Engineering use of Piezocone data in North Sea clays
Essais Piézocone dans les argiles de la Mer du Nord

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SYNOPSIS

Cone penetration testing with measurements of pore pressure (piezocone) is now used routinely in North Sea soil investigations. The interpretation of the test results should include consideration of the position of the piezoelement and pore pressure effects on cone readings. Data from four major soil investigations show that the pore pressure parameter, Bq, correlates to some extent with over-consolidation ratio, OCR, and "total cone" factor N^, based on CAUc triaxial tests. Bq correlates well with "effective cone" factor, Nkg and "excess pore pressure" cone factor N^u, and recommendations based on these correlations are given for computing undrained shear strength, su. These correlations should be checked in other clay types. Effective stress strength parameters are interpreted with Senneset and Janbu's (1984) theory. Based on correlations to CAUc triaxial data, the authors recommend to adjust Senneset and Janbu's interpretation diagram.

INTRODUCTION

Cone penetration testing with measurements of cone tip resistance and sleeve friction has been a very important part of North Sea soil investigations since 1972. Over the years several site specific correlations between cone resistance and undrained shear strength have been developed (e.g. Lunne and Kleven, 1981).

As an example Fig. 1 shows computed Nk-factors vs depth for hard overconsolidated structured clay at Gullfaks (Lunne et al, 1983), where the water depth is 135 m. The data in Fig. 1 come from five major platform site investigations within the same area. A wide scatter in the Nk factors can be observed, although clearly Nk tends to decrease with depth.

Soil investigations carried out by Statoil over the last 5 years at the Sleipner and Gullfaks fields have resulted in continuously updated cone factors.

In the last two years, piezocone testing (where pore pressure is measured in addition to cone resistance and sleeve friction) has become standard practice in the North Sea. High quality testing is ensured through detailed equipment/procedure requirements from the oil companies and quality control schemes from the in situ testing contractors.

The measurement of pore pressure facilitates important corrections to cone resistance and sleeve friction so that results from different cone equipments can be brought to a common basis (Aas et al, 1984). Interpretation in terms of effective stresses also becomes possible (Senneset and Janbu, 1984).

This paper summarizes the experience with piezocone testing from four major soil investigations at the Sleipner and Gullfaks fields. Recommendations are given for the interpretation of piezocone tests in similar North Sea clays.

DESCRIPTION OF EQUIPMENT

A cone penetration test in the North Sea is performed either by jacking the cone into the seabed or into the bottom of a borehole. In the first case, 40-45 m penetration in soft clays and 10-20 m in dense sand is possible when using a seabed jack with a total thrust capacity of about 200 kw. The jack can also be modified to be used for "down-the-hole testing" through a drillstring at any predrilled depth, with a maximum stroke of 3 or 6 m at each test depth.

Fig. 1 Cone factor N_k versus depth, Gullfaks. su determined from CAUc triaxial compression tests.
TREATMENT OF PIEZOCONE DATA

Recent work (Robertson and Campanella, 1983 and Aas et al., 1984) has shown that the pore pressures generated during penetration of the cone influence the measured cone resistance, \( q_c \), and sleeve friction, \( f_s \). Since water pressure acts on an area immediately behind the cone, \( q_c \) should be corrected for the effect of pore pressure:

\[
q_T = q_c + k \cdot u(1-a),
\]

(1)

where \( a = F_B/F_A \) = area ratio (see Fig. 2)

\( u = \) measured pore pressure

\( k = \) correction factor (see below).

Only a jointless cone would measure \( q_T \) directly. A similar effect exists for the sleeve friction. The cones used in North Sea soil investigations have \( a \) - values ranging from 0.42 to 0.78. The above correction reduces considerably the differences in \( q_c \) and \( f_s \) measured by different cone types in the same soil. Since the pore pressure with the cones in Fig. 2 is not measured at the location where the pore pressure correction should be made, an estimate is made of \( k \), the ratio of the pore pressure immediately behind the cone and the measured pore pressure. Tests in different clay types in Norway (e.g. Aas et al., 1984) have shown an average \( k \)-value of 0.8 for both the McClelland and Fugro cones.

Surface cone penetration readings are zeroed at the seabed, and down-the-hole readings are zeroed at the bottom of the borehole. Adjusting to the same reference as surface tests, \( q_T \) from a down-the-hole test becomes:

\[
\begin{align*}
q_T &= q_c + k(u + \gamma_w h)(1-a) + \alpha \gamma_w h \\
&\approx q_c + k \cdot u(1-a) + \gamma_w h
\end{align*}
\]

(2)

where \( \gamma_w = \) unit weight of water

\( h = \) depth of borehole

It is also possible to define an "effective cone resistance":

\[
q_g = q_T - (u + \gamma_w h)k
\]

(3)

Several definitions have been suggested for the pore pressure ratio (e.g. Baligh et al., 1981 and Robertson and Campanella, 1983). In this paper the authors have chosen to use the pore pressure ratio proposed by Senneset and Janbu, 1984:

\[
B_q = \frac{\Delta u}{q_T - p_o}
\]

(4)

where \( p_o = \) total overburden pressure with reference to the seabed, \( \Delta u = \) excess pore pressure with reference to pore pressure reading immediately behind the cone tip.

SOIL CONDITIONS AT SITES INVESTIGATED

The correlations presented in this paper are taken from soil investigations at four platform locations on the Sleipner and Gullfaks fields in the North Sea:

Gullfaks A, water depth \( \sim 135 \) m:

Below the top gravel layer, the soil consists of stiff to hard overconsolidated glaciomarine silty clay. As described by Lunne et al (1983), the clay at this location has a pronounced macrofabric which considerably influences the strength measurements. Figure 3 shows the results of index tests, consolidated undrained triaxial (CAUc) and direct simple shear (DSS) tests as well as piezocone penetration tests.
Gullfaks C, water depth ~ 220 m:

As shown on the boring profile in Fig. 4, an 8 m thick layer of soft lightly overconsolidated clay underlies 2 m cover of sand. A medium dense clayey sand is found from 10 to 40 m which in turn overlies a moderately overconsolidated clay layer.

Sleipner Area 2, water depth ~ 107 m.

The upper 6m consist of silty sand with clayey sand layers. Below are found glaciomarine clay layers with varying degree of overconsolidation ratio. Figure 5 gives a soil profile at Area 2.

Sleipner Area 4, water depth 82 m:

Below 22 m of dense fine sand is a stiff to very stiff glaciomarine silty clay with varying overconsolidation ratio (See Fig. 6).

Table 1 summarizes index properties of the clay layers at these four locations. The soil investigations described above were performed from the two Norwegian soil survey vessels "Ferder" and "Bucenatour". McClelland Ltd. or Fugro B.V. performed the sampling and in-situ tests, whilst NGI carried out the laboratory tests.

**Table 1 Summary of soil classification data for clay layers used in correlations.**

<table>
<thead>
<tr>
<th>Location</th>
<th>Depth below seabed (m)</th>
<th>Water content w, %</th>
<th>Liquid limit WL, %</th>
<th>Plasticity index Ip, %</th>
<th>% clay particles &lt;2u</th>
<th>Over-consolidation ratio, OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gullfaks &quot;A&quot;</td>
<td>5-17</td>
<td>18-25</td>
<td>43-50</td>
<td>26-32</td>
<td>35-43</td>
<td>5-15</td>
</tr>
<tr>
<td>Gullfaks &quot;C&quot;</td>
<td>17-40</td>
<td>21-32</td>
<td>41-50</td>
<td>22-28</td>
<td>15-40</td>
<td>3-5</td>
</tr>
<tr>
<td>Sleipner A-4</td>
<td>6-40</td>
<td>19-23</td>
<td>36-44</td>
<td>20-25</td>
<td>22-43</td>
<td>1-3</td>
</tr>
</tbody>
</table>

**Correlations between piezocone and laboratory test results**

Several authors have suggested that the pore pressure parameter can be related to the overconsolidation ratio, OCR (e.g. Baligh et al., 1981). Robertson and Campanella (1983) postu-
lated that the relationship between pore pressure and OCR will be influenced by variations in soil plasticity and sensitivity since the excess pore pressure is a function of rigidity index, $I_r$. Generally for the same OCR, $I_r$ decreases with increasing plasticity.

Figure 7 plots the pore pressure parameter $B_q$ against the best estimate of the in situ overconsolidation ratio OCR. Generally the preconsolidation stress has been estimated from oedometer tests interpreted using Casagrande's and Janbu's constructions or from relations among undrained shear strength, $s_u$, plasticity index, $I_p$, and OCR. Admittedly there are significant uncertainties in the values of OCR determined this way. The data show that $B_q$ generally decreases with increasing OCR.

Undrained shear strength

To date the undrained shear strength, $s_u$, has been computed from the measured cone resistance, $q_c$, with the following bearing capacity equation:

$$ q_c = s_u \cdot N_k + p_0 $$

where $N_k$ is the empirical cone factor and $p_0$ is the total overburden pressure with reference to the seabed.

With the corrected cone resistance, $q_T$, the cone factor is expressed as:

$$ N_{kT} = \frac{q_T - p_0}{s_u} $$

Similarly the "effective cone resistance", $q_E$, and the excess pore pressure $\Delta u$, can be used to define cone factors:

$$ N_{kE} = \frac{q_E}{s_u} $$

and

$$ N_{\Delta u} = \frac{\Delta u}{s_u} $$

Fig. 7 Pore pressure parameter vs. overconsolidation ratio.

Fig. 8 Cone factor $N_{kT}$ vs. pore pressure parameter $B_q$

Fig. 9 Cone factor $N_{kE}$ vs. pore pressure parameter $B_q$

Fig. 10 Excess pore pressure cone factor $N_{\Delta u}$ vs. pore pressure parameter $B_q$
The $N_k$ factor has been reported to vary with overconsolidation ratio and soil type. Due to the uncertainty involved in determining OCR, the authors have explored the relationships between the various cone factors and the pore pressure parameter, $B_q$. Once a profile of in situ density has been obtained, $B_q$ can be determined directly from $\nu_c$ and $u$. For this study all cone factors have been computed based on $u$ from CAUC triaxial tests.

Figure 8 shows $N_{kT}$ plotted as a function of $B_q$. $N_{kT}$ tends to decrease with increasing $B_q$ corresponding to an increase in $N_{kT}$ with increasing OCR.

Figures 9 and 10 show that $N_{kE}$ and $N_{u}$ correlate better with $B_q$ than $N_{kT}$. To increase the range of $B_q$ values in this study, values from the Emmerstad quick clay have also been included (Aas et al., 1984). The authors believe that the relationships between $B_q$ and $N_{kE}$ and between $B_q$ and $N_{u}$ show considerable promise and should be explored further in other types of clay. The ratio $N_{u}/B_q$ equals $N_{kT}$.

At present it is recommended to use the solid lines in Figs. 9 and 10 if one requires a best estimate of $u$ or the dotted lines if one requires upper and lower bounds for $u$. It should be borne in mind that the $N_u$ values obtained by this interpretation method correspond to that obtained from a CAUC triaxial test on a sample from the same depth.

**Effective stress strength parameters**

Senneset and Janbu (1984) have proposed a bearing capacity formula in terms of effective stresses:

$$q_T - P_0 = (N_q - 1)(P_0' + a) - N_u u$$  \hspace{1cm} (9)

where $N_q = \tan^2 (45^\circ + \phi'/2)\tan(\phi' - 2\theta)$, and $\theta$ defines the size of the failure zone as shown in Fig. 12. The formula for $N_u$ is strictly valid for plane strain conditions, but Senneset and Janbu (1984) suggest no correction for a cylindrical cone. The theoretical value of $N_u$ has been reported by Senneset and Janbu (1984) to be $6\tan^2(1 + \tan \phi')$ for a penetrating cone. The cohesion, $c^t = a \cdot \tan \phi'$ ($a$ = attraction).

Introducing the cone resistance number,

$$N_m = \frac{N_q - 1}{1 + N_u B_q}$$

equation (9) becomes:

$$q_T - P_0 = N_m (P_0' + a)$$  \hspace{1cm} (10)

or

$$N_m = \frac{q_T - P_0}{P_0' + a}$$

Using $\theta = 0$ and the formula for $N_u$ given above, Senneset and Janbu (1984) presented the relationships shown with dotted lines in Fig. 12 for interpretation of effective stress strength parameters from piezcone data. First one has to estimate the attraction and compute $N_m$ and $B_q$; $\tan \phi'$ can then be found from Fig. 12.

The authors have used the Senneset and Janbu interpretation procedure to compare piezcone test results to the results of CAUC triaxial tests consolidated to in situ stresses.

Figure 11 plots $N_q$ vs. $\tan \phi'$ with $q_T$ obtained from the piezcone tests and $\tan \phi'$ and $a$ from the CAUC triaxial tests. A set of constant $B_q$ contour lines is also given. For comparison values for tests in sand ($B_q = 0$) based on results given by Lunne and Christoffersen...
The cone factors are also plotted. The solid contour lines given in Fig. 12 are meant to be an empirical updating of the curves presented by Senneset and Janbu (1984). Figure 13 illustrates how the new set of curves give a better fit between the cone readings and the pore pressure readings than Senneset and Janbu's relationships. The attraction, a, must be determined before \( N^m \) can be computed. If \( q_c \) increases linearly with OCR, the attraction can be found as proposed by Senneset et al. (1982). If this method cannot be used the authors tentatively recommend to estimate attraction from the degree of overconsolidation.

Normally consolidated and slightly overconsolidated clay, \( a = 2-10 \text{ kN/m}^2 \)

Moderately overconsolidated clay, \( a = 10-20 \text{ kN/m}^2 \)

Heavily overconsolidated clay, \( a = 20-100 \text{ kN/m}^2 \)

CONCLUSIONS

Piezocone data have been compared to laboratory test results on good quality clay samples from four major North Sea soil investigations.

The cone readings should be corrected for pore pressure effects, and the pore pressure readings have been corrected to correspond to measurements made just behind the cone.

The pore pressure parameter, \( B_q \), tends to decrease with increasing OCR, but other factors also influence this relationship. Presently only an indication of OCR can be obtained from piezocone data.

The cone factors \( N_U \) and especially \( N_E \) and \( N_AU \) (based on \( CAU \), triaxial tests) vary systematically with \( B_q \). The authors recommend that these relationships should be explored further in other clay deposits, using high quality data.

Based on the available data the authors recommend the average or the bands given in Fig. 9 to compute undrained shear strength, \( s_u \). Figure 10 can be used to check the computed \( s_u \).

The authors recommend adjusting the interpretation diagram given by Senneset and Janbu (1984) as shown in Fig. 12 to compute effective stress strength parameters.

ACKNOWLEDGEMENT

The authors are grateful to Statoil for supporting this work and for permission to publish the test data. The cooperation of the specialist contractors McClelland Ltd. and Fugro B.V. is acknowledged. The authors are also indebted to numerous NGI colleagues. Their major contribution to the data presented here is gratefully recognized.

REFERENCES