) for the disturbance of sample as,

\[
\theta_{\text{UC}} = \frac{\theta_{\text{UC, aged}}}{2 \sigma_0} \left( \frac{\text{OCR} \cdot 1 + 2 \mathcal{K}_c}{1 + 2 \mathcal{K}_c} \right)^{1/4}, \tag{13}
\]

The correction factor \( \mu_{\text{UC}} \) for the absence of external confining pressure during shearing was interpreted to be equal to \( \alpha_0 \) proposed by Nakase et al. (1972). Likewise, the correction factor for the strain rate effect was expressed by \( \mu_{\text{SR}} \) proposed by Bjerrum (1972), since the time required to bring the clays up to failure in the UC tests and the vane tests is of the same order. And the correction factor \( \mu_{\text{SR}} \) for the stress release was found to be 1 because the theoretical UC strength of “perfect” sample corresponds to that of “ideal” sample. These correction factors are plotted against the plasticity index PI in Fig. 7 (Ohta et al., 1989). Therefore, the measured UC strength \( \theta_{\text{UC, measured}} \) for NC young clays can be corrected to the ideal UC strength comparable to Eq. (11) as,

\[
\theta_{\text{UC, measured}} = \frac{\theta_{\text{UC, measured}}}{2 \sigma_0} \left( \frac{\text{OCR} \cdot 1 + 2 \mathcal{K}_c}{1 + 2 \mathcal{K}_c} \right)^{1/4}, \tag{13}
\]

And, SBT strengths can be estimated from ideal UC strengths as,
in which \( \mu_{v} \) is obtained by dividing Eq.(8) by Eq.(11).

The vane shear tests at the site were performed under ten times as quick rate as usual (1 degree) in order to prevent drainage of pore water from boundary surfaces. Such obtained measured strengths have to be corrected to the vane strengths expected under the usual shear rate of 0.1 degree/sec and then they are divided by 1.15 which is the increase ratio of undrained strength to the strain rate shown by Skempton and Bishop (1954). On the other hand, the ideal vane strengths can be evaluated from the vane strengths obtained under the usual shear rate and Bjerrum's \( \mu_{v} \) in Fig.7. Therefore, the measured vane strengths obtained under the shear rate of 1 degree/sec are corrected to ideal vane strengths comparable to Eq.(10) as,

\[
\sigma_{mv} = \mu_{v} \cdot \sigma_{mv}^{o}, \quad \mu_{v} = \frac{\sigma_{mv}^{o}}{\sigma_{mv}} = 1.15
\]

in which \( \sigma_{mv}^{o} \) is obtained from comparing Eq.(10) with Eq.(8) and \( \mu_{v} \) is the correction factor for the extremely quick shear rate of 1 degree/sec.

5 CONVERSION TO SBT STRENGTH

The soft clay deposit at the site is over-consolidated as seen in Fig.1. But, since it is holocene clay, the aging effect would be negligible to consider. The strength of OC clays can be converted to that of NC clays by,

\[
\frac{\sigma_{nc}}{\sigma_{oc}} = \frac{\sigma_{oc}}{\sigma_{oc}^{o}} \cdot \frac{S_{oc}}{S_{oc}^{o}} \cdot \frac{E_{oc}}{E_{oc}^{o}}
\]

in which \( \sigma_{oc}^{o} \) is the (vertical) effective overburden stress, OCR is the over-consolidation ratio defined by \( \sigma_{oc}^{o}/\sigma_{oc}^{cr} \), \( \sigma_{oc}^{cr} \) is the swelling index obtained from the oedometer test, \( S_{oc} \) and \( S_{oc}^{o} \) are strengths of NC and OC clays, respectively.

Both vane shear and UC strengths shown in Fig.1 can be converted to SBT strengths as follows:

- for vane shear strength data:
  \[
  \left( \frac{S_{oc}}{\sigma_{oc}^{o}} \right) = \left( \frac{S_{o}}{\sigma_{oc}^{o}} \right) \frac{S_{oc}}{S_{oc}^{o}} \text{ by Eq.(16),}
  \]
  \[
  \text{and} \quad \left( \frac{S_{oc}}{\sigma_{oc}^{o}} \right) = \left( \frac{S_{oc}}{\sigma_{oc}^{o}} \right) \text{ by Eq.(15).}
  \]

- for UC strength data:
  \[
  \left( \frac{\sigma_{uc}}{\sigma_{uc}^{o}} \right) = \left( \frac{\sigma_{uc}}{\sigma_{uc}^{o}} \right) \text{ by Eq.(16),}
  \]
  \[
  \left( \frac{\sigma_{uc}}{\sigma_{uc}^{o}} \right) = \left( \frac{\sigma_{uc}}{\sigma_{uc}^{o}} \right) \text{ by Eq.(15).}
  \]

Fig.8 compares the converted strengths indicated by the solid line in the figure. Since the converted strengths well explain the SBT strengths, the validity of theoretical strengths and correction factors would be confirmed.

Figure 8. Comparison of converted SBT strengths.

Figure 9. Prediction of UC, vane and SBT strengths.

Figure 10. Ratio of vane strength to UC strength.
6 COMPUTED AND MEASURED STRENGTHS

Fig. 9 shows comparison of measured and computed strengths. Solid lines in the figure indicate the computed strengths divided by correction factors. Plots are the measured values converted to strengths for NC clays by using Eq.(16). The computed UC strengths in Fig. 9 (a) are obtained from Eq.(11) and Eq.(15) and the computed vane strengths in Fig. 9 (b) are given by Eq.(10) and Eq.(15). Herein, since empirical relations: 

\[ M = 1.75A \]  

(17)

Karaboe, 1975), 

\[ N = 0.44 + 0.64(P/100) \]  

(18)

Massarach, 1974) and 

\[ \sin\theta = 0.01 - 0.233\log(P) \]  

(19)

Kennedy, 1959) are employed to express the theoretical strengths as the function of only \( P \), the computed values might be no more than rough estimate.

Fig. 10 demonstrates ratio of vane strength to UC strength. The theoretical ratio of vane strength to UC strength can be expressed from Eq.(10), Eq.(11), Eq.(13) and Eq.(15) as:

\[
\frac{2N_{vane}}{N_{UC}} = \frac{2}{3} ~ \exp \left( \frac{A}{M} \right) \frac{\mu_s - \mu_o}{\mu_s - \mu_c} .
\]

(17)

The measured strengths converted to those for NC clays are plotted in the figure. The vane strength data shown in Fig. 1 are converted to those under the shear at usual strain rate. The measured data at other sites (Tanaka et al., 1954) are added in the figure. The computed values indicated in Fig. 10 have a band arising from \( \mu_o \) in Fig. 7. Empirical relations by Karaboe (1975) and Massarach (1974) are employed again to express Eq.(17) as the function of only \( P \). The authors feel that satisfactory agreement between the measured and the computed values would be obtained.

7 CONCLUDING REMARKS

This paper presents the evaluation of undrained strengths obtained from the site on a reclaimed soft clay ground from the viewpoint of the elasto-plastic constitutive model proposed by Sekiguchi and Ohta (1977). Theoretical expressions of undrained strengths are shown and the correction factors which enable the measured strengths to be converted to the ideal strengths comparable to the theoretical strengths are introduced. Throughout such evaluation of undrained strengths, attempts has been made to establish a reliable set of constitutive parameters required for the elasto-plastic model.

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Analysis of a survey on sand compressibility and its application in predicting drained friction angle of sand based on CPT

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ABSTRACT: In the present study, an empirical approach for evaluating sand compressibility based on the result of a survey is established. Expert opinions are collected and analyzed. Four factors, namely mineral characteristics, particle angularity, gradation, and particle roughness, are adopted for evaluating sand compressibility. Criteria for the evaluation are established based on the results of the survey. A methodology that can account for the sand compressibility in the determination of the drained friction angle is developed. The basic concept of this methodology is to aggregate existing methods that are known to be applicable to sands of low, medium, and high compressibility, respectively. An example is presented to illustrate the proposed methodology.

1. INTRODUCTION

It has been well recognized that the interpretation of CPT data for design soil parameters such as relative density and drained friction angle of sand is affected by the compressibility of the sand (Hald, et al., 1982; Robertson and Campanella, 1983; Juang et al., 1996). In a recent study, Huang and Juang (1996) have shown that different empirical methods for calculating the friction angle of sand may be "aggregated" based on the sand compressibility to improve reliability of the result. In their study, sand compressibility is characterized by friction ratio (R_F) obtained from CPT. However, the compressibility of sand is a rather complex behavior; even the term "sand compressibility" is not well defined. This paper presents the result of a survey that was used to obtain opinions on sand compressibility. The purpose of the survey was to establish a simple approach by which the compressibility of a sand deposit could be approximately determined.

Sand compressibility is known to be influenced primarily by mineral type, particle angularity, gradation, particle roughness, and stress history. Although important, the factor "stress history" was not included in the questionnaire for two reasons: 1) its precise effect on the sand compressibility is difficult to assess, and 2) only an indication of compressibility was sought in the framework envisioned. Thus, opinions were sought only on the effects of the four remaining factors, namely mineral characteristics, particle angularity, gradation, and particle roughness. The collected expert opinions were analyzed using fuzzy sets and an empirical approach to assess the compressibility of sand was established.

2. ANALYSIS OF THE SURVEY

2.1 Factors Affecting Sand Compressibility

In this survey, the respondents were asked to name possible factors that would affect sand compressibility. Besides the four factors mentioned in the questionnaire (mineral characteristics, particle angularity, gradation, and particle roughness), the following factors were also mentioned by the respondents (in no particular order): porosity, particle shape, macro structure, depositional environment, cementation, and particle size. Although the significance of these factors is well recognized, the factors such as "depositional environment", "macro structure", and "cementation" were excluded from the proposed framework because they are difficult to assess and we judged them to be relatively less
important than the four factors we adopted. The factor “particle shape” could be lumped in with the adopted factor “angularity”, and in fact, the terms were considered to be practically synonymous. Similarly, the factor “particle size” could be considered together with “gradation”, although its effect on the compressibility appears to be less obvious than that of the latter. The factor “porosity” is definitely important. However, we viewed it as being a combined effect of “gradation/size” and “angularity”, and we felt that inclusion of this factor in the proposed framework was not likely to yield a more accurate indication of the compressibility.

In summary, we considered the adopted factors to be adequate for providing an indication of sand compressibility, although the influence of other factors mentioned by the respondents should not be overlooked.

2.2 Linguistic Grades of Sand Compressibility

In the proposed framework, sand compressibility is assessed with linguistic grades such as “high”, “medium”, and “low”. Each respondent was asked to define these terms and the opinions were “averaged”. Figure 1 shows the result of this survey.

![Figure 1. Definition of the three linguistic grades](image)

These linguistic grades are defined over an arbitrarily selected independent variable called the “sand compressibility index” (SCI), as shown in Figure 1. The linguistic grades are “mapped” into a fuzzy set of SCI, not a single SCI value. Each fuzzy set is characterized by a membership function. For example, a sand judged to be “low” in compressibility can be associated with many SCI values, but each with a different degree of support (called membership grade). The most appropriate SCI value is 0 for a sand judged to be “low” in compressibility. In other words, the membership grade of an SCI value of 0 is the greatest in the fuzzy set that represents the linguistic grade of “low”. The membership grade decreases as the SCI value increases in this fuzzy set, and it reduces to 0 as the SCI value reaches 0.5. The fuzzy sets (and their membership functions) that represent the linguistic grades “medium” and “high” can be interpreted in the same way. In addition to the three main linguistic grades, borderline grades such as “low to medium” and “medium to high” (not shown in Figure 1) may be used. However, no new definition is needed, as the borderline grades may be taken as the average of the corresponding main grades.

2.3 Relative Importance of the Four Adopted Factors

In the survey, the respondents were asked to assess the relative importance of the four adopted factors, mineral characteristics, particle angularity, gradation, and particle roughness. Each opinion was expressed in a pairwise comparison matrix and a computer program called Fuzzy Decision Support (FDS), developed by Jiang, et al. (1995), was used to determine the weights of the four factors. Two steps were involved: 1) from each opinion matrix, the weights of the four factors were first determined; and 2) the weights obtained based on individual opinions were then aggregated. The results of the FDS analysis of the opinions are shown in Table 1. Note that the weights obtained from FDS are beta-M fuzzy sets (Jiang, et al., 1992). A beta-M fuzzy set is defined by four parameters, namely the lower bound, the upper bound, and the shape parameters α and β. However, because the fuzzy sets obtained are almost symmetric (graphs not shown, but evidenced by the fact α = β), and the ranges over which they are defined are narrow, they may be substituted with their modes, which are non-fuzzy, in practical applications.

2.4 Criteria to Evaluate Sand Compressibility

In the present study, the mineral characteristics are evaluated with two attributes, hardness and mica content. The respondents of the survey were asked to give a rating scale based on the hardness of the mineral, as well as the mica content in the sand, for the assessment of sand compressibility. The opinions were averaged and the result is shown in Table 2. To
assess the compressibility of sand, one enters the hardness value and the mica content into Table 2. The result of the evaluation is a linguistic grade that can be mapped into a fuzzy set based on Figure 1.

The respondents were also asked to give a rating scale each for assessing sand compressibility according to “particle angularity”, “gradation”, and “roughness”, respectively. The opinions given by the respondents were analyzed in a similar manner and the result is summarized in Table 3.

Table 1. Weights of the four factors

<table>
<thead>
<tr>
<th>Factors</th>
<th>Weight (fuzzy set)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>min</td>
</tr>
<tr>
<td>Mineral characteristics</td>
<td>0.54</td>
</tr>
<tr>
<td>Particle angularity</td>
<td>0.19</td>
</tr>
<tr>
<td>Particle gradation</td>
<td>0.12</td>
</tr>
<tr>
<td>Particle roughness</td>
<td>0.09</td>
</tr>
</tbody>
</table>

Table 2. Criteria for assessing compressibility based on hardness and mica content

<table>
<thead>
<tr>
<th>Hardness (H)</th>
<th>Mica Content, MC (%)</th>
<th>Low</th>
<th>Medium</th>
<th>High</th>
</tr>
</thead>
<tbody>
<tr>
<td>H ≥ 7</td>
<td>MC &lt; 5</td>
<td>Low to Medium</td>
<td>Medium</td>
<td>High</td>
</tr>
<tr>
<td>H = 6</td>
<td>5 ≤ MC &lt; 10</td>
<td>Low to Medium</td>
<td>Medium</td>
<td>High</td>
</tr>
<tr>
<td>H = 5</td>
<td>10 ≤ MC &lt; 15</td>
<td>Medium</td>
<td>Medium to High</td>
<td>High</td>
</tr>
<tr>
<td>H = 4</td>
<td>MC ≥ 15</td>
<td>Medium to High</td>
<td>High</td>
<td>High</td>
</tr>
<tr>
<td>H ≤ 3</td>
<td>High</td>
<td>High</td>
<td>High</td>
<td>High</td>
</tr>
</tbody>
</table>

Table 3. Criteria for assessing sand compressibility based on angularity, gradation, and roughness

<table>
<thead>
<tr>
<th>Criteria on Particle Angularity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angularity/Shape</td>
</tr>
<tr>
<td>rounded</td>
</tr>
<tr>
<td>rounded to angular</td>
</tr>
<tr>
<td>angular</td>
</tr>
<tr>
<td>Grade</td>
</tr>
<tr>
<td>low</td>
</tr>
<tr>
<td>medium</td>
</tr>
<tr>
<td>high</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Criteria on Particle Gradation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gradation</td>
</tr>
<tr>
<td>well graded</td>
</tr>
<tr>
<td>uniform</td>
</tr>
<tr>
<td>gap graded</td>
</tr>
<tr>
<td>Grade</td>
</tr>
<tr>
<td>low</td>
</tr>
<tr>
<td>medium</td>
</tr>
<tr>
<td>high</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Criteria on Particle Roughness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roughness</td>
</tr>
<tr>
<td>smooth</td>
</tr>
<tr>
<td>medium</td>
</tr>
<tr>
<td>rough</td>
</tr>
<tr>
<td>Grade</td>
</tr>
<tr>
<td>low</td>
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<tr>
<td>medium</td>
</tr>
<tr>
<td>high</td>
</tr>
</tbody>
</table>

4. DETERMINATION OF DRAINED FRICTION ANGLE OF "NC" SAND

As an example to illustrate possible applications of the survey results, attention is now directed to the estimation of design parameters such as the drained friction angle \( \phi' \) of a normally consolidated (NC) sand from CPT. The effect of sand compressibility on the \( \phi' \) derived from CPT data has been well-recognized (Robertson and Campanella, 1983). Huang and Juang (1996) have shown that the methods by Durgunoglu and Mitchell (1975), Robertson and Campanella (1983), and Chen and Juang (1996) may be used to determine \( \phi' \) from CPT for sand of low, medium, and high compressibility, respectively. The three methods for determining the \( \phi' \) of NC sands may be expressed as:

1275
\[
\tan \varphi = \frac{1}{C_T} \ln \left( \frac{q / \sigma'_{all}}{C_T} \right) 
\]

where the constants \( C_T \) and \( C_T^* \) are 7.629 and 0.194, respectively, in the Dargush and Mitchell method. In the Robertson and Cananella method, the constants are 6.820 and 0.266, respectively; in the Chen and Juang method, they are 6.079 and 0.340, respectively. Thus, the three methods can be described with the same equation. These methods are referred to herein as the LC method (for sand of low compressibility), the MC method (for sand of medium compressibility), and the HC method (for sand of high compressibility).

For a sand with a given compressibility, the \( \varphi' \) may be determined by aggregating the values obtained by the three methods (LC, MC, and HC). The aggregation operation is defined as follows:

\[
\varphi = \varphi_L W_L + \varphi_M W_M + \varphi_H W_H 
\]

where \( \varphi' \) is the drained friction angle obtained by this aggregation operation for the sand with the given compressibility, \( \varphi_L, \varphi_M, \) and \( \varphi_H \) are the angles obtained by the LC, MC, and HC methods, respectively; and \( W_L, W_M, \) and \( W_H \) are the weights that are assigned to the LC method (which produces \( \varphi_L \)), the MC method (which produces \( \varphi_M \)), and the HC method (which produces \( \varphi_H \)), respectively. The weights are determined based on a "distance" model defined as (Huang and Juang, 1996):

\[
d_k = \frac{1}{N+1} \sum_{i=1}^{N} \left[ (C_{\alpha_{all}} - K_{\alpha_{all}})^2 + (C_{\alpha_{all}} - K_{\alpha_{all}})^2 \right] 
\]

Here, the distance is a measure of similarity between the compressibility fuzzy set \( C_{\alpha} \) obtained from Equation 1 and each of the three compressibility grades defined in Figure 1. Thus, for example, \( d_k \) is the distance between \( C_{\alpha} \) and the fuzzy set that represents the linguistic grade "Low" defined in Figure 1 (note that the subscript \( K \) becomes 1 in this case). The terms \( C_{\alpha_{all}} \) and \( K_{\alpha_{all}} \) are the lower and upper bounds of the \( \alpha \)-level interval of the fuzzy set \( C_{\alpha} \), while the terms \( K_{\alpha_{all}} \) and \( K_{\alpha_{all}} \) are the lower and upper bounds of the \( \alpha \)-level interval of each of the fuzzy sets defined in Figure 1. \( N \) is the number of increments of \( \alpha \) value from 0 to 1. For example, if the increment of \( \alpha \) is taken as 0.2, then \( N \) is 5 and the denominator of the right-hand-side of Equation 4, \( N+1 \), is 6.

The basic concept of the similarity model is that a smaller distance value means a greater similarity, and thus, the result from the corresponding method (LC, MC, or HC) should be given a greater weight. Following this concept, the weight is defined as:

\[
w_k = \frac{1}{\sum_{i=1}^{N} d_i} 
\]

where, again, the subscript \( K \) refers to each of the three fuzzy sets defined in Figure 1. Once the weight is determined for each of the three methods (LC, MC, and HC), the \( \varphi' \) is then calculated by Equation 3. Figure 2 shows a flowchart of the methodology presented above.

5. NUMERICAL EXAMPLE

A sandy soil site consists of a top layer of fill of about 1.5 m thick and a deep layer of fine to medium sand. The groundwater is about 2 m below the ground surface. The natural water content of the saturated sand is about 23%, and the average saturated unit weight of the sand is about 19 kN/m³. The \( q_c \) values
from CPT at several depths, along with the effective stress at these depths, are shown in Table 4.

To apply the methodology presented above, the drained friction angles \( \phi' \) at various depths are first calculated based on the LC, MC, and HC methods. The results are also shown in Table 4. The next step involves an assessment of the compressibility of the sand. The evaluation based on Tables 2 and 3 results in linguistic grades of "low", "medium", and "medium" under the criteria of "mineral characteristics", "sphericity", and "gradation", respectively. No data is available on the particle roughness and the grade is assumed to be "medium" here. Using the definition of linguistic grades defined in Figure 1 and the weights shown in Table 1, the overall compressibility can be computed with Equation 1. For precise computation involving fuzzy sets, the FDS program may be used. However, non-fuzzy weights are used in the example presented herein for simplicity (note: this is acceptable as per the discussion in Section 2.3).

The weights for the four factors are 0.57, 0.20, 0.13, and 0.10, respectively (see Table 1). Thus, the overall compressibility (\( C_C \)) is 0.57 x "low" = 0.20 x "low" + 0.13 x "medium" + 0.10 x "medium". This is a simple computation involving constants and intervals. For simplicity, only 5 α-increments are taken in the determination of \( C_C \). The result is shown as a set of α-level intervals in Table 5. As an example to illustrate how the α-level intervals of \( C_C \) are obtained, consider the case where α = 0.2. At this level, the interval for "Low" is [0.05, 0.37], and in [0.19, 0.81] for "Medium". The corresponding α-level interval of \( C_C \) is calculated as:

\[
CA = 0.77 \times \text{"Low"} + 0.23 \times \text{"Medium"} = 0.77 \times [0.037] + 0.23 \times [0.19, 0.81] = [0.04, 0.47]
\]

The next step is to calculate \( d_k \) and then \( W_k \), using Equations 4 and 5. As an example, the distance between \( C_C \) and the fuzzy set that represents the linguistic grade "Low", denoting as \( d_k \), is calculated as follows:

\[
d_k = \left[ (0.05 - 0.62)^2 + (0.04 - 0.57)^2 \right]^{1/2} + \left[ (0.06 - 0.40)^2 + (0.47 - 0.37)^2 \right]^{1/2} + \left[ (0.09 - 0.25)^2 \right]^{1/2} + \left[ (0.12 - 0.12)^2 \right]^{1/2}/6 = 0.123
\]

Similarly, \( d_{0.4} = 0.404 \) and \( d_{0.9} = 0.919 \). Thus, the weight \( W_k \) is:

\[
W_k = (1/d_k) / \left[ (1/d_k) + (1/d_{0.4}) + (1/d_{0.9}) \right] = 0.696
\]

Similarly, \( W_{0.4} = 0.211 \) and \( W_{0.9} = 0.093 \). These weights are used to calculate \( \psi' \) and the result is shown in Table 4.

In summary, this example illustrates two important aspects of the proposed methodology. One is the evaluation of the compressibility of sand and the other is its potential application.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>( \sigma_c ) (MPa)</th>
<th>( \sigma_M ) (kPa)</th>
<th>( \phi' ) (deg)</th>
<th>( \phi^* ) (deg)</th>
<th>( \phi^* ) (deg)</th>
<th>( \psi' ) (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.0</td>
<td>5.03</td>
<td>52.0</td>
<td>38.2</td>
<td>39.5</td>
<td>41.0</td>
<td>38.7</td>
</tr>
<tr>
<td>4.5</td>
<td>4.07</td>
<td>66.7</td>
<td>34.4</td>
<td>37.6</td>
<td>39.1</td>
<td>36.9</td>
</tr>
<tr>
<td>6.0</td>
<td>5.03</td>
<td>81.4</td>
<td>36.4</td>
<td>37.5</td>
<td>39.0</td>
<td>36.8</td>
</tr>
<tr>
<td>7.5</td>
<td>6.64</td>
<td>96.1</td>
<td>36.7</td>
<td>37.8</td>
<td>39.3</td>
<td>37.1</td>
</tr>
<tr>
<td>9.0</td>
<td>8.86</td>
<td>110.0</td>
<td>37.1</td>
<td>38.2</td>
<td>39.7</td>
<td>37.6</td>
</tr>
</tbody>
</table>

Table 4. Determination of \( \psi' \) from CPT

<table>
<thead>
<tr>
<th>α</th>
<th>Main Linguistic Grade (Fig. 1)</th>
<th>Overall Compressibility (( C_C ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>[0.05, 0.37]</td>
<td>[0.05, 0.37]</td>
</tr>
<tr>
<td>0.2</td>
<td>[0.19, 0.81]</td>
<td>[0.63, 1.0]</td>
</tr>
<tr>
<td>0.4</td>
<td>[0.28, 0.72]</td>
<td>[0.70, 1.0]</td>
</tr>
<tr>
<td>0.6</td>
<td>[0.34, 0.66]</td>
<td>[0.75, 1.0]</td>
</tr>
<tr>
<td>1.0</td>
<td>[0.38, 0.42]</td>
<td>[0.80, 1.0]</td>
</tr>
</tbody>
</table>

Table 5. The α-level intervals of the main linguistic grades and the overall compressibility

6. CONCLUDING REMARKS

An attempt to establish an empirical approach for estimating sand compressibility was made possible.
with the opinions provided by knowledgeable geotechnical engineers in the survey reported herein. An analysis of these opinions has led to the development of the present empirical approach. This approach consists of two steps: (1) evaluate the compressibility of a sand based on the criteria listed in Tables 2 and 3, and (2) compute the overall compressibility using Equation 1. The result is expressed in the form of a fuzzy set. To interpret this result, the distance between this fuzzy set and a set of predefined linguistic grades (defined as fuzzy sets such as those shown in Figure 1) is calculated. The linguistic grade that yields the smallest distance value best describes the level of compressibility. In addition, the Cc fuzzy set and the similarity measures may be used directly in the determination of design parameters.

A simple methodology is presented that uses the obtained compressibility fuzzy set. This methodology may be applied to estimation of design parameters of sands such as relative density, friction angle, and constrained modulus, and to estimation of settlement, provided that empirical methods are available for such estimations for sands of low, medium, and high compressibility. In the present paper, an example is presented for estimation of drained friction angle of sand. While the presented methodology still needs to be validated with more quality data, it appears to be a promising approach where the compressibility of sand can be evaluated and accounted for directly in a geotechnical analysis.

ACKNOWLEDGMENTS

The impetus for this study was a joint research project entitled “A new approach for determining soil parameters from in-situ tests” and supported by the U.S. National Science Foundation through Grant No. MSS-9026252. This study could not have been completed without the thoughtful responses to our questionnaire provided by a group of friends and colleagues. This group includes (in alphabetical order): Dr. A.G. Franklin of the USAE Waterways Experiment Station, Profs. D. Frost, G. Rix, and P. Mayne of Georgia Institute of Technology, Prof. G. A. Leonards of Purdue University, Prof. D. Leiskehink of University of Delaware, Dr. T. Lunne of Norwegian Geotechnical Institute, Oslo, Norway, Prof. S. Marchetti of L’Aquila University, Italy, Dr. K. R. Massarch of Royal Institute of Technology, Stockholm, Sweden, Prof. I.K. Mitchell of Virginia Polytechnic Institute, Prof. E. Senneset of Technical University of Norway, Trondheim, Prof. T.L. Wolf of Michigan State University, Profs. R.D. Woods and R. Hryciw of University of Michigan. The authors express their sincere gratitude to all these experts.

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7. REFERENCES


Deformational characteristics of granite weathered residual deposit

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Jea-Young Park
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ABSTRACT: Both laboratory and field tests were performed at a granite weathered residual deposit to evaluate the deformational characteristics at strains from 10% to 1%. A special undisturbed sampling device was developed. Degree of weathering varied significantly and obtaining representative undisturbed samples was quite difficult in the site. The procedure for evaluating the deformational characteristics of the granite weathered residual deposit was proposed. In this procedure in situ average moduli determined by crosstape and PMT were effectively combined with laboratory normalized modulus reduction curve which was insensitive of weathering and sampling disturbance.

1 INTRODUCTION

In Korea, granite is occupied around one-third of the country, and granite weathered residual soils are widely distributed. Undisturbed sampling of this soil has been extremely difficult because of the sensitive pedostructure, and this difficulty have kept the researchers from investigating the deformational characteristics of these deposits in detail.

For the evaluation of deformational characteristics of soils, seismic and pressuremeter tests have been used in the field, and triaxial compression (static and/or cyclic), resonant column, and torsional shear tests have been mostly employed in the laboratory. However, different testing techniques are likely to have different ranges of reliable strain measurements, different applied stress level, and different loading frequencies, and the deformational characteristics of soils can be affected by these variables. Therefore, the effects of strain amplitude and loading frequency on the deformational characteristics need to be considered, and measured values should be effectively adjusted to the strain amplitude and frequency where the geo-system is working.

Field tests can be used to determine the modulus of soils under in-situ conditions, but the applicable strain ranges are limited. Laboratory tests can be used to determine the modulus over whole strain amplitude range (on the order of 10% to 1%), but it is very difficult to obtain the representative undisturbed samples for the site. In order to evaluate a reliable non-linear deformation characteristics of a site, moduli determined by field and laboratory tests need to be effectively combined.

Both field and laboratory tests were performed at a typical granite weathered residual deposit to evaluate the deformational characteristics over a whole strain range. Both undisturbed and reconstituted samples were obtained and test results were compared considering the effects of strain amplitude and loading frequency. A special undisturbed sampling device was developed. A procedure to evaluate the deformational behavior of granite weathered residual deposit was suggested based on this study.

2 TESTING EQUIPMENTS

2.1 Resonant Column and Torsional Shear Tests

The resonant column and torsional shear (RC/TS) testing equipment of a fixed-free type was used to investigate the effects of strain amplitude and loading frequency on the deformational characteristics of soils. The RC/TS apparatus used in this study has several advantages. Both RC and TS tests can be performed with the same setup simply by changing frequency of forcing function. Small torque can be applied using coil-magnet system without introducing mechanical compliances, thus results between RC and TS tests can be easily compared over a wide range of strains. The effect of loading frequency on modulus can be investigated.
effectively with simply changing loading frequency in TS test from 0.01Hz to 10Hz. Testing equipment and measurement techniques were explained in detail by Kim (1994).

2.2 Triaxial Compression Test with LDT
Young’s modulus was determined using triaxial compression test with local deformation transducer (LDT). Tatsuoka and his group have developed and widely used LDT (Tatsuoka and Shibuya, 1992). The average local strain was measured without bedding error using LDT and the axial load was measured using an external load cell. LDT is a thin phosphor-bronze strip where four strain gauges are attached close to its mid-height two on each side. A pair of LDTs are positioned between two small rings glued on the membrane diametrically opposite each other on the specimen and the axial deformation is determined by the bending of LDT sensed by strain gauges. In this study, LDT was used in the strain ranges from about 0.01% to 1%. 

2.3 Pressuremeter Test
The pressuremeter test (PMT) is a unique method for directly determining the variation in situ shear modulus with strain amplitude. TEXAM pre-bored PMT which is volume control monoeell type was used in this study.

Several studies (Robertson and Hughes 1986; Bellotti et al, 1989) have shown that the unload-reloading portion of PMT curve appears to be insensitive to the disturbance caused by installation and effective to determine the small strain shear modulus as low as about 10%. They suggested a interpretation technique of moduli from unload-reloading curve accounting for the relevant stress and strain levels acting around the PMT probe during the test. The average plane strain effective stress, $\sigma'_n$, and shear strain, $\gamma_{nr}$, existing in the plastic zone around the probe at the start of the unload-reload loop can be given by:

$$\sigma'_n = \sigma'_n + \alpha (\sigma'_n - \sigma'_n)$$  \hspace{1cm} (1)

$$\gamma_{nr} = \beta \gamma_c$$  \hspace{1cm} (2)

where, $\sigma'_n$, $\sigma'_n$ are effective horizontal stress and effective cavity stress at the start of unloading, respectively. $\Delta \gamma_c$ is shear strain increment during unload-reload cycle at the cavity wall, $\alpha$ and $\beta$ are reduction factors. In this study, $\alpha$ and $\beta$ were determined to be 0.18 and 0.5, respectively, for the granite weathered residual soil.

2.4 Crosshole Test
The most widely used seismic method to determine accurate and detailed shear wave velocity profile for engineering analyses is a crosshole test. In this method, the time for shear wave to travel between several points at the same depth within a soil mass is measured. With travel times, wave velocities are calculated after travel distances have been determined. Once the value of shear wave velocity, $V_s$, is determined, maximum shear modulus can be calculated from the following:

$$G_{max} = \rho V_s^2$$  \hspace{1cm} (3)

where, $\rho$ is the mass density of the soil

3 TEST SITE AND TESTING PROCEDURES
A test site was selected near Taejon, Korea, considering the geological condition. The site consisted of granite weathered residual soils. Weathering have progressed from the ground surface down, and the degree of weathering usually decreased with depth. The tested soil was classified as SP-SM. The depth of site concerned in this study was about up to 2m below the ground.

At first, crosshole test was performed, and the shear wave velocity and maximum shear modulus profiles of the site were determined. In this study, four boresholes with about 2 m spacing were constructed in a linear array as shown in Figure 1.

![Figure 1. Boreholes array for field tests](image)

The thin-walled stainless steel casings were installed with a good coupling between surrounding soils. The 3-component geophones used as receivers were tightly fixed to the casing with proper orientations using airbag system. Top of a rod which was embedded to the bottom soil at the same height as receivers was struck by hammer and the shear wave was generated. Test results were interpreted
using interval travel times between two receivers.

After the crossthroat tests were completed, stainless steel casings were pulled out and the PMT probe was carefully installed at a depth of 0.9m. The variation in in-situ modulus with strain amplitude of the soil was determined at strains larger than about 10^-5% using unloading and reloading curves.

A special undisturbed sampling device was constructed as shown in Figure 2. The sampler consisted of three parts: trimmer, specimen holder, and end adapter. The sampler was connected with a support frame which was anchored to the ground. The carefully curved undisturbed soil intruded into the sharp edge of the trimmer with self weight or an application of minimal pressure. The specimen holder was designed as split mold type with membrane attached tightly to the mold by vacuum during sampling. Once the soil intruded to the end of adapter, bottom end was cut carefully and the sealed sampler was brought to the lab.

The end adapter and trimmer were disconnected and both ends of specimen were cut off with flat perpendicular to the specimen axis. Removal of the ends had beneficial effect of eliminating disturbed part. Because the membrane has already been installed during sampling, the specimen can be tested right away. The size of the specimen was 7.1cm in diameter and 12cm long.

Figure 2. Undisturbed sampling device

Undisturbed samples were obtained at various locations around the tested area. As summarized in Table 1, the variation of void ratio of the specimens, which related to the degree of weathering, was high. and obtaining the representative undisturbed samples on this site was quite difficult.

Rereconstituted specimens of same density and water content were constructed by static compaction method in order to minimize the particle crushing during compaction. To evaluate the effect of sample disturbance on deformational characteristics, laboratory tests were performed on both undisturbed and reconstituted specimens.

RC/TTS tests were performed to investigate the effects of strain amplitude (on the order 10^-3 to 10^-1%) and loading frequency on the deformational characteristics. Each specimen was grouted to the top cap and bottom pedestal using hydrostone paste to achieve a fixed-free boundary condition. RC and TS tests were performed under isotropic confining pressures of 20, 40 and 80 kPa. At a confining pressure of 20 kPa, a series of high amplitude TS tests were performed at 0.5 Hz. At a strain amplitude of about 0.001%, loading frequencies were varied at 0.05, 0.1, 0.5, 1.0, 5, 10 Hz to investigate the frequency effect on modulus. After completion of high amplitude TS testing, high amplitude RC testing was performed with increasing strain amplitudes. Then, the confining pressures were increased to 40 kPa and 80 kPa, the same testing procedures were repeated.

The triaxial compression test with LDT was performed under isotropic confining pressures of 20, 40 and 80 kPa. A couple of hinged attachments were glued to the membrane, and were cured overnight before testing. The loads were applied to specimen with constant strain rate of 0.01%/min.

4 TEST RESULTS

4.1 Frequency effect on modulus

To investigate the effects of loading frequency on stiffness, the variation in normalized modulus with loading frequency was plotted in Figure 3. The shear modulus was normalized by the value at a loading frequency of 0.5 Hz. The modulus increases almost linearly as a function of the logarithm of loading frequency.

To quantify the frequency effect, least-squares curve fitting was performed and fitting curve yields a frequency effect of 8.3% and 6.3% per log cycle of loading frequency for undisturbed and reconstituted specimens, respectively.

Typical variations in shear modulus with strain amplitude determined by RC and TS tests are shown in Figure 4(a). Modulus determined by RC test were
Figure 3. Frequency effect on modulus

- KD #1, FE = 8.3%
- RM #1, FE = 4.5%

\[ G/G_0(5.5 Hz) = 1 + FE \cdot LOG(Freq) \]

loading frequency, Hz

Figure 4. Typical variation in shear modulus with strain amplitude as determined by RC and TS tests

(a) Before considering frequency effect

- RC (93 Hz)
- TS (0.5 Hz)

shearing strain, \( \gamma \) %

(b) Adjusted to loading frequency of 0.5 Hz

- RC
- TS

shear modulus, G, MPa

Figure 5. Variation in G with \( \gamma \) for various soils

The normalized modulus reduction curve with strain amplitude (\( G/G_0 vs \log \gamma \)) has been widely used to evaluate a non-linear site response in soil dynamics and earthquake engineering. Figure 6 shows the normalized modulus reduction curves of various specimens. Test results existed within a
narrow band over a whole strain amplitude independent of the effects of void ratio and sample disturbance in a practical sense.

![Graph](image)

Figure 6. Variations in $G_{m,n}$ with $\gamma$ for undisturbed and reconstituted specimens

4.4 Evaluation of Deformational Characteristics Based on Field and Laboratory Tests

The variation in maximum shear modulus with depth are obtained by crosshole test and summarized in Table 2. The crosshole test provided a reliable average $G_{m,n}$ profile under in-situ condition, but it was determined at strains below elastic threshold, not over a whole strain range.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Maximum shear modulus, $G_{m,n}$, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>case 1</td>
</tr>
<tr>
<td>0.3</td>
<td>72.77</td>
</tr>
<tr>
<td>0.6</td>
<td>75.68</td>
</tr>
<tr>
<td>0.9</td>
<td>78.72</td>
</tr>
<tr>
<td>1.2</td>
<td>95.08</td>
</tr>
</tbody>
</table>

![Graph](image)

Figure 7. Typical unloading-reloading curve in PMT

Table 2. Variation in $G_{m,n}$ with depth

$\sigma_{m,n}$ is in situ mean effective stress, $\hat{\sigma}_{m,n}$ is average mean octahedral effective stress around the cavity, and $n$ is the modulus exponent. The value of $n$ was directly determined as 0.49 by RC tests using undisturbed specimens.

$\sigma_{m,n}$ is in situ mean effective stress, $\hat{\sigma}_{m,n}$ is average mean octahedral effective stress around the cavity, and $n$ is the modulus exponent. The value of $n$ was directly determined as 0.49 by RC tests using undisturbed specimens.

$G_{m,n}^C = G_{m,n} \left( \frac{\sigma_{m,n}}{\sigma_{m,n}} \right)^n$ (5)

Figure 8. Variations in moduli with strain amplitude determined by laboratory and field tests

Moduli determined by crosshole and PMT tests which provides the average property of the site, matched well with RC results for UD sample #3 at the corresponding strain amplitude. It indicates two interesting points: 1) moduli determined by crosshole, PMT, and RC tests are equivalent if the
effects of strain amplitude, loading frequency, and confinement are considered in the comparison. 2) Degree of weathering of the site is similar to that of UD sample #3 and it seems difficult to obtain the representative UD samples on the granite weathered residual deposit where degree of weathering significantly varies. For the reliable evaluation of deformation characteristics of this site, in situ average modulus determined by either crosshole or PMT test needs to be effectively combined with laboratory modulus reduction curve which is insensitive to the weathering and sample disturbance. A procedure for the evaluation of deformation characteristics of the granite weathered residual deposit based on laboratory and field tests is suggested as follow:

1. In the field, either determine Gla, profile from crosshole test or determine G - ω relations (usually ω > 10−4 s) at various depths using PMT.
2. Obtain the undisturbed samples for lab tests, and determine the range of in situ void ratio and water content.
3. Determine G/Gla, - ω relation over the whole strain range of interest and frequency effect on modulus from RCT/TS and LDT triaxial compression tests using either undisturbed or reconstituted specimens prepared at the similar density and water content with in situ using static compaction method.
4. Adjust in situ moduli to the values of design frequency considering the frequency effect.
5. Determine the representative G - ω relations of the site at various depths combining the adjusted moduli from field tests with G/Gla, - ω relation from laboratory tests.

Figure 9 shows the representative G - ω relations of the site at various depths determined by the proposed method.

5. CONCLUSIONS

The following conclusions can be drawn from this study.

1. Moduli determined by crosshole and PMT were equivalent with RC test results if the effects of strain amplitude, loading frequency, and confinement are considered in the comparison.
2. Degree of weathering varied significantly in the granite weathered residual deposit and obtaining representative undisturbed samples of the site was quite difficult.
3. The procedure for evaluating the deformation characteristics of the granite weathered residual deposit was proposed. In this procedure in situ average moduli determined by crosshole and PMT were effectively combined with laboratory normalized modulus reduction curve which was insensitive to the weathering and sample disturbance.

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Estimating method of shearing resistance parameters of compacted gravelly soils in field by laboratory tests

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ABSTRACT: It is not easy to know shearing resistance parameters $c_f$ and $\phi_f$ of compacted gravelly soils in field because of difficulty in getting the undisturbed samples for the laboratory test and also in performing the in situ shear test. In order to estimate shearing resistance parameters $c_f$ and $\phi_f$ of compacted gravelly soils in field from shear test of compacted soils in laboratory, the author carried out shear tests ($d=150\text{mm}, l=400\text{mm}$) with change of the gradation and the maximum grain size of compacted soils, and compared the test results with those of direct shear test in field using a $300\text{mm} \times 300\text{mm} \times 180\text{mm}$ shear box. From a series of laboratory tests using Eliminated Gradation Samples (cut off larger particles simply) and Substitute Gradation Samples (replaced larger gravel-particles by smaller gravel-particles of the same weight), it is known that in situ strength parameters $c_f$ and $\phi_f$ of gravelly soils can be estimated reasonably by laboratory tests with a few kinds of Eliminated Gradation Samples.

1 INTRODUCTION

When high embankment with slope is constructed, it is common to carry out stability analysis using shearing resistance parameters of compacted soils. However, if the compacted soil is a gravelly one with large gravels, it is difficult to pick the undisturbed samples from compacted ground for shear tests in laboratory, and it is not easy to carry out large shear tests in field. Moreover, even though laboratory shear tests is carried out using disturbed samples from field, it is difficult to find the correct mechanical property of gravelly soils by soil test apparatus with the current standard size. Therefore, large scale triaxial compressive test apparatus (e.g., $d=300\text{mm}, l=600\text{mm}$) are sometimes applied, but there is a problem with labor and cost. Hence, estimating method of shearing resistance parameter in field is desired with more simplicity and availability (LRR 1990).

For test of gravelly soils, it is required to make laboratory shear test apparatus large-sized. But, because the change of test apparatus size has a limitation, the test has to be done with soils of which maximum grain size is matched for particular test apparatus.

About how adjustment of test gradation and test method have to be, and how we can estimate the shearing property in field from results in laboratory test, an experimental result estimating mechanical property of compacted soils in field by the standard direct shear test ($d=60\text{mm}, l=30\text{mm}$) have been reported (Moshands & Nafalsa 1989). However, there is difficulty in getting the available data from other researches for gravelly soils the author has dealt with.

The author has investigated an estimating method of shearing resistance parameters $c_f$ and $\phi_f$ obtained from in situ direct shear tests using the parameters obtained from laboratory direct shear test (a triaxial apparatus using shear box of almost same size as the one of CBR test under the JIS) for the compacted ground soils which include gravelly particles of $63-70\text{mm}$ size.

2 GROUND SOILS

The physical properties of gravelly soil used for this study are given in table 1. Also, the grain size distribution curves of ground soils with one of Eliminated Gradation Samples and the one of Substitute Gradation Samples used for laboratory tests are shown in Figure 1 and Figure 2 respectively. According to the soil group symbol by triangular coordinates of Japanese Unified Soil Classification System, compacted soils in field is gravel soils with fines (GF) as illustrated in Figure 3. In detail, the name of soil group is classified as clayey gravel (GC) (JSSMFE 1990). Gravel is subrounded to rounded, with maximum size of $63.5\text{mm}$. Sand and fines are also contained considerably.

3 IN SITU DIRECT SHEAR TEST

In situ direct shear test is carried out simultaneously at the same site, using the $30\text{cm} \times 30\text{cm} \times 18\text{cm}$ shear box as shown in Figures 4 and 5. In the in situ test, in order to prevent the rotating and tension moment at the shear plane, shearing load is loaded at an oblique angle. Hence, when arranging the test results, the
Table 1 Physical properties of test soils

<table>
<thead>
<tr>
<th>Name of Soil Group</th>
<th>Clayey gravel (GGA)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density of soil particles</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Maximum particle size mm</td>
</tr>
<tr>
<td></td>
<td>Gravel</td>
</tr>
<tr>
<td></td>
<td>Sand</td>
</tr>
<tr>
<td></td>
<td>Fine</td>
</tr>
<tr>
<td></td>
<td>Plastic Liquid</td>
</tr>
<tr>
<td></td>
<td>Liquid Limit</td>
</tr>
<tr>
<td></td>
<td>Plastic Index</td>
</tr>
</tbody>
</table>

![Figure 1 Grain size distribution curve of ground soils and Eliminated Gradation Samples](image1)

Figure 1 Grain size distribution curve of ground soils and Eliminated Gradation Samples

![Figure 2 Grain size distribution curve of ground soils and Substitute Gradation Samples](image2)

Figure 2 Grain size distribution curve of ground soils and Substitute Gradation Samples

Shearing load is determined with the following revised horizontal load: \( P_{\theta} = P \cos \theta - \mu \cdot P \cdot \sin \theta \). Where \( P \): the oblique load, \( \mu \): the friction coefficient between flange of shear box and base plate, \( \theta \): the angle between the horizontal plane and the oblique loading direction (Takehashi et al. 1976). A general arrangement of the apparatus is shown in Figure 5. The relationship between the vertical stress \( \sigma_v \) and the shear strength \( \tau_v \) is shown in Figure 6. In the figure, the shear strength in No.3 is larger than the one of No.2. The value of shear strength \( \tau_v \) in No.3 comes out smaller. It can be inferred that the density of test soils in No.3 is smaller than the others and causes this result as shown in Figure 6. Therefore, when the strength parameters \( \phi_v \) and \( c_v \) are calculated using Coulomb's failure criterion, the other 3 tests

![Figure 3 Grading single of test soils by Japanese United Soil Classification System](image3)

Figure 3 Grading single of test soils by Japanese United Soil Classification System

![Figure 4 Shear box and oblique load](image4)

Figure 4 Shear box and oblique load

![Figure 5 Test apparatus of the in situ direct shear test](image5)

Figure 5 Test apparatus of the in situ direct shear test

![Figure 6 Test result of the in situ direct shear test](image6)

Figure 6 Test result of the in situ direct shear test
of compacted test soils, installation conditions of the mould containing compacted soil in the shear box, and the mechanism of laboratory direct shear test apparatus.

5 LABORATORY DIRECT SHEAR TEST

5.1 Test soils for laboratory test
Test soils for laboratory test are picked from the same site as in situ direct shear tests. Grain size distribution curve of ground soils and test soils for laboratory tests are shown in Figures 1 and 2. Soils picked in field are adjusted to 2 kinds for laboratory direct shear tests: Eliminated Gradation Samples in which the maximum particle size is limited within 25.4, 19.1, 9.52, 4.76, 2.0mm by cutting off large particles simply from ones with maximum particle size up to 63.5 mm. Substitute Gradation Samples of which maximum particle size is 25.4, 19.1, 9.52, 4.76mm by replacing larger gravel-particles by smaller gravel-particles of the same weight. Figure 3 shows locations of these test soils in triangular coordinates of Japanese Unified Soil Classification System. In Eliminated Gradation Samples, as the maximum particle size becomes smaller, gravel soils with fines (GF) become classified as sand soils with fines (SF). On the other hand, in Substitute Gradation Samples, though the maximum diameters are changed, locations in the triangular coordinates are constant. The goal of this study is to estimate failure resistance parameters by reproducing the condition of compacted ground soils in field and laboratory. Thus, in Eliminated Gradation Samples, assuming that gravel particles with exceeded maximum particle size applied for adjustment of Eliminated Gradation Samples are cut off from compacted ground soils, density of finer fraction than the maximum particle size is considered as an initial condition of test soils for laboratory test. In Substitute Gradation Samples, supposing that the content of gravel particles with diameter larger than 2.0mm control shear strength, density of which the degree of compaction (compacted dry density / maximum dry density for each test soil) is same as the one in field, is considered as an initial condition of test soils. Table 2 shows the maximum particle size and maximum dry density of compacted soils used in field and laboratory tests, and initial conditions for

![Diagram of laboratory direct shear apparatus](image-url)

Figure 7: Diagram of laboratory direct shear apparatus

except No.3 are adopted. As a result, the value of φ₀ =43.1, c₀=60.8kPa are obtained.

4 TRIAL APPARATUS FOR LABORATORY DIRECT SHEAR TEST

Laboratory direct shear test apparatus used in this study is designed to be sheared with mould to just the state that test soil is compacted in mould. Inner diameter of the mould is 150mm and the height is 100mm. So because of the largeness, it is possible to examine gravelly soils with quite big particle size. Figures 7(a),(b),(c) illustrate the mould for preparation
each test soil. In addition, water content in each test soil is adjusted so that water content of matrix in test soils becomes equal to the one in compacted ground soils.

5.2 Comparison test

Figures 8 and 9 give the results of compaction tests obtained using so-called standard Proctor (compactive effort £c=54.9 kN/m²) for each test soil. Test conditions are as follows: rammer weight=2.5kg, drop height of rammer=30cm, base diameter of rammer=5cm, mould diameter=15cm, mould volume=2250cm³, number of compactive layer=3, number of rammer drop per each layer=55. In order to obtain the maximum dry density of compacted ground soils with the maximum particle size of 63.5 mm, density adjustment by method of Walther-Holz (Walther & Holz, 1951) is done based on the result of compaction tests for Eliminated Gradation Samples of which the maximum particle size is 25.4mm in laboratory. As a result, the maximum dry density of compacted ground soils (ρ_max) is obtained 2024 g/cm³. When the degree of compaction in field (C_\text{f}}/C_{\text{max}}) is estimating using this value, the mean value from No.1 to No.4 given in table 2 is 95.1%. In Eliminated Gradation Samples, smaller the maximum particle size is, smaller the maximum dry density tends to be, as shown in table 2. The same phenomenon can be found in Substitute Gradation Samples. These phenomena are caused by the fact that a ratio of void among particles increases due to reduction of the maximum particle size.

5.3 Shear tests

Shear tests are performed under a fixed vertical load. Those tests are carried out under vertical stresses of 100, 200, 300, 400 kPa to the test soils adjusted in order to have the same gradation and density. From the results, cohesion c_d and angle of internal friction \phi_d are obtained. To prepare the test soils with designated gradation and density, test soils is made by compacting each layer with rammer after applying soils equivalent to each 1/3 of mould height given in Figure 7(a). And after setting test soils in a test apparatus as shown in Figures 7(b, c, d) and compressing with each vertical stress for 30 minutes, the upper and the lower parts of shear box are spaced about 0.2mm and sheared. The test procedure is done after confirming that compression has been almost finished. Shearing speed is 0.50 mm/min so that the same strain speed as in in-situ direct shear tests.

6. COMPARISON OF FIELD TEST RESULTS WITH LABORATORY TEST RESULTS

In this test, shear stress \tau does not have a peak in most of the case. In each case, test is stopped at shear displacement of 18.6mm, 15% of decrease ratio of shear area in test soil, and the shear stress \tau at this point is considered as shear strength \tau_c. The relationship between horizontal displacement and shear stress, and the relationship between horizontal displacement and vertical displacement for Eliminated Gradation Samples and Substitute Gradation Samples are given in Figures 10 and 11. Despite field test results indicate volume expansion at shearing when vertical stress is 100 kPa, in laboratory tests of Eliminated Gradation Samples, all the shearing is done with volume contraction as shown in Figure 10. On the other hand, in laboratory tests of Substitute Gradation Samples, all test results show a property of volume expansion as shown in Figure 11. The property of volume expansion at shearing in field tests considered as a precompression effect by compaction. However, the property of volume expansion in Substitute Gradation Samples is considered as a property at shearing by engaging with gravel particles, rather than by precompression effect due to compaction. The relationship between the maximum particle size \Delta_{\text{max}} of test soils and angle of internal friction \phi_d is shown in Figure 12, and the relationship between the maximum particle size \Delta_{\text{max}} of test soils and cohesion c_d is shown in Figure 13.

From three Figures, there is a straight relationship
between the maximum particle size $D_{max}$ on one logarithm and angle of internal friction $\phi$ if the maximum particle size is smaller than 9.52mm, 1/15 of the diameter of test soils, and it is possible to estimate the angle of internal friction in the field at extension of the straight relationship. Moreover, there is also a straight relationship between the maximum particle size $D_{max}$ on one logarithm and cohesion $c$, if the maximum particle size is smaller than 9.52mm, 1/15 of the diameter of test soils, and it is also possible to estimate the cohesion of test soils in the field at extension of the straight relationship. This is because increase in engagement of coarse particles induces the increase of $\phi$, and decrease in cohesive action of fines induces the decrease of $c$, as the maximum particle size $D_{max}$ increases. With its diameter of test soils (150mm) in this laboratory shear test, only the ones with the maximum particle size up to 9.52mm can obtain the available test results, and if the maximum particle size is bigger than 9.52mm, result shows excessive restriction effect to gravel particles. These results are considered as a reason why the shear resistance parameters of test soils with their maximum particle size exceeded more than 9.52mm are not on the straight relationship, as explained above. While the angle
of internal friction $\phi$ ~43.1° and cohesion $c_v$ ~60.8 kPa obtained in direct shear tests in field, $\phi$ ~40.5° and $c_v$ ~44.1 kPa are estimated from test results in laboratory.

On the other hand, in case of Substitute Gradation Samples (test soils with same gravel content), the values of angle of internal friction both in laboratory tests and in field tests are almost same, between the maximum particle size $D_{max}$ on one logarithmic and angle of internal friction $\phi$ with the maximum diameter less than 9.52mm, 1/15 of a diameter of test soils (150mm). While angle of internal friction is 43.1° in field, mean angle of internal friction of the maximum particle size smaller than 9.52mm in laboratory is 42.5°. The cause why $\phi$ in laboratory tests becomes larger than the one in field tests when the maximum particle size is bigger than 9.52mm, is because excessive restriction effect of gravel grains by shear box of 150mm size induces excessive shear strength, as well as in Eliminated Gradation Samples. Moreover, the apparent mean value of cohesion in laboratory tests with the maximum particle size smaller than 9.52mm is 95 kPa, and this value is bigger than cohesion 60.8 kPa in field tests. Shear test by Substitute Gradation Samples is the method in which the main object is to obtain $\phi$ in field from laboratory shear test, and thereby is considered as $\phi$ material and tests must have $c_v$ ~0 as side side.

Furthermore, the maximum particle size $D_{max}$ has to be smaller than 1/15 of the diameter of test soils in laboratory shear tests in order to estimate the results in field correctly from laboratory tests. According to this relationship, the maximum particle size has to be smaller than 20mm for application of shear box of which shear area is 300mm×300mm in field test. It may be the ideal to apply the shear box with inner diameter (or side length) larger than 1m for ground soils of which the maximum particle size is 63.5mm. However, since it is not easy to prepare a vertical load for direct shear test in which the shear area is 300mm×300mm in field test, assuming that approximately 20% coarse gravel (19-75 mm) in gravelly soils does not have much effect, the values by this test apparatus utilized from the past are regarded as shearing strength parameters $c_v$ and $\phi$ in field (Ishii et al. 1991). Considering the possibility that results of in situ direct shear test are kind of bigger, $c_v$ and $\phi$ in field, by the estimation in laboratory test of Eliminated Gradation Samples in this study, are considered as the appropriate estimation values.

7 CONCLUSION

The author has experienced the study on estimating method of shearing strength parameters $c_v$ and $\phi$ of compacted gravelly soils in field (gravel about 50%, sand about 25%, fines about 20%) including large gravel particle (the maximum size of gravel particle is approximately 60-75mm) obtained by in situ direct shear test (shear area: 300mm×300mm) from laboratory tests. Using trial apparatus of laboratory direct shear test for this study purpose, the author performed a number of experiments by Eliminated Gradation Samples and Substitute Gradation Samples based on gradation of ground soils, and analyzed the relationship with shearing strength parameters $c_v$ and $\phi$ obtained from field tests. As a result, if the laboratory tests are carried out considering Substitute Gradation Samples having the same degree of compaction (D value) as in field, the angle of internal friction $\phi$ can be estimated almost appropriate, but estimation of cohesion $c_v$ tends to be excessive and inappropriate. In this case, the maximum particle size of Substitute Gradation Samples is smaller than 1/15 of the diameter of test soils for laboratory direct shear test. This method can be available when estimating only $\phi$, assuming $c_v$ ~0 to $\phi$ material. On the other hand, shear test result using Eliminated Gradation Samples under the same condition as in field is available to estimate appropriately shearing resistance parameters $c_v$ and $\phi$ in field. In this case, the Eliminated Gradation Sample is varied in the maximum particle size of 2-3 types smaller than 1/15 of the diameter of test soils (150mm). For a long time, estimating mechanical property in field using Substitute Gradation Samples has been generally supposed. However, considering estimating method by Eliminated Graduation Samples can obtain better results and is simple gradation adjustment of test soils in laboratory, the estimating method of shearing resistance parameters of compacted gravelly soils in field from laboratory tests using Eliminated Gradation Samples can be considered as a reasonable estimating method for engineering.

8 ACKNOWLEDGE

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Shear modulus and damping of a stiff marine clay from in situ and laboratory tests

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ABSTRACT: This paper describes and compares the results of in situ and laboratory investigations which were carried out in order to determine the soil profile of the Saline di Augusta site (also Saline site), with special attention being paid to the variation of shear modulus and damping ratio with depth. The soil deposits at this site mainly consist of a medium stiff, overconsolidated (OCR ~ 2.0 to 6.0), Pleistocene marine clay with low to medium PI, overlaid by a 15 m thick Holocene silty clay stratum. The Saline site is located near the city of Augusta on the east coast of Sicily which is one of the most seismically active areas of Italy.

1. INTRODUCTION

The site under consideration is located on the east coast of Sicily, which is one of the most seismically active areas of Italy. Since 1169, the city of Augusta has been struck by three disastrous earthquakes with an MKS intensity from IX to XI (Postischl 1985).

The 13th December 1990 earthquake, with moderate magnitude (M=5.4) and where the epicentre was located 10 km offshore from the city of Augusta (Amato et al. 1991), caused 19 victims and severe damages to buildings and infrastructures with an MKS intensity equal to VII (De Riceis et al. 1991). In particular, reinforced concrete buildings with 228 flats located in the Saline site, were damaged by this earthquake of moderate magnitude. In order to study the possible amplification phenomena of the Saline site, a comprehensive laboratory and in situ investigation has been carried out to obtain a soil profile with special attention being paid to the variation of the shear modulus (G) and damping ratio (D) with depth. The results of such an investigation have partially been published (Maugeri et al. 1994, Frenna and Maugeri 1995, Maugeri and Frenna 1995, Costelli et al. 1995, Cavallaro and Maugeri 1995, Lo Presti et al. 1996, 1997a, 1997b, Cavallaro 1997). This paper tries to summarise this information in a comprehensive way in order to provide a case record of site characterization for seismic response analysis.

2. GEOLOGY

The deposits under consideration lie over the Ille-Ile Plateau which consists of cretaceous-miocene limestone with intercalations of vulcanite (Carbone 1985). To the East, the Ille-Ile Plateau borders the Ille-Ile-Malteese escarpment, to the West the Scieli-Ragusa faults to the North-West the Scordia-Lentini fault and to the South the graben of the Channel of Sicily (Ghisetti and Vezzani 1980). The seismic activity of the region is mainly linked to the tectonic stresses which develop at the border between the Eurasian and African plates. The Ille-Ile Plateau represents a contact area between these two plates.

The thickness of the deposits varies from between 50 to 300 m (Carbone 1985). The upper part of these deposits (less than 15 m) consists of recent and actual alluvial soils which overhang a
Pleistocene marine clay which is locally called Augusta blue clay.

The Holocene deposits mainly consist of alternating layers of grey silty clay and silty sands. Stiff, CaCO₃ cemented, layers of sand were found at depths of between 10 and 15 m.

The Pleistocene deposits mainly consist of a medium stiff, overconsolidated (OCR = 2.0 to 6.0), marine clay with low to medium PI.

3. INVESTIGATION PROGRAM AND BASIC SOIL PROPERTIES

The investigated area has plane dimensions of 4100 m² and a maximum depth of 80 m. The area pertaining to the investigation program and the locations of the boreholes and field tests are shown in Fig. 1.

Undisturbed samples were retrieved by means of an Osterberg (1973) piston sampler and by means of an 86 mm Shelby tube sampler.

The general characteristics and index properties of the Augusta clay are shown, as a function of depth, in Fig. 2. The clay fraction (CF) is prevalently in the range of between 60 - 70%. This percentage decreases to 30 - 40% at certain depths where a sand fraction of 15 - 30% and a gravel fraction of 2 - 10% are observed. The silt fraction is in the range of about 25 - 40%. The values of the natural moisture content wₑ prevalently range from between 30 and 35%. Characteristics values for the Atterberg limits are: wₑ=60 - 65% and wₑ=22 - 26%, with a plasticity index of PI=30 - 40%. The data shown in Fig. 2 clearly indicate a very high degree of homogeneity of the deposit. This indication is also confirmed by comparing the penetration resistance qₑ from mechanical cone penetration tests (CPT) performed at different locations over the investigated area (Fig. 3). The variation of qₑ with depth clearly shows the existence of layers with very different mechanical characteristics. The upper Holocene silty clay has very poor mechanical characteristics with qₑ of about 0.3 to 0.6 MPa. The lower Pleistocene clay has qₑ values of about 2 to 4 MPa. A transition zone with interbedded stiff sand layers (qₑ=15 to 35 MPa) exists between these two strata at depths of about 10 and 15 m. The soil deposits can be classified as inorganic clay of high plasticity.
4. STRESS HISTORY

The preconsolidation pressure $\sigma'_p$ and the overconsolidation ratio OCR = $\sigma'_p / \sigma_{oc}$ were evaluated from the 24th compression curves of 7 incremental loading (IL) oedometer tests. Moreover, 10 Marchetti's flat dilatometer tests (DMT) were used to assess OCR and the coefficient of earth pressure at rest $K_s$, following the procedure suggested by Marchetti (1980).

The information obtained from laboratory and in situ tests is summarised in Fig. 4.

For depths of up to 10 m, DMT results show an OCR from 1 to 3 ($K_s = 0.3 - 1.0$), which means that the upper Holocene deposit is normally consolidated or lightly over consolidated. For the lower Pleistocene clay, the OCR values obtained from DMT range from 5 to 7 ($K_s = 1.0 - 1.5$) with an average value equal to 6 up to about 30 m depth.

The OCR values inferred from oedometer tests are lower than those obtained from in situ tests. One possible explanation for these differences could be that lower values of the preconsolidation pressure $\sigma'_p$ are obtained in the laboratory because of sample disturbance.

5. SHEAR MODULUS AND DAMPING RATIO

The small strain shear modulus $G_s$ was determined from an in situ Cross Hole (CH) test. The equivalent shear modulus ($G_{eq}$) and damping ratio $D$ were determined in the laboratory by means of a Refined Column test (RCT) and cyclic loading torsional shear tests (CLTST) performed on Shelby tube specimens by means of a Refined Column/Torsional shear apparatus (Lo Presti et al. 1993). Monotonic loading torsional shear tests (MLTST) were also performed on Shelby tube specimens using the same apparatus, obtaining the measurement of the secant shear modulus $G_{ssec}$. RCTs were also performed at times of Bergamo on Osterberg specimens. Moreover it was attempted to assess $G_s$ by means of empirical correlations, based either on penetration test results or on laboratory test results (Jamekloowski et al. 1995).

Small strain shear modulus $G_s$: in situ vs. laboratory measurements

Figure 5 shows the values of $G_s$ obtained in situ from a CH test and those measured in the laboratory.
from RCT, CLTST and MLTST performed on undisturbed specimens which were isotropically reconsolidated to the best estimate of the in situ mean effective stress. The $G_s$ values are plotted in Fig. 5 against depth. In the case of laboratory tests, the $G_s$ values are determined at shear strain levels of less than 0.001 %. It is possible to see that quite a good agreement exists between the laboratory and in situ test results. A better agreement between the laboratory and in situ test results is observed in the case of the Osterberg samples than for the Shelby tube samples. On average the ratio of $G_s$ (Lab) to $G_s$ (Field) was equal to 0.86 with a Standard Deviation of 0.095.

![Figure 5. $G_s$ from laboratory and in situ tests.](image)

The $G_s$ values for the upper Holocene silty clay steadily increase from 20 to 80 MPa with depth. In the transition zone, where stiff sand layers exist, $G_s$ increases up to 110 MPa. It is worthy to note that CHI tests results can ignore the existence of soft layers interbedded between stiff layers because of the occurrence of reflection phenomena. In the lower Pleistocene blue clay $G_s$ values are in the interval of 80 to 120 kPa, slowly increasing with depth.

Shear modulus and damping ratio from laboratory tests

The laboratory test conditions and the obtained small strain shear modulus $G_s$ are listed in Table 1. In some cases, the same specimen was first subjected to MLTST, then to CLTST, after a rest period of 24 hrs with opened drainage, and eventually, after another 24 hrs of rest with opened drainage, to a RCT. The size and shape of the specimens are also indicated in Table 1.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$G_s$ (kPa)</th>
<th>$e$</th>
<th>MLTST $G_s$ (1)</th>
<th>CLTST $G_s$ (2)</th>
<th>RCT $G_s$ (3)</th>
<th>Speed of sound (m/sec)</th>
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* U= Undrained, $G_s$ (1) from MLTST, $G_s$ (2) from CLTST, $G_s$ (3) from RCT. H= Hollow cylindrical specimen ($R_e$ = 25 mm, $R_i$ = 15 mm, $h$=100 mm). S= Solid cylindrical specimen ($R_e$ = 25 mm, $h$=100 mm).

The $G_s$ values, reported in Table 1, indicate a moderate but measurable influence of strain rate and type of loading even at very small strains where the soil behaviour is supposed to be elastic. In order to appreciate the rate effect on $G_s$, it is worthwhile to remember that the equivalent shear strain rate ($\dot{\gamma} = 240 \cdot \frac{1}{\sqrt{G_s}}$) experienced by the specimens during RCT can be three orders of magnitude greater than those adopted during CLTST. The effects of the rate and loading conditions become more and more relevant with an increase of the shear strain level, as can be seen in Fig. 6 where the $G-\dot{\gamma}$ curves obtained from MLTST, CLTST and RCT are compared. It is possible to notice that the lowest decay of $G$ with $\dot{\gamma}$ is observed in RCT, while the maximum decay occurs during MLTST. In order to quantify the rate effect, the coefficient of strain rate $\alpha(\dot{\gamma})$ was computed from CLTST and RCT, as shown in Fig. 6.

The experimentally determined $\alpha(\dot{\gamma})$ values are summarised in Table 7. The data available in

![Figure 6. $G-\dot{\gamma}$ curves from MLTST, CLTST and RCT tests.](image)
literature are also shown in this figure. These data were obtained by means of undrained compression tests or CLTST-RCT performed on both remoulded and undisturbed clays. The data, on the whole, clearly show that $\alpha(\gamma)$ increases with the plasticity index and, for a given soil, with the shear strain level. Considering that, at small strains, $\alpha(\gamma)$ is not greater than 5%, it is possible to conclude that very similar values of $\alpha_0$ can be obtained from different kinds of test such as MLTST, CLTST, RCT.

Figure 7. (The coefficient of strain rate $\alpha(\gamma)$).

A comparison between the damping ratio values obtained from RCT and those obtained from CLTST is shown in Fig. 8. It is possible to see that the damping ratio from CLTST, at very small strains, is equal to about 2%. Greater values of $\beta$ are obtained from RCT for the whole investigated strain interval. After a correction of the experimental data for equipment-generated damping ($D_{eq}$), according to Stokoe et al. (1995), still large differences remain between the CLTST and RCT results.

Figure 8. Damping ratio from CLTST and RCT tests.

Figure 9. Damping ratio from CLTST and RCT tests.

A comprehensive comparison between the results of the CLTST and RCT is shown in Fig. 9. Considering that the influence of $N$ on $D$ has been found to be negligible, in the case of clayey soils for strain levels of less than 0.1%, it is supposed that RCT provide larger values of $D$ than CLTST because of the rate (frequency) effect, in agreement with data shown by Shibuya et al. (1995) and Tatsuoka et al. (1995). According to these researchers the nature of soil damping in soils can be linked to the following phenomena:

- Non-linearity which governs the so-called hysteretic damping controlled by the current shear strain level. This kind of material damping is absent or negligible at very small strains.
- Viscosity of the soil skeleton (creep) which is relevant at very small strain rates.
- Viscosity of the pore fluid which is relevant at very high frequencies.

Soil damping, at very small strains, is mainly due to the viscosity of the soil skeleton or of the pore fluid, depending on the strain rates or frequencies. Moreover, according to Tatsuoka and Kohata (1995) and Tatsuoka et al. (1995) a partial drainage condition can provide very high values of the damping ratio. Shibuya et al. (1995) indicate that, for a given strain level, the damping ratio of cohesive soils increases when the loading frequency is smaller than 0.1 Hz (because of the creep effects), is more or less constant for loading frequencies between 0.1 and 10 Hz (non linearity is dominant) and increases for frequencies greater than 10 Hz (because of pore fluid viscosity).

Fig. 10 shows the damping ratio of Vallericca clay (Italy) vs. frequency, for a strain level of 0.01% and consolidation pressure between 100 and 800 kPa. This data was obtained by d'Ocaofrio 1996. The considered soil is a stiff, highly overconsolidated, Pleistocene marine clay with a PI of about 26%. In the same figure the data of Augusta clay, obtained in this research and those of Pisa clay
obtained by Lo Presti et al. (1997b) have been reported. The trend of the whole data is in good agreement with the findings of Shibuya et al. (1995).

\[
Q_1 = \frac{530}{T_1^{0.83}} - 1.8 \frac{K}{\gamma} (\sigma' / \gamma) - 2.7 \gamma \frac{\gamma_0}{\gamma}
\]

where: \( \gamma \) and \( \gamma_0 \) are expressed in the same unit; \( \gamma_0 \) = 1 bar is a reference pressure; \( \gamma_0 \) and \( K_0 \) are respectively the unit weight and the coefficient of earth pressure at rest, as inferred from DMT results according to Marchetti (1980).

c) Mayne and Rix (1993)

\[
G_v = 400 \frac{Q_{D'}^{0.17}}{\alpha_{30}^{0.17}}
\]

where: \( G_v \) and \( Q_{D'} \) are both expressed in [kPa] and \( \alpha_{30} \) is the void ratio. Eq. (4) is applicable to clay deposits only.

d) Jamiolkowski et al. (1995)

\[
G_v = 600 \frac{Q_{D'}}{\alpha_{30}^{0.17}} P_k^{0.3}
\]

where: \( \alpha_{30} = (\sigma'_{c} + 2 \sigma_{I}) / 3; \) \( P_k = 1 \) bar is a reference pressure; \( G_v \), \( \sigma'_{c} \), and \( P_k \) are expressed in the same unit. The values for parameters which appear in eq. (5) are equal to the average values that result from laboratory tests performed on quaternary Italian clays and reconstituted sands. A similar equation was proposed by Shibuya and Tanaka (1996) for Holocene clay deposits.

Eqs. (3) to (5) incorporate a term which expresses the void ratio; the coefficient of earth pressure at rest usually appear in eqs. (3) and (5). However only eq. (3) tries to obtain all the input data from the DMT results.

The \( G_v \) values obtained with the methods above indicated are plotted against depth in Fig. 11. The method by Jamiolkowski et al. (1995) was applied considering a given profile of void ratio and \( K_0 \). The coefficient of earth pressure at rest was inferred from DMT. The method by Mayne and Rix (1993) was applied only to the cohesive strata, disregarding the high values of \( Q_{D'} \) encountered in the sandy layers that exist between 10 and 15 m. The \( N_{20} \) values, experimentally determined during SPT, did not show any important variation in the transition zone at depths between 10 and 15 m, where thin layers of stiff sand exist. Standard Penetration Tests were performed at intervals from 1.5 to 3.0 m. The quite large interval used could explain why the thin sand layers were not detected. Consequently, the obtained \( G_v \) values, in the transition zone, resulted to be quite low. The DMT material index indicated the presence of sandy layers at depth of about 10 and 15.5 m and at the same depth the dilatometer
modulus greatly increased (Maugeri et al. 1994). However, the method by Hysing (1990) was not capable of detecting these stiff strata as can be seen in Fig. 11.

![Graph](image)

Figure 11. $G_s$ from different empirical correlations.

All the considered methods show very different $G_s$ values of the Holocene and Pleistocene clay strata. On the whole, eq. (4) and (5) seems to provide the most accurate trend of $G_s$ with depth, as can be seen comparing Fig. 5 with Fig. 11. It is worthwhile to point out that the considered equations overestimate $G_s$ for depths greater than 20 m.

6. CONCLUDING REMARKS

A site characterisation for seismic response analysis has been presented in this paper. On the basis of the data shown it is possible to draw the following conclusions:

- the small strain shear modulus obtained from MLTST, CLTST and RCT is moderately influenced by the strain rate.
- it was attempted to quantify the effect of strain rate on the shear modulus by means of the parameter $a(\gamma)$ which increases with the plasticity index and, for a given soil, with the shear strain level.
- the small strain shear modulus measured in the laboratory is on average 0.86 of that measured in situ by means of CH tests. A better agreement between laboratory and in situ measurements is observed in the case of stiff cohesive samples retrieved with an Osterberg sampler.
- the damping ratio, measured in the laboratory, resulted to be mainly influenced by rate effects.
- empirical correlations between the small strain shear modulus and penetration test results were used to infer $G_s$ from SPT, CPT and DMT. The values of $G_s$ were compared to those measured in a CH test. This comparison clearly indicates that a certain relationship exists between $G_s$ and the penetration test results, which would encourage one to establish empirical correlations for a specific site. This approach makes it possible to consider the spatial variability of soil properties in a very cost effective way.
- relationships like those proposed by Jamiolkowski et al. (1995) or Shibuya and Tsuchida (1996) seems to be capable of predicting $G_s$ profile with depth in both Holocene and Pleistocene deposits. The accuracy of these relationships could obviously be improved if the parameters which appear in the equations were experimentally determined in the laboratory for a specific site.

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Which soil-investigation methods can we trust: Field tests or laboratory tests?

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ABSTRACT: Many laboratory and field testing methods have been developed for soil investigation. However, choosing the right soil parameters for analysis and design is one of the most difficult tasks for geotechnical engineers. In this paper, the question is examined and discussed for the case of shallow footings on sand. Static load tests on shallow footings with different size were performed. Soil properties, back-calculated from the static load tests using the FLAC program, are compared with those determined from field and laboratory tests.

1. INTRODUCTION

Choosing the right soil parameters for analysis and design is a very difficult task for geotechnical engineers. Laboratory testing is normally used as a basis for evaluation of soil properties. However, soil samples taken for laboratory tests will always be disturbed during the sampling process, especially in the case of non-cohesive soil. Many field test methods have been developed, which provide with fairly reliable information about the relative stiffness of the soil layers and their thickness. However, the evaluation of the soil properties from the field test results have been based on the correlation between the field tests and the laboratory tests. Which ones can we trust, the laboratory tests or the field tests? In this paper, the question is examined and discussed for the case of shallow footings on sand.

A comprehensive study of piled footings with settlement-reducing piles in sand were performed by Phung (1993). The experiential part of the study consisted of three test series (T1, T2, T3) on single piles, pile groups and piled footings. In each series a static load test on a shallow footing (called C) was also included. The soil was well prepared and well controlled by different laboratory and field testing methods. The effective angle of internal friction $\phi'$ and the modulus of elasticity $E$ of soil, interpreted from the results of different test methods were then compared with each other and with those back calculated from the static load tests using FLAC, a two-dimensional explicit finite difference code.

2. SOIL INVESTIGATION

The test site is an abandoned sand pit at Gråbo, 35 km north-east of Gothenburg, Sweden. The site is a glaciofluvial delta created during two phases of the latest deglaciation. The delta is built up mainly of sand, but lenses of clay and coarser gravel are also found. The test pit was about 3 m by 3 m at the base and 3 m deep with a wall inclination of 2:1. The excavation was filled with sand which was compacted 0.2 m thick layers. The soil properties were determined by means of a volumeter and different field methods: cone penetration test (CPT), dilatometer test (DMT) and pressuremeter test (PMT). In the first test series, the relative density of sand was about 38%, in the second 67%, and in the third 62%.

2.1 Laboratory tests

The grain size distribution investigated by sieve analyses showed an average coefficient of uniformity, $C_u = 3.4$, and a mean grain diameter, $d_3 = 0.34$ mm. The specific gravity $\rho$, determined by the pycnometer method had a mean value of 26.8 kN/m$^3$.

<table>
<thead>
<tr>
<th>Table 1. Average basic soil properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk density (kN/m$^3$)</td>
</tr>
<tr>
<td>Dry density (kN/m$^3$)</td>
</tr>
<tr>
<td>Water content (%)</td>
</tr>
<tr>
<td>Void ratio e</td>
</tr>
</tbody>
</table>
The constrained modulus of compressibility \( M \) can be determined from oedometer tests and expressed as

\[
M = m \alpha_0 (\sigma'_v / \alpha_0)^\beta
\]

(1)

where, \( m \) = modulus number, \( \beta \) = stress exponent, \( \sigma'_v \) = effective vertical stress, and \( \alpha_0 \) = reference stress = 100 kPa. For Gröbo sand, Liedberg (1991) found that \( m = 64.2 \ e^{-0.02 \ e} \), and \( \beta = 0.41 \), where \( e \) = initial void ratio.

From triaxial tests, Young's modulus \( E_c \) can be expressed as:

\[
E_c = k_c \sigma_0 (\sigma'_v / \sigma_0)^{n_c}
\]

(2)

where, \( k_c \) = modulus number, \( n_c \) = stress exponent, \( \sigma'_v \) = minor principal stress. For Gröbo sand, it was found that \( k_c = 16.9 \ e^{-0.04} \), and \( n_c = 0.58 \) (Liedberg, 1991).

Poisson's ratio \( \nu \) can be calculated from the equation \( M = E(1-\nu)(1+\nu)/(1-2\nu) \). Assuming that \( \nu \) varies linearly with the void ratio, it was found that \( \nu = (e - 0.163)/1.518 \) for Gröbo sand. This equation is valid for a maximum void ratio of 0.92.

Ekstrom (1989) performed a number of direct shear tests and triaxial tests to evaluate the angle of internal friction \( \phi' \). From the results of direct shear tests, the \( \phi' \) value varied from 37° to 46°, when the dry density of soil changed from 15.5 to 18.1 kN/m³. From the results of triaxial tests on sand with density 16.0 to 18.3 kN/m³, the \( \phi' \) value changed from 40° to 47°, which is rather similar to the results obtained by Liedberg (1991). The following correlation was obtained by Ekstrom using regression analysis

\[
tan\phi' = 1/(1.47e + 0.237)
\]

(3)

2.2. Field tests

Totally 22 CPTs, 14 DMTs and 3 PMTs were carried out in all the three test series. In this section only a short descriptions of the tests performed is presented. For more detailed, see Phang (1993).

Pressuremeter Tests

PMTs were performed before each test series using a Marshland mini-pressuremeter. The pressuremeter was installed by direct insertion of the probe inside a driven slotted tube, which is a rather common way of installation in Sweden. Cone Penetration Tests

Two types of the CPT equipment were used in the test field: mechanical and electrical. The electrical one, in which the point resistance \( q_p \) and the skin friction \( f_s \) were measured separately, was used before the second and the third test series. The mechanical penetrometer, a Swedish Geotech, in which only the total penetration force is measured, was used before and after every test series. Since the skin friction of the rod of the mechanical penetrometer was extremely small for the shallow depths in question, the total penetration force could be assumed equal to the point resistance. Comparison with the results obtained from the electrical penetrometer indicated that this assumption is reasonable.

Dilatometer Tests

DMTs were carried out after each soil preparation, i.e. before each test series, as a control of the homogeneity of the sand. The tests were also performed inside and outside the pile groups after each test series so that the soil properties before and after the tests could be compared. A total of about 160 measurements were performed in all the three test series. The main results are given in Section 4, where the interpreted soil properties obtained from the different soil investigation methods are discussed.

3. STATIC LOAD TESTS

The footings were made of pre-fabricated reinforced concrete and were absolutely rigid. Footings with three different sizes were used: 0.46 m by 0.46 m in plan and 0.3 m in thickness (test T1C); 0.63 m by 0.63 m by 0.35 m (T2C); and 0.8 m by 0.8 m by 0.4 m (T3C). Four 0.4m-diameter end-tapered piles E400, which were placed at a depth of 10 m below the ground surface, were used as reaction piles. To avoid the effects of the reaction piles on the test results, casings down to a depth of 8 m were used to isolate the pile steel rods from the soil. As main reaction beam and secondary beams, an H-beam HE6600 with a height of 0.6 m and a length of 6.5 m and two double U-320 beams with a length of 2m were used.

Displacements of the upper surface of the footing and also of the ground surface were measured by means of electric resistance transducers, with a stroke of 25, 50, or 100 mm. The displacements were measured against two reference beams, which were protected against wind and sunshine by tarpaulins. The reference beams were founded on footings that
rested 0.3 m under the soil surface. Levelling showed that movements of the reference beams were negligible. A hydraulic jack with a maximum capacity of 600 kN was used. Total loads applied in all the tests were monitored by an independent electronic load cell. The acquisition system consisted of a data-logger, a personal computer and a printer. The logger had up to 200 channels, of which at most 40 channels were used in the tests. The logger was set up and controlled by the PC computer. The set-up routine is a program written in the logger’s control language which defines the type and timing of logging and control routines, as well as parameters for all channels. In the tests, all channels were measured at 15 second intervals.

All the tests were carried out using the same standard procedure as the quick maintained load test. In this method of testing, the applied load is increased every fifteen minutes by a constant amount, approximately 5% of the estimated ultimate load. Settlement gauges are read 0.5, 1, 2, 4, 8 and 15 minutes after application of a new load increment has been started. A new load increment is applied immediately after the 15 minutes reading. Using the computer-based data acquisition system, however, all data from the settlement transducers and from the pile load transducers were automatically collected every fifteen seconds. And each load step was only maintained for eight minutes. An advantage of using electrical displacement transducers and a data logger is that the creep behaviour can be interpreted with great accuracy.

4. COMPARISONS OF TEST RESULTS

All the field test results will be compared with the laboratory results in the form of graphs, in which both the test data and the best fit lines are drawn. The laboratory tests used in the comparison were performed by Ekström (1989) and Liedberg (1991). The dry density for each field test data (each point) in Figures 1 and 2 was interpolated from the volumeter tests at the same depth as the field test. Of course, this way of comparison will result in a quite large scatter, but it is a good way of evaluating the soil properties calculated from field test results.

4.1 Angle of Internal Friction

In the CPTs, the \( \phi' \) value can be calculated from the correlation proposed by Robertson & Campanella (1983), which is converted to Equation (4) by the author.

\[
\tan \phi' = 0.38 \log(\frac{q}{\sigma'_v}) + 0.10
\]  (4)

The \( \phi' \) values estimated according to Eq (4) are compared with those obtained from the triaxial tests, made by Ekström (1989) and Liedberg (1991) in Figure 1. In this figure the dry densities corresponding to the CPT-based \( \phi' \) values are interpolated from the volumeter tests for the corresponding depths. The figure indicates that \( \phi' \) estimated from CPT are in rather good agreement with the results of triaxial tests.

![Fig. 1 Internal friction angle \( \phi' \) estimated from CPT - Comparison with laboratory tests.](image)

There are four different procedures for estimation of the effective friction angle from DMT. In the first procedure, for soils with the dilatometer material index \( I_d > 1.2 \), the \( \phi' \) value can be estimated from \( 
\end{equation}

\[
\phi' = 25 + 0.19 (R_d - R_{c1} - 100)^{0.8}
\]  (5)

where, \( R_d = \frac{500}{E_d + 1.5 \sigma'_v - 500} \) if \( \sigma'_v > 50 \) or \( E_d < \sigma'_v < 500 \), \( R_{c1} = E_d \sigma'_v \) if \( \sigma'_v < 50 \) or \( E_d \sigma'_v > 500 \).

Schnettmann (1982) suggested a second method of calculating a plane strain, effective friction angle \( \phi'_{c1} \) from DMT combined with measurement of the penetration thrust. Marchetti (1985) proposed a third method, a procedure for evaluating axi-symmetrical effective friction angle \( \phi'_{c2} \) - so-called CPT-linked method which is similar to Schnettmann’s method, but based on the \( q_0 \) value from the CPT instead of the DMT penetration thrust. In a fourth procedure, the calculated friction ane may be normalised to a reference stress values (Schnettmann, 1988).
In this paper, the first method and the third method are used. The calculations were made at every 0.2 m depth and the results are also compared with the triaxial tests. Figures 2a and b). In these figures, the dry densities are also interpolated from the volumeter tests for the corresponding soil layers. The $\phi'$ value estimated according to Marchetti & Craps (1981) is found to be too small in comparison with the triaxial tests. The CPT-linked method of Marchetti (1985), gives a better agreement with the results of the laboratory tests. However, in a number of cases unreasonable values, both of $K_s$ and $\phi'$, were obtained. This can be explained by the fact that, since sand deposits normally have a high variability, the use of even closely spaced soundings involves an inherent error in obtaining "matching" $q_s$ and $K_s$ values in this method” (Schuernmann, 1988).

In Figure 2b, the values out of practical limits, e.g., corresponding to negative $K_s$, values, were disregarded.

The large scatters in Figures 2a and 2b may be explained by the unmatchability of the calculated $\phi'$ values and the dry densities interpolated from the volumeter tests. Besides, in the figures, the stress dependence of $\phi'$ values is not taken into account.

In the PMT test the angle of internal friction $\phi'$ can be obtained, e.g., according to Eq. (6), proposed by Centre d’Etudes Menard (Bagnellin et al., 1978)

$$\phi' = 25 \cdot 2 \cdot \frac{p_{s}^{2}}{p_{v}^{2}}$$

(6)

where, $p_{s} = p - p_{r}$, net limit pressure, in bar

The $\phi'$ values estimated from Eq. (6) are, however, too small to be true. In Table 2 the average $\phi'$ values estimated from different methods are compared. The comparison shows that $\phi'$ values obtained from CPT are in best agreement with the laboratory test results.

### Table 2. Average angle of internal friction $\phi'$

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>37.7</td>
<td>36.7</td>
<td>35.1</td>
<td>36.3</td>
<td>23.0</td>
</tr>
<tr>
<td>T2</td>
<td>41.8</td>
<td>33.8</td>
<td>39.6</td>
<td>42.0</td>
<td>28.4</td>
</tr>
<tr>
<td>T3</td>
<td>41.0</td>
<td>32.8</td>
<td>40.5</td>
<td>41.6</td>
<td>26.8</td>
</tr>
</tbody>
</table>

4.2 Deformation characteristics

The deformation moduli are estimated from the different field test methods and compared with the laboratory test results in the same way as regarding the angle of internal friction.

In the dilatometer test, the constrained tangent modulus $M_{\text{const}}$ can be determined as:

$$M_{\text{const}} = R_l \cdot E_0$$

(7)

where, $E_0$ = dilatometer modulus; $R_l$ = correlation factor (Schuernmann, 1988). The modulus $M_{\text{const}}$ estimated from DMT according to Eq (7) is compared with the laboratory test results in Fig. 3. The figure indicates that the $M_{\text{const}}$ is often larger than the $M$ value derived from oedometer tests. Similar results can be seen in Ekström’s study.

In the pressuremeter test, the pressuremeter modulus can be estimated as (Hansbo & Pranmorg, 1990):

$$E_{pm} = 2(1 + v) \left( \sum_{i} \left( \Delta P_i \right) \frac{v_i}{\Delta P_i} \right) \Delta P / \Delta P'$$

(8)
where, $V_e$ = initial volume of measuring cell; $V_i$ = effective volume of tube; $V_c = V_e + \Delta V/2$ = mean value of $V$ over the straight line portion of the pressure meter curve, corrected for the stiffness of the slotted tube; $\rho V' = \text{inclination of the straight line portion of the corrected pressure meter curve}; V = \text{Poisson's ratio of the soil}.$

\[ M = \alpha q. \]  

(9)

where, $\alpha = \text{correlation factor. There are different suggestions regarding the } \alpha \text{ value. Vesic (1970), for example, proposed } \alpha = 2(1 + f_0^2), \text{ where } f_0 \text{ is the relative density of soil. However, comparison between the } M \text{ values from the laboratory tests and the } q_e \text{ values in Fig. 5a indicates that the correlation factor } \alpha \text{ strongly depends on the } q_e \text{ value, and the equation of the best fit line is:}

\[ \alpha = 5.909 q_e^{0.333} \]  

(10)

This may be because $q_e$ was quite low in this study: 0.5 to 6.0 MPa. With $q_e > 2 \text{ MPa the mean value of } \alpha \text{ is less than 4. Using Eqs. (9) and (10), the modulus } M \text{ is estimated and compared with the laboratory tests in Fig. 5b.}

\[ M \text{ is estimated as:} \]

\[ M = \alpha q. \]  

(9)

where, $\alpha = \text{correlation factor. There are different suggestions regarding the } \alpha \text{ value. Vesic (1970), for example, proposed } \alpha = 2(1 + f_0^2), \text{ where } f_0 \text{ is the relative density of soil. However, comparison between the } M \text{ values from the laboratory tests and the } q_e \text{ values in Fig. 5a indicates that the correlation factor } \alpha \text{ strongly depends on the } q_e \text{ value, and the equation of the best fit line is:}

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As regards the CPT test, it is commonly suggested that the modulus $M$ is estimated as:

\[ M = \alpha q. \]  

(9)

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As regards the CPT test, it is commonly suggested that the modulus $M$ is estimated as:
Table 3 Comparison of deformation modulus estimated from different methods, MPa

<table>
<thead>
<tr>
<th>Method</th>
<th>$M_{sys}$</th>
<th>$M_{est}$</th>
<th>$M_{calc}$</th>
<th>$E_0$</th>
<th>$E_r$</th>
<th>$E_m$</th>
<th>$E_{10}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>6.0</td>
<td>8.5</td>
<td>5.5</td>
<td>3.4</td>
<td>2.1</td>
<td>18.0</td>
<td>6.5</td>
</tr>
<tr>
<td>T2</td>
<td>10.4</td>
<td>29.8</td>
<td>9.3</td>
<td>7.9</td>
<td>7.1</td>
<td>30.4</td>
<td>18.7</td>
</tr>
<tr>
<td>T3</td>
<td>10.2</td>
<td>27.9</td>
<td>9.1</td>
<td>7.6</td>
<td>5.6</td>
<td>23.5</td>
<td>14.2</td>
</tr>
</tbody>
</table>

Fig. 6 Analysis of shallow footing T2C using FLAC - Element grid

For comparison, the secant modulus of elasticity $E_s$ is also back-calculated from the tests on the shallow footings. The settlement of a rigid square footing on a semi-infinite homogeneous elastic solid can be estimated as:

$$ s = \frac{815 \rho b (1 - v^2)}{E_s} \approx \frac{0.815 \rho (1 - v^2)}{E_s b} $$

where $s$ = settlement of footing, $b$ = width of footing, $\rho$ = uniform applied load, $\rho$ = concentrated applied load.

The average values of $M$ and $E_s$ estimated by different methods are compared in Table 3. This table, the secant modulus of elasticity $E_{sec}$ and $E_{cal}$ are back-calculated from the static load tests using Eq. (11) at 25 and 50% of the failure load, which is about 20 kN for T1C, 150 kN for T2C and 200 kN for T3C. $M$-modulus can be calculated from the corresponding $E_s$-modulus and vice versa using $v = 0.37$ for T1C, and 0.3 for T2C and T3C. The table shows that, in comparison with the laboratory modulus, the dilatometer modulus is much higher. The modulus estimated from CPT is in good agreement with the laboratory test results provided that a reasonable $\alpha$-values is chosen, e.g. according to Eq. (10). In comparison with the modulus back-calculated from the static load tests, the modulus obtained from the laboratory tests, and the CPT's are all much smaller. The $M$ value evaluated from the dilatometer tests seems to be of comparable size, but it is still smaller than the back-calculated modulus.

The pressuremeter modulus can not be directly compared with the modulus of elasticity because of a different method of settlement analysis. Settlement predicted according to the pressuremeter method also includes ten years of creep. According to Briaud (1992), the elastic modulus in compression $E$ is 2 to 3 times larger than the pressuremeter "first-load" modulus.

5. NUMERICAL ANALYSIS WITH FLAC

FLAC (Fast Lagrangian Analysis of Continua) is a two-dimensional explicit finite difference code, based on a Lagrangian calculation scheme (ITASCA, 1991). FLAC has several built-in constitutive models. For the analysis of the shallow footings, axisymmetrical geometry is used. The square footing is replaced by an equivalent circular one with the same area. The soil media is divided by a mesh composed of quadrilateral elements as shown in Fig. 6. The left vertical boundary represents the axis of symmetry. To minimise the boundary effect, the radius to the right vertical boundary and the depth of the soil mass in question are both chosen as large as ten times the radius of the equivalent footing. A linearly elastic perfectly plastic material according to the Mohr-Coulomb failure criteria is used for modelling the soil. The in-situ stress condition is assumed to be caused by the self-weight of the soil.

To simulate a rigid footing, a constant velocity boundary condition is applied in the negative y-direction across the footing width, i.e. the grid points representing the footing are moved rigidly. In order to minimize shocks to the system thus modelled, the velocity $V$ is kept as small as $10^{-6} m/s$. The number of steps $N$, required to reach a given displacement $D$ of the footing, is then $N = D/V$. 

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For Test T1C, FLAC was run with 18 different combinations of the values of the elastic modulus $E_i$, the internal friction $\phi'$, and the dilation angle $\psi$, with $E_i$ ranging between 13 and 26 MPa, $\phi'$ between 32° and 38°, and $\psi$ between -18° and 0°. The Poisson’s ratio of soil is 0.37 for T1C and 0.3 for T2C and T3C. The elastic shear modulus $G$, and elastic bulk modulus $K$, used in the program, are calculated from the Young’s modulus and the Poisson’s ratio according to the common elasticity formulas:

$$G = E/2(1 + \nu)$$

$$K = E(3\nu + 1)/(3\nu + 1)$$

The parameter study for Test T1C, shown in Fig 7a, indicates that the footing behaviour calculated by FLAC using the soil properties: $E_i = 22.8$ MPa, $\phi' = 34°$, and $\psi = -18°$ (the data file t1c7a) is in best agreement with the observed behaviour.

For Test T2C, FLAC was run with 21 different combinations of the values of $E_i$, $\phi'$, and $\psi$, with $E_i$ ranging between 30 and 42 MPa, $\phi'$ between 35° and 43°, and $\psi$ between 0° and 20°. The parameter study for Test T2C, presented in Fig 8a, shows that the footing behaviour calculated using $E_i = 37.6$ MPa, $\phi' = 38°$ to 39°, and $\psi = 0°$ to 3° is in best agreement with the observed behaviour (the data files t2c7a and t2c7a).

For Test T3C, FLAC was run with 23 different combinations of the values of $E_i$, $\phi'$, and $\psi$, with $E_i$ ranging between 30 and 35 MPa, $\phi'$ between 35° and 41°, and $\psi$ between 0° and 16°. The parameter study for Test T3C, shown in Fig 8c, we find that the behaviour of the footing can be best predicted by FLAC using $E_i = 30$ MPa, $\phi' = 37°$ and $\psi = 0°$ (the data file t3c7).

**Table 3. Soil parameters back-calculated by FLAC**

<table>
<thead>
<tr>
<th>Test</th>
<th>$E_i$ (MPa)</th>
<th>$\nu$</th>
<th>$\phi'$ (°)</th>
<th>$\psi$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>22.8</td>
<td>0.37</td>
<td>34</td>
<td>-18</td>
</tr>
<tr>
<td>T2</td>
<td>37.6</td>
<td>0.30</td>
<td>38-39</td>
<td>0-3</td>
</tr>
<tr>
<td>T3</td>
<td>30.0</td>
<td>0.30</td>
<td>37</td>
<td>0</td>
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</tbody>
</table>

From the parameter study, some remarks can be drawn. FLAC can predict the load-settlement behaviour of shallow footings excellently well if reasonable soil parameters are chosen and if the soil is modelled as a Mohr-Coulomb material. The modulus of elasticity of the soil is of the same order of magnitude as that back-calculated by Eq. (11) from the load tests on the shallow footing at loads between 18% and 25% of the failure load.

Among the soil investigation methods, the dilatometer tests seems to give a modulus whose size is comparable to the $E_i$ value given in Table 4, although it is still smaller than $E_{soil}$. The angle of internal friction $\phi'$ evaluated from the dilatometer tests according to the CPT-linked method, Marchetti (1995), is closest to the value back-calculated by FLAC, although it is slightly larger. For loose sand, a negative dilation angle $\psi$ should be used, while for medium dense or quite dense sand a value equal to, or slightly higher than, zero can be used. The $E_i$ and $\phi'$ obtained from the laboratory tests and back-calculated using FLAC are compared in Table 5. The comparison indicated that the back-
calculation gave lower friction angles and much higher modulus of elasticity than the laboratory values. This may be explained by the fact that all the static load tests were carried out according to the quick maintained load test, in which each load increment is maintained for 15 minutes. A slower testing procedure would influence the $E_r$ value.

<table>
<thead>
<tr>
<th>$T_1$ ($f_r=38%$)</th>
<th>$T_2$ ($f_r=67%$)</th>
<th>$T_3$ ($f_r=62%$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_r$ (MPa)</td>
<td>$\phi'$ (°)</td>
<td>$E_r$ (MPa)</td>
</tr>
<tr>
<td>Lab. FLAC</td>
<td>Lab. FLAC</td>
<td>Lab. FLAC</td>
</tr>
<tr>
<td>3.5</td>
<td>22.8</td>
<td>38</td>
</tr>
</tbody>
</table>

6. CONCLUSIONS

In this study, all the tests have been well performed and the test results are reliable. However, the two studied parameters (friction angle and deformation modulus) estimated from the various methods are quite different. The laboratory tests, especially the triaxial test, are commonly considered as the standard tests, with which other field tests can be compared. However, the study indicates that the laboratory parameters are not good enough for simulating the load-settlement curve of a shallow footing on sand. If Mohr-Coulomb soil model is used, a friction angle lower than the value obtained from the laboratory tests should be used, while the modulus of elasticity should be taken higher than the laboratory one. In this study only the problem of shallow footings on sand is examined. For other geotechnical problems, similar studies should be made especially a soil parameter study using numerical methods which are based on well-instrumented load tests. This can give us soil parameters to be relied upon.

ACKNOWLEDGEMENTS

The work described in this paper is a part of a research project carried out at Department of Geotechnical Engineering of Chalmers University of Technology (CTH) and at the Swedish Geotechnical Institute (SGI), and sponsored by the Swedish Council for Building Research (BFR), which are all gratefully acknowledged by the Author. Sincere thanks is also expressed to Prof. Sven Hansbo for his critical reading of the manuscript.

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Evaluation of cone penetration testing as an in situ liquefaction resistance measurement

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ABSTRACT: Cone penetration tests were conducted at three sites in which high quality undisturbed samples had been obtained by the in situ freezing method. Cone penetration resistance are compared with liquefaction resistance that were determined in the laboratory with the undisturbed samples. Several of the CPTs were tested at each site in order to observe the variation of the test data.

1 INTRODUCTION

Various empirical correlations using the cone penetration test (CPT) data have been proposed for evaluating the liquefaction resistance of a sand deposit. Robertson and Csampanella (1985), Ishihara (1985), Seed and De Alba (1986) and Shibata and Togaraki (1988) developed the correlation curves based on the field performance data during the past earthquakes. More recently, Suzuki et al. (1995a, 1995b) and Tokinai and Tsutsumi (1995) have examined the correlation based on the field performance data and the laboratory test data and have shown that the correlation curves obtained from the respective data agreed well. Comparison of the CPT data and the liquefaction resistance ratio obtained from the high quality undisturbed samples enables one to quantitatively examine the applicability of the CPT as an in situ liquefaction resistance measurement. However, Suzuki, Tokinai and the co-worker's data (Suzuki et al. (1995a), Tokimatsu et al. (1995)) lie in the range of relatively low to medium liquefaction resistance. In Japan, since the occurrence of the 1995 Hyogoken Nambu earthquake, there have been a need of the liquefaction potential assessment of a field against strong ground motion as large as 0.5 to 0.8 g. Thus, the correlation curve should be verified in the range of relatively high liquefaction resistance as well. The paper presents the correlation between the CPT data and the liquefaction resistance of the high quality undisturbed samples. Variation of the CPT data is also presented.

2 IN SITU TESTS

2.1 Test sites

The tests were conducted at three sites, all of which were located in flood basin of major rivers in Japan. Location of the sites are given in Fig. 1. The sites were selected as the authors had carried out the in situ freezing sampling of sandy soils for use in laboratory liquefaction resistance tests, and the standard penetration tests (Matsuo et al. (1996)). The in situ freezing sampling method developed by Yoshimi et al. (1989) yields high-quality undisturbed sand samples with which reliable liquefaction resistance can be obtained from the laboratory tests. As shown later in Fig.3, the sites consist of clean sands and present relatively large SPT N-values. The soils tested were Holocene deposits, except that the soil below an elevation of -4 m at the Edogawa site was Pleistocene deposit.

2.2 Field tests and equipments

At each site, the cone penetration tests, P- and S-wave velocity tests, standard penetration tests, and the other tests were conducted. The P- and S-wave velocity tests were performed with the resonance-type logger. Five types of cone penetrometer were used. Table 1 and Fig. 2 present the specifications and general view of the CPTs, respectively. The CPTs measure tip resistance, qc, porewater pressure, u, and sleeve friction,
Table 1. Specifications of CPTs used

<table>
<thead>
<tr>
<th>Type</th>
<th>Cone tip resistance</th>
<th>Penetration resistance</th>
<th>Penetration rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>20 100 150 0.30 0.35 4500 0.00 0.00</td>
<td>20 100 150 0.30 0.35 4500 0.00 0.00</td>
<td>2</td>
</tr>
<tr>
<td>B</td>
<td>20 100 150 0.30 0.35 4500 0.00 0.00</td>
<td>20 100 150 0.30 0.35 4500 0.00 0.00</td>
<td>1</td>
</tr>
<tr>
<td>C</td>
<td>20 100 150 0.30 0.35 4500 0.00 0.00</td>
<td>20 100 150 0.30 0.35 4500 0.00 0.00</td>
<td>1</td>
</tr>
<tr>
<td>D</td>
<td>20 100 150 0.30 0.35 4500 0.00 0.00</td>
<td>20 100 150 0.30 0.35 4500 0.00 0.00</td>
<td>2</td>
</tr>
</tbody>
</table>

\( q_s \): cone tip resistance, \( q_t \): penetration resistance, \( v \): penetration rate.

Fig. 2. Diagram of CPT

2.3 Test results

Fig. 3 shows the soil profiles and the test data of each site, together with location of the soundings. In the figures, the soil gradation data were those obtained from the SPT core samples. The cone tip resistance, \( q_s \), is the corrected one of the measured \( q_t \) for effective sectional area of a porous filter. It is generally observed in the figures that the tendency of cone tip resistance with depth is quite similar to those of the SPT blowcounts. The deviation of cone tip resistance is relatively small in spite of different types of CPTs used. It seems that the deviation of cone tip resistance is smaller than that of SPT N-value. The deviation of cone tip resistance is large at depths with a SPT N-value of around 30 or larger. At the Tonegawa and Edogawa sites, the deviations of sleeve friction and porewater pressure are large compared with that of tip resistance. This indicates that the measurement of sleeve friction and porewater pressure are sensitive to the mechanical configuration of CPT core. If we look at the data for the Natori site, the data obtained from a particular site seems more stable. In the case of Type A CPT at the Natori site, the penetration was stopped to measure the shear wave velocity of soil every 0.5 m depth. This obviously reflected to the measurements. If the penetration was performed continuously, smoother and lower deviated data would have been expected. Although in the cases of Type-D CPT at the Tonegawa and Natori sites, the penetration rates were varied: D1 - 1 cm/s, D2 - 2 cm/s, the difference of the data was very small. This suggests that the difference of the rate is insensitive to the measurements.

3 LABORATORY TEST

The undisturbed samples retrieved from the sites were tested in the laboratory. Undrained cyclic triaxial tests were carried out with an isotropic confining stress equal to the effective overburden pressure in situ. The liquefaction resistance ratio, \( R_s \), that was defined as a cyclic shear stress ratio required to cause 5% double amplitude axial strain in 20 cycles was determined. The depths for the laboratory test samples and the \( R_s \) values are demonstrated in Fig. 3.

4 CORRELATION BETWEEN IN SITU AND LABORATORY TEST

Fig. 4 shows the correlation between corrected cone penetration resistance \( q_t \) and liquefaction resistance ratio obtained from the laboratory tests. The \( q_t \) corrected for an effective overburden pressure of 1 kgf/cm\(^2\) (98 kPa) is defined as:

\[
q_t = (1.7(a - 0.7)) \times q_t
\]

where \( a \) is effective overburden pressure (in kgf/cm\(^2\)). The corrected cone penetration resistance data for all the CPTs corresponding to the depths for the laboratory test samples are plotted in the figure. Although the data are limited to a range of \( q_t \) over 100 kgf/cm\(^2\), it seems that the liquefaction resistance ratio, \( R_s \), increases considerably when \( q_t \) exceeds around 100-150 kgf/cm\(^2\). This agrees well with the previous study by Tokimatsu et al. (1995). Empirical correlations between CPT data and liquefaction resistance have been proposed by several researchers. Fig. 5 compares the experimental data and the empirical correlation curves for clean sands proposed by Robertson et al. (1985), Ishihara (1985),
Fig. 3-1 Test results: Natorigawa site
(a) Location (plan view)

(b) Soil profile and test data

Fig. 3-2 Test results: Toegeawa site
Fig. 4 Correlation between liquefaction resistance ratio and corrected CPT data

Fig. 5 Comparison of the CPT data with the empirical correlations

Fig. 6 Correlation between liquefaction resistance ratio and corrected SPT N-value

Seed et al. (1986), Shibata et al. (1988) and Suzuki et al. (1995b). In plotting the curves, the in situ liquefaction resistance $\tau / \sigma'$ was converted to the laboratory liquefaction resistance $R_l$ by using the following relationship (Yoshimi et al., 1989):

$$\tau / \sigma' = 0.9 (1 + 2K_s) / R_l$$

(2)

where 0.9 is a correction factor for the effect of multidirectional shear, and $K_s$ is the coefficient of earth pressure at rest, assumed here to be 0.5. Although the curves show for a specific range of mean grain size $D_{50}$ and/or fines content Fe, the curves by Shibata et al. and Suzuki et al. agree fully well with the experimental data. On the other hand, the curves by Robertson, Ishihara, and Seed et al. tend to overestimate the liquefaction resistance.
The liquefaction resistance are plotted against the corrected SPT N-value in Fig. 6, together with the empirical correlation curve proposed by Matsuo et al. (1996). The extent of deviation for Rc-N relation seems to be somewhat less than that for Rf-N relation.

5 CONCLUSIONS

Cone penetration tests were conducted at three sites of sandy soils along with the other field and laboratory tests. The results may be concluded in the following:

(1) Cone penetration resistance presented fairly good consistency. Even among CPTs of different types, variation of cone tip resistance was small. Measurements of sleeve friction and porewater pressure was influenced by the difference in the cone configuration.

(2) The cone penetration resistance was correlated with liquefaction resistance that were determined in the laboratory with undisturbed samples. It was confirmed that liquefaction resistance increased significantly when corrected cone penetration resistance exceeded 100-150 kgf/cm².

(3) The correlation of liquefaction resistance with SPT N-value was better than that with CPT resistance. The difference, however, was not considerable.

ACKNOWLEDGEMENT

The authors are indebted to the technical committee members of the Japan Geotechnical Consultant Association for conducting the field tests, and in particular to Mr. Akio Yamamoto, Oyo Corporation, who helped compiling the data. They are grateful to the members for allowing to present a great amount of data.

REFERENCES


Characterization of Welland Clay at Forkes Road

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ABSTRACT: The results of index tests and physical tests on Welland clay taken from nine boreholes put down along a proposed tunnel alignment over a horizontal distance of 365 m are presented. The index tests included natural water content, Atterberg limits and unit weight. The physical tests included the determination of undrained shear strength (triaxial compression, unconfined compression, in situ vane), the determination of effective shear strength parameters, and oedometer tests. Correlations between some of the parameters are included. Triaxial and unconfined strengths were similar. In situ vane strengths were lower than triaxial or unconfined strengths at shallow depths but about twice as much at depths of about 30 m. Comparisons are made with data obtained from nearby sites.

1 INTRODUCTION

During the late 1960's, the Ministry of Transportation of Ontario (MTO) completed a preliminary foundation investigation at the proposed Forkes Road crossing of the Welland Canal By-Pass near Welland, Ontario (Fig. 1). At the time of the investigation, a tunnel crossing and a high-level bridge crossing of the canal were being considered. The results of the investigation were included in a Ministry report (Stumne 1967) and consisted of detailed determinations of index properties, unit weight, undrained shear strength obtained by unconfined compression, triaxial compression and vane tests, shear strength parameters in terms of effective stresses, and compressibility. The laboratory tests were made on tube samples that were taken in nine boreholes that were put down along a line having a horizontal distance of about 365 m. Subsequently, other geotechnical investigations were made in the Welland area at the locations shown in Fig. 1.

This paper presents a synthesis of the factual data that was obtained in the investigation, provides correlations and comments which may be useful for future studies on Welland Clay.

2 DESCRIPTION AND GEOLOGY OF SITE

The site is located in the Haldimand Clay Plain (Chapman and Putnam 1966) which lies between the Niagara Escarpment and Lake Erie. The ground surface was flat to gently undulating and was poorly drained. Geological profiles for all, or part of, the by-pass between Port Robinson and Port Colborne have been published by Owen (1969), Gorman and Code (1972), and Freeman (1981). The Pleistocene lithographic sequence from top to bottom was described as an upper glacioeustricine clay and till unit, an upper clayey silt till, a lower glacioeustricine clay, silt, and sand unit, a lower gravelly till with a sand-silt matrix, and bedrock consisting of dolostone, shale, and gypsum of the Silurian Selina formation. At the Forkes Road site, the till above the lower till ranged in thickness from about 20 to 25 m. Stumne (1967) separated the clay deposit into five distinct zones (Designated Zones 1 to 5, inclusive) as shown in Fig. 2. These zones are described in subsequent sections.

3 STRUCTURES

The proposed canal and tunnel grades are illustrated in Fig. 2. Construction of the tunnel was to proceed using a cut-and-cover technique and a dewatering scheme that incorporated deep wells in the upper bedrock. The alternative high-level bridge scheme required a minimum grade elevation of about 217 m at the centroid of the canal and this scheme would require high approach embankments. At the time of the investigation, embankment heights of 20 m, or more, were being considered, if technically feasible, because large quantities of fill would be available from the canal excavation at reasonably low cost.
LEGEND

1. Department of Highways Test Shaft (Lo and Milligan 1967)
2. Department of Highways Forkes Rd. Tunnel (Sternac 1967)
3. Department of Highways Main St. E Tunnel (Davis and Milligan 1968)
4. Owen 1969
5. St. Lawrence Seaway Authority Test Excavation (Kwan 1971)
6. St. Lawrence Seaway Authority Test Shaft (Conlon Et Al 1971)

Figure 1. Location plan of Forkes Road and other sites.

Figure 2. Section showing stratigraphy, proposed Welland Canal cut and proposed tunnel grade.
4 FIELD AND LABORATORY INVESTIGATIONS

In general, the upper 15 to 18 m of each borehole was drilled with a continuous flight auger and the remainder of the borehole was drilled with a conventional diamond drilling rig. The clays were sampled with 50-mm I.D. Shelby tubes that were pushed, where possible. Otherwise, a 50-mm O.D. split-spoon sampler was used. In situ vane strengths were determined with a standard Ministry vane. An extra borehole was put down adjacent to Borehole 8 in order to permit comparisons of in situ strengths that were obtained with a Norwegian vane. Representative samples were tested in the laboratory. The tests included determinations of natural water content, Atterberg limits, unit weight, grain size distribution, undrained shear strength (unconfined and triaxial compression), compressibility (oedometer), and effective shear strength parameters.

5 GENERAL SUBSOIL CONDITIONS

Visual descriptions of the soil and variations in soil properties were used to define the zone boundaries shown in Fig. 2. Zone 1 was typically mottled, brown to grey-brown, and included well-defined, but discontinuous, layers (between dotted lines in Fig. 2) or pockets of stratified clay with red, brown, and grey colouring. Oxidation and desiccation were apparent throughout the zone and occasional thin vertical gypsum deposits were noted. The undrained strength of the clay was relatively high and typically, the consistency was stiff to hard. Zone 2 was brown to reddish-brown in colour and contained occasional gravel up to about 25 mm in size. The presence of occasional pockets, or seams, of silt was noted in several boreholes. The soil in Zones 3, 4 and 5 were similar but Zones 3 and 5 included layers of red, brown, and grey clay. Zone 4 was brown to reddish-brown with some mottling and included some pockets of silt. The ground water levels were essentially at ground surface.

6 TEST RESULTS

The natural water content (w_n) increased from about 23 percent at the ground surface to about 35 percent at the bottom of Zone 4. A slight decrease in water content was noted in Zone 5. In stratified Zones 1, 3 and 5, sudden increases in water content (spikes) were noted owing to increased plasticity.

The average unit weight (γ) decreased from about 19 kN/m³ at the ground surface to about 18 kN/m³ at the base of Zone 4 with a slight increase through Zone 5. The following correlation was found γ

\[ (\text{kN/m}^3) = 31.3 - 8.29 \log_{10} w_c (\%) \]

Clay fractions (≤0.002 mm) ranged from about 17 to 69 percent, the higher clay fractions were associated with the more-plastic layers in the stratified zones. Statistical analyses were made on the Atterberg limits and the results are shown in Fig. 3. A histogram for each zone is shown in Fig. 3a and the data indicate that the mean liquid limit (w_l) of about 48-52 is similar in Zones 1, 3, 4 and 5, but lower (39) in Zone 2 with the standard deviation being significantly greater (as expected) in Zones 1, 3, and 5 wherein stratified clays were noted. Excellent correlations (Fig. 3b) were obtained between liquid limit and plasticity index (w_c).

![Figure 3](image)

Figure 3. (a) Histograms of liquid limit, (b) correlations between liquid limit and plasticity index.

Conventional consolidated undrained triaxial compression tests with pore pressure measurements were made. All samples were tested in the same orientation as in situ. The results are summarized in Table 1. The strength parameters c' and φ' were evaluated by straight-line regression analyses.

<table>
<thead>
<tr>
<th>Zone</th>
<th>No. of Tests</th>
<th>c' (kPa)</th>
<th>φ' (deg)</th>
<th>w_c (%)</th>
<th>w_l (%)</th>
<th>w_r (%)</th>
<th>Coeff</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>14</td>
<td>0.952</td>
<td>10</td>
<td>25.5</td>
<td>32</td>
<td>26</td>
<td>52</td>
</tr>
<tr>
<td>2</td>
<td>19</td>
<td>0.986</td>
<td>3</td>
<td>26</td>
<td>27</td>
<td>18</td>
<td>33</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>0.905</td>
<td>15</td>
<td>16</td>
<td>46</td>
<td>22</td>
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<td>34</td>
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<tr>
<td>5</td>
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<td>4</td>
<td>19.5</td>
<td>45</td>
<td>25</td>
<td>54</td>
</tr>
</tbody>
</table>
Conventional odometer tests were made on tube samples. Casagrande's graphical construction was used to determine the preconsolidation pressure ($\rho_0$). A correlation between preconsolidation stress and triaxial compression undrained shear stress ($\tau_0$) is given by $\tau_0 = 6.5 \rho_0$.

Undrained shear strengths were obtained by in situ vane, on laboratory unconfined compression tests and on triaxial compression tests. In general, the undrained shear strengths were a few hundred kPa near the ground surface, gradually decreased through Zones 1 and 2, and tended to be more uniform through Zones 3, 4 and 5. In the lower half of the profile, some strength values were in the 20-30 kPa range. Only a limited number of vane tests were made in the upper part of the profile due to the hard consistency.

The strength values from each borehole were normalized with respect to the triaxial strengths where $R_t = \text{triaxial/unconfined shear strength}$ and $R_v = \text{triaxial/in situ vane strength}$. The average $R_t$ varied from about 0.92 at Elev. 172 m to 1.06 at Elev. 146 m; this indicates that the average ratios between triaxial and unconfined strengths were less than 8 per cent and were not influenced significantly by depth. The average $R_v$ varied from about 1.22 at Elev. 172 m to 0.46 at Elev. 146 m; this indicates the differences between the triaxial and in situ vane strengths are substantial. The average vane strengths were less than the triaxial strengths in the upper part of the profile but were greater than the triaxial strengths in the lower part of the profile; at Elev. 146 m, the in situ vane strengths were about twice the triaxial strengths. The magnitude of the vane strengths obtained with the Norwegian vane and the Ministry vane in Borehole 8 were comparable.

Owing to space limitations, additional data and discussion are not included herein.

CONCLUSIONS

The clay overburden at the Forkes Road site was subdivided into Zones 1 to 5, with increasing depth. The vertical boundary between each zone could be essentially defined on the basis of visual observations alone owing to the presence of stratification in Zones 1, 3 and 5.

The stratified zones contained relatively thin layers which had marked differences in index properties such as natural water content, unit weight, plasticity and grain size distribution. These variations had a significant influence on the undrained shear strength.

Excellent correlations were obtained between liquid limit and plasticity index, water content and unit weight, and between triaxial undrained shear strength and preconsolidation pressure.

The undrained shear strengths as measured by triaxial compression and unconfined compression tests were similar. With respect to the triaxial and unconfined strengths, the in situ vane strengths were lower at shallow depths and about double at depths of about 30 m. The clay is highly overconsolidated near the ground surface and is essentially normally consolidated at depths of about 25 m.

REFERENCES


Iowa borehole shear testing in unsaturated soil

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ABSTRACT: Research is being conducted at the University of Oklahoma to investigate the influence of matric suction on the soil shear strength parameters determined using the Iowa Borehole Shear Test (BST). This paper presents results of a laboratory study during which the BST was conducted in an unsaturated silt soil compacted within a rigid wall calibration chamber. Results of a field investigation are presented for comparison. Experimental observations indicate that the Mohr-Coulomb strength parameters determined with the BST are greatly affected by the matric suction. The BST friction angle tended to increase with increasing matric suction up to a threshold value, above which the friction angle changed very little. Cohesion intercept values from BSTs, while small, tended to decrease with increasing matric suction.

I. INTRODUCTION

Many parts of the US, and the world in general, consist of arid or semiarid climates where unsaturated soils are predominant. In these areas the application of saturated soil mechanics to the behavior of foundations, slopes or other geotechnical systems will result in unrealistic predictions of behavior. A more realistic approach would be to use a method for predicting expected soil moisture conditions during the design life of a system within the framework of unsaturated soil mechanics. Thus, the moisture dependency of soil properties, such as shear strength and stiffness, is logically incorporated into the design process.

Most of the available methods for approaching unsaturated soil mechanics problems are founded in laboratory determinations of unsaturated soil properties with little emphasis on in situ testing. The complexity involved with constructing equipment and using laboratory testing methods for evaluating unsaturated soil properties, may hinder the use of unsaturated soil strength testing in mainstream geotechnical practice. However, the application of unsaturated soil mechanics principles to the interpretation of in situ test methods may have some appeal, since, many of these tests are simple to perform and are becoming more commonplace among consulting firms and other organizations practicing geotechnical engineering.

This paper presents results of research that demonstrate the importance of matric suction on shear strength parameters determined with the BST. A long range goal of this research is to provide a practical framework for interpreting BST results obtained in unsaturated soils, thereby providing a methodology to predict shear strengths under moisture conditions different from those existing at the time of in situ testing.

2. BACKGROUND

2.1 The Iowa Borehole Shear Test

The Iowa Borehole Shear Test (BST), developed by Richard Handy and his co-workers (Handy and Fox 1967), can be used in situ to determine the friction angle and cohesion intercept of soil. The BST is the only invasive type in situ test that can produce a Mohr-Coulomb failure envelope by direct determination of forces on the failure plane, and
generally, the envelopes are highly linear and reproducible (Luton and Tinian 1987).

The BST is a fairly simple test in concept and is analogous to performing a direct shear test on the sidewalls of a borehole. A borehole in which the BST is deployed, is typically 76 mm in diameter and can be readily advanced in most soils using a hand auger, Shelby tube or small solid stem augers, although, minimal disturbance to the borehole wall is desired. Stage testing is conducted by lowering a shear head into the hole and expanding diametrically opposed, curved, serrated shear plates against the borehole wall. During a test, the total normal stress is increased incrementally, and the corresponding shear strength is determined by drawing the shear head upward while measuring the maximum force obtained. The shear and normal stress is calculated using the area of the plates and the shear and normal forces determined during the test. Borehole shear test strength parameters have been found to compare favorably with the results of laboratory determined effective stress-strength parameters for medium to stiff clays below the water table (e.g., Miller and Luton and Tinian 1994). The test offers considerable advantage over laboratory tests because it can be deployed rapidly. The authors have found that BSTs conducted in unsaturated silt and clay soils, while still producing highly linear and reproducible failure envelopes, have resulted in friction angles and cohesion intercepts that appear to be a function of the matric suction in the soil.

2.2 Shear Strength of Unsaturated Soil

According to Fredlund and Rahardjo (1993), unsaturated soil mechanics encompasses soil with negative pore water pressures. The behavior of unsaturated soils depends on the nature of the water and air phase. The air phase can be continuous or exist as occluded bubbles within the water phase.

It is now widely accepted that the stress state for an unsaturated soil can be adequately described by two independent stress state variables which are the net normal stress and the matric suction. The net normal stress is the difference between the total stress and total air pressure ($\sigma_{1}-\sigma_{3}$) acting in the soil while the matric suction is the difference between the pore air and pore water pressure ($u_{a}-u_{w}$). Two independent stress tensors can be used to fully describe the state of stress at point within a soil mass (Fredlund and Rahardjo 1993). The stress state variables can be used to describe the shear strength of the soil in equation form as follows:

$$\tau = c' + (\sigma_{3} - u_{w}) \tan \phi' + \left( u_{a} - u_{w} \right) \tan \phi''$$

(1)

where: $\tau$ = shear stress at failure, $\phi'' = $ effective cohesion, $\sigma_{3}$ = total stress normal to the failure plane, $u_{a}$ = pore air pressure, $\phi' = $ angle of internal friction associated with the net normal stress, $u_{w}$ = pore water pressure, and $\phi'' = $ angle of internal friction associated with the matric suction. Note, for a saturated soil, $u_{a} = u_{w}$ and $c'$ and $\phi'$ become effective-stress strength parameters.

Equation (1) describes a plane on a three-dimensional plot with shear strength on the vertical axis and matric suction and net normal stress on the horizontal axes. Evidence suggests that this linear representation of shear strength may be applicable over a limited range of suction due to non-linearity in the shear strength-suction relationship (Fredlund and Rahardjo 1993). The evaluation of unsaturated soil shear strength in the laboratory is no trivial matter and requires both control of the pore air and pore water pressures.

Inspection of Eq. (1) reveals that to be able to predict the shear strength, at moisture conditions other than those existing during testing, requires knowledge of the soil-water characteristic behavior, i.e., the relationship between water content and matric suction. It can be seen that for the case where the pore air pressure is zero (atmospheric pressure), an increase in matric suction will result in an increase in the cohesion intercept projected on the net normal stress-shear stress plane; however, the slope of the failure envelope in this same plane remains constant.

2.3 Calibration Chamber Testing

Calibration chamber testing involves the creation of soil beds in a laboratory for the purpose of carefully studying soil response to in situ testing or other types of loading. Originally, calibration chambers were developed to calibrate cone penetrometers under simulated field conditions (Holden 1993). The goal of chamber testing is to create consistent, uniform soil beds that can be heavily instrumented to measure soil response to testing under controlled boundary conditions. Calibration chambers offer an excellent way to study the influence of different soil conditions on the test results, because, variables that may otherwise be difficult to determine in situ, such
as the state of stress, stress history, or soil homogeneity, are to a large extent under the control of the experimental. Calibration chambers used to date, incorporate soil test beds with diameters that range from 0.5 to 2 m and heights that range from 0.8 to 2.9 m (Ghiouso and Jamilolakowski 1991). The presence of the chamber boundary and the type of boundary have been found to greatly affect in situ test results. Chambers can have rigid or flexible boundaries. Research with sandy and clayey soils indicates that the boundary effects are more severe for rigid, radial boundaries. Ghiouso and Jamilolakowski (1991) suggest that in silice sand of low to moderate compressibility, a ratio of the chamber to cone penetrometer diameter greater than 30 to 35 is required to avoid boundary effects. Schmahl and Houfshy (1999) found that finite chamber dimensions affected the ultimate cavity pressure determined with a pressuremeter to roughly the same extent that the tip resistance of a cone penetrometer is affected.

While a rigid wall chamber with a diameter of 0.3 m was used in this study, it is believed that the boundary effects were minimal because the BST incorporates a relatively small proportion of soil around the borehole. Furthermore, the stress on the failure plane is known throughout the BST, and the expansion of the shear head results in relatively little lateral deformation.

3.0 RIGID WALL CALIBRATION CHAMBER EXPERIMENTS

3.1 Experimental Setup

At the University of Oklahoma, BSTs were conducted in an unsaturated silty soil contained within a rigid wall calibration chamber. Soil beds were prepared by carefully compacting silty soil into 5-cm thick layers to a height of 30 cm. Soil was compacted to a height of 0.3 m using a technique designed to produce a consistent, homogeneous fabric. The target matric suction values were determined from the soil-water characteristic curve. A low plastic silt having a plasticity index of four was selected so that values of matric suction in the soil bed would be within the range that could be measured with a miniature tensiometer inserted near to the test location. To produce a uniform borehole, a 76-mm diameter tube was fixed in the center of the chamber and soil was compacted in the annulus between the center tube and chamber wall.

Following compaction of the soil bed, the bottom of the chamber served as a piston through which a simulated overburden pressure of about 160 kPa was applied via a hydraulic jack. A period of 24 hours was allowed to achieve moisture equilibrium in the soil. Immediately prior to testing, the central tube was carefully removed and the BST was initiated.

3.2 Experimental Results

A summary of calibration chamber soil data determined for the conditions existing after application of the overburden pressure, just prior to testing is given in Table 1. Borehole shear test results are shown in Table 2.

Table 1. Calibration Chamber Soil Test Bed Data

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Average Water Content (%)</th>
<th>Void Ratio (%)</th>
<th>Degree of Sat. (%)</th>
<th>Vol. Water Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>13.1</td>
<td>0.74</td>
<td>45.1</td>
<td>19.2</td>
</tr>
<tr>
<td>2</td>
<td>22.8</td>
<td>0.58</td>
<td>99.8</td>
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<tr>
<td>3</td>
<td>20.0</td>
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<td>81.2</td>
<td>31.5</td>
</tr>
<tr>
<td>4</td>
<td>18.1</td>
<td>0.71</td>
<td>65.2</td>
<td>27.0</td>
</tr>
<tr>
<td>5</td>
<td>15.6</td>
<td>0.73</td>
<td>54.9</td>
<td>23.1</td>
</tr>
<tr>
<td>6</td>
<td>20.1</td>
<td>0.62</td>
<td>83.0</td>
<td>31.7</td>
</tr>
<tr>
<td>7</td>
<td>17.4</td>
<td>0.71</td>
<td>62.8</td>
<td>26.0</td>
</tr>
<tr>
<td>8</td>
<td>15.5</td>
<td>0.70</td>
<td>56.8</td>
<td>23.3</td>
</tr>
</tbody>
</table>

Table 2. Borehole Shear Test Results

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Matric Suction (kPa)</th>
<th>Friction Angle (deg.)</th>
<th>Cohesion (kPa)</th>
<th>Coeff. Of Corr., r²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>56</td>
<td>46.0</td>
<td>1.1</td>
<td>0.9994</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>32.6</td>
<td>4.6</td>
<td>0.9912</td>
</tr>
<tr>
<td>3</td>
<td>10</td>
<td>39.3</td>
<td>6.2</td>
<td>0.9997</td>
</tr>
<tr>
<td>4</td>
<td>15</td>
<td>42.8</td>
<td>4.9</td>
<td>0.9991</td>
</tr>
<tr>
<td>5</td>
<td>32</td>
<td>44.7</td>
<td>2.3</td>
<td>0.9931</td>
</tr>
<tr>
<td>6</td>
<td>*</td>
<td>36.5</td>
<td>3.8</td>
<td>0.9972</td>
</tr>
<tr>
<td>7</td>
<td>*</td>
<td>43.8</td>
<td>2.8</td>
<td>0.9965</td>
</tr>
<tr>
<td>8</td>
<td>24</td>
<td>43.7</td>
<td>3.6</td>
<td>0.9990</td>
</tr>
</tbody>
</table>

* Tensiometer Failed to Work

In Fig. 1, the BST failure envelopes are plotted. Values of the correlation coefficient, r², shown in Table 2, indicate that the envelopes are nearly linear. The data listed in Tables 1 and 2 are shown graphically in Figures 2-4. The soil-water characteristic curve presented in Fig. 2 is typical of silty soil and clearly portrays the dependency of matric suction on the moisture content of the soil.
The relationships between matric suction, volumetric water content, and BST strength parameters are shown in Figs. 3 and 4. Some interesting observations are noted below.

1) The BST friction angle increases rapidly from 33° to 45° as the matric suction increases from zero to 15 kPa. Beyond a matric suction of 15 kPa, the friction angle increases gradually to 46° at a matric suction of 36 kPa. A matric suction of 15 kPa appears to represent a threshold, above which additional increases in matric suction lead to small increases in shear strength.

2) The BST cohesion intercept tends to decrease with increasing matric suction.

3) Both the BST friction angle and cohesion intercept show a significant degree of correlation to the volumetric water content; however, the degree of correlation is greater for the friction angle.
If the effective stress friction angle and cohesion intercept are assumed to be relatively constant for the range of void ratio investigated, and the pore air pressure is assumed to be equal to atmospheric pressure, then based on Eq. (1), the slope of the BST failure envelopes should be the same, and only the cohesion intercept should increase with increasing matrix suction. This is not the case with the data obtained.

The reasons for why the BST friction angle increased as the matrix suction increased are not fully understood, but possible explanations are offered. First, the effective stress friction angle may have increased as the matrix suction increased, due to variations in fabric; however, this seems unlikely given that the void ratio of test beds, shown in Table 1, was higher for higher values of matrix suction. Void ratios were higher, for higher values of matrix suction, due to the fact that relatively less compression of the test beds occurred under application of the overburden pressure. Second, the matrix suction on the failure plane may have changed during application of the normal stress, and/or, during shearing. Thus, the vertical offset of the failure envelope due to the strength contributed by matrix suction, may be different for each normal stress increment. Changes in matrix suction may have resulted from soil fabric alterations that occurred when the normal stress was increased, or, from volume change tendencies during shearing.

The trend of the friction angle with increasing matrix suction shown in Fig. 3 is similar to that seen in the results presented by Marshour et al. (1996) for triaxial compression tests on highly plastic clayey soil. Under the assumption of constant matrix suction during shearing, they offered the following explanation for this phenomena. At low matrix suction, water is abundant and acts as a lubricant between soil grains, thus, reducing friction. Drying of the soil has a dual affect on shear strength. It results in an increase of matrix suction and an increase of friction due to less lubricant (water) between grains. Hence, the friction angle increases with increasing matrix suction. Furthermore, it was suggested that the decrease in the growth rate of the friction angle at higher matrix suction, occurred beyond the residual saturation value. Beyond this value, slight changes in water content result in large changes in matrix suction. While this physical explanation of the phenomena in question, is reasonable, the possibility of other mechanisms such as shear induced changes in matrix suction must be explored. For this reason, efforts are currently being made to modify the BST apparatus so that suction can be measured on the face of the shearing plates.

The trend of decreasing BST cohesion intercept with increasing matrix suction is opposite to that observed by Marshour et al. (1996). This result may be due in part to variations in soil fabric, from test to test, that resulted during compaction of the soil under different moisture conditions.

### 4.0 FIELD OBSERVATIONS

The BST was evaluated for predicting the uplift resistance of four, 80-mm diameter drilled shafts installed in an unsaturated, highly plastic, clayey soil. The prediction method used, was that presented by Lunetegger and Miller (1994). This method incorporates estimates of shear strength parameters from the BST and lateral stresses from the prebored pressuremeter test. Soils at the test site were relatively homogeneous with increasing depth, except with regard to the moisture conditions, which changed from relatively dry to relatively moist with depth. A plot of the water content versus BST strength parameters is shown in Fig. 5. The trend of data is similar to that observed in Fig. 4, where the BST friction angle is seen to decrease and the cohesion intercept is seen to increase, with increasing water content. While the friction angles shown in Fig. 5 are quite high, their use resulted in accurate predictions of the uplift shaft capacity, as shown in Fig. 6. The accuracy of these predictions is likely the result of the fact that the moisture content of the soils, and hence the matrix suction, changed very little over the time elapsed between in situ testing and pile load testing.

![Figure 5. BST Results from the Shaft Uplift Test Research Site.](image-url)
The results indicate that the strength parameters determined with the BST, in unsaturated soil, may accurately depict the soil shear strength corresponding to a specific water content.

5 SUMMARY

Results of preliminary calibration chamber studies at the University of Oklahoma indicate that the matrix suction has a significant influence on the friction angle and cohesion intercept determined with the BST. The friction angle tends to increase with increasing matrix suction, while the cohesion intercept tends to decrease. Borohole shear test results from the chamber studies showed strong resemblance to results from actual field tests. Based on limited field testing, the unsaturated, soil strength parameters from the BST appear to be representative of the soil shear strength corresponding to the water content at the time of testing. The relationships between matrix suction, volumetric water content, and strength parameters determined with the BST, appear to be predictable, and are similar to those obtained by other researchers using triaxial testing to define the strength.

REFERENCES

Comprehensive slope stability analysis in unsaturated silts and sands

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ABSTRACT: A method has been presented which facilitates the determination of the shear strength of unsaturated soils by using the water retention curve and the angle of internal friction without performing elaborate laboratory tests (Öberg and Sällfors, 1995). To further improve this method, a concept, the effective matrix suction will be introduced and a relationship between the d50-value from the grain size distribution and this so-called effective matrix suction will be presented. This relationship facilitates a rough determination of the characteristic matrix suction to be used in a design situation without having to perform measurements of matrix suction. Finally, it is illustrated how all these results can be used in a stability analysis. Guidelines for engineering practice are given, i.e., for the manner in which comprehensive stability analyses should be made.

1. INTRODUCTION

Usually, conventional slope stability analysis in non-cohesive soils is based on the effective strength parameters of the natural sediment. For many silt and sand slopes, which to a great extent are unsaturated, a conventional analysis does not explain the existence of the steep slopes. Since experience in Scandinavia concerning the unsaturated zone and its matrix suction has been limited, no account has normally been taken of matrix suction in slope stability analysis. Consequently, these analyses have resulted in unrealistically low factors of safety, which have not been in agreement with the observed sliding activity. Therefore for practical engineering purposes a method was formulated by Öberg and Sällfors (1995), where the shear strength of unsaturated silt and sands can be determined exclusively based on the water retention curve. The shear strength can then be calculated as

\[ \tau_f = c' + (\sigma - u_w) \tan \phi' + (u_s - u_w) \gamma_s \tan \phi' \quad (1.1) \]

A similar approach was reported by Varapalli et al. (1996). To further simplify the determination of the shear strength a relationship between the d50-value from the grain size distribution and the effective matrix suction will be introduced. This relationship facilitates a rough determination of the characteristic matrix suction without having to perform measurements of matrix suction. Finally, it will be illustrated how these results can be used in a stability analysis.

2. MEASUREMENTS OF MATRIX SUCTION

Measurements of matrix suction have successfully been performed at different test sites in Sweden. The test sites are situated in the middle of Sweden in areas with major sand and silt deposits, and the unsaturated zone is as deep as 20 meters.

In Figure 2.1 and 2.2 results from matrix suction measurements at two test sites are presented, the Skedomsravinne test site and the Ransätter test site. These two test sites represent the extremes in the sense that the ground water table is situated about 24 m below the groundwater table (the Skedomsravinne test site) contrary to 5 m below the ground water table (the Ransätter test site). Consequently, the piezometers at the Skedomsravinne test site are placed in the intermediate zone and at the Ransätter test site in the capillary zone.

To be noticed from the measurements at the Skedomsravinne test site is that normal precipitation and ground water fluctuation does not seem to affect the matrix suction in the intermediate zone, and no clear seasonal variation can be observed, see Fig.2.1. The reason for this is that the intermediate zone is not
completely saturated. Thus, the pores are to a great extent filled with air and the pore water mostly exists as a thin film surrounding the solids. The matrix suction profile becomes complex and no hydrostatic relationship seems to exist. The matrix suction varies irregularly with depth and is affected by the immediate surrounding soil conditions, i.e. porosity, relative density and water content.

![Figure 2.1](image1.png)

Figure 2.1 Measured matrix suction at the Skeðsforsøgene test site.

In the capillary zone there is a clear seasonal variation of the matrix suction, see Fig. 2.2.

![Figure 2.2](image2.png)

Figure 2.2 Measured matrix suction at the Ransäter test site.

Here the pore water phase is continuous, which means that there is contact between the ground water table and the pore water phase. In the saturated part of the capillary zone the matrix suction seems to be hydrostatic.

Another observation made is that it is difficult to perform long-term measurements of matrix suction particularly in the intermediate zone and presumably also in the soil water zone. The significant amount of air in these zones appears to diffuse through the pore water into the chamber of the piezometers causing the matrix suction to reach values near the atmospheric pressure. It is therefore of the utmost importance to de-air and re-fill the piezometers with de-aired water on a regular basis in order to obtain reliable results. The same problem was not experienced with piezometers installed in the capillary zone, presumably because the pores are almost completely filled with water and consequently, the air phase is limited.

3. DEFINITION OF THE EFFECTIVE MATRIX SUCTION

The advantage of using the degree of saturation, as in Eq. 1.1, for determining the positive effect of matrix suction on the shear strength is that it is phenomenologically simple to understand. The degree of saturation reflects the pore area filled with water, i.e. the area where the matrix suction in the soil acts. In conformity with the effective stress principle for saturated soils, it would be possible to write the effective stress for an unsaturated soil as 

$$\sigma' = \sigma_{eff}$$

where $\sigma_{eff}$ is a so-called effective matrix suction. If $\sigma_{eff}$ is defined this effective matrix suction can be defined as the degree of saturation multiplied by the corresponding matrix suction. If $\sigma_{eff}$ is defined the effective matrix suction becomes

$$\sigma_{eff} = \sigma_{r}(1 - S_r) + \sigma_{w}S_r$$  \hspace{1cm} (3.1)

In Fig. 3.1 the water retention curve (●) for a silt is given. If the matrix suction is multiplied by the degree of saturation, the curve (●) also presented in Fig. 3.1 is obtained. It can be noticed that the effective matrix suction versus the degree of saturation reaches a fairly constant value. By comparing the two curves in Fig. 3.1 it becomes clear that the constant effective matrix suction is approximately equal to the air entry value of the soil. However, the matrix suction multiplied by the degree of saturation can reach values both somewhat higher and somewhat lower than the air entry value. The exact shape of the curve representing the effective matrix suction versus the degree of saturation is governed by the shape of the water retention curve. A well graded soil has a water retention curve with a steep slope.
Consequently, the effective matrix suction becomes greater than or equal to the air entry value. Whereas, a poorly graded soil has a fairly flat slope of water retention curve, and thus the effective matrix suction has a value lower than or equal to the air entry value. However, the differences are relatively small.

![Water retention and effective matrix suction curves for a well graded and a poorly graded material.](image1)

The effective matrix suction value of the soil is theoretically the lowest matrix suction that can be obtained at almost full saturation above the ground water table. Every measured matrix suction corresponds to a certain point on a matching water retention curve. From a designer's point of view, this indicates that it may be sufficient to reduce the measured matrix suction by multiplying it by the degree of saturation in order to obtain an estimate of the lowest matrix suction and consequently the lowest shear strength that can be expected.

4. RELATIONSHIP BETWEEN THE GRAINSIZE DISTRIBUTION AND THE WATER RETENTION CURVE

For engineering practice it is important to be able to roughly estimate the matrix suction for a soil without costly time consuming measurements in a periscopic investigation.

In order to study a possible relationship between the grain size distribution, degree of saturation and the matrix suction, a compilation of more than ten grain size distributions with appertinent water retention curves was made. To be noticed is that the soils compiled are Scandinavian silts and sands.

The compilation showed that all the water retention curves had almost the same slope for degrees of saturation less than 80% when presented in a logarithmic scale. In Fig. 4.1 the effective matrix suction curves for the soils compiled are given. For most of the soils tested there is a clear plateau, where the effective matrix suction versus degree of saturation is almost constant. This plateau is approximately equal to the air entry value of the soil.

![Effective matrix suction curves for the soils tested.](image2)

If the $d_{50}$-value is evaluated and taken as a measure of grain size and compared with the effective matrix suction (or air entry value), the relationship given in Fig. 4.2 is obtained. The compilation of the water retention curves indicated that the air entry value for the soils mainly seems to occur at a degree of saturation of about 90%. Moreover, by the use of Fig. 4.2 and the fact that the water retention curves have almost the same inclination, empirical water retention curves can be constructed.

The first step is to determine the $d_{50}$-value for the soil of interest. The effective matrix suction can then be estimated by use of Fig. 4.2. The corresponding degree of saturation for the air entry value is assumed to be 90%. For example according to Fig. 4.2 a soil with a $d_{50}$-value of 0.04 mm has an effective matrix suction value of 21 kPa or an air entry value of 23 kPa (21/0.9~23 kPa). This gives the filled circle in Fig. 4.3. Furthermore, Line 1 is drawn through origo. Finally, Line 2 is constructed by use of the light grey lines, which have the same inclination as the water retention curves determined in the laboratory.
In the upper part of the soil profile, in the so-called soil water zone, the matrix suction is influenced by evaporation and precipitation. The knowledge of the effect of precipitation on matrix suction is still limited, and an assumption on the safe side would be to set the matrix suction in the soil water zone to zero.

In the capillary saturated zone, the matrix suction is affected by the ground water fluctuations. Due to the pore water phase being continuous the matrix suction values can be assumed to be hydrostatic in this zone. This was also confirmed by field measurements conducted at different sites, see Fig. 2.7. Therefore, in the capillary zone the characteristic matrix suction values in a design situation would be best chosen according to the highest ground water level that can be expected. However, it is of the utmost importance to know the extension of the capillary zone to be able to estimate the matrix suction profile in a design situation.

The discussion in section 3 and the results presented in section 4 together with the fact that the matrix suction values in the intermediate zone are fairly constant, see Fig. 2.1, indicate that for engineering purposes it is only necessary to reduce the measured matrix suction with the degree of saturation in order to obtain the characteristic matrix suction. Therefore, the recommendation in a design situation for the intermediate zone is that the air entry value or the calculated effective matrix suction should be chosen as the characteristic matrix suction value.

5. DETERMINATION OF CHARACTERISTIC MATRIX SUCTION

5.1 Direct determination

The main difficulty when performing a stability analysis with respect to matrix suction is to decide to what extent the matrix suction can be utilised. In the following, recommendations for the determination of characteristic matrix suction to be used in a design situation are given.

Estimation of characteristic matrix suction by the use of the $d_{50}$ value from the grain size distribution.

This method can be used if the soil has a clay content which is less than 5% and if the air entry value is expected to be between 10–100 kPa (which is the case for most Swedish silts and fine sands).

By determining the $d_{50}$ value for the soil at hand and by using Fig. 4.2 to evaluate the effective matrix suction...
suction (or air entry value) the empirical water retention curve can be constructed as described in section 4. Thereby, it is possible to roughly estimate the shear strength for various degrees of saturation, using Eq. 1.1.

Estimation of characteristic matrix suction by the use of data presented by Anderson and Wilkert (1972)

This method in which a data base presented by Anderson and Wilkert (1972), containing about 400 water retention curves with belonging grain size distributions is utilized, is to be preferred when the clay content is moderately higher than 5%. Again, the grain size distribution for the soil must be determined. Then, by matching the grain size distribution for the soil at hand with the grain size distributions of the soils presented by Anderson and Wilkert, a reference soil with a corresponding water retention curve can be found. It is then possible to estimate the characteristic matrix suction of the soil, and to calculate the shear strength for different degrees of saturation in accordance with Eq. 1.1.

6. SLOPE STABILITY ANALYSIS

In order to illustrate the difference in calculated factors of safety when matrix suction values are taken into account and when not, a stability analysis has been carried out for one test site. The slope stability analysis have been made using the computer program SLOPE/W (Geo-slope, 1991-1992). The objective of the stability analyses has not been to design the slope but merely to compare the results in terms of calculated factors of safety for the different approaches used for determining the characteristic matrix suction values. For a conventional drained analysis the lowest calculated factor of safety is $F_s = 0.97$, see Fig. 6.1.

In Table 6.1 the characteristic matrix suction values from the in situ measurements are presented as the measured matrix suction multiplied by the degree of saturation (i.e. effective matrix suction). The characteristic matrix suction in the soil water zone has been set to zero and an estimate of the extension of the capillary zone has been made by the use of measurements and water retention curves.

Indirect determination of the characteristic matrix suction values in the intermediate zone have been made using both the $\phi_{sr}$ value from the grain size distribution and the data presented by Anderson and Wilkert. The $\phi_{sr}$ values have been evaluated from the grain size distribution for each depth. By use of

![Figure 6.1](image.jpg)

The most critical global slip surface for a drained analysis when matrix suction is not taken into account.

Furthermore, by matching the grain size distributions determined in the laboratory with the compositions of the soils presented by Anderson and Wilkert, the corresponding water retention curves were found for the reference material. An estimate of the air entry value multiplied by the corresponding degree of saturation for each depth was made. The characteristic matrix suction values are presented in Table 6.1 as the effective matrix suction values.

<table>
<thead>
<tr>
<th>Depth [m]</th>
<th>Characteristic matrix suction determined by the use of $\phi_{sr}$ value Anderson and Wilkert</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>-1</td>
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<tr>
<td>5.7</td>
<td>-3</td>
</tr>
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<td>-6</td>
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<td>13.6</td>
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</tr>
<tr>
<td>21</td>
<td>-22.5</td>
</tr>
<tr>
<td>22.5</td>
<td>-15</td>
</tr>
<tr>
<td>24</td>
<td>0</td>
</tr>
</tbody>
</table>

The discrepancy between the calculated effective matrix suction from the in situ measurements of matrix suction and the effective matrix suction determined indirectly by the use of the $\phi_{sr}$ value and the data presented by Anderson and Wilkert depends on the fact that the soil in situ is probably somewhere between wetting and drying. The matrix suction values determined indirectly are determined from so-called drying curves which represents the upper limit of the water retention curve and consequently higher values of matrix suction are obtained for a certain degree of saturation.
The calculated factor of safety for the slip surface presented in Fig. 6.1, when matrix suction values from in situ measurements are taken into account, is $F_{s} = 1.06$. This corresponds to an increase in factor of safety by 10%, even though the matrix suction values are fairly modest.

When the matrix suction is estimated from the data presented by Anderson and Wiklert, the factor of safety for the same slip surface becomes somewhat higher $F_{s} = 1.09$. If the matrix suction values evaluated by use of the relationship between the $d_{50}$-value and the effective matrix suction is used, a further increase in factor of safety is obtained $F_{s} = 1.14$.

CONCLUSIONS

There is a relationship between the $d_{50}$-value from the grain size distribution and the effective matrix suction. This relationship together with the fact that the determined water retention curves had almost the same inclination in a logarithmic scale, made it possible to develop a new method for estimating an empirical water retention curve based on Figs. 4.2 and 4.3.

Recommendations are given of how to choose characteristic matrix suction in slope stability analysis have been given. Methods for both a direct and an indirect determination of characteristic matrix suction have been proposed.

If only a rough estimate of the stability is to be performed, available geotechnical investigations and an indirect determination of the matrix suction may be sufficient. However, the stability analyses showed, as expected, that when the characteristic matrix suction values were determined indirectly, the factor of safety was slightly overestimated as compared to the case when the characteristic matrix suction was determined directly. This is due to the fact that the indirect determination of the characteristic matrix suction is based on so-called drying curves which represent the upper limit of the matrix suction versus degree of saturation. Obviously, in order to perform more elaborate stability analyses in unsaturated silt and sand slopes, the traditional site investigations should be supplemented with field measurements of the matrix suction and determination of the degree of saturation.

However, the main conclusion is that the matrix suction values have to be taken into account in stability analyses, in order to obtain more relevant factors of safety. The result thus obtained becomes more concordant with the existing steep configurations and observed sliding activities, than the results obtained by conventional stability analysis. The effects of cementation and root systems ought to be investigated and included in the stability analyses.

REFERENCES


A correlation of soil strength between different sounding tests on embankment

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ABSTRACT: The Standard penetration test and the Swedish weight sounding test are both generally used on many railway lines. Since these tests are relatively labor-intensive, however it is difficult to perform them at many locations on one site. At the same points on many railway embankments, both a Portable dynamic cone penetration test, which is light and portable, and the Swedish weight sounding test were made. Correlations in soil strength between these two tests were analyzed and a relationship proposed for equations between $N_c$ for the Portable dynamic cone penetration test and $W_{sw}$, $N_{sw}$ for the Swedish weight sounding test. A conversion formula between $N_c$ and $N$-value in the Standard penetration test or $q_u$ for unconfined compression strength was also proposed.

1. INTRODUCTION

Many slope failures have occurred during the rainy season or typhoon storms in Japan. In order to determine the causes of this problem and to estimate the soundness and stability of the slope, a common procedure has been to conduct a sounding test to investigate the depth and strength of the surface layer.

The Standard cone penetration test and the Swedish sounding test have generally been used for this purpose, however these tests are relatively labor intensive. In fact, since it is difficult to perform many sounding tests on a slope with a large area, the soil characteristics are represented from sounding tests at only one or several locations. Although information thus obtained is limited, it is commonly used as a basis for design and maintenance.

In this study two types of sounding tests were performed, namely: the Swedish sounding test and a Portable dynamic cone penetration test, at the same point on the embankment. The characteristics of soil strength of the embankment surface were analyzed. Based on the results, the correlations between the Portable dynamic penetration test and the Swedish sounding test, etc. were investigated.

2. PREVIOUS RESEARCH ON CORRELATION BETWEEN DIFFERENT SOUNDING TESTS

In summary previous research on soil strength from sounding tests, Inada (1960) proposed formulas between $N$-value of the Standard penetration test, and $W_{sw}$ (N) and $N_{sw}$ (half-turn/m) of the Swedish weight sounding test for the Meishin highway as follows:

$$N = 0.002 W_{sw} + 0.067 N_{sw}$$

(Gravel, sand and sandy soil)

$$N = 0.003 W_{sw} + 0.50 N_{sw}$$

(Clay and clayey soil) \[(1)\]

In addition, Inada (1960) proposed a formula between $q_u$ (kN/m$^2$) of unconfined compression strength, and $W_{sw}$ (N) and $N_{sw}$ (half-turn/m) as follows:

$$q_u = 0.04 W_{sw} + 0.75 N_{sw}$$ \[(2)\]
Miki (1959) proposed the following formulas:

\[ N = \frac{1}{10} N_{sw} \text{ (for } N_{sw} < 90) \]  
(Loam)

\[ N = \frac{1}{12} N_{sw} \text{ (Sand and gravel)} \]  
(3)

Those formulas are linear equations. Ueda (1957) proposed the following nonlinear equation.

\[ N = 0.318 N_{sw}^{0.715} \text{ (Clay, sand and gravel)} \]  
(4)

A direct comparison between \( N_{sw} \) for the Swedish sounding test and \( q_{s} \) (MN/m²) of the Dutch double-tube cone penetration test is seldom attempted. Miki (1959) obtained this formula for loam,

\[ q_{s} = \frac{1}{2} N_{sw} \]  
(5)

Muromachi (1971) proposed a formula based on his data measured along railway lines.

\[ q_{s} = 0.67 + 0.049 N_{sw} \]  
(6)

The relationship between the Portable dynamic cone penetration test and the Standard penetration test is expressed by the Japanese Ministry of Construction Office as follows.

\[ N_{c, 30} = 3 \sim 10 N \]  
(7)

where \( N_{c, 30} \) is number of strokes to a penetration depth of 30 cm.

The relationship between the portable dynamic cone penetration test and the Swedish weight sounding test is shown in the Portable dynamic cone penetration test manual (Sabo Technical Center, 1982) as:

- Surface layer soil \( N_{sw} < 90 \text{ kN} : N_{c} < 5 \)
- Sandy loam \( N_{sw} = 90 \text{ kN} : 5 < N_{c} < 15 \)
- Sand layer \( 100 < N_{sw} \text{ kN} : 15 < N_{c} \)

But these values only give an order and are not quantitative.

No equation which combines the Swedish sounding test and the Portable dynamic cone penetration test is presently available for railway embankment sounding.

3. DESCRIPTION OF SOUNDING TEST FOR A RAILWAY EMBANKMENT

3.1 Sounding test

The Swedish weight sounding test and the simplified penetration test were used for slope investigation.

The latter was one of the dynamic cone penetration test. The test instrument designed by the Public Works Research Institute of the Ministry of Construction is called a Portable dynamic cone penetration test instrument because it uses a portable rig, light weight and is small in size as shown in Fig. 1 (Sabo Technical Center, 1983). The instrument whose total mass is about 15 kg consists of a cone head connected with a rod and a driving weight for penetration. The angle of the cone is 60 degrees, and the diameter is 16 mm. The number of strokes \( N_{c} \) where by the cone is penetrated 10 cm when a mass of 5 kg drops from a 50 cm height is counted.

![Fig. 1 Portable dynamic cone penrometer](image)

3.2 Sounding test sites

Embankments for the investigation included counts 65 locations along Japan Railways (Okada et al., 1992). Collapses due to heavy rainfall occurred during the past 10 years. A few points were chosen on the middle slope site near the collapsed embankments and both the Swedish
weight sounding tests and the Portable dynamic cone penetration tests were carried out at each site.

The distributions of frequencies for $N_c$ of the Portable dynamic cone penetration tests were investigated for the collapsed embankments.

4. RELATION BETWEEN THE PORTABLE DYNAMIC CONE PENETRATION TEST AND THE SWEDISH WEIGHT SOUNDING TEST

4.1 Purpose

A comparison of the Portable dynamic cone penetration test with the Swedish weight sounding test at the same location is rarely executed, and no correlation equation has been proposed. For the purpose of determining a quantitative estimate equation between these two sounding tests, the soil strengths mentioned above were analyzed, which were obtained at many sites of embankment failures along Japan Railway lines.

Some equations for conversion from $N_c$ value to the other sounding test values for each soil classification were proposed.

4.2 Method of analysis on correlation between different sounding tests

On the relationship between $N_{sw}$ and $N_c$, as $N_{sw}$ is measured for each 25 cm depth, $N_{sw}$ is supposed to equal an average of $N_c$ in that section.

In some special cases where the cone head of the Portable cone penetrometer hits at a large gravel, $N_c$ is a very large and the standard deviation between 10 cms is a large value because of the light weight 49 N. This is affected not only by the ratio between the dynamic compressive pressure and the shear strength during penetration but also by grain size etc. (Muromachi, 1982). By abandoning these singular data, the mean values for all the measured points were analyzed.

For the Swedish weight sounding test, the inertia force of the settling weight may cause the cone to stop at a position deeper than where it stops naturally. Accordingly it is assumed that $W_{sw}$ at a position when the weight begins to settle is equal to $N_c$ at that position.

For the conditions mentioned above, a correlation between $W_{sw}$, $N_{sw}$ from the Swedish weight sounding test and $N_c$ from the Portable dynamic cone sounding test was evaluated. An example of this correlation is shown in Fig. 2.

The correlation between $N_c$ and $W_{sw}$ (kN) or $N_{sw}$ (half-turns/m) is generally expressed as follows:

$$N_c = \alpha W_{sw}$$

$$N_c = \beta N_{sw} + \gamma$$

As $W_{sw}=0.98\text{kN}$ and $N_{sw}=0$ must be equal to $N_c$, the constant $\gamma$ in Eq.(9) is $\gamma = 0.98 \alpha$. Accordingly Eq.(9) is given as follows:

$$N_c = \beta N_{sw} + 0.98 \alpha$$

Using $W_{sw}$ and $N_c$ as $(x_1, y_1)$ at settlement under self weight, and $N_{sw}$ and $N_c$ as $(x_{21}, y_{21})$ at rotation. The sum of squares of the residuals is given as,

$$S = \sum_{i=1}^{\frac{1}{2}} (y_{1i} - \bar{y}_1)^2 + \sum_{i=1}^{\frac{1}{2}} (y_{2i} - \bar{y}_2)^2$$

$$= \sum_{i=1}^{\frac{1}{2}} (x_{1i} - a x_{11})^2 + \sum_{i=1}^{\frac{1}{2}} (x_{2i} - a x_{21})^2$$

$$= \sum_{i=1}^{\frac{1}{2}} (x_{1i} - a x_{11})^2 + \sum_{i=1}^{\frac{1}{2}} (x_{2i} - a x_{21} - 0.98 \alpha)^2$$

(11)
where $\lambda$ and $\mu$ are the data for settlement under self weight and for rotation respectively, $Y_1$ and $Y_2$ are averages of $N_c$ in each case.

Executing a partial differential by equation with $\alpha$ and $\beta$ of Eq.(11) for a minimum of the residuals, we can obtain simultaneous equations for $\alpha$ and $\beta$ as follows:

$$
\alpha = (U - \Delta) \left\{ \frac{1}{2} \sum_{i=1}^{2} x_{1i} y_{1i} + 0.98 \sum_{i=1}^{2} y_{2i} \right\} \sum_{i=1}^{2} x_{2i}^2
- 0.98 \left( \sum_{i=1}^{2} x_{2i} \right) \sum_{i=1}^{2} y_{2i}
$$

$$
\beta = (U - \Delta) \left\{ \frac{1}{2} \sum_{i=1}^{2} y_{1i} + 0.98 \frac{\lambda}{\mu} \sum_{i=1}^{2} x_{2i} \right\} \sum_{i=1}^{2} x_{2i}^2
- 0.98 \left( \sum_{i=1}^{2} x_{2i} \right) \sum_{i=1}^{2} y_{2i}\right\} \sum_{i=1}^{2} x_{2i}^2
$$

where

$$
\Delta = \left( \frac{1}{2} \sum_{i=1}^{2} x_{2i} + 0.98 \frac{1}{2} \sum_{i=1}^{2} y_{2i} \right) \sum_{i=1}^{2} x_{2i}^2
- 0.98 \left( \sum_{i=1}^{2} x_{2i} \right) \sum_{i=1}^{2} y_{2i}\right\} \sum_{i=1}^{2} x_{2i}^2
$$

As the result, we can determine $\alpha$ and $\beta$ from Eq. (12) which expresses a correlation between the Portable dynamic cone penetration test and the Swedish weight sounding test.

4.3 Correlation for each soil classification

Widely scattered data on soil strength which was collected in the collapsed embankments at 67 sites along Japan Railway lines was analyzed. In these data, soils were classified into gravelly grained soil, sandy grained soil and fine grained soil based on the Japanese unified classification system.

By using Eqs.(10) and (12), we can obtain the following equations for the soil classification.

$$N_c = 0.20N_{sw} + 4.0W_{sw} \text{(Gravelly grained soil)} \quad (14)$$
$$N_c = 0.22N_{sw} + 3.0W_{sw} \text{ (Sandy grained soil)} \quad (15)$$
$$N_c = 0.12N_{sw} + 4.0W_{sw} \text{ (Fine grained soil)} \quad (16)$$

Fig.3 shows the relationship in Eq.(14) for gravelly grained soil. The correlation coefficients in Eqs.(14) - (16) are 0.79, 0.76 and 0.87 respectively.
The relationship given by some previous studies mentioned above has a wide spread variation as shown in Fig.4.

The other hand, newly proposed equations Eq.(14)~(16) will give a more quantitative estimate than any previous ones. Accordingly, for the investigation of an embankment slope, we can estimate soil strengths from the Portable dynamic cone penetration test as well as the Swedish weight sounding test.

5. EQUATION FOR CONVERSION FROM $N_{c}$ TO $N_{v}$-VALUE OF THE SPT AND $q_{c}$ OF UNCONFINED COMPRESSION TEST

5.1 An equation for conversion from $N_{c}$ to $N_{v}$-value

(1) Conversion from $N_{c}$ to $N_{v}$-value

Some relationships between $W_{sw}$ and $N_{sw}$ from the Swedish weight sounding test and $N_{v}$-value from SPT were obtained by Iida (1966), Miki (1969) and Ueda (1967) et al., as mentioned above.

An equation for conversion from $N_{c}$ to $N_{v}$-value with an interpolation of Iida's formula is proposed. By using the relationship equations of Eq.(14)~(16) between $W_{sw}$, $N_{sw}$ and $N_{c}$, some conversion equations from $N_{v}$-value to $N_{c}$ are obtained by the substitution of the Iida's equations of Eq.(1) between $W_{sw}$, $N_{sw}$ and $N_{v}$-value.

In the case of $W_{sw} \approx 0.98kN$, a equation for gravelly graded soil is given by substituting 0.98 for $W_{sw}$ in Eq.(14):

$$N_{sw} = 20 + 5N_c$$  \hspace{1cm} (17)

and Eq.(1) becomes as follows;

$$N = 2 + 0.067N_{sw}$$  \hspace{1cm} (18)

Accordingly, using Eq.(14)~(18),

$$N = 0.7 + 0.34N_c \quad (Gravelly \text{ graded soil}) \quad (19)$$

Similarly, for sandy graded soil and fine graded soil,

$$N = 1.1 + 0.30N_c \quad (Sandy \text{ graded soil})$$
$$N = 1.7 + 0.33N_c \quad (Fine \text{ graded soil}) \quad (20)$$

In the case of $W_{sw} < 0.98kN$, we can obtain the following equations similarly;

$$N = 0.58N_c \quad (Gravelly \text{ graded soil})$$
$$N = 0.96N_c \quad (Sandy \text{ graded soil})$$
$$N = 0.75N_c \quad (Fine \text{ graded soil}) \quad (21)$$

The relationships for Eq.(19)~(21) are given in Fig.5. It is useful, then to adopt Eq.(21) at $N \leq 4$ and Eq.(19) and (20) at $N > 4$.

(2) Comparison between the proposed equations and the observations

To clarify the practical usage of the proposed equations, Eq.(19)~Eq.(21), these equations were compared using the results of in-situ sounding tests.

One site is an embankment on the Railway Company, where the subsurface embankment consist of soils contains silty sand in an upper layer and dense silty cobbles in a lower layer. The other site is an alluvial deposit in Tokyo where the soils contain sandy silt in the upper layer and sand in the lower layer.

The Standard penetration test and the Portable dynamic cone penetration test were investigated at the same locations at these sites. One of the results for the sounding tests are shown in Fig.6. The relationship between the $N_{v}$-value and $N_{c}$ value is shown in Fig.5. It is recognized that the proposed equations agree with the observations although the observed $N_{v}$-values of $N > 10$ are slightly larger than the proposed equations for the same $N_{c}$ values.

5.2 Equation for conversion from $N_{c}$ to $q_{c}$

An equation for conversion from of $N_{c}$ to $q_{c}$ (kN/m²) is proposed with interpositions of Eq.(2) of Iida's formula and Eq.(16) for fine graded soil. The results indicates,

$$q_{c} = 2S + 5N_c \quad (W_{sw} \approx 0.98kN)$$
$$q_{c} = 1N_c \quad (W_{sw} < 0.98kN) \quad (22)$$

The relationship of both the equations is shown
6. CONCLUSION

The correlation of soil strength between the portable dynamic cone penetration test and the Swedish weight sounding test was analyzed.

By using the proposed equations, we can determine some relationships between both the sounding tests, and we can confidently conduct sounding tests by the portable dynamic cone penetration test at many points with little labor.

The results obtained in this paper are summarized as follows:

(1) The relationship between the portable dynamic cone penetration test and the Swedish weight sounding test can be expressed by Eqs. (14) – (16). A sounding value obtained by the portable cone dynamic penetration test can therefore be related quantitatively to one by the Swedish weight sounding test.


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Geotechnical and seismic surveys for site characterization

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ABSTRACT: This paper presents combined study of many geotechnical (SPT, CPT), geophysical (seismic refraction (SR), surface waves inversion (SWI), cross-hole (CH), down-hole (DH) and seismocone (SC)) surveys and laboratory (resonant column (RC)) tests for site characterization. These tests have been mainly employed to a test site in order to compare their respective results, to propose specific relationships between geotechnical and seismic parameters and to estimate accurately the geometry and the dynamic soil characteristics.

1. INTRODUCTION

The interpretation of various field and laboratory techniques, for the reliable, project oriented, accurate and low cost, site characterization is of prior importance in geotechnical and recently environmental engineering. Microzonation studies, and the analysis of seismic behavior of spatially extended infrastructures (highways, railways, harbors, pipelines, tunnels, bridges etc.), or even the stockpile of waste polluted materials, necessitate the combined use of many field and laboratory surveys. Most of the geotechnical in situ and laboratory tests and seismic prospecting are conceptually different and their interpretation for engineering application necessitates judicious interpretation of their results.

Due to soil heterogeneity and the lateral variation of the soil formations, Vs and damping measurements from surface wave inversion (SWI) or refraction (SR) techniques, may be slightly different from the measurements performed by the cross-hole (CH) technique, or the discrete values given by laboratory tests. While each method is accurate with respect to its principle, one may seek for local or global information, according to the nature of the problem. A complete site characterization necessitates both. For example, the seismic design of a pile bridge foundation needs very good knowledge of the dynamic properties of the local soils but its response is influenced by the wave-field and the soil-structure interaction effects which necessitate the global knowledge of the site.

The aim of the paper is to present and discuss the results from field and laboratory techniques, for a complete site characterization (geometry and dynamic soil properties). Different field techniques are compared to each other and also with laboratory tests. Cross-hole measurements (Vs) are also compared with seismocone testing. Finally, the paper presents correlation relationships between N60, P and Vs values for silts & sands and clays from a large data base established for Greek natural soils.

2. FIELD AND LABORATORY TECHNIQUES

2.1 Seismic prospecting

The refraction technique is well known from geophysics (Mota 1954). Recently, its use has been extended to engineering oriented projects. An important shortcoming of the method is its inability to determine low velocity strata beneath or between higher velocity strata.

The SWI is widely used in seismology and recently in geophysical prospecting. Theoretical and practical aspects of the method have been described extensively by many researchers (e.g. Mokhtar et al. 1988, Stocker et al. 1988 and Raptakis et al. 1996). This method uses both group and phase velocities of the surface waves in order to retrieve the distributions of shear wave velocity with depth (Herrmann 1985).

As it can be seen in Figure 1, the good fit between experimental and theoretical dispersion curves, is
necessary for the acceptance of an accurate and reliable $V_s$ profile, as it is depicted in Figure 2.

The seismocone testing is a recent and promising development based on the classical cone penetration and piezocone technique. Two geophones separated by a distance of one meter are mounted on a conventional piezocone and they are recording the seismic waves produced by a shear wave generator at the free surface. The source is a horizontally oriented piston acting in two directions and thus providing polarized waves for the accurate estimation of the first SH arrival (Robertson et al. 1985).

2.2 Geotechnical tests

Field geotechnical tests comprising boring, sampling, SPI and CPT measurements were performed following the ASTM standards. Laboratory tests included, besides the conventional classification tests, mainly RC tests. The RC tests were performed using a Drneich type apparatus, following also the corresponding ASTM standard.

3. INVESTIGATION SITES

The data presented herein comes essentially from a test site (EUROSEISTEST), which has been recently established near Thessaloniki, to perform experimental and theoretical studies in engineering seismology and earthquake engineering. The test site lays at a normally consolidated sedimentary valley, 5km wide. The maximum depth of the deposits in the center of the valley is of about 200m. The surficial low rigidity soils consist mainly of silty clays, clays and loose sands. The deeper layers are essentially medium to stiff clays.

Active and seismic faults are crossing the valley, which is a tectonically active, with an "opening velocity" of 0.7cm per year. The estimation of the location of faults and lateral discontinuities is of major importance when studying spatially extended structures. For all these reasons EUROSEISTEST is an excellent site to investigate the most commonly used methods. An extensive investigation program has been performed with all the aforementioned methods (Polidis 1995, Raptakis 1995 & Jongmans et al. 1997).

The comparison between CH and laboratory RC $V_s$ measurements as well as the correlation relationships between N$_{63}$ values and $V_s$ CH measurements refer to an extended data base of natural soils from various sites in Greece. Finally, a comparative study between CH and SC measurements of $V_s$ was performed at another specific site.
4. RESULTS OF 1D AND 2D PROFILES.

The application of the aforementioned methods, data interpretation and techniques in one specific location (EUROSEISTEST) is given in Figure 3.

Numerous field measurements have been performed at various locations at this reference site (EUROSEISTEST). Synthesizing the available geophysical and geotechnical results, the 1D models were constructed for each specific location (Figure 4). These models were built up mainly according the $V_p$ values in conjunction with the geotechnical data.

The construction of the 2D valley cross-section, shown in Figure 5, is based on a previous extensive study of the accuracy of each field technique as well as the analysis of weak and strong motion data from
5. COMPARISONS OF $V_s$ DETERMINED FROM FIELD TESTS

Figures 6 and 7 show that the SWI results are comparable with those from SH refraction and borehole seismics (CH and DH).

5.1. COMPARISONS OF $V_s$ FROM SR (SH WAVES) AND SWI TECHNIQUE

5.2. COMPARISONS OF $V_s$ BETWEEN CH AND SC TESTS
The results of $V_s$ derived from SC and CH tests (Figure 8) are well compared for non-cohesive materials (SG, SM, SP with $N_{SPT} < 2$) as well as for clays (mainly CL, with IC=$10-25$ and $N_{SPT} = 2-21$). For stiffer clays, it seems that the seismic $V_s$ values are in general lower than the corresponding values from CH.

6. COMPARISON BETWEEN FIELD AND LABORATORY TESTS

Shear wave velocity values determined from RC tests on undisturbed good quality specimens are similar with the corresponding CH $V_s$ values at the same depths (Figure 9).

![Figure 9. Comparison of $V_s$ from CH and RC tests.](image)

The material damping is an important parameter, especially for the site effect study. Table 1 presents a comparison between damping measurement from in situ (SWAS) and laboratory (RC) tests. The comparison is successful which makes the SWAS technique very interesting for practical applications.

Table 1: Comparison of critical damping and quality factor between RC test and SWAS tests correspondingly.

<table>
<thead>
<tr>
<th>No</th>
<th>ISCS</th>
<th>Damp. RC (%)</th>
<th>Damp. SWAS (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>CL-ML</td>
<td>7.7</td>
<td>2.0</td>
</tr>
<tr>
<td>2</td>
<td>CL</td>
<td>18</td>
<td>1.5</td>
</tr>
<tr>
<td>3</td>
<td>CL</td>
<td>23</td>
<td>1.2</td>
</tr>
<tr>
<td>4</td>
<td>CL-CL</td>
<td>21</td>
<td>9</td>
</tr>
</tbody>
</table>

A correction factor of 0.75 was applied to correct the measured SPT values due to differences between the actual delivered energy to the theoretical free fall energy. No correction factor for the overburden pressure was used. The data comes from many sites in Greece.

7. $N_{SPT}$-$V_s$ CROSS-HOLE CORRELATIONS

Several empirical $N_{SPT}$-$V_s$ relationships have been proposed the past 20 years (e.g. Inna 1977, Ohia and Goto 1978, Lee 1992 and Raptakis et al. 1994). Figures 10 and 11 present the specific relationships proposed for natural cohesive and non-cohesive soils.

![Figure 10. Correlation between $N_{SPT}$ and $V_s$ from CH tests values for clays.](image)

![Figure 11. Correlation between $N_{SPT}$ and $V_s$ from CH tests values for silts & sands.](image)

CONCLUSIONS

The combined use of field and laboratory tests at specific sites in Greece aiming to test their adequacy and to estimate accurately the geometry and the dynamic soil properties at low shear strains, gave the
opportunity for valuable comparisons and correlations. The main conclusions can be summarized as follows:

1) The level of site characterization is related to the type of structure and the study undertaken.
2) For site response analyses, for which a 2D knowledge is necessary, the low cost surface wave inversion technique offers a very good quantitative knowledge of the main parameters. This makes the above method very attractive for practical applications.
3) Geophysical surveys are indispensable in geotechnical engineering when the global knowledge of the local geology is necessary.
4) SC testing gives comparable results with CH tests except for rather rigid cohesive materials ($V_s > 250$ m/s).
5) RC measurements of $V_s$ are well compared to field CH measurements for normally consolidated recent soil deposits.

6) Although there is a limited number of comparative damping measurements both from in situ and laboratory (RC) tests, the comparison between them seems to be quite good.

7) The correlations between $N_D$-$V_s$ are very useful in practical applications and are similar with relationships proposed by other researchers. The proposed empirical formulae are for Greek soils.

Concluding, the use of advanced geophysical surveys appears to be a low cost tool for a satisfactory knowledge of few basic parameters that we are seeking for in a site characterization.

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New technology of determination of physical and mechanical characteristics of soils

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ABSTRACT: In the given paper some ideas are presented which concern the creation of the new generation of the devices for testing of the soils in situ creating a certain stress-strain state (SSS) in the basement area being tested. The test results of some kinds of the soils are given as well as the measurement result processing theory. In order to raise accuracy and safety of design of the basements and the foundations a new device has been created as well as a method of the determination of physical and mechanical properties of the soils which provide both a complete similarity of the material being tested in the device as compared with its operation in situ as a known SSS. An absolutely flexible and smooth die in an elastic halfspace is the main idea of the device, but the technical realization of this idea is connected with definite difficulties. Besides, a number of problems arise when the strength characteristics of the basement soil are determined (adhesion and angle of internal friction). The device being worked out by the authors and the calculation method allow to determine Young’s modulus, adhesion and angle of internal friction under the field condition in a hole.

1 INTRODUCTION

The standardized tests of the soils with the help of the static loads envisage the measurement of the following physical and mechanical properties of the foundation material:

- modulus of general deformation;
- Poisson’s ratio;
- adhesion;
- angle of internal friction.

The reliable result can be obtained only in the case if the stress-strain state (SSS) created by the device in a sample of soil is known. The majority of the devices used for the tests of the soils (compression devices, adf devices, triaxial apparatus, tough dies) create the stressed deformed condition which has free peculiar points and the regions with the undetermined values of the stresses from the point of view of the strict solutions of the theory of elasticity.

Moreover, sometimes the stressed deformed condition created by a device does not correspond to the conditions of the soil operation in situ.

For example, a device for the soil test with the help of radial thrust in the wells including a pressure chamber with an elastic casing, a measuring fixture for the registration of the radial displacements, a compression station for the specified pressure (see Fig. 171, page 104 [1]).

The impossibility of the soil tests in the pits and on the surface of the soil mass as well as the uncertainty of the SSS of the basement in the field of the edges of the cylindrical pressure meter are the drawbacks of the above-mentioned device.

A device for the soil tests under the conditions of three-axis loading is most similar to the suggested one. It includes the pressure chamber with the elastic casing, the loading and measuring fixture arranged in the lower part of the chamber with the possibility of the interaction of the strain gauge with the elastic casing of the working chamber [2].

The necessity of the preparation of the samples of the unbroken structure is a drawback of the known device of three-axis test of the soils. The soils should be extracted from the geological wells or pits. In its case it leads to the change (breaks) of the natural SSS of the soils.

The purpose of this paper is a working out of such a device which can create a certain SSS in the soil mass which allows to determine the physical and mechanical characteristics of the soils in situ.
2 NEW DEVICE FOR SOIL TEST IN SITU

In order to create a certain stress-strain state (SSS) in the soil mass the authors have worked out a device for the soil tests under the conditions of the dimensional (three-dimensional) operation of the foundation including the pressure chamber, the working chamber with the elastic casing, the upper cover and the lower one, the load and measuring device with the possibility of the interaction of the stress gauge with the elastic casing. The pressure chamber is made in the form of the ring bellows; the working chamber is made as cylindrical bellows; the elastic casing consists of two parts: the ring part and the round one, the stress gauges are situated in the centre and on the adjoining boundary of the working chamber and the pressure chamber. Besides, the upper edge is made as a rest for the upper parts of the working chamber and the pressure chamber. A supposed device is shown in Fig. 1. The device includes upper cover 1, pressure chamber 2, working chamber 3, ring elastic casing 4, round elastic casing 5, strain gauges 6, 7, 8, compressor station 9, pressure reducing valves 10, 11 for the creation of different pressure in the working chamber and the pressure chamber.

The device operates in the following way. The device should be placed with its elastic casings 4 and 5 on the prepared soil platform 12 which can be both in the well (pit) and on the surface of the soil basement. The position of the upper cover is rigidly fixed with the help of rest 13. On the first stage of the tests an equal pressure from the compressor station 9 is delivered into working chamber 3 and pressure chamber 2. The strains are registered with the help of gauge 8 and modulus of elasticity of the soil basement is determined. Then without the pressure change in pressure chamber 2 one should increase the pressure in working chamber 3 with the help of actuator valve 11. The strains are measured with the help of gauges 6 and 7. The time of the plastic strains is determined according to the difference of the readings of gauges 6 and 7, and according to the pressure in working chamber 3 and pressure chamber 2 the strength characteristics of the soil - cohesion and angle of internal friction - are calculated which correspond to three-dimensional stress-strain state of the soil basement.

Thus, we obtain the dimensional mechanical characteristics of the soil under the conditions which are similar to the actual SSS of the basement of the buildings and structures.
3 MEASUREMENT RESULT PROCESSING

As a result of the soil tests with the help of the suggested device, we obtain the following data:
- acting uniformly distributing load on the basement;
- geometry of load;
- time basement settlements in points A, B, C (Fig. 2).

\[ u_i - \int_{\Gamma}^{i} \sigma_i \, d\Gamma, \quad i = 1, 2, 3, \quad (3) \]
\[ \sigma_j - \int_{\Gamma}^{j} \sigma_j \, d\Gamma, \quad j = 1, 2, \ldots, 6, \quad (4) \]

where
- \( u_i, \sigma_j \) are displacements and stresses in point \((x_i, y_i, z_i)\) from the action of a single vertical force \(P\) in point \((x_j, y_j, z_j)\);
- \( P \) is acting uniformly distributed load (pressure in the working chamber and the pressure chamber of the suggested device);
- \( \Gamma \) is boundary of the device contact with elastic medium.

Let us convert the expressions (3) and (4) having taken the constants outside integral and having dropped the coordinates in the brackets:

\[ u_i = \left(\frac{A_e}{E} \right) \int_{\Gamma}^{i} \varphi_i \, d\Gamma, \quad i = 1, 2, 3 \quad (5) \]
\[ \sigma_j = \left(\frac{A_e}{E} \right) \int_{\Gamma}^{j} \varphi_j \, d\Gamma, \quad j = 1, 2, \ldots, 6 \quad (6) \]

The expression (5) is the initial one for the determination of Young’s modulus of the soil basement:

\[ E = \left(\frac{A_e}{u_i} \right) \int_{\Gamma}^{i} \varphi_i \, d\Gamma. \quad (7) \]

In the last formula, Poisson’s ratio being a part of the expression \(A\), is also known (besides, Young’s modulus). But its value for different types of soils is known and is in the limits from 0.3 to 0.4 taking the least values for sands and the largest ones for clays. The integration of Boussinesq’s and Mindlin’s functions in the equation (6), (7) is rather a complicated problem in the general case. One can resort to numerical integration having compiled the necessary tables (charts) for different values of Poisson’s ratio, different depths of the device position and different geometry of the contact surface of the device.

Now let us consider the charts obtained during the laboratory tests of the experimental sample of the device in the tray of the sand soil (Fig. 3).

As it is clear from Fig. 3, “load-displacement” dependence in all the points of the elastic working surface of the device \((A, B, C)\) is of a linear character till the pressure in the device chamber reaches \(p_0 = 60\) kPa. That’s why it is better to determine Young’s modulus according to formula (7) in this very load interval. As for the strength characteristics of the basement material (cohesion - c and angle of internal...
Fig. 3. Results of laboratory tests of sand soil in tray

\[ \tau_{w} = \sigma_{w} \phi + c_{w} \]

where 
\[ \tau_{w} = \frac{1}{3} [(\sigma_{1} - \sigma_{2})^2 + (\sigma_{2} - \sigma_{3})^2 + (\sigma_{3} - \sigma_{1})^2] \]

is tangential stress on the octahedron plate; 
\[ \sigma_{w} = \frac{1}{3}(\sigma_{1} + \sigma_{2} + \sigma_{3}) \]

is mean normal stress; 
\( \sigma_{1}, \sigma_{2}, \sigma_{3} \) are principal stresses.

There are two unknown quantities (\( c \) and \( \phi \)) in formula (8), that's why in order to determine them it is necessary to carry out two tests under different values of \( p_{0} \). The principal stresses \( \sigma_{1}, \sigma_{2}, \sigma_{3} \) are calculated according to the known expression via the components of stress vector \( q_{j} \) which in their turn are calculated according to the formula (6) at the corresponding values \( p_{0} \) and \( p_{f} \).

4 CONCLUSIONS

1. The device suggested by the authors for the soil tests creates a certain stress-strain state (SSS) in the soil mass which allows to determine the strength and strain characteristics of the basement material under the conditions which are similar to the real space work of the foundation basement.

2. The SSS created by the device is determined in accordance with the elastic isotropic halfspace theory by means of the numerical integration (according to the formulas (5) and (6) of the fundamental Boussinesq's (Mindlin's) solutions along the boundary of the device contact with the soil.

3. The deformation characteristic of the soil (Young's modulus) is calculated according to the formula (7) for the load which corresponds to the linear part of curve C on the chart of Fig. 3.

4. Strength parameters of the basement material (cohesion \( c \) and angle of internal friction \( \phi \)) acting on the octahedron plate are determined according to the formula (8) from two tests when the plastic deformations take place (pressure \( p_{0} \)) under the different values of pressure \( p_{f} \).

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Undrained monotonic behavior of sand from self-boring pressuremeter

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ABSTRACT: An analytical procedure for estimating undrained stress-strain behavior of sand from drained self-boring pressuremeter tests (SBPMT) data is proposed. Data from two sites have been analyzed using the procedure to predict the mechanical behavior of an axisymmetric element over a wide range of deformation. The results are compared with laboratory triaxial test data on frozen samples to validate the procedure.

1 INTRODUCTION

Empirical correlations are available for estimating undrained strength of granular soils from in-situ index tests, e.g., piezocene penetration test (CPTU) or standard penetration test (SPT). However, none of these procedures provides information about how the undrained shear strength develops with increasing deformation. In addition, these relationships do not account for stress, stress path and fabric dependency of mechanical response of granular materials. A procedure has been proposed in this paper to overcome these deficiencies. The procedure is based on calibration of a simple stress strain relationship from SBPMT data and supplementary information from seismic CPT. The calibrated model can be used in deformation analysis of an earth structure.

2 MODELING ELASTIC BEHAVIOR

For isotropic elastic material, the strain increment is given by

\[ d \varepsilon = C_{ijkl} d \sigma_{ij} \]

where \( \sigma \) is the effective Cauchy stress and \( D_{ijkl} \) is the elastic compliance tensor. The elastic stiffness tensor, \( C_{ijkl} \), is given by

\[ C_{ijkl} = G \left( \frac{2 \nu}{1 - 2 \nu} \right) \delta_{ij} \delta_{kl} + \delta_{ik} \delta_{jl} + \delta_{il} \delta_{jk} \]

(2)

where \( \delta_{ij} \) represents Kronecker's delta, and \( \nu \) is the Poisson's ratio. The elastic tangent shear modulus, \( G \), for a hyperelastic material can be expressed as

\[ G = K_{CS} P_A \left[ 1 + \left( P_A \right)^{n_k} \right]^{n_k} \]

(3)

where \( K_{CS} \) (elastic shear modulus number) and \( n_k \) (elastic exponent) are model parameters, \( P_A \) is the atmospheric pressure and \( P_1 \) (the first invariant) is the trace of the effective stress tensor.

The value of \( n_k \) is about 0.5 for a number of sands. A value of 0.2 can be assumed for \( \nu \) for many sands irrespective of the void ratio. Parameter \( K_{CS} \) mainly depends on soil type and the state of packing. The appropriate in-situ estimate of \( K_{CS} \) can be found from shear wave velocity measurements from seismic CPT and a reasonable knowledge of in-situ state of stress. To account for the influence of the state of packing on \( G \), the value of \( K_{CS} \) is updated during the deformation process in proportion with \( (2.17 - e)^{(1+e)} \), where \( e \) is the current void ratio.
3 MODELING PLASTIC BEHAVIOR

The plastic behavior in distortion is often modeled by adopting a loading surface that has an appearance similar to a cone in the principal stress space with its apex at the origin. Since the conical loading surface opens out along the hydrostatic axis, the distortion mechanism does not predict plastic deformation in isotropic compression. To account for the plastic deformation in isotropic compression, a second loading surface is introduced. For isotropic materials, the loading surface for isotropic compression takes a spherical shape in the principal effective stress space and is sometimes called the "cap". The plasticity mechanisms for distortion and isotropic compression are assumed to be mutually non-interactive and of the strain hardening type.

3.1 Distortional Behavior

We begin with the definition of the Spatially Mobilized Plane (SMP). SMP is defined such that at its intersection with the plane normal to the principal direction "k" of the effective stress tensor, the ratio, \(\sigma_{k}\sigma_{k}^{-}\gamma_{k}\gamma_{k}^{-}\), is maximized (Nakai and Matsuo, 1983). \(\sigma_{k}\) and \(\gamma_{k}\) are the principal values of \(\sigma_{k}\). The normal (\(\sigma_{\text{SMP}}\)) and the shear (\(\tau_{\text{SMP}}\)) components of the effective stress tensor on this plane and the ratio \(\sigma_{\text{SMP}}/\tau_{\text{SMP}}\), \(\eta\) (often referred to as the stress ratio), are given by

\[
\sigma_{\text{SMP}} = \frac{3}{2} \frac{I_{1}}{I_{2}}, \quad \tau_{\text{SMP}} = \sqrt{I_{1}I_{2}} - \frac{9}{2} \frac{I_{1}}{I_{2}}
\]

\[
\eta = \sqrt{I_{1}/(9I_{2})} - 1 \tag{4}
\]

The second invariant of effective stress tensor, \(I_{2}\), is equal to \((\sigma_{1}\gamma_{1})^{2}\gamma_{1}^{-} + \gamma_{1}\gamma_{2}\) and the third invariant, \(I_{3}\), is equal to the determinant of \(\gamma_{1}\). Constant \(\eta\) lines represent the loading surfaces, i.e., yielding occurs when \(\partial\gamma_{1}\delta\sigma_{1}\geq 0\). Assuming the particles are mobilized to the maximum extent on the average along the SMP, it follows from micro mechanics (Matsuo, 1974) that

\[
\eta = -\lambda (\frac{d\sigma_{\text{SMP}}}{d\gamma_{\text{SMP}}} + \mu)
\]

where \(d\sigma_{\text{SMP}}\) and \(d\gamma_{\text{SMP}}\) are the components of \(d\sigma\) normal and parallel to the SMP. \(\sigma_{\text{SMP}}\) denotes the principal values of the strain increment of the mechanism for distortional plasticity, \(\lambda\) and \(\mu\) are model parameters. Assuming a hyperbolic relationship between \(\gamma_{\text{SMP}}\) and \(\eta\) the instantaneous slope, \(G_{\eta}\), can be calculated from (Salgado, 1990)

\[
G_{\eta} = \frac{G_{\eta} (1 - R_{\eta} \eta / \eta_{p})}{K_{\eta} (\sigma_{\text{SMP}} / P_{\eta})^{n} (1 - R_{\eta} \eta / \eta_{p})^{n}} \tag{6}
\]

where \(G_{\eta}\) is the initial slope of the \(\eta-\gamma_{\text{SMP}}\) curve, \(\eta_{p}\) is the asymptotic value of stress ratio as \(\gamma_{\text{SMP}}\) increases. \(K_{\eta}, n, R_{\eta}\) and \(P_{\eta}\) are model parameters. The stress ratio at failure, \(\eta_{f}\), is assumed to depend on \(I_{4}\), the trace of the effective stress tensor at failure, as follows

\[
\eta_{f} = \eta_{u} - \Delta \eta \log \left( \frac{I_{4}}{(3P_{\eta})} \right) \tag{7}
\]

where \(\eta_{u}\) and \(\Delta \eta\) are model parameters. From Eqs. (5) and (6)

\[
d\sigma_{\text{SMP}} = -\lambda (\sigma_{\text{SMP}} + \mu) d\eta \tag{8}
\]

Assuming the principal directions of \(\sigma_{k}\) and \(\gamma_{k}\) to be the same, the direction cosines of \(d\sigma_{k}\) can be calculated from Eq. (5). Assuming further that \(d\sigma_{\text{SMP}}\) and \(d\gamma_{\text{SMP}}\) are coaxial, \(d\sigma_{k}\) can be calculated from

\[
d\sigma_{k} = \sqrt{\eta_{u}} G_{\eta} (\frac{\eta}{\eta_{u}}) d\sigma_{\text{SMP}} \frac{\sigma_{k} - \gamma_{k}}{\gamma_{k}} = \sqrt{\eta_{u}} G_{\eta} ((-\eta) / \lambda + (\sigma_{k} - \gamma_{k}) / \sigma_{\text{SMP}}) d\eta \tag{9}
\]

where \(\eta_{u}\) is the direction cosine of a unit normal to the SMP with respect to the principal axes and is equal to \(I_{4}((\mu / (\lambda + \mu))^{1/2}\). Differentiating \(\eta\) from Eq. (4) and substituting into Eq. (9), an incremental relationship between the principal strain and the effective stress is obtained. Transformation of the strain increment tensor in the principal stress space, \(d\sigma_{j} = d\sigma_{k} \frac{\partial \alpha_{j} \partial \alpha_{k}}{\partial \alpha_{k} \partial \alpha_{j}}\), to the coordinate axes using \(d\alpha_{j} = N_{k} d\alpha_{k}\) yields

\[
d\sigma_{j} = C_{ij} d\sigma_{j} \tag{10}
\]

where \(N_{i}\) is the direction cosine of the principal direction "i" with respect to the j-th coordinate. Parameters \(\lambda\) and \(\mu\) are not affected by the inherent anisotropy (Nakai and Matsuo, 1983). Studies also suggest that the quantity \(\eta_{u}\) may not be significantly
affected by inherent anisotropy (see, e.g., Been and Jefferies, 1985). Thus, Eq. (8) only needs to be modified to capture inherent anisotropy. Following modification to Eq. (6) can thus be proposed

\[
G_{yy} = G_{yy}(1 - R_{y} \eta / \eta_y)^2
\]

\[
= \eta_A K_{yy} (1 - R_{y} \eta / \eta_y)^2
\]

(11)

where the multiplier \( \eta_A \) depends on \( \theta \) (the angle between the depositional direction and the normal to the SMD) and an additional model parameter, \( n_A \), as follows for \( \theta = 0^\circ \) or \( 45^\circ \) (for \( \theta = 45^\circ \), \( n_A = 1 \))

\[
n_A = 1 - (n_A - 1)(2 \cos^2 \theta - 1)
\]

(12)

The value of \( \theta \) is updated at the end of a time step.

3.2 Behavior in Isotropic Compression

To evaluate the component of plastic strain, \( \epsilon_{pl} \), due to isotropic compression, an associated plasticity model proposed by Lade (1977) for isotropic materials is used. The loading surface for this formulation is given by

\[
f_c - I_1^2 - 2I_2 = \sigma_1' \sigma_2' \]

(13)

The relationship essentially represents a family of spherical surfaces in the effective principal stress space. Section of the loading surfaces for the distortion (AOD) and consolidation mechanisms (AB) in triaxial plane are shown schematically in Figure 1. Abbreviations “EXC” and “TXC” are used to denote triaxial compression and extension, respectively. It can be shown that the plastic strain increment, \( \epsilon_{pl} \), can be found from

\[
\epsilon_{pl} = \frac{C_C}{2} \left( \frac{\sigma - \sigma_{yy}}{P_A} \right)^{n_A} \left( \frac{\sigma_{yy}}{P_A} \right)^{n_A - 1} \frac{\partial \sigma_{yy}}{\partial \sigma_{yy}} \frac{\partial \sigma_{yy}}{\partial \sigma_{yy}} \epsilon_{yy}'
\]

(14)

Strain increments from Eqs. (1), (10), (14) are added together to calculate the total strain increment.

3.3 Plastic Model Parameters

Two triaxial compression, one triaxial extension, and two isotropic compression on undisturbed samples are needed for calibration of the stress-strain relationship described above. Such an elaborate laboratory testing program is seldom feasible. An approximate calibration procedure in the absence of necessary and sufficient information is outlined below. Information available for calibrating a stress-strain relationship often include data from undrained laboratory element tests on undisturbed samples or in-situ self-boring pressuremeter tests, which are not conventionally used in calibration of constitutive models. These data can be used in a calibration exercise if reasonably precise information about the bounds of values of the model parameters is available. Therefore, existing information on approximate values of model parameters needs to be documented.

C and p: Examination of isotropic consolidation tests shows that the parameter, C, is mainly affected by the relative density, \( D_r \), and grain compressibility. The exponent, \( p \), on the other hand, is affected primarily by grain compressibility only. A relationship is proposed in Figure 2 between parameter C and \( D_r \) that can be used in the absence of more precise information. A value of 0.9 for parameter \( p \) appears to be appropriate for granular materials with medium compressibility. The corresponding value for highly compressible soils is 0.65.

Guidelines for selecting approximate values of \( \lambda \), \( \mu \), \( \eta_1 \), \( \eta_2 \), \( \lambda_1 \), \( \eta_1 \), and \( \eta_2 \), from a minimal material specific information are as follows. Once the approximate values of these parameters are identified, model calibration simplifies greatly because iteration over the remaining model parameter, \( K_m \), is only necessary to fit the model to data. \( \lambda \) and \( \mu \): Nakai and Matsukawa (1983) showed that Eq. (5) does not depend on void ratio or sample fabric and it is affected only by soil type. Hence \( \lambda \) and \( \mu \) can be
evaluated from a suitable drained element test, e.g., triaxial or plane strain, on reconstituted specimens. Typical values of \( \lambda \) and \( \mu \) for many types of sand have been summarized in Roy (1997).

\[ \eta = 2 \sqrt{2} \tan \phi' / 3 \]  

(15)

\( \eta_{w} \) and \( \Delta \eta \) in TNC, \( \eta \) relates to the mobilized effective stress friction angle, \( \phi' \), by

Since \( \phi' \) and \( \phi_{\text{cv}} \) relates to the relative density, \( \phi_{\text{p}} \), and \( \phi_{\text{cv}} \) are estimated from the knowledge of \( \phi_{\text{p}} \). The relative density of a deposit can be estimated from the cone tip resistance measured in a cone penetration test. Symbols \( \phi' \), \( \phi_{\text{p}} \), and \( \phi_{\text{cv}} \) denote the values of the effective stress friction angle and \( \phi_{\text{p}} \) at failure and the steady state friction angle, respectively. Failure is defined as the instance when the peak value of \( \phi' \) is mobilized. The value of \( \phi_{\text{cv}} \) is estimated from the knowledge of mineralogy. Estimates of \( \phi_{\text{cv}} \) are in fact available in the literature for several types sand (see, e.g., Salgado, 1990; Santhasar et al., 1994).

The drained angle of internal friction measured in a laboratory element test is usually higher than the corresponding undrained value. Since \( \eta \) relates directly to the peak friction angle in triaxial compression via Eq. (15), model parameters \( \eta_{w} \) and \( \Delta \eta \) are expected to be depend upon the drainage condition. Examination of a number of laboratory element test data on several sands leads to the following relationship (coefficient of determination between the two variables, \( R^2 = 0.94 \))

\[ (\eta_{w} - \eta_{\text{cv}})_{\text{undrained}} = 0.46 (\eta_{w} - \eta_{\text{cv}})_{\text{drained}} \]  

(16)

The quantity, \( \eta_{w} \), is obtained using \( \phi_{\text{cv}} \) instead of \( \phi' \) in Eq. (15). Estimates of \( \eta_{w} \) and \( \Delta \eta \) pertinent to a certain drainage condition can be obtained by modifying the correlation suggested by Bolton according to Eq. (16).

\( \phi_{\text{p}} \) and \( \phi_{\text{cv}} \), primarily depends on \( \phi_{\text{p}} \). Since the parameter governs the magnitude of irreversible distortion at peak stress ratio, which in turn is not significantly affected by sample fabric and stress path, the effect of stress path and fabric on \( \phi_{\text{p}} \) is expected to be minimal. Experience with the use of stress strain relationship similar to that described earlier (Sridhar, 1994) appears to indicate that a value of 0.75 is appropriate for \( \phi_{\text{p}} \) for very dense cohesionless soils and a value near unity for very loose deposits. In the absence of material specific information, for an approximate estimate of \( \phi_{\text{p}} \), linear interpolation is used in this study setting \( \phi_{\text{p}} = 0.0 \) at \( \phi_{\text{p}} = 0 \% \) and \( \phi_{\text{p}} = 100 \% \). Previous experience also suggests that \( \phi_{\text{p}} \) ranges between -0.3 and -0.6 for many sands (Salgado, 1990; Sridhar, 1992). Roy (1997) could simulate a large number of laboratory triaxial and plane strain tests on several sands using \( \eta_{w} = 0.5 \).

Analysis of a large number of laboratory triaxial tests on undisturbed samples indicates that a value of 2.0 is typical for deposits formed in a hydraulic deposition process such as spitsotting of mine tailings or fluvial deposition of channel sands (Roy, 1997).

4 UNDRAINED BEHAVIOR FROM SBPMT

The stiffest and softest cavity expansion test data from self-boring pressuremeter tests at J-Pit (a deposit spitted Syncrude Sand near Port McMurray, Alberta) and KIDD #2 (a Holocene channel deposit of Fraser River Sand near Vancouver, BC) were analyzed as plane strain problem. At these sites \( D_{\phi} \) varies between 30 and 65 % and the sand are of medium compressibility. Details of the self-boring pressuremeter testing and the numerical model used in their back-analysis can be found in Roy (1997). The back-analysis procedure involves fitting the model response in cylindrical cavity expansion by varying \( K_{\text{cv}} \) manually until a reasonable match between the
computed and observed material response is obtained. Estimates of the other parameters are obtained from seismic measurements and estimates of relative density from seismic piezocone penetration tests performed at adjacent locations and guidelines listed above. The results of back analysis are shown in Figure 3, in which the cavity strain (radial deformation at the cavity wall divided by the original radius of the cavity) is plotted against the corresponding cavity pressure. The depth

![Figure 3. Simulation of SBPMT](image)

using the parameters obtained from the back analysis, the undrained triaxial behavior of Syncrude tailings and Fraser River sand is estimated. (shown as broken lines in Figure 4). Undisturbed samples were extracted via ground freezing from locations adjacent to the SBPMTs and triaxial compression and extension tests were conducted in the laboratory. Tests on samples with relative density at consolidation, D_{rc}, similar to those pertaining to the SBPMTs are shown in Figure 4. The samples were anisotropically consolidated to a vertical effective stress, \(\sigma_{v,con}^{'}\), as shown in Table 2 and a horizontal effective stress (\(\sigma_{h,con}^{'}\)) of 0.5 \(\sigma_{v,con}^{'}\). The reasonable agreement between predicted and measured response in Figure 4 suggests that fundamental soil parameters obtained of SBPMTs are also indicated in the figure. Table 2 shows the parameters used in back analysis are listed in Table 1.

At J-Pit, the effective vertical geostatic stress, \(\sigma_{v}^{'}\), at 4.53 and 5.18 m were taken as 44 and 54 kPa, respectively with a K_{0} of 0.5. At KIDD # 2, for the soft layer (14.37 m) \(\sigma_{v}^{'}\) and \(K_{0}\) were taken as 137 kPa and 0.5, respectively. The corresponding values at 16.375 m depth are 155 and 0.7.

![Figure 4. Predicted and Measured Triaxial Stress-Strain Curves](image)

**Table 1. Model Parameters from SBPMT**

<table>
<thead>
<tr>
<th>Site</th>
<th>D_{rc} %</th>
<th>K_{0}</th>
<th>(\eta_{P}^{'})</th>
<th>(\Delta \eta^{'})</th>
<th>K_{sp}</th>
<th>R_{P}</th>
</tr>
</thead>
<tbody>
<tr>
<td>J-Pit</td>
<td>30</td>
<td>400</td>
<td>0.52</td>
<td>0.01</td>
<td>175</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>500</td>
<td>0.62</td>
<td>0.06</td>
<td>1000</td>
<td>0.87</td>
</tr>
<tr>
<td>KIDD</td>
<td>30</td>
<td>750</td>
<td>0.71</td>
<td>0.06</td>
<td>240</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>1100</td>
<td>0.79</td>
<td>0.10</td>
<td>1000</td>
<td>0.82</td>
</tr>
</tbody>
</table>

Note: For Syncrude Sand, \(\phi_{w}=28^\circ\), \(\lambda=0.85\) and \(\mu=0.29\). The corresponding values for Fraser River Sand are 32\(^\circ\), 0.77 and 0.39, respectively.

**Table 2. Particulars of Triaxial Tests**

<table>
<thead>
<tr>
<th>Site</th>
<th>Test No.</th>
<th>D_{rc} %</th>
<th>(\sigma_{v,con}^{'}) kPa</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>J-Pit</td>
<td>F55C1B31</td>
<td>31</td>
<td>44</td>
<td>TXE</td>
</tr>
<tr>
<td></td>
<td>F55C1B32</td>
<td>31</td>
<td>44</td>
<td>TXC</td>
</tr>
<tr>
<td></td>
<td>F526C211</td>
<td>58</td>
<td>40</td>
<td>TXE</td>
</tr>
<tr>
<td></td>
<td>F526C2B4</td>
<td>65</td>
<td>48</td>
<td>TXC</td>
</tr>
<tr>
<td>KIDD</td>
<td>K94F1C70</td>
<td>67</td>
<td>150</td>
<td>TXE</td>
</tr>
<tr>
<td></td>
<td>K94F1C23</td>
<td>53</td>
<td>120</td>
<td>TXC</td>
</tr>
<tr>
<td></td>
<td>K94F3C4B2</td>
<td>36</td>
<td>156</td>
<td>TXE</td>
</tr>
<tr>
<td></td>
<td>K94F2C2A</td>
<td>40</td>
<td>145</td>
<td>TXC</td>
</tr>
</tbody>
</table>
from pressuremeter tests can be used to estimate undrained element response. The estimated values of the parameters are useful in finite element or finite difference effective stress analysis of static liquefaction problem.

5 CONCLUSIONS

A realistic stress-strain model is proposed in this study for frictional materials with inherent anisotropy. Guidelines for the selection of appropriate values of a majority of model parameters are also documented. Necessary and sufficient data are seldom available for calibration of a stress-strain model such as that described in this paper. In such a situation, the constitutive model can be reasonably calibrated utilizing the guidelines and available test data. A procedure is suggested for calibrating the model from self-boring pressuremeter data. The procedure has been validated by comparing the computed triaxial response with model parameters back figured from SBPMT's with the observed laboratory behavior of undisturbed samples. The proposed method based on inverse modeling of SBPMT can be useful in the assessment of static liquefaction potential; a problem in which the traditional empirical procedures based on in-situ tests such as SPT and CPTU have not been very successful.

ACKNOWLEDGEMENTS

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REFERENCES


Estimation of cyclic strength of sand from self-boring pressuremeter tests

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ABSTRACT: The angle of dilatancy is easily estimated from drained self-boring pressuremeter test (SBPMT) data. The angle of dilatancy is found to correlate quite well with the normalized and energy corrected blow count, \( (N)_d \), measured in a standard penetration test (SPT) for world wide data. Using the correlation between self-boring pressuremeter can be used as a screening tool for identifying deposits susceptible to cyclic liquefaction.

1 INTRODUCTION

The state of packing has a significant influence on the volumetric behavior of granular deposits and is therefore a very important index for liquefaction resistance of the material. Vaid et al. (1981) demonstrated that the dilation angle at large strain, \( \gamma \), from laboratory simple shear data can be correlated to the state of packing. Based on this observation it was suggested that \( \gamma \) can also be used as an index for resistance of granular soils against cyclic liquefaction. Since the dilation angle at large strain can also be estimated from back analysis of SBPMT, it appears that self-boring pressuremeter can also be used as a screening tool for identifying liquefi able deposits. Examination of world wide data (Table 1 provides a summary of the study sites) indicates that the value of \( \gamma \) estimated from SBPMT using a very simple isotropic linear elastic perfectly plastic stress strain relationship due to Carter et al. (1986) correlates rather well with \( (N)_d \). Using the correlation a chart between cyclic strength of granular deposits and the value of large strain dilation angle estimated from SBPMT is proposed. The proposed chart is validated using two recent earthquake case history.

It should be noted that index tools such as SPT and piezocone are in use for a number of years and it is not the objective of this paper to suggest that SBPMT is a better alternative. It needs to be recognized however that SBPMT provides stress deformation data over a wide range of deformation and can in principle be used to calibrate a stress-strain relationship. One such procedure is already developed by Roy (1997) for monotonic loading. If a calibration procedure can be developed for cyclic loading, the calibrated stress-strain model can be used in an elaborate deformation analysis for those deposits perceived to be susceptible to cyclic liquefaction without the necessity to undertake an expensive laboratory testing program.

The study undertaken here is expected to show that the conclusions drawn from analysis of SBPMT are consistent with the inference from tests such as SPT at the elementary level. However, due to the reasons mentioned above, SBPMT data can be used in a more comprehensive manner as and when practical procedures for calibration of a realistic stress-strain model using SBPMT data becomes available.

2 BACK ANALYSIS OF SBPMT

For isotropic linear elastic perfectly plastic Mohr-Coulomb material without cohesion governed by Rowe's dilatancy, Carter et al. (1986) derived the following relationship between the cavity strain, \( \varepsilon_c \), (radial deformation at cavity wall divided by the
Table 1. Site particulars

<table>
<thead>
<tr>
<th>Site</th>
<th>F.C</th>
<th>Dₚ₀</th>
<th>φᵥυ</th>
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Note: 1. F.C = percent (by weight passing # 200 US sieve)
2. φᵥυ (constant volume friction angle) relates to the effective stress friction angle, φ⁰, and v by Kᵥ = Kᵥ = Kᵥ, where
   Kᵥ = (1-sin φ')(1+sin φ')
   Kᵥ = (1-sin φ')(1+sin φ')
   Kᵥ = (1-sin φ')(1+sin φ')
   Kᵥ = (1-sin φ')(1+sin φ')

original cavity radius), and effective cavity pressure (gas pressure within the inflatable section of the probe minus the ambient pore water pressure), P⁰:

\[ \tau₀ = \frac{\phi_0 \cdot \sin \phi'}{2G} \left[ \left( P/P' + \sigma''_n \right)^{1-K_0} \times \frac{X}{Z} - \left( P/P' + \sigma''_n \right)^{1-K_0} \right] \]

where X = 2(1+Ω(Κᵥ×Κᵥ+1)), Y = 2Ω(Κᵥ×Κᵥ+1), Z = 1+Κᵥ×Ω = (1-μ)(Κᵥ×Κᵥ+1)-μ(Κᵥ×Κᵥ+1), σ''_n = σ''_n(1 + sin φ'), and φ₀ = effective horizontal geostatic stress.

The model uses a single valued shear modulus, G, and Poisson’s ratio, μ, over the entire volume of deforming soil surrounding the cavity. This stress-strain model is based on small strain assumption and pertains to undisturbed deposits. As evident from Eq. (1), the model parameters include average values of the shear modulus and Poisson’s ratio representative of the entire soil mass, the peak angle of internal friction and the dilatation angle.

Using G to be approximately equal to 0.7 times the undrained shear modulus for 20% unload modulus at εᵥ=5%, and μ = 0.25, very reasonable values of the strength parameter φ₀ can be derived for many types of sand (da Cunha, 1994, Hughes et al., 1994). Analysis of a self-boring pressmeter cavity expansion response essentially involves varying φ₀, v and φ₀ until a reasonable match is obtained between the response predicted by Eq. (3) and that observed SHPMT. In the following it is examined whether the angle of dilatancy estimated following the procedure described above can be used as an index for liquefaction resistance.

The stress-strain relationship used in this study does not capture the stress path and fabric dependency in soil behavior. Hence, the results of the back analysis are only applicable if the problem geometry is similar to that in a cylindrical cavity expansion, i.e., plane strain extension. However, noting the fact that at large deformation the fabric effects tend to vanish, the value of the large strain dilatation angle estimated from back analysis of SHPMT appears to be generally applicable in a plane strain problem. In a critical application, the data can in principle be analyzed using a constitutive model capable of accounting for stress path and fabric dependency to avoid this limitation.

3 RELATIONSHIP BETWEEN V AND (Nᵥ₀)

Vaid et al. (1981) identified a correlation between the value of the dilatation angle at shear strain of 10% and (Nᵥ₀) proceeding as follows. A linear relationship between v₀ (the value of the dilatation angle normalized for an effective vertical stress of 1 atmosphere pressure) and relative density, Dᵥ₀, from laboratory simple shear experiments on reconstituted Ottawa Sand samples was first postulated. The stress level
correction for the dilation angle was approximated by \( \nu - \nu_p = 2.5^\circ \). Then utilizing a linear relationship between the relative density and \( \log(N_d) \) that is not specifically developed for Ottawa sand, a linear relationship between \( \log(N_d) \) and \( \nu_p \) was proposed. The symbol, \( N_d \), is used to denote SPT blow count normalized for 100 kPa effective vertical stress. Since \( N_d \) is a reasonable index of liquefaction resistance (Seed, 1975), Vaid et al. (1980) suggest a correlation between the large strain dilation angle and liquefaction potential based on the relationship they identified between \( N_d \) and \( \nu_p \). Except for the following approximations in this approach, the procedure can be readily adapted for SBPMT.

- The scheme for stress normalization for the dilation angle is approximate.
- Although a relationship between \( \nu_p \) and \( D_k \) for Ottawa Sand was used, to develop a relationship between \( \nu_p \) and \( N_d \), a global correlation between \( N_d \) and \( D_k \) was utilized. Unless the sands for which the correlation between \( N_d \) and \( D_k \) is developed are of the same compressibility as Ottawa sand, its use may lead to ambiguity (see e.g., Vaid et al., 1985).
- The correlation proposed by Vaid et al. does not use energy-corrected SPT blow count. Since energy delivered to the drill rod while conducting an SPT can vary over a wide margin depending on testing equipment and procedure, energy correction to SPT blow count is at present perceived to be of minimum necessity for ensuring accuracy.

A direct correlation between an estimate of dilation angle from SBPMT and \( (N_d)_o \) can largely resolve these problems. In the simple isotropic elastic perfectly plastic stress strain relationship used in this study to back analyze SBPMT, \( \nu \) remains constant after failure, prior to which there is no relevance of this parameter. It is the constant post-failure value of \( \nu \) that is used in what follows as an index of resistance to cyclic liquefaction. Since the dilation angle derived from SBPMT is only used here as an index for soil liquefaction resistance, the fact that the stress strain relationship does not account for variation of \( \nu \) with strain in a realistic manner should not be viewed as an approximation.

To obtain \( \nu_p \) from \( \nu \) determined from back-analysis of SBPMT, the curvature in the Mohr-Coulomb failure envelope needs to be accounted for. For a large number of cohesionless soils, the difference between \( \phi^* \) and \( \phi_o \) decreases to zero as the effective mean normal stress increases from 1 atmosphere to 100 atmosphere (Byrne et al., 1987, and Vesic and Clough, 1968). This observation is utilized in this study to normalize the dilation angle for the effective vertical stress.

To explore the possibility of developing a relationship between \( \nu_p \) and \( (N_d)_o \) for the statistical behavior of these variables is first examined. Figure 1 shows the cumulative frequency distributions of \( \nu_p \) and \( (N_d)_o \) for in-situ test data from eleven sand sites. The vertical scale of the plot is such that a normally distributed data set will plot on a straight line. The linearity of the cumulative frequency distribution of the log-transformed variables indicates a log-normal distribution of \( \nu_p \) and \( (N_d)_o \). Thus, a correlation between \( \log(\nu_p) \) and \( \log((N_d)_o) \) can be postulated. Linear regression of \( \log(\nu_p) \) and \( \log((N_d)_o) \) yields the following relationship

\[
\log \nu_p = 0.57 \log ((N_d)_o) + 0.25
\]

The value of the coefficient of determination, \( R^2 \), for this relationship is 0.82. Eq. (2) pertains to non-plastic cohesionless materials and the correlation does not depend on the fines content. Data used in the development of Eq. (2) and the correlation are plotted in Figure 2.
4 LIQUEFACTION POTENTIAL FROM SBPMT

Seed et al. (1985) proposed a chart relating \( (N_d)_{eq} \) with the cyclic stress ratio needed to trigger liquefaction, that essentially embodies the current state of practice for identifying non-plastic or low-plasticity liquefiable deposits from SPT. A modification to the chart has been proposed recently by the National Center for Earthquake Engineering Research (NCEER), which introduces a cut-off for the critical stress ratio axis at \( v_\alpha = 0.05 \) (Finn, 1996).

Using Eq. (2), the NCEER chart has been replotted as shown in Figure 3 with \( v_\alpha \) as the index for liquefaction resistance instead of \( (N_d)_{eq} \). Figure 3 can be used to evaluate the liquefaction potential of cohesionless soils from self-boring pressuremeter data.

5 VALIDATION

Widespread liquefaction have been reported at Kobe in the magnitude 7.2 Hyogoken Nanbu event of January 17, 1995. Although liquefaction was reported at the north-northwestern and southeastern areas of Treasure Island near San Francisco, the deposit at other areas performed reasonably well during the magnitude 7.1 Loma Prieta event of October 17, 1989 (Hryciw et al., 1991). SBPMTs at a site in Kobe and a location in Treasure Island that did not liquefy during the Loma Prieta earthquake are used to validate the proposed chart (Figure 3). Laboratory cyclic simple shear test data on samples extracted by ground freezing from Massey Tunnel and J-Pit (Vaid et al., 1990) are also used to check the accuracy of the proposed procedure. The observed field performance of the deposits in Kobe and Treasure Island and the cyclic simple shear data are superposed to the proposed chart (Figure 3). A reasonable performance of the chart is apparent from the comparison.

The above exercise should not be construed as a suggestion that SBPMT is a better screening tool for identifying deposits susceptible to cyclic liquefaction than SPT. However, as mentioned earlier, SBPMT data pertain to stress-strain measurement over a large range of strain at a given depth. In contrast, index measurements such as SPT \( (N_d)_{eq} \) is essentially a single measurement pertaining to a certain average value of strain in the surrounding soil. A potential therefore exists of calibrating a realistic stress-strain relationship from SBPMT data. A calibration procedure has already been developed for monotonic loading by Roy
6 CONCLUSIONS

A relationship has been identified between the angle of dilatancy estimated from inverse modeling of the self-boring pressuremeter data and standard penetration test blow count using data from eleven sand sites. Using this correlation, a chart has been proposed to assess the liquefaction resistance of cohesionless deposits from self-boring pressuremeter tests. Laboratory test data on frozen samples from two sites in Western Canada, field performance of a site in Kobe during the Hyogoken Nambu event of 1995 and field performance at Treasure Island near San Francisco during the 1989 Loma Prieta event were used to validate the procedure.

It is evident from the results that the inference drawn from SBPMT data are consistent with the observations in SPT at several sand sites worldwide. SBPMT is thus a good screening tool for identifying deposits susceptible to cyclic liquefaction as SPT. However, since an SBPMT stress strain data are measured over a wide range of deformation at a given depth, there is a potential to develop a procedure for calibration of a realistic stress strain relationship from SBPMT. The calibrated model can be used to estimate deformation response of the deposit for important structures on soils susceptible to liquefaction if needed.

ACKNOWLEDGMENTS

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Characterizing stiffness and strength of soft Bangkok clay from in-situ and laboratory tests

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ABSTRACT: This paper presents some preliminary results of in-situ and laboratory tests on soft Bangkok clay obtained through an internationally co-operation research program. Various kinds of in-situ tests as well as soil samplings were carried out in an attempt to characterize profile of stiffness and strength with depth at two sites, AIT campus and Nong Ngu Hao (NNH) site; the latter is planned to be Second International Airport in Bangkok. The results show that the profile of elastic shear modulus, \( G_{max} \), was about 5.5 to 11 times of dilatometer modulus, 1.5, and the rigidity index, \( G_{max}/\sigma_s \), ranged from 300 to 500. The undrained shear strength, \( S_u \), from in-situ vane test can also be correlated to the cone tip resistance using the cone factor, \( N_{ct} \), of 11 and 17 for AIT area and NNH area, respectively.

1. INTRODUCTION
Soft Bangkok clay is well-known world-wide in geotechnical engineering field. However, little information is currently available on the stiffness at small strains, despite that this property is the key for the design of diaphragm wall being constructed in some on-going mega-projects in Bangkok area such as metro, sewage tunneling, etc. This fact triggered the execution of this internationally co-operative site investigation program, some preliminary outputs of which are described in this paper. The site investigation was carried out in December, 1996 at two sites, AIT campus located about 40 km to the North from the city centre and Nong Ngu Hao (NNH) site about 15 km to the East.

2. GENERAL SOIL PROFILE
Subsoil in Bangkok area consists of Quaternary deposits which originated from sedimentation at the delta of the ancient river in the Chao Phraya. The Chao Phraya Plain in Bangkok consists of a broad deep basin which filled with sedimentary soil deposits from alternate layers of sand, gravel and clay. The depth of bed rock surface is approximately to be between 550 m to 2010 m. The marine clay is the uppermost clay layer, and it is generally found in the lower deltaic area of the Bangkok Plain which extends from 200 to 250 km in the East-West direction and 250 to 300 km in the North-South direction. The thickness of the soft to medium stiff clay in the upper layer varies from 12 to 20 m while the total clay layer including the lower stiff clay is about 15 to 30 m. (Balasubramaniam et al., 1996).

The main clay minerals in the Bangkok clay as reported by Holmberg (1977) was chlorite, illite, with some montmorillonite. The organic content varies from 3% to 5%. Sensitivity of the marine clays can be high, as much as a factor of 8, but quick clay is not observed in this region. According to Bjerrum (1973), Bangkok clay may be termed as "Aged Normally Consolidated Clay".

2.1 Properties of Soil at Nong Ngu Hao Test Site
The soil profile at NNH was characterized into sub-layers (Fig.1). The weathered clay serves as crust from the ground surface to about 2 m depth. It is a brownish-gray weathered crust. The soft clay layers which extend from 2 to 11.5 m depth are dark grey color with shells and organic matter. The natural water content reaches in excess of 100% being close to the liquid limit. The undrained shear strength from field vane shear test increases with depth from 13 kPa to 27 kPa. Stiff clay layer lies at the depth about 15.5 to 21 m depth with light-brown color.
It is overconsolidated clay. Dense sand layer is found at 21 m depth until the end of boring at 30 m depth. The layer is brownish color consisting of fine to medium sand.

The ground water level at this site investigation was at 0.6 m depth. The piezometric pressure distribution is shown in Fig. 3. Due to the excessive pumping, the piezometric pressure is lower than the hydrostatic pressure below 6 m depth.

2.2 Properties of Soil at AIT Test Site

The soil profile with depth is shown in Fig. 2. Weathered crust starts from ground surface to 3.5 m depth, underlain by soft clay layer of thickness
about 3.5 m. The thickness of medium clay is about 3.5 m extending from 7 to 10 m in depth. The thickness of the stiff clay extends from about 10 to 17 m then underlaid by sand layer. Below 20 m is the hard soil. Sand lenses of 2 cm to 3 cm thick can be found.

To the depth of 8m, the liquid limit of soil varies from 82% to 103%, and the plasticity index varies from 58% to 78%, which classifies the soil as high plastic clay (CH). However, in some layers, organic soil can be seen which groups the soil in high plastic clay (OH). The natural water content lies within the range between 35% and 96%.

The ground water level was found at 1.9 m depth and the piezometric pressure was also lower than the hydrostatic pressure below 8 m depth (Fig.3).

3. TESTING PROGRAM
The program consists of in-situ and laboratory tests. At each site, the following tests were carried out.
In-situ tests:

1-1 A total of four piezocone tests using the standard cone with 10 cm² tip base area, 150 cm² surface area of sleeve friction and 60° apex angle. The rate of penetration was 0.02 m/s. The output results obtained were cone resistance q_c, sleeve friction f_s, and pore pressure u_w.

1-2 A total of three dissipation tests at 3 locations using the piezocone to estimate horizontal coefficient of consolidation c_h and permeability k_h. The piezocone was stopped occasionally for monitoring the decrease of the excess pore pressure with time.

1-3 Field vane test at 3 locations with 0.5 m interval in depth. The vane had the dimension of 5 cm in diameter and 10 cm in height. The vane was rotated at an approximate rate of 0.17°/sec.

1-4 Two flat dilatometer tests (DMT) to estimate undrained shear strength S_u and Young’s modulus, E. The dimension of the conventional Marchetti’s dilatometer blade was 94 mm wide, 15 mm thick and 235 mm long.

1-5 Seismic cone developed at PHRI (Tanaka et al., 1994) was used to estimate the elastic shear modulus G_m. The measurement was taken at every 0.5 m in depth.

Laboratory tests

1-1 Constant volume direct shear box test to estimate undrained shear strength S_u.
1-2 Unconfined compression test to obtain undrained shear strength S_u.
1-3 Bender element test to evaluate G_m of laboratory samples.
1-4 Triaxial compression/extension test to obtain S_u and G -γ relationship.
1-5 Oedometer tests to estimate the overconsolidation ratio, OCR.
1-7 Index tests

Methods for soil sampling

The samples were taken using Shelby tube sampler with a diameter of 76 mm. Alternative method using a fixed piston thin wall tube sampler having a diameter of 76 mm was also employed. It aimed at evaluating sample quality retrieved by these two different sampling methods.

4. DESIGN PARAMETERS

4.1 Overconsolidation Ratio (OCR)

Fig.6: Profile of overconsolidation ratio

At NNH site, the OCR value estimated from conventional oedometer test using the sample taken from Shelby tube was very high about 7 at the crust layer and decreased quickly to 2.7 at the depth of 2.5 m. The value further decreased with depth to the value close to unity. At the same site, the comparative results using samples taken by the fixed piston sampler as tested by the method of the Constant Rate of Straining (CRS) gave higher values of OCR (Fig.6).

At AIT site, the OCR value near to the crust was about 3. It decreased with depth, except at the depths of 5.5 m and 7.5 m, where the value of OCR suddenly increased.

4.2 Elastic Shear Modulus at very small strain G\text{\scriptsize{max}}

The profile of G\text{\scriptsize{max}} with depth was directly measured using the seismic cone, from which G\text{\scriptsize{max}} is given by

\[ G_{\text{\scriptsize{max}}} = \left( \gamma / \gamma_c \right)^{-\frac{1}{2}} \]  

γ, γ_c: total unit weight of soil, gravitationnal acceleration, V_s: in-situ shear wave velocity.

Tanaka et al. (1994) have proposed empirical correlations of \( G_{\text{\scriptsize{max}}} = 50(\varphi, \sigma) \) and \( G_{\text{\scriptsize{max}}} = 7.5E_0 \) on the basis of a series of site investigation conducted in soft marlne clays in Japan. As can be seen in Figs. 7, 8 and 9, the following correlations may be seen for soft Bangkok clay:

\[ G_{\text{\scriptsize{max}}} = (55 + 11)E_0 \]  

(NNH) (AIT)

\[ G_{\text{\scriptsize{max}}} = (19 + 17)(\varphi, \sigma) \]  

(NNH) (AIT)
Alternatively, $G_{m,0}$ may be estimated using appropriate void ratio functions (Shibuya and Tanaka, 1996; Shibuya et al., 1997):

$$G_{m,0} = \alpha_1 \varepsilon_1 \varepsilon_0 ^{2.5} (OCR)^{0.36} \quad \text{(in kPa)} \quad (4.3)$$

$$G_{m,0} = \beta (1 + \varepsilon_1) - 2 \sigma' \varepsilon_1 ^{0.5} (OCR)^{0.36} \quad \text{(in kPa)} \quad (4.4)$$

Note that the prediction of $G_{m,0}$ in the above equations needs information solely on the available borehole data: i.e., in-situ void ratio, $e_1$, and the effective overburden pressure, $\sigma'$. The average values of $\alpha$ and $\beta$ of 5,000 and 24,000 have been obtained on the basis of field and laboratory investigation on Japanese and European clays. Fig. 10 shows the results for soft Bangkok clay, which give:

$$G_{m,0} = (5,000 \rightarrow 6,200) \varepsilon_1 ^{1.1} \sigma' ^{0.5} \quad \text{(AIF) (NH)} \quad (4.5)$$

$$G_{m,0} = (2,000 \rightarrow 3,100)(1 + \varepsilon_1) ^{0.35} \sigma' ^{0.5} \quad \text{(AIF) (NH)} \quad (4.6)$$

In Fig. 10, the data points denoted by star symbol represent the $G_{m,0}$ value from laboratory bender element test performed on Shelby tube samples recompressed to in-situ $\sigma'$. Effects of sample disturbance on the measurement of $G_{m,0}$ appears to be insignificant when judged from close coincidence of the $G_{m,0}$ values between in-situ and laboratory measurements.

![Fig. 7. a) $G_{m,0}$ from DMT and SCPT
b) $G_{m,0}$ from CPT and SCPT](image)

![Fig. 8. Correlation of $G_{m,0}$ to $E_b$ and $(q_s - \sigma'_v)$](image)

![Fig. 9. a) $G_{m,0}$ from SCPT and CPT, DMT;
   b) Correlation of $G_{m,0}$ to $E_b$ and $(q_s - \sigma'_v)$](image)

![Fig. 10. Predicted shear modulus $G_{m,0}$](image)
4.3 Undrained Shear Strength $S_u$

Fig. 11: a) Shear strength from in-situ vane test correlated to CPTU. b) Shear strength from direct shear test correlated to CPTU (NNH site).

In general, $S_u$ may be correlated to the cone resistance in the following form:

$$S_u = \frac{q_c - q_{pr}}{N_{ef}}$$  \hspace{1cm} (4.7)

$q_c$: corrected cone resistance taking account of pore water pressure

$N_{ef}$: piezcone factor

As can be seen in Fig. 11, $N_{ef}$ ranged from 11 to 17 against corrected vane shear strength at NNH site and AIT site, respectively.

Correlating the piezcone data to $S_u$ from constant volume direct shear box test, $N_{ef}$ was found 7.8 at NNH site.

4.4 Rigidity index ($G_{max}/S_u$)

Fig. 12 shows values of rigidity index at two sites. It may be seen that $G_{max}/S_u$ in soft Bangkok clay ranged from 300 to 500.

5. CONCLUSION

Geological records show that soft Bangkok clay has never been subjected to mechanical overconsolidation. However, the overconsolidation ratio when examined in CRS test on high-quality samples was about 1.6 on average at Naung Ngu Hao site. The depth profile of elastic shear modulus, $G_{max}$, was successfully estimated with borehole data, the cone tip resistance and the dilatometer modulus, through empirical correlations previously proposed by the authors. In estimating undrained shear strength, $S_u$, the cone factor, $N_{ef}$, was 17 and 11 at AIT and NNH sites, respectively. The rigidity index, $G_{max}/S_u$, ranged from 300 to 500.

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7. REFERENCES


Deformation and strain fields beneath an embedded circular plate

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ABSTRACT: An extensive series of full scale load tests using buried circular plates has been conducted over the last six years at the Department of Geotechnical Engineering of the Norwegian University of Science and Technology. These tests have explored the effects of plate shape, installation disturbance, and unload/reload cycles in dry Hokksund sand at various densities. A portion of the test series focused on the direct measurement of the deformation field developed beneath a flat circular plate during loading. Axissymmetric deformation fields are constructed using the composite data from five load tests conducted at each of three densities. These deformation fields are then used to calculate strain fields.

1 INTRODUCTION

The field compression meter (FCT), an in situ at depth plate load test, has been the focus of a renewed research effort of the Department of Geotechnical Engineering at the Norwegian University of Science and Technology (NTNU). The FCT provides in situ estimates of deformation moduli in silty and sandy soils. The modulus is measured using vertical loads, differing from the radial or horizontal loading imposed by other field tests. Anticipated design load levels are used during testing.

This research effort has explored the effects of shape, installation disturbance and unload/reload response of buried plates in dry Hokksund sand. A series of 57 full scale load tests using screw-shaped and flat plates have been conducted over a 6 year period. Three densities of dry sand and three installation methods were used.

A portion of the test series focused on the measurement of point displacements in the soil body beneath the plate. Displacements beneath the plates were measured in the three densities, however plate shape and installation method were held constant. A flat circular bearing plate with a diameter of 16 cm built in place during the filling process representing an ideal or "wished in place" installation was used.

2 EXPERIMENTAL SERIES

The tests were conducted in a large scale model lab capable of full scale tests of field equipment. This lab consists of a large tank (4 x 4 x 3 m) and an automated handling system for filling and emptying the tank. Soil is placed in the tank by raking from a distributor which passes slowly back and forth over the tank. The available soil is dry Hokksund sand, a relatively homogeneous medium sand. Reference geotechnical parameters for Hokksund sand are given in Table 1, and the grain size distribution shown in Figure 1.

<table>
<thead>
<tr>
<th>Table 1. Hokksund sand (Moen, 1978).</th>
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<td>Grain density, ( \rho_s )</td>
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<tr>
<td>Laboratory water content, ( w )</td>
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<td>Maximum porosity, ( n_{max} )</td>
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<td>Minimum porosity, ( n_{min} )</td>
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<td>Coefficient of uniformity, ( C_u )</td>
</tr>
<tr>
<td>Constrained modulus, ( M^c ) (loose)</td>
</tr>
<tr>
<td>Constrained modulus, ( M^c ) (dense)</td>
</tr>
<tr>
<td>Dry friction angle, ( \phi ) (loose)</td>
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<tr>
<td>Dry friction angle, ( \phi ) (dense)</td>
</tr>
<tr>
<td>Cohesion, ( c )</td>
</tr>
</tbody>
</table>

*500 kPa reference vertical stress, oedometer test

The fill density is chosen by varying the size of the nozzles in the distributor. Historical data, as well as density controls during these tests indicate that an homogeneous density is obtained throughout the depth of the tank. This is consistent with studies showing that above a critical height the fill distance has no effect on sample density (Rud and Truu, 1985). The test densities are given in Table 2.
The bearing plates were placed in the soil during the filling process. After filling the tank to 1 m below the final surface elevation (approximately 2 m depth in the tank), the surface was carefully leveled and the plate placed directly on the sand. The bearing plates were then stabilized by guy wires attached to the top of the 1.1 m long drill rod. After the four plates were placed and stabilized, an additional 1 m of sand was mined into the tank and the guy wires removed. The placement plan and the bearing plate assemblies are illustrated in Figure 2.

A hydraulic jack mounted to a reaction frame spanning the tank was used to apply incremental loads. The free moving PVC sleeve on the drill rod eliminated side friction, allowing the full load to be transferred to the plate. This also isolated the loading to only the soil surrounding the plate. The number of load increments was typically about 5.

The point displacements beneath the plates were measured using small copper disks built into the soil body during filling of the tank. The disks were connected to dial gauges at the surface via light gauge copper wire run through thin plastic tubing. Each disk provided displacement in one direction. Six disks were used in a single bearing plate test to measure radial and vertical displacements in 3 points (Figure 3). By combining the data from five bearing plate tests, each with 3 unique points, an axisymmetric network of 15 points with measured vertical and radial displacements develops, as shown in Figure 4.

3 DATA PREPROCESSING

The point displacement data required preprocessing to correct two problems: differences in the medium density and inconsistent load increments. The difference in density is readily apparent from Table 2, where it is seen that Hoff (1992) and Kvalsvik (1991) report different medium densities. Differences in the load increments arise because the data is a composite of many individual tests, and some variances in applied load occurred from test to test. These two problems are moderated by preprocessing of the data.
The same density at all points in the data set is required for a meaningful analysis. The 'loose' and 'dense' conditions for Hoff (1992) and Kvalsvik (1991) tests are identical, however the 'medium' density is 1.69 g/cm³ in the Hoff (1992) tests and 1.63 g/cm³ in the Kvalsvik (1991) tests. As shown in Figure 4, the Kvalsvik (1991) data represents three points beneath the edge of the plate. Since the Kvalsvik (1991) tests could not be repeated, an estimate of the load-deformation response of these three points was made using an interpolation between the Kvalsvik (1991) data at 1.63 g/cm³ and at 1.74 g/cm³ to obtain an estimate of the load-displacement response at 1.69 g/cm³, the 'medium' density used by Hoff (1992).

The variations in load increment for each test are easily accommodated by fitting a smooth curve to the discrete load-displacement data. For each initial point \( p(u,v) \), two third order polynomials are used to describe the displaced location of the point \( p(u,v) \) as a continuous function of load \( f \)

\[
p(u,v) = p(u + u(f), v + v(f))
\]

(3.1)

The functions \( u(f) \) and \( v(f) \) are defined by

\[
u(f) = a_2 f^2 + b_2 f + c_2
\]

(3.2)

\[
v(f) = a_3 f^2 + b_3 f + c_3
\]

(3.3)

where \( a, b, c \) are curve fitting constants.

These polynomials allow the calculation of the deformation field at arbitrary load levels within the load limits defined by the discrete data sets. For plotting and visualization purposes, the calculated deformations \( u(f) \) and \( v(f) \) may be exaggerated by multiplying with a magnification constant.

In Figure 4 it is seen that the two points at the surface do not correspond to the regular grid, the points must be adjusted radially by 40 cm. The first

is corrected by linear interpolation between the measured points. This interpolation process is also used to develop the deformation fields and is fully described in section 4. The second point, which must be located at 24 cm from the centerline of the plate, cannot be interpolated as it does not lie between existing data points. An extrapolation is made, assuming that the load-displacement response of the extrapolated point will follow the trends defined by the adjacent measured points. The Kvalsvik (1991) data points vary slightly (up to 10 cm) from the regular grid; these are also adjusted by interpolation.

For a complete rectangular grid, additional data points are required to define the boundaries of the field. These data points lie along the centerline beneath the plate, and along the boundary 32 cm from the centerline. These data points are estimated using an extrapolation from the existing data. Two premises are used: First, the estimated data points will have similar trends in load-displacement response to the adjacent measured data points. Second, points directly beneath the centerline of the plate have only vertical displacements, radial displacements are zero.

Clearly the creation of these grid points is uncertain and relies on judgment. These data points may not accurately reflect the deformations which occur. However, it is felt that estimations of the deformations outside the measured grid provides a better basis for the calculations of the deformation fields and subsequently the calculation of associated strains. Figure 5 presents a typical input data set.

![Figure 5. Diagram of input grid.](image-url)
4 DISPLACEMENT FIELDS

The loading of the plate causes displacements throughout a volume of soil beneath and radially outward from the plate, and it is of interest to visualize this continuous displacement field. The measured displacements can be used to represent this field at the grid points shown in Figure 4. However, the spacing of these points is relatively coarse, limiting the ability to visualize the field.

An impression of the continuous displacement field can be obtained from a plot of a relatively tightly spaced network of point displacements. The effect is the same as that obtained in photographs of pin model tests. A fine grid of point displacements may be obtained by interpolating between measured points using finite element analysis techniques.

Discrete points in a continuous displacement field provide a basis for estimating displacements occurring at any point within the region defined by the points. The geometry of the points shown in Figure 5 represent a simple case, where the measured points define the corners (or “nodes”) of rectangular regions. The geometry of a region is defined in Figure 6. The horizontal displacements \( u = u(x) \) and the vertical displacements \( v = v(x) \) at a point within this region are estimated by interpolating from the nodal values \( u_i \) and \( v_i \) using linear shape functions:

\[
    u = \sum_{i=1}^{4} N_i u_i \quad \text{and} \quad v = \sum_{i=1}^{4} N_i v_i
\]

(4.1)

where the functions \( N_i \) are defined as (cf. Figure 6):

\[
    N_1 = \frac{(a-x)(b-y)}{4ab} \quad N_2 = \frac{(a+x)(b-y)}{4ab} \\
    N_3 = \frac{(a+x)(b+y)}{4ab} \quad N_4 = \frac{(a-x)(b+y)}{4ab}
\]

(4.2)

5 mm settlement of bearing plate (kPa) 180 820 1100
Estimated bearing capacity (kPa) 150 1000 1200

By using the interpolation described above, and expressing the measured point deformations as a function of load level (equations 3.1 to 3.3), the deformation occurring at an arbitrary point and load level may be calculated for the system. A fine mesh of displaced points at a chosen load level, for example at the estimated bearing capacity, can be generated to visualize the deformation field. Although impossible to present in this article, a dynamic visualization is also possible where the deformation field is calculated and plotted for a series of increasing load levels, allowing the development of the deformation field to be viewed as a continuous process.

The displacement fields are calculated at the stress level causing 5 mm vertical displacement of the bearing plate in each density. This load level corresponds well with the estimated bearing capacity of the plates, see Table 3. In the following figures only two interpolation points between nodes are used. A magnification factor of 15 was applied to the calculated displacements shown in the figures 7-9.

Table 3. Stress levels for deformation calculations.

<table>
<thead>
<tr>
<th>Bearing plate (kPa)</th>
<th>Loose</th>
<th>Medium</th>
<th>Dense</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 mm settlement</td>
<td>180</td>
<td>820</td>
<td>1100</td>
</tr>
<tr>
<td>Estimated bearing capacity (kPa)</td>
<td>150</td>
<td>1000</td>
<td>1200</td>
</tr>
</tbody>
</table>

Figure 6. Bilinear element (Cook, et al. 1989).

Figure 7. Deformations in loose Hokkaido sand at plate stress of 180 kPa, magnification factor 15.
The volume of deformed soil extends to approximately 350 mm (2.2 plate diameters) below the plate in the loose sand (Figure 7). The largest deformations are concentrated in an approximately cylindrical volume beneath the plate. The deformations are primarily vertical, decreasing rapidly confining the zone of influence to a relatively small spherical volume beneath the plate.

The depth of influence is greater in the middle density (Figure 8), estimated at about 450 mm, or 2.8 plate diameters. This lies outside of the plotted region and is an estimate. The pattern is less spherical, tending towards a conical radiation from the plate, however the largest deformations are still contained in a cylindrical volume beneath the plate. A noticeable widening of the influenced volume indicates a tendency towards radial displacements.

The depth of influence in the dense sand (Figure 9) is even greater, similarly estimated to perhaps 550 mm (3.3D) or more. The tendency towards radial displacements decreases from the medium density (cf. Figures 8 and 9). They are primarily vertical, with a slight tendency of radial displacement.

The changes in the influence volume (sketched in Figure 10) are likely attributable to changes in the response of the soil to the shearing cause by loading of the plate. At the loosest state, the sand is likely to be contractant. The structure of the soil collapses, effectively damping out the stress field by restructuring of the soil, perhaps even increasing the plastic deformations by reducing the mean stress level. The sand at its densest state is likely to be dilatant, which under shearing would cause increases in the mean effective stress. This may increase the elastic response, carrying stresses deeper in the soil. The medium case presents an uncertainty - although the Knudsvik (1991) data is corrected for porosity, this correction is a linear interpolation and does not account for changes in behavior caused by dilatancy.

Figure 8. Deformations in medium Hokksund sand at plate stress of 820 kPa, magnification factor 15.

Figure 9. Deformations in dense Hokksund sand at plate stress of 1100 kPa, magnification factor 15.

Figure 10. Sketch of Influence region.
5 STRAIN FIELDS

The displacement data presented in section 4 are used for calculating the strain fields. The strains are calculated at each grid point (both original and interpolated); these strains are then contoured to represent the continuous strain field.

The linear interpolation of deformations used in section 4 produces constant vertical and radial strains within each rectangular area. These strains are calculated from the displacement data. For point 4 in Figure 11a, the strains are calculated as:

\[
\begin{align*}
\varepsilon_r &= \frac{u_4 - u_3}{2a} \\
\varepsilon_\theta &= \frac{v_4 - v_3}{2b} \\
\gamma &= \frac{v_4 - v_3 - u_4 + u_3}{2b}
\end{align*}
\]

where \( \varepsilon_r, \varepsilon_\theta \) and \( \gamma \) are radial, vertical and shear strains.

![Figure 11. Strain calculations.](image)

The strain values calculated for any grid point (or corner node in Figure 11b) depend on which of the adjacent areas the point is considered to be a member of. The strains calculated by (5.1) are constant, thus strains calculated for adjacent areas are generally discontinuous. A single grid point may have up to eight possible strain values calculated for it, two each of vertical and radial strains, and four possible shear strains depending on the arrangement of adjacent grid points, cf. Figure 11b. However, contouring the data requires that each point must have a unique value for each of the principle strains.

An averaging scheme using the subgrid of interpolated displacement points is adopted to produce a continuous strain field from the calculated values. When the point is within a network of points, the strains at the point are the averages of the values calculated with respect to each of the rectangular elements connected to that point. For the center point in Figure 11b, the averages are calculated as:

\[
\begin{align*}
\varepsilon_r &= \frac{\varepsilon_r(0) + \varepsilon_r(9)}{2} \\
\varepsilon_\theta &= \frac{\varepsilon_\theta(0) + \varepsilon_\theta(9)}{2} \\
\gamma &= \frac{\gamma(0,0) + \gamma(0,9) + \gamma(9,0) + \gamma(9,9)}{4}
\end{align*}
\]

where the subscripts in parentheses indicate the nodal point (or points) the strain calculation is based upon. Absolute values are used for shear strains.

This average scheme spreads the discontinuity at the point to produce two regions with linearly varying strain on either side of the point. The amount of spreading depends on the spacing of the interpolation points, as illustrated in Figure 12 which shows the effect for axial strains in a rod. A large number of interpolation points approximates the discontinuous original field, while maintaining continuity in the averaged field. At least one interpolation point is required, otherwise, the maximum strain will not be calculated, cf. Figure 12b.

![Figure 12. Effect of the average scheme in (5.2).](image)

A fourth quantity representing the overall strain level at the point may be defined. This quantity is the strain magnitude \( \varepsilon_{\text{max}} \) and is defined as the norm to the axisymmetric small strain tensor \( \varepsilon_s \):

\[
\varepsilon_{\text{max}} = \sqrt{\varepsilon^2_r + \varepsilon^2_\theta + \gamma^2}
\]

The strains at 5 mm vertical displacement of the bearing plate (approximately bearing capacity of the plate) are calculated for the three densities using the 2 by 2 interpolated grid from section 4. The load is applied incrementally, and the quantities in (5.2) and (5.3) evaluated at each load step for all grid and interpolation points. The maximum values are extracted, and as seen in Table 4 shear strains dominate and radial strains are negligible. The vertical compression shows moderate mobilization.
Table 4. Maximum calculated strains and strain magnitudes at 5 mm vertical plate displacement.

<table>
<thead>
<tr>
<th>Strain (%)</th>
<th>Loose</th>
<th>Medium</th>
<th>Dense</th>
</tr>
</thead>
<tbody>
<tr>
<td>$e_x$</td>
<td>2.17</td>
<td>1.79</td>
<td>2.09</td>
</tr>
<tr>
<td>$e_y$</td>
<td>0.09</td>
<td>0.44</td>
<td>0.15</td>
</tr>
<tr>
<td>$e_z$</td>
<td>5.50</td>
<td>6.21</td>
<td>5.68</td>
</tr>
<tr>
<td>$e_{in}$</td>
<td>5.79</td>
<td>6.22</td>
<td>5.90</td>
</tr>
</tbody>
</table>

Contour plots of the strains were prepared for the three densities at the final load increment (load at 5 mm vertical displacement). The strain fields were similar in form between the three densities. The strain fields $e_x$ and $e_y$ in only the loose sand is presented. The radial strains are not plotted; the calculated values are so low that they fall below the lowest contour value and the plot is empty.

The vertical strains are predominant in a roughly cylindrical volume beneath the plate, whereas the shear strains predominate in a ring-shaped volume surrounding this cylinder, as shown in Figures 13 and 14. These strains correspond to a column of soil beneath the plate being compressed and pushed downwards through the surrounding soil mass.

The strain magnitude $e_{in}$ field is plotted in Figures 15 to 17 for the three densities. The response in the loose state and the dense state at the estimated bearing capacity is similar, although the strains in the dense sand decrease faster than in the loose. The difference in response may be due to the dilatancy effects discussed in section 4.

Figure 13. $e_x$ contours, loose Hokksund sand.

Figure 14. $e_y$ contours, loose Hokksund sand.

Figure 15. $e_{in}$ contours, loose Hokksund sand.
with increasing density; in the dense sand this depth is perhaps greater than 3.5 plate diameters. The strains beneath the plate consist primarily of a cylindrical volume in vertical compression, circumscribed by a cylindrical shell volume of much higher shear strains. The overall strain response, represented by the strain magnitude \( \varepsilon_{xy} \), shows a consistent pattern between the loose and the dense states, with some indication that strain levels may drop slightly faster with depth in the dense sand.

Figure 16. \( \varepsilon_{xy} \) contours, dense Hokksund sand.

The middle density (Figure 17) is not consistent with the loose or the dense case, nor is it consistent with an intuitive sense of what the strain distributions are likely to be. The distortion in the strain field is likely caused by the extensive preprocessing of the input data required for this density. As discussed in section 4, the creation of data points at the correct density was done using linear interpolation between measured data points at higher and lower densities than the desired density. This linear interpolation does not account for dilatancy effects. Shear is the predominant strain (cf. Table 4 and Figure 14), and tendencies towards volume change during shearing would have an effect on the measured deformations.

Figure 17. \( \varepsilon_{xy} \) contours, medium Hokksund sand.

6 SUMMARY AND REMARKS

The analysis presented in this paper is concerned with the calculation and graphical representation of the deformation and strain fields occurring beneath a buried circular plate in a dry homogeneous sand at three densities. These fields may be calculated and plotted at arbitrary load levels within the range of loads used in the laboratory tests. The figures are prepared for a vertical load causing 5 mm of vertical plate displacement, this load compares well to bearing capacity estimates for the plates.

The results show a generally consistent behavior between the lowest and highest density. Deformations consist primarily of vertical movement, with increasing radial deformations in the dense sand. The depth of influence increases

7 REFERENCES


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Correlation between SPT and seismic CPT

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ABSTRACT: Correlations of cone penetration resistance with physical soil properties, SPT N-value, and shear wave velocity are investigated, using either the soil behavior type index \( I_b \) proposed by Robertson, fines content, or mean grain size as an index to specify the soil type. The following conclusions are drawn: (1) The soil behavior type index \( I_b \) shows good correlation with fines content and mean grain size. (2) The CPT-SPT resistance ratio \( q_s/N_v \) varies not only with soil type but also with SPT N-value or cone penetration resistance. (3) The shear wave velocity shows good correlation with cone penetration resistance when the effect of soil behavior type index \( I_b \) is taken into account.

1 INTRODUCTION

The standard penetration test (SPT) has been widely used for many years in Japan, and most foundation designs have been based on SPT N-values and physical properties of soils recovered in the SPT sampler. One reason why the SPT is so popular is that many correlations between SPT N-values and soil properties required for foundation design have been proposed and used. However, the SPT has some disadvantages such as potential variability of measured resistances depending on operator variability, and possibility of missing delicate changes of soil properties owing to the inevitable discrete record. Moreover, it is unsuitable for cohesive soil having unfavorably small N-values.

As a result, the cone penetration test (CPT) has become increasingly popular in recent years because it provides a continuous record which is free from operator variability. Thus, the CPT provides a more consistent evaluation of the strength and stiffness of soils than does the SPT. However, since it has few correlations with dynamic soil properties, more correlations are required for foundation design. Furthermore, there is a clear need to establish a good CPT-SPT correlation so that the CPT can be used in conjunction with the SPT.

This paper proposes correlations among cone penetration resistance \( q_s \), physical and mechanical properties of soil, i.e., fines content \( F_C \), mean grain size \( D_{50} \), SPT N-value, and shear wave velocity \( V_s \), based on the seismic CPT.

2 OUTLINE OF CONE PENETRATION TEST

The CPT tests were conducted at 15 sites where SPT N-values, fines content \( F_C \), and mean grain size \( D_{50} \) were known from SPT tests. Seismic CPT tests, which combine shear wave velocity measurements with the CPT, were also conducted at 57 sites in Japan. These sites included a wide variety of soils ranging from clays to sandy soils, but primarily sandy soils with SPT N-values less than about 60.

Fig. 1 outlines the seismic cone penetrometer. The seismic CPT tests were conducted at a penetration rate of 2 cm/sec, and recorded continuous records with depth of three components: cone penetration resistance \( q_s \), sleeve friction \( f_s \), and pore water pressure \( P_w \). It can also evaluate shear wave velocity \( V_s \), when penetration is temporarily halted at intervals of 50 cm or 1 m. The shear wave generated from the surface by the plunger hammering method was observed with a geophone within the cone.

Fig. 2 shows an example of a seismic CPT result, along with an SPT result at the same site. The seismic CPT result is presented in the form of distributions of the above three components, i.e., cone penetration resistance \( q_s \), sleeve friction \( f_s \), and pore water pressure \( P_w \). In addition to the plots of shear wave and the corresponding shear wave velocity \( V_s \), the variation of SPT N-value with depth shows a very similar tendency to that of cone penetration resistance \( q_s \), as shown in Fig. 2. The present method can produce a clear waveform of the shear wave.
3 RELATION TO PHYSICAL PROPERTIES

Figs. 3 and 4 show relations between soil behavior type index I_s and physical properties of soils, i.e., fines content FC and mean grain size D_{50}. The soil behavior type index I_s, proposed by Robertson et al. (1995) is defined as:

\[ I_s = \left( 3.47 - \log Q_4 \right)^2 + \left( \log F_R + 1.22 \right)^2 \]  
(1)

\[ Q_4 = \frac{\sigma - \sigma_v}{\sigma_v} \]  
(2)

\[ F_R = \frac{f_c}{\sigma_c} \times 100 \]  
(3)

where \( F_R \) is normalized penetration resistance (dimensionless), \( F_R \) is normalized friction ratio (%), \( Q_4 \) is core penetration resistance (MPa), \( f_c \) is sleeve friction (MPa), \( \sigma_c \) is overburden pressure (MPa), and \( \sigma_v \) is effective overburden pressure (MPa).

As the value of soil behavior type index \( I_s \) increases, the fines content FC tends to increase, as shown in Fig. 3, while the mean grain size \( D_{50} \) tends to decrease, as shown in Fig. 4. This suggests that fines content FC or mean grain size \( D_{50} \) can be approximately estimated from soil behavior type index \( I_s \).

4 CPT-SPT CORRELATION

Fig. 5 shows relations between SPT N-value and cone penetration resistance \( q_c \) in terms of fines content FC, mean grain size \( D_{50} \) and soil behavior type index \( I_s \). The cone penetration resistance \( q_c \) and soil behavior type index \( I_s \) are the average values over a length of 30 cm where the corresponding N-values were measured. The N-values used here were corrected to an energy efficiency of 78% (Sned et al. 1985). There is a fairly well defined tendency between SPT N-value and cone penetration resistance \( q_c \), when either fines content FC, mean grain size \( D_{50} \) or soil behavior type index \( I_s \) is taken into account. Also shown in Fig. 5 are curves to represent these relations, which are drawn as center lines for the data. The curves shift downward with increasing fines content FC or soil behavior type index \( I_s \), or decreasing mean grain size \( D_{50} \). Furthermore, the slope of each curve gradually decreases as the SPT
N-value increases, which suggests that the $q_v/N$ ratio may vary with SPT N-value.

Figs. 6 and 7 show relations of $q_v/N$ ratio with fines content FC and mean grain size $D_{50}$ in terms of SPT N-value. Also shown in the figures are curves to represent the relations, which are drawn as center lines for the data. The correlations derived from previous studies (Muromachi et al. 1982 and Robertson et al. 1983) are also shown in these figures. It is known from the previous studies that the $q_v/N$ ratio appears to decrease with increasing fines content FC (Muromachi et al. 1982), or with decreasing mean grain size $D_{50}$ (Robertson et al. 1983). The figures, however, indicate that the $q_v/N$ ratio varies not only with fines content FC or mean grain size $D_{50}$ but also with SPT N-value, particularly for sandy soils with small fines content FC or large mean grain size $D_{50}$.
The $q_u/N$ ratios of those soils tend to decrease with increasing SPT N-value. For example, the $q_u/N$ ratio with fines content FC = 10% may be close to 1.0 for SPT N-values less than 10, whereas they decrease to 0.5 for SPT N-values greater than 30. The reason for this may be caused by 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 well known that the drained strength vs. undrained strength ratio for sands decreases with increasing soil density (Tatsuoka et al. 1982). This appears to relate to the above tendency in which the $q_u/N$ ratio decreases as SPT N-value increases. There are few data for cohesive soil with large SPT N-value. However, the change in the $q_u/N$ ratio with SPT N-value is likely to be small for large fines content FC or small mean grain size $D_{50}$, since both CPT and SPT can produce undrained conditions in cohesive soil.

Fig. 8 shows relations between the $q_u/N$ ratio and soil behavior type index $I_b$ in terms of cone penetration resistance $q_c$. Also shown in Fig. 8 are curves to represent the relations, which are drawn as center lines for the data. Fig. 8 indicates a tendency in which the $q_u/N$ ratio decreases with increasing soil behavior type index $I_b$ or cone penetration resistance $q_c$. Thus, it may be possible to estimate the SPT N-value from CPT data alone.

5 CPT-Vs CORRELATION

Fig. 9 shows the relation between cone penetration resistance $q_c$ and shear wave velocity $V_s$. It indicates that shear wave velocity $V_s$ increases as cone penetration resistance $q_c$ increases. A previous study has shown that the relation between SPT N-value and shear wave velocity $V_s$ varies depending on soil type (Imai 1977). The relations between cone penetration resistance $q_c$ and shear wave velocity $V_s$ in terms of soil behavior type index $I_b$ are shown in Fig. 10. The cone penetration resistance $q_c$ and soil behavior type index $I_b$ are average values over intervals of 50 cm or 1 m, where the corresponding shear wave velocities were measured. Fig. 10 includes straight lines to represent these relations, which are drawn as center lines for the data. Also shown in Fig. 10 (d) by a broken line is the relation for cohesive soils proposed by Mayne et al. (1995). Fig. 10 indicates that the line shifts toward as the soil behavior type index $I_b$ increases. Moreover, the relation for $I_b \geq 2.6$ shown in Fig. 10 (d) is consistent with that proposed by Mayne et al. (1995).
Fig. 11 shows the relation between shear wave velocity vs. cone penetration resistance ratio $V_s/q$ and soil behavior type index $I_s$. There is a fairly well-defined trend in which the $V_s/q$ ratio increases as the soil behavior type index $I_s$ increases. To specify a more definite tendency, the relations between $V_s/q$ ratio and soil behavior type index $I_s$ are shown in Fig. 12 in terms of cone penetration resistance $q_s$. Also shown in Fig. 12 are straight lines to represent the relations, which are drawn as center lines for the data. The line shifts downward and left with increasing cone penetration resistance $q_s$. Thus, it may be possible to estimate the shear wave velocity $V_s$ from the CPT result alone.

Fig. 11. Relation between soil behavior type index and $V_s/q$ ratio.

6 CONCLUSION

CPT tests were conducted at 15 sites where SPT N-values with grain size characteristics were available. Seismic CPT tests, which combine shear wave velocity measurements with the CPT, were also conducted at 57 sites in Japan. The effects of fines content FC, mean grain size $D_{50}$, and soil behavior type index $I_s$ on the correlations among cone penetration resistance $q_s$, SPT N-value, and shear wave velocity $V_s$ are investigated. The following conclusions may be made:

1. The soil behavior type index $I_s$ shows good correlation with fines content FC and mean grain size $D_{50}$. Soil behavior type index $I_s$ tends to increase with increasing fines content FC or decreasing mean grain size $D_{50}$. 

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2. The \( q/N \) ratio varies depending on soil type represented by either fines content FC, mean grain size \( D_{50} \) or soil behavior type index \( I_b \), which is consistent with previous findings. In addition, the \( q/N \) ratio for the same sands increases with decreasing N-value, probably reflecting the difference in drainage conditions between CPT and SPT tests. Thus, the CPT-SPT resistance ratio \( q/N \) varies not only with soil type but also with SPT N-value or cone penetration resistance \( q_c \).

3. The shear wave velocity \( V_s \) shows good correlation with cone penetration resistance \( q_c \), when the effect of soil behavior type index \( I_b \) is taken into account.

REFERENCE


Characterization of a saprolitic soil from Porto granite by in situ testing

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ABSTRACT: An extensive site investigation has been made in a relatively homogeneous weathered profile of a typical saprolitic soil from granite of Porto region, which included penetration testing such as DP, SPT and CPT, pressuremeter (PMT) and SBPT, dilometer (DMT), cross-hole seismic testing (CH) and Loading Tests (PLT over 30 and 60 cm diameter plates and a 120 cm diameter concrete footing, carefully instrumented). Identification and classification based on such testing methods and criteria of interpretation of the results in order to define mechanical properties and geotechnical design parameters – mainly for shallow foundations purposes – are discussed. Most attention is made on the adaptation of current criteria for evaluation of geotechnical properties, developed for transported soils with similar physical properties, to the particularities of these residual soils, characterized by singular fabric and structural (cementation) characteristics.

1. SELECTION OF THE EXPERIMENTAL SITE.
GENERAL DESCRIPTION OF THE PROFILE

The experimental work was carried out at a site (approximately 50 x 30 m²) in which a homogeneous saprolitic soil 6 m thick was encountered by the previous SPT and DP exploring campaign.

Geologically, the parent rock is a very well characterized granite from Porto region (Viana da Fonseca et al., 1994). Details of geological data as well as of grain size distribution and Atterberg limits of the saprolitic soil can be found in Viana da Fonseca et al. (1997). Generally, results obtained with specimens taken from the SPT sampler and from blocks reveal a fairly homogeneous ground profile (well graded) classified as SM (silty sand) or SM-SC (silty clayey sand), according to the ASTM Classification for Engineering Purposes. Natural physical properties were evaluated from these samples with values of 0.60 to 0.85 for the void ratio and 15 to 29% for the water content, which correspond to degrees of saturation of 70 to 100%.

The natural particle arrangement is characterized by open continuous voids on a cemented structure and low values of relative density (D<sub>r</sub> ≤ 28%)

2. PARAMETERS FROM IN SITU TESTS

2.1 Resume of the executed tests

Table 1 resumes the executed tests. Figure 1 shows the variation of SPT, CPT, CH and DMT values with at rest effective vertical stress, σ<sub>v</sub>, and depth, z.

<table>
<thead>
<tr>
<th>Test</th>
<th>Number (penetration, blow count)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPT</td>
<td>46 (46 k bl)</td>
</tr>
<tr>
<td>DP</td>
<td>2 (52.0 m)</td>
</tr>
<tr>
<td>DPL</td>
<td>15 (66.3 m)</td>
</tr>
<tr>
<td>CPT</td>
<td>9 (53.9 m)</td>
</tr>
<tr>
<td>PLT (0.01 m)</td>
<td>3</td>
</tr>
<tr>
<td>PLT (0.3 m)</td>
<td>3</td>
</tr>
<tr>
<td>PI</td>
<td>1 (3 bl)</td>
</tr>
<tr>
<td>PMT</td>
<td>5 (3 bl)</td>
</tr>
<tr>
<td>SBHT</td>
<td>4 (2 bl)</td>
</tr>
<tr>
<td>CH</td>
<td>32 (2 bl)</td>
</tr>
<tr>
<td>DMT</td>
<td>12 (24 bl)</td>
</tr>
</tbody>
</table>

2.2 Standard Penetration Testing (SPT)

The variation of (N')<sub>90</sub> reflects a range of values for the angle of shearing resistance of 35° to 40°, with an average value of 38° (Décourt, 1989). These values agree with those deducted from back-analysis of PLT and tracial tests over undisturbed samples (Viana da Fonseca, 1990). Nevertheless, cohesion intercept calculated from these tests is significant (c' = 8-12 kPa), which is a characteristic of residual soils and, therefore, strength solely defined by q' will be rather conservative.

Values of (N')<sub>90</sub> plotted on Skempton (1986) abacus, Fig. 2, would indicate medium to high densities while direct evaluation of relative density
(OCR > 0.3) indicates high porosity. It is believed that analysis ruled by distal theories will be conservative. Besides, the association of OCR values greater than one, on characterization of these soils from SPT, deviates the results towards abnormal low strength values.

On correlating N_{SPH} values with stiffness it is now well stated that the relations between penetration parameters and maximum shear modulus (G_m) are the ones which best assure some independence on unavoidable misleading factors, such as scale effects, non-linearity, etc. (Jamiolkowski et al., 1988; Robertson, 1991). From the experimental data a linear relation could be obtained:

\[ G_m (MPa) = 98 + 0.42 \cdot N_{SPH} \]  \hspace{1cm} (2.1)

The variation of G_m with effective mean stress (\(\sigma_{eff}^{*}\)) is rather less accentuated than the correspondent penetration parameters, such as N_{SPH}, with the consequence that correlations between G_m and N_{SPH} for the relevant values of \(\sigma_{eff}^{*}\) on shallow foundations strongly underestimate elastic stiffness of the soil (Shrodi, 1988).

2.3 Dynamic Probing (DPSPH and DPL)

The two regions most common energies from ISSMFE standards were used: DPH and DPL.

The variation of DPH parameters with depth is less pronounced than the N_{SPH} (CPT) variation. Even an analysis based on the specific energy of penetration (Nixon, 1988), particularly between the results of DPH and SPT, is not conclusive. On the other hand, the results of DPL tests, executed aside of CPT and SPT tests, interpreted in terms of the dynamic cone resistance in the unit of pressure (P_{K} – Dutch formula) are very similar to q_{c} values. This trend has been regionally verified on sandy soils and on saprolitic soils from granite strongly weathered with mainly sandy matrix. The fact that this trend is undetected on DPH test is associated to the very high energy which leads to a number of blows rather low and nonsensitive to natural variation of soil characteristics through depth.

2.4 Cone Penetrometer Testing (CPT)

Results of CPT (Fig. 1b) denote an approximately linear growth of q_{c} with q_{c}. It is also observed that values of sleeve friction grow smoothly with the at rest effective stress, with ratios of f_{s}/q_{c} = 0.77, revealing fair homogeneity of friction over this depth.

Soil classification based on K_{S}, OCR = f_{s}/q_{c} values, with reference to proposals developed for transported soils (Maswood & Mitchel, 1993), indicates high values of OCR, which are not congruent with the genesis of these soils and with the low values of coefficient of at rest earth pressure regionally observed. Classification by Robertson (1990) chart identifies this material as cemented, aged or very stiff natural soil, with a grain size distribution typical of sands or silt/sand mixtures, although its density index values are low (Fig. 3).

Correlations between CPT and SPT results are usually expressed by the ratio q_{c}(MPa)/N_{o}. In the present case values were found between 0.7 and 0.8, which is far higher than the average values for transported soils with similar grain size (reported by
Burland & Burridge, 1985). This can be explained by the fact that in these residual-cemented materials the static point resistance is more sensitive to the cohesion than the dynamic resistance, expressed by $N_{pr}$. These results confirm Brazilian reports on this subject (pseudos residual soils), with values of $q_c/N_{pr}$ 10% higher (Roche Filho, 1986).

![Graph showing normalized cone resistance ($q_c/N_{pr}$) vs. normalized friction ratio ($N_f/q_c - q_p$)].

**Fig. 3. Soil classification (Robertson, 1990).**

The evaluation of the angle of shearing resistance by means of Robertson & Campanella’s (1983) proposal conducted to higher values of $q_s$ (44 - 45°) than those obtained on triaxial tests, where the cohesive component is non-negligible. This reflects the simultaneous sensitivity of $q_s$ towards frictional and cohesive components. This methodology, being strictly frictional, does not explicitly deduct the cohesive intercept, although the value of the angle of shearing resistance has to be taken as a recent value on the stress space and, consequently, this late parameter decreases with the increasing of the vertical effective stress, fact that is very well defined on the results plotted in the formerly referred abacus. Nevertheless, care should be taken on the adoption of this solely frictional resistance for the evaluation of the ultimate load of shallow foundations as it is well recognised the strong influence of cohesion on the evaluation of $q_{ph}$ as well as the strong non-linearity of the bearing capacity factors with increasing values of $q_s$.

An attempt was made to configure the lateral resistance parameters by back-analysing CPT results in order to obtain the best pair of strength parameters using formulations based on Janbu & Semen's theory. Values of $c'$ between 15 and 40 kPa were obtained, which clearly overestimate those obtained by backanalysing multiple PLTs and triaxial tests over undisturbed samples (Viana da Fonseca, 1996).

On the same level, Puppiah et al. (1993) interpretation has resulted in a very strong overevaluation of effective cohesion which can be explained by the different cementation conditions of these natural formations towards the artificially cemented sands used by the authors. From the interpretation of the CPT results by the state parameter concept, $q_s$, the values taken from the normalised point resistance have conducted to higher strength than the ones evaluated directly from the natural void ratio, associated to the fact that mechanical properties of these residual soils are not exclusively conditioned by volumetric factors but mostly by fabric and structure.

![Graph showing $N_{fr} = q_c/(q_c^s - q_p)$ vs. $q_s$].

**Fig. 4. $G_s/q_s$ versus $q_s = q_c/(q_c^s - q_p)$ (Robertson, 1991).**

For stiffness evaluation through conepenetration results and for the same reasons referred to before, values of $q_s$ were correlated with $G_s$ and the similar low degree of dependency with penetration resistance was detected. From the integration of these values in Robertson’s (1991) proposal for transported soils with different overconsolidation ratios (Fig. 4), it can be verified that measured values of $G_s$ are substantially higher than the presented correlations, revealing a higher stability of elastic properties to penetration parameters than strength.

Correlations between $q_c$ and Young modulus, established in different stress-strain levels by triaxial tests (CIT e CDT) with local strain measurement, confirmed the very strong influence of non-linearity on $E/\gamma_q$ ratios (Viana da Fonseca, 1996).

### 2.5 Pressuremeter Testing – Prebored (Michigan – PMT) and Selfbored (Cambridge – SBPT)

The evaluation of strength parameters from PMT by means of yield or limit pressures methods, lead to a rough underestimation of strength, even taken as exclusively frictional. On the other hand, results from Cambridge SBPT interpreted by the Hughes-Wringle-Windle method (for purely frictional materials), has conducted to very similar values of peak angle of shearing resistance to the ones obtained by CPT interpretation, but still remains the
\[ E_{\text{ref}} = E \times (e/10)^{0.25}, \quad e > 10^{-4} \] (2.3)

the SBPT unload-reload moduli correspond to secant values for shear strain of 5.7-7.4 \times 10^{-4} (±0.06%). One of these values is imprinted in the relation of \( G_s \) versus \( \varepsilon_s \) on Fig. 6, using Jardine (1992) model.

<table>
<thead>
<tr>
<th>( \varepsilon_s )</th>
<th>( P_L )</th>
<th>( f_s )</th>
<th>( P_s )</th>
<th>( E_{\text{ref}} )</th>
<th>( E_{\text{ref}}/P_s )</th>
<th>( E_{\text{ref}}/f_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.350</td>
<td>14.6</td>
<td>1.4</td>
<td>10.6</td>
<td>1.4-1.9</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 6. Non-linearity - \( G_0 \) over relation \( G_{\text{ref}} \) versus \( \varepsilon_s \)


2.6 Cross-Hole Seismic Test (CH)

The results of cross-hole tests are very homogeneous and reveal a very smooth growth of \( G_0 \) with depth. In Fig. 7 these variations are compared with the lines corresponding to Ishihara’s (1986) proposals, for

\[ G_0 (\text{Ishihara}) = 4.94 \times 10^5 \times (P_s)^{0.83} \] (2.4a)

\[ G_0 (\text{Ishihara}) = 9 \times 10^5 \times (P_s)^{0.83} \] (2.4b)

Effective at rest versus strain, \( P_s = 0.64 \times (1 + \text{Equiv}_s) \) (Ishihara, 1986)

Fig. 7. Comparison of observed trends of \( G_0 \) versus \( P_s^k \) and Ishihara (1986) proposals.
Fig. 8. $V_s (CH)$ versus $V_s (N_{60 S P T})$ over depth.

natural alluvial sands, aged and cemented, corresponding to:

$$G_s (MPa) = (7.9 \pm 14.3) \cdot 0.4 \left[ (17.2 - d) \cdot \sqrt{0.4 \cdot 10^4} \right]^{1.4}$$

while for the present results the following is found:

$$G_s (MPa) = 65 \cdot \left[ (17.2 - d) \cdot \sqrt{0.4 \cdot 10^4} \right]^{1.4}$$

It can be seen that the value of the constant for the maximum shear modulus expression is much higher than the corresponding for sandy transported soils while the factor $(n)$ of dependency on the octahedral at rest effective stresses is far lower.

From the reinterpretation of the observed trends – see Fig. 8 – with reference to the most common correlations between $N_{60}$ and $V_s (CH)$ for transported aged soils (Seed et al., 1986), new values for the constants were found: fixing $c = 0.84$ and $s_1 = 1.3$, one gets $s_1 = 1.5$. Reinterpreting the same values with the direct correlations between $G_s$ and $N_{60}$ (Stroud, 1988), the following was obtained:

$$G_s (MPa) = 57 \cdot N_{60}^{1.32}$$

concluding that it is the distinct dependency of both $G_s$ and $N_{60}$ on $d$ that explains such a reformulation of the correlation between them.

2.7 Dilatometer Test (Marchetti - DMT)

Material classification in grain size terms based on Marchetti $f_d$ index is fairly consistent $(\delta_d)$ opposing the results based on CPT parameters, which identify finer groups from $f_d < 0.6$, probably due to high percentage of laminar particles (mica) which induce higher sleeve resistances on these soils. Campanella & Robertson (1991) proposal for strength evaluation gives very accurate values, only if the relation between $q_i^{\alpha = 0.8}$ and $K_{dp}$ is changed to the new trends found on the experimental campaign, i.e.:

$$q_i / \sigma_{ur}^{\alpha = 0.8} \cdot K_{dp}$$

against $q_i / \sigma_{ur}^{\alpha = 0.33} \cdot K_{dp}$ proposed by the authors for transported soils. With the same relation the evaluation of the coefficient of earth pressure at rest, $K_{dp}$, by means of $K_{dp} = 0.736 + 0.024 \cdot K_{dp} - 0.00172 \cdot q_i / \sigma_{ur}^{\alpha = 0.33}$ (2.7)

Recent trends for association of $K_{dp}$ with $K_{dp} = 0.736 + 0.024 \cdot q_i / \sigma_{ur}^{\alpha = 0.33}$ (2.7)

Stiffness classification for shallow foundations settlement assessment, having recourse to the correlations between the moduli $E_{mod}$ and $G_s$ or $E_{mod}$ (Baldi et al., 1989), resulted in new correlations with much higher ratio between those parameters, than the referenced proposals:

$$G_s / E_{mod} = 16.7 \cdot 10^{-3} \cdot \log (E_{mod})$$

$$E_{mod} (P_s) = 2.35 - 0.21 \log (P_s)$$

This last correlation, however, is situated between the laws that define – for those authors – the behaviour of NC and OC transported soils.

3. INTERPRETATION OF PLT

3.1 Failure definition

Shallow foundations failure on these residual soils is usual of punching type, which consequences on very little definition of an inflexion zone of the load-settlement curve, by which a log-log presentation is the proper way to detect it. From the PLT tests on circular rigid plates of 36 and 60cm diameter and on a concrete footing of 120cm diameter (detailed description in Vianna da Fonseca et al., 1997) the following values were defined: $q_f = 709$, 821 e 950KPa, respectively.

Using the bearing capacity equation taking account the water level position, three expressions are obtained. These were optimized to get the two strength parameters range: $c' = 6.3 - 6.9$KPa and $d' = 36.5 - 37'$, revealing a fair agreement with the previously referred laboratorial results and in situ testing interpretations.

3.2 Settlement evaluation from SPT, CPT, PIVMT, SBPT and DMT results

Some conclusions from the literature, which use SPT parameters for settlement evaluation, were tested and the following was concluded:

(i) Terzaghi & Peck proposal, has conducted to settlements 2 to 4 times higher than observed;
(ii) Parry’s (1978) proposal, assuming $\alpha = 0.3$, has given reasonable results for the very early load levels (up to 20% of failure – before yield loss – definition in Vianna da Fonseca et al., 1997), being strongly unconservative for higher ones;
(iii) Burland & Burbridge (1985) proposal (average $\alpha = 1.7$) is roughly conservative, with values of predicted settlements 2 to 3 times higher than the observed ones (for loads up to serviceability
limits which has been adapted from the Boston code, $\alpha=0.755\%$, suggesting a lower value for
$\alpha=0.835$, in accordance with similar trends in
Brazilian residual soils (Rocha Filho, 1986).
From the CPT based semi-empirical solutions for
determination of intermodular settlements, Schmertmann's method
was tested with great accuracy (fine layer discretization)
for the most representative PLT ($\alpha=0.69\%$ and
1.2h). An excellent reproduction of the observed
curves was obtained (even in non-linearity terms)
since the value of $E_{50}=6$ is modified to 4.0 to 4.5,
which are higher than the proposed and are
associated to fabric and cemented-structure effects
(Viana da Fonseca et al., 1997).
The most adapted Ménard's geological factors (a)
for correction of PLT modulus in order to get the
best convergence between observed settlements on
PLT tests (in serviceability load levels) and calculated
by means of the classical elastic solution taking
into account the concept of settlement centre
(Viana da Fonseca, 1956), was found to be typical
of silty soils (a=0.82), corresponding to the actual grain
size distribution of this unripe soil. The use of
PMT unload-reload modulus happens to give the
direct values of the Young modulus to be taken in
the same solutions. On the other hand, the values of
STPT unload-reload modulus reproduce the
behaviour of intermediate cycles in PLT tests.
Finally, a load-settlement analysis of the most
significant PLT, similar to CPT interpretation but
using DMT Modulus ($E_{50}$) was made. The non-linear
method from Leonards & Frost (1988) -- based on
Schmertmann's influence diagrams - and Robertson
(1991) were used and the best adjustment of the
experimental results were obtained for a factor of
$E_{50}^d\%=2.34$, which is an intermediate value between
NC and Gc sandy deposits (Berardi et al., 1991).
The non-linearity of both PLT curves ($E_{50}=0.69\%$ and
1.2h) was also reproduced from DMT results.

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Strength profiling in soft offshore soils

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ABSTRACT: This paper presents results from a centrifuge and 1 g testing program to investigate different soil characterization apparatus in offshore sediments. The research is aimed at developing site investigation techniques for the offshore industry. Testing devices used include the cone penetrometer and vane test, which are compared with results obtained from T-bar and ball penetrometer tests, as well as results obtained from model plate loading tests. The results are extremely consistent across a range of soil types, but it is found that adjustments need to be made to theoretical factors relating the shear strength to the primary force or torsion measurement, possibly due to strain anisotropy or other effects such as strain-softening.

1. INTRODUCTION

The most commonly used apparatus to profile offshore soils is the cone penetrometer (CPT), supported by the vane test in soft sediments. However, there are a number of limitations associated with CPT testing, particularly in very deep water and in the soft calcareous sediments found in moderately deep water off the northern and western coasts of Australia. Firstly, the cone must withstand extreme ambient water pressures yet still be sensitive enough to measure very low soil resistance in the upper layers. Secondly, large corrections, often of similar magnitude to the net cone resistance, are needed for the overburden stress, and excess pore pressure developed behind the cone. These factors have led to alternative devices, such as the T-bar (Stewart and Randolph, 1994) and ball penetrometers, where a differential penetration pressure is measured directly. The work reported here, carried out at the University of Western Australia, covers five different site investigation tools in three different soil types. The aim has been to calibrate the new penetrometer devices against each other, against direct measurements of shear strength using a miniature vane, and against theoretical relationships relating the differential pressure and undrained shear strength.

2. IN SITU SOIL TESTING APPARATUS

2.1 Penetrometer devices

Figure 1 shows the three penetration devices used in the current study, and general soil deformation mechanisms. Key differences are that:
- for the cone, the soil flow is axisymmetric but is prevented from closing behind the conical head by the full width shaft;
- for the T-bar, the flow is essentially plane strain and closes fully behind the cylindrical bar, apart from a small region around the central shaft;
- for the ball, the flow is axisymmetric, and nearly closes behind the ball, apart from a shaft that is about 14% of the cross-sectional area of the ball.

Figure 1 Penetration devices and deformation mechanisms

The cone is similar to conventional field cones used offshore. It is 10 mm in diameter, with a 60° head, and a load cell is located just behind the head.

The T-bar and ball penetrometer device consists of a load cell located at the end of a thin (4.5 mm diameter) shaft. A screw connection at the tip of the load cell enables various types of probe to be connected and penetrated into the soil. For this study a sand-blasted ball, 12 mm in diameter, and a sand-blasted T-bar, 20 mm long and 5 mm in diameter, were used.
2.2 Rotation devices

A rotary actuator has been designed and constructed at UWA specifically for centrifuge testing (Watson, 1996). The actuator fits within existing actuators that provide vertical and horizontal movement, and contains a motor-tacho-gearbox system that can provide in excess of 10 Nm of torque at speeds ranging from 0.2 to 340 deg/s. A slip-ring assembly enables recording of axial and torsional load, measured by load cells located directly behind the test probe, during continuous rotation. The rotation and angular velocity are recorded using an encoder connected to the main shaft.

At present two devices have been developed, as illustrated in Figure 2. Vane tests are performed using a traditional cruciform vane with height, \( h \), of 10 mm and diameter, \( d \), of 15 mm.

![Figure 2: Rotation devices](image)

In addition, a torsional plate load test (TPLT) has been fabricated to investigate the monotonic and cyclic response of the soil. The plate can be penetrated into the soil to various depths, then giving a bearing pressure - displacement response. At any stage, the plate can be rotated monotonically or cyclically, to give a measurement of shear strength similar to a vane test, under either constant vertical load, or fixed penetration. In the present paper, only the bearing response has been considered.

3. DERIVATION OF SHEAR STRENGTH

3.1 Penetration devices

For the penetration devices, the undrained shear strength, \( s_u \), can be obtained from

\[
  s_u = q/N
\]

where \( q \) is the net bearing pressure and \( N \) is a factor representing the relationship between shear strength and net bearing pressure for the separate devices. Subscripts of \( c \), \( t \) and \( b \) will be used for the cone, T-bar and ball respectively.

In the case of the T-bar and ball penetrometer, soil flows around the probe during penetration. Thus there is no need to correct for overburden pressure, and the measured bearing resistance, \( q_m \), is equal to the net bearing resistance, \( q_w \) or \( q_b \). The same is true of the TPLT, except that minor correction is required for friction on the side wall.

When analysing the CPT a correction must be made for both the pore pressure acting at the shoulder of the cone (pore pressure area correction) and the overburden pressure. This can be expressed as follows (Robertson and Camppanella, 1983)

\[
  q_c = q_m - \frac{\sigma' + u_w (1 - \alpha)}{1 - \alpha} \tag{2}
\]

where \( \alpha \) is the effective overburden stress, \( u_w \) is the hydrostatic water pressure, \( \sigma' \) is an experimentally calculated area correction factor for the pore pressure acting on the back of the cone and \( \beta \) represents the excess pore pressure behind the cone, expressed as a ratio of the net bearing pressure.

As will be shown later, the adjustment of the measured bearing resistance to obtain \( q_c \) introduces significant uncertainty in estimating the shear strength of soft soils, where the ratio of strength to overburden stress is low. In the current work, the area correction factor, \( \alpha \), was evaluated by comparing calibrations of the cone using direct load and fluid pressure, and was found to be 0.131. The excess pore pressure ratio, \( \beta \), was taken as unity from previous experience with piezocene tests in calcareous silt and kaolinite. Note that correction of the cone resistance using equation (2) will typically lead to net cone resistance that is some 20 per cent higher than that obtained by just subtracting the overburden stress from the measured resistance.

Values of \( N \) for the various devices have been determined from a combination of experimental and numerical analysis. For the cone penetrometer, \( N_c \) can vary widely, depending on the sensitivity and internal structure of the soil, with quoted values in the range 11 to 17 (Robertson and Camppanella, 1983). For the reconstituted soils considered here, experience has indicated cone factors in the range 10 to 12, and an average value of 11 has been adopted.

An exact plasticity solution for the T-bar, confirmed by finite element analysis, has yielded a range of \( N_t \) between 9.5 (smooth) and 11.9 (rough). A value of 10.5 has been selected here, as suggested by Stewart and Raudolph (1994).

Preliminary finite element analysis of the ball penetrometer indicates that for any given roughness,
the ball factor, $N_b$, is about 25% greater than that for the T-bar, and that a value of 13 to 13.5 should be appropriate. However, an intriguing outcome of the present research has been that the experimental results indicate identical values for the T-bar and ball factors.

Bearing capacity factors for the T-bar were derived using the solutions presented by Housby and Wroth (1983) for a circular footing bearing on the surface of soil with strength varying linearly with depth. The bearing capacity factor will reduce as the plate penetrates, since the effect of strength increase below the plate reduces. The actual profile of $N_b$ is presented later.

Direct use of bearing capacity factors for a surface foundation is somewhat questionable for a plate that is ultimately embedded by 4 or 5 diameters, and will undoubtedly underestimate the bearing capacity factor at high embedment. This is partly compensated for by allowance for side friction, which has been taken as 40% of the soil shear strength at the plate-soil interface, and the full soil shear strength over the 3 mm skirt depth (where there is soil-soil shearing). However, as will be demonstrated, using $N_b$ factors based on those for shallow footings produces an estimate of shear strength in excess of those derived from the other devices, and the use of a global $N_t$ equal to 10.5 produces a more acceptable shear strength profile. Further studies are needed in this area.

3.7 Vane shear

The vane shear apparatus simply measures the torque, $T$, required to overcome the shear resistance on the surfaces of the cylinder circumscribing the cruciform blades of the vane. Assuming full shear strength mobilised on each surface, the deduced shear strength may be expressed as

$$\tau = \frac{T}{\pi d^2 (h/2 + d/6)} \tag{4}$$

It should be noted that, as for the other devices, no account is taken of strength anisotropy, or progressive failure, in the derivation of equation (4).

4. SOIL TYPES

Soil characterisation tests have been undertaken in three different soil types, both in the geotechnical centrifuge at UWA and also on the laboratory floor at 1 g. Calcareous clay and calcareous silt have been tested at 120 g and 150 g respectively, while kaolin has been tested at 1 g.

The calcareous sediments come from bulk samples taken from separate seabed locations on the North-West Shelf and the Timor Sea off the coast of Australia. They are characteristic of the weaker types of sediment found in deeper water (200 m to 900 m). Due to the limited data base of laboratory tests on these two materials, kaolin was selected as a baseline material for the research program.

5. RESULTS

All tests were conducted under nominally undrained conditions. Coefficients of consolidation lie in the range 1 to 3 m²/year (noting that silicon oil was used as the pore fluid for the calcareous silt), and a standard penetration rate of 3 mm/s was adopted for the probes. This section will focus primarily on the tests in the calcareous clay, with supporting data from the calcareous silt and kaolin tests.

5.1 Calcareous clay

5.1.1 CPT, T-bar and ball penetrometer

Figure 3 shows the bearing pressure profiles for the T-bar, ball and CPT.

Note that in deducing the net bearing resistance profile for the CPT the effective unit weight of the soil was taken as 5.5 kN/m², while the correction parameters, $\alpha$ and $\beta$, were taken as 0.131 and 1.0 respectively, as discussed previously.

As can be seen, the resistance measured directly by the CPT is significantly higher than for the T-bar and ball penetrometers, highlighting the required correction for overburden pressure and pore pressure build up. When this correction is made, the deduced net bearing resistance profiles are virtually identical.

The deduced shear strength profiles for the calcareous clay are shown in Figure 4, together with the various factors. As can be seen, the three penetration devices predict a similar shear strength profile for the calcareous clay, and indeed it would seem that a cone factor of 10.5 might be more appropriate than the value of 11 adopted.
A tentative conclusion is that the bearing resistance is independent of the shape of the penetrometer, in contrast to finite element studies that indicate a higher bearing capacity factor for the axisymmetric ball compared with the plane strain T-bar. An additional benefit of the T-bar and ball penetrometers is that information can be obtained on the remoulded strength of the material during extraction of the probe. Figure 5 illustrates the pullout behaviour of the T-bar and ball in calcareous clay. The remoulded strength is approximately 70% of the undisturbed strength (sensitivity ~ 1.4).

5.1.2 Vane shear tests

The vane shear device allows multiple tests, vertically and at different locations in plan, to be conducted during a single centrifuge test. Although not presented here, tests in the calcareous silt and kaolin samples have indicated minimal effects on the observed shear strength of (a) the wait time after installation (prior to testing), and (b) rotation rate.

Figure 6 below illustrates the observed vane response in the calcareous clay at various depths. As can be seen, a clear peak is reached after approximately 15 degrees of rotation, followed by strain-softening with further rotation. The degree of strain-softening increases markedly with depth, with the ratio of peak to residual ranging from 2.1 to 3.5, with an average of 2.4.

The strain-softening in the vane test, and trend of increased 'sensitivity' with depth, is much greater than that observed for the T-bar and ball penetrometer. However, that is to be expected, given the well defined shear planes in the vane test. As the vane rotates, the tendency of the soil to exhibit volumetric collapse will lead to reduced normal effective stresses on the shear planes, and gradually reducing friction as the vane rotates.

Figure 7 plots the observed peak and residual shear strengths deduced from the vane shear tests, compared with the penetrometer strength profiles shown previously. Also shown are 'factored' vane strengths, obtained by reducing the peak strengths by 17%. It may be seen that this leads to a strength profile that is consistent with those from the penetrometers, with a strength gradient of approximately 2.5 kPa/m. The need to factor the peak vane strengths is discussed further in Section 6.
5.2 Calcereous silt

A similar range of soil characterisation tests has also been conducted in calcereous silt, in centrifuge model tests at 150 g. In addition, the torsional plate loading test (TPLT) has been used to obtain a profile of bearing capacity with depth. The results from the TPLT will be discussed first, before comparing the strength profiles obtained from all the tests.

5.2.1 TPLT

As described previously, the TPLT consists of a circular plate (diameter 30 mm, thickness 3 mm) with vane type skirts penetrating an additional 3 mm (10 % of the diameter) into the soil, in the manner of a torvane. At a centrifuge acceleration of 150 g, the plate represents a 4.5 m diameter foundation. The shear strength profile of the soil can be deduced from the bearing response curve as the foundation is penetrated into the soil.

The theory used to deduce shear strength from net bearing pressure has been outlined in Section 3 above. Visual observations confirmed that the soil flows over the top of the foundation during penetration, and hence the measured bearing resistance is identical to the net bearing pressure, q. The profile of net bearing pressure, and effective bearing capacity factor, Nq (including allowance for side friction) is shown in Figure 8. As commented earlier, the limiting value of 6.7 at large penetration makes insufficient allowance for the depth of burial, and further refinement is needed for the TPLT.

Figure 8  Deduced q and Nq profiles for TPLT

5.2.2 Comparison of deduced shear strength

Figure 9 details the results obtained from the various soil characterisation tests in determining s_u. In the analysis, T-bar and ball N factors of 10.5 were used, while a correction factor of 0.83 has been applied to the peak vane strengths. These factors are consistent with those used for the calcereous clay. Although the residual strengths for the vane are not shown, the average sensitivity observed was about 1.1 with almost no strain softening evident.

As can be seen, the results from the various tests agree well, particularly in the upper 12 m. The s_u gradient of the calcereous silt is 0.85 kPa/m.

An s_u profile from the TPLT using Nq factors from Figure 8 is also shown, and compared to that deduced using a constant Nq equal to 10.5 for the full penetration. The results indicate that using Nq equal to 10.5 provides a far better comparison with the other devices.

Figure 9 Shear strength of calcereous silt from various soil testing devices

5.3 Kaolinite

A third set of tests was conducted (at 1 g) on a pre-consolidated sample of kaolinite to compare the various devices. Again, identical factors were used for the T-bar and ball penetrometers, and for factoring the vane shear strengths. For the vane tests, an average sensitivity of 2.1 was observed.

Figure 10 Shear strength of kaolinite from various soil testing devices
Figure 10 shows the test results, with excellent agreement between the two probe tests, and a tendency for the factored vane strengths to lie just below the penetrometer values. This is possibly due to swelling of the sample at 1 g.

The average strength (ignoring the upper 50 mm) is about 25 kPa, which is 10% below the value of 28 kPa expected for this kaolin, for the consolidation stress of 150 kPa (Stewart, 1992).

6. DISCUSSION

The observed CPT and T-bar data agree with existing theory. The bar factor of 10.5 for the T-bar is consistent with the expected strength for the kaolin sample (from the consolidation stress), and also with the vane strengths. Any larger factor for the T-bar would lead to unacceptably low strengths.

Perhaps the most intriguing aspect of the tests is the apparently identical factors for the ball and the T-bar, in spite of numerical analysis that indicates ball factors 25% higher than for the T-bar. At this stage, it is recommended that a ball factor of 10.5 be adopted. This factor is supported by additional laboratory testing (not included in this paper) using ball penetrometers of varying size, and also from field vane and ball penetrometer tests conducted in soft mine tillng deposits (Newson et al., 1998). The low factor, compared with theoretical estimates, may be due to strength anisotropy, or differences between plane strain and axisymmetric shearing.

The vane tests give shear strengths which are between 10% and 20% higher than those derived from the penetrometer devices. The magnitude of the reduction in peak vane strength is consistent with factors suggested by Azzouz et al. (1983), from the back-analysis of three-dimensional embankment failures. The discrepancy in the strengths measured by the vane and the various penetrometers may be attributed partly to strength anisotropy and partly to strain-softening effects.

The vane tests showed a significant amount of strain-softening, particularly in the calcareous soils. In comparison with the penetration devices, which measure some form of average shear strength, it is thus likely that the peak strength measured by the vane represents an upper limit on shear strength. This implies that some form of average between residual and peak shear strength would be better for comparison with the penetrometer devices.

Davies et al. (1989) demonstrated how strength anisotropy can be important, comparing results from vanes of different aspect ratio. They showed that reducing the aspect ratio from 0.78 to 0.4 (compared with 0.87 in the present study) caused an apparent reduction in shear strength by 30%.

7. CONCLUSIONS

This paper has presented results from several soil characterisation devices in various soil types. The soil types are typical of those found on the North-West Shelf and Timor Sea off the coast of Western Australia, and the results generated have shown the potential benefits of alternative penetration devices in characterising soft offshore soil deposits.

For soft soils, the large corrections required in the analysis of CPT results, where the overburden pressure is of similar magnitude to the net bearing pressure, leads to the conclusion that devices such as the T-bar or ball penetrometers are superior to the cone, since they measure a differential bearing resistance directly.

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9. REFERENCES


Advanced interpretation for flow and consolidation
Potentiometric surface mapping with a CPT: Case studies and a word of caution

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ABSTRACT: The use of pore pressure measurements with a Cone Penetrometer (CPT) for mapping water tables and vertical hydraulic gradients was tested in clay-silt-sand sequences in the mid-western U.S.A. Dissipation tests were conducted in the vicinity of existing monitoring wells. The data indicate that an apparent value for equilibrium pore pressures may be obtained which may be lower or higher than the true value, depending upon the composition and grain size of the aquifer material, the depth of the dissipation test within the saturated zone, and the use history of the porous filter in the CPT assembly. Consequently, the dissipation data must be carefully calibrated and tested with measurements in monitoring wells before they are used for hydraulic mapping at a given site.

1 INTRODUCTION

Characterization of subsurface hydrogeology is one of the major objectives of remedial investigations at hazardous waste sites with soil and/or groundwater contamination. This characterization includes determining the depth to water table or the potentiometric surface elevation, and the horizontal and vertical hydraulic gradients in the aquifer. Conventional approaches to hydrogeologic characterization use drilling of soil borings and monitoring wells for hydrologic measurements and testing. These methods are environmentally intrusive, expensive, and provide hydraulic parameters that are averaged on a relatively large scale.

The Cone Penetrometer (CPT) is increasingly being used for rapid hydrogeologic characterization in environmental investigations (Janicki et al., 1985; Struybisky and Sainey, 1990; Sharp et al., 1992; Chiang et al., 1992). A potentially powerful use of the CPT is for mapping the horizontal and vertical hydraulic gradients in an aquifer by using the measurements of pore pressure (Campanella et al., 1982; Robertson et al., 1986). In the saturated zone, the pore pressure refers to the hydraulic head at the depth where measurements are made. Measurement of pore pressures at spatially distributed locations or from several depths at a single location, therefore, provide the magnitude of the horizontal and vertical hydraulic gradients. Whereas the use of this technique has been implemented in field investigations, a systematic evaluation of the technique does not appear to have been conducted. In this paper, we present the results of a study for hydraulic mapping by using pore pressure measurements at two sites in the midwestern U.S.A. It is concluded that while the use of the CPT is extremely attractive for hydraulic mapping, careful calibration of the CPT measurements with monitoring well data must be obtained at each site.

2 BACKGROUND

A CPT consists of a cone at the end of one-meter sections of steel rods that are pushed into the ground using hydraulic rams (Meigh, 1987). In its basic configuration, the cone is fitted with strain gauges and a pressure transducer. The strain gauges measure the tip resistance and sleeve friction during penetration. A pressure transducer is located just behind the tip to record pore fluid pressures through a porous filter. Tip resistance, sleeve friction and pore pressures measurements are recorded continuously as the cone is being advanced. Because of the disturbance of the material around the cone during penetration, in situ pore pressures are altered. Significantly high pore pressures are developed in cohesive, lower permeability materials such as clays, due to consolidation around the cone (Campanella et al., 1982; Elsworth, 1991; Aggarwal et al., 1994). In higher permeability materials such
as coarse-grained sand layers, cone penetration may result in both consolidation and dilation of materials being penetrated (Aggarwal et al., 1994). When dilation occurs, the instantaneous pore pressures are lower than equilibrium pore pressure.

The anomalous instantaneous pore pressures generated during cone penetration dissipate or equilibrate over time to a lower or higher value as the materials reach equilibrium. The change in pore pressure with time at a given depth can be recorded by arresting the penetration of the cone. Dissipation time required to reach equilibrium depends upon the hydrological and mechanical properties (permeability, compressibility, strength, etc.) of the materials (Campanella et al., 1982; Robertson et al., 1986; Elsworth, 1992, Aggarwal et al., 1994) and can be used to map the distribution of saturated and unsaturated layers as well as vertical differences in permeability of the materials (Aggarwal et al., 1994).

The equilibrium pore pressure measured in the saturated zone provides an estimate of the hydraulic head at the depth of measurement. The hydraulic head due to a column of fluid is equal to \( \rho gh \), where \( \rho \) is the fluid density, \( g \) is acceleration due to gravity and \( h \) is the height of the column. Thus, for dilute waters in shallow aquifers, the pore pressure at a given depth in the saturated zone is related to the height of a water column by a factor of 2.308, i.e., a pressure of one pound per square inch (psi) is exerted by a 2.308 ft column of water. The height of the water column obtained from pore pressure measurement can be subtracted from the depth of measurement to determine the depth to water table (or potentiometric surface in confined aquifers). By using this methodology, multiple.
measurements at spatially distributed locations or at several depths at a single location can be used to map horizontal and vertical hydraulic gradients.

3 SITE DESCRIPTION

Pore pressure measurements were obtained at two areas near Lincoln, Nebraska in the midwestern U.S.A. The areas of study are designated Site A and Site B. Loess, clay, and silty-clay with alternating sand and clay layers form the unsaturated zone at both sites. The saturated zone consists of fine- to medium-grained sand with clay, silt, and silty clay layers. Shallow groundwater occurs under unconfined or semi-confined conditions with the depth to water table being about 33 ft at Site A and about 72 ft at Site B. A network of monitoring wells with screened intervals of 5 ft installed at various depths within the saturated zone was available at both sites. Dissipation tests were conducted at both sites in the vicinity of existing monitoring wells. Water level measurements in these wells indicated that groundwater flow was primarily horizontal and that the vertical hydraulic gradient was negligible.

4 RESULTS AND DISCUSSION

The instantaneous pore pressures at both of the sites indicated compaction and dilation as a result of cone penetration, depending upon the nature of the aquifer material. Typical dissipation curves observed at the two sites are shown in Figures 1 and 2. Equilibrium pore pressure was defined as the pressure when the slope of the dissipation curve was nearly zero for more than about 10 minutes of observation.

4.1 Site A

Equilibrium pore pressure at Site A was measured using the CPT at 3 or 4 depth intervals at three different locations. The expected pore pressure at each of the CPT testing depths was calculated from the measured depth to water table in an adjoining monitoring well and the mid-point of the screened interval. Figure 3 shows the equilibrium pore pressure obtained by using the CPT and the expected pore pressure. If the pore pressure measurements reflect in situ pore pressure, then all of the observations should lie along the "ideal curve" in Figure 3. However, pore pressure measurements only at the first depth interval at a given location plotted on the "ideal curve". Subsequent dissipation tests at deeper depth intervals within the same hole plot below the "ideal curve", indicating that the pore pressure equilibration occurred at a pressure below that expected from the depth of the water table in the monitoring wells. This difference in multiple measurements was not due to a vertical gradient in the aquifer because, as noted previously, water level measurements in a cluster of monitoring wells installed at different depths indicated that there was no significant vertical gradient in the aquifer.

Because the first dissipation test always produced the equilibrium pore pressure expected from measuring well data, it was postulated that the porous filter in the CPT assembly perhaps was partially clogged during penetration, creating a pressure differential across the filter. This pressure differential may cause the apparent equilibration of the pore pressure at a value higher or lower than that in the aquifer.

The hypothesis was tested by measuring the pressure of a 2.308 ft column of water using the cone assembly before and after a penetration test. With a fresh filter before the test, a pressure of 1 psi was recorded, as expected. However, after a single dissipation test, the recorded pressure was much lower, ranging from 0.45 to 0.65 psi. This indicated that the ceramic filter in the cone should be used for only a single dissipation test.

Pore pressure dissipation measurements with a new filter for each test were then obtained from several locations at Site A. The water table map constructed from these measurements was in excellent agreement with the water levels measured in the monitoring wells.

4.2 Site B

Dissipation tests at Site B were performed at several depths, but at a single location. All measurements at different depths were made with a new filter. Measured and expected pore pressures at Site B are shown in Figure 2. Even with a new filter, a reproducible measurement of equilibrium pore pressure could not be obtained at different depths. Additionally, the measured equilibrium pore pressure at several depth intervals was greater than the expected value. This higher value of the pore pressure did not result from an early termination of the dissipation test because none of the dissipation tests ended when the values were unstable.

In addition, inconsistent results were obtained in repeated measurements at the same depth interval. This indicates that in addition to the use history of the porous filter, the depth to water table and composition of the aquifer materials (clay-silt-sand proportions) may impact the equilibration of pore pressures recorded in a cone penetrometer. Consequently, one must carefully evaluate the applicability of the pore pressure dissipation
5 CONCLUDING REMARKS

Under ideal conditions and if excess pore pressures are not developed during penetration in the saturated zone, the instantaneous pore pressure should be zero at the water table. Below the water table, the pore pressure curve should have a slope equal to the increase in hydrostatic pressure with depth. Such conditions, however, are rare in actual field situations. The ability to map water table or potentiometric surface using equilibrium pore pressure data appears to depend on site specific conditions and may be possible only if carefully tested and calibrated against monitoring well data.

6 REFERENCES


Evaluating the state of consolidation of clay at a reclaimed site

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ABSTRACT: Reclaimed sites in Singapore are usually underlain by highly compressible marine clay. The evaluation of the state of consolidation of the underlying clay is important for the development of recently reclaimed sites. Three in-situ tests, the piezoecone penetration test (CPTU), the flat dilatometer test (DMT), and the field vane test (FVT) were used for evaluating the vertical yield stress of the foundation clay underlying a large reclaimed site in Singapore using interpretation methods developed on the basis of Cam clay model. Interpreted values of overconsolidation ratio (OCR) from the CPTU and the FVT were slightly higher or comparable to those deduced directly from oedometer tests and in-situ porewater pressure measurements using field piezometers and the piezocone. The OCR values interpreted from the DMT were generally lower than those directly measured. The in-situ tests were found useful for evaluating the state of consolidation of clay.

1. INTRODUCTION

Reclaimed land created typically by the placement of sandfill over existing seabed forms a significant part of Singapore's total land space. Reclaimed sites in Singapore are generally covered by newly placed hydraulic sand fill and underlain by highly compressible foundation clay layers consisting primarily of the Singapore marine clay. The evaluation of the state of consolidation of the clay, as part of site characterization, is important for newly reclaimed sites not only because it dictates the amount of future settlement but also because it controls the behaviour of the clay when it is subject to future loading. Traditionally, the state or the degree of consolidation of a clay stratum is evaluated either by direct measurements of the in-situ excess pore water pressures from field piezometers or by direct evaluation of the preconsolidation pressures from oedometer tests on undisturbed samples. Occasionally, settlement records can also be used if the distribution of settlement with depth is available. In addition to technical problems such as reliability of the field piezometers and difficulty of obtaining high quality undisturbed samples, these traditional methods are costly.

With the rapid development of field testing techniques in the past two decades, in-situ tests have gradually become the major means of site characterization even in soft to medium stiff clays for which the determination of various key soil parameters relies traditionally on laboratory tests. Because of the relevance of in-situ test measurements to the stress history of clays, results of many selected in-situ tests provide an alternative to the evaluation of the vertical yield stress. The success of in-situ tests however depends on the method of interpretation.

The objective of this paper is to investigate the usefulness of in-situ tests for the evaluation of the state of consolidation of consolidating clays. Data collected from three in situ tests including the piezocone penetration test (CPTU), the flat dilatometer test (DMT), and the field vane test (FVT) carried out at two test locations at a large reclaimed site in the eastern part of the Singapore island are used in the investigation. By re-defining the overconsolidation ratio (OCR) as the in-situ vertical yield stress over the future equilibrium vertical effective stress for clay underlying reclaimed sites, appropriate methods of interpretation can be developed or selected for estimating the OCR or the vertical yield stress based on the measurements from these tests. Interpreted results can be compared with
vertical yield stresses measured in oedometer tests and deduced from in-situ porewater pressures observed in field piezometers and piezocone dissipation (CPTUD) tests for verifying the validity of the interpretation methods.

2. SITE CONDITIONS

The Changi East reclamation site is located at the eastern end of Singapore. The mean sea level was +1.6 mCD in the area. The seabed, originally at -3.0 mCD, was underlay by the Singapore marine clay. The marine clay comprises of the upper marine clay layer, the intermediate sandy clay layer, and the lower marine clay layer. The total thickness of the marine clay varies from several metres to as much as 40 metres across the site. At two locations FT-8 and FT-9 that were selected for the investigation, the total clay thickness is around 25 metres.

Land reclamation of the area started in March 1993. At the two selected locations, FT-8 and FT-9, which are 70 metres apart, the reclamation work was completed to an elevation of +4 mCD in August 1994. Subsequently, vertical drains with square grids of a spacing of 1.5 m were installed at FT-8 location, which was followed by the placement of an additional 6 m of surcharge to +10 mCD in December 1994. In the area surrounding FT-9, where no vertical drain was installed, a surcharge of the same height was placed in October 1994. The site was subsequently investigated by different testing and monitoring techniques in the period between May and November 1996.

3. METHODS OF INVESTIGATION

A cone penetrometer rig was used for the piezocone penetration test and the flat dilatometer test. A standard 10cm² cone with a filter element located at the base immediately behind the conical tip was used in the CPTU, whereas a Marshetti dilatometer was used in the DMT. In both tests, a standard penetration rate of 20 mm/s was adopted.

For the FVT, a Geonor vane was used and a standard rotation rate of 0.27/s was adopted during shear.

Standard one-dimensional consolidation tests with 24-hour constant duration were carried out using fixed ring oedometer for determining the pressure-void ratio relationship from which the preconsolidation pressure was assessed. Sineo pneumatic piezometers were used for monitoring the in-situ pore pressures. Special piezocone dissipation tests in which the pore pressure decay was monitored continuously up to 30 hours were carried out using a standard 10cm² cone with the filter at the base for the determination of the equilibrium pore pressures.

4. METHODS OF INTERPRETATION

For CPTU and DMT, the adopted interpretation methods are based on a consideration that penetration of these penetrometers in clay can be modelled as an expansion of a cavity in modified Cam clay. For FVT, the rotation action of a vane in clay is treated as a simple shear in modified Cam clay in the development of the interpretation method.

4.1 Piezocone test

Both the cone penetration resistance (qₚ) and the penetration pore pressure at the base of the cone, as obtained from the piezocone test (CPTU) using a standard cone are believed to be strongly related to the stress history of the soil (Wroth, 1984). These data can therefore be used to estimate the overconsolidation ratio (OCR).

According to Chang et al. (1996), for a normally consolidated clay with the vertical preconsolidation pressure (σₒc) equal to the initial effective vertical stress (σₑₒ), the cone penetration may be simulated by the expansion of a spherical cavity, and the cone resistance q can be expressed as

\[ q = \frac{2}{3} \alpha_s \sigma_{vo} M (1/2)^B (\ln I_v + 1) + \sigma_{vo} \]

(1)

where \( \alpha_s \) is the strain rate correction factor for cone penetration, I is the rigidity index, M is the slope of the critical state line and A is the plastic volumetric strain ratio.

Based on the expansion of a spherical cavity, the penetration pore pressure on the cone face u can be expressed as

\[ u = q - \alpha_s (0.67M + 1) \sigma_{vo} (1/2)^B \]

(2)

At reclamation sites, the preconsolidation pressure (σₒc) of the consolidating clay may be assumed to be equal to the in-situ effective stress (σₑₒ) acting on the clay. The difference between the equilibrium effective stress σₑₒ and the existing preconsolidation pressure (σₒc) is the excess pore pressure (uₑ). By
re-defining OCR as $\sigma'_{uc}/\sigma_{un}$, the cone resistance and the penetration pore pressure, respectively, in a consolidating clay become

$$q_s = \frac{2}{3} \alpha_u \sigma_{un} M(1/2)^\alpha \text{OCR}(\ln l_i + 1) + \sigma_{uu}$$

(3)

$$u_s = q_s - \alpha_u (0.67M + 1)\sigma_{un}(1/2)\text{OCR}$$

(4)

For a soft clay which is still undergoing consolidation, the difference between the pore pressure on the cone face $u_s$ and the pore pressure at the cone base $u_{base}$ is generally very small. Based on a limited number of tests carried out using a dual-filter cone at another reclaimed site in Singapore, Chang (1995) found that the maximum difference was around 12 kPa. In such cases, it may be reasonable to assume that $u_{base}$ is equal to $u_s$ and that it may be evaluated by using the following expression

$$u_{base} = q_s - \alpha_u (0.67M + 1)\sigma_{un}(1/2)\text{OCR}$$

(5)

From Equation (5), the overconsolidation ratio (OCR) of a clay which is undergoing consolidation may be expressed in terms of $q_s$ and $u_{base}$ as

$$\text{OCR} = \frac{q_s - u_{base}}{\alpha_u \sigma_{un}(1+0.67M)(0.5^\alpha)}$$

(6)

In Equation (6), $\alpha_u$, $M$, and $A$ are required as input parameters. The strain rate factor ($\alpha_u$) can be taken as 1.64 for the standard cone penetration rate of 20 mm/s. The critical state parameters $M$ (slope of the critical state line) and $A$ (plastic volumetric strain ratio) can be evaluated from laboratory triaxial compression and oedometer tests, respectively. The value of $M$ is directly related to the effective angle of internal friction $\phi'$, which can be evaluated by CIUC tests.

4.2 Dilatometer test

The lift-off pressure measured immediately after the penetration of a flat dilatometer blade in the dilatometer test, $P_n$, can be assumed to be equal to the limit pressure developed in the expansion of a cylindrical cavity in modified Cam clay (Cao, 1997). For a normally consolidated clay with $\sigma'_{oc}$ equal to $\sigma'_{up}$, the lift-off pressure $P_n$ can be expressed as

$$P_n = \sigma'_{up} + \frac{1}{\sqrt{3}} \alpha_u M \sigma'_{up} \left( \frac{1}{2} \right) \text{OCR}(\ln l_i + 1)$$

(7)

where $\alpha_u$ is the strain rate correction factor for the penetration of a standard dilatometer.

For a consolidating clay, Eq. (7) becomes

$$P_n = \sigma'_{up} + \frac{1}{\sqrt{3}} \alpha_u M \sigma'_{up} \left( \frac{1}{2} \right) \text{OCR}(\ln l_i + 1)$$

(8)

Using Equation (8), the overconsolidated ratio of a consolidating clay can be estimated as

$$\text{OCR} = \frac{\sqrt{3}(P_n - \sigma'_{up})}{0.5^\alpha \sigma'_{up} M(\ln l_i + 1)}$$

(9)

In Equation (9), $\alpha_u$ can be taken as 1.57 for a standard dilatometer test with a blade penetration rate of 20 mm/s. The critical state parameters $M$ and $A$ can be evaluated from laboratory tests, as described earlier. The rigid index $l_i$ can also be evaluated from the triaxial compression test.

4.3 Field vane test

According to Cao (1997), the rotation of a vane in soft to medium stiff clay can be modelled as simple shear in modified Cam clay. The undrained shear strength from the field vane test in a normally consolidated clay can be expressed as

$$(\phi)_{fv} = \frac{1}{\sqrt{3}} M \sigma'_{up} \left( \frac{1}{2} \right) \cos \phi'_{ps}$$

(10)

where $\phi'_{ps}$ is the effective friction angle for plane strain compression, which can be estimated as between 1.3 $\phi'$ and 1.4 $\phi'$ for Cam clay (Cao, 1997).

Using the new definition of OCR for a clay which is still undergoing consolidation, Eq. (10) can be rearranged as follows for the prediction of OCR at reclaims sites

$$\text{OCR} = \frac{2^{1/2} \sqrt{3} (\phi_{fv})_{fv}}{\sigma'_{up} \cos \phi'_{ps}}$$

(11)
5. MEASURED AND INTERPRETED RESULTS

Preconsolidation pressures deduced from laboratory oedometer tests on undisturbed samples and vertical yield stresses inferred from in-situ porewater pressures as observed in piezometers are used as the reference values for evaluating the validity of methods of interpretation for various in-situ tests. In addition, equilibrium porewater pressures as observed from a number of specially conducted long duration piezocone dissipation tests (CPTUDs) are also included for comparison.

For the Singapore marine clay at Changi, the \( q' \) value was found to range from 20° to 25° and the average value is 22.8° for the upper and the lower marine clays. This corresponds to an average M value of 0.89. The value of \( A \) ranges from 0.72 to 0.94, with an average value of 0.89 for both the upper and the lower marine clays (Choo, 1997). The rigidity index \( I_r \), on the other hand, was found to average at 66 and 79, respectively, for the upper and the lower marine clays. These typical parameters were used in the prediction of OCR values from the in-situ tests.

Figure 1 shows the OCR profiles predicted from \( q' \) and \( M_\tau \) measured in two post-reclamation CPTUs at FT-8 location. The predicted OCR values are slightly higher than those directly measured in oedometer tests, which were conducted 5 month earlier than the CPTU. The predicted OCR values are comparable to those deduced from the equilibrium pore pressure recorded in long-duration piezocone dissipation tests (CPTUDs) and in-situ pore pressures measured in field piezometers.

Figure 2 shows the comparison of OCR profiles predicted using Eq. (9) from the dilatometer test data with discrete OCR values from oedometer tests and in-situ pore pressure measurements at FT-8 location. The estimated OCR values, which were generally on the order of 0.3 to 0.4, were much lower than the directly measured values of around 0.9 at the time of dilatometer testing. The reason for such a discrepancy is not clear. The fact that the trend line as indicated by the DMT predicted OCR profile is similar to that of directly measured data suggests that a systematic error such as improper seating of the sensing disc in the dilatometer blade might have contributed to an error in the dilatometer measurements.

Figure 3 shows the predicted OCR profile from the field vane test data by Eq. (11) at FT-8 location. The predicted OCR values are in good agreement with other directly measured data.

Similar comparisons on the OCR profiles interpreted from CPTU, DMT, and FVT with the directly measured OCR values at FT-9 location are shown in Fig. 4, 5, and 6, respectively. Both the CPTU and the FVT provide OCR values slightly higher than those directly measured. The DMT interpreted OCR profile indicates values that were only marginally lower than those directly measured.
6. STATE OF CONSOLIDATION

The value of OCR, defined as \( \sigma'_v / \sigma'_u \), is an index of the state of consolidation of a consolidating clay. The degree of consolidation, defined as the ratio of \( (\sigma'_v - \sigma'_u) / \sigma'_u \), can be deduced from the OCR if \( \sigma'_v / \sigma'_u \) is known.

It is interesting that in Fig. 1, the OCR values are relatively consistent with depth. The directly measured OCR values are on the order of 0.7 to 0.8 based on oedometer tests in May 1996. These OCR values correspond to a degree of consolidation of 65 to 80% in the upper marine clay and 40 to 69% in the lower marine clay at FT-8 location at the time of investigation. The directly measured OCR values as deduced from in-situ pore pressure measurements
in October 1996 are around 0.9. The corresponding minimum degree of consolidation is about 88% for the upper marine clay and 80% for the lower marine clay. The rapid increase in the degree of consolidation over a period of five months clearly indicates the effectiveness of the vertical drains.

Comparing the OCR profile as indicated by the direct measurements in Fig. 4 with that in Fig. 1, the OCR profile at FT-9 location is different from that at FT-8 not only in the range of OCR values but also in its variation with depth. The directly measured OCR increases approximately linearly with depth and varies from around 0.3 on the top of the upper marine clay to 0.6 at the lower part of the lower marine clay at FT-9 location. The corresponding minimum degree of consolidation are only about 10% in the upper marine clay layer and 5% in the lower marine clay layer. Comparing these values to the degrees of consolidation of the clay layers at FT-8 location where vertical drains have been used to accelerate the consolidation process, the effectiveness of the vertical drains is clearly illustrated.

7. CONCLUSIONS

This study has illustrated the usefulness of in-situ tests for characterizing the state of consolidation of clay at the Changi East Reclamation site in Singapore. Comparing the OCR values predicted from measured data in the piezocone test, the dilatometer test and the field vane test in consolidating clay using the proposed methods with directly measured OCR data from oedometer tests, long duration piezocone dissipation tests, and piezocone measurements, one can draw the following conclusions:

1) The OCR values interpreted from the corrected cone resistance \( q_c \) and the penetration pore pressure at the cone base \( u_b \) measured in the CPTU are slightly higher or comparable to the directly measured values.

2) The lift-off pressure \( (P_d) \) from the dilatometer test provides OCR values that are generally lower than the directly measured values.

3) The OCR values interpreted from the untriaxial shear strength measured in the field vane test are slightly higher or comparable to the directly measured values.

8. ACKNOWLEDGEMENT

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9. REFERENCES


Pore pressure dissipation rates as a discriminant in evaluating permeability distributions from piezocene tests

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ABSTRACT: Results are presented to demonstrate the use of pore pressure dissipation rate \((\frac{dp}{dt})\), rather than conventional pore pressure dissipation \((p(t))\), in defining the hydraulic parameters of consolidation coefficient and coefficient of permeability from CPT data. The primary advantage is that early time dissipation data may be used, rather than waiting for full dissipation to proceed to a post arrest null overpressure. This procedure may be used to accelerate the execution of dissipation tests, resulting in a significant time saving in fine grained soils. A dislocation model (Elsworth, 1993) is used to evaluate the form of the pore pressure field developed around an advancing penetrometer during partially drained penetration, enabling construction of a series of nondimensional type curves defining dissipation rate, \((\frac{dp}{dt})\), versus time following penetrometer arrest. The proposed data reduction method is applied to a series of five dissipation tests conducted at York and Navarro, Kansas (Aggarwal et al., 1994). The maximum pressure dissipation rate magnitudes are recovered and used to evaluate the hydraulic conductivity. These are compared directly with hydraulic conductivity magnitudes evaluated from the peak pressure magnitude (Elsworth, 1993) and from pump test results. The general correspondence is encouraging.

1 INTRODUCTION

A variety of methods are available to evaluate the parameters of hydraulic conductivity (analogous to permeability) and consolidation coefficient (analogous to hydraulic diffusivity) of saturated soils using cone penetrometers. These methods include both the use of specially equipped cones capable of performing in situ slug type tests and inference of parameters from post arrest pressure dissipation behavior (Tørrissen et al., 1987; Szewczyk et al., 1989; Lefondre and Baligh, 1986; Elsworth, 1993). The advantage of using dissipation records is that no special equipment is required to perform the test, and consolidation coefficient may be reliably determined (Tch and Houbly, 1991) from \(t_0\), provided the ratio of shear modulus to undrained shear strength \((G/S_0)\) may be evaluated. The appropriateness of these evaluations has been demonstrated, and in reality only weak dependence on soil stiffness or strength (defined through the rigidity index, \(I = G/S_0\) (Tch and Houbly, 1991)) is apparent. (Robertson et al., 1997; Jones and Rust, 1993).

Evaluation of hydraulic conductivity from piezocene data is more tenuous since the dissipation process following undrained insertion is controlled (in theory) only by consolidation coefficient, \(c_v\). Correspondingly, correlations based on \(t_0\) (Schmertmann, 1978, for example) yield relatively inconclusive results (Robertson et al., 1992). Alternatives using the magnitude of initial pre-arrest pore pressure as an index to determine hydraulic conductivity have also been attempted (Elsworth, 1993), and correlate well with available data for undrained and drained piezocene insertion.

Application of all of the previously mentioned methods is limited in fine-grained soils where excessive time is required to reach full dissipation. Pore pressure dissipation to existing static magnitudes is necessary to define both the magnitude of the penetration induced pore pressure (used to evaluate hydraulic conductivity) and the progression of dissipation (used to evaluate consolidation coefficient) as indexed through \(t_0\).

An alternative to waiting for full dissipation is to use pressure dissipation rate, \((\frac{dp}{dt})\), as the index parameter. This form of data analysis may pro-
ceed without knowledge of the initial static pore fluid pressures, and therefore drastically reduce the duration of the dilation test. In the following, the theory is developed in support of this hypothesis, and data from a series of piezocene soundings (Aggarwal et al., 1994) constrained against pumping test results, are used to illustrate the applicability of the technique.

2 PORE PRESSURE RESPONSE

2.1 Theory

The pore pressures, $p$, developed in excess of static pore pressure, $p_s$, around a circular rod of radius, $r$, inserted at a velocity, $U$, into a porous medium may be defined (Elsworth, 1991) in terms of the non-dimensional parameters of excess pressure, $P_D$, advance rate, $U_D$, and time, $t_D$. These are defined as,

$$P_D = \frac{4(p - p_s)k}{UR}$$

$$U_D = \frac{U}{c_T}$$

$$[x_D, y_D, z_D] = \frac{1}{f} [x; y; z]$$

in terms of the material characteristics of permeability, $k$, fluid dynamic viscosity, $\mu$, and coefficient of consolidation, $c_T$ (with $c_{T0} = c_T$). The coordinate system is centered on the moving tip of the penetrometer (Elsworth, 1991, Figure 1) with the positive $x$-axis defining the penetrometer shaft, with advance at rate $U$ in the direction of the negative $x$-axis. This representation enables the pressure dissipation following penetration arrest to be determined (Elsworth, 1991, equation (17)) as,

$$P_D \mathcal{R}_D = \frac{R_s}{R_0} \frac{2}{\sqrt{\pi} \mu} \int_{R_0/R_s}^{R_s/R_s} e^{-\pi (r_0 + z_0)^2} d R_0$$

where $R_0 = z_0 + y_0 + x_0$ and non-dimensional time defines both the current time, $t_D$, and the time of penetrometer arrest, $t_D'$, as

$$t_D - t_D' = \frac{4c_T(t - t')}{r^2}.$$  (5)

The geometry relative to the arrested configuration is denoted by the overbar as

$$[\bar{x}_D, \bar{y}_D, \bar{z}_D] = [x_0 + \frac{1}{2} U_D (t_D - t_D'), y_0, z_0]$$  (6)

enabling the dissipation behavior to be determined relative to the arrested geometry of the penetrometer.

2.2 Type curves

The significant non-dimensional groupings of parameters that control the dissipation behavior may also be determined. From equation (4) it is apparent that the functional dependence will uniquely link pressure response as a single static location on the penetrometer shaft (fixed $x_D$, and $x_D \geq 0$) as,

$$P_D \mathcal{R}_D = \mathcal{F} \left[ \frac{t_D'}{x_D}, \frac{t_D - t_D'}{x_D}, \frac{U_D}{x_D}, \mathcal{P}_D \right].$$

Figure 1: Pore pressure dissipation response around an arrested distortion for the limiting behavior as $t_D'/x_D \gg (t_D - t_D')/x_D$.

The dissipation behavior asymptotes as the penetration induced pore pressure bulb approaches a constant form, representing a near-uniform initial condition for the dissipation behavior. This criterion is met for $t_D'/x_D \gg (t_D - t_D')/x_D$, where the resulting pressure dissipation results are as illustrated in Figure 1. Apparent from Figure 1 is that static pore fluid pressures must be known, a priori, to define both $t_D$ and the induced pore pressure magnitude, $p - p_s$, from the field-recorded data. To avoid this constraint, the rates of change of pore pressure may be evaluated directly from equation (4) and Figure 1, and plotted in Figure 2 for $U_D/x_D \leq 1$ and in Figure 3 for $U_D/x_D \geq 1$. 

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These figures may be used directly in data reduction, where the two plots make use of the change in asymptotic behavior as the parameter $U_D X_D$ transits unity (see Figure 1).

Figure 2: Variation in pore pressure dissipation rate with time. The pore pressure behavior of Equation 4, as represented in Figure 1, is transformed as $-d[P_d X_D]/d[(t - t_0)/2D]$ on the ordinate.

Figure 3: Variation in pore pressure dissipation rate with time. The pore pressure behavior of Equation 4, as represented in Figure 1 and in rate form in Figure 2, is shifted by a factor of $1/U_D X_D$ in pressure-rate (ordinate) and a factor of $U_D X_D$ in time (abscissa).

2.3 Data reduction method

Magnitudes of permeability, $k$, may be evaluated directly from the type curve analyses, and converted to hydraulic conductivity, $K$, as desired, through $k/\mu = K/\rho g$. The ability to determine parameters relates to the shape of the type curves, as apparent in the different forms of Figures 2 and 3. These parameters are, as follows.

$U_D X_D \leq 1$: Apparent from Figure 2 is that a single peak pressure rate develops with a steady sequence of pre-peak pressure rate magnitudes prior to the peak pressure rate. This common peak magnitude is defined as,

$$\frac{d[P_d X_D]}{d[t - t_0]/2D}_{\text{peak}} \approx \frac{1}{4}$$

and this may be rearranged to determine hydraulic conductivity, $K$, as

$$k = K = \frac{1}{\mu} \frac{U_C r^3}{\rho g} \frac{d[p - p_0]}{d[t - t^*]}_{\text{peak}}$$

where consolidation coefficient must also be determined. Noting that the peak dissipation rate occurs as

$$\frac{(t_d - t_0^*)}{2D} = \frac{4C_C (t - t^*)}{2D} \approx 1$$

enables equation (10) to be substituted into equation (9) and results in the final form of the conductivity relationship as

$$k = K = \frac{U_C r^3}{16(t - t^*)_{\text{peak}}} \frac{d[p - p_0]}{d[t - t^*]}_{\text{peak}}$$

$U_D X_D \geq 1$: From Figure 3 it is apparent that the curves collapse onto an approximately common form with the transformations used for the axes. Under this transformation, the relationship defining permeability, $k$, and hydraulic conductivity, $K$, may be developed on noting from Figure 3 that

$$\frac{d[P_d X_D]}{d[t - t_0]/2D}_{\text{peak}} \approx \frac{1}{2} U_D X_D$$

resulting in the relationship,

$$k = K = \frac{1}{\mu} \frac{U_C r^3}{\rho g} \frac{d[p - p_0]}{d[t - t^*]}_{\text{peak}}$$

This enables hydraulic conductivity magnitudes to be determined from peak pressure rate, $dp/dt$, without prior knowledge of consolidation coefficient, $c_C$. This is fortuitous, since, using early dissipation rate data, the magnitude of the coefficient of consolidation may not be determined.
3 ANALYSIS

The proposed data reduction method is applied to a series of five dissipation tests conducted at York and Navarre, Kansas (Aggarwal et al., 1994). The materials were penetrated using a 60° cone of \( \frac{3}{4} \) inch diameter and at a standard, pre-arrest penetration rate of \( U = 2 \) cm/s. From these dissipation data, the time derivatives of pressure change may be evaluated, and are plotted for the five tests in Figure 4. The figures are censored to remove excessive noise from the traces, following the evaluation of rate magnitudes from the pressure data. Apparent from the form of the pressure dissipation rate traces is the replication of the average dissipation slope of 7.4, identified in Figure 3. The maximum pressure dissipation rate magnitudes are removed from Figure 4 and used with \( \beta = 1 \) in equation (13) to evaluate the hydraulic conductivity, as recorded in Table 1. Also compared in the data evaluation are the hydraulic conductivity magnitudes evaluated from the peak pressure magnitude (Elsworth, 1993) and from pump test results. The general correspondence is encouraging.

<table>
<thead>
<tr>
<th>Site</th>
<th>Depth (ft)</th>
<th>Pump test</th>
<th>Peak pressure</th>
<th>Pressure rate (this study)</th>
</tr>
</thead>
<tbody>
<tr>
<td>York</td>
<td>74</td>
<td>( 6 \times 10^{-2} )</td>
<td>( 1.68 \times 10^{-2} )</td>
<td>1.11</td>
</tr>
<tr>
<td></td>
<td>73</td>
<td>( 6 \times 10^{-3} )</td>
<td>( 1.40 \times 10^{-3} )</td>
<td>1.43</td>
</tr>
<tr>
<td></td>
<td>83</td>
<td>( 3 \times 10^{-3} )</td>
<td>( 4.16 \times 10^{-3} )</td>
<td></td>
</tr>
<tr>
<td>Navarre</td>
<td>76</td>
<td>( 5 \times 10^{-4} )</td>
<td>( 4 \times 10^{-4} )</td>
<td>( 3.98 \times 10^{-3} )</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>( 5 \times 10^{-4} )</td>
<td>( 7.6 \times 10^{-3} )</td>
<td>( 1.2 \times 10^{-3} )</td>
</tr>
</tbody>
</table>

Table 1: Determination of hydraulic conductivity magnitudes, \( K \), for piezocene dissipation data using peak pressure (Elsworth, 1993) and pressure dissipation rate (this study) methods, and comparison with field pumping test results (Aggarwal et al., 1994).

Figure 4: Variation in pressure dissipation rate, \(-dp/dt\), with time for a series of tests at York (a-c) and Navarre (d), Kansas (Aggarwal et al., 1994).

4 CONCLUSIONS

Post arrest dissipation behavior is defined under the hypothesis that pressure dissipation rate may be used to determine permeability and hydraulic conductivity magnitudes. Results from post-arrest dissipation tests confirm the anticipated form of the dissipation rate curves and enables hydraulic conductivity magnitudes to be determined directly from the assembled data. The principal advantage of the method is that only early-time records require to be used, eliminating the need for extending the dissipation test to completion. In all examples reported in this work, the peak pressure rate is available within the first minute of the dissipation test, with \( \beta = \beta_0 \) at least an order of magnitude greater. This acceleration opens the possibility of completing more tests in a given period.

The theoretical behavior accounts for the important influence of partial drainage that may occur during the penetration process and the effect this exerts on the initial conditions for dissipation. Correspondingly, different behavior is recorded for slow (undrained, \( U < U_0 \)) penetration and fast (drained, \( U > U_0 \)) penetration. The appropriate relations for determining hydraulic conductivity are equations (11) and (13), respectively. Despite the advantage in rapid evaluation, and that the need to know reference (static) pore pressure magnitudes is eliminated, the use of pressure dissipation rate does not perform as well as the method of peak pressure magnitudes.

ACKNOWLEDGMENTS

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Hydraulic conductivity assessment at the Trino Vercellese power station site: Large and small scale tests

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ABSTRACT: To design the dewatering system for the planned 2 GW nuclear power station of Trino Vercellese evaluation of the hydraulic conductivity of the soil was performed on the basis of pumping tests carried out by means of 2 pumping wells and of a large number of inflow tests in piezometers.

1 INTRODUCTION

The design of the 2 GW nuclear power plant at Trino Vercellese in Italy required a detailed recognition and evaluation of drainage, in order to forecast the influence of large area dewatering on the surroundings. It is at present well recognised in geotechnical engineering that, because of the dominating influence of the macrofabric features of deposits, the most reliable way to assess the mass conductivity of coarse grained soil is properly program and conduct large scale pumping tests. Borehole outflow and inflow tests are also used in this soils, but they yield, at best, only values representative of the order of magnitude of the in situ conductivity. Within this context this paper reports mainly on the large scale pumping tests carried out at the mentioned site of Trino Vercellese, whereas results obtained from piezometer tests are only referred to, in order to support some assumptions introduced into analysis.

2 SOIL PROFILE

The investigated site is located in the upper part of Po river valley, close to the town of Trino Vercellese. The stratigraphic section in Figure 1 schematically shows the soil sequence at the site, mainly composed of gravel and sand with small percentage of silt up to 40-42 m depth, and silty sand up to the maximum investigated depth. As a result of this composition, in situ tests have been mainly limited to cross hole tests and Standard Penetration Test. Based on geological information, it can be argued that soil strata within the depth of 5 to 30 m below ground level has not been subjected to any appreciable erosion, even if values of the overconsolidation ratio of order of 1.5 could be expected due to ground water level oscillations.

3 PUMPING TESTS

Pumping tests consist in withdrawing the water, usually at constant rate, from one well and
observing the water level decline in one or more observation wells or piezometers. In the present case two pumping tests have been carried out in two wells (EP2566, EP2567) located as shown in Figure 2, with observational piezometric lines located at radial distance from the well equal to 25, 50 and 100 m. Each vertical has been equipped with two piezometers at depths of 20 and 30 m from ground level.

![Figure 2. Location of pumping wells and piezometers.](image)

The water level decline with time is indicative of the hydraulic properties of the aquifer. As an example, data reported in Figures 3 and 4 are indicative of a confined flow pattern, even if for drawdown values higher than 10 meters, the flow pattern seems to indicate deviation from a purely confined condition.

Figure 3. Pumping wells discharge characteristics and discharge curves.

Even if response of the aquifer during pumping tests was similar to that of a confined aquifer, a long period, such as three days, was planned as pumping duration, with pumping rates equal to 17 x 10³ m³/s, 20 x 10³ m³/s (well EP2567), 45 x 10³ m³/s, 72 x 10³ m³/s (well EP 2966).

4. INTERPRETATION OF PUMPING TESTS DATA

A pumping test is a transient flow problem, the solution of which requires to consider the conservation of water mass, the state relation of the pore water, the dynamic equilibrium of the pore water, the interaction between the pore water and the soil skeleton and equilibrium of the porous body.

By defining the specific storage S, the volume of water that a unit volume of saturated aquifer release under a unit change of hydraulic head, the general equation of transient flow through a saturated anisotropic medium is given by:

\[ \frac{\partial k}{\partial x} + \frac{\partial k_y}{\partial y} + \frac{\partial k_z}{\partial z} + \frac{\partial h}{\partial t} = S \frac{\partial h}{\partial t} \]

(1)

being h the hydraulic head.

When interpreting the experimental data of
pumping tests in a confined aquifer, the following hypotheses are introduced:
- the hydraulic head is uniform throughout the aquifer, before pumping;
- the storage within the well can be neglected if compared to the volume of aquifer;
- the well fully penetrates the aquifer;
- the pumping rate is constant with time.
If the aquifer thickness is equal to \( b \) the transmissivity \( T \) is defined by:

\[
T = k b
\]

and storativity is given by:

\[
S = S_0 b
\]

With these two definitions and the above mentioned hypotheses, the general equation reduces to:

\[
\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = \frac{S}{T} \frac{\partial h}{\partial t} + \frac{S}{T} \frac{\partial^2 h}{\partial t^2}
\]

or, if converting into cylindrical coordinates:

\[
\frac{\partial^2 h}{\partial r^2} + \frac{1}{r} \frac{\partial h}{\partial r} = \frac{S}{T} \frac{\partial h}{\partial t} + \frac{S}{T} \frac{\partial^2 h}{\partial t^2}
\]

It is well known in literature that Theis (1935) first obtained a solution for equation (3). As an alternative approach Cooper and Jacob (1946), suggested that when \( T \) is large or \( r \) is small, the solution of equation (3) can be expressed in the format:

\[
S = \frac{23Q}{4\pi T} \left( \frac{225T}{s^2} \right)
\]

being:
- \( Q \) = well discharge
- \( s \) = drawdown
so that by plotting the drawdown against the time on a semilogarithmic scale (Figure 4), a straight line is obtained, and the change as for tenfold increase time and the intersection \( t_0 \) with horizontal axis \( s = 0 \) are determined. Then the transmissivity is given by:

\[
T = \frac{23Q}{4\pi \Delta s}
\]

or, if converting into cylindrical coordinates:

\[
S = \frac{225Q}{r^2}
\]

Figure 4. Drawdown versus time/well distance.
is also of interest to note that, when the Cooper and Jacob (1946) approximation holds true, i.e. when:

\[
\frac{ny}{s} > 2.5
\]

(7)

if equation (7) is satisfied within a distance \( r_4 \) for a given \( t \), then considering two piezometers located at distances \( r_1 \) and \( r_2 \) equation (7) can be rewritten:

\[
\phi(t) = \frac{Q}{4\pi T} \ln \left( \frac{r_2}{r_1} \right) - \frac{Q}{2\pi T} \ln \left( \frac{r_3}{r_4} \right)
\]

(8)

This equation shows, as suggested by Neuman (1988), that the relative drawdowns change logarithmically with the radius in a time-independent fashion. This means that the drawdown cone around the well is in a pseudosteady state condition, and equation (8) is analogous to the well known Thiem (1939) equation for stationary condition.

5 REMARKS

Results obtained at the site under consideration are summarized in table 1.

<table>
<thead>
<tr>
<th>CONFINED FLOW</th>
<th>WELL</th>
<th>WELL DISCHARGE</th>
<th>( K_b )</th>
</tr>
</thead>
<tbody>
<tr>
<td>FULLY PEERTING WALL</td>
<td>EP2566</td>
<td>45</td>
<td>7 + 0</td>
</tr>
<tr>
<td></td>
<td>EP2567</td>
<td>72</td>
<td>7 + 0</td>
</tr>
<tr>
<td>UNSTEADY STATE</td>
<td>EP2566</td>
<td>17</td>
<td>3 + 0</td>
</tr>
<tr>
<td></td>
<td>EP2567</td>
<td>20</td>
<td>3 + 10</td>
</tr>
<tr>
<td>PSEUDOSTEADY STATE</td>
<td>EP2566</td>
<td>45</td>
<td>4 + 7</td>
</tr>
<tr>
<td></td>
<td>EP2567</td>
<td>72</td>
<td>5 + 8</td>
</tr>
<tr>
<td></td>
<td>EP2567</td>
<td>47</td>
<td>2 + 11</td>
</tr>
<tr>
<td></td>
<td>EP2567</td>
<td>20</td>
<td>2 + 13</td>
</tr>
</tbody>
</table>

They refer to the above mentioned assumptions of confined flow and fully penetrating well. The first assumption, unsteady state, based on the experimental aquifer response, is also

Figure 5. Vs, G versus depth.

Figure 6. Results in the piezometer tests.
supported by the obtained storativity values, being $S = 7\times10^{-4}$. This value is in agreement with the theoretical ones, deduced by assuming for the present case a thickness $b = 44$ m for the aquifer, water compressibility $\beta = 4.32 \times 10^{-4}$ (MPa)$^{-1}$, a soil porosity $n = 0.3$ and by deducing soil compressibility on the basis of the cross hole test reported in Figure 5. By combining a shear modulus $G = 750$ MPa with Poisson ratio $\nu = 0.18$ one gets a soil compressibility $\alpha = 1 \times 10^{-3}$ (MPa)$^{-1}$, with theoretical storativity of $S = 9 \times 10^{-4}$.

The second assumption, pseudosteady state, has been in some way corroborated by the large number of inflow constant-head tests and inflow falling-head permeability tests, carried out by means of piezometers at various depths in order to get a rough idea of changes of conductivity in relative meaning. Results referred in Figure 6 they all shown that vertical conductivity can be expected to be at least one order of magnitude lesser than horizontal one.

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A new CPT method for estimating soil hydraulic properties

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ABSTRACT: Complete characterization of contaminated sites should include determination of soil hydraulic properties that describe behavior under unsaturated conditions. We present a new cone penetrometer test (CPT) method for estimating soil-moisture characteristic and hydraulic conductivity curves from in situ measurements. A modified cone penetrometer is used to inject water from a screen into the soil. The cumulative flow and pressure heads at two locations are recorded in time. The soil hydraulic properties are obtained via numerical inversion of Richards’ equation. Here we discuss field-scale tests in a laboratory aquifer that demonstrate the device and solution technique. Results show that the saturated hydraulic conductivity is well estimated and the soil moisture characteristic curve lies between the wetting and drying curves obtained from other standard laboratory methods.

1 INTRODUCTION

Definition of the hydraulic properties of unsaturated soils is increasingly necessary for geotechnical applications. Knowledge of the soil-moisture characteristic and hydraulic conductivity curves, $\theta(h)$ and $K(h)$, is particularly important for accurate numerical modeling of variably saturated flow and contaminant transport processes. While these soil properties can be determined in the laboratory, in situ methods are often preferred.

Direct measurement of point data on the soil-moisture characteristic curve, $\theta(h)$, and/or the hydraulic conductivity curve, $K(h)$, may be obtained using instantaneous profile, crust, or infiltrimeter methods, among others (Klute & Dirksen 1986; Benson & Gribb 1997). The Guelph permeameter and double ring infiltrometer are commonly used to obtain saturated hydraulic conductivity values (Reynolds 1993; Bouwer & Jackson 1974).

Parameter optimization is an indirect approach that makes it possible to obtain $K(h)$ and $\theta(h)$ simultaneously from transient flow data (Kool et al. 1987). A flow event is modeled with an appropriate governing equation and analytical models of $K(h)$ and $\theta(h)$. The unknown parameters of $K(h)$ and $\theta(h)$ are obtained by minimization of an objective function describing the differences between some measured flow variables and those simulated with a model. This methodology was originally applied to one-step and multi-step column outflow data in the laboratory (see for example, Kool et al. 1985; Parker et al. 1985; van Dam et al. 1992, 1994; and Ecting & Hopmans 1993). In the field, this method has been used with ponded infiltration (Bohne et al. 1992), tension disc infiltrometer (Šimůnek & van Genuchten 1996, 1997, and Šimůnek et al. 1997), and multi-step soil water extraction data (Izuno et al. 1997) obtained in the near surface.

Gribb (1993, 1996) proposed a new cone penetrometer tool (e.g., cone permeameter) and use of parameter optimization to estimate soil hydraulic properties at depth. The prototype was further developed by Leonard (1997) as shown in Figs. 1 and 2. The cone permeameter is pushed into the soil to the test depth, and a constant head is then applied to the 5-cm long screen. Cumulative inflow volume is determined from scale readings of the mats of water removed from a bottle. Pore water pressure increases are measured with tensiometer rings 5 and 9 cm above the screened section. An inverse solution method is used to predict the soil hydraulic
properties. Here we present observed data and numerical analysis of tests performed by Leonard (1997) in a laboratory aquifer system under unsaturated conditions. Results are compared with independent measurements of the soil hydraulic properties to benchmark performance.

2 THEORY

HYDRUS-2D (Šimůnek et al. 1996) is used to simulate the cone permeameter test in unsaturated soil. The governing flow equation for radially symmetric, isothermal Darcian flow in an isotropic, rigid porous medium, assuming that the air phase plays an insignificant role in the liquid flow process, is (Richards 1931):

\[
\frac{1}{r} \frac{\partial}{\partial r} \left( r K \frac{\partial h}{\partial r} \right) + \frac{\partial}{\partial z} \left[ K \left( \frac{\partial h}{\partial z} + i \right) \right] = \frac{\partial \Theta}{\partial t}
\]

where \( r \) = radial coordinate, \( z \) = vertical coordinate positive upward, \( t \) = time, \( h \) = pore water pressure head, and \( K \) and \( \Theta \) = hydraulic conductivity and volumetric moisture content, respectively. The initial pressure head distribution in the domain is determined from initial tensiometer readings. A constant head boundary condition is applied to represent the source. Exterior boundaries are located far enough away from the source as not to influence the solution, and are defined as no-flow boundaries.

The van Genuchten (1980) expressions for the moisture content and hydraulic conductivity \( \Theta(h) \) and \( K(\Theta) \) are used in this work:
\[ \theta_i = \frac{\delta (h_i) \cdot \theta_i}{\theta_i - \theta_o} = \frac{j}{(1 + a \cdot h_i^{m})} \quad h < 0 \]

\[ \theta_i = 1, \quad h \geq 0 \]  

(2)

and

\[ K(\theta) = K_i \cdot \theta_i \left[ \frac{1 - \phi_i}{1 + \phi_i} \right]^n \quad h < 0 \]

\[ K(\theta) = K_i, \quad h \geq 0 \]  

(3)

where \( \theta_i \) = effective moisture content, \( K_i \) = saturated hydraulic conductivity, \( \theta_o \) and \( \theta_e \) = residual and saturated moisture contents, respectively, \( a \) and \( m \) \((= 1 - \frac{1}{n})\) = empirical parameters. The hydraulic characteristics defined by (2) and (3) contain five unknown parameters: \( K_0, \alpha, \beta, n, \) and \( \theta_e \).

To derive estimates of the hydraulic parameters using parameter optimization, an objective function, \( \Phi \), expressing the differences between flow responses measured with the prototype and those predicted by a numerical model with hydraulic parameter inputs, is minimized:

\[ \Phi(\theta, q_i) = \sum_{i=1}^{n} \left[ \sum_{j=1}^{m} \left( q_j (t_i) - q_j (t_i, \theta, \beta) \right)^2 \right] \]

(4)

where \( k \) = number of different sets of measurements, such as cumulative inflow volume, or the pressure head measurements; \( q_i \) = number of measurements in a particular set; \( q_j (t_i) \) = specific measurements at time \( t_i \) for the \( j \)th measurement set; \( \beta \) = vector of optimized parameters (e.g., \( K_0, \alpha, \beta, \theta_e \) and \( n \) in this work); \( q_j (t_i, \theta, \beta) \) = corresponding model predictions for the parameter vector, and \( v_j \) = weights associated with a particular measurement set. Minimization of the objective function \( \Phi \) is accomplished using the optimization routine developed by Simunek & van Genuchten (1997).

4 RESULTS AND DISCUSSION

Results of the optimization processes and

![Graph showing the results of the optimization processes](image)

Fig. 3. Measured cumulative inflow (I), and pressure head data (LT = lower tensiometer, UT = upper tensiometer) and simulated flow responses (IS1 = Inverse Solution 1, IS2 = Inverse Solution 2) for Test A.

![Graph showing the results of the optimization processes](image)

Fig. 4. Measured cumulative inflow (I), and pressure head data (LT = lower tensiometer, UT = upper tensiometer) and simulated flow responses (IS1 = Inverse Solution 1, IS2 = Inverse Solution 2) for Test B.

3 METHODS

Prototype tests were conducted in a laboratory aquifer measuring 4.7 m x 4.7 m x 2.6 m. The aquifer material is a sandy soil with occasional kaolin pockets, underlain by 20 cm of gravel. A description of the aquifer and properties of the soil were previously presented by Gribb et al. (1997). A cone penetrometer was continuously pushed into the soil to a depth of 70 cm by a drill rig. Two representative tests were selected for presentation here. For a first test (Test A) a water pressure head of 30 cm was supplied to the center of the screen. The following day, a second test (Test B) was performed with a water pressure head of 50 cm. Flow data were electronically collected every 5 sec for 400 sec (Leonard, 1997).

Two inverse solutions for each test were performed. In the first case, Inverse Solution 1 yielded estimates of the unknown parameters, \( \alpha \), \( \beta \), \( n \), and \( \theta_e \), for \( \theta_e = 0.008 \) cm²/cm. In the second case, Inverse Solution 2 yielded estimates of \( \alpha \), \( n \) and \( \theta_e \), for \( \theta_e = 0.35 \) cm²/cm and \( \theta_e = 0.008 \) cm²/cm.
representative hydraulic property data obtained using standard techniques are shown in Figs. 3-5 and in Table 1. Measured and simulated cumulative flow and pressure head data in time from the two inverse solutions are plotted in Figs. 3-4. The estimated retention curves for solutions of the two cone permeameter tests, along with those independently determined with capillary rise and pressure plate tests (Singleton, 1997) are presented in Fig. 5. Table 1 shows the hydraulic parameters $\alpha$, n, $\theta_0$, and $K_c$ estimated via inverse modeling, hydraulic parameters $\alpha$, n and $\theta_0$, of the retention curves obtained with the standard laboratory methods and the mean values of $K_c$ determined from slug tests (Scaturo, 1993; Guelph permeameter tests (Scaturo, 1993) and laboratory constant head tests (Singleton, 1997).

The estimates of parameters $\alpha$, n, $\theta_0$, and $K_c$ obtained solely from the cone permeameter flow responses (Inverse Solution 1) provided a better fit of measured data for both numerical solutions. The resulting retention curves for both tests followed the same shape, but the estimated saturated moisture content $\theta_0$ was much smaller than that obtained from other test methods.

Nonuniqueness of $\theta_0$ values was anticipated by the results of earlier numerical experiments with error-free synthetic data. Gribb (1986) showed that an objective function similar to (4) was least sensitive to $\theta_0$ and n, and more sensitive to $K_c$ and $\alpha$. Since $\theta_0$ was not reliably estimated for this test, Inverse Solution 2 was performed to investigate the influence of $\theta_0$. In this case, $\theta_0$ was set equal to 0.35 cm$^3$/cm$^3$ and $K_c$, n, and $\alpha$ were optimized. The simulated cumulative flow data fitted the measured data very well, and the modeled pressure heads at the lower tensiometer position tracked closely to observed data. On the other hand, the simulated pressure heads at the upper tensiometer showed earlier, and more gradual, progressions of the wetting front when compared to the measured data. Comparison of soil hydraulic properties resulting from Inverse Solutions 1 and 2 shows that the optimized parameter $K_c$ increased with the larger fixed value of $\theta_0$, for both tests. It is interesting to note that higher values of $K_c$ were obtained for Test B. This test was performed with a higher applied pressure head, which resulted in larger pressure head increases in the soil than in the case of Test A. However, the values of $K_c$ obtained were very close for both tests and solutions. Values of $\alpha$ and n decreased and the characteristic curves had more gradual slopes with the higher, fixed value of $\theta_0$. The shapes of the curves obtained from both tests were similar for each solution.

Disturbance of the soil surrounding the cone occurs during placement. However, it seems that soil structure changes due to placement were not significant for this soil. This was evident from the results of the numerical inversions, which were within the range of hydraulic properties obtained with other standard techniques. Saturated hydraulic conductivity values were similar to those obtained with Guelph permeameter (Scaturo, 1993) and laboratory constant head tests (Singleton, 1997). The soil-moisture characteristic curves were most similar to the wetting retention curve determined with the capillary rise test. The curves for fixed $\theta_0 = 0.35$ were within the two limiting branches of the characteristic curves obtained with the capillary rise (wetting) and pressure plate (drying) tests. The
shapes of the curves close to the limiting or scanning wetting branches of the soil-moisture characteristic curves were expected, because of the wetting character of the cone permeameter test.

5 CONCLUSIONS

We present here a new cone tool for simultaneous determination of the soil-moisture characteristic and hydraulic conductivity curves in unsaturated soil. Our results in a laboratory aquifer composed of sandy loam showed that the saturated hydraulic conductivity was well estimated. In addition, the soil-moisture characteristic curves obtained were between the wetting and drying curves obtained from other standard laboratory methods. It is well known that the soil fabric is disturbed due to cone penetration; however, the results of these few tests in sandy soil show little effect of disturbance on the value of Ks returned from the optimization procedure. It is likely that disturbance effects will be more significant in other soil types. This will be studied further in the field.

ACKNOWLEDGMENTS

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REFERENCES


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Pressuremeter testing in onshore ground investigations: A report
by the ISSMGE Committee TC16

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ABSTRACT: This report describes the use of pressuremeters in onshore ground investigation. It describes
the types of pressuremeters, the methods of calibration, installation techniques, test procedures, analysis
of expanding cavities, interpretation of tests and application of results in design. It is a statement on the general
practice of pressuremeter testing and is not intended to be a standard.

FOREWORD

This report on pressuremeter testing is issued under the auspices of the ISSMGE Technical Committee
TC16 (Ground Property Characterisation from In Situ Testing). The report was authored by Barry
Clarke (UK) and Michel Gamba (France), with additional input from other members of the
committee and practitioners of pressuremeter testing.

The background to the report is as follows. The
current TC16 was re-constituted under the
chairmanship of Peter Robertson (Canada) by the
ISSMGE in 1994. This committee superseded both
the previous Technical Committees TC16 (Penetration Testing), and TC27 (Pressuremeter and Dilatometer Testing). The terms of reference of the
new committee included the following:

- to promote co-operation and exchange of
  information about the execution and
  interpretation of in situ testing for site
  and ground property characterisation; and

- to establish two working parties, the second
  on the self-boring devices and those installed in
  pre-drilled holes (e.g. self-boring
  pressuremeter, Menard pressuremeter, etc.).

At the first meeting of the new committee, held in
London on March 30, 1995, a working party on
pressuremeter testing was established, with Martin
Fahy (Australia) being appointed chairman. The
working party was charged with the task of
preparing the report on pressuremeter testing. The
aim was to have the final report available for
presentation at the 14th International Conference in
Hamburg in 1997.

A European Committee (EC4) had presented a
report on pressuremeter testing in Europe at the 10th
European Conference on SMFE in Florence (Amar et
al, 1991). The previous ISSMGE TC27 had grown
cut of the European EC4, with Michel Gamba
(France) as chairman. This committee had been
working on expanding the EC4 report to include
non-European input when the committee was
disbanded.

A decision was therefore made to make this
previous work the basis of the report to be prepared
by the working party on pressuremeters. Following
the first meeting of TC16, Barry Clarke (UK) and
Michel Gamba (France) finalised the first draft of the
report that they had previously been working on
under the previous TC27. This draft was presented
at the second meeting of the current TC16, which
was held in Sherbrooke, Canada on May 17, 1995
(during the 4th International Symposium on
Pressuremeters). Members of the committee and
practitioners were invited to review the draft and
provide comments. This draft of the report has been
edited by the original authors, taking these comments
into account.

AIMS OF THE REPORT

The main aim of the report is to present a general
"state of good practice" for pressuremeter testing for
onshore applications, dealing with prebored, self-
boring and full displacement pressuremeters. The
report is intended to promote good practice in
pressuremeter testing, to allow the full potential of
the pressuremeter to be utilised where appropriate. The content of the report is heavily influenced by the experience of the two main authors, and is therefore mainly a reflection of European (particularly UK and French) experience and practice. In preparing the report for TC27 views were sought from all member societies of the ISSMGE. It was concluded that pressuremeter testing was either carried out by companies following national guidelines and specifications or individuals who had developed their own expertise. For example, the majority of pressuremeter testing in Francophone countries, and Japan has been based on direct design rules which are now described in national and international standards. In the UK, contractors operating pressuremeters tend to follow similar specifications and interpret tests using a limited number of accepted methods. Elsewhere much of the pressuremeter testing and the associated interpretation is undertaken using in house specifications.

This report is not intended to be a Standard. Nor is it envisaged that a single closely-defined Reference Test Procedure will evolve from this report, at least not in the near future. There are several national standards for the prebored pressuremeter such as ASTM D4719-87 (USA), AFNOR P94-110 (France) and GOST 20276-85 (Russia). Further standards covering the self-boring pressuremeter and the rock dilatometer are being prepared by ASTM.

The Working Party is strongly of the view that further research and development in pressuremeter technology, and in interpretation and application of test results, should be encouraged. There are two approaches used in interpreting and using pressuremeter results: one consists of developing a set of empirical design rules based on measurements made in a very standard way with a standard Ménard pressuremeter, while the other approach is to use analytical methods to derive "fundamental" soil properties (strength, stiffness etc.) from the test, for use in understanding the soil behaviour and in analysing soil/structure interaction. While some individual members of the working party strongly support one approach or the other, the authors of the report, and the Working Party, have taken the view that both approaches have application under different circumstances. This diversity of opinion is regarded as healthy.

1 INTRODUCTION

The modern pressuremeter was developed in France and Japan in the 1950s with commercial investigations starting in France in 1956. The first published investigations were by Ménard (1957) who undertook studies for structures in North America. Since the 1950s there have been significant developments in equipment, installation, interpretation and application such that pressuremeter testing is now an accepted part of routine ground investigations.

There are several types of pressuremeter, a number of test procedures, international and national standards and many methods of interpretation of test data. In practice, tests are interpreted using either simple models to give ground properties or semi-empirical rules to give design parameters. The most widely known and used standards are those for the Ménard type pressuremeter (AFNOR, 1991, ASTM, 1987, 1994).


This report, prepared for the ISSMGE Committee TC16, is a brief introduction to the current state of international practice of pressuremeter testing in onshore ground investigations. The report describes equipment, site operations, interpretation and application.

2 TYPE OF PRESSUREMETER

The pressuremeter probe is usually defined as a cylindrical device that can apply a uniform pressure to the pocket wall, the pocket being created specially for the pressuremeter test. The word pocket is used rather than borehole to distinguish between the pocket created specially for pressuremeter testing and the borehole created for advancing between test positions. A borehole diameter is either equal to or greater than the pocket diameter. Furthermore, the methods used to create the pocket often differ from those used to create the borehole. The separate identification of pocket and
borehole has contractual implications in the UK and elsewhere where pressuremeter testing is paid for on time-based rates.

The probe is connected to the surface by rods which are used to lower, drill, push or hammer the probe into place. An umbilical cable connects the probe to the testing equipment at the surface which includes a pressure supply, control unit and data logger. Tests can either be stress and strain controlled tests. In stress controlled tests, the applied pressure is increased and the displacement of the membrane is monitored to give the response of the ground to a change in pressure. Strain controlled tests made using gas filled pressuremeters are actually stress controlled but the pressure increments are small and the rate of increase in pressure is controlled by the rate of expansion of the membrane. In true strain controlled tests using liquid filled pressuremeters volumetric increments of liquid are injected into the probe and the pressure required monitored.

Pressuremeter probes are grouped according to the method of installation (prebored, self-bored or pushed in) and method of measuring displacement (volume or radius). There are instances, however, when one type of pressuremeter may be assigned to one group based on the most common method of installation yet be installed, in certain situations, in a different way. Table 1 gives details of commercially available pressuremeters indicating the group to which they belong, their displacement measuring system and the ground in which they can operate. Installing a pressuremeter can change the properties of the ground adjacent to the probe. The measured ground response can be affected by the measuring system used. Thus, the information from a pressuremeter test will depend on the type of pressuremeter specified as well as the ground conditions in which it is used.

Probes are very similar in appearance but can differ significantly in detail. The inflated diameter of most probes is between 40 and 80 mm depending on the probe and the length to diameter ratio of the expanding section of all probes usually exceeds five. The total length of a probe varies between 0.5m and 2m depending on the type of probe. The dimensions of a probe govern the diameter of the borehole and the diameter and length of the test pocket.

The expanding membrane, usually made of rubber or adiprene, is clamped onto the body of the probe. In some instances it is protected by steel strips or reinforced to prevent damage during installation and testing. The membrane is inflated by a fluid which can be oil or water or gas. The fluid is usually pressurised at the surface though some probes have electrically operated down hole pistons. Probes are usually designed for testing either soil or rock. Probes designed for testing rock have a maximum pressure capacity of 20 MPa; probes designed for testing soil usually have a pressure capacity of less than 5 MPa. In some cases the same probe can be used in both soils and rocks provided a pressure transducer of the correct sensitivity is fitted.

The deformation of the membrane is monitored using either displacement transducers (radial displacement type) or volume change gauges (volume displacement type). Among the volume displacement types in commercial use are the Ménard type pressuremeters, of which there are several in production, the TETAM, and the Lateral Load Tester all of which are prebored pressuremeters, the PAF-76, a self-bored pressuremeter, and the Apagao minipressionmeter, a full displacement pressuremeter. The radial displacement types in commercial use include the prebored pressuremeters, the Elastimeter and High Pressure Dilatometer, the Cambridge type self bored pressuremeters and the full displacement push in pressuremeters or cone pressuremeters.

The development of pressuremeters has coincided with the development of electronics resulting in improvements in measuring and interpreting pressuremeter tests. Data loggers are now routinely used to monitor pressuremeter tests. This enables an operator to process the data on site and rapidly forward the information to a client.

Table 1 only lists those pressuremeters readily available in commercial practice. Other

Figure 1 Types of prebored pressuremeters: (a) a tricell probe; (b) a monocell probe (after Clarke, 1995)
pressuremeters not listed in Table 1 include research pressuremeters and pressuremeters for specific uses. These other pressuremeters, however, operate on similar principles to those described here.

2.1 Prebored Pressuremeters (PBP)

These pressuremeters are lowered into a pocket that is bored specially for the test. They can be used in any ground conditions provided a stable pocket can be bored. The Ménaud pressuremeter (MPM) (Ménaud, 1957) is a volume displacement pressuremeter that contains three expanding sections (Figure 1a). The expansion of the central test section is monitored; the guard cells are inflated to ensure that the length of the test section remains constant and, ideally, it will then expand as a cylinder. There are two types of this probe, one for testing soil (GC) and the other for rock (GB). The TExAM pressuremeter (Capelle, 1983) is a single cell volume displacement pressuremeter (Figure 1b) in which the fluid-filled membrane is expanded by a screw driven piston.

The Elastimeter (Suyama et al., 1966), the High Pressure Dilatometer (HPD) (Hughes and Ervin, 1980) and the GA20 (Jewell and Fubey, 1984) are radial displacement type probes used for testing rock. The movement of the rubber membrane of the Elastimeter is monitored with two spring loaded feeler arms. Three pairs of spring loaded plates are used in the HPD to monitor the displacement of the adiabene membrane which is protected by steel strips known as a Chinese lantern. Two pairs of spring loaded potentiometers are used in the GA20 to monitor the displacement of the reinforced rubber membrane. These probes are monometer probes (Figure 1b).

Stress or strain controlled tests can be carried out with PBP's in soils and rocks. The classic S shaped test curve obtained from a PBP test is a consequence of testing in a prepared pocket (see Figure 6). The initial part of the curve represents the pressure required to inflate the membrane and overcome the resistance of any drilling mud in the pocket. The initial movement of the ground is a function of the ground properties which may have been changed due to preboring and stress relief. The latter part of the loading curve represents the response of the least disturbed ground.

2.2 Self-bored Pressuremeters (SBP)

These pressuremeters, first developed in the 70's and shown in Figure 2, are drilled into the ground thus creating their own pocket. These probes can be used in any soil provided the gravel content is small, and in some weak rocks. Theoretical tests would seem to be no radial movement of the ground during installation, therefore the curve obtained from a test should represent the true ground response. In practice, friction between the probe and the ground, especially in granular materials, and localised effects of drilling can create significant strains within the ground adjacent to the pressuremeter which can result in a change in ground properties. The probes and drilling techniques are designed, however, to minimise these strains and hence changes to the ground properties. In clays, these changes are usually small and only affect the theoretical interpretation of the initial portion of a test; in sands the disturbance during installation is generally greater.

The two Cambridge Self-boring Pressuremeters (CSBP) (Wroth and Hughes, 1973) and Weak RockSelf-boring Pressuremeters (RSBP) (Clarke and Allan, 1989) are both radial displacement type monometer probes. The PAP-76 (Hedéquet et al., 1968, and Baguelin et al., 1978) is a volume displacement type monometer probe. These three probes are drilled into place with rotary rigs.

The expansion of the adiabene membrane of the CSBP is monitored by three spring loaded feeler arms at the centre of the expanding section with the arms being at 120° spacing. Recent models of the
probe include arms at 60° spacing, arms at different levels within the probe and proximity transducers. The membrane can be protected by a Chinese Lantern when operating in sands, stiff clays and clays containing some gravel. The CSBP can also include a pore pressure transducer mounted on the membrane so that pore pressure changes can be measured during drilling and testing.

Three spring loaded plates are used to measure the movement of the reinforced rubber membrane of an RSBP inflated by oil under pressure. The movement of the membrane of the PAF-76 is determined by measuring the volume of oil pumped under pressure into the probe. This allows true strain controlled tests to be carried out with the PAF-76.

2.3 Full Displacement (Pushed-in) Pressurimeters (FDP)

These pressurimeters were first developed in the late 1970s (HéQuéquel et al, 1982 and Withers et al, 1986) in order to increase the speed of installation and create repeatable disturbance. There are two types of push in pressurimeter, a thin walled tube which partially displaces the soil and a solid cone pressurimeter which displaces all the soil radially.

These devices are now predominantly cone pressurimeters with the probes being mounted behind electrical cones. They are either pushed or hammered into the ground disturbing the soil during the installation process. The curve obtained from a test represents the response of the ground disturbed during installation.

The full displacement cone pressurimeter (FDP) (Briaud and Shields, 1980, Hughes and Robertson, 1985; Withers et al, 1986), Figure 3, operated from a cone penetration test (CPT) truck, is a radial displacement monocell probe mounted behind a 15cm² static cone. It can only be used in soils in which it is possible to push a static cone. The membrane is inflated by air or oil under pressure and the movement recorded using three displacement transducers. The membrane is protected by a Chinese Lantern. A friction reducer, mounted between the probe and the cone, can be used to reduce the shear between the soil and the probe. At the start of a test the membrane may not be in contact with the ground because of the friction reducer; the ground response curve will then be an S-shaped curve.

The Apago mini-pressuremeter is a volume displacement monocell probe mounted behind a 10cm² conical tip. It is hammered or pushed into the soil. The tip diameter is greater than the probe diameter, therefore an S shaped expansion curve is obtained.

3 CALibrATIONS

Data from a pressurimeter test need to be converted to applied pressure and strain at the pocket walls and corrected for system or membrane compliance, details of which are given in Table 2. The three most common groups of calibrations are:

i) pressure and displacement transducers,
ii) membrane stiffness,
iii) and membrane compression or system compliance.

Other calibrations such as effects of temperature and changes in membrane properties with use are sometimes necessary especially when testing soft clays or when trying to measure absolute rather than relative pressures.

Displacement transducers are calibrated by comparing the output with measured displacement. Membrane stiffness is measured by inflating the probe in air. All other calibrations are obtained by pressurising the probe in a rigid cylinder. An instrument register giving the history of all
calibrations for a particular probe is usually maintained by the operator. All radial displacement type probes contain transducers. The voltage output from a transducer is converted to pressure or strain by a calibration factor determined at the beginning and end of a contract and whenever there is a major repair of the equipment.

The pressure exerted on the ground by the membrane will not equal the recorded pressure because of membrane thickness, that is, the pressure required to inflate the membrane in air. A membrane thickness calibration should be carried out whenever a membrane is replaced. This can occur several times during a contract since membranes can be damaged during installation and testing. Membrane thickness calibrations are only significant for tests in soft clays or loose sands or tests near the ground surface but, in practice, are routinely applied.

The thickness of a membrane changes as the internal pressure increases and/or the membrane expands. Therefore the measured movement of the inner surface of a membrane is not equal to that of the membrane/ground interface. Membrane compression, the change in thickness of a membrane due to pressure, is small and only significant for tests in hard soil and rock. It is unnecessary to carry out this calibration when testing soil. It is only necessary to determine the membrane compression correction for radial displacement type probes since the system compliance correction for volume displacement type probes includes the membrane compression correction. A further membrane compression correction (thinning due to the stretching of the membrane as it expands) is applied to take into account the change in thickness during expansion. This small correction is only significant when testing rocks with radial displacement type probes.

System compliance represents the volume changes that occur in the supply lines, testing equipment and probe as the pressure is increased. This correction applies to all volume displacement type probes and includes the correction for the membrane compression.

System compliance in radial displacement type pressuremeters is a function of the displacement transducers and membrane. In practice, it is usually ignored when testing soft clays and loose sands. It is necessary, however, to correct for system compliance when testing stiff soils and rocks and to indicate the magnitude of that correction. Furthermore, if the correction is too large then an upper limit to the modulus may be given. For example, in the UK corrected stiffnesses exceeding 3MPa are unlikely to be quoted.
4 INSTALLATION

Installation has a significant effect on the shape of a test curve and, therefore, on the interpreted parameters. Ideally, the installation technique should be consistent and designed to minimise disturbance to the surrounding ground or produce repeatable disturbance. The ground types in which pressuremeters can be used are given in Table 1.

Pressuremeters are usually installed in boreholes drilled by rotary rigs. The exception to this is when an SBP or FDP can be installed directly from the surface in suitable ground conditions; the former with a purpose-built rotary rig or a conventional rotary rig with an adapter for SBP drilling, the latter with a CPT truck. It is usual to specify tests at particular horizons to obtain a profile of ground properties.

4.1 Prebored Pressuremeters (PBP)

It is usual to specify the pocket length and diameter when using a PDP and, possibly, the method of creating the pocket especially if soil samples are required. The pocket can be created using a variety of techniques but the best technique is that which removes all material and minimises disturbance to the pocket wall. The borehole is drilled and the test pocket created from the base of the borehole using one of the preferred methods listed in Table 1. A typical sequence of operations is shown in Figure 4.

Menard pressuremeter tests are usually performed at every metre. In some instances, in particular in sands and gravels below the water table, the Menard pressuremeters probe can be driven into the ground though it is protected by slotted casing during driving.

4.2 Self-bored Pressuremeters (SBP)

The sequence of operations for SBPs and FDPs can be similar since both types of pressuremeter create their own pocket. An SBP is either drilled into place using an internal bit driven by either rotating inner rods (Figure 5) or a downhole motor, or is jetted into place (Benoit et al., 1995). The internal bit cuts the ground as the probe advances; the jet breaks the ground up. Mud is pumped down to flush the cuttings back through rods to the surface where they are collected in a settling tank. The mud pressure, speed of advance, and cutter position and speed are adjusted to ensure the SBP replaces the ground as the probe advances. These are under the control of the operator therefore it is usual to specify minimum disturbance drilling and test depths and not to specify the drilling parameters. The cuttings in the return fluid are the only indication of ground through which the probe is drilled or jetted but the shape of the test curve does give an indication of the quality of the drilling process and the ground in which the test is carried out.

4.3 Full Displacement (Pushed-in) Pressuremeters (FDP)

The cone pressuremeter can be pushed into soil at a constant speed, usually 2 cm/sec, using a standard cone truck, either from the base of a borehole or the ground surface. The cone displaces the soil therefore no samples are available for inspection unless the pocket is overcored.
<table>
<thead>
<tr>
<th>Group</th>
<th>Name</th>
<th>Pressure Capacity MPa</th>
<th>Strain Capacity %</th>
<th>Diameter mm</th>
<th>Total Length m</th>
<th>Measuring Length mm</th>
<th>L/D</th>
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<td></td>
<td></td>
<td></td>
<td>80</td>
<td>0.9</td>
<td>600</td>
<td>7.5</td>
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<tr>
<td></td>
<td>Elastometer 100</td>
<td>10</td>
<td>12</td>
<td>66</td>
<td>520</td>
<td>7.9</td>
<td>one diameter</td>
<td>stiff clays, dense sands and weak rocks</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Elastometer 200</td>
<td>20</td>
<td>66</td>
<td>520</td>
<td>7.9</td>
<td>one diameter</td>
<td>weak to moderately strong rocks</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Goldsers GA20</td>
<td>20</td>
<td>20</td>
<td>70</td>
<td>420</td>
<td>6</td>
<td>two diameters</td>
<td>weak to moderately strong rocks</td>
<td></td>
</tr>
<tr>
<td></td>
<td>High Pressure Dilatometer</td>
<td>20</td>
<td>25</td>
<td>73</td>
<td>1.5</td>
<td>455</td>
<td>6.1</td>
<td>three diameters</td>
<td>weak to moderately strong rocks</td>
</tr>
<tr>
<td>Self-bored</td>
<td>Cambridge self-boring</td>
<td>4.5</td>
<td>15</td>
<td>84</td>
<td>500</td>
<td>6</td>
<td>three radii</td>
<td>all soils containing little or no gravel</td>
<td></td>
</tr>
<tr>
<td>pressuremeter</td>
<td>PAF-76</td>
<td>2.5</td>
<td>6</td>
<td>132</td>
<td>264</td>
<td>2</td>
<td>volume</td>
<td>all soils containing little or no gravel</td>
<td></td>
</tr>
<tr>
<td></td>
<td>weak rock self-boring</td>
<td>20</td>
<td>10</td>
<td>73</td>
<td>400</td>
<td>5.5</td>
<td>three radii</td>
<td>hard clays, very dense sands and weak rocks</td>
<td></td>
</tr>
<tr>
<td></td>
<td>pressuremeter</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pushed-in</td>
<td>Cambridge cone pressuremeter</td>
<td>4.5</td>
<td>50</td>
<td>44</td>
<td>450</td>
<td>10</td>
<td>three radii</td>
<td>all soils except gravel</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Pneum pressuremeter</td>
<td>2.5</td>
<td>30</td>
<td>31</td>
<td>124</td>
<td>4</td>
<td>volume</td>
<td>monaxial</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Fugro McClelland cone</td>
<td>10</td>
<td>50</td>
<td>44</td>
<td>450</td>
<td>10</td>
<td>three radii</td>
<td>all soils except gravel</td>
<td></td>
</tr>
<tr>
<td></td>
<td>pressuremeter</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>mini-pressuremeter</td>
<td>2.5</td>
<td>66</td>
<td>32</td>
<td>330</td>
<td>10</td>
<td>volume</td>
<td>all soils except gravel</td>
<td></td>
</tr>
</tbody>
</table>
**Table 1b** Commercially available pressuremeters - Manufacturers and Suppliers

<table>
<thead>
<tr>
<th>Name</th>
<th>Country</th>
<th>Manufactured Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Agapeo</td>
<td>France</td>
<td>Minard pressuremeter, cone pressuremeter</td>
</tr>
<tr>
<td>Geomatec</td>
<td>France</td>
<td>Minard pressuremeter</td>
</tr>
<tr>
<td>Cambridge In Situ</td>
<td>UK</td>
<td>probed pressuremeter, self-bored pressuremeter, cone pressuremeter</td>
</tr>
<tr>
<td>OYO Corporation</td>
<td>Japan</td>
<td>probed pressuremeter, self-bored pressuremeter</td>
</tr>
<tr>
<td>Rockset</td>
<td>Canada &amp; USA</td>
<td>probed pressuremeter, self-bored pressuremeter, cone pressuremeter</td>
</tr>
</tbody>
</table>

---


<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Hand Auger with Mud</th>
<th>Flight Auger</th>
<th>Driven Sampler</th>
<th>Driven Slotted Tube</th>
<th>Pushed Sampler</th>
<th>Pilot Hole and Pushed Sampler</th>
<th>Pilot Hole and Slotting</th>
<th>Core Barrel</th>
<th>Rotary Percussion</th>
<th>Open Hole Drift Bit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soft</td>
<td>2B</td>
<td>1</td>
<td>NR</td>
<td>NR</td>
<td>NR</td>
<td>2B</td>
<td>2</td>
<td>NR</td>
<td>NR</td>
<td>2B</td>
</tr>
<tr>
<td>Firm to stiff</td>
<td>1B</td>
<td>1</td>
<td>1B</td>
<td>NR</td>
<td>NR</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td>NR</td>
<td>NR</td>
</tr>
<tr>
<td>Stiff to hard</td>
<td>NA</td>
<td>NA</td>
<td>1B</td>
<td>2</td>
<td>NR</td>
<td>2</td>
<td>1</td>
<td>1</td>
<td>1B</td>
<td>NR</td>
</tr>
<tr>
<td>Silts</td>
<td>NA</td>
<td>NA</td>
<td>1B</td>
<td>2</td>
<td>NR</td>
<td>2B</td>
<td>2</td>
<td>2</td>
<td>NR</td>
<td>NR</td>
</tr>
<tr>
<td>Above GWL</td>
<td>1</td>
<td>2</td>
<td>NR</td>
<td>2</td>
<td>NR</td>
<td>2</td>
<td>2B</td>
<td>NR</td>
<td>NR</td>
<td>1B</td>
</tr>
<tr>
<td>Under GWL</td>
<td>1</td>
<td>2</td>
<td>NR</td>
<td>2</td>
<td>NR</td>
<td>2B</td>
<td>NR</td>
<td>2</td>
<td>NR</td>
<td>1B</td>
</tr>
<tr>
<td>Sands</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose and above GWL</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>NR</td>
<td>NR</td>
<td>2</td>
<td>NR</td>
<td>2</td>
<td>NA</td>
<td>1B</td>
</tr>
<tr>
<td>Loose and below GWL</td>
<td>NR</td>
<td>1</td>
<td>NR</td>
<td>NR</td>
<td>NR</td>
<td>NR</td>
<td>NR</td>
<td>2</td>
<td>NA</td>
<td>1B</td>
</tr>
<tr>
<td>Medium to dense</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td>NR</td>
<td>NR</td>
<td>2</td>
<td>NR</td>
<td>2</td>
<td>2B</td>
<td>1B</td>
</tr>
<tr>
<td>Sand and Gravel</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NR</td>
<td>2</td>
<td>NA</td>
<td>NA</td>
<td>2</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Dense</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NR</td>
<td>1D</td>
<td>NA</td>
<td>NA</td>
<td>2</td>
<td>NR</td>
<td></td>
</tr>
<tr>
<td>Rock</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weathered rock</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>2B</td>
<td>2</td>
<td>1</td>
<td>1B</td>
<td>2B</td>
</tr>
<tr>
<td>Strong Rock</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>1</td>
<td>2B</td>
<td>2B</td>
</tr>
</tbody>
</table>

1 - recommended; 2 - acceptable; NR - not recommended; NA - not applicable; B - conditional; D - pilot hole drilled first
failure of the soil in extension. Typically the stress range for an unload-reload cycle in clay is equal to the undrained shear strength and in sands, about 40% of the effective pressure at the start of unloading. Theoretically, the limit to the unloading cycle is a function of the strength of the soil but in practice this is unknown at the time of the pressuremeter test therefore a reduced limit is assumed.

Pressuremeter tests were developed primarily to determine the stiffness of the ground, and that is still the main purpose. Stiffness can be derived from the latter part of a test curve but if strength is the main parameter required from a test then unload-reload cycles for stiffness should not be conducted since the effects of creep and consolidation may affect the results.

In most pressuremeter tests the membrane is expanded until the maximum average cavity strain specified is reached. This is usually the strain capacity of the probe. A test can be terminated at a smaller average strain for one of the following reasons.

1) The maximum pressure capacity of the probe is insufficient to cause the ground to yield. This is common when testing rock.

2) The membrane bursts because of damage caused either during installation or by discontinuities in the ground or by expansion up the pocket.

3) The total volume capacity is reached because of non-uniform expansion.

4) The pocket is too large due to difficulty in creating the pocket.

5) The test specification requires modulus only and therefore the operator terminates the test to prevent possible damage.

Typical shapes of PDP test curves demonstrating termination criteria are shown in Figure 6.

5.1 Ménard Test

This is a special stress controlled test forming part of a complete specification covering the probe, installation, test and interpretation. It is used to obtain design parameters directly from volume displacement triaxial probes, or by using the same technique but giving the results provided they are of adequate length (Yeung and Carter, 1990). The triaxial probe is most common in France where the test was
<table>
<thead>
<tr>
<th>Calibration</th>
<th>Measuring System</th>
<th>Frequency</th>
<th>Additional Calibrations</th>
<th>Soft Ground</th>
<th>Most Ground</th>
<th>Strong Ground</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>volume radial</td>
<td></td>
<td></td>
<td>Max Pres 1 MPa</td>
<td>Max Pres 4 MPa</td>
<td>Max Pres 20 MPa</td>
</tr>
<tr>
<td>transducers</td>
<td>fixed yes</td>
<td>1 start and finish of project 2 regular intervals during project</td>
<td>1 any change in the transducers 2 change to the lead connecting the probe to the surface 3 following damage to probe which involves thorough cleaning and drying</td>
<td>critical</td>
<td>critical</td>
<td>critical</td>
</tr>
<tr>
<td>system compliance</td>
<td>yes *</td>
<td>1 start and finish of project 2 regular intervals during project</td>
<td>1 change to the lead connecting the probe to the surface 2 change to the control unit</td>
<td>important</td>
<td>important</td>
<td>critical</td>
</tr>
<tr>
<td>membrane stiffness</td>
<td>yes yes</td>
<td>1 start and finish of project 2 regular intervals during project 3 every time a membrane is replaced</td>
<td></td>
<td>critical</td>
<td>important</td>
<td>not important</td>
</tr>
<tr>
<td>membrane compression</td>
<td>no yes</td>
<td>1 start and finish of project 2 regular intervals during a project 3 every time a membrane is replaced</td>
<td></td>
<td>unnecessary</td>
<td>not important</td>
<td>critical</td>
</tr>
<tr>
<td>membrane thinning</td>
<td>no yes</td>
<td></td>
<td></td>
<td>not important</td>
<td>not important</td>
<td>critical</td>
</tr>
<tr>
<td>zero</td>
<td>yes yes</td>
<td>1 prior to lowering into borehole</td>
<td></td>
<td>critical</td>
<td>critical</td>
<td>critical</td>
</tr>
</tbody>
</table>

* depends on stiffness of ground
<table>
<thead>
<tr>
<th>Name</th>
<th>Type</th>
<th>Probe</th>
<th>Ground Conditions</th>
<th>Recording Intervals</th>
<th>Pressure Increment</th>
<th>Volume Increment/Strain Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>AFNOR (1991)</td>
<td>stress</td>
<td>MPM</td>
<td>all</td>
<td>15, 30, 60 sec</td>
<td>1 atm/10</td>
<td></td>
</tr>
<tr>
<td>ISRM (1987)</td>
<td>stress</td>
<td>PBP</td>
<td>rock</td>
<td>not specified</td>
<td>not specified</td>
<td></td>
</tr>
<tr>
<td>ASTM D4719-94 (1994)</td>
<td>stress</td>
<td>PBP</td>
<td>all</td>
<td>20 sec</td>
<td>1 atm/7 to 1 atm/10</td>
<td></td>
</tr>
<tr>
<td>ASTM D4719-94 (1994)</td>
<td>strain</td>
<td>PBP</td>
<td>all</td>
<td>30 sec</td>
<td>0.05 to 0.1 x Vₘ</td>
<td></td>
</tr>
<tr>
<td>GOST 20276-85 (1985)</td>
<td>stress</td>
<td>PBP</td>
<td>soils</td>
<td>varies</td>
<td>25 kPa</td>
<td></td>
</tr>
<tr>
<td>Stress</td>
<td>stress</td>
<td>all</td>
<td>all</td>
<td>1 to 10 to 30 sec</td>
<td>varies</td>
<td></td>
</tr>
<tr>
<td>Strain</td>
<td>strain</td>
<td>SBE</td>
<td>all</td>
<td>1 to 10 to 30 sec</td>
<td>1%/min</td>
<td></td>
</tr>
<tr>
<td>Strain</td>
<td>strain</td>
<td>FDP</td>
<td>all</td>
<td>1 to 10 to 30 sec</td>
<td>5%/min</td>
<td></td>
</tr>
<tr>
<td>Holding</td>
<td>strain</td>
<td>all</td>
<td>all</td>
<td>1 to 10 to 30 sec</td>
<td>1% min during loading; 0% during holding</td>
<td></td>
</tr>
</tbody>
</table>
Figure 7  A stress controlled test

The use of monocell probes for this method is common in Japan.

The probe is lowered into the prebored pocket and expanded in about ten to fourteen equal stress increments until the volume of the pocket is doubled in size. Each increment is maintained for one minute with readings of volume being recorded at 15 sec, 30 sec and one minute after applying the increment. Figure 7, a typical Ménard test, shows the stress and strain rate during a test and the curve used for interpretation. The French standard (AFNOR, 1991) and American standard (ASTM, 1987, 1994) describe the test in detail. It is usual in France to plot volume change against pressure since the volume change is the dependent parameter but to be consistent in this report pressure is shown plotted against volume change.

5.2 Stress Controlled Test

There are usually more increments of stress in other types of stress controlled tests than in the Ménard test and each increment may be maintained for a longer period, possibly up to two minutes. Tests can include unload-reload cycles and a final unloading curve. There is no international standard for this test but a number of procedures are recognised as good practice (e.g. ASTM, 1994). The form of the curve is similar to that shown in Figure 7 though, in the case of an SBP or FDP test, the membrane begins to move once the corrected internal applied pressure exceeds the external ground pressure.

5.3 Strain Controlled Test

Strain controlled tests can be controlled by either the volume of injected liquid or feedback from the displacement transducers. In the former test procedure, the volume of the probe is increased in equal increments of volume of liquid. The
5.4 Holding Test

A holding test, a form of strain controlled consolidation test shown in Figure 9, is used in clay to determine the horizontal coefficient of consolidation. The decay in total, and possibly pore, pressure with time is monitored while maintaining the expanding section at a constant volume. The membrane is expanded in the usual manner possibly with an unload-reload cycle, up to about 10% cavity strain. In volume displacement type probes the volume is then held constant. In radial displacement type gas filled probes the rate of expansion is reduced to zero by reducing the pressure in a controlled manner. The interpretation of the test is based on the time required to achieve 50% dissipation of the excess pore pressure generated during expansion. Practically 50% of the pore pressure dissipates in the same time that it takes for the total pressure to reduce by 50% of the difference between that when holding starts and that at equilibrium conditions when all excess pore pressures have dissipated. It is often easier to measure total pressure rather than pore pressure.

6 THE TEST CURVE

The data recorded during a test are converted to internal applied pressure and volumetric or cavity strain and then corrected, if necessary, for membrane stiffness, and either compression (due to pressure and membrane stretching) or system compliance to produce the test curve. Data are either recorded at equal intervals of time, usually 5, 10 or 20 sec, independent of the test type or at intervals related to the application of increments of volume or pressure. In the former case all data are plotted to give a test curve; in the latter case the data at the end of each increment are plotted.

Typical curves for PBP, SBP and FDP tests are shown in Figure 10 to indicate the differences between the shape of the curves and the volumetric strains occurring within the different type of tests. The shape of these curves indicates the quality of the test; examples of different quality PBP test curves are shown in Figure 6. The shape of a SBP test curve can sometimes indicate the quality of the test.

It is possible to identify the type of ground from the shape of the curve (e.g. Baguidin et al 1978) but it is more common to use information from adjacent boreholes or, in the case of PBP tests, the samples or cuttings obtained from the test horizon since the
Figure 10  Typical shapes of pressuremeter tests

7 APPLICATIONS OF THEORIES OF EXPANDING CYLINDRICAL CAVITIES TO THE INTERPRETATION OF TESTS

A pressuremeter test is assumed to be the expansion of a cylindrical cavity, the theories of which are well documented. It is assumed that the cavity is infinitely long and displacements take place in the radial direction, that is plane strain conditions apply. Most pressuremeters are installed in vertical boreholes therefore expansion takes place on a horizontal direction. The vertical geostatic stress is assumed to be the intermediate stress. The geometry of the problem and the co-ordinate system used are shown in Figure 11. Further references to examples of theories of expanding cavities are given in Table 5.

Measurements of pressure and displacement are recorded within the membrane. The displacement is usually expressed in terms of either cavity strain, the ratio of the displacement to the original cavity radius, or volumetric strain, the ratio of the change in volume to the original volume of the test pocket. The cavity radius at the in situ total horizontal stress is the reference datum which is shown in Figure 12 for the three group of pressuremeters. This can only be determined directly from good quality SBP tests when the probe is installed in the ground with minimal disturbance. In all other cases the reference datum has to be either assumed or calculated, usually by an iterative procedure.

Equations of equilibrium and compatibility are established and these, together with an assumed relationship between stress and strain, are used to predict the pressuremeter curve. Table 5 lists a
<table>
<thead>
<tr>
<th>Author</th>
<th>Interpretation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lamb (1852)</td>
<td>Linear elastic material</td>
</tr>
<tr>
<td>Bishop et al (1945)</td>
<td>Cohesive material</td>
</tr>
<tr>
<td>Monard (1957)</td>
<td>Frictional cohesive material</td>
</tr>
<tr>
<td>Cassan (1960)</td>
<td></td>
</tr>
<tr>
<td>Veizé (1972)</td>
<td></td>
</tr>
<tr>
<td>Gibson and Anderson (1961)</td>
<td>Linear elastic perfectly plastic material with no volume changes</td>
</tr>
<tr>
<td>Windle and Wrench (1977)</td>
<td></td>
</tr>
<tr>
<td>Jefferies (1983)</td>
<td></td>
</tr>
<tr>
<td>Hoilseby and Withers (1988)</td>
<td></td>
</tr>
<tr>
<td>Denby and Clough (1980)</td>
<td>Non linear elastic perfectly plastic material with no volume change</td>
</tr>
<tr>
<td>Ferreira and Robertson (1992)</td>
<td></td>
</tr>
<tr>
<td>Prevost and Hoog (1975)</td>
<td>Elastic-plastic with strain hardening or softening with no volume change</td>
</tr>
<tr>
<td>Lavagni (1963)</td>
<td>Linear elastic perfectly plastic material with volume changes</td>
</tr>
<tr>
<td>Salcicano (1966)</td>
<td></td>
</tr>
<tr>
<td>Veizé (1972)</td>
<td></td>
</tr>
<tr>
<td>Hughes et al (1977)</td>
<td></td>
</tr>
<tr>
<td>Robertson and Hughes (1986)</td>
<td></td>
</tr>
<tr>
<td>Hoilseby et al (1986)</td>
<td></td>
</tr>
<tr>
<td>Palmer (1972)</td>
<td>Cohesive material with no volume changes</td>
</tr>
<tr>
<td>Bugiuclin et al (1972)</td>
<td></td>
</tr>
<tr>
<td>Lavagni (1972)</td>
<td></td>
</tr>
<tr>
<td>Manasero (1989)</td>
<td>Cohesicless material with volume changes</td>
</tr>
<tr>
<td>Arnold (1981)</td>
<td>Hyperbolic pressuremeter test curve</td>
</tr>
<tr>
<td>Fahey and Carter (1993)</td>
<td>Hyperbolic pressuremeter test curve</td>
</tr>
<tr>
<td>Sadegh and Clarke (1995)</td>
<td>Parabolic pressuremeter test curve</td>
</tr>
</tbody>
</table>
This cycle can also be used to produce the variation of shear modulus with strain (Muir Wood, 1990).
The latter part of an ideal pressuremeter curve when plotted as applied pressure against the
logarithmic value of volumetric strain is linear and the slope is equal to the undrained shear strength of
the clay test. This is independent of the deformation model used. It represents the post peak value
of undrained shear strength.
These models were developed for loading from in
situ stress and can only be reliably applied to quality
SBP tests and, in the case of the stiffness to any test
that contains an unload-reload cycle. Complete
unloading curves can be obtained from all
pressuremeter tests in clays. The advantage of using
this portion of the curve is that it is independent of
the effects of installation therefore the reference
datum is no longer required. Linear elastic perfectly
plastic (Jefferys, 1988, Houlsoy and Wittmers, 1988)
and hyperbolic unloading (Ferreira and Robertson,
1992) models have been developed for interpreting
SBP and FDP tests.

7.1 Tests in Clays
Tests in clays are generally interpreted using the
assumption that the deformation is undrained, that is
there are no volume changes within the clay. A
further assumption is that there are no inherent creep
effects. A pressuremeter test curve can be
interpreted directly to give a stress-strain curve
(Baquelin et al., 1972, Ladanyi, 1972 and Palmer,
1972). This has the advantage that it is unnecessary
to assume a deformation model for the clay. The
main disadvantages are the need to identify the
reference datum and the assumption the clay is
unaffected by installation. In practice, this method is
only used to interpret parts of a pressuremeter test
curve and not to give a complete shear stress-strain
curve. It can be used to interpret both PBP and SBP
tests though it is most commonly restricted to SBP
tests.

The most common assumption when interpreting
pressuremeter tests in clays is that the clay is a linear
elastic perfectly plastic material (Gibson
and Anderson, 1961). This allows the stiffness
and undrained shear strength of a clay to be obtained
directly. A correction can be applied to take into
account the finite length of the membrane (Yeung
and Carter, 1990). It is usual to obtain the stiffness
from an unload-reload cycle since the slope of this
cycle is twice the shear modulus (Palmer, 1972).
<table>
<thead>
<tr>
<th>Installation</th>
<th>self-boring</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1. reduction of in situ stress</td>
</tr>
<tr>
<td></td>
<td>2. softening of ground adjacent to probe</td>
</tr>
<tr>
<td></td>
<td>3. erosion of weaker layers</td>
</tr>
<tr>
<td></td>
<td>4. non cylindrical pocket</td>
</tr>
<tr>
<td></td>
<td>5. reference (strain) datum unknown (not necessary for MPN)</td>
</tr>
<tr>
<td>pushing in</td>
<td>1. over drilling giving reduction in in situ stress (of self-boring)</td>
</tr>
<tr>
<td></td>
<td>2. under drilling causing cavity expansion (of pushing in)</td>
</tr>
<tr>
<td></td>
<td>3. shear between probe and ground</td>
</tr>
<tr>
<td>probe alignment (vertical)</td>
<td>1. non cylindrical expansion</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Ground Conditions</th>
<th>1. non cylindrical expansion</th>
</tr>
</thead>
<tbody>
<tr>
<td>anisotropy</td>
<td>1. not a continuum</td>
</tr>
<tr>
<td>discontinuities</td>
<td>1. non cylindrical expansion</td>
</tr>
<tr>
<td>variation in ground type over test section</td>
<td>1. non cylindrical expansion</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Process</th>
<th>1. vertical stress not intermediate stress in heavily overconsolidated soils</th>
</tr>
</thead>
<tbody>
<tr>
<td>membrane length</td>
<td>1. undrained</td>
</tr>
<tr>
<td></td>
<td>2. non cylindrical expansion</td>
</tr>
<tr>
<td>tensile cracking at shallow depth</td>
<td>1. discontinuities form</td>
</tr>
<tr>
<td>tensile cracking in heavily overconsolidated soils</td>
<td>1. discontinuities form</td>
</tr>
<tr>
<td>ratio</td>
<td>1. ground properties are strain rate dependent</td>
</tr>
<tr>
<td></td>
<td>2. partial drainage</td>
</tr>
<tr>
<td>volume displacement type probes</td>
<td>1. average expansion</td>
</tr>
<tr>
<td>radial displacement type probes</td>
<td>1. expansion at position of transducers</td>
</tr>
</tbody>
</table>
The method proposed by Schnaid and Houlby (1990) allows a realistic estimate of $\phi'$ to be made.

The shear modulus is usually taken from an unload-reload cycle assuming the sand being tested is elastic. Shear modulus varies with the effective stress at which it is measured. As the membrane expands, the effective stress in the sand increases thus the shear modulus increases. Further, with clays, the modulus is non-linear, that is it varies with the strain range over which it is measured. Ballotti et al (1989) have proposed a method of obtaining an elemental shear modulus normalized with respect to the in situ horizontal stress.

7.3 Tests in Rocks

Rocks range from highly weathered residual soils, through moderately weathered rock with unweathered inclusions in a weathered matrix, to hard rocks which may contain discontinuities. The expansion of most pressuremeter tests in the moderately to unweathered rocks is limited since the pressure capacity of most pressuremeters is insufficient to cause the rock to yield. For this reason most pressuremeter tests in rocks are used to measure the stiffness of the rock.

Tests are analysed as if the rock were behaving elastically. In practice, rocks often contain discontinuities which implies the expansion curve is a function of the rock mass behaviour which could include development of fissures and opening and/or closing of existing discontinuities. The modulus from an unload-reload cycle, in general, represents the stiffness of the intact rock (Johnston and Haberfeld, 1990), that is an upper bound value. The initial modulus may represent the stiffness of the rock mass.

A pressuremeter test in weak rock can cause failure of the rock. It is commonly assumed that if this happens then there are no volume changes within the rock (undrained) therefore the analyses developed for clays could be used to give an undrained shear strength. However, because of the stiffness of rock and presence of discontinuities most rock tests will be partially or fully drained (Johnston and Haberfeld, 1990). Tests should be interpreted to give cohesion and angle of shearing resistance but it is not possible to determine these values without an independent assessment of those values. For those reasons strengths derived from tests in rock should be treated with caution.

8 FACTORS AFFECTING INTERPRETATION

There are several reasons, summarised in Table 6, why a pressuremeter test cannot be simply interpreted using theories of cavity expansion. Errors arising because of site and testing operations and ground conditions are common to all tests including pressuremeter tests, the magnitude of each effect varying between tests.

Tests in soils and rocks are often carried out to classify the ground and to obtain parameters for design. Design methods are based on test results and modified to take into account actual performance. Only recently, with improved models and numerical methods has it been possible to begin to attempt to model ground response exactly. Thus, the results obtained from pressuremeter tests can only be validated by either comparing results from pressuremeter tests with results from other tests, or comparing predictions based on pressuremeter test results with observations of actual performance. These two approaches are discussed under Interpretation and design.

9. INTERPRETATION

Pressuremeter tests can be analysed as expanding cavities but because of the factors given in Table 6 it is common to apply empirical methods or simple models in the interpretation. There are three methods used to interpret pressuremeter tests:

i) Empirical interpretation to give design parameters directly.

ii) Assume a shear stress-strain model and integrate that model to give a pressuremeter curve which is then made to fit the test data.

iii) Fit a curve to the pressuremeter data and differentiate that to give a shear stress-strain curve.

The first method, originally developed by Ménard (1963) and subsequently expanded by others (e.g. Bagnold et al, 1978 and Brizand, 1992), is based on correlations between observations of full scale structures and pressuremeter test curves. The parameters taken from the test curve are a function of the probe, installation and test procedure and are used in design methods specially developed for that pressuremeter test. Any variations from the specified procedure will invalidate the results.

The second method enables the stiffness and strength of the ground to be quantified. The three properties of the ground obtained from a
Table 7 Commonly used methods to interpret pressuremeter tests

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Parameter</th>
<th>Probe</th>
<th>Method</th>
<th>Reference</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>all soils</td>
<td>G&lt;sub&gt;u&lt;/sub&gt; or G&lt;sub&gt;c&lt;/sub&gt;</td>
<td>all probes</td>
<td>unload-reload cycle</td>
<td>0.5 (dp/dn&lt;sub&gt;0&lt;/sub&gt;)</td>
<td></td>
</tr>
<tr>
<td>clay</td>
<td>( \sigma_3 )</td>
<td>PBP</td>
<td>curve-fitting</td>
<td>Marsland and Randolph (1977)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( \sigma_0 )</td>
<td>SBP</td>
<td>directly from curve</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>( \sigma_0 )</td>
<td>FDP</td>
<td>unloading curve</td>
<td>Housby and Withers (1988)</td>
<td>( p = \sigma_0 [1 + \ln(G_{sh})] )</td>
</tr>
<tr>
<td></td>
<td>G&lt;sub&gt;u&lt;/sub&gt; or G&lt;sub&gt;c&lt;/sub&gt;, v. e.</td>
<td>all probes</td>
<td>unload-reload cycle</td>
<td>Muir Wood (1990)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( \sigma_0 )</td>
<td>PBP</td>
<td>Menard limit pressure</td>
<td>Amar et al (1975)</td>
<td>((p_{cm} - \sigma_0) &lt; 300 \text{ kPa} ) ( (p_{cm} - \sigma_0)/5.5 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>((p_{cm} - \sigma_0) &gt; 500 \text{ kPa} ) ( 25 + (p_{cm} - \sigma_0)/10 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( u_c )</td>
<td>SBP</td>
<td>latter part of loading curve</td>
<td>Windle and Wroth (1977)</td>
<td>( (p - \sigma_0) = s_i [1 + \ln(G_{sh} + \ln(\Delta V/V))] )</td>
</tr>
<tr>
<td>sand</td>
<td>( \sigma_0 )</td>
<td>PBP, FDP</td>
<td>not recommended</td>
<td>Bellati et al (1989)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( \sigma_0 )</td>
<td>SBP</td>
<td>directly from curve (with caution)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>G&lt;sub&gt;u&lt;/sub&gt; or G&lt;sub&gt;c&lt;/sub&gt;, v. e.</td>
<td>all probes</td>
<td>unload-reload cycle</td>
<td>Bellati et al (1989)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( q^\prime )</td>
<td>FBP</td>
<td>not recommended</td>
<td></td>
<td>( p_s = 2.5 \gamma B - 24V4 )</td>
</tr>
<tr>
<td></td>
<td>( \phi^\prime )</td>
<td>SBP</td>
<td>latter part of loading curve</td>
<td>Hughes et al (1977)</td>
<td>( \sin \phi^\prime = \frac{1}{s} )</td>
</tr>
<tr>
<td></td>
<td>( \phi^\prime )</td>
<td>FDP</td>
<td>unloading curve</td>
<td>Housby and Nutt (1995)</td>
<td>( \theta_{sh} = \theta_{sh0} = 2.21 + 19.33 \text{ Dr} )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \theta_{sh} = \theta_{sh0} = 3.80 + 9.84 \text{ Dr} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \theta_{sh} = \theta_{sh0} )</td>
<td></td>
</tr>
</tbody>
</table>
pressuremeter test should be the in situ total horizontal stress, stiffness and strength. Perfectly plastic, linear elastic perfectly plastic, hyperbolic and parabolic soil stress-strain models have been suggested. The effects of installation and differences between actual and assumed behaviour can invalidate the assumptions made. The natural variability of the ground is such that variation in in situ properties can exceed the difference in properties obtained using more complex and, possibly, realistic models compared to the simpler models. A number of techniques to correct for installation effects have been proposed but there is little practical evidence that the improvement in the quality of the results justify the application of any of these techniques. For those reasons the simpler models given in Table 7 together with the relevant equations, are more common. These models can be used to give parameters adequate for design by interpreting selected parts of a test curve.

The third method would be the preferred method if a test represented the true ground response at the in situ geostatic conditions. In practice, the parameters obtained are dependent on the effects of installation. This approach is often limited to parts of a test curve, those that are not dependent on installation, for example interpretation of unload-reload cycles.

Thus, the interpretation of a pressuremeter test is based on theory modified by experience to take into account effects of installation and ground conditions. The results obtained depend on the method of interpretation which implies that the use of the results in design is dictated by the method of interpretation.

It is common to independently select parameters from different parts of the test curve. For example, stiffness may be taken from an unload-reload cycle whereas strength is taken from the latter part of the curve. The parameters are dependent since they are a function of the geological history. Two methods have evolved to take this dependency into account. The first is to select the parameters and then ensure they are compatible using a framework developed from theories of soil and rock mechanics. The second, known as computer aided modelling (CAM), is one in which a theoretical model is made to fit the test data and the curve is then differentiated to produce the stress-strain curve. This could include regression techniques as well as numerical methods such as the finite element method.
<table>
<thead>
<tr>
<th>Ground Type</th>
<th>Description</th>
<th>Category</th>
<th>$P_u$, MPa</th>
<th>Non Displacement Piles and Caissons</th>
<th>Full Displacement Piles</th>
</tr>
</thead>
<tbody>
<tr>
<td>clay, silt</td>
<td>soft clay or silt</td>
<td>A</td>
<td>&lt; 0.7</td>
<td>1.1</td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td>medium stiff clay or silt</td>
<td>B</td>
<td>1.2 - 2.0</td>
<td>1.2</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>hard clay or silt</td>
<td>C</td>
<td>&gt; 2.5</td>
<td>1.3</td>
<td>1.6</td>
</tr>
<tr>
<td>sand, gravel</td>
<td>loose sand or gravel</td>
<td>A</td>
<td>&lt; 0.5</td>
<td>1.0</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td>medium dense sand or gravel</td>
<td>B</td>
<td>1.0 - 2.0</td>
<td>1.1</td>
<td>3.7</td>
</tr>
<tr>
<td></td>
<td>dense sand or gravel</td>
<td>C</td>
<td>&gt; 2.5</td>
<td>1.2</td>
<td>3.2</td>
</tr>
<tr>
<td>chalk</td>
<td>soft chalk</td>
<td>A</td>
<td>&lt; 0.7</td>
<td>1.1</td>
<td>1.6</td>
</tr>
<tr>
<td></td>
<td>weathered chalk</td>
<td>B</td>
<td>1.0 - 2.5</td>
<td>1.4</td>
<td>2.2</td>
</tr>
<tr>
<td></td>
<td>hard chalk</td>
<td>C</td>
<td>&gt; 3.0</td>
<td>1.8</td>
<td>2.6</td>
</tr>
<tr>
<td>marl</td>
<td>bristle marl</td>
<td>A</td>
<td>1.5 - 4.0</td>
<td>1.8</td>
<td>2.6</td>
</tr>
<tr>
<td></td>
<td>hard marl</td>
<td>B</td>
<td>&gt; 4.5</td>
<td>1.8</td>
<td>2.6</td>
</tr>
<tr>
<td>weak rock</td>
<td>weathered rock</td>
<td>A</td>
<td>2.5 - 4.0</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td></td>
<td>fissured rock</td>
<td>B</td>
<td>&gt; 4.5</td>
<td>*</td>
<td>*</td>
</tr>
</tbody>
</table>

* choose the factor from most closely related soil type
The first method, the empirical method, is commonly applied to PBP tests in soils since non-homogeneous conditions exist around the probe due to installation. The second method, based on soil models, is usually applied to SBP tests and parts of FDP and PBP tests. The third method, based on a model pressuremeter curve, has had little use in practice but the development of CAM may result in increasing application of this method.

9.1 The Ménard Pressuremeter Modules and the Ménard Limit Pressure

This method of interpretation is restricted to volume displacement tricell PBP tests that comply with the specification developed by Ménard, though Japanese experience has demonstrated that it can be successfully applied to radial displacement monocell probes. The pressuremeter modulus, Em, is originally defined as the slope of the linear portion of the strain profile on the onset of the elastic response and yield as shown in Figure 13a. This stress range is defined as the range over which the rate of volume change during the increments of stress are a minimum. Note that Figure 13a is produced in accordance with AFNOR (1991) rather than the more conventional pressure versus strain curve. AFNOR (1998) specifies the pressure range to calculate Em as shown in Figure 13b.

The Ménard limit pressure, pvl, is the pressure required to double the volume of the cavity (AV/Vc = 1). It is not the same as the true limit pressure which is the pressure required to cause continuous expansion (AV/Vc = 1).

9.2 Total Horizontal Stress

Total horizontal stress is the in situ geostatic stress. Its selection is, at best, subjective since it depends on identifying a single point on the test curve. There are a number of techniques (Clarke, 1995) to help identify that point but they are based on assumptions which may be invalidated either by the installation process or by the difference between the assumed model and the actual ground response.

Figure 12 shows the position of the horizontal stress on the ideal curves produced by the three types of pressuremeters. The horizontal stress can only be identified directly from an SBP test in which the installation disturbance is minimal. The best way to identify that point is to review several tests from one borehole and recognize the signature of each transducer which is a function of the transducer design, drilling and in situ stress. It is often possible to identify clearly the horizontal stress from one test, usually a shallow test, and relate this to the signature of the transducer. This then enables the horizontal stress to be identified from all other tests. Further checks are carried out on the results to ensure they comply with geological information. It is possible to use curve fitting methods to identify the horizontal stress but the values must be treated with caution since it has been found that the results are inconclusive.

It is not possible to independently measure horizontal stress directly therefore any value chosen must be subjectively chosen. Installation disturbance increases as the soil stiffness and particle size increases. For these reasons the credibility of the value of horizontal stress is less for stiff soils and sands than it is for soft clays. The interpretation of tests, especially when deriving horizontal stress, should be done by those experienced in operating and interpreting pressuremeter tests.
<table>
<thead>
<tr>
<th>Soil Type</th>
<th>$p_{f,m}$</th>
<th>Borad Concrete</th>
<th>Borad and Fined</th>
<th>Driven</th>
<th>Grouted</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MPa</td>
<td>Concrete</td>
<td>Steel</td>
<td>Concrete</td>
<td>Steel</td>
</tr>
<tr>
<td>soft clay</td>
<td>0 - 0.7</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>stiff clay</td>
<td>0.7 - 1.2</td>
<td>A, (B)</td>
<td>A, (B)</td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>very stiff clay</td>
<td>&gt; 2.5</td>
<td>B, (C)</td>
<td>A, (B)</td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>loose sand</td>
<td>0 - 0.7</td>
<td>-</td>
<td>B</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>medium dense sand</td>
<td>0.7 - 1.2</td>
<td>-</td>
<td>B*</td>
<td>A</td>
<td>C</td>
</tr>
<tr>
<td>very dense sand</td>
<td>&gt; 2.5</td>
<td>-</td>
<td>C*</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>completely weathered chalk</td>
<td>0 - 0.7</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>-</td>
</tr>
<tr>
<td>partially weathered chalk</td>
<td>1 - 2.3</td>
<td>C, (D)</td>
<td>B, (C)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>marl</td>
<td>1.5 - 4</td>
<td>C</td>
<td>C</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>stiff marl</td>
<td>&gt; 4.5</td>
<td>D, (E)</td>
<td>-</td>
<td>C</td>
<td>D</td>
</tr>
<tr>
<td>weathered rock</td>
<td>2.5 - 4</td>
<td>F</td>
<td>-</td>
<td>D</td>
<td>D</td>
</tr>
</tbody>
</table>

Curves in brackets only apply for well constructed piles.

* Take lower curve when pile is more than 30m deep.

It is recommended that unit skin friction for piles in chalk should be obtained from loading tests.
The horizontal stress can only be found from PBP and PDIP tests by fitting models to the data (e.g. CAM). However, the results are often inconclusive therefore any quoted horizontal stress should be viewed with caution.

9.3 Shear Modulus

The pressuremeter is an ideal test for measuring the stiffness of the ground in situ since the stiffness obtained, the shear modulus, is theoretically the same for undrained and drained conditions. The pressuremeter has increasingly been used to measure stiffness of all soils and rocks. An average stiffness, which represents the average secant stiffness of all the elements radiating from the probe, can be determined directly. An elemental stiffness which represents the stiffness of a single element, of the stiffness of a triaxial specimen, can be deduced.

The shape of the initial loading portion of any pressuremeter test curve is a function of the installation disturbance therefore the stiffness is generally taken from an unload-reload cycle as shown in Figure 14. In Ménard tests the pressuremeter modulus is taken from the initial loading curve. In tests in rocks the modulus from an unload-reload cycle represents the modulus of the intact rock whereas the initial modulus may represent the mass modulus. In that case the initial modulus may be more relevant to design.

The unload-reload cycle approximately represents elastic behaviour if the stress range is insufficient to cause yield in extension. An average stiffness, $G_{av}$, is obtained by fitting a line to all the data forming the cycle. The slope of that line is approximately twice the shear modulus. This modulus is an average secant stiffness. This modulus may be similar to the average modulus deduced from observations of the performance of structures (O’Brien and Newman, 1990 and Clarke, 1993).

A more applicable value can be obtained by selecting the data over a specified strain range typical of that expected in the ground. The magnitude of the unload-reload cycle is normally specified in terms of pressure as explained in Section 5. Rather than use the complete cycle to give $G_{av}$ a portion of the unloading or reloading curve can be used to give an average secant stiffness over a specified strain range. It is recommended that the slope is calculated from the reloading portion using the minimum cavity strain as one point since this gives more consistent results (Clarke, 1993). Normally a cavity strain range of 0.1% is used since this represents typical average strains in the ground beneath or adjacent to a well designed structure. When designing laterally loaded piles or retaining structures it may be necessary to use a cavity strain range in excess of 0.1% since the deformations may be greater.

The unload-reload cycle may be interpreted to give a non linear stiffness curve (Muir Wood, 1990, Bellotti et al, 1989, Fahey and Carter, 1993). Simply plotting the variation of the secant of an unload-reload cycle using either the minimum or maximum cavity strain as a common point produces a variation in average stiffness with cavity strain. This can be
Figure 16 Correlations between the Ménard limit pressure and undrained shear strength from PBP tests in clays (after Amar et al., 1975)

converted to an elemental stiffness directly comparable to that measured in a triaxial test (Muir Wood, 1990; Bellotti et al., 1989). Generally the profile obtained from tests in stiff clay is similar to that obtained from local strain measurements in triaxial tests if the modulus is normalised to the in situ effective stress or consolidation pressure. The non-linear stiffness curve corrected for stress level and normalised with respect to the in situ stress is the relevant design curve for sands.

9.4 Undrained Shear Strength

A peak and post peak undrained shear strength can be obtained from SBP tests in clay. However, the peak strength obtained is sensitive to any installation disturbance and is usually ignored. The post peak strength can be obtained from the latter part of the loading curve assuming a perfectly plastic model, as shown in Figure 15.

It is possible to derive a strength from a PBP test but the expansion must be sufficient to ensure clay unaffected by installation is loaded. Further, the reference datum has to be correctly identified. It is possible to adjust the reference datum until the derived undrained shear strength is compatible with the reference datum chosen (Mansfield and Randolph, 1977). It is more usual to obtain an undrained strength by an empirical correlation such as that shown in Figure 16 where \( p_{85} \) is the applied pressure required to double the cavity size and \( p_{u} \) is the estimated in situ total horizontal stress (Ménard, 1965).

The undrained shear strength can be obtained from the unloading curve of a FDP test as shown in Figure 17. It is also possible to use the unloading curve from PBP and SBP tests if computer aided modelling is used with an unloading soil model.

9.5 Angle of Shearing Resistance

The angle of shearing resistance can be obtained from the latter part of the loading curve of an SBP test in sand (Hughes et al., 1977) as shown in Figure 18. This method only applies to dense sands though a correction can be applied from tests in loose sands (Robertson and Hughes, 1986). There must be some disturbance during installation which implies that the value obtained must be treated with caution.
It is not possible to obtain directly an angle of shearing resistance from PBP and FDP tests because of installation disturbance. However, empirical correlations (Gambin, 1977, Houlby and Nutt, 1995) exist which are being further developed as more testing is undertaken.

9.6 Coefficient of Consolidation

A horizontal coefficient of consolidation can be obtained from a strain holding test in clay. Ideally the pore pressure decay curve is used to estimate the time for 50% dissipation of pore pressure though it is possible, within experimental accuracy, to use the total pressure decay curve. This, together with the time factor derived from Figure 19 using results obtained from the expansion stage of the test, gives the coefficient of consolidation.

9.7 Computer Aided Modelling (CAM)

An alternative approach to the interpretation of pressuremeter results involves the matching of the pressure expansion curve and sometimes the pressure contraction curve with a numerically generated curve. This method is known as computer aided modelling (CAM). Generally CAM is limited to SBP test results and sometimes to FDP results in clays. The results from CAM depend on the model used. The minimum number of parameters will be three, representing the in situ stress, stiffness and strength. A number of combinations of those three parameters will produce a fit therefore there must be some input during the iterative process to ensure the parameters obtained are compatible with one another and known soil behaviour.

10 DESIGN BASED ON RESULTS OF PRESSUREMETER TESTS

The pressuremeter test can be used in design to help solve a number of geotechnical engineering problems. The relevance of pressuremeter tests to geotechnical problems is given in Table 11. The displacements in the ground beneath a vertically loaded shallow foundation are similar to those created by the expansion of a spherical cavity in that the vectors of displacement radiate from the foundation. The variation in increase in pressure with distance from the foundation is similar to that

Figure 18 The derivation of angle of shearing resistance from SBP tests in sands

Figure 19 The time factor for 50% dissipation of pore pressure around an expanded cavity in clay (after Clarke et al, 1979)
### Table 10: Soil properties obtained from pressuremeter tests

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Soft</th>
<th>Stiff</th>
<th>Loose</th>
<th>Dense</th>
<th>Gravel</th>
<th>Weak</th>
<th>Strong</th>
</tr>
</thead>
<tbody>
<tr>
<td>Probe</td>
<td>PBP</td>
<td>SBP</td>
<td>FDP</td>
<td>PBP</td>
<td>SBP</td>
<td>FDP</td>
<td>PBP</td>
</tr>
<tr>
<td>$\sigma_h$</td>
<td>A</td>
<td>CE</td>
<td>C</td>
<td>A</td>
<td>CE</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>$q_d$</td>
<td>BE</td>
<td>A</td>
<td>BE</td>
<td>A</td>
<td>BE</td>
<td>CE</td>
<td>B</td>
</tr>
<tr>
<td>$e'$</td>
<td>B</td>
<td>B</td>
<td>CE</td>
<td>A</td>
<td>CE</td>
<td>CE</td>
<td>A</td>
</tr>
<tr>
<td>$G_t$</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>$G_{ur}$</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>C</td>
<td>N</td>
</tr>
<tr>
<td>$P_t$</td>
<td>BE</td>
<td>B</td>
<td>BE</td>
<td>B</td>
<td>BE</td>
<td>CE</td>
<td>B</td>
</tr>
<tr>
<td>$e_h$</td>
<td>B</td>
<td>A</td>
<td>A</td>
<td>B</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
</tbody>
</table>

A - excellent; B - good; C - possible; N - not possible; E - empirical

### Table 11: Geotechnical problems in which pressuremeter test results have been used

<table>
<thead>
<tr>
<th>Problem</th>
<th>PBP</th>
<th>SBP</th>
<th>FDP</th>
</tr>
</thead>
<tbody>
<tr>
<td>shallow foundations</td>
<td>A</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td>deep foundations</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>retaining walls</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>slope stability</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>embankments</td>
<td>C</td>
<td>C</td>
<td>C</td>
</tr>
<tr>
<td>excavation</td>
<td>N</td>
<td>C</td>
<td>N</td>
</tr>
</tbody>
</table>

A - excellent; B - good; C - possible; N - not possible
Pressurerometers have also been used to calibrate instruments such as total pressure cells, determine changes in properties following ground improvements and to measure changes in the ground during construction. The main use of pressurerometer testing, however, is to obtain a ground response curve to produce design parameters.

11 THE DIRECT METHOD OF DESIGN

The direct method of design was originally developed by Ménard and is continually being updated as improved methods of analysis and performance data become available. The method is based on theories and numerical analyses of cavity expansion modified to take into account actual performance of structures. The most comprehensive database of foundation performance linked to pressurerometer testing is maintained by Laboratoire Central des Ponts et Chaussées who also produce design guidelines (LCPC SETRA, 1983). These rules have now been updated and incorporated into the French Ministère de L'Equipement, du Logement et des Transports, Fascicule 6.2, Titre V du CCTG (MELT, 1993) which are explained by Gakónin and Frank (1995).

The rules have been applied to SBP test results by comparing the SBP test curve with a PBP test curve (Baguelin, 1982). This approach has not been well developed because SBP test results tends to be used to obtain ground properties directly. The direct method has also been applied using results of FDP tests. It is likely that this will continue since the interpretation of these tests will probably be empirical because of the major changes to ground properties due to installation.

12 THE MÉNARD METHOD

The Ménard method is based on results of volume displacement type tri-cell PBP tests carried out in accordance with a standard specification. It is assumed that the ultimate bearing capacity of a shallow foundation is related to the Ménard limit pressure, \( p_{m} \), and the settlement to the pressurerometer modulus, \( E_{m} \). A pressurerometer bearing capacity factor, \( k \), used to relate the ultimate bearing capacity to the Ménard limit pressure, depends on the ground conditions, depth and shape of foundation and method of construction. The value of \( k \) varies with foundation depth though, below a critical depth, it is
Figure 21 The bearing capacity factor, k, for shallow foundations (see Table 8 for soil categories)

constant. The method also applies to the base capacity of deep foundations though, in that case, there is additional capacity due to the friction between the sides of the foundation and the ground.

The ground response obtained directly from a pressuremeter test can also be used to predict settlement of foundations. The calculated settlement is a function of the pressuremeter modulus, the increase in pressure due to the foundation loading, and a shape factor which is a function of the size of the foundation and ground conditions.

A number of pressuremeter tests are carried out in one borehole to give a profile of results. An average net limit pressure and pressuremeter modulus are determined over the zone of influence of the foundation to take into account the variation in properties with depth. Further details of the methods are given by Baguelin et al. (1978), Briaud (1992) and Clarke (1995).

Figure 22 A comparison between measured settlement and that predicted by the Ménard method (after Baguelin et al., 1978)

12.1 Shallow Foundations

The ultimate bearing capacity, $q_u$, for a vertically loaded foundation can be given as:

$$q_u = k (p_{\text{red}} - \sigma_v) + \sigma_v$$

where $p_{\text{red}}$ is an average net Ménard limit pressure from test results within 1.5B of the formation level of a foundation of width B, $\sigma_v$ the total horizontal stress and $\sigma_v$ the total vertical stress. The factor $k$ can vary from 0.8 for foundations at the surface to 1.7 to 2.2 for foundations at depth in sand and gravel as shown in Figure 21. This equation is further modified if the foundation is subject to an inclined load or near an excavation or slope.

The Ménard limit pressure, $p_{\text{red}}$, is obtained from volume displacement type PBP tests carried out in accordance with the Ménard specification. It is also possible to estimate the shear strength of the soil from these tests and use the results in bearing capacity formulae such as those proposed by Brinch Hansen. The two methods of predicting ultimate bearing capacity generally give similar results (Briaud, 1992).

The settlement of a shallow foundation is a function of the increase in isotropic stress and deviatoric stress and the isotropic and deviatoric stiffnesses. The pressuremeter modulus is a function of these stiffnesses. Under a rigid footing (Ménard and Rousseau, 1962) the settlement, $w$, is given as:
Figure 23 Values of unit skin friction derived from the Ménard limit pressure (after MELT, 1993)

\[ w = \frac{2qB_e \left( \frac{B}{B_e} \right)^2}{9E_e} \]

where \( B_e \) is a reference dimension, \( B \) is the breadth or diameter, \( q \) the increase in stress, and \( \lambda_0 \), \( \lambda \), and \( \lambda_e \) are coefficients.

A study of twenty-six shallow foundations including strip, pad and raft foundations founded in a variety of soils and weak rocks shows that the Ménard method predicted the observed settlement reasonably well (Figure 22). This method predicts the long term settlement therefore consolidation and creep movements are included. Note that in the Ménard method the derivation of the pressuremeter modulus is based on the assumption that \( v = 0.33 \). The prediction of long term settlements of foundations on clay soils may be improved possibly by taking into account a different Poisson's ratio for different soils.

12.2 Axially Loaded Piles

The capacity of axially loaded piles is a function of the end bearing capacity and friction between the pile and the ground. The end bearing capacity can be estimated from Equation 1 taking values of \( k \), which vary between 1.0 and 4.2, from Table 8. The bearing capacity factor is adjusted if the ground beneath the pile is layered or if the penetration of the pile into the bearing stratum is insufficient to mobilise the full capacity.
which the Ménard rules developed for AFNOR (1988) are based.

The settlement of axially loaded piles can be estimated using a load transfer method (Frank and Zhao, 1982) which is based on a numerical method assuming bi-linear elastic behaviour. Predictions tend to agree with observations (Frank et al., 1991) and details are given by LCPC and others (Briaud, 1992, and Clarke, 1995).

12.3 Horizontally Loaded Piles

The direction of loading in a pressuremeter test produces similar deformation patterns to an element adjacent to a horizontally loaded pile. The resistance to deflection, $p_h$, can be estimated from the modulus of subgrade reaction, $k_s$, which is a function of the pressuremeter modulus, pile diameter and ground conditions (Gambin, 1979).

$$ p_h = k_s y $$

where $y$ is the deflection, $p_h$ is equal to 0.5 $p_u$ at the ground surface and increases to $p_u$ at a critical depth of 2B in clays and 4B in sands where B is the pile diameter. The modulus of subgrade reaction can be estimated using:

$$ \frac{1}{k_s} = \frac{2B_s}{9E_{so}} \left( \frac{2B}{B_s} \right)^{0.67} + \frac{\alpha B}{6E_{so}} $$

where $\alpha$ is a coefficient dependent on ground type.

A number of other empirical methods have been developed to predict the capacity of horizontally loaded piles both from PB and PDP tests (Suaya et al., 1982, and Briaud et al., 1985) and FDP tests (Robertson et al., 1986, and Meyerhoff and Santry, 1987). The methods are based on either the modulus of subgrade reaction or a ground response curve. In the former case the modulus of subgrade reaction varies with depth to take into account increasing deflection near the surface. Both methods use the complete pressuremeter curve and show that predictions agree with measurements in the short term based on a limited number of case histories.

13 THE INDIRECT METHOD OF DESIGN

The indirect method of design uses the ground properties of in situ stress, stiffness and strength in design rules or numerical methods. Table 10 lists the parameters that can be obtained from pressuremeter tests and the quality of those parameters.

Many design rules were developed from a theoretical interpretation of foundation performance modified to take into account results of a particular test and actual performance. If pressuremeter test results are to be used in those design rules then the results must be similar to results from the tests on which the design methods were based. If the pressuremeter results are similar then they can be used in the design rules, if they are not then either a correction factor has to be applied or the design rules have to be modified. The former method is commonly applied to the interpretation of in situ tests (for example using cone factors based on correlations between cone resistance and undrained shear strength from triaxial tests).

Comparisons between results of pressuremeter tests and results of other tests are made

1. to justify the method of interpreting the pressuremeter test curve,
2. to show that the quality of the data is acceptable and
3. to demonstrate that the pressuremeter test results are similar so that they can be used in the chosen design rules.

Pressuremeter test results may be different from results of other tests since the rate of loading, stress path and drainage conditions are different.

13.1 Total Horizontal Stress

It is impossible to measure the in situ horizontal stress since any form of intrusive instrument will change the in situ stresses. Careful installation, however, can reduce any installation disturbance to a minimum. The SPB is the only in situ intrusive testing device designed to minimise disturbance to the ground prior to testing. The stress acting on the probe should equal the total in situ stress though it may not be a principal stress because of shear between the probe and the ground (Clarke and Wroth, 1984).

The selection of total horizontal stress from a SPB curve is subjective. The ground response is a function of the in situ stress; therefore the results of the test can be used in a framework based on theories of soil and rock mechanics to assess the quality of the selected value of in situ stress. Alternatively comparisons can be made with results of
Figure 25 A comparison between undrained moduli of elasticity derived from average secant shear moduli taken from unload-reload cycles in SBP tests with laboratory tests and values derived from observations of structures (Clarke, 1993).

interpretation of other tests or geological history. Examples are given by Jeffreys et al. (1987), Clarke et al. (1990), Benoit and Lutengger (1993) and Clarke (1993).

13.2 Shear Moduli

One of the prime purposes of carrying out many pressuremeter tests is to determine the stiffness of the ground. The initial slope of any test curve is a function of the properties of the ground changed by the installation process. Theoretically, the initial slope of a SBP test represents the stiffness of undisturbed ground but in practice there is some disturbance which, though small (less than 0.2% cavity strain), affects the initial slope. The shear modulus is, therefore, usually taken from an unload-reload cycle. An exception may be when the initial modulus from a PPB test in rock is used to represent the mass modulus.

Shear moduli vary with the effective stress at, and the strain range over which they are measured. The scatter in many published profiles of shear modulus against depth is due to the fact that no account was taken of the stress level and strain range when quoting the results. Figure 25 shows such a profile from tests in London Clay in which the shear modulus has been converted to undrained modulus of elasticity. The pressuremeter results are scattered but they do fall within the zone of moduli back figured from observations of structures. Figure 26 shows tests in the same soil but in this case the secant moduli were calculated over a fixed strain range. There is very little scatter and, if the correct strain range is chosen, the results conform with those back figured from observations of performance of full scale structures. This suggests that shear moduli from unload-reload cycles can be used directly in predictions of settlement especially if the relevant strain range is chosen.

Numerical techniques such as the finite element method permit the use of non-linear stiffness profiles such as those shown in Figure 27. The secant shear modulus from an unload-reload cycle is converted to an elemental modulus which, for London Clay, is similar to that from a triaxial test. Note that the stiffness is normalised with respect to the effective stress at the start of the test. In clays the effective stress remains essentially constant once yield occurs.
Figure 27 Non linear stiffness profiles of London Clay taken from unload-reload cycles in SBP tests and triaxial tests (after Hight et al, 1993)

This is not the case in sands where the effective stress increases as the membrane expands. Reducing the shear modulus to an elemental secant modulus at the in situ stress and normalising it with respect to the maximum modulus shows that pressuremeter test results in sands conform with a theoretical variation of modulus with strain (Bellotti et al, 1989).

13.3 Undrained Shear Strength

Pressuremeter tests are rarely carried out exclusively to determine the strength of a clay since there are cheaper and easier alternative methods. Nevertheless undrained strengths are routinely obtained from pressuremeter tests in clays. Generally, the undrained strengths from pressuremeter tests are similar to those from triaxial tests on stiff clay and greater than those from triaxial tests on soft clay. The strength depends on the strain range over which it is measured, the method of interpretation and the type of probe. All methods of interpretation give the same strength if the latter part of a SBP curve is used. Strengths from PYP tests are derived using empirical correlations (e.g. cons). The strength from FDP tests, taken from the unloading curve, may not be equal to that from the latter part of the loading curve of an SBP test. Limited results (Howshary and Nutt, 1993) suggest that, as expected, the shear strength obtained in unloading differs to that on loading.

13.4 Angle of Shearing Resistance

Empirical correlations are used to evaluate angles of shearing resistance from PYP and FDP tests. The results are not consistent and should be treated with caution. However, they may improve as the databases are extended.

The interpretation of SBP tests in sands often produces scatter in a profile of angle of shearing resistance with depth since the values are very sensitive to the choice of reference datum used. Only results from good quality SBP tests should be used. The limited evidence suggests that angles of shearing resistance from SBP tests, though high, produce acceptable predictions of foundation performance.

13.5 Applications of the Indirect Method of Design

The indirect design methods based on pressuremeter results are essentially similar to methods based on soil parameters from other tests, triaxial tests for example. Unlike the direct approach it is unnecessary to develop a database to enhance empirical correlations. Though fundamental ground properties from pressuremeter tests have been used in design very few case studies have been published. Clarke and Wroth (1984), Thompson and Leach (1991) and Hight et al (1993) give examples of case studies of retaining walls, tunnels and structures in the UK.

14. CHOOSING A PRESSUREMETER

Pressuremeters are used to obtain:

i) design parameters directly (Minard test),
ii) ground properties,
iii) shear modulus,
iv) non linear stiffness profiles,
v) total horizontal stress,
vii) and either undrained shear strength of clays or angle of shearing resistance of sands.
The type of geotechnical problem in which pressurermeter test results can be used is indicated in Table 10. A form of pressurermeter can be used in any ground condition but not all pressurermeters can be used in all ground conditions. Thus in deciding which pressurermeter to use an engineer must take into account the likely ground conditions and the parameters required. Figure 28 is a chart showing the criteria followed when choosing a pressurermeter.

All other controlling factors are usually left up to the operator.

Calibrations and test procedures are specified though it is usual to use accepted procedures such as those given by APNOR (1991), ASTM (1994) and Clarke and Smith (1992). Clarke (1995) gives a comprehensive specification for all pressurermeters in any ground condition.

15 SPECIFICATION

Different brands of the same type of pressurermeter from different manufacturers may give similar results in the same ground conditions. It is better to specify the type of pressurermeter and the pressure and displacement capacity, rather than a particular version, otherwise the specification may be too restrictive which could lead to an expensive ground investigation.

The method of installation depends upon the ground conditions, the probe and drilling equipment. Correct installation is essential if test results are to be of any use. The experienced operator should install the probe in such a way that there is minimum or repeatable disturbance to the ground. It is usual to specify the maximum size and method of creating a prebored probe and the speed of pushing the FDAP.

16 BILL OF QUANTITIES

Pressurermeter testing is usually itemised to cover mobilisation, calibrations, installation and testing. Table 12 gives the main items for a Bill of Quantities. Note that creating a test pocket is in addition to the creation of the borehole since they are two distinctly different operations, except in MPF operations.

Not all tests are completed according to the specification. This does not imply they have failed since the ground conditions dictate the test. Overburden and discontinuities can lead to membrane bursts; the pressure required to expand the membrane may exceed the capacity of the probe because of the stiffness of the ground. In all these cases the 'failure' is due to ground conditions and not the fault of the operator. If failure is due to an operator's lack of care, for example damaging the membrane on lowering the probe into the borehole,
<table>
<thead>
<tr>
<th>Item</th>
<th>Item Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Establishment on site of the drilling rig and crew</td>
<td>item</td>
</tr>
<tr>
<td>2</td>
<td>Establishment on site of pressuremeter equipment and operator</td>
<td>item</td>
</tr>
<tr>
<td>3</td>
<td>Moving of the drilling rig to the site of each exploratory borehole</td>
<td>no</td>
</tr>
<tr>
<td>4</td>
<td>Moving of pressuremeter equipment to the site of each exploratory borehole</td>
<td>no</td>
</tr>
<tr>
<td>5</td>
<td>Open hole drilling/boring or rotary core drilling to advance the borehole to a test location</td>
<td>m</td>
</tr>
<tr>
<td>6</td>
<td>Manufacturer's test for straightness and sensitivity</td>
<td>item</td>
</tr>
<tr>
<td>7</td>
<td>Calibration of pressuremeter prior to establishment on site and following the completion of testing on site</td>
<td>item</td>
</tr>
<tr>
<td>8</td>
<td>Carry out additional calibrations as instructed by the Engineer</td>
<td>item</td>
</tr>
<tr>
<td>9</td>
<td>Provide copies of the probe calibration register</td>
<td>hr</td>
</tr>
<tr>
<td>10</td>
<td>Standing time of pressuremeter equipment</td>
<td>m</td>
</tr>
<tr>
<td>11</td>
<td>Backfilling to pressuremeter borehole</td>
<td>per test</td>
</tr>
<tr>
<td>12</td>
<td>Additional data processing, analysis and reporting as specified</td>
<td>item</td>
</tr>
<tr>
<td>13</td>
<td>Prebored Pressuremeter</td>
<td>m</td>
</tr>
<tr>
<td>14</td>
<td>Drilling as specified to form a test pocket</td>
<td>m</td>
</tr>
<tr>
<td>15</td>
<td>Provide orientation data for pressuremeter tests</td>
<td>m</td>
</tr>
<tr>
<td>16</td>
<td>Expansion test of duration not exceeding 1.5 hrs.</td>
<td>test</td>
</tr>
<tr>
<td>17</td>
<td>Extra over item 13 for expansion tests in excess of 1.5 hrs.</td>
<td>hr</td>
</tr>
<tr>
<td>18</td>
<td>Extra over item 13 for carrying out additional unload-reload cycles as specified</td>
<td>per cycle</td>
</tr>
<tr>
<td>19</td>
<td>Self-drilling to form test pocket</td>
<td>m</td>
</tr>
<tr>
<td>20</td>
<td>Provide orientation data for pressuremeter tests</td>
<td>no</td>
</tr>
<tr>
<td>21</td>
<td>Provide pore pressure cell data during installation</td>
<td>test</td>
</tr>
<tr>
<td>22</td>
<td>Expansion test of duration not exceeding 1.5 hrs including 30 min standing</td>
<td>test</td>
</tr>
<tr>
<td>23</td>
<td>Extra over item 13 for expansion tests in excess of 1.5 hrs.</td>
<td>hr</td>
</tr>
<tr>
<td>24</td>
<td>Extra over item 13 for carrying out additional unload-reload cycles as specified</td>
<td>per cycle</td>
</tr>
<tr>
<td>25</td>
<td>Establishment on site of the cone truck and crew</td>
<td>item</td>
</tr>
<tr>
<td>26</td>
<td>Moving of the cone truck to the site of each exploratory borehole</td>
<td>no</td>
</tr>
<tr>
<td>27</td>
<td>Pushing the cone to form test pocket</td>
<td>m</td>
</tr>
<tr>
<td>28</td>
<td>Provide orientation data for pressuremeter tests</td>
<td>no</td>
</tr>
<tr>
<td>29</td>
<td>Expansion test of duration not exceeding 1.5 hrs.</td>
<td>test</td>
</tr>
<tr>
<td>30</td>
<td>Extra over item 12 for expansion tests in excess of 1.5 hrs.</td>
<td>hr</td>
</tr>
<tr>
<td>31</td>
<td>Extra over item 12 for carrying out additional unload-reload cycles as specified</td>
<td>per cycle</td>
</tr>
<tr>
<td>32</td>
<td>Analysis of cone data</td>
<td>per test</td>
</tr>
</tbody>
</table>

Rates for all * items usually given for depths 0 to 10m, 10 to 20m etc.
then the operator should bear the cost of the time taken to retrieve the situation. This could include repairing the equipment and standing time for the drilling rig and crew. It is important to agree quantities on site while all parties involved are present.

17. SUMMARY

1. A pressuremeter is a cylindrical device with a flexible membrane which is inflated under pressure within a test pocket created specifically for the test.
2. There are three groups of pressuremeters: prebored, self-bored and pushed in.
3. A form of pressuremeter can be used in any type of ground.
4. The type of pressuremeter, method of installation, test procedure and method of interpretation as well as the ground conditions influence the parameters obtained.
5. Test can be either stress or strain controlled or some combination.
6. Tests can be interpreted using theories of cavity expansion to produce ground properties. In general simple models are used.

18 ACKNOWLEDGMENTS

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**LIST OF SYMBOLS**

- B  dimension
- b  reference dimension
- c'  cohesion
- C  coefficient of consolidation
- E  Modulus of soil
- G  modulus of soil
- Gs  initial shear modulus
- Gs  average shear modulus from an unloading curve
- Gs  average shear modulus from a reloading curve
- h  height
- h  modulus of subgrade reaction
- m  applied pressure
- M  limit pressure
- Mw  mean weight
- MWV  volumetric strain
- n  coefficient of friction
- q  angle of shearing resistance
- q  cavity strain
- λ  coefficient
- e  total horizontal stress
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