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Kjell Karlsrud

Strength and deformation properties of Norwegian clays from laboratory tests on high quality block samples.

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Strength and deformation properties of Norwegian clays from laboratory tests on high quality block samples

The 23rd Bjerrum Lecture presented in Oslo 26 November 2010

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ABSTRACT

For a long time it has been a challenge for the geotechnical profession to determine the true behaviour of clays based on soil sampling and laboratory testing. The main difficulty has been to retrieve sufficiently undisturbed samples that maintain the clay structure and correctly depict the *in-situ* stress-strain and strength behaviour of the clay under any kind of imposed drained or undrained loading conditions. The extent and impact of the disturbance caused by sampling depends on a number of factors such as the specific sampling equipment and sampling procedures that are used, index properties of the clay (sensitivity and plasticity index in particular), and sampling depth. Experience with laboratory tests on conventional piston samples shows that sample disturbance commonly affects the test results. In the mid 1970's, the University of Sherbrooke developed a special block sampler that was shown to give samples of very high quality. Between 1982 and 2010, the Norwegian Geotechnical Institute (NGI) has used the Sherbrooke block sampler at 22 different sites in Norway and 1 site in the UK. Two to five block samples were collected at each site, on which oedometer, undrained triaxial and direct simple shear (DSS) test were carried out in the laboratory. Essentially all samples tested showed far superior quality as compared to conventional piston samples. It was therefore considered to be valuable to summarize test results obtained on such high quality samples, as presented herein. Deformation and strength parameters from individual tests are summarized, and compared against index data for the different clays tested. This has resulted in a series of correlation diagrams. The index parameters found of most relevance to use as correlation parameters are the natural water content and clay sensitivity. The latter is expressed in two categories: medium to low sensitivity (S_t<15) and high sensitivity (S_t>15). The overconsolidation ratio is another key parameter. The correlation diagrams presented include the following: i) pre-consolidation pressure, compressibility, permeability and permeability change index as derived from oedometer tests, ii) normalized peak and post-peak shear strength, peak effective friction angle, stiffness and strain at failure from undrained triaxial and DSS tests.

When design analyses are to be based on the results of tests on high quality block samples, as compared to conventional piston samples of poorer quality, it is important to ensure that the end result complies with past semi-empirical experiences.

INTRODUCTION

The traditional way of sampling soft clays in Norway since the mid 1950's has been to use fixed piston samplers. At the Norwegian Geotechnical Institute (NGI), experiences with the standard 54 mm piston sampler confirm that it cannot produce undisturbed good quality samples when the sample depth exceeds about 10-20 m in clays of medium to high plasticity, and in the most silty type Norwegian clays such samples essentially cannot be produced.

Note: This written presentation of the lecture is also published in Canadian Geotechnical Journal (2013), Vol. 50, Number p. 1-21, and was co-authored by Francisco Gabriel Hernandez-Martinez.

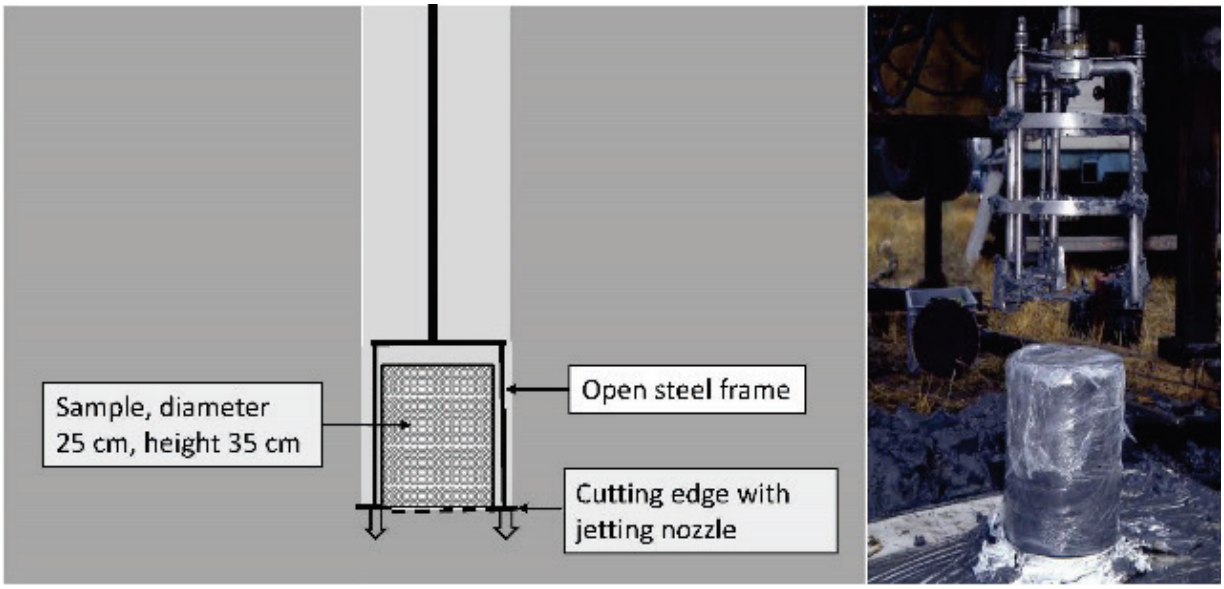

Figure 1. Principal sketch and photo of Sherbrooke block sampler.

High sensitive low-plastic clays are the worst.

In the late 1960's NGI introduced the thin-walled sharp-edged 95 mm steel piston sampler, which definitely gave improved sample quality compared to the traditional 54 mm sampler (Bjerrum 1973). It was also shown that if 95 mm samples are extruded and built into the testing apparatus very shortly after sampling (less than 1-2 hours), that would also significantly enhance the sample quality, especially in the most difficult lean and sensitive clays. The reason is that such short storage time limits the possibility for the sample to reconsolidate and stick to the inner wall of the cylinder during extrusion.

In 1976, NGI and the first author became involved in a project in James Bay, Canada where dams and dikes were planned and built on soft and partially high sensitivity clays. In this collaboration, NGI undertook a series of laboratory tests on samples that were taken using a special block sampler developed at the University of Sherbrooke (Lefebvre and Poulin 1979). The laboratory test results obtained on these block samples were in NGI's experience rather extraordinary in terms of the small strain at failure and extremely brittle collapse they showed in undrained tests, and the very pronounced volumetric collapse the samples showed when stressed beyond the apparent preconsolidation pressure in oedometer tests, (Eide and Andresen 1977; Karlsrud *et al.* 1984). Several years later NGI approached Guy Lefebvre at University of Sherbrooke, who agreed to come to Norway and use this block sampler at a number of Norwegian sites (see Fig. 1). The three sites covered then were in the quick clay deposits at Ellingsrud and Emmerstad and in non-sensitive clay at Onsøy, places where NGI had previously taken both 54 and 95 mm piston samples.

The results of NGI's oedometer, undrained direct simple shear (DSS), and triaxial tests on these block samples were also very impressive in terms of sample quality and test results (Lacasse *et al.* 1985), and convinced NGI that this block sampler would be very valuable tool to use also on other projects in the future. When NGI became involved in the design of a new railway to the new airport at Gardermoen, which along most of its 40 km length would involve many cuts and fills through marine clay sediments, the importance of obtaining high quality samples as basis for design was recognized. NGI

therefore bought such a block sampler from the University of Sherbrooke in 1992 for use in this and other future projects.

Up until 2010 NGI has used the block sampler at 22 different sites in Norway and one in the UK. Most applications have been in connection with commercial projects but some have also been pure research and development. Laboratory testing on the block samples NGI has retrieved and tested over the years, have mainly consisted of continuous or incremental loading oedometer tests, undrained triaxial compression and extension tests, and DSS tests. The data from these tests on block samples show that the tested samples in general have been of high quality (NGI 2012). It was therefore considered to be of great value to the profession to establish a database presenting key results from tests on such good high quality samples. The two main objectives of this study have been:

1. To collect, organize and summarize all existing data from laboratory tests carried out on high quality block samples taken by NGI at 23 different sites in the period 1982 to 2010. The laboratory tests include results of constant rate of strain (CRS) and incremental loading (IL) oedometer tests, consolidated anisotropic undrained compression (CAUC) and extension (CAUE) triaxial tests, and DSS tests.

2. To develop empirical correlations between various key engineering soil parameters like volumetric compressibility, overconsolidation ratio, permeability, undrained strength and shear stiffness, with the index properties of the various clays tested.

EFFECTS OF SAMPLE DISTURBANCE

Sample disturbance effects are not only caused by the sampling operation itself, but also by some inevitable effective stress changes and disturbance caused by trimming and handling of the specimens in the laboratory prior to testing. (e.g. Berre *et al.* 1969; La Rochelle and Lefebvre 1970; Lunne *et al.* 1997, 2006; Ladd and DeGroot 2003).

Sample disturbance has a considerable effect on the stress-strain response and strength of the soil compared to its *in-situ* characteristics as discussed quite early by Hvorslev (1949) and Bjerrum (1954), Ladd and Lambe (1963), Skempton and Sowa (1963), and Noorany and Seed (1965). In later years, it has also been an important research topic. For instance, Berre *et al.* (1969) compared results of oedometer tests on Norwegian soft marine clays recovered with the new 95 mm NGI sampler and those recovered with the 54 mm NGI piston sampler. They concluded that the yield stresses derived from the oedometer results on soil obtained with 95 and 54 mm samplers were not very different except that the scatter in the results was higher for the 54 mm samples.

To illustrate the impact of sample quality, Fig. 2 compares tangent modulus versus axial stress from CRS oedometer tests on a high-quality block sample and a 54 mm piston sample of poorer quality on samples taken at the Onsøy site. The effects of sample disturbance appear as: i) a very significant reduction in the initial tangent modulus, M_0, when loading up towards the preconsolidation pressure (see also definitions in the section titled "Results of oedometer tests"), ii) an increase in the minimum tangent modulus, M_L, reached before the modulus starts to increase again, and iii) an increase in tangent modulus also as the stress level increase into the virgin consolidation range. The reason for the increase in stiffness of the disturbed 54 mm sample as compared to the high-quality block sample at stress levels beyond the minimum breaking point lies in the large volume change that has occurred as a result of the disturbance during loading up to that stress level. This volume change gives a denser soil fabric and consequently a larger stiffness.

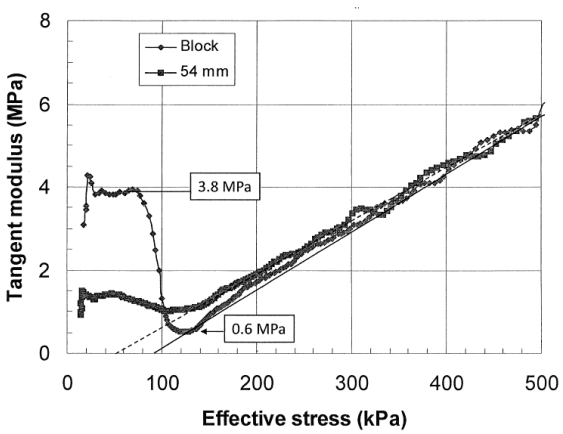

Figure 2. Example of oedometer test results on block sample vs 54 mm sample.

Figure 3 presents a similar comparison of sample disturbance effects on the results of CAUC triaxial tests on Onsøy clay, as suggested by: i) a significant reduction in the peak strength, ii) a significant increase in the strain at failure, and iii) an increase in the shear stress at large strains. The explanation for the increase in post-peak shear stress for the disturbed piston samples is, as for the CRS results, the larger volume change upon consolidation to *in-situ* stresses for the 54 mm sample than for the block sample, leading to a denser soil fabric.

For lean silty clays, sample disturbance will often change the behaviour rather dramatically, from contraction and strain softening

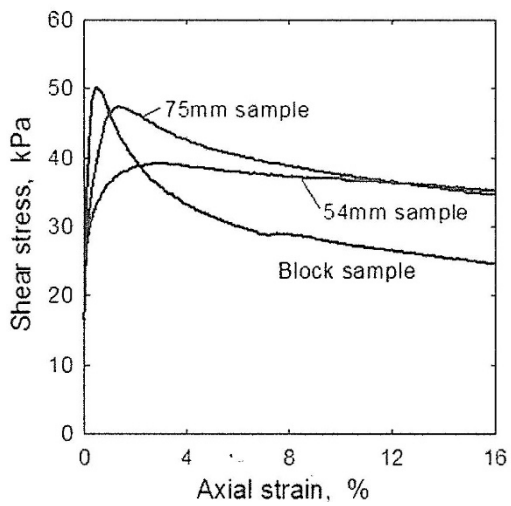

Figure 3. Example of triaxial test results on block sample vs 75 mm and 54 mm sample, Onsøy clay (after Lunne et al. *1997).*

of a good quality sample to dilation and strain-hardening behaviour of a poor quality sample. Fig. 4 shows a typical example of this. The impact of sample quality on the undrained strength is in general, less pronounced in CAUE triaxial extension tests and DSS tests than in the CAUC triaxial compression tests. The reason for this lies in the principle stress rotation that occurs during such tests, which tends to gradually break down the original clay structure. Table 1 gives a qualitative indication of the relative effect on the strength for different modes of shearing for 54 mm piston samples compared to high-quality block samples. The quality of 54 mm piston samples generally decreases with i) increasing sample depth, ii) increasing sensitivity, and iii) decreasing water content or plasticity index.

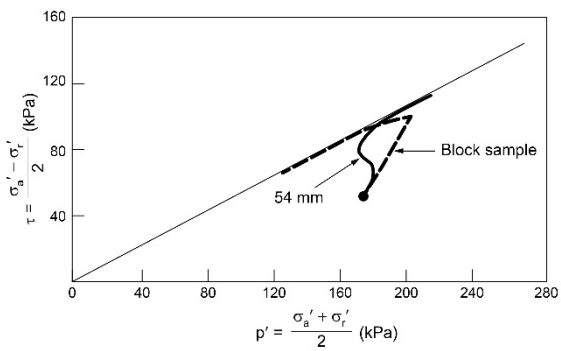

Figure 4. Comparison of effective stress paths for high and poor quality CAUC triaxial tests on a lean silty NC clay.

Table 1. Relative impact of strength of 54 mm piston sample as compared to block samples of high quality.

Test type	% higher strength of block sample compared to 54 mm piston sample
CAUC-triaxial compression	10-50
CAUE-triaxial extension	0-10
DSS-direct simple shear	5-20

Other factors that can significantly affect quality of piston samples are i) detailed

sampling operations in the field, ii) thickness and shape of the edge of the sample cylinders, iii) transportation, handling and storage time of the cylinders, iv) trimming and handling during preparation for testing.

For further discussion of sample disturbance effects the reader is referred to Lunne *et al.* (1997, 2006), Lunne and Long (2006), Lunne and Andersen (2007), and Berre *et al.* (2007).

BLOCK SAMPLING SITES

Sixteen of the sites where NGI has taken block samples are located within the marine clay deposits in southern Norway. Six sites are located in Trøndelag close to Trondheim. One site is located outside Norway at Bothkennar west of Edinburgh. Seven of the 23 sites are reference sites used by NGI for other research purposes. This includes the sites at Daneviksgt. and Lierstranda near Drammen, the three Onsøy sites near Fredrikstad, and the Ellingsrud and Emmerstad sites. The Bothkennar site is also a reference research site established by the Building Research Establishment in the UK. A complete issue of Géotechnique (1992) was devoted to a description of the Bothkennar site.

For all sites, determination of the *in-situ* vertical effective stress at the time of sampling was in most cases based on independent piston sampling to get a continuous distribution of total unit weights, combined with measurements of *in-situ* pore pressure, or as a minimum observed ground water level. In a few cases, the *in-situ* pore pressure distribution deviated considerably from hydrostatic. Thus, the effective stress levels are in these cases different from what one would normally have expected.

LABORATORY TESTS CARRIED OUT

Index testing on the block samples include in all cases unit weight, water content and plastic limits, and in most cases fall cone tests on undisturbed and remoulded material. Sensitivity values have primarily been derived from the fall cone tests. When no fall cone tests were conducted, the sensitivity was taken from in *in-situ* vane borings (when available). The remoulded shear strength determined by *in-situ* vane borings will in general be higher than from fall cone tests, especially for the very high-sensitivity clays. The reason is that remoulded shear strengths less than about 0.5 kPa cannot be read by the *in-situ* vane equipment, and rod friction can also contribute to excessively high remoulded values. When there were obvious discrepancies, the liquid limit was used to set the sensitivity value. *In-situ* cone penetration tests with pore pressure measurement (CPTU) borings were undertaken at many of the block sampling sites. These CPTU data have previously been systemized and correlated to undrained strength and overconsolidation ratio determined for the block samples, ref. Karlsrud *et al.* (2005).

All Atterberg limits data plot above the A-line in the Casagrande plasticity chart, confirming that the clays are generally inorganic. For all samples tested there was very good agreement between measured total unit weights, γ_t, and *in situ* water content, w, according to Eq. [1] applicable to fully saturated soils, for a specific unit weight in the range 2.65 to 2.70.

$$[1] \qquad \gamma_t = 9.81\left[\left(\frac{1+w}{\frac{1}{\rho_s}+w}\right)\right]$$

Figure 5 shows, as expected, that the clay sensitivity, S_t, is closely related to the liquidity index, LI (data is limited to 19 of the 23 sites). According to the definition used by the Norwegian Geotechnical Society, a clay is defined as very or highly sensitive when S_t >30, which from Fig. 5 is the case when the liquid limit exceeds about LI=1.35. To define a clay as quick, the

remoulded shear strength must also be less than 0.5 kPa.

The grain size curve has been determined for about 60% of the block samples. Figure 6 shows that the clay content, defined as particle size less than 0.002 mm, varies from 22% to 65% for the samples tested. As expected, the clay content tends to increase with the water content.

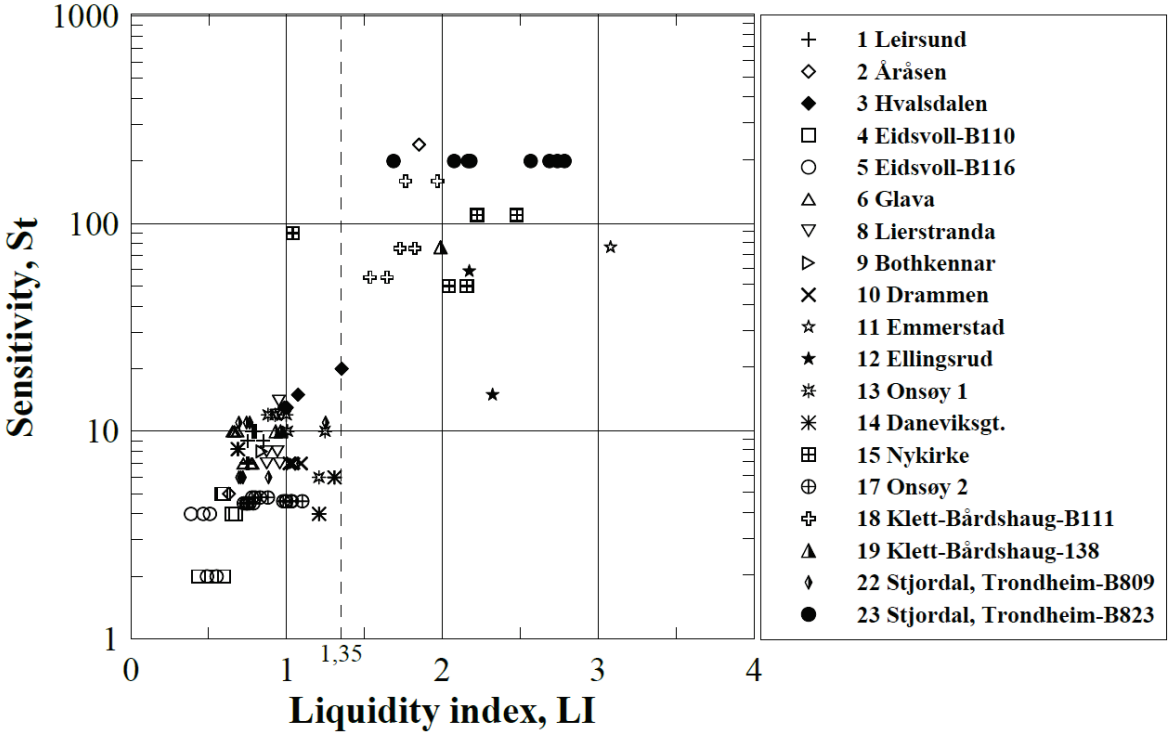

Figure 5. Relationship between sensitivity and liquidity index.

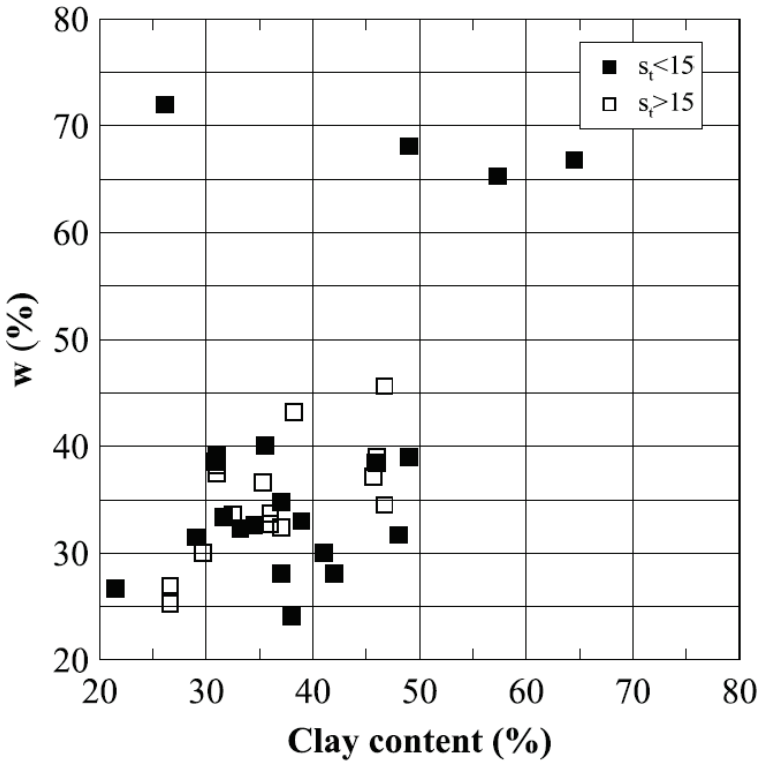

Figure 6. Clay content versus water content.

QUALITY OF TESTS

Lunne *et al.* (1997) have proposed the criterion in Table 2 for evaluation of sample disturbance based on the normalized change in voids ratios, $\Delta e/e_0$, when a test specimen is consolidated to the assumed *in-situ* effective stress level. Figure 7 presents values of $\Delta e/e_0$ versus depth for the triaxial tests covered in this study. The data suggest that the quality of the block samples generally reduce with sample depth. The DSS and oedometer tests showed similar results. As the level of unloading during sampling will increase with sample depth, the volume change during re-loading to *in-situ* stresses will also increase with depth. It may be assumed that volume change resulting from such re-loading, and not physical disturbance, will have less impact on the quality of the tests than physical sample disturbance.

The shape of the oedometer curves is another strong indicator of sample quality. A key parameter in that respect may be the ratio of maximum modulus, M_0, observed when loading up towards the preconsolidation pressure, p'_c, to the minimum modulus, M_L, observed during loading beyond the apparent pre-consolidation pressure, as defined in the section titled "Results of oedometer tests". Table 2 presents a proposed correlation between the ratio of M_0/M_L and sample quality. Figure 8 shows values of the ratio of M_0/M_L plotted against

Table 2. Sample quality assessed on basis of modulus values from oedometer tests.

Sample quality	Ratio $\Delta e/e_0$	Ratio M_0 / M_L
Very good to excellent	0-0.04	>2
Good to fair	0.04-0.07	1.5-2
Poor	0.07-0.14	1-1.5
Very poor	>0.14	<1

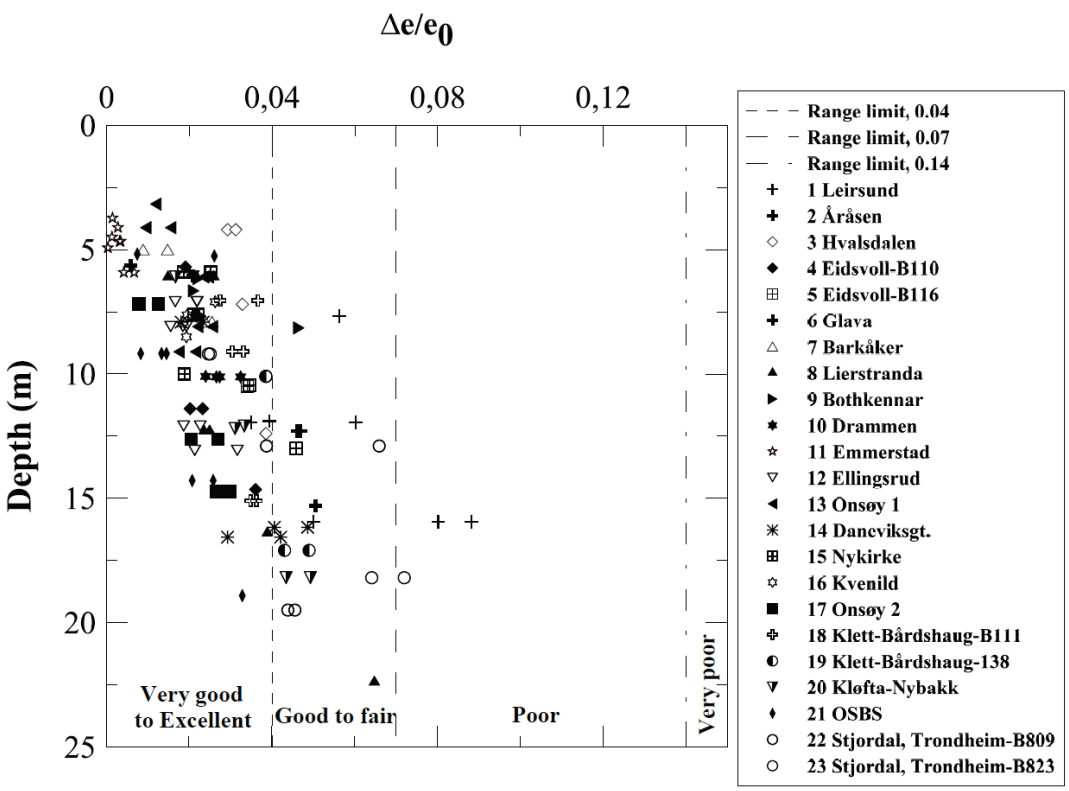

Figure 7. Quality of block samples as defined according to Lunne et al. *(1997) based on volume change observed in triaxial tests.*

depth for all oedometer tests. Both criteria given in Table 2 were used when assessing the quality of the individual samples tested. Only results of tests on samples classified good to excellent have been included in the results presented herein.

INDEX PARAMETERS USED IN CORRELATIONS

The plasticity index, I_p, has traditionally been used in Norwegian and international practice as a key correlation parameter for strength, compressibility and other engineering properties of clays, e.g. Bjerrum (1972, 1973). For leached and sensitive

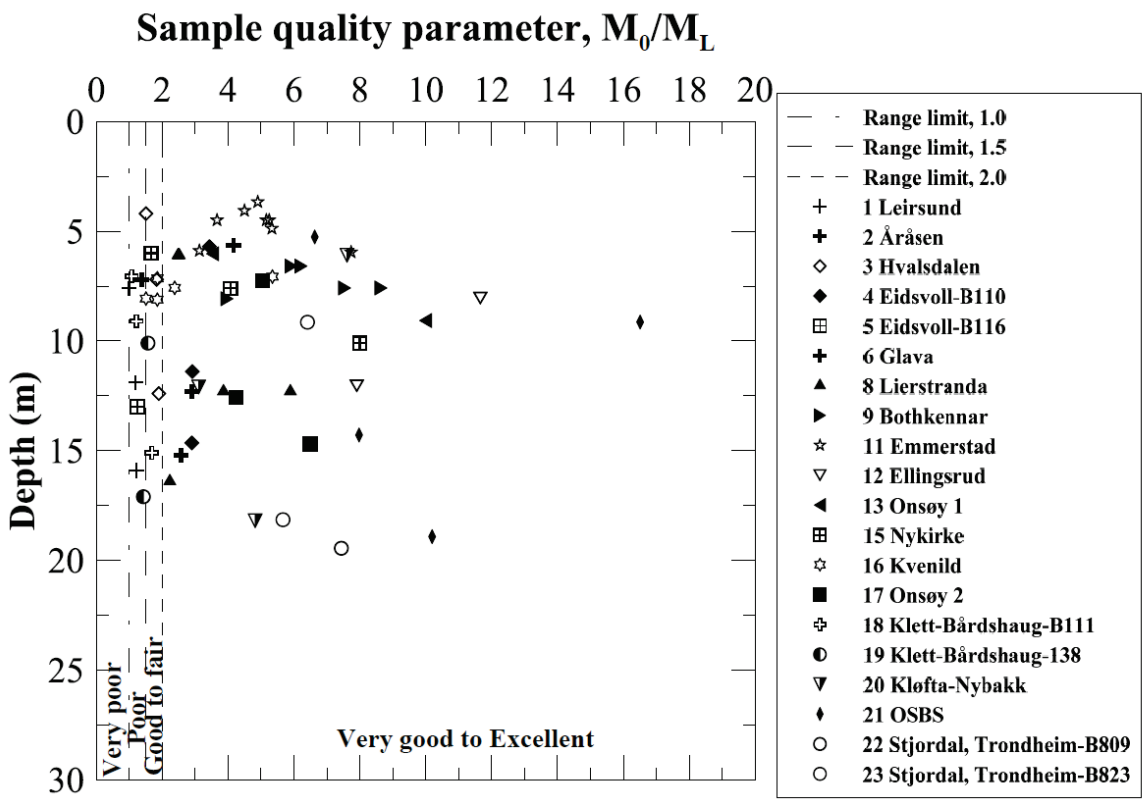

Figure 8. Ratio of maximum to minimum modulus versus depth from oedometer tests.

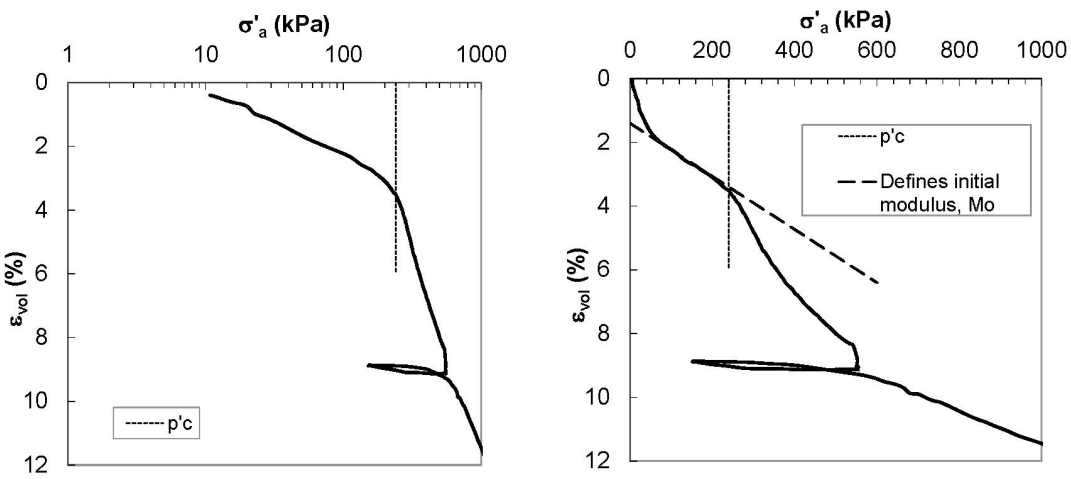

Figure 9. Example of stress-strain curve from CRS test on high quality block sample (log scale left, linear scale right.)

clays, the liquid limit, w_L, is significantly lower than that of an otherwise identical non-leached clay with normal sensitivity, which then also significantly reduces the plasticity index. As the current study includes tests on both leached sensitive (quick) clays and non-leached clays, the use of I_p as a key correlation parameter was found to correlate poorly to the results of the different test types.

After trying out various other index parameters for correlating test results, it was concluded that the best index parameter was the natural water content when it was combined with the overconsolidation ratio, OCR, determined from the oedometer tests. This is because, as will be shown later, the sensitivity has surprisingly little impact on most engineering parameters considered in this study.

RESULTS OF OEDOMETER TESTS

Figure 9 shows a typical example of stress-strain curves for a CRS-type oedometer test plotted in logarithmic scale (Figure 9a) and linear scale (Figure 9b). From the dashed line in the linear diagram it appears that after an initial "seating effect" the strain develops essentially linearly with stress, as represented by an initial modulus M_0.

The initial "seating effect", which is observed on all samples, is partly a result of less than perfect smoothness and fit between the trimmed specimens and the top and bottom caps. During trimming, a portion of the clay thickness closest to the trimmed surfaces will to some extent be remoulded or disturbed. This remoulded clay will undergo some extra volume change, especially during the initial stages of loading. This will also contribute to the "seating effect".

The tangent modulus concept proposed by Claesson (2003) was used for representing the volumetric compressibility characteristics. This represents a modification of the original concept developed by Janbu (1963). Figure 10 presents an example of how the tangent modulus develops with applied effective stress, and how key parameters were defined from each oedometer test in terms of apparent preconsolidation pressure, p'_c, and modulus values. The reason for using the term "apparent preconsolidation pressures" is to underscore that it may result from effects other than physical pre-loading, such as creep and chemical weathering. The following gives some further comments to the modulus behaviour and definitions used in Fig. 10:

1. During loading from zero to the *in-situ* vertical effective stress, σ'_{v0}, the modulus generally increases gradually and then tends to reach a plateau defined as the maximum re-loading modulus, M_0. (The low initial values mostly reflect seating effects as discussed earlier). The modulus then drops off more or less linearly to a minimum level defined as M_L, with corresponding stress defined as σ'_{ML1}. After this stress is reached the modulus increases linearly, but for some clays the modulus is constant up to a stress level defined as σ'_{ML2} before it starts to increase linearly. Janbu's modulus number, *m*, defines the rate of increase beyond this point. This line defines an M=0 intercept on the stress axis defined as p'_r , which is the same definition as used by Janbu (1963). Note that for very stiff as well as disturbed clays, p'_r may be negative.

2. The procedure used for defining the apparent preconsolidation pressure, p'_c, was first proposed by Karlsrud (1991). As seen from Fig. 10, this method simply takes the preconsolidation pressure as the average stress at which the tangent modulus starts to drop off, until it starts to climb up again along the virgin modulus line. A comparative study of different other methods for defining p'_c from oedometer tests was presented in a report by NGI (2005). This report

included the classical Casagrande (1936) method, the Becker *et al.* (1987) energy method, and a linear curvature method, which is based on point of maximum curvature of the change in tangent modulus in relation to change in strain. When applied to 15 of the oedometer tests in the database, the Karlsrud (1991) method comes out just about equal to the average of the three other methods (0.8% on the high side). A main reason for choosing the Karlsrud (1991) method in this study is its simplicity. This method of interpretation was also used for establishing the CPTU-correlations based on samples that were presented by Karlsrud *et al.* (2005).

[2] $$m_0 = \frac{\ln_{10}}{\left(\frac{Cc}{1+e_0}\right)}$$

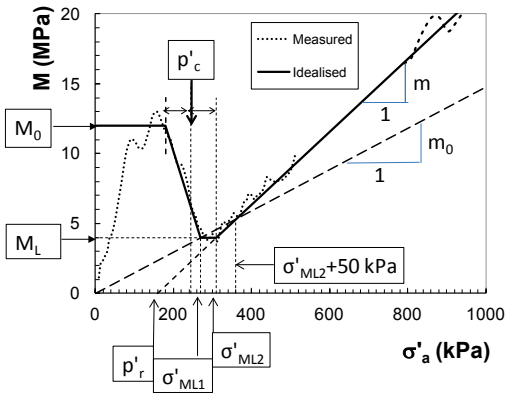

Figure 10. Definition of modulus relationships from oedometer tests.

3. The classical virgin compression index, $C_C/1+e_0$, defined by a straight line in the semi-log plot, is through Eq. [2] directly related to the modulus number for a case when $p'_r = 0$.

For correlation studies it can also be convenient to define the virgin consolidation behaviour by a modulus number, m_0, defined by assuming that $p'_r = 0$. As shown in Fig. 10, in this study it was chosen to define m_0 for each test by a line that starts at zero stress and passes through the observed tangent modulus curve at a stress corresponding to σ'_{ML2} + 50 kPa. The reason for this choice is that it may represent a typical limiting "acceptable" stress or strain level when it comes to practical consolidation-settlement problems.

Most of the oedometer tests in this study were of the (CRS) type, with a rate of straining in the range 0.2%/h to 0.7%/h. This means that it typically takes about 1 to 3 days to run a CRS test, as compared to about 1 week for the standard 24 h (IL) test. Standard IL tests were also carried out on some of the block samples. When CRS and IL tests were carried out on the same block sample, the pre-consolidation pressures derived from the IL tests were found to be 10% to 18% lower than for the CRS tests, which is a direct effect of the difference in rate of loading or rate of straining.

The vertical permeability is defined by the permeability, at zero volumetric strain (i.e. at *in-situ* void ratio), and a permeability change index defined from a semi-log plot ε_a versus k plot and given by Eq. [3]:

[3] $$\beta_k = \frac{(\log k_1 - \log k_2)}{\Delta\varepsilon_a}$$

This means that the permeability in relation to the permeability at zero strain is given by Eq. [4]:

[4] $$\log k_i = \log k_0 - \beta_k \cdot \varepsilon_a$$

Modulus correlations

Figure 11 presents normalized values of the maximum initial tangent modulus, defined as $M_0/(m_0 \cdot p'_c)$, as a function of the water content, w. This normalized modulus ratio varies from 2 to 7, and tends to increase somewhat with water content. The sensitive clays with S_t>15 fall in the same band as the

nonsensitive clays with S_t<15, but are maybe more dependent upon water content.

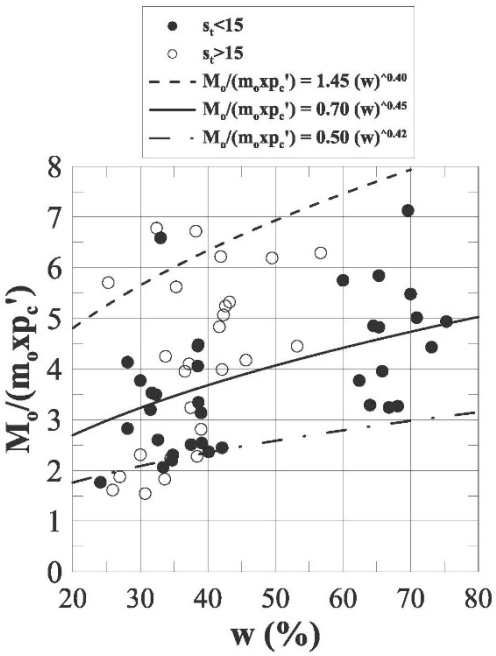

Figure 11. Normalized re-loading modulus in relation to water content.

It is possible that the lowest normalized values of $M_0/ (m_0 \cdot p'_c)$ reflects some effects of sample disturbance, as discussed in the section above dealing with sample quality. General experiences suggest that silty low plastic clays are more prone to disturbance than high water content plastic clays, which could partly explain the tendency for the increase in normalized $M_0/ (m_0 \cdot p'_c)$ with increasing water content.

Figure 12 presents values for the modulus number, m (as defined in Fig. 10) as a function of the water content. The modulus number decrease as expected with natural water content. There is a fair amount of scatter, typically ± 25% for a given water content. There is no apparent impact of clay sensitivity on the modulus number.

A typical range for m-values previously suggested by Janbu (1985) is also included in Fig. 12. These data broadly follow the same trend as the block samples, but suggest larger scatter and notably lower m-values when the water content increases beyond about 40%. Janbu's data are based mostly on oedometer tests on conventional 54 mm piston samples, implying that these data are more affected by sample disturbance. Note in this respect that one effect of sample disturbance is that both m and p_r' values decrease with increasing impact of disturbance (ref. Fig. 2).

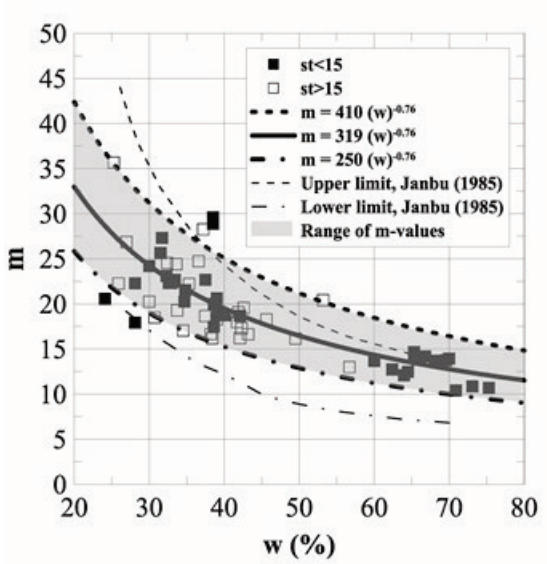

Figure 12. Modulus number, m, versus water content.

The modulus number, m, for a specific test is coupled to a specific reference pressure, p_r' (as defined in Fig. 10). Figure 13 shows that measured normalized values of p_r'/ p_c' tend to increase with increasing water content. This relationship also appears independent of the clay sensitivity. Figure 14 shows, as one would expect from Figs. 12 and 13, that p_r' increases with increasing modulus number, m.

Figure 15 shows that there is also a close correlation between the defined m_0- values (see definition in Fig. 10) and water content. For given water content the scatter is about the same as in the m-values in Fig. 12, but the m_0-values are by definition lower. In settlement analyses it is therefore very im-

portant to always use values for the modulus number, m, that are coupled with a value of p_r'. If the stress changes are only moderately larger than the preconsolidation pressure, the alternative may be to use values of m_0 coupled with p_r'= 0.

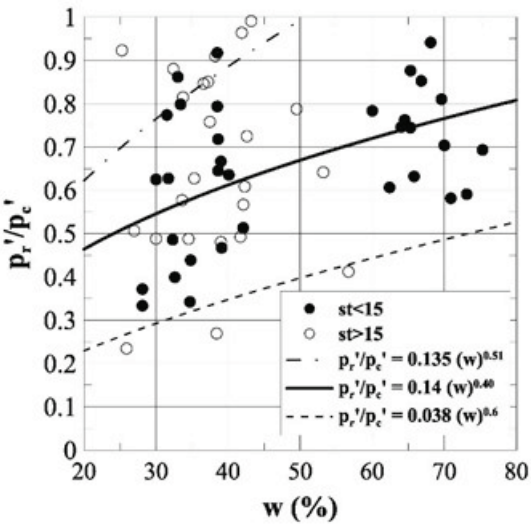

Figure 13. Normalised reference pressure, p'_r/p'_c, versus water content.

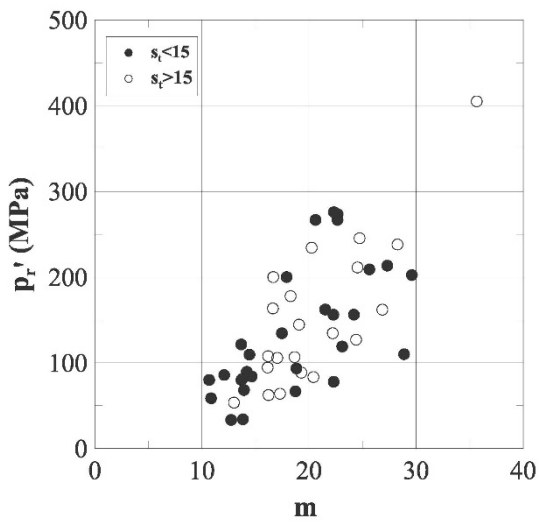

Figure 14. Combined correlation graph for m and p_r' in relation to water content.

The minimum modulus, M_L, reached in any test (as defined in Fig. 10), is related to the reference stresses and modulus number through Eq. [5].

[5] $$M_L = m \cdot (\sigma'_{ML2} - p'_r)$$

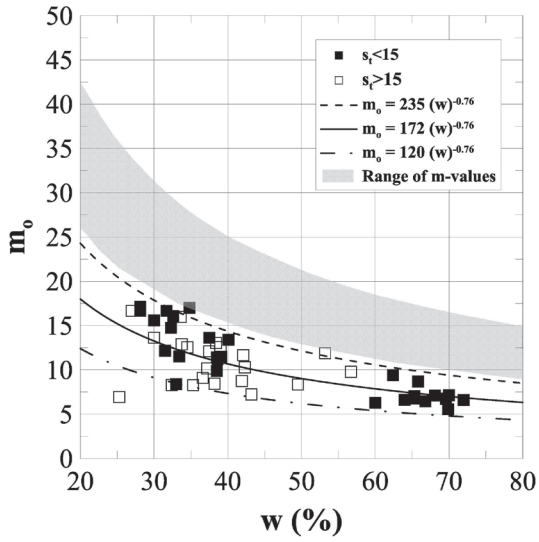

Figure 15. Comparison between modulus numbers m_0 and m.

Figure 16 shows that the normalized value of σ'_{ML2}/p'_c decreases with increasing water content. What this ratio really shows is how fast the modulus drops from the peak M_0-value to the minimum M_L-value, which is a reflection of the brittleness of the clay structure. The smaller the ratio σ'_{ML2}/p'_c, the more brittle the clay. Seen in this context, it is also rather surprising that the clays with high sensitivity (S_t>15) show no distinct difference to the clays with low to medium sensitivity (S_t<15).

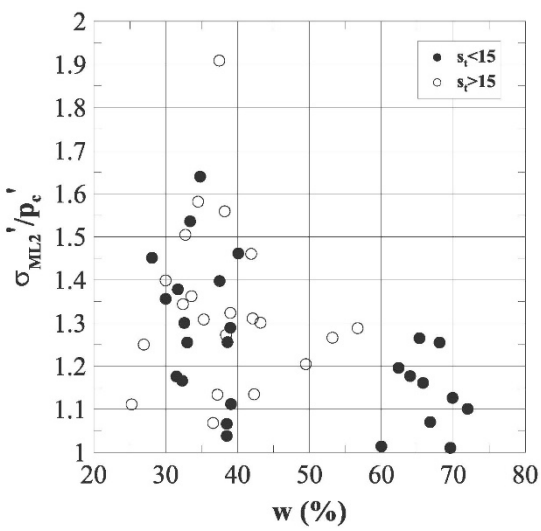

Figure 16. Measured normalized stress level σ'_{ML2}/p'_c in relation to water content.

Figure 17 shows that the stress levels at which the modulus is constant and at its minimum value before it starts to climb up again (see Fig. 10), as defined by the normalized ratio $\sigma'_{ML2}/\sigma'_{ML1}$, tend to decrease with increasing water content. Detailed examination of the data suggests that this ratio also seems to be independent of both sensitivity and OCR of the clay.

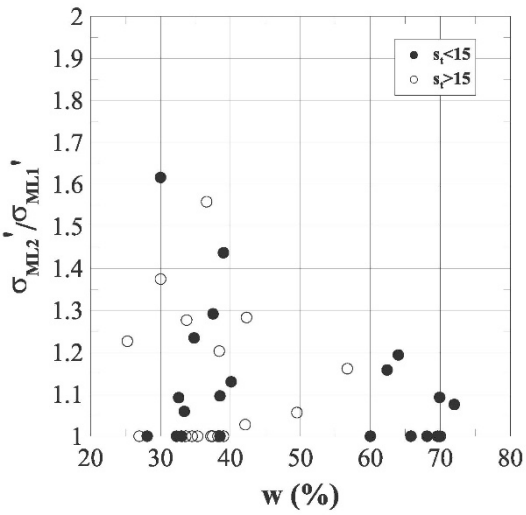

Figure 17. Normalized stress ratio $\sigma'_{ML2}/\sigma'_{ML1}$, in relation to water content.

Permeability correlations

Figure 18 presents values of *in-situ* vertical permeability, k_0, in relation to the water content. These k_0-values were determined by extrapolating directly measured permeability values at two different stress or strain levels using Eqs [3] and [4], combined with values calculated continuously during the CRS oedometer tests. Sandbækken *et al.* (1986) have described the procedure used for this purpose. The overall average and one standard deviation (std. dev.) of the deduced *in-situ* permeability values up or down correspond to:

$$k_0 = 1.77 \cdot 10^{-9} \pm 1.14 \text{ m/s}$$

A linear regression analysis of all data gives an average line ± 1 std. dev. that suggest only a slight increase in permeability with water content. The main reason for this is that the clay content also increases with water content (Fig. 6). Tavenas *et al.* (1983) and Mesri *et al.* (1994) also showed that the permeability tends to increase with water content or voids ratio.

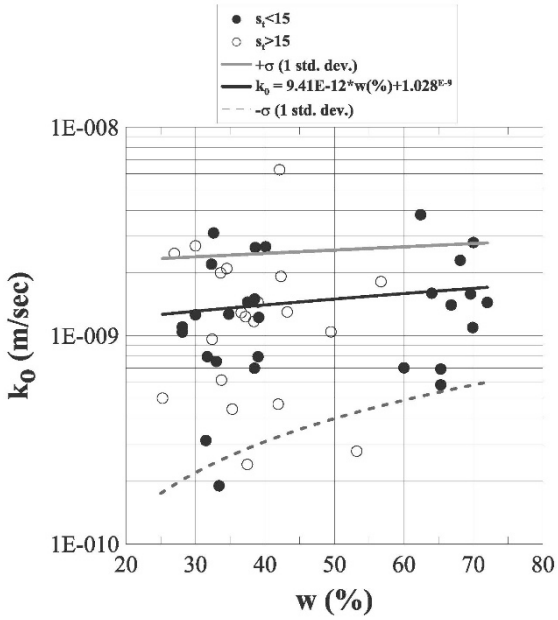

Figure 18. Permeability, k_0, determined from oedometer tests in relation to water content.

The scatter in the permeability data suggests that not only void ratio or water content impacts the permeability of clays, but also a number of other factors such as type of clay minerals and the complete grain-size distribution curve. Tavenas *et al.* (1983) and Mesri *et al.* (1994) have drawn similar conclusions.

Figure 19 suggests that the permeability change index, as defined by Eq. [3], decreases somewhat with increasing water content, when the four lowest values are disregarded. The measured values lie mostly within the range 3 to 6, which tends to agree with the findings of Mesri *et al.* (1994).

Results of undrained tests

The block samples used for CAUC and CAUE triaxial testing were trimmed to a height of 140 mm and diameter of 70 mm. Prior to undrained shearing all triaxial test

specimens were anisotropically consolidated to a stress level corresponding to the best estimate of the *in-situ* vertical and horizontal effective stress, e.g. $\sigma'_{ac} = \sigma'_{v0}$ and $\sigma'_{rc} = \sigma'_{h0} = K_0 \cdot \sigma'_{v0}$. The undrained triaxial tests were carried out under a constant rate of axial straining of $\dot{\varepsilon}_a$= 1.43%/, corresponding to a rate of shear strain of $\dot{\gamma}$= 2.15%/hr.

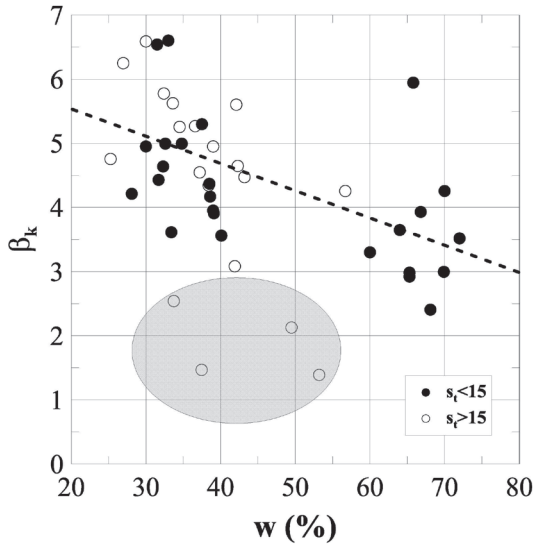

Figure 19. Permeability change index, β_k, as function of water content.

The DSS specimens were trimmed to 16 mm in height and diameter of 65 mm. They were run in the standard NGI/Geonor DSS apparatus using a reinforced rubber membrane. DSS specimens that were anticipated to be overconsolidated, were first loaded to a stress that was about 80%-90% of the assumed preconsolidation pressure, and then loaded back to $\sigma'_{ac} = \sigma'_{v0}$ prior to undrained shearing. The purpose of this consolidation procedure is to try to restore the correct *in-situ* horizontal effective stress in the specimen, which can have a significant impact on the DSS test result. Since the late 1970's this been a standard procedure used at NGI for DSS testing of overconsolidated clays. The DSS specimens were sheared undrained by maintaining constant volume (DSS-CCV tests, where CCV stands for consolidation followed by shearing at constant volume). The rate of shear straining corresponds to $\dot{\gamma}$ = 4.7%/h, which is about four times faster than in the triaxial tests. The original background for this choice was to give comparable times to reach failure in the triaxial CAUC and DSS tests.

Peak undrained strength

For systemizing data on undrained strength it is most convenient to work with the normalized strength ratio defined as s_u/σ'_{ac}. Based on undrained tests on clay samples that were artificially overconsolidated in the laboratory prior to undrained shearing, Ladd and Foot (1974) proposed that the normalized strength is closely related to the OCR, through Eq. [6]:

$$\frac{s_u}{\sigma'_{ac}} = S \cdot OCR^m \qquad [6]$$

The principle behind Eq. [6] was developed by Ladd and Foott (1974), called "Stress History and Normalized Soil Engineering Properties" (SHANSEP).

Although the SHANSEP concept was originally developed to study the undrained strength of artificially overconsolidated clays, it was considered that the same framework would be useful for comparing undrained strengths derived from undisturbed high-quality block samples. The OCR-values used in the correlations stem from preconsolidation pressures derived from the CRS type oedometer tests. At one site (Kvenild) only IL tests were available. Based on the comparison between IL and CRS tests given above, the pre-consolidation pressures from these IL tests were upgraded by 12% to correspond to CRS tests results.

Figure 20 presents the normalized strength versus OCR for the CAUC triaxial compression tests. The figure shows the range of values of the constant, S, and power, *m*, that captures all the data. The average line and the approximate upper and lower ranges are represented by the following:

Average: S= 0.30, *m*= 0.70
Upper: S= 0.35, *m*= 0.75
Lower: S= 0.25, *m*= 0.65

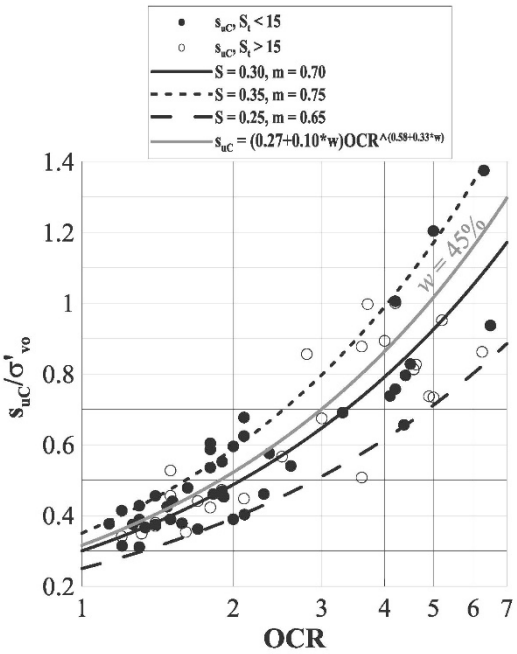

Figure 20. Normalised strength versus OCR from CAUC tests.

The data in Fig. 20 show some scatter. For a given OCR the normalized strength ratio typically deviates by ± 20% up and down. There is no apparent effect of clay sensitivity. Linear regression analyses of the data suggest some dependency on water content as given by Eq. [7]. Broadly speaking, for a given OCR the variation in water content can explain about ± 10%, or half of the variation in normalized strength.

[7] $s_{uC} = (0.27 + 0.10 \cdot w) \cdot OCR^{(0.58+0.33 \bullet w)}$

One can always argue that due to possible variation in sample quality there is some uncertainty in both the strength and OCR values determined for each specimen. However, considering the good quality of all samples tested, variable quality is not likely to explain the remaining scatter in the results. As also concluded in relation to compressibility characteristics, the main reasons for the scatter most likely lie in the detailed mineralogical and geochemical characteristics of the clays, including possible effects of cementation and/or clay fabric.

Ladd and Foot (1974), Ladd (1991) and Ladd and DeGroot (2003) have presented normalized strength for different artificially overconsolidated clays. Their data suggest a range of S- and m-values that are surprisingly similar to what is shown in Fig. 20. Figure 21 shows normalized strength data for the CAUE triaxial extension tests. These results also show significant scatter for a given OCR, as given by the following values for S and m:

Average: S= 0.12, *m*= 0.80
Upper: S= 0.16, *m*= 0.94
Lower: S= 0.08, *m*= 0.75

Although the scatter in the normalized CAUE strengths in Fig. 21 is even larger than for the CAUC strengths in Fig. 20, around ± 35%, the linear regression analysis suggests that the observed scatter in this case is mainly related to the variation in water content, as reflected by Eq. (8).

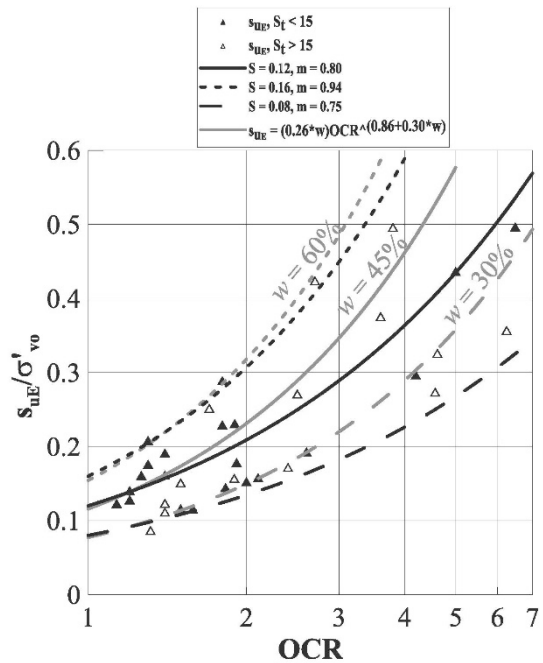

Figure 21. Normalised strength versus OCR from CAUE tests with results of regression analysis on impact of water content.

[8] $s_{uE} = (0.26 \cdot w) \cdot OCR^{(0.86+0.30 \bullet w)}$

$$s_{uE} = (0.26 \cdot w) \cdot OCR^{(0.86+0.30 \cdot w)}$$

This is an observation of some interest. In a CAUE test, the stress rotation leads to much more degradation of the original clay structure by the time the peak strength is reached as compared to a CAUC test. Consequently, the strength is to a larger extent reflected by the water content at the start of undrained shearing.

Only a limited number of DSS tests were carried out on the block samples. Thus, the normalized strength diagram in Fig. 22 for the DSS tests contains fewer data points. The scatter is also large for this case as indicated by the following values for S and m:

Average: S = 0.22, *m*= 0.80
Upper: S = 0.26, *m*= 0.90
Lower: S = 0.16, *m*= 0.75

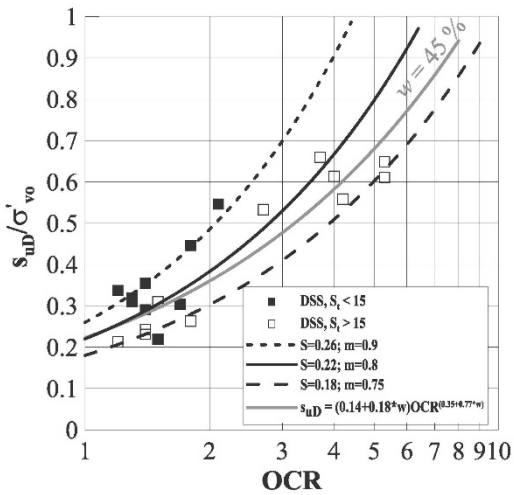

Figure 22. Normalised strength versus OCR from DSS tests with result of regression analysis on impact of water content.

As for the CAUE tests, the linear regression analysis suggests that a large part of the scatter is related to the variation in water content, as reflected by Eq. [9]. There is a tendency that the sensitive clays with S_t > 15 show the lowest normalized strengths.

[9] $s_{uD} = (0.14 + 0.18 \cdot w) \cdot OCR^{(0.35+0.77 \bullet w)}$

Figure 23 shows the anisotropic strength ratios of s_{uE}/s_{uC} and s_{uD}/s_{uC} seen in relation to the OCR. The scatter in the data is, however, fairly large. The CAUE tests suggest very low values of s_{uE}/s_{uC} = 0.22 to 0.32 for the sensitive clays with OCR < 2, increasing to an average around 0.42 (range: 0.32 to 0.52) at higher OCR values. For clays with low sensitivity, the anisotropy ratio seems more independent of OCR and is typically given by s_{uE}/s_{uC}=0.4 (range 0.30 to 0.52).

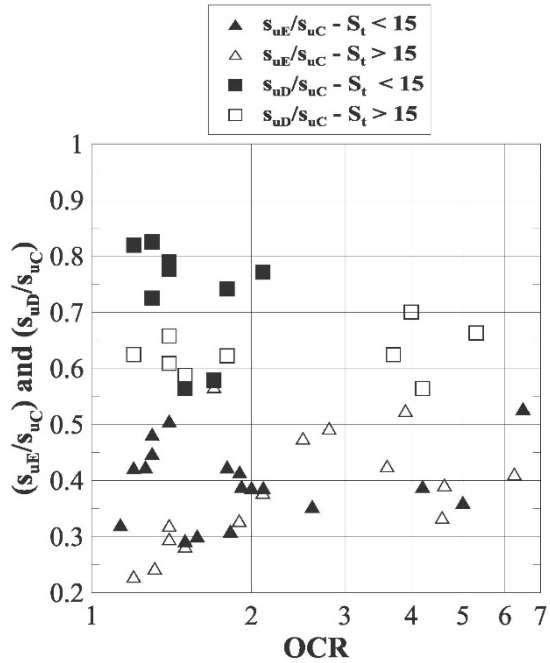

Figure 23. Anisotropic strength ratios against overconsolidation ratio.

The more limited number of DSS tests included in this study suggest s_{uD}/s_{uC} = 0.56 to 0.66 for the sensitive clays with S_t > 15, and s_{uD}/s_{uC} = 0.57 to 0.82 for clays with S_t < 15. Also, in this case there is a tendency that the sensitive clays show the lowest strength. There seems to be no clear dependence of the s_{uD}/s_{uC} ratio upon the OCR-value.

Figure 24 shows that the anisotropic strength ratios s_{uE}/s_{uC} and s_{uD}/s_{uC} tend to increase with the water content of the clays, and as typically represented by Eqs (10 and 11).

[10] $$\frac{s_{uD}}{s_{uC}} = (0.00447 \cdot w) + 0.4547$$

[11] $$\frac{s_{uE}}{s_{uC}} = (0.0029 \cdot w) + 0.277$$

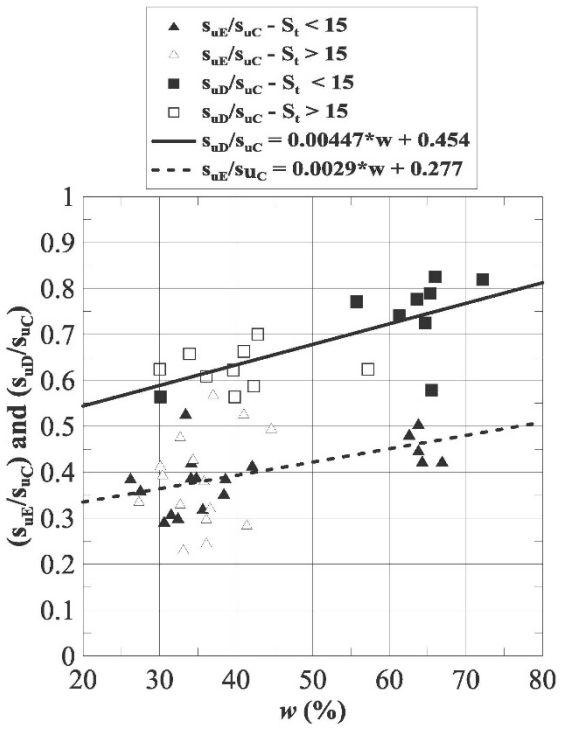

Figure 24. Anisotropic strength ratios against water content.

Table 3, taken from Lunne and Andersen (2007), compares anisotropic strength ratios deduced from some of the block samples to values obtained for the same clays from testing of 54 and 95 mm piston samples.
The data clearly suggest less anisotropy effect for the poorer quality piston samples than for the block samples. The explanation for this effect for these poorer quality samples is that the principal stress rotation that takes place in CAUE and DSS test leads to a gradual breakdown of the clay structure, and thus less impact of sample quality on the strengths from-CAUE and DSS tests as compared to a CAUC test.

Post-peak undrained strength

The stress-strain curves for most clays tested show a steady decline in shear stress following a peak, and a decline that has not levelled off at the end of the tests. In this post-peak residual strength, τ_{pp} at the same level of shear strain in both triaxial and DSS tests, and taken as γ_{pp} =20% as shown in Fig. 25. For the triaxial tests this means an axial post-peak strain corresponds to ε_{app}= $\gamma_{pp}/1.5$ = 13.3%.

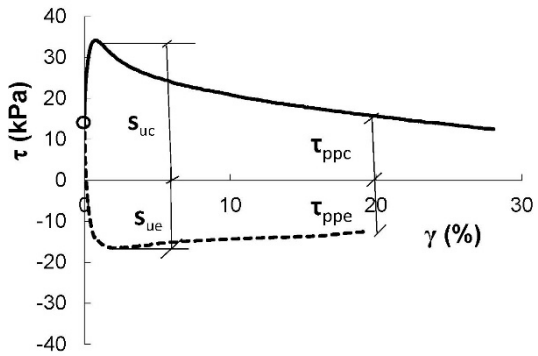

Figure 25. Definition of peak and post-peak strength from triaxial tests.

By normalizing the post-peak residual strength by the peak strength for all test types as shown in Fig. 26, the following observations can be made:

- The post-peak strength reduction is largest for the CAUC tests and smallest for the CAUE tests, with DSS somewhere in-between.

Table 3. Comparison between anisotropic strength ratios observed on piston sample as compared to high quality block samples (from Lunne and Andersen, 2007).

Anisotropy ratio	Block samples			54/95 mm samples		
	Mean	n	r^2	Mean	n	r^2
s_u^{DSS}/s_u^{CAUC}	0.69	20	0.989	0.74	16	0.987
s_u^{CAUE}/s_u^{CAUC}	0.42	19	0.984	0.50	12	0.974

- As expected, for all types of tests the post-peak strength reduction is larger for high-sensitivity than low-sensitivity clays.
- The post-peak strength reduction decreases with increasing OCR.

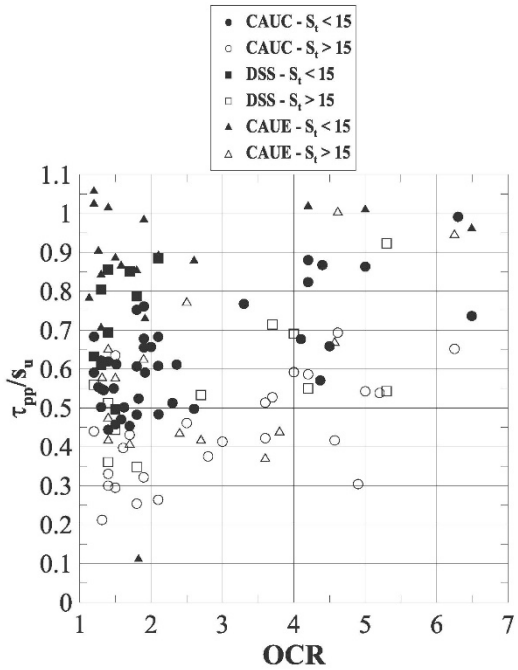

Figure 26. Normalised post-peak strength versus overconsolidation ratio for all tests.

The post-peak strengths show no clear correlation to the water content.

Effective strength parameters

The Mohr-Coulomb (M-C) effective strength failure envelope is herein defined by an effective friction angle, φ, and attraction, a. The attraction, a, was defined by Janbu (1963) as the intersection of the M-C failure line on the stress axis. The attraction is related to the effective cohesion intercept, c, on the shear stress axis by the expression $a = c\cos\varphi$.

Herein only the maximum set of effective shear strength parameters that can be defined from the each test, is presented which generally occur at fairly large strains in an undrained test, well beyond the level when the peak undrained strength is reached. It can in some cases be difficult to define combined values of maximum friction angle, φ, and attraction, a, from a single test. In such cases, it may be necessary to use the results of several different tests to define these parameters. Figs. 27 to 29 show examples of effective stress paths from different sites. For both the normally consolidated Onsøy 2 clay site (Fig. 27), and the overconsolidated quick clay from the Klett-Bårdshaug site (Fig. 28), a common M-C failure line that captures both the CAUC and CAUE tests could be well defined. The attraction intercept for these two cases is very small.

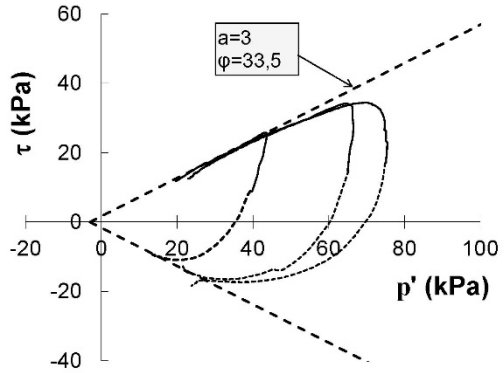

Figure 27. Effective stress paths from triaxial tests on Onsøy 2 clay.

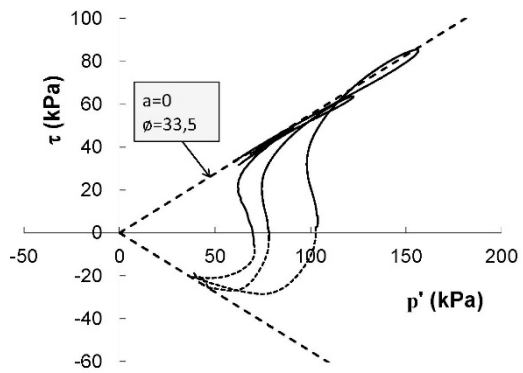

Figure 28. Effective stress paths from triaxial tests on Klett-Bårdshaug quick clay

From the CAUC and CAUE tests on Emmerstad quick clay shown in Fig. 29, no unique failure line can be defined. If the effective friction angle is taken as the same φ = 33.5° for all tests, the attraction intercept will in this case vary from 5 to 25 kPa.

The lowest attraction applies to the CAUE tests, and the highest to two of the CAUC tests. The shape of the CAUC effective stress paths suggest that the attraction partly breaks down at large strains, and approaches the minimum value. A possible explanation for this behaviour is that the Emmerstad quick clay to some extent is cemented.

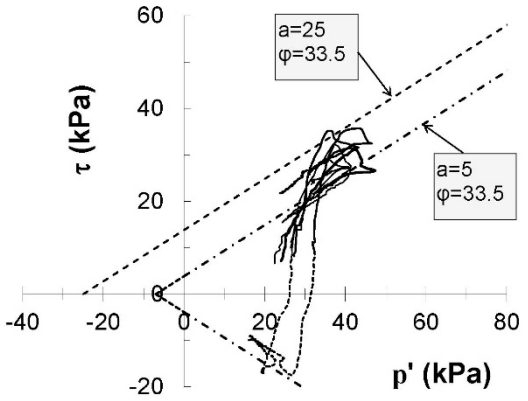

Figure 29. Effective stress paths from triaxial tests on Emmerstad quick clay.

It is complicated to compare effective stress parameters derived from the different clays tested when both an apparent friction angle and an attraction is used to define results of individual tests. It was therefore found more convenient for each test to define an apparent maximum friction angle, φ_{max}, by fitting a tangent M-C line to the effective stress paths assuming zero attraction (i.e. a = 0). Using this approach, Fig. 30 shows that the deduced values of φ_{max} tend to increase with water content. Some of the sensitive clay specimens with water content in the range 36% to 53% show higher values than the non-sensitive clays, which is a bit surprising.

The radial effective stress is not known in a standard DSS test. It is therefore not possible to deduce precisely the true mobilized maximum effective friction angle in DSS tests. However, special DSS tests instrumented to attain the radial stress suggest that at large strains the ratio of radial to axial effective stress in the DSS apparatus approaches unity (e.g. Dyvik *et al.* 1987). If that is the case, it can be shown that the maximum effective friction angle that is mobilized in the specimen is given by Eq. [12].

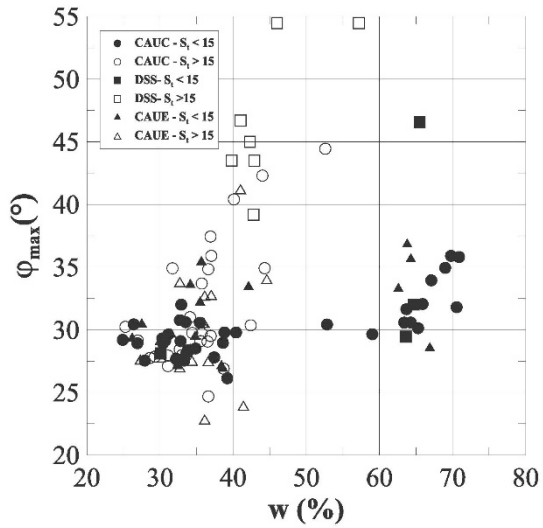

Figure 30. Maximum effective friction angle from triaxial and DSS tests in relation to water content assuming zero attraction intercept.

[12] $$\sin\varphi_{max} = \tan\delta_{max}$$

where:

$\tan\delta_{max}$ is the maximum value of τ_h/σ'_a during the test. Where there are direct comparable results, the deduced values of φ_{max} from the DSS and CAUC tests generally agree closely (Fig. 30). Six DSS values for samples with water content 40% to 57% show however, relatively high friction angles as compared to the CAUC tests. These values apply to the Emmerstad quick clay deposit, and tend to confirm that this clay is influenced by cementation effects.

Shear strain at failure

To enable a direct comparison between triaxial and DSS tests, Fig. 31 shows the shear strain at failure, γ_f, for all tests and test types in relation to OCR.. Some observations that may be noted are:

- The CAUC tests show as expected the smallest strain at failure, and a fairly

clear trend for γ_f to increase with OCR, which is not so obvious from the CAUE and DSS tests

- The shear strain at failure is in general smallest for the clays with S_t>15 and as observed less than 3%
- The CAUE tests on low sensitivity clays show the largest shear strain at failure, but for the clays with S_t>15 are only slightly larger than the comparable CAUC tests
- The DSS tests fall in-between the CAUC and CAUE tests.

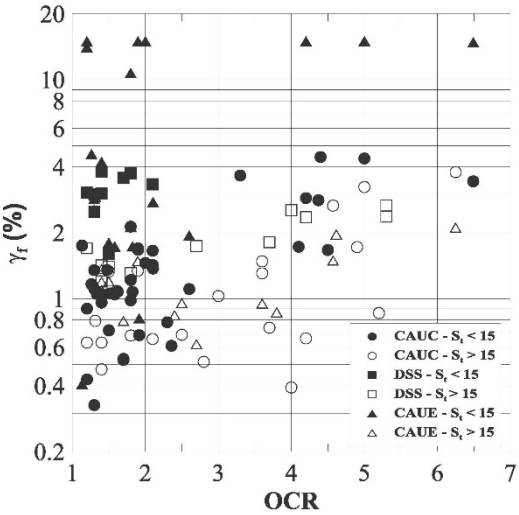

Figure 31. Summary of strain at failure versus OCR, all tests.

The data suggest no clear dependency between strain at failure and water content for any test type.

Undrained shear modulus

For presenting shear modulus data it was in this study chosen to use the undrained secant shear modulus as a basis, to study how that depends on the imposed shear stress level for all tests types. Fig. 32 illustrates the definition of the secant shear modulus at different degrees of strength mobilization. Notice that for triaxial tests, where undrained loading starts at an initial shear stress defined by $\tau 0 = (\sigma a0 - \sigma r0)/2$, the degree of shear strength mobilization refers to the applied increase in shear stress to reach failure, $\Delta\tau f$, as defined in Fig. 32.

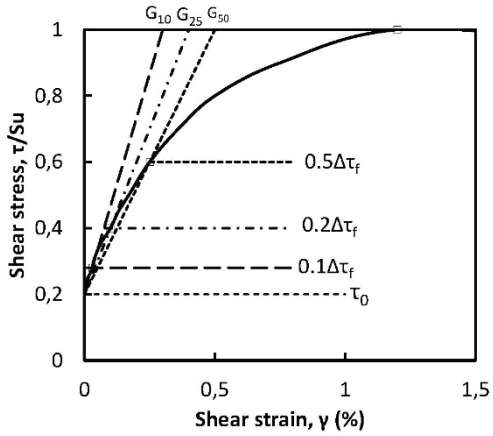

Figure 32. Definition of secant shear modulus at different degrees of shear strength mobilisation.

Figure 33 shows values for the G_{50} modulus normalized with respect to the absolute value of the peak undrained shear strength in the respective tests. It may appear surprising that the CAUE triaxial tests show the highest values of G_{50}/s_u and the CAUC tests the lowest. The main reason is that the undrained shear strengths are lowest for the CAUE tests. Figure 33 otherwise suggests that:

- G_{50}/s_u decreases with OCR
- G_{50}/s_u tends to be higher for the high-sensitive clays with S_t > 15 than for clays with S_t < 15
- The test data suggest that there is no clear correlation with water content.

Figure 34 shows that when the G_{50} values are normalized to the *in-situ* vertical effective stress rather than the undrained shear strength, the different test types group more together although scatter is still large. The CAUE tests still tend to give G_{50}/σ'_{v0} values on the high side, with G_{50}/σ'_{v0} values from the CAUC and DSS tests being more similar.

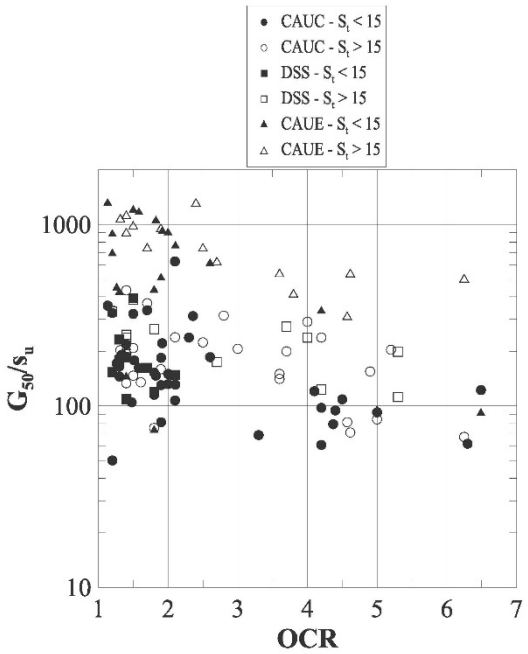

Figure 33. Values of shear modulus at 50% mobilisation normalised to the undrained strength (G_{50}/s_u) in relation to OCR, triaxial tests.

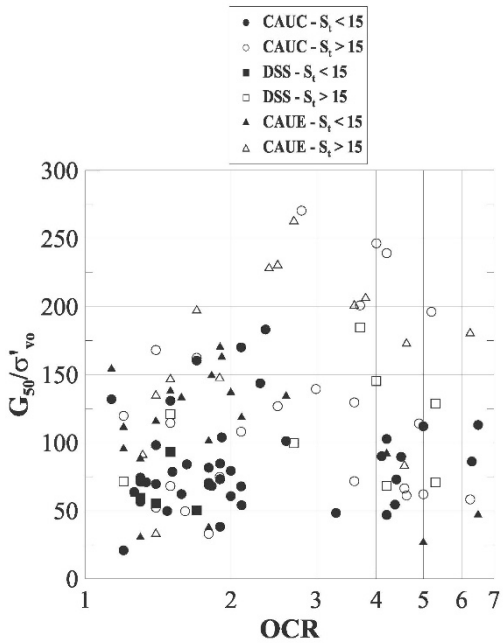

Figure 34. Values of shear modulus at 50% mobilisation normalised to in-situ *vertical effective stress, ($G_{50}/\ \sigma'_{v0}$) in relation to OCR, triaxial tests.*

APPLICATION TO DESIGN

There is no doubt that laboratory tests on high-quality block samples give increased apparent pre-consolidation pressure, shear modulus and undrained strength of clays as compared with tests on poorer quality piston samples. As most geotechnical designs in Norway during the past 50 years or so were based on piston samples, it is necessary to carefully consider if there will be a need to adjust the material factors and design principles if the design is based upon higher quality samples. This issue is briefly discussed in the following in relation to settlement and stability analyses.

Settlement analyses

As was shown in the earlier section on oedometer results, tests on high-quality block samples enhance the pre-consolidation pressure, but also give a more dramatic decrease in tangent modulus when the pre-consolidation pressure is exceeded. This more pronounced structural breakdown also causes enhanced creep rates, as compared to poorer quality samples, when the pre-consolidation pressure is exceeded. For settlement analyses based on block samples, it therefore becomes even more important to account for creep in the analyses than for analyses based on poorer quality samples. The impact of creep is illustrated by Fig. 35,

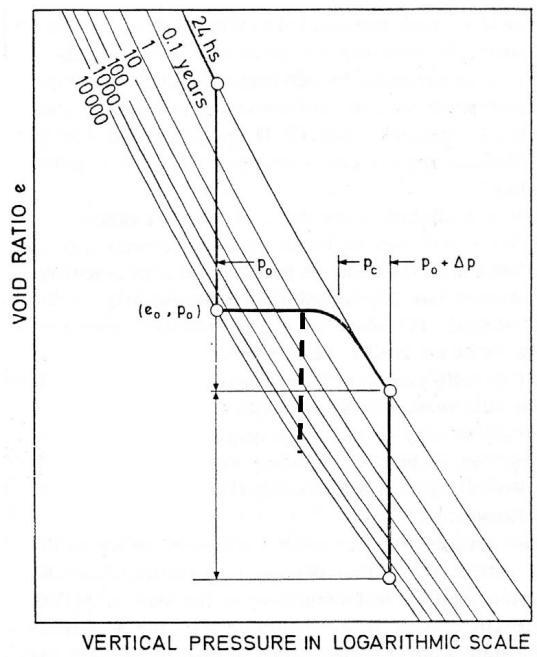

Figure 35. Illustration of impact of creep or rate of loading on volume changes in oedometer test (based on Bjerrum 1967).

taken from Bjerrum (1967), in which it is also illustrated that significant creep deformations will also occur for stresses well below the apparent pre-consolidation pressure.

In an attempt to get some direct field evidence of how much settlement and creep would occur as function of load level Bjerrum (1967) undertook a study of settlement of buildings in Drammen, Norway, where there was a rather homogeneous clay layer covering large areas. Figure 36 presents time-settlement curves for the various buildings. The figure also shows values for the net load imposed by the different buildings, presented as degree of mobilization (in percentage) of the assumed apparent pre-consolidation pressure. This is expressed by the loading ratio $R = \Delta p/(p'_c - \sigma'_{v0})$, where Δp represents the net load imposed by the building.

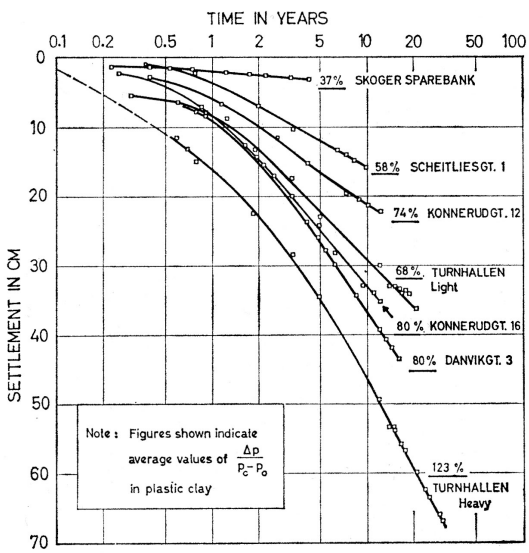

Figure 36. Measured settlement versus time for buildings in Drammen (from Bjerrum 1967).

Figure 37 presents the settlement observed after 1 and 5 years. The data suggest a pronounced increase in settlements when the loading ratio increase beyond approximately R = 0.5 for the 1 year data and R = 0.4 for the 5 year data. A tentative conclusion that can be drawn on this basis is that long-term creep settlements will be relatively small and acceptable if the loading ratio is less than about R = 0.4.

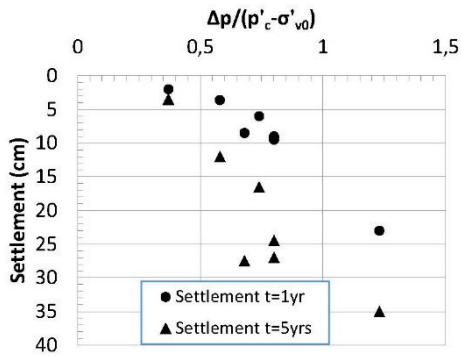

Figure 37. Settlements measured after 1 and 5 years for buildings in Drammen (based on data from Bjerrum 1967 in Figure 7.2).

Stability analyses

Undrained stability analyses have in the past often been based on semi-empirical approaches where the undrained strength has been taken from *in-situ* remote vane borings, and a correction factor is introduced to the calculated safety factor. Bjerrum (1972, 1973) proposed a correction or reduction factor to the calculated safety factor that increases with increasing plasticity index of the clay. Aas *et al.*, (1986) proposed an alternative approach where the correction factor was based on the normalized vane shear strength ratio, s_{uvane}/σ'_{v0}. This correction factor increases with increasing values of s_{uvane}/σ'_{v0}.

In undrained stability analyses, it is important to account for possible impact of the following factors on the input parameters and the calculated safety factor:

- Time to failure or rate of loading
- Progressive failure or stress-strain compatibility
- Triaxial versus plain strain conditions
- Three-dimensional geometry effects

- Effects of swelling on undrained strength (at bottom cuts or excavations).

It is well established (e.g. Bjerrum 1972; Andersen and Stenhamar 1982; Lacasse 1995; Ladd and DeGroot 2003) that the rate of loading or time to failure has a significant impact on the peak undrained strength in all types of strength tests. Figure 38 shows impact of time to failure on the undrained shear strength as observed in CAUC triaxial tests on a range of different clays as presented by Lunne and Andersen (2007). The strengths are normalized to the strength observed in a standard CAUC test loaded to failure in typically 140 minutes. For slow long-term loading, as relevant for many field situations, the data suggest that the undrained strength may level off at about 85% of the strength observed in a standard CAUC test. There are, however, many practical design situations where the peak load acts in a much shorter time than in a standard CAUC test, and at a much higher strength. This applies for instance to earthquake loading, peak wave and wind loads and other variable live loads. As an example, Fig. 38 suggests that the undrained strength can be 30% to 60% larger than what is observed in a standard CAUC test if the load acts for only for 15 seconds.

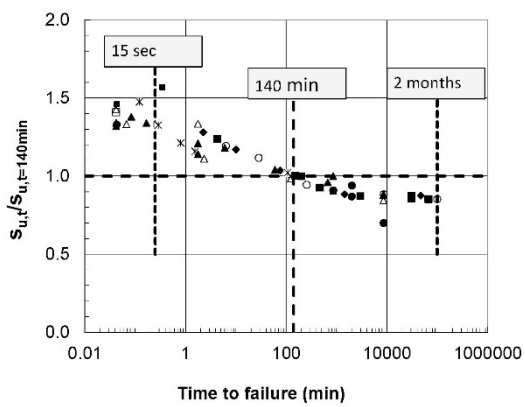

Figure 38. Effects of time to failure on undrained strength, based on Lunne and Andersen (2007).

Due to the difference in stress-strain behaviour, including shear strain at failure and post-peak strength reduction, it will in general be necessary in stability analyses to ensure that there is reasonable compatibility in terms of stresses and strains along a potential failure surface. One way of dealing with this in a limit equilibrium analyses, is to define the relevant undrained strength in CAUC, CAUE and DSS tests for either of the following:

1) A shear strain level that is the same in all test types and corresponds to the shear strain at failure in the CAUC test, which will be the smallest. At that shear strain, the peak strength is rarely reached in CAUE and DSS tests.

2) When the average shear stress at a given shear strain level in the three test types, defined as $\tau_{ave} = (\tau_C + \tau_E + \tau_D)/3$, is at its maximum.

Karlsrud (2003) as well as Ladd and DeGroot (2003) have both suggested that the second approach may be the most reasonable to use.

Over the past 10 years or so, significant efforts were made to try to account directly for strain- softening effects in stability analyses. Most of the focus has been on the use of advanced soil models in finite element analyses that can follow the post-peak behaviour of the clay up to relatively large strains. The problem has however, turned out to be quite challenging. The most difficult issue is how to properly handle strain localization and shear band development as discussed by for instance, Andresen *et al.* (2002); Jostad and Andresen (2004); Jostad *et al.* (2006), Grimstad *et al.* (2010); Jostad and Grimstad (2011). Further experiences with back-analyses of actual failures are needed to see how well these models can capture reality.

As a further comment to the possible importance of strain-softening, in 1972 NGI undertook a test fill on quick clay at the Ellingsrud site on the outskirts of Oslo, as

described by Bjerrum (1973). As shown by the typical cross section in Fig. 39, some excavation and trenching were carried out under the fill and in the toe, area of the fill to try to provoke a failure, but no failure occurred. The stability of the embankment was at that stage calculated on the basis of CAUC, CAUE and DSS tests undertaken on 95 mm piston samples. This gave a calculated safety factor of SF = 1.12 based on the peak laboratory strengths. Based on that, Bjerrum (1973) concluded that "this indicates that progressive failure is a factor of minor importance, even in this high sensitive clay."

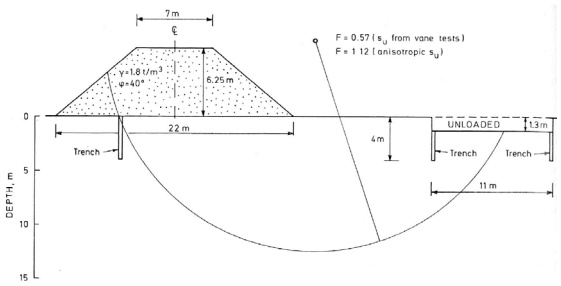

Figure 39. Cross-section of test fill at Ellingsrud (after Bjerrum 1973).

Bjerrums's conclusion may still be questioned. The laboratory tests on the 95 mm piston samples were most likely impacted by some sample disturbance effects. The strengths used could probably be on average about 10%-20% lower than what would be obtained for high quality block samples (ref. Table 1). Furthermore, possible three-dimensional 3D effects were not accounted for. The embankment was 72 m long, or about four times its average width, which could lead to 10% or so geometry effect on the calculated safety factor. In addition, no account was taken of rate effect and plane strain versus triaxial loading conditions. These two last factors may approximately cancel each other out. Thus the real theoretical of safety factor based on peak strength from block samples and geometry effects could have been more like 1.1·(1.1 to 1.2)·1.1.2 = 1.35 to 1.47 and not 1.12 as given by Bjerrum (1973).

A further interesting aspect of the Ellingsrud test fill is that it was eventually brought to failure in 1974 by extending the trench beyond the toe to a depth of 7 m. Back-analyses of that failure were presented by DiBiagio and Stenhamar (1975), which gave a calculated safety factor at failure in the range of SF = 0.87 to 1.09, based on the best estimate at the time of the actual undrained strength. The range reflects different assumptions regarding the strength in the upper about 4 m thick weathered clay crust. Correcting this safety factor for likely sample disturbance effects and 3D effects, the calculated safety factor becomes 1.05 to 1.43. This could suggest that strain softening or progressive failure did have some impact on the failure.

The Ellingsrud example illustrates that to draw clear conclusions from back-analyses of actual failures and if or how that can explain the impact of isolated effects like sample quality, strain- softening, rate, and geometry effects is far from straight forward. The uncertainty in such back-analyses could be reduced in the future if they are based on undrained strengths derived from high-quality (block) samples. Assuming the design is based on results of tests on high quality block samples as presented herein, Table 4 presents a tentative recommendation for the amount of correction that may be relevant to the different peak undrained strengths to account for stress-strain compatibility or progressive failure effects. This strictly pertains to engineering problems, like embankments and footings, where the potential failure surface is reasonably well balanced with respect to involving both active (compression), passive (extension), and DSS modes of shearing.

It is well documented that the undrained strength is larger in plane strain tests than in the standard triaxial tests. Data summarized by Ladd (1991) shows for instance, on average 9% larger strength for compression-type loading and 22% larger for extension-type loading. This aspect has to the

author's knowledge, not been commonly accounted for in design practice. Such potentially positive impact on stability should therefore be used with caution.

Table 4. Suggested reduction (in %) of peak undrained strength from high quality block samples to account for stress-strain compatibility and strain softening.

Test type	Highly sensitive clays $S_t > 15$	Low sensitive clays $S_t < 15$
CAUC	10-15	0-10
CAUE	0-5	0
DSS	5-10	0-5

Three-dimensional or geometry effects are normally accounted for in analytical bearing capacity equations for footings and deep excavations. For slopes and cuts it has been less common to account for possible 3D effects, but there should be no reason not to do so. For slopes of limited length it is suggested to use a simplified correction of the infinite case corresponding to the relative impact on the bearing capacity number, N_c, as applicable for footings or bottom heave stability analyses. That means, for a potential failure body, the definition of its thickness, width and length, and comparison of stability numbers for the assumed length to that of an infinite case. As an example, for a case where the depth and width of a failure are approximately equal, the stability number N_c will increase by 20% as compared to the infinite case if the length of the failure body is about the same as the assumed width.

Full 3D continuum finite element analyses should in principal give a more correct answer to 3D effects. Karlsrud and Andresen (2008) showed, as an example, very close agreement between safety factors computed using full 3D FEM of a deep strutted excavation, with what was obtained from the closed form solutions for bottom heave stability.

The clay below the bottom of an excavation will start swelling once the excavation is carried out. The modulus in swelling is quite high, which means that at least the first phase of the swelling and effective stress reduction below the bottom of an excavation will occur quite fast (within a few months for a typical clay). A potential failure of an excavation in clay will normally occur rapidly, within minutes or hours, and in most cases represent an undrained loading case. It is however, important to account for possible reduction in undrained strength resulting from the swelling and effective stress reduction under the bottom of the excavation. For a given change in vertical effective stress, the impact of swelling on the undrained strength can be estimated using the SHANSEP expression, Eq. [6]. Conventional consolidation/swelling analyses are used to determine the relevant pore pressure condition and new vertical effective stresses resulting from the excavation. In such swelling analyses, it is important to account for the strong impact the levels of effective stress reduction have on the swelling modulus, and consequently the swelling coefficient of consolidation.

CONCLUSIONS AND RECOMMENDATIONS

Some general conclusions that can be drawn from the established database and correlation studies are as follows:

1. According to the criteria proposed by Lunne *et al.* (1997), the quality levels of most of the block samples tested are classified as very good to excellent, with some also in the category good to fair. Samples that were defined to have poor quality were not included in the correlation studies.

2. The clay sensitivity and natural water content were the index properties found of most general relevance in the correlations studied herein. The overconsolida-

tion ratio, OCR, is another key correlation parameter used in this study. A reason for not using the plasticity index, as otherwise commonly used by NGI and others as a key index parameter, is that it is impacted by the clay sensitivity. For practical purposes it was found sufficient to group the sensitivity into two main categories of "high" (here taken as S_t>15), and "low" (here taken as S_t<15).

3. From oedometer tests results, the modified tangent modulus concept proposed by Claesson (2003), was used for defining the volumetric compressibility during re-loading up to the preconsolidation pressure and during further virgin loading. There is a clear correlation between the maximum or initial re-loading modulus, M_0, water content, and preconsolidation pressure. The modulus number, m, and the corresponding reference pressure, p'_r, that define the virgin compression line, are likewise closely correlated to the water content. Rather surprisingly, the compressibility parameters appear to be independent of the clay sensitivity.

   The clay permeability is defined from the oedometer tests by the permeability at zero volumetric strain, k_0, and a permeability change index, β_k. The permeability of the tested samples is on average around $k_0 = 1.77 \cdot 10^{-9}$ m/s, but shows surprisingly large scatter, with a standard deviation of about 60%. The permeability seems rather independent of the water content, but seems to decrease slightly with the clay content. The permeability change index tends to decrease from around 4 to 6.5 for the lowest water content (30%) to 2.5 to 4 for the highest water content (70%).

4. The normalized peak undrained shear strength parameters obtained from triaxial and DSS results were expressed by the SHANSEP concept (ref. Eq. [6]) and outcomes are summarized in Table 5. The normalized shear strength tends to increase somewhat with the water content of the clay, but that cannot explain the relatively large scatter in the data.
   The normalized strengths also seem minimally dependent on the sensitivity of the clay. The variability in normalized strengths therefore seems mostly to come from local effects of cementation or differences in diagnetic bonds, possibly arising from subtle differences in mineralogy and pore-water chemistry of the different clays.

   The anisotropic strength ratios lie mostly in the range of suE/suC = 0.23 to 0.53 and suD/suC = 0.56 to 0.82. These strength ratios show a clear increase with increasing water content, and to a lesser extent increase with increasing OCR. There is also a clear trend that high-sensitivity clays give lower anisotropic strength ratios (e.g. are more anisotropic) than low sensitivity clays. This may be explained by a larger impact of the principal stress rotation that occurs during CAUE and DSS tests for the more brittle sensitive clays.

Table 5. Range of SHANSEP parameters defined from the different test types.

Test type	Typical average		Upper range		Lower range	
	S	m	S	m	S	m
CAUC	0.30	0.70	0.35	0.75	0.25	0.65
CAUE	0.12	0.80	0.16	0.94	0.08	0.75
DSS	0.22	0.80	0.26	0.90	0.16	0.75

5. The post-peak strength reduction, here defined as the normalized shear stress, τ_{pp}/s_u, reached at a shear strain of 20%, is much more pronounced for CAUC- than CAUE-type triaxial tests. The DSS tests fall in-between. There is also a clear tendency that τ_{pp}/s_u is significantly lower for clays with high sensitivity than for low-sensitive clays. Both for low and high sensitivity-clays, τ_{pp}/s_u increase with increasing OCR. The water content seems to have little impact on the post-peak strength.

6. Effective shear strength parameters are herein only defined by a maximum effective friction angle, assuming for simplicity that the attraction intercept (e.g. effective cohesion) is taken as zero. The peak effective friction angle is generally reached at fairly large strains. This maximum effective friction angle tends to increase somewhat with increasing water content. Most data lie in the range of φ = 28º to 35º. Some clays show, however, extremely high values φ= 55º. This most likely reflects a significant attraction or effective cohesion, which may be due to cementation or chemical bonding effects, similar to what was discussed in relation to undrained strengths.

7. The shear strain at failure in the undrained tests varies within wide limits, from a minimum of γ_f around 0.4% to a maximum of 15%. The CAUC tests clearly show the smallest values and a very clear trend of increasing strain at failure with increasing OCR, and decreasing with clay sensitivity. The scatter in the data is very large for the CAUE tests, but typically show two to five times the strain of failure compared to the CAUC tests. The DSS tests fall in-between.

8. The undrained secant shear modulus values are most conveniently compared by normalizing the values with respect to the vertical effective stress, but data normalized with respect to the undrained strength are also presented herein. The majority of the data suggest values of G_{50}/σ'_{v0} in the range 50 to 200. The CAUC tests tend to show the smallest values, and the CAUE the highest, with DSS in-between. There is a general tendency for G_{50}/σ'_{v0} to increase with increasing sensitivity, and to be rather independent of OCR and water content.

As an overall observation, the results show that there is more variability in all key engineering parameters than had first been anticipated at the onset of this study. The main cause of this variability may lie in subtle differences in mineralogy and geochemical characteristics between the clays tested, and to a lesser extent in differences in actual sample quality. Further studies are needed to sort out such possible impacts of mineralogy and geochemistry.

When using the data as presented herein for specific design purposes, it is recommended to use data from clay sites that are geographically closest to, and have the most similar overall characteristics as a site included herein. Furthermore, if no new high quality block samples are tested as part of a project, one should assume empirical values that are on the conservative side.

The good correlations that have previously been established between results of CPTU tests and the undrained CAUC strength and OCR values determined on high-quality block samples (e.g. Karlsrud *et al.* 2005) will also greatly help in selecting empirical undrained strength parameters for a given site. Special considerations of the possible impact of creep, strain-softening, rate and geometry effects should be made when it comes to the application of results of tests on high-quality block samples in design practice.

Acknowledgements

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Laurits Bjerrums Minnefond

Statutter

§ 1 Opprettelse

Laurits Bjerrums minnefond er opprettet av Norsk geoteknisk forening, dels ved egne midler, og dels ved gaver fra private firmaer og offentlige institusjoner, samt fra enkeltpersoner.

§ 2 Formål

Fondets avkastning skal benyttes til å fremme geoteknisk forskning og stimulere det geotekniske miljø ved følgende tiltak:

(a) Laurits Bjerrums ærespris tildeles for et fremragende enkeltarbeid, eller for flere betydelige arbeider som sammen har fremmet faget geoteknikk og fundamentering.

(b) Laurits Bjerrums stipendium benyttes fortrinnsvis til å stimulere yngre, lovende geoteknikere til forskning innen faget.

(c) Laurits Bjerrums minneforedrag holdes av fremstående geoteknikere som inviteres og honoreres.

§ 3 Styre

Fondets midler forvaltes av et styre på 3 medlemmer valgt av generalforsamlingen i Norsk geoteknisk forening for en periode på 5 år, med mulighet for gjenvalg av de enkelte styremedlemmer én gang. Norsk geoteknisk forenings sekretær, kasserer og revisor fungerer som sådanne også for fondsstyret. Fondsstyret skal holde minst ett møte pr. år. Sekretæren innkaller til møtene og deltar i disse.

Fondsstyret tar avgjørelser i alle saker som vedrører bruk av fondets avkastning til ovennevnte formål Resultatet av avgjørelsen meddeles til Norsk geoteknisk forening som skal være arrangør ved minneforedragene og ved utdeling av ærespris og stipendium.

§ 4 Anvendelsesprinsipp

Anvendelsen av fondets avkastning skal så vidt mulig skje i Laurits Bjerrums ånd. Regelmessighet og rutine ved utdelingen bør vike prioritet for originalitet og oppfinnsomhet. Det eksepsjonelle skal honoreres fremfor mengde, og ved de respektive seremonier og tilstelninger skal det legges vekt på å skape særpreg og festivitas.

§ 5 Statuttendringer

Bestemmelsene i disse statutter kan etter år 2000, endres av Norsk geoteknisk forening i henhold til foreningens egne statutter. Fondsstyret skal på forhånd enstemmig ha erklært seg enig i endringsforslaget.

Statutter vedtatt av Norsk geoteknisk forenings generalforsamling 18. september 1973