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# CPTU correlations for clays

## *Corrélations CPTU pour les argiles*

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ABSTRACT
New correlations between various CPTU factors, undrained shear strength and overconsolidation ratio have been developed for soft to medium stiff clays. The correlations are based on comparing the CPTU results against undrained traxial compression strength and preconsolidation pressure determined on block samples of very high quality taken with the Sherbrooke 250 mm block sampler. The data base covers samples from 17 different sites ranging from soft to medium stiff clays with plasticity index from 10 to 50 % and sensitivity from 3 to about 200. The data clearly show that the measured excess pore pressure gives the best and most consistent correlation to the measured undrained strength. The cone resistance shows fairly large scatter, which may partly be an equipment/measurement problem and partly reflect that the cone resistance is a more complex parameter than the pore pressure response and which depends on more subtle clay characteristics. The cone factors also depend on the clay sensitivity and the plasticity index of the clays tested and the type of cone used.

RÉSUMÉ
De nouvelles corrélations entre les facteurs du piézocône CPTU et la résistance au cisaillement non-drainée et le rapport de surconsolidation sont proposées pour les argiles molles à moyennement raides. Ces corrélations ont été obtenues en comparant les résultats CPTU avec la résistance au cisaillement non-drainée en compression triaxiale et la pression de surconsolidation mesurées sur des blocs de haute qualité obtenus avec l'échantillonneur bloc 250 mm de l'Université de Sherbrooke. La base de données compte 17 différents sites d'argile, avec indice de plasticité entre 10 et 50 % et sensibilité entre 3 et 200. Les résultats montrent clairement que la pression interstitielle mesurée par le CPTU présente la corrélation la plus consistante avec la résistance au cisaillement non-drainée. La résistance en pointe du cône apparaît plutôt variable, en partie dû à l'équipement et la mesure elle-même, en partie dû au fait que la résistance en pointe est un paramètre plus complexe que la réponse des pressions interstitielles, et dépend de caractéristiques plus subtiles de l'argile. Les facteurs du piézocône dépendent aussi de la sensibilité et plasticité de l'argile.

## 1 INTRODUCTION

The cone penetrometer with measurement of pore pressure, called CPTU test, was developed in the late 1970's. The first publication on the CPTU was by Roy et al (1980). The CPTU test was initially a research tool which gradually came into commercial use, and first in connection with site investigations for offshore structures in the early 1980's. In Norwegian onshore commercial practice it took until around 1990 before it found its place. It has by now (2004) gradually become the most common in-situ testing tool for determination of undrained strength of clay deposits in Norwegian site investigations, and has to a large extent replaced in-situ vane borings. This is mainly a result of the introduction of the wireless cone penetrometers manufactured by the Swedish companies ENVI (stores data in memory in the cone) and Geotech (transmits signals acoustically), which have greatly enhanced the efficiency of operations. In this context it may be mentioned that in Norway the CPTU-tests are almost exclusively carried out with one-manned advanced multi-purpose rigs. The production rate typically lies in the range 80-100 m pr. day.

The results of CPTU tests can in principal be used to derive a number of soil parameters related to stress history, shear strength and moduli values. Dissipation tests can also be used to assess the coefficient of consolidation and permeability. The derivation of such parameters in clays is generally based on semi-empirical correlations against parameters established through conventional soil sampling and laboratory tests. This paper deals only with CPTU tests in relatively soft to medium stiff clay deposits and the derivation of undrained shear strength and pre-consolidation stress based on comparisons against parameters determined exclusively by laboratory tests on very high quality block samples.

## 2 PROPERTIES OF CLAYS TESTED

In order to enable development of reliable correlations between CPTU-results and real in-situ soil parameters it is vital that such parameters are obtained by laboratory tests on high quality undisturbed soil specimens. Experience shows that it can be difficult to obtain samples of sufficiently good and indisputable quality with conventional commercial piston sampling, or standard Shelby tube sampling. The sample quality normally reduces with decreasing plasticity index and increasing sample depth and clay sensitivity (Karlsrud, 1991; Lunne et al 1997a). The effect of sample disturbance on measured undrained strength, pre-consolidation stress and stress-strain relations depend on the clay type. For many years the Norwegian Geotechnical Institute (NGI) has used the change in volume a sample undergoes during re-consolidation to in-situ effective stresses as an indicator of sample disturbance (Andresen and Kolstad, 1979). Lunne et al (1997a) proposed a modified scale for sample quality equal to the change in void ratio normalised by the initial void ratio.

To obtain high quality samples for establishment of more reliable reference parameters for CPTU correlations, NGI has since 1983 made use of the special Canadian Sherbrooke block sampler, Lefebvre and Poulin (1979), at several sites in Norway, (Lacasse et al., 1985), also in parallel with CPTU testing at the same sites (Karlsrud et al.,1996). According to the classification proposed by Lunne et al (1997a), the samples tested and included in this correlation study can be classified as very good to excellent.

The undrained strength determined by triaxial compression tests on samples consolidated anisotropcally to the present in-situ effective stresses, $s_u^{CAUC}$ (hereafter for simplicity defined as $s_{uc}$), has been chosen as the main parameter for comparison against the CPTU results, combined with the pre-consolidation stress, $p_c$' determined by oedometer tests. All triaxial samples were sheared at a rate of about 0.6% pr. hour. In contrast with what can be the case for disturbed samples, all triaxial compression tests on the block samples showed a pronounced peak at failure. The axial strain at failure tended to increase with OCR from typically 0.3 to 1.2% for OCR less than 2 to 0.3 to 3.0% for OCR of 5. Figure 1 show an example of stress-strain curve for a block sample compared to piston samples.

The oedometer tests were in almost all cases constant rate of strain (CRS) tests, loaded at an axial strain rate of 0.5-1% pr.hour. For all samples tested the oedometer curves gave a very clear definition of the stress level at which the pre-consolidation stress, $p_c$' (or yield stress as defined by some, e.g. Burland, 1990), as illustrated by a typical example in Figure 2. The precise pre-consolidation stress was generally defined as an average from the Casagrande (1936) method and the Becker et al (1987) method, but these methods gave nearly identical results. It may be noted that the pre-consolidation pressure depends on the rate of straining, similar to the undrained strength. Thus the CRS tests used in this study normally give 5 to 20 % larger pre-consolidation pressures than conventional 24 hour incremental oedometer tests (Leroueil et al., 1983; Lunne, 2002).

The combination of CPTU-testing and block sampling has so far been carried out by NGI at 16 different locations in Norway. Table 1 presents typical range of index properties at the different locations. NGI has also been involved with block sampling and laboratory testing at the Bothkennar test site in the UK, which has also been included in this study. At each location block samples were taken from 2 to 5 different levels, giving a total of 58 tests to compare against CPTU results at the same locations and levels.

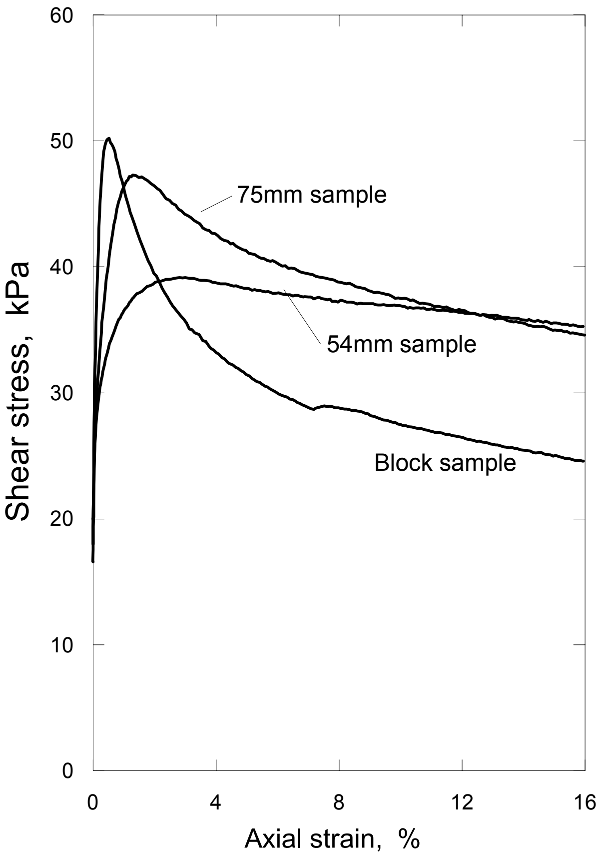


Figure 1. Example of CAUC triaxial test result on block sample compared to piston samples. Lierstranda z=12.3 m (from lunne et al 2002)

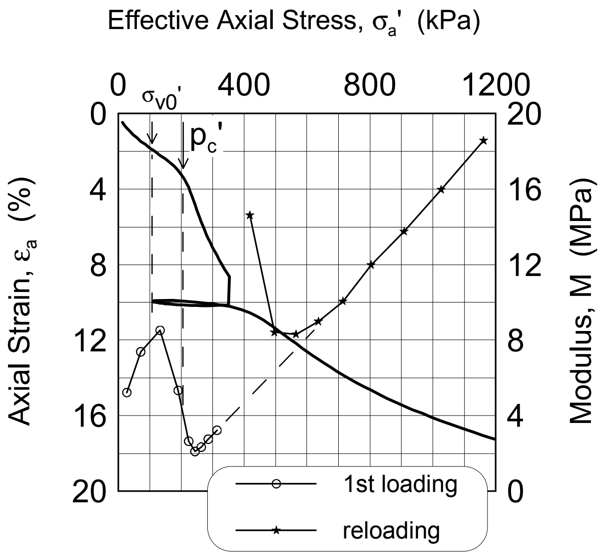


Figure 2. Example of oedometer test result. Lierstranda, z=12.3 m

Table 1: Index properties of clays tested

| Location | Depth interval, m | Water content, % | $I_p$, % | Particles < 2 μm | $S_t$ |
|---|---|---|---|---|---|
| Leira | 7.2–15.9 | 30–34 | 12-14 | 36–49 | 7-10 |
| Hvalsdalen 1 | 7.2-15.2 | 31-39 | 9-17 | 44-49 | 5-240 |
| Hvalsdalen 2 | 7.2-12.4 | 36-38 | 12 | 40-46 | 15-20 |
| Eidsvoll 1 | 5.7–14.7 | 25–33 | 13–19 | 37–48 | 2–5 |
| Eidsvoll 2 | 10.4-13 | 25-26 | 13-14 | 38-42 | 2-4 |
| Lierstranda | 6.1–22.4 | 33–40 | 14–20 | 36–44 | 8–12 |
| Bothkennar,UK | 6.0–8.3 | 69–72 | 42–53 | 17–35 | 8–13 |
| Emmerstad | 4.1–5.9 | 37–44 | 7–11 | 40 | 77 |
| Onsøy 1 | 3.1–9.1 | 59–72 | 28–44 | 21–35 | 7–12 |
| Glava | 5.6-6 | 33 | 17-18 | 29-34 | 7-10 |
| Daneviks gt. | 7.9-16.6 | 36-53 | 18-29 | 36-41 | 4-8 |
| Nykirke | 6.0-10.1 | 25-37 | 4-9 | 27-35 | 100-180 |
| Kvenild | 7.1-8.5 | 31-37 | 10-14 | 31-47 | 22-63 |
| Onsøy 2 | 7.2-14.7 | 63-64 | 36-42 | 49-65 | --- |
| Buvika 1 | 7.1-15.1 | 29-31 | 6-7 | 29-30 | 55-160 |
| Buvika 2 | 7.6-17.1 | 31-33 | 8-13 | 28-33 | 10-105 |
| Kløfta-Nybakk | 6.1-18.2 | 32-39 | 8-18 | 33-46 | 7-135 |

It should be noted that the sensitivities in Table 1 were determined from fall cone tests. The very high sensitivity of some samples is due to leaching of the original marine clay deposits. The sensitivity varies considerably with depth at some locations. Note also that the plasticity index of clays is influenced by the sensitivity. Normally or lightly over consolidated leached clays with high sensitivity will in un-leached state have a plasticity index which is typically a factor of 1.5 to 2.0 larger than the leached high sensitive clay.

The mostly Norwegian marine clays tested were deposited in the sea after the last glaciation period about 10,000 years ago. The clay mineral content of these Norwegian clays is dominated by Chlorite and Illite/Muskovite (e.g. Kenney,1967), and they have a low organic content (less than 2%). At the Bothkennar site in the UK the clay was deposited in estuarine conditions, Hight et al. (2003), and the clay has a different mineralogy compared to the Norwegian clays.

The clays tested showed undrained strength, $s_{uc}$, ranging from 15 to 150 kPa, and overconsolidation ratio, OCR, ranging from 1.2 to 6.3. The highest OCR-values were due to removal of overburden, otherwise it is generally due to secondary creep and/or chemical weathering.

Figure 3 presents the undrained triaxial compression strength determined on the various block samples in relation to the pre-consolidation stress in a normalised form of $s_{uc}/\sigma_{v0}'$ versus overconsolidation ratio OCR = $p_c'/\sigma_{v0}'$, where $\sigma_{v0}'$ is the in-situ vertical effective overburden stress. The measured data in Figure 3 are also compared to a range of correlation functions based on the SHANSHEP concept (Ladd et al, 1977), and defined by the equation:

$$s_{uc}/\sigma_{v0}' = \alpha \cdot OCR^m \qquad (1)$$

where $\alpha = s_{uc}/\sigma_{v0}'$ for OCR=1.0, corresponding to a young truly normally consolidated clay which has not had the opportunity to develop any apparent pre-consolidation pressures due to secondary consolidation (e.g. Bjerrum, 1972).

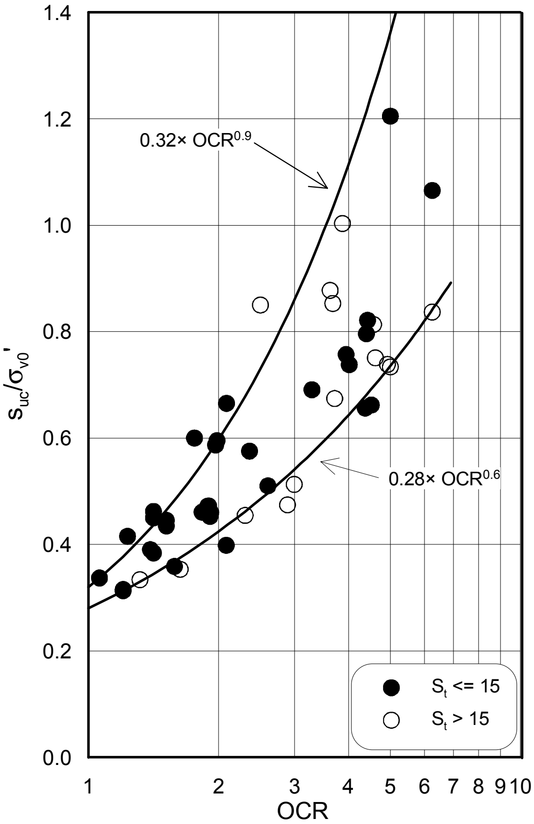


Figure 3. Normalized CAUC strength values, $s_{uc}/\sigma_{v0}'$, for block samples in relation to OCR

From Figure 3 it is apparent that these natural clays do not show such a unique relationship between undrained strength and OCR as has been indicated by testing clays that have been pre-consolidated artificially in the laboratory (e.g Ladd et al 1977). The reason may be that soil structure and possible local chemical bonding or cementation effects plays a role in-situ, which is lost when a sample is artificially pre-consolidated in the laboratory.

On many of the block samples the undrained strength has also been determined by triaxial extension tests (CAUE), $s_{ue}$, and direct simple shear tests (DSS), $s_{uDSS}$. Figure 4 compares these strengths to the triaxial compression strength. The data suggest somewhat larger anisotropy for the clays with high sensitivity (here taken as $S_t$>15), and that the anisotropy decreases with increasing plasticity index, $I_p$. No clear dependence of the anisotropy on the overconsolidation ratio has been observed.

## 3 CPTU RESULTS AND CORRELATIONS

### 3.1 *Definitions and CPTU factors considered*

In the years passed since the CPTU test was first introduced, different cone factors have been used to relate the measured values of cone resistance and excess pore pressure generated during penetration of the probe to the undrained strength of clay deposits, e.g. Lunne et al. (1997b).

In terms of measured pore pressures in CPTU tests the undrained strength is generally correlated to the pore pressure factor

$$N_{\Delta u} = \frac{u_2 - u_0}{s_u} \qquad (2)$$

where $u_2$ = measured pore pressure at the location just behind the neck of the cone and $u_0$= in-situ pore pressure. If a cone with pore pressure measurement in the tip is used, it may be possible to make a fair estimate of the $u_2$ pore pressure by application of a correction factor as given in Lunne et al. (1997b).

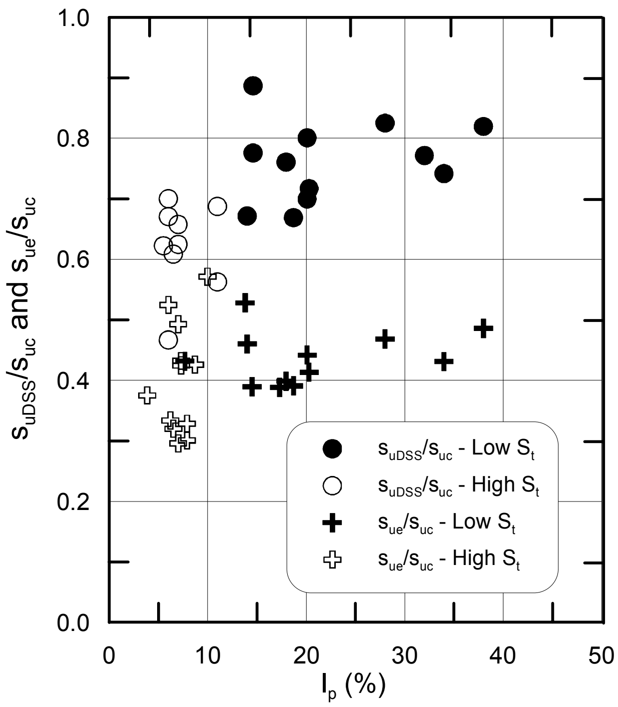


Figure 4. Measured anisotropic strength ratios $s_{ue}/s_{uc}$ and $s_{uDSS}/s_{uc}$ for the block samples

In relation to measured cone resistance it has become common practice to relate the undrained strength to the corrected cone resisitance, $q_t$, rather than the directly measured tip resistance, $q_c$, through the normalised expression:

$$N_{kt} = \frac{q_t - \sigma_{vo}}{s_u} \qquad (3)$$

where $\sigma_{v0}$ = Total vertical overburden pressure

$q_t = q_c + u_2(1\text{-}a)$

and a= area ratio of the cone related to the area of the central part of the cone as compared to the gross area. The area ratio is determined by calibration tests in the laboratory as described in Lunne et al. (1997b). This area correction reduces or eliminates some of the observed difference in cone resistance obtained by using cones from different manufacturers.

In previous studies it has also been attempted to relate the undrained strength to a combination of the measured tip resistance and pore pressure through the expression

$$N_{ke} = \frac{q_t - u_2}{s_u} \qquad (4)$$

In some previous studies (Lunne et al.,1985; Karlsrud et al., 1996) correlation plots between the corrected tip resistance factor $N_{kt}$ on one axis against the pore pressure factor $B_q$ on the other axis have been shown. $B_q$ is defined as:

$$B_q = \frac{(u_2 - u_o)}{(q_t - \sigma_{vo})} \qquad (5)$$

$B_q$ is also equal to $N_{\Delta u}/N_{kt}$. A plot of $N_{kt}$ against $B_q = N_{\Delta u}/N_{kt}$ is therefore misleading in the sense that it does not reflect the cone resistance factor $N_{kt}$ at all because $N_{kt}$ appears as a parameter on both axis. Such a plot actually only reflects how measured excess pore pressure vary with undrained strength. Correlation plots of $N_{kt}$ and $B_q$ are therefore, not included in this paper.

In previous studies (Lunne et al., 1989 and 1997b; Mayne, 1991) the overconsolidation ratio, OCR, has been tried correlated to the pore pressure parameters $B_q$, and $(u_2\text{-}u_0)/u_0$ and the parameter $Q_t$ defined as:

$$Q_t = \frac{(q_t - \sigma_{vo})}{\sigma'_{vo}} \qquad (6)$$

These OCR correlations are also used and discussed later in this paper.

At the majority of the locations in this study the CPTU tests were carried out with the wireless ENVI $5^t$ cone which has an area ratio of a= 0.69 (Elmgren, 1995).

### 3.2 *Note on reliability of measured CPTU parameters*

It is very important that the CPTU data are as reliable as possible. The best way to achieve this is to carry out the tests following the requirements and guidelines given in the International Reference Test Procedure (IRTP) published by the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE, 1999).

Most equipment used for commercial soil investigations follow the above guidelines, but it is particularly important that the schemes for calibration of sensors and the procedures for recording zero readings before and after each test are adhered to. Reliable measurement of penetration pore pressures depends very much on good procedures for saturation of the pore pressure measurement system. Test results should be examined shortly after testing to identify possible errors.

However, even if the CPTUs are carried out following the IRTP, experiences indicate that results can vary with the cone penetrometer made from one manufacturer to another, even with present (2004) modern equipment.

NGI(2002) recently carried out a test programme at the Onsøy2 soft clay test site to investigate the variability between different cone penetrometers produced by different manufacturers. The Onsøy2 site has been used by NGI for many years and is a very uniform site. Four to five cone penetrometer tests were done with each of the 9 different cone penetrometers tested. Table 2 presents typical examples of variability in measured cone resistance and pore pressure.

Table 2 Typical results of comparative CPTUs at Onsøy2 test site (Based on NGI, 2002)

| Depth, m | Corrected cone resistance, $q_t$ | | | Measured pore pressure, $u_2$ | | |
|---|---|---|---|---|---|---|
| | Overall average (kPa) | Range sensores (%) 1) | Total range (%) 2) | Overall average (kPa) | Range sensores (%) 1) | Total range (%) 2) |
| 10 | 589 | ±15.6 | ±16.5 | 452 | ±6.9 | ±12.8 |
| 20 | 898 | ±14.3 | ±14.9 | 681 | ±7.0 | ±13.2 |

1) Range between average readings from 9 different cone penetrometers
2) Total range including variability of all individual readings from different cone penetrometers (45 different CPTU soundings).

The results in Table 2 illustrate that the total range in cone resistance, $q_t$, is larger than the range in pore pressure, $u_2$, and also that the measured pore pressure is significantly less dependant on the type of cone penetrometer compared to the cone resistance. The reason why there is larger variation in cone resistance compared to the pore pressure is not fully understood, but it is a fact that the measured pore pressure is generally a much higher percentage of the capacity of the pore pressure sensor compared to the cone resistance and capacity of the load cell.

For some of the sites listed in Table 1 the CPTUs were carried out more than 20 years ago. Since considerable equipment improvement has taken place since then it is to be expected that the inherent uncertainties associated with the CPTU measurements may be larger than illustrated in Table 2. This also applies to the ENVI cone used in most of this study.

### 3.3 *Undrained strength correlations*

The data from this study clearly show that the pore pressure response gives the most consistent and best correlations to the undrained $s_{uc}$ strength determined on the block samples. Figure 5 shows the derived values of $N_{\Delta u}$ versus measured overconsolidation ratio, OCR, on the block samples. These data have been grouped into two ranges of sensitivity of $S_t$ smaller or larger than 15, which has a systematic impact on the excess pore pressure and $N_{\Delta u}$ . This is seen more clearly in figure 6 which shows $N_{\Delta u}$ directly against sensitivity for different ranges of OCR. Figure 7 shows that $N_{\Delta u}$ also to some extent depend on the plasticity index, $I_p$. As discussed in Section 2 of this paper $I_p$ is a parameter that can be misleading for clays with high sensitivity. Figure 7 also suggest that the plasticity index is a secondary parameter for the sensitive clays. On basis of a detailed study of the combined impact of OCR, $S_t$ and $I_p$ on the results, it is proposed typical average correlations accounting for their combined effects as follows:

For low sensitive clays ($S_t$<15)

$N_{\Delta u}$= 6.9 – 4.0logOCR+0.07($I_p$) $\quad I_p$ in % (7a)

For high sensitive clays ($S_t$>15)

$N_{\Delta u}$ = 9.8- 4.5logOCR (7b)

These correlations are illustrated in fig.5 for an average $I_p$-value of 22.4% for low-sensitive clays. Typical curves based on eq's 7a) and 7b) are also shown in Figs. 6 and 7 for ranges of OCR values, and using an overall average $I_p$=22.4 for $S_t$<15. The exact average OCR value corresponding to the different ranges in the data base are as follows:

| Range OCR | Average OCR |
|---|---|
| 1-2 | 1.57 |
| 2-4 | 2.92 |
| >4 | 4.90 |

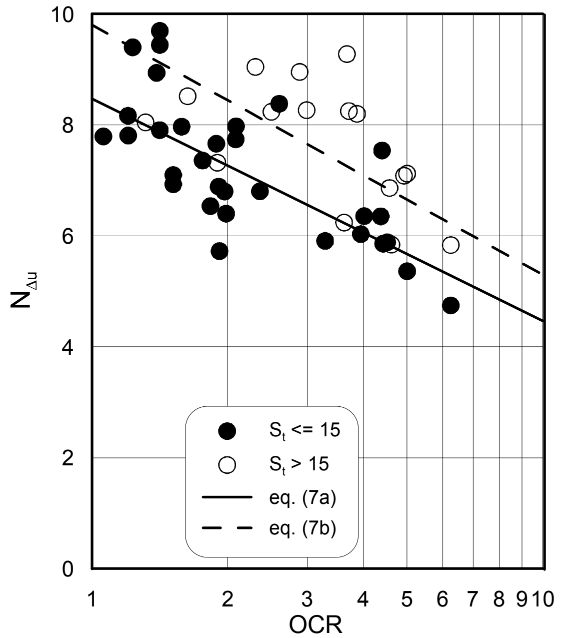


Figure 5. Relationship between $N_{\Delta u}$ and OCR

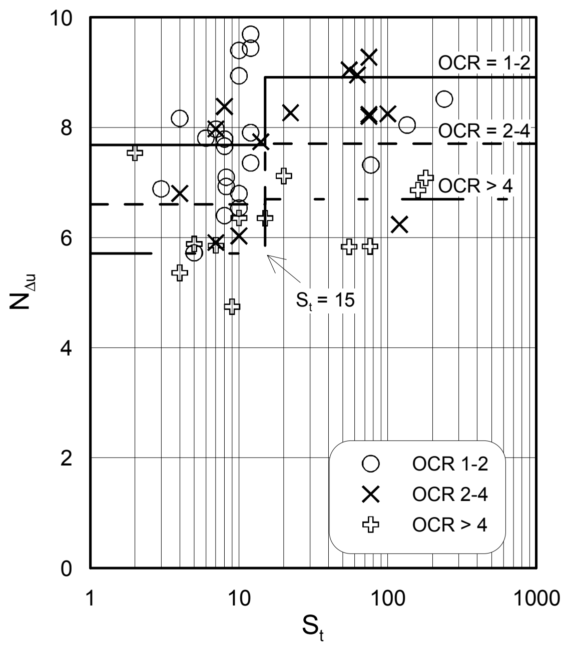


Figure 6. Influence of sensitivity on $N_{\Delta u}$

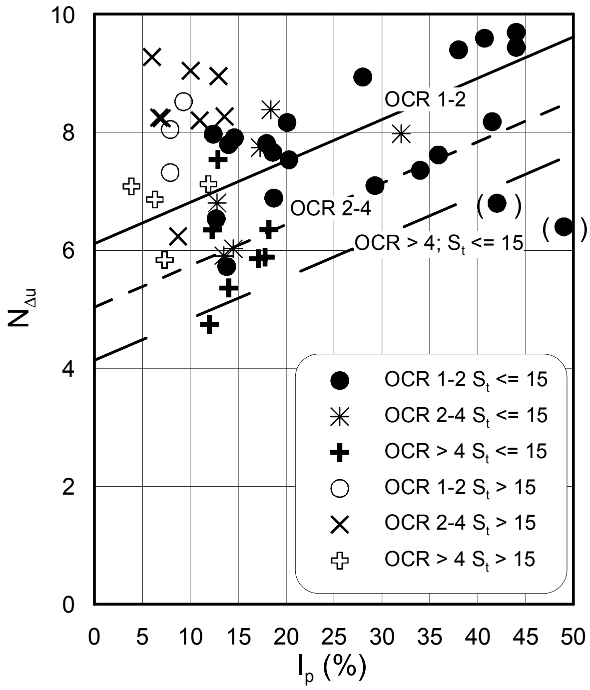


Figure 7. Influence of plasticity index on $N_{\Delta u}$

Assessment of scatter in the data suggests that there is a variation in strength of about ±10-15% when the correlations above are applied. Some of this variation is believed to be equipment and procedure related, and can depend on the cone penetrometer used and workmanship as discussed in section 3.2 above. Some of the scatter in the correlations may also be due to changes in design and improved accuracy of the cones used over this 20 year time period.

A special comment is made to the two points put in bracket in Fig.7. These represent the Bothkennar UK site. Hight et al. (2003) have suggested that the plasticity index as measured by standard test procedures could be much too high, because it has high organic content. These points were therefore given no weight when the effect of $I_p$ was assessed. Finally, it is also quite possible that other and more subtle soil parameters in reality influence the results and contributes to some of the scatter. This could for instance be the detailed stress-strain relation for the clay, including the large strain post-peak behaviour, and the anisotropic nature of the undrained shear strength. Even for the block samples there may also be some scatter due to disturbance effects, but this is believed to be less than ±5%.

It is a bit unfortunate that the cone factor $N_{\Delta u}$ depends of the overconsolidation ratio, which requires good sample quality to determine precisely. The dependency is however, not so strong. A fair estimate of OCR based on geologic history can be made in some cases. An alternative is to first tentatively estimate OCR, then use the first estimated $N_{\Delta u}$ and calculated $s_{uc}$ value to estimate a new OCR on basis of Figure 3, and use that OCR as new basis for assessing a revised $N_{\Delta u}$ and undrained shear strength. One or two iterations may be needed to define the final value. The first estimate of OCR can also be made on basis of the CPTU correlations described later in the paper.

Figures 8, 9 and 10 show similar relationships between the cone factor $N_{kt}$, OCR, $S_t$ and $I_p$. It is readily observed that these relationships show far more scatter than the $N_{\Delta u}$ relationships in Figs. 5 to 7. This could either be due to less accuracy of the sensors measuring cone resistance, or that the cone resistance even more than the pore pressure depends on other and more subtle soil parameters than OCR, $S_t$ and $I_p$. Tentative best fit correlations are as follows:

For low sensitive clays ($S_t$<15)
$N_{kt}$= 7.8+2.5logOCR+0.082$I_p$ (8a)

For high sensitive clays ($S_t$>15)
$N_{kt}$ = 8.5 +2.5logOCR (8b)

Figures 8-10 also show typical correlation functions according to eq's 8a) and 8b). The variation in calculated undrained strength based on these correlations typically lie around ±15 % for the high sensitive clays and ± 30% for the low sensitive clays.

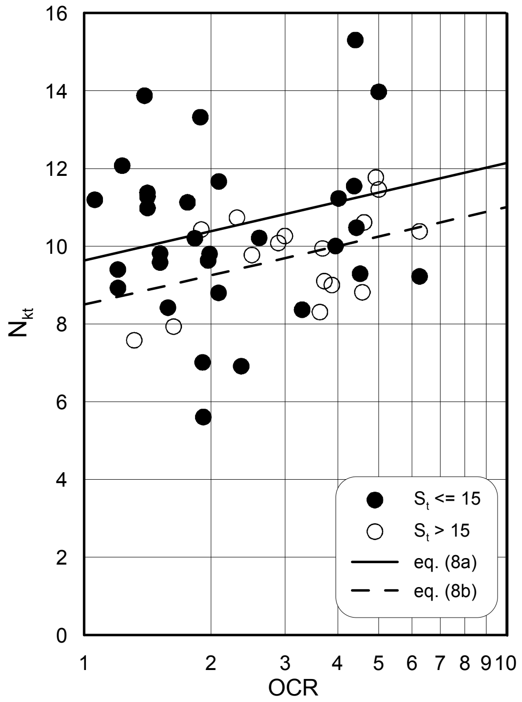


Figure 8. Relationship between $N_{kt}$ and OCR

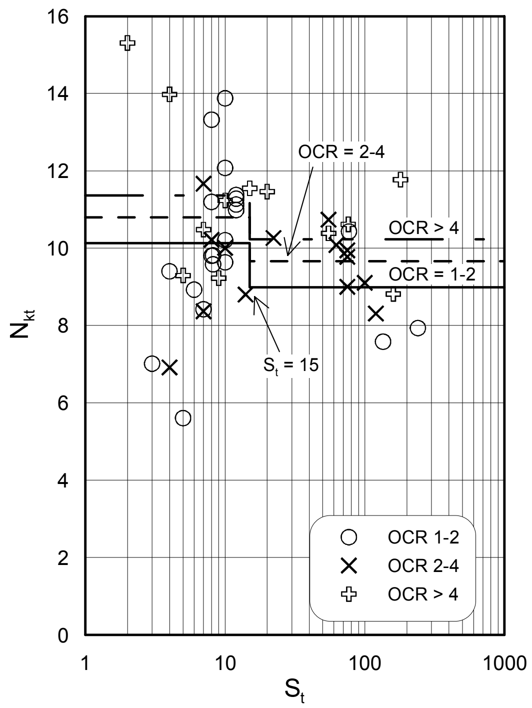


Figure 9. Influence of sensitivity on $N_{kt}$

Figure 11 compares shear strengths calculated from a series of 35 different CPTU-tests carried out in the Oslo harbour basin in a clay deposit that in geologic terms should be essentially normally consolidated. The CPTU tests cover an area of about 100 by 250 m. The water depth is typically 8-10 m, and the clay is rather homogenous with a water content in the range 35-45 %, generally decreasing with depth. The strengths were computed on basis of equations 7a) and 8a) accounting for a decrease in plasticity index from about 25% in the top to 15 % in the bottom and taking OCR=1.3 and constant with depth. Figure 11 confirms the larger scatter in strength from $N_{kt}$ than from $N_{\Delta u}$ that was also suggested by Table 2, with a variation of typically ±20% and ±10% respectively. It may however, be noted that the absolute variation is close to constant with depth. Another interesting observation that can be made from figure 11 is that the undrained shear strengths, $s_{uc}$, based on $N_{\Delta u}$ increase essentially linearly with depth, as was expected for this normally consolidated deposit. The $s_{uc}$ values based on $N_{kt}$ does not show the same linear trend and is higher than based $N_{\Delta u}$ in the top, but then fall off with depth relative to the $N_{\Delta u}$ strength profile. This suggests a depth influence on $N_{kt}$. Figure 12 compares the strength ratios directly. Herein is also included a depth correction factor $I_z$ applied to the strength arrived at from $N_{kt}$ to try to match the strength from $N_{\Delta u}$. $I_z$ is defined by the expression:

$$(s_{uc})_{Nkt} = I_z\,(s_{uc})_{N\Delta u} \tag{9}$$

$I_z=(1-a(z-z_{ref})/z)^b$ with $z_{ref}$=16m, a=0.45, b=0.75 as best fit

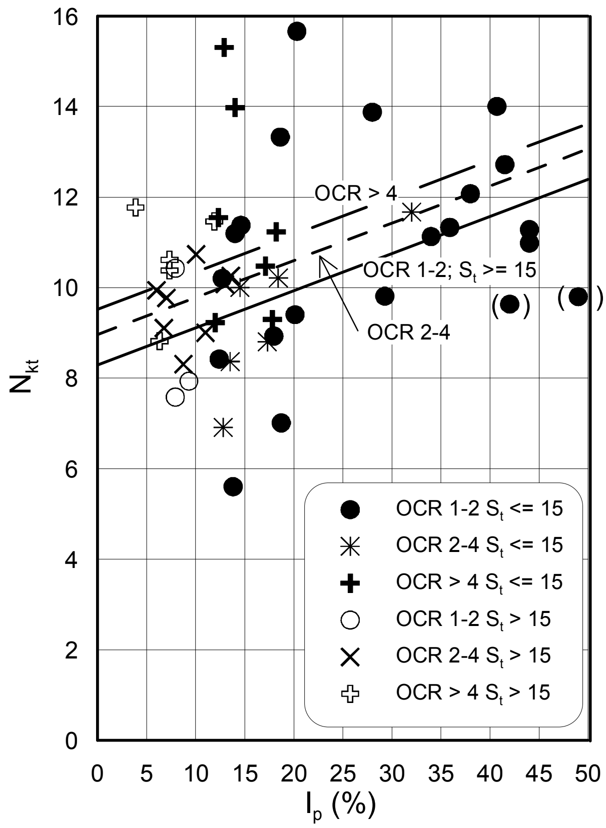


Figure 10. Influence of plasticity index on $N_{kt}$

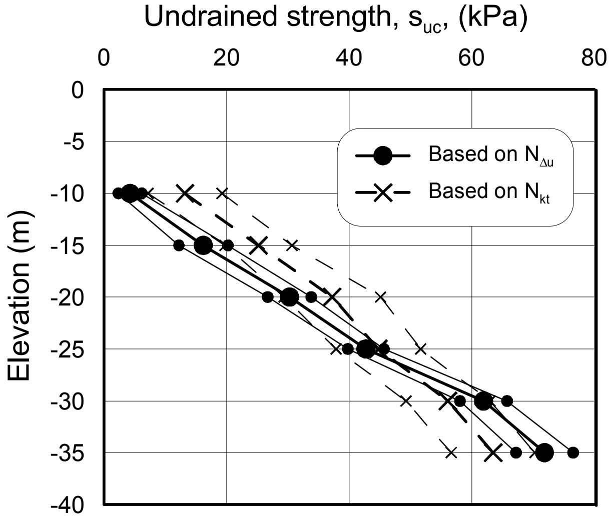


Figure 11. Comparison between strengths from cone resistance and pore pressure in Bjørvika, Oslo harbour

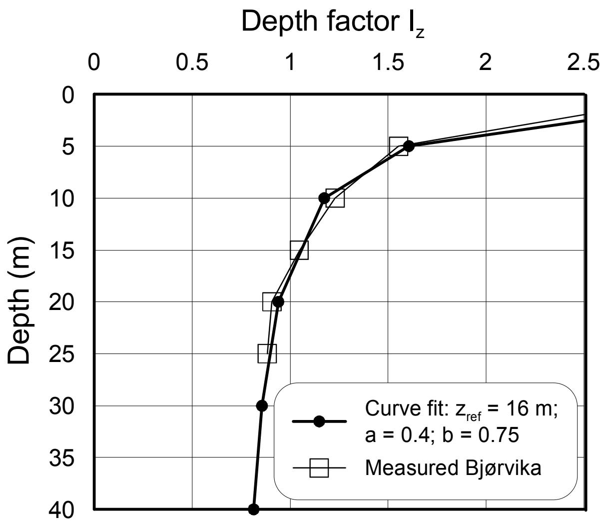


Figure12. Depth factor on $I_z$ suggested by data from Bjørvika in

The block sampling data presented in this paper has not made it possible to sort out if this is a general depth effect, and if the expression (9) is representative for other clay profiles or other cone penetrometer types. Further comparative and statistical studies at different sites and with different cone penetrometers are needed to get a better grasp on this potential depth influence.

The cone factor $N_{ke}$ contains both corrected cone resistance and excess pore pressure, equation (4). Figure 13 shows that this cone factor gives very large scatter when plotted against OCR. Figure 14 shows on the other hand that $N_{ke}$ is better related to the pore pressure factor $B_q$ The advantage of this correlation is that it does not require any independent estimate of OCR. It has not been possible to identify any significant dependency of this correlation on plasticity index of the clay, but it depends somewhat on sensitivity. The typical average lines in Figure 14 are represented by the following equations:

For low sensitive clays ($S_t$<15)
$N_{ke} = 11.5 – 9.05B_q$ with $N_{ke}$=2.0 as a lower limit (10a)

For high sensitive clays ($S_t$>15)
$N_{ke} = 12.5 – 11.0B_q$ with $N_{ke}$=2.0 as a lower limit (10b)

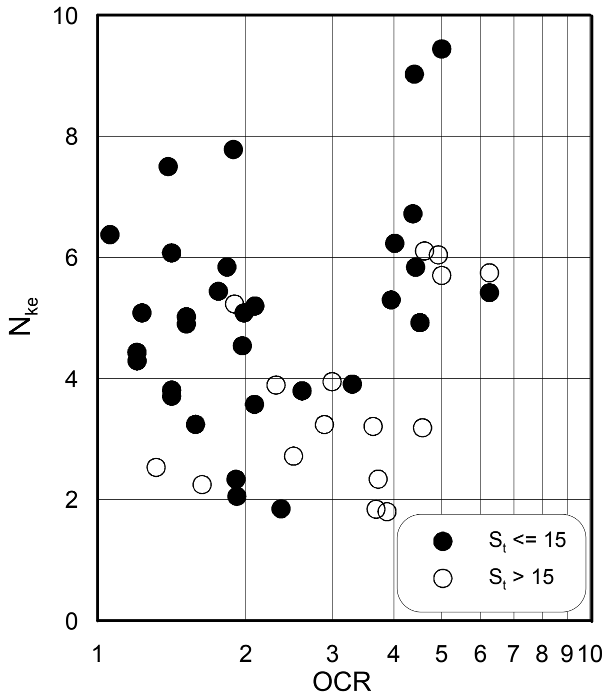


Figure 13. Relationship $N_{ke}$ and OCR

These correlations should be used with caution for values of $B_q$ less than about 0.6, as there is little data in that range, and the data also suggest a non-linear trend for low $B_q$ values. It may also be noted that the variation in $N_{ke}$ around the average line is about ±1.0 at all levels of $B_q$. This means as an illustration, that the variation in strength increase from ± 15% at $B_q$ =0.6 to ± 33% at $B_q$ =0.9. This variation or uncertainty in calculated strength is larger than when using the pore pressure response alone, but is generally better than when using the cone resistance alone when $B_q$ is less than about 0.7-0.8.

In engineering practice the authors recommend that all three cone factor correlations (based $N_{\Delta u}$, $N_{kt}$, and $N_{ke}$) are used to determine the undrained strength, but that most weight is given to strengths arrived at from $N_{\Delta u}$. A comparison between these strengths may also help to reveal apparent inconsistencies or problems with the individual CPTU tests. For instance will low $s_{uc}$ values based on $N_{\Delta u}$, compared to the other two often give an indication of poor saturation of the piezometer system.

It is also important to bear in mind that the undrained strength determined on high quality block samples can be considerably larger than on samples taken with conventional piston sampling, and that the block samples also show far more pronounced strain softening beyond peak than poorer quality samples. A designer must keep that in mind when using the strengths derived from the CPTU correlations proposed herein.
If the correlations are to be applied to distinctly different clays from the mostly Norwegian marine clays covered in this study, it is recommended to verify the applicability of the proposed correlations by similar block sampling and testing as described herein. The cone factors have also to some extent been found to depend on the cone penetrometer used (e.g. Table 2 and NGI,2002), which may also warrant some independent verification of the applicability of the correlations. Even for large and important projects in Norway it is the authors' recommendation to carry out block sampling and verify the applicability of the correlations for specific CPTU equipment used. Examples of combination of CPTU testing and block for actual projects were presented by Karlsrud (1999) and Jensen (2001). These examples showed that the undrained strengths as a result of such investigations could be upgraded by 20-30%, which led to very substantial cost savings for the projects in question.

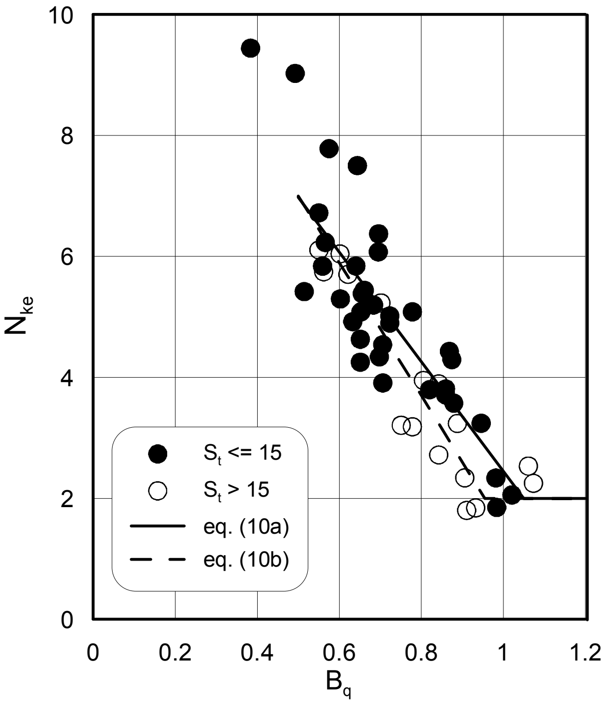

Figure 14. Relationship $N_{ke}$ and $B_q$

### 3.4 *Correlations for overconsolidation ratio, OCR*

Figure 15 shows that there is a correlation between the cone factor $B_q$ (eq. 5) and OCR, but the scatter is fairly large, which also agrees with data presented in Fig.5.14 of Lunne et al. (1997b). A similar and slightly better correlation was found between the normalised excess pore pressure $(u_2\text{-}u_0)/\sigma'_{vo}$ and OCR, Figure 16, but the best correlation was found between the cone factor $Q_t$ (eq. (6)) and OCR, Figure 17. The typical average value of OCR according to Fig.17 corresponds to:

For low sensitive clays ($S_t$<15)
$$OCR = (Q_t/3)^{1.20} \tag{11a}$$

For high sensitive clays ($S_t$>15)
$$OCR = (Q_t/2)^{1.11} \tag{11b}$$

Even use of Figure 17 will give significant uncertainty in estimated OCR, and relatively speaking much larger than for the correlations between CPTU results and the undrained shear strength. One reason for that may be the inherent variability in the actual relation between in-situ undrained strength and OCR, as shown by Figure 3. As for the undrained shear strength it is recommended to use all correlations on specific projects.

## 4 CONCLUSIONS

The correlations presented herein between various cone resistance factors and the true in-situ undrained triaxial compression strength of samples consolidated anisotropically to in-situ effective stresses prior to shearing, $s_{uc}$, show that the pore pressure factor gives the most consistent correlations, and is recommended as the best factor for determination of $s_{uc}$. The cone factor $N_{\Delta u}$ depends however on the overconsolidation ratio, OCR, the sensitivity, $S_t$, , and the plasticity index, $I_p$, of the clay. The reliability of the strength arrived at lie around ±10%, provided of course that the CPTU equipment and procedures used deter-

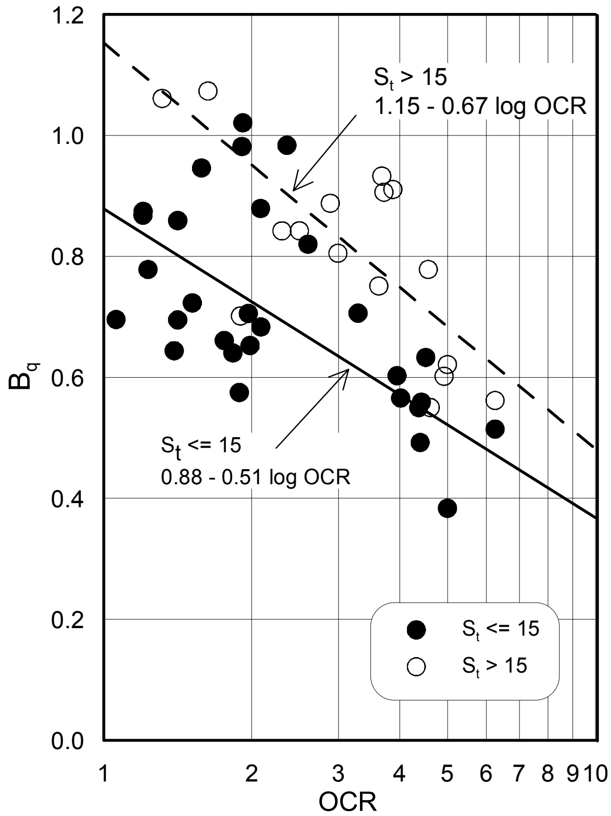

Figure 15. Relationship $B_q$ and OCR

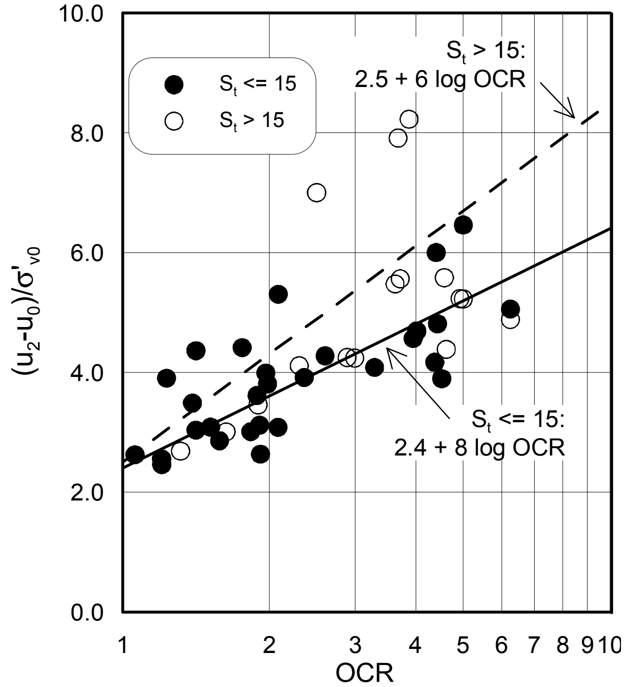

Figure 16. Relationship normalised excess pore pressure $(u_2\text{-}u_0)/\sigma'_{v0}$ and OCR

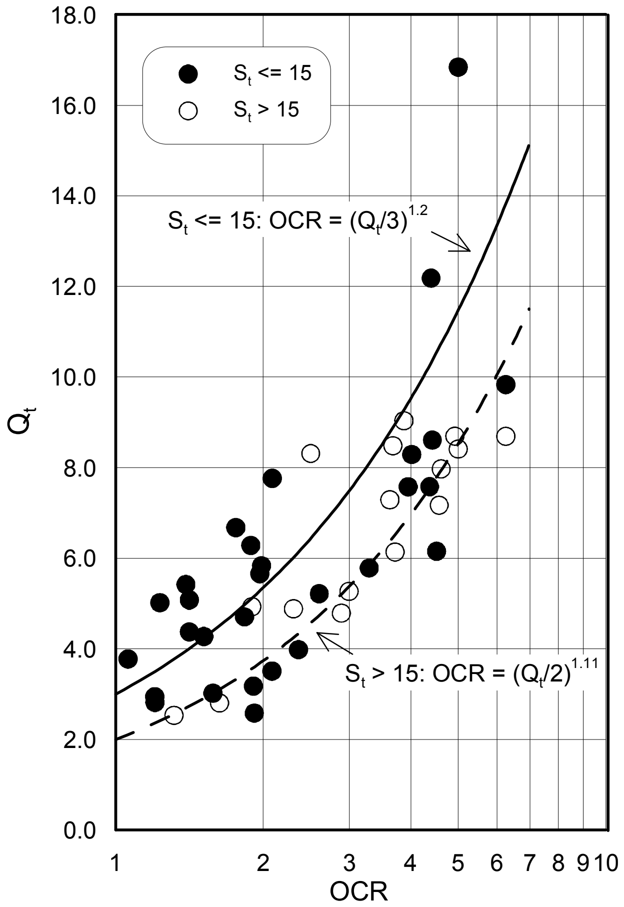


Figure 17. Relationship $Q_t$ and OCR

mines the pore pressure in a reliable manner. For most cones on the market today, this should not be a problem, but it requires good follow up and checking of saturation procedures, zero readings and calibration factors.

The cone resistance factor $N_{kt}$, give in general poorer correlations against undrained strength than the pore pressure response. This may partly be an equipment problem but possibly also reflect that the cone resistance depends on more subtle soil parameters like for instance the stress-strain relation up to and beyond peak. The data from Oslo harbour (Figures 11 and 12) also indicate that there may be a depth influence on $N_{kt}$ which should be investigated further.

The effective cone factor $N_{ke}$, combining both cone resistance and pore pressure response, lies somewhere in between the $N_{\Delta u}$ and $N_{kt}$ factors when it comes to variability. However, for values of $B_q$ larger than 0.7-0.8 the variability in $N_{ke}$ becomes rather large. It is still recommended to use both the $N_{ke}$ and the $N_{ke}$ correlations to verify the undrained strengths arrived at from $N_{\Delta u}$.

When using the strengths derived from the proposed CPTU correlations, a designer must keep in mind that the undrained strength determined represent the peak undrained triaxial compression strength on high quality samples.

The data suggest that the best estimate of overconsolidation ratio can be made from the normalized cone resistance factor $Q_t$, but the correlation is not nearly as good as for predicting the undrained strength.

It should finally be noted that the correlations herein have been developed from a data base mainly containing results for Norwegian marine clays, and mainly using the cone penetrometer by ENVI. If applied to distinctly different clays and/or using different CPTU equipment it is encouraged to verify the applicability of the proposed correlations by similar comparisons between results of block sampling and CPTU results as described herein. Even for large and important projects in Norway it is the authors' recommendation to verify the applicability of the correlations in such a manner.

It can finally be mentioned that it is NGI's and the authors' intention to regularly update the database, also including data that in the future may become available through others.

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## REFERENCES

Andresen, A. and Kolstad, P. 1979. The NGI 54 mm samplers for undisturbed sampling of clays and representative sampling of coarser materials. *Proc. Int. Symposium on Soil Sampling*, Singapore 1979, pp. 13-21.

Becker, D.E., Crooks, J.H.A., Been, K. and Jefferies, M.G. 1987. Work as a criterion for determining in situ and yield stresses in clay. *Canadian Geotechnical Journal*, Vol. 24, p. 549, 1987.

Bjerrum, L. 1972. Embankments on soft ground, state-of-the-art report. *Proc. Specialty Conference on the Performance of Earth and Earth-Supported Structures*, Lafayette, Indiana, Vol. 2, pp. 1-54.

Burland, J.B. 1990. On the compressibility and shear strength of natural clays. Rankine Lecture. *Geotechnique*, Vol. 40, No. 3, pp. 329-378.

Casagrande, A. 1936. The determination of the pre-consolidation load and its practical significance. *Proc. First International Conference on Soil Mechanics and Foundation Engineering*, Vol. III, Discussion D-34, p. 60. Boston 1936.

Elmgren,, K. 1995. Slot-type pore pressure CPTU filters. Behaviour of different filling media. *Proc. Int. Symposium on Cone Penetration Testing CPT '95,* Linköping, Sweden 1995, Vol. 2, pp. 9–12. (Also: SGF Report 3:95.)

Hight, D., Paul, M.A, Barras, B.F., Powell, J.J.M., Nash, P.R., Smith, P.R., Jardine, R.J. and Edwards, D.H. 2003. The characterisation of the Bothkennar clay. *Proc. Workshop on Characterisation and Engineering Properties of Natural Soils*, Singapore, 2003, Vol. 1, pp. 543-598.

ISSMGE International Society of Soil Mechanics and Geotechnical Engineering. 1999 International Reference Test Procedure for the Cone Penetration Test (CPT) and the Cone Penetration Test with Pore Pressure (CPTU). Report of the ISSMGE Technical Committee 16 on Ground Property Characterisation from In-situ Testing, *Proc. 12th European Conference on Soil Mechanics and Geotechnical Engineering*, Amsterdam, 1999, Vol. 3, pp. 2195-2222.

Jensen, T.G. 2001. CPTU combined with block sampling resulted in cost saving solutions at Nykirke railway link. *Young Geotechnical. Engineers Conf.*, Bulgaria 2001.

Karlsrud, K. 1991. Sammenstilling av noen erfaringer med prøvetaking og effekt av prøveforstyrrelse i norske marine leirer. *NGI report 521500-6.*

Karlsrud, K. 1995. Blokkprøvetaking i kombinasjon med CPTU gir nye muligheter. *Fjellsprengningsteknikk – Bergmekanikk - Geoteknikk 1994, Geoteknikkdagen 1995, Oslo*. (Also: NGI Report 521674-1.)

Karlsrud, K. 1999. General aspects of transportation infrastructure. Keynote lecture. *Proc. 12th European Conference on Soil Mechanics and Geotechnical Engineering*, Amsterdam.1999, Vol.1, pp.17-30.

Karlsrud, K., Lunne, T.and Brattlien, K. 1996. Improved CPTU interpretations based on block samples. *Proc. 12th Nordic Geotechnical Conference*, Reykjavik 1996. Vol. 1, pp. 195-201. (Also NGI, Oslo. Publ. 202).

Kenney, T.C. 1967. The influence of mineral composition on the residual strength of natural soils. *Proc. Geotechnical Conf. Oslo 1967*. Vol. 1, pp. 123-129. (Also NGI, Oslo. Publ. 76).

Lacasse, L. , Berre,T. and Lefebvre,G.1985. Block sampling of sensitive clays. *Proc. 11th International Conference on Soil Mechanics and Foundation Engineering,* San Francisco 1985. Vol. 2, pp. 887- 892. (Also NGI, Oslo. Publ. 163, 1986).

Ladd, C.C., Foott, R., Ishihara, K., Schlosser, F. and Poulos, H.G. 1977. Stress-deformation and strength characteristics. State-of-the-art report. *Proc. 9th International Conference on Soil Mechanics and Foundation Engineering*, Tokyo 1977, Vol. 2, pp. 421-94.

Lefebvre, G. and Poulin, C. 1979. A new method of sampling in sensitive clay. *Canadian Geotechnical Journal*, Vol. 16, pp. 226-233.

Leroueil, S., Samson, L. and Bozozuk, M. 1983. Laboratory and field determinations on preconsolidation pressure at Gloucester. *Canadian Geotechnical Journal*, Vol. 20, No. 3, pp. 477-490.

Lunne, T. 2002. Engineering properties of lean Lierstranda clay. *Proc. International Symposium on Coastal Geotechnical Engineering in Practic,* Yokohama, Japan 2000, Vol. 2, pp. 177- 186.

Lunne, T., Christophersen, H.P. and Tjelta, T.I. 1985. Engineering use of piezocone data in North Sea clays. *Proc. 11th International Conference on Soil Mechanics and Foundation Engineering*, San Francisco, 1985, Vol. 2, pp. 907-912.

Lunne, T., Eidsmoen, T:, Gillespie D. and Howland J.D. 1986

Laboratory and field evaluation of cone penetrometers. *Proc. ASCE Spec. Conference In Situ'86*, Blacksburg, Va. 1986, pp. 714-729.

Lunne, T., Lacasse. S. and Rad, N.S. 1989. SPT, CPT, pressuremeter testing and recent developments on in situ testing of soils. General Report. *Proc. 12th International Conference on Soil Mechanics and Foundation Engineering*, Rio de Janeiro 1989, Vol. 4, pp. 2339-2403.

Lunne, T.; Berre, T. and Strandvik, S. 1997a. Sample disturbance effects in soft low plastic Norwegian clay. *Proc. International Symposium on Recent Developments in Soil and Pavement Mechanics,* Rio de Janeiro 1997, pp. 81-102. (Also NGI, Oslo. Publ. 204).

Lunne, T.; Robertson, P.K. and Powell, J.J.M. 1997b. *Cone penetration testing in geotechnical practice*. London, Spon Press. 312p.

Mayne, P.W. 1991. Determination of OCR in clays by piezocone tests using cavity expansion and critical state concepts. *Soils and Foundations*, Vol. 31, No. 2, pp. 65-76.

Norwegian Geotechnical Institute 2002. Quality of CPTU. Statistical analyses of CPTU data from Onsøy. *NGI Report No. 20001099-2*, dated 5 September 2002.

Norwegian Geotechnical Institute 2004 SIP8- Offshore Geohazards-Correction methods for oedometer tests, theory and verification. *NGI Report 20021023-11* (in printing).

Roy, M., Tremblay, M., Tavenas, F. and La Rochelle, P. 1980. Induced pore pressures in static penetration tests in sensitive clay. *Preprint Volume. 33rd Canadian Geotechnical Conference,* Calgary 1980, p.p. 11.3.1 to 11.3.13.