A procedure for the assessment of the undrained shear strength profile of soft clays

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ABSTRACT
In the geotechnical research community, there is widespread agreement that the choice of the characteristic undrained shear strength (c_{uA}) is very important and plays an essential role in the design and stability analysis of various geotechnical constructions placed in or on soft clay deposits. Thus, the choice of undrained shear strength, both conservative and non-conservative, could have major economic (and social) consequences in many projects. This paper summarizes the work carried out by engineers and scientists representing various institutions in Norway to provide a sound engineering method to determine characteristic shear strength of soft clays. This paper presents a recommendation on how to determine a characteristic c_{uA} profile based on laboratory and in situ testing methods and discuss the impact of stress history, strain rates, Atterberg's limits, and sample disturbance on the undrained shear strength of soft clays measured at a single borehole location. The discussion is supported by the results obtained from laboratory and field-testing. This paper highlights the key issues related to the extrapolation of undrained shear strength of soft clay from a given borehole location to a large soil volume.

Keywords: Soft clays, undrained shear strength, sample disturbance.

1 INCEPTION

The concept of shear strength goes back to 1773 when Coulomb proposed the following equation:

$$\tau_f = c + \sigma\, tan\varphi \qquad (1)$$

This was the first time that shear strength (τ_f) was regarded as consisting of two parts, i.e., cohesive resistance (c) and frictional resistance (φ), that increase proportionally with normal pressure (σ). However, the strength parameters c and φ, as introduced by Coulomb, remained difficult to determine, especially when cohesive soils were involved. Based on numerous direct shear tests from 1934 to 1937, Hvorslev reached the two main conclusions that

- cohesion (c) depends merely on water content;
- the angle of internal friction (φ) is a soil characteristic.

Hvorslev replaced the normal pressure in Eq. 1 with effective normal stress or the difference of total stress and pore pressure as

$$\tau_f = c + \sigma' tan\varphi = c + (\sigma - u)\, tan\varphi \qquad (2)$$

Here, c is the true cohesion; σ' is the effective normal stress on the failure plane; σ is the total normal stress on the failure plane; φ is the true angle of internal friction.

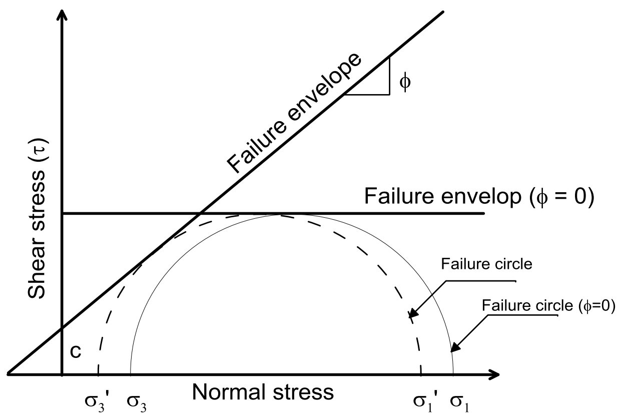

Figure 1. Failure envelopes for geomaterials. Superscript' in the figure refers to effective stresses.

Although this equation has been universally accepted to deduce the drained shear strength of geomaterials, the undrained shear strength of soft clays remains a concern. The nature of shear strength saw considerable discrepancy at the Second International Conference of Soil Mechanics in 1948. At the conference the concept of "$\varphi = 0$ analysis" was raised by Skempton (1948) because saturated cohesive soils exhibit an angle of internal resistance $\varphi = 0$ when brought to failure under undrained shearing. The corresponding strength revealed is the undrained shear strength. Specific to the $\phi = 0$ concept, it was assumed that for a saturated clay specimen under undrained conditions increases in confining stress were carried by the pore water in the sample, with the effective stress in the sample remaining unchanged. This was found consistent with the Terzaghi's effective stress principle; if the effective stress in a sample does not change, the deviatoric stress required to cause failure in the sample does not change. With the development of testing techniques, especially triaxial tests, and the accumulation of data, the fundamental behavior of soft clay was found to follow the effective stress envelope as non-cohesive soils, while the total stress envelope reflects the pore water pressures that develop during undrained shear and the fundamental behavior in terms of effective stresses (e.g. Bell 1915; Fellenius 1922; Terzaghi 1943; Skempton 1948; Lambe 1960; Bjerrum 1961, Aas 1965; Bishop 1966; Janbu 1967; Tavenas and Leroueil. 1987). The undrained shear strength is often determined through field and laboratory tests, such as the triaxial test, simple shear test, direct shear test, cone penetration test with and without pore pressure measurements (CPT /CPTU), field vane test and dilatometer test, fall cone tests, and uniaxial and plane strain test. However, these tests may become expensive if a large number of tests are to be conducted. Therefore, several researchers have developed empirical correlations between undrained shear strength and typical soil properties that can be relatively obtained with index tests. The reader is referred to Lacasse (2016), a paper in this conference, for further details.

2 PROBLEM DEFINITION

In the geotechnical research community, there is widespread agreement that the choice of the characteristic undrained shear strength is very important and plays an essential role in the calculation for the design and stability analysis of various geotechnical constructions placed in or on soft clay deposits. Thus, the choice of undrained shear strength (c_{uA}), both conservative and non-conservative, could have major economic (and social) consequences in many projects.

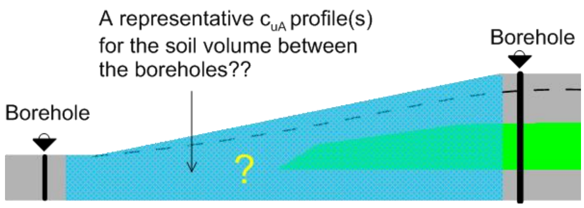

Figure 2 Problem definition.

However, the influence of the stress history, soil fabric, strain rates, Atterberg's limits, sampling technique, and sample disturbance is great on the undrained shear strength of soft clays. Consequently, the assessment of a representative c_{uA} of soft clays has been challenging since the inception of the concept. A simplified approach is therefore adopted for design purposes. It needs to be emphasized that c_{uA} of the soil is the undrained shear strength that is assumed to be mobilized along the slip surface 45^0 inclined from the major principle stress.

The undrained shear strength c_{uA} profiles with depth are usually established at some selected boreholes where the information is collected. These c_{uA} profiles are later interpolated to the soil volume between the boreholes. Thus, the accuracy of a c_{uA} profile at the boreholes is crucial because the soil volume between the boreholes depends on the representativeness of the c_{uA} profiles at the boreholes. As a first step, this paper presents a procedure for assessing the c_{uA} profile at a borehole. A discussion follows on the important aspects to consider when the c_{uA} profile is estimated for a large soil volume. This paper summarizes the work carried out by engineers and scientists representing various institutions in Norway. The overall aim of the work presented in this paper has been to provide a method to make a sound engineering judgement related to the determination of the characteristic shear strength of soft clays.

3 UNDRAINED SHEAR STRENGTH ESTIMATION

In this paper, a characteristic c_{uA} profile refers to the active undrained shear strength profile most likely to occur, deduced based on available and relevant measurements and experience data. In many cases, this is a mean value or a weighted mean of available data. If measurements (interpreted strength values) show relatively great variation with depth, additional caution must be taken when selecting the c_{uA} profile.

An empirical relationship widely used in Norway is SHANSEP, which stands for stress history and normalized stress engineering parameters (Ladd and Foot 1974). This SHANSEP principle is expressed by the following formula:

$$c_{uA} = \alpha \, OCR^{m} \, p_o' \tag{3}$$

Where

α = constant
m = constant
OCR = p_c'/p_o' (over consolidation ratio)
p_c' = effective pre-consolidation pressure
p_o' = effective vertical stress.

The SHANSHEP formula suggests that c_{uA} is governed by three parameters—soil density, pore pressure, and stress history—and two empirical constants that have been shown to vary significantly between different clays. For Norwegian sensitive clays, the values of α and m vary between 0.25 and 0.35 and between 0.65–0.75, respectively (see Figure 3).

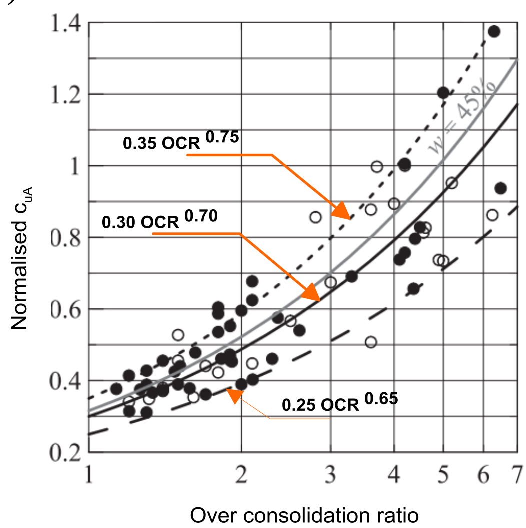

Figure 3 c_{uA} normalized with effective overburden stress versus the soil's stress history for Norwegian clays as suggested by Karlsrud and Hernandez-Martinez (2013). The filled and the open circles in the figure refers to the data for sensitive clays having the sensitivity more than and less than 15, respectively.

4 ASSESSMENT PROCEDURE FOR THE C_{uA} PROFILE AT ONE LOCATION

In this section, a stepwise assessment procedure is provided to establish c_{uA} profiles.

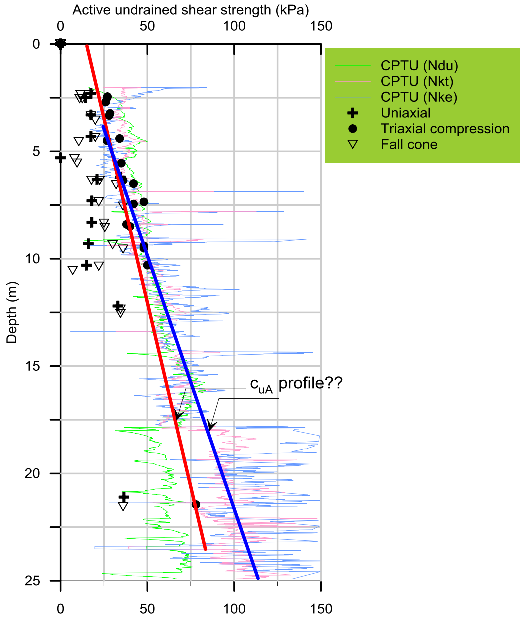

Figure 4 c_{uA} profile at one borehole location.

4.1 Quaternary geologic map

Since there is a close connection between clay strength and pre-consolidation pressure (p_c'), it is important to understand the Quaternary geology of the area of interest. This includes knowledge about the soil depositional history and subsequent natural processes that have shaped the landscape due to, for example, erosion and landslide activity.

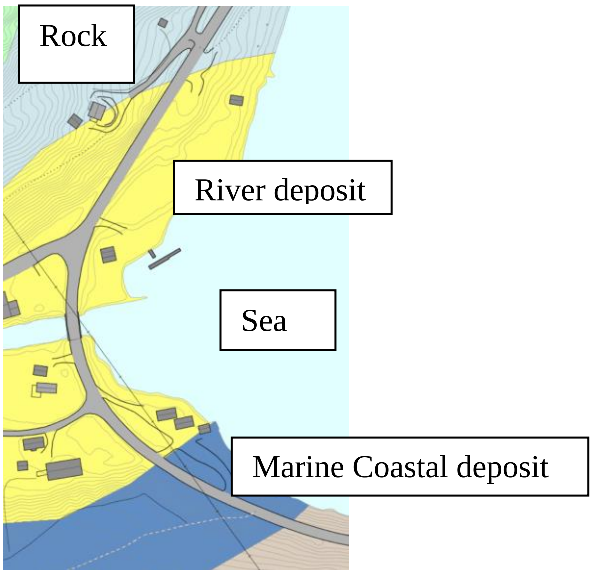

Figure 5. Quaternary geologic map (www.ngu.no).

The assessment of the c_{uA} profile, therefore, often begins with Quaternary and topographic maps. However, the reader must be aware that the quaternary map provides information only on the top layer of sediment deposition. Therefore, it is important to point-out that it often can be marine clay under alluvial deposits and coastal deposits that it is highlighted in Figure 5. In addition, human activity in many places has resulted in significant changes of natural terrain forms. In such cases, useful information may be found in old maps and reports that record what has occurred in an area over the years. In places with clear signs of landslide activity and/or ravines, the landscape often gives certain indications of what can be expected of the OCR and whether masses can be landslide affected. A knowledge about the former seabed level is a useful reference in view of to get an idea about p_c' in soil deposit.

4.2 On-site inspection

An excursion is a prerequisite for planning an effective site investigation. Unfortunately, the geotechnical engineers often undermine this step. It should be realized that excursion could help in updating the information available on maps and aid in making a site investigation plan that is executable.

The soil investigation is a dynamic process in which the measure must be adapted to what is revealed as the planned program is executed. This requires good monitoring and good communication between the field technicians and geotechnical engineers. If a borehole(s) is moved during sampling, this should be justified with proper reasoning. Therefore, the excursion before a site investigation could help avoid such issues. Site investigations should be planned so that a preliminary interpretation of the CPTU results is made before samples are collected.

4.3 Site investigation

A detailed site investigation plan for field and laboratory investigation is prepared based on the additional information gathered through steps 4.1 and 4.2. A strategy should be made to establish total stations. The total station refers to the reference boreholes where the following information must be collected:

- ✓ Total or rotary sounding
- ✓ Pore pressure measurements (in at least two levels)
- ✓ CPTU, field vane shear tests
- ✓ Soil sampling, using preferably Ø72 mm or Ø54 mm tube sampling of good quality
- ✓ Index parameters, including the Atterberg's limits
- ✓ Odometer test from at least two levels
- ✓ Anisotropically consolidated undrained compression/extension triaxial tests from at least two levels.

For the slope stability calculation, at least two total stations, one at the top and another at the toe of the slope, are recommended. The site investigation should be consistent with the critical sections where stability calculations are to be performed. An assessment is needed of how large the soil volume/area will be because the soil investigation should provide a basis for assessing layering and properties potentially involving soil volume.

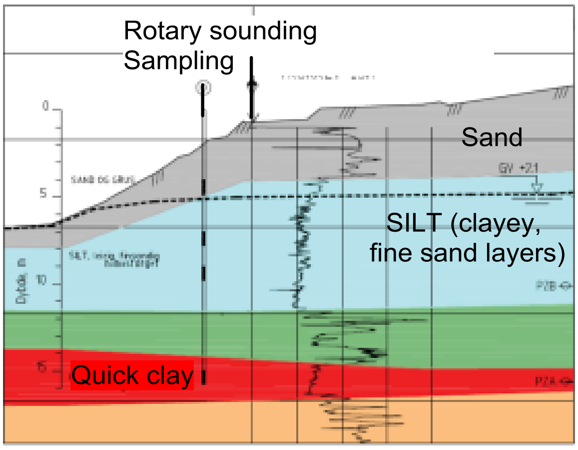

Figure 6 Detection of weak soft clay layers (NIFS report 46/2012).

The detection of weak layers using rotary/total sounding methods can be challenging. Figure 6 shows rotary sounding results taken from an NIFS report (46/2012). These indicate that the tip resistance is decreasing with depth from 5 to 12 m in the layer in light blue; this indicates a layer of sensitive clays. However, the laboratory investigation revealed that this layer consists of silt. On other hand, a similar response was seen between a depth of 15 and 17 m, and this layer was found to be a quick clay layer (highly sensitive soft clay). Therefore, supplementing sounding results with sampling or additional field tests to facilitate and improve the qualitative assessment of the sounding is advisable.

The CPTU is often the first field result relates to the strength profile that goes to geotechnical engineers; it provides a basis for drawing up a characteristic c_{uA} profile. When interpreting the CPTU results, the following questions should be asked: are there indications of weak layers and/or sections of the profile that should be given extra attention? The CPTU also provides a good basis for assessing the soil sampling program. One of the strengths with the CPTU is that the in situ pore pressure (u_0) has little effect on the interpretation. For instance, the interpretation of c_{uA} based on cone resistance does not require information regarding in situ pore pressure. A pore pressure-based assessment of c_{uA} is proportional to Δu ($= u_2 - u_0$). Since u_2 is usually several times greater than u_0, typically four to five times or more for soft clays, the uncertainty in the estimate of u_0 usually constitutes a small proportion of Δu.

A general difficulty with site investigation methods is the number of result uncertainties, which can be aleatory or epistemic. Aleatory uncertainties are unavoidable, whereas the epistemic uncertainty is related to our lack of knowledge. For example, the resulting c_{uA} profile from the CPTU test can depend on the choice of methods, correlations, and testing procedures. Moreover, not all types of equipment are appropriate for all types of materials, which should be tested with equipment and procedures that satisfy quality class 1 (e.g. Sandven et al. 2012). Based on the preliminary interpretation of the CPTU results from the total stations and at the other boreholes, samples should be taken from the desired depth.

4.4 Soil characterization and sample quality assessment

Literature e.g., Berre et al. (1969), La Rochelle and Lefebvre (1970), Bjerrum (1973), Leroueil et al. (1979), Nagaraj et al. (1990, 2003), Lunne et al. (1997), Ladd and DeGroot (2003), Leroueil and Hight (2003), Karlsrud and Hernandez-Martinez (2013), Amundsen et al. (2015), and Amundsen et al. (2016) suggest that soft clays could be prone to sample disturbance—especially when sampled using tube samplers (54, 76, or 95 mm diameter). On the contrary, block sampling in soft clay is considered to capture a more realistic soil behavior can be captured in the laboratory, as illustrated in Figure 7.

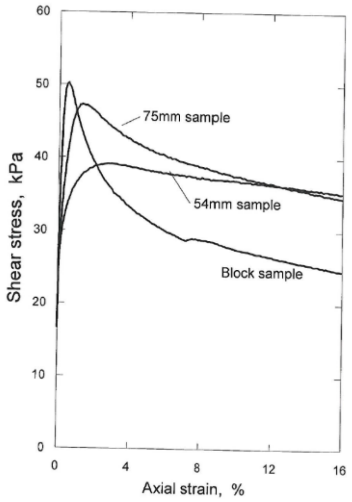

Figure 7 Example of an aniostropically consolidated, undrained, compression, triaxial test carried out by Lunne et al. (2002) on a block sample compared to piston samples from Lierstranda (depth 12.3 m).

Table 1. $\Delta e/e_0$ based sample quality criteria suggested by Lunne et al. (1997) (source: Amundsen et al. 2015)

Class	Description	$\Delta e/e_0$ (OCR 1-2)	$\Delta e/e_0$ for OCR 2-4
1	Very good to excellent	<0.04	<0.03
2	Good to fair	0.04–0.07	0.03–0.05
3	Poor	0.07–0.14	0.05–0.10
4	Very poor	>0.14	>0.10

Given that block sampling is expensive and less suited for sampling at greater depths, effort should be made to take 76-mm diameter samples. Atterberg's limits, odometer tests, and triaxial tests under compression and extension should be performed. Quality assessment of tests should be based on volume change during reconsolidation.

4.5 Assessment of c_{uA} based on laboratory testing

The most reliable laboratory method to assess c_{uA} is triaxial testing. Odometer test(s) should be performed before triaxial testing so that pre-consolidation pressure of the material is known. Pre-consolidation pressure can be measured by different methods, including Casagrande's method (1936), Janbu's method (1963), or Salfors's method (1975). A typical odometer result for a soft clay is shown in Figure 8; this illustrates Casagrande's method to estimate pre-consolidation pressure.

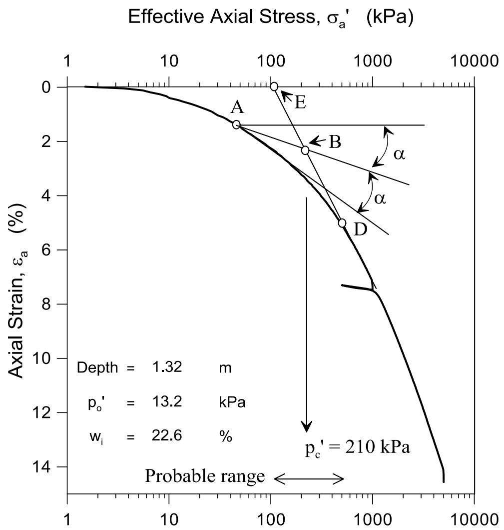

Figure 8 Casagrande's method (Holtz and Kovas, 1981).

In addition, knowledge about the in situ effective stress (pore pressure measurement) at the sample collection depths for the triaxial tests is valuable. In this way, one can estimate an accurate OCR, which can also be used to estimate K_0' for use in triaxial testing. Estimation of the correct K_0' is demanding. However, one can use the approach suggested by Brooker and Ireland (1965) that provides a relatively easy way to estimate K_0' based on soil plasticity (I_p) and the OCR. Caution is needed in using a relatively high K_0' because this will result in a higher average effective stress in the sample, resulting in a high c_{uA}.

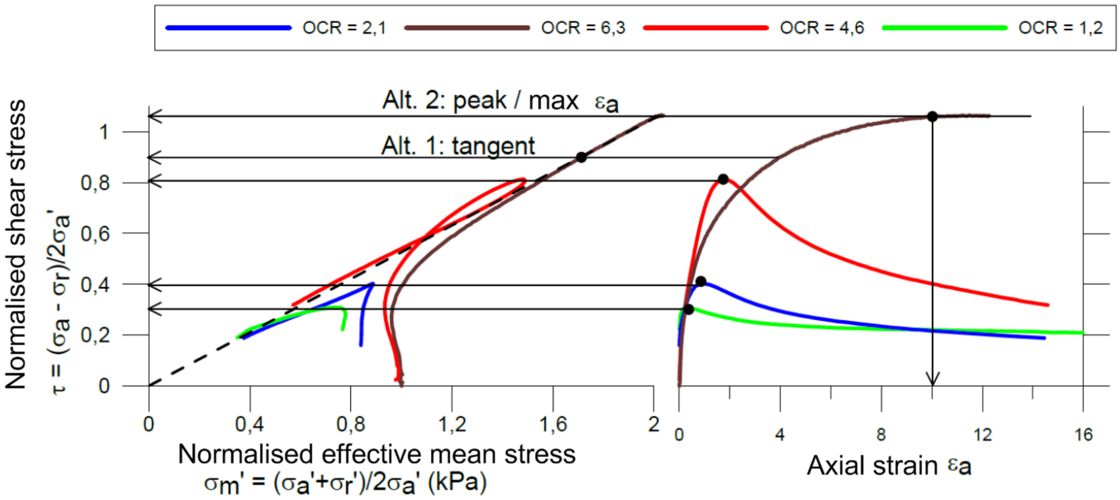

Figure 9 Estimation of c_{uA}.

As in the odometer test, the results of a triaxial test depend on the strain rate (rate dependence). An increase in the strain rate generally results in increased maximum undrained strength and brittle behavior. Lunne and Andresen (2007) showed that there might be a factor of approximately 1.5 in the estimate c_{uA} for very fast rate to a very slow rate. Commonly used strain rates in Norway have been 0.7% to 3.0% per hour. In practice, there is little difference between these experiments, and normally it is not distinguish between these strain rate and strain softening behavior. However, while estimating c_{uA} from triaxial tests, a distinction must be made between dilating and contracting behavior. For the tests exhibiting contracting behavior (positive excess pore pressure build-up during testing), it is recommended to obtain c_{uA} at the maximum measured shear resistance.

A test that first exhibits a contracting behavior but then shows dilation is an indication of sample disturbance. In such cases it is advisable to not go beyond the c_{uA} that shows the point representing the transition between contractancy and dilatancy (see Figure 9). Indeed, dilating materials (highly over-consolidated clays) often attain high strengths; however, this is normally related to high strains. In these cases, it is recommended that the c_{uA} is defined by a given strain, for example 10%.

4.6 Assessment of c$_{uA}$ *based on field testing*

The most common field test to estimate c_{uA} is the CPT apart from vane shear testing. In Scandinavian countries, the CPT with pore pressure measurements (CPTU) is considered to be the common method for estimating c_{uA} due to its ability to provide information that can help in establishing a continuous c_{uA} measurement along with the depth. The value of c_{uA} is based on three cone factors: N_{kt}, $N_{\Delta u}$, and N_{ke}.

The total tip resistance based c_{uA} is calculated as

$$c_{uA} = \frac{q_T - \sigma_{vo}}{N_{kt}} \tag{4}$$

Here, q_T is tip resistance and σ_{vo} is the total vertical pressure. The pore pressure measurement based c_{uA} is calculated as

$$c_{uA} = \frac{u_2 - u_o}{N_{\Delta u}} \tag{5}$$

Here u_2 is the measured pore pressure and u_o is the in situ pore pressure. The effective tip resistance based c_{uA} is calculated as

$$c_{uA} = \frac{q_T - u_2}{N_{ke}} \tag{6}$$

More information regarding the testing and interpretation of the CPTU can be obtained in the literature (Lune et al. 1997, Karlsrud et al.

2005). It is clear from Eqs. 4–6 that c_{uA} will depend on the cone factors. Several correlations exist for the N_{kt}, N_{ke}, and $N_{\Delta u}$ parameters to calculate c_{uA}. Some of these are discussed below.

The widely accepted method in Norway was proposed by Karlsrud et al. (2005), who compared the CPTU with c_{uA} from block samples and suggested the cone factors N_{kt} and $N_{\Delta u}$ based on the OCR and soil plasticity index (I_p). The N_{ke} parameter was correlated with B_q (pore pressure parameter) and it can be defined as

$$B_q = \frac{u_2 - u_o}{q_T - \sigma_{vo}} \tag{7}$$

Separate suggestions, as listed below, were made for soil having sensitivities less than or more than 15:

For S_t<15

$$N_{kt} = 7.8 + 2.5 \cdot \log OCR + 0.082 \cdot I_p \tag{8}$$

$$N_{\Delta u} = 6.9 - 4.0 \cdot \log OCR + 0.07 \cdot I_p \tag{9}$$

$$N_{ke} = 11.5 - 9.05 \cdot B_q \tag{10}$$

$$B_q = 0.88 - 0.51 \cdot \log OCR \tag{11}$$

For S_t>15

$$N_{kt} = 8.5 + 2.5 \cdot \log OCR \tag{12}$$

$$N_{\Delta u} = 9.8 - 4.5 \cdot \log OCR \tag{13}$$

$$N_{ke} = 12.5 - 11.0 \cdot B_q \tag{14}$$

$$B_q = [1.15 - 0.67 \cdot \log OCR] \tag{15}$$

Authors are referred to Karlsrud et al. (2005), Sandven et al. (2014) for an elaborated information regarding the CPTU interpretation using Eqns 8-15. Similarly, the Swedish Geotechnical Institute (2007) has suggested the following equation to calculate N_{kt} :

$$N_{kt} = 13.4 + 6.65\, w_L \tag{16}$$

Here, w_L is the liquid limit. If w_L is unknown, then one can assume N_{kt} = 16.3 for clays and 9.4 for silty material. In comparing Eqs. 8 and 12 with Eq. 16, a correlation has been established between w_L versus I_p for the Norwegian clays (Figure 10), and c_{uD}/c_{uA} = 0.63 ($I_p \leq$ 10%) and c_{uD}/c_{uA} = 0.63 to 0.80 ($10 < I_p \leq 80\%$) have been assumed on the basis of NIFS (2014). Here c_{uD} refers to direct shear undrained shear strength.

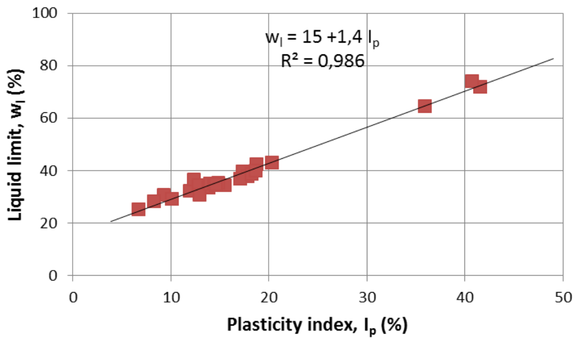

Figure 10 Correlation between w_L and I_p for the Norwegian clays based on the NGI block sample database.

Figures 11 and 12 show that the derived N_{kt} from the SGI is higher than recommended by Karlsrud et al. (2005); thus, the SGI is more conservative in the calculation of c_{uA} in this instance.

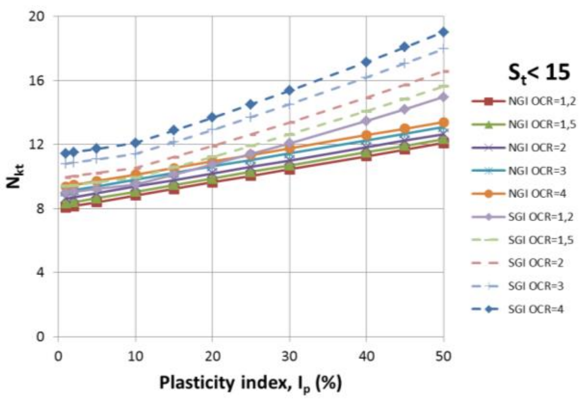

Figure 11 A comparison between the N_{kt} factors suggested by Karlsrud et al. (2005) and SGI (2007) for the clays having S_t <15.

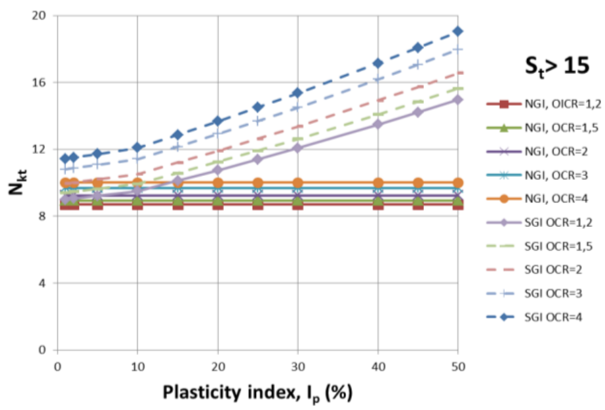

Figure 12 A comparison between the N_{kt} factors suggested by Karlsrud et al. (2005) and SGI (2007) for the clays having S_t >15.

Norwegian geotechnical consultancy firms have also established their own practice to estimate the cone factors. In addition to Karlsrud et al. (2005), Multiconsult and Norconsult use B_q to estimate $N_{\Delta U}$, N_{kt}, and N_{ke} based on Lunne et al. (1997). Tables 2 and 3 present their recommendations, and Figure 13 illustrates these correlations along with the data from Lunne et al. (1997). Some differences are obvious for the N_{ke} and $N_{\Delta u}$ parameters recommended by these two firms.

Table 2. Multiconsult's recommendation (Source: NGF seminar, 2010)

Basis	**Factors**	**Cone factors (based on B_q)**
Pore pressure	$N_{\Delta u}$	$N_{\Delta u} = 1.8 + 7.25 \cdot B_q$
Total tip resistance	N_{kt}	$N_{kt} = 18.7 - 12.5 \cdot B_q$
Effective tip resistance	N_{ke}	$N_{ke} = 13.8 - 12.5 \cdot B_q$

Table 3. Norconsults's recommendation (Source: NGF seminar, 2010)

Basis	**Factors**	**Cone factors (based on B_q)**
Pore pressure	$N_{\Delta u}$	$N_{\Delta u} = 1.0 + 9.0 \cdot B_q$
Total tip resistance	N_{kt}	$N_{kt} = 19.0 - 12.5 \cdot B_q$
Effective tip resistance	N_{ke}	$N_{ke} = 16.0 - 14.5 \cdot B_q$

Before selecting a value for the cone factors, the basis of selection and the available information in terms of routine investigations, in situ pore pressure, odometer, and triaxial tests are needed, and the choice of the interpretation method needs consideration and justification. Moreover, the validity of the interpretation of factors N_{kt} and $N_{\Delta u}$ and especially for $N_{\Delta u}$ at low B_q must be considered. Vane shear testing is another approach to obtained DSS tests at a desired depth. However, interpretation of undrained shear strength is not very reliable for clays with *Ip* < 10% as a proper correction factor for low plastic clay has not been established. The reader is referred to Gylland et al. (2016), a paper in this conference, for further details.

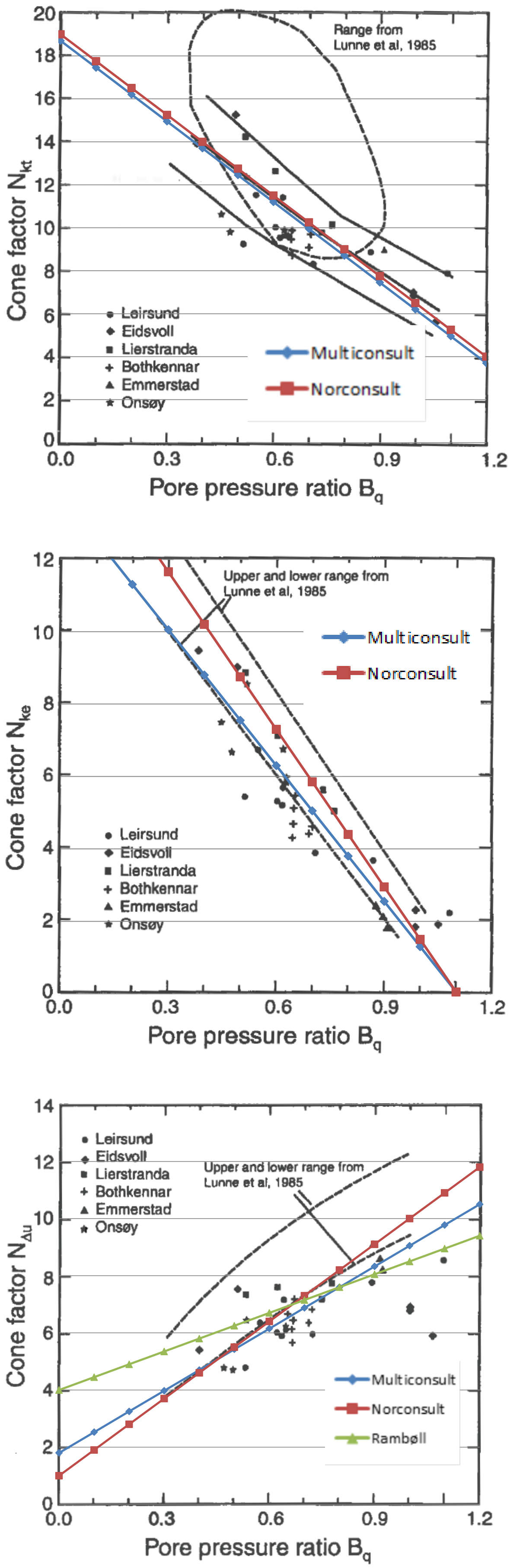

Figure 13 Cone factors versus B_q.

4.7 *Presentation and assessment of the* c_{uA} *profile*

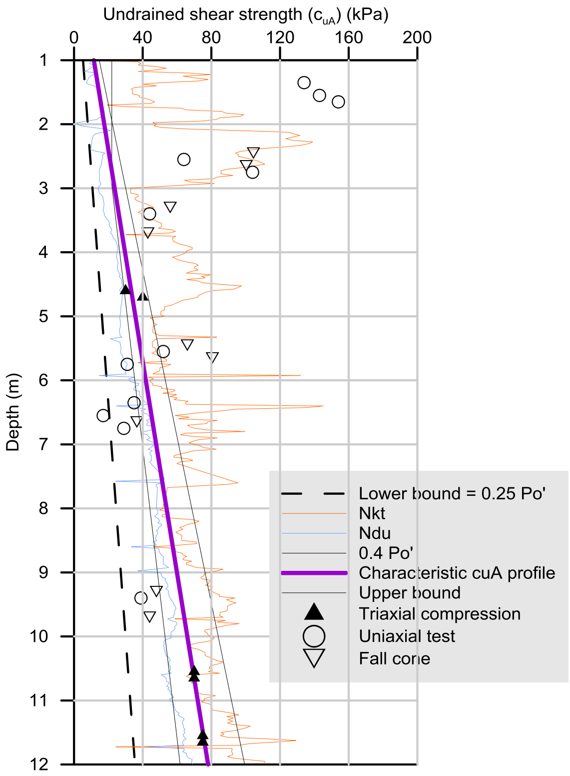

Figure 14 Estimation of a c_{uA} *profile.*

Field and laboratory data used for the interpretation of c_{uA} profiles must be chosen so that the profiles that emerge represent the soil volume to be included in the calculation. It must be substantiated that the data sample is representative of the current profile and soil volume; this includes topography, distance to calculation profile, contour level, previous level of the terrain in the area, effective stress level, pore pressure, variation in soil conditions in the area, soil geological information, and depositional history. Caution should be exercised with the use of index data from routine testing (such as uniaxial tests and the fall cone) to the interpretation of the characteristic c_{uA} profile. The interpretation of the relevant measurements and the experience-based data should be plotted together (as shown in Figure 14). Before selecting the most likely c_{uA} profile based on these data, preference ranking of the measurements and the empirical data should be done in accordance with the following:

1. Triaxial tests (sample quality class 1)
2. CPTU (quality class 1)
3. Experience-based c_{uA}/p_o' or SHANSEP
4. Fall cone/uniaxial test/vane test.

One plot for each borehole is normal, but it could be more appropriate to interpret several boreholes in the same plots. The selected c_{uA} profile should normally lie between the estimated lower and upper bounds of c_{uA} profiles. Experience indicates that the lowest c_{uA} profile for Norwegian clays is $0.25 \cdot p_o'$.

5 ASSESSMENT PROCEDURE FOR C_{uA} FOR A LARGE SOIL VOLUME

Different approaches can be taken to estimate/interpret c_{uA} for a large soil volume. For example, geostatistical approaches allow estimation of soil properties, including c_{uA} using the autocorrelation function and the kriging method (DNV, 2012). For example, if c_{uA} has been measured in a number of positions within the soil volume but is in principle unknown in all other positions, the autocorrelation function for c_{uA} can be established based on the available observations of the c_{uA} profile; using the Kriging technique allows estimating the value of the c_{uA} profile in positions where the c_{uA} profile has not been measured. For each specified position of estimation, the kriging technique provides both an estimate of the c_{uA} profile and a standard error in the estimation, accounting for spatial correlations. This technique is adopted in offshore purposes to estimate various soil properties. However, more research is needed before the method can be practiced as a tool for the engineers working on onshore geotechnics. Moreover, the current state of practice in Norway suggests that one should use engineering judgement when assessing c_{uA} for a large soil volume. In doing so, the followings issues should be considered:

- ✓ Should c_{uA} profiles represent one point or a soil volume?
- ✓ How large a soil volume should firmness profiles represent?

- ✓ Could it be that consistently weak layers will have significance?
- ✓ Are there large variations in the layering and soil conditions detected, or can we interpolate between points where we have data?

Identify how many boreholes must be established and which data need to be included in determining the c_{uA} at each borehole. The number of boreholes for the c_{uA} profile depends on topographic conditions and variations in soil conditions.

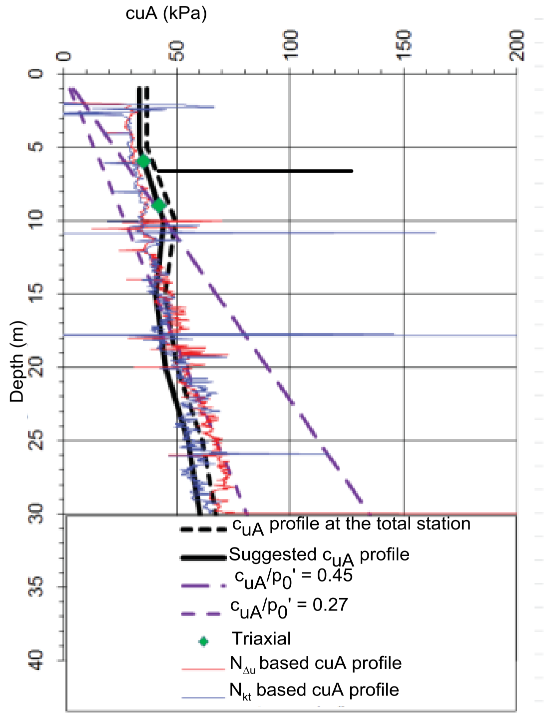

Figure 15 Representative $c_{uA\ profile}$ at a location where insufficient information is available.

For the stability calculation on a hillside, a minimum of two c_{uA} profiles have been established, one representing the shear strength at the top of the slope and one at the bottom of the slope. If there is a large variation in topography and/or soil conditions along the slope, then more strength profiles should be established between the profiles at the top and bottom of the slope. At times, some boreholes between the total stations will have a limited amount of information. In such cases, one needs to be cautious and conservative in establishing the c_{uA} profile. This is exemplified in Figure 15 in which only the CPTU data and two triaxial results are available for a borehole. The level of conservatism is based on the number and quality of the underlying information. In case of doubt, the best solution is to carry out additional field and laboratory testing.

For sensitive clay, the strain-softening behavior needs to be taken into account. If the c_{uA} profile is determined on the basis of laboratory tests on block samples with sample quality "very good to excellent" or data CPTU correlations against such tests, the current practice suggests 15% reduction of c_{uA} to account for the time effect and strain softening. This reduction may be included in the calculation program such as GeoSuite Toolbox by adding a factor of 0.85 at c_{uA} in the calculation.

Eurocode 7, Chapters 2.4.3 and 2.4.5.2 suggest some useful and relevant points to be taken into account in the selection of c_{uA} profiles. These points are summarized here:

- ✓Selection of characteristic c_{uA} should be based on results from the field and the laboratory data, either directly or through correlations/theory/empirical or other well-established and relevant experience.
- ✓The scope of investigations and the type and number of tests/experiments must be taken into account/reflected in the choice of the characteristic c_{uA}.
- ✓The test method for the determination of c_{uA} should be consistent with published and recognized information (practice) for test methods.
- ✓Factors that could cause differences between the measured value in laboratory/field and real behavior in soil shall be taken into account.
- ✓Consideration should be given to geologic information and data from other projects and/or areas with similar soil conditions.
- ✓The order of magnitude of the selected characteristic c_{uA} should be compared with relevant published data and local and general experience. Measured

values should be reconciled with empirical data to verify that the size falls within the expected range of variation.

- ✓The estimated characteristic c_{uA} should be based on a conservative estimate of the values that have the greatest impact on relevant issue/analysis/calculation.
- ✓The magnitude of soil volume included in the calculations/analyses should be reflected in the choice of c_{uA}.
- ✓Because the lab and field tests in most cases will only occupy a very small proportion of the volume of soil that will be involved in an analysis/calculation, so should the characteristic c_{uA} be chosen as a conservative estimate for the mean values of measured/derived values found by experiments in the field.
- ✓It is important to focus on weak areas and/or weak layers that may be significant for the analysis.
- ✓Caution should be exercised in the use of values that require large deformations.

6 CLOSING REMARKS

This paper presents a procedure to assess the undrained shear strength profile of soft clays. In doing so, various aspects relevant for the estimation of the strength profile are discussed in light of existing literature and field and laboratory data. The work here is mainly applicable but not limited to stability problems in Norwegian clays. Finally, this paper suggests that the representativeness of the estimated c_{uA} profile depends on how well the geotechnical information, experience, and engineering judgement are combined.

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8 REFERENCES

Aas, G. (1965) “Study of the Effect of Vane Shape and Rate of Strain on Measured Values of In Situ Shear Strength of Clays,” Proc. 6th Int. Conf. on Soil Mech. Found. Engg., Montreal, Vol. I., 141-145.

Amundsen, H. A., A. Emdal, R. Sandven and V. Thakur (2015). On engineering characterization of a low plastic sensitive soft clay. GeoQuebec, Quebec.

Amundsen, H. A., V. Thakur and A. Emdal (2016). Sample disturbance in block samples of low plastic soft clays. 17th Nordic Geotechnical Meeting, Island.

Bell A L (1915). Lateral pressure and resistance of clay and the supporting power of clay foundations. Mi. Proc. Institute for Civil Engineers. Vol 149, 233.

Berre T, T. Lunne, K.H. Andersen, S. Strandvik and M Sjursen (2007). Potential improvements of design parameters by taking block samples of soft marine Norwegian clays. Canadian Geotechnical Journal, Volume 44(6), 698-716.

Berre, T., K. Schjetne and S. Sollie (1969). Sampling disturbance of soft marine clays. Proc. of the 7th ICSMFE, Special Session, Mexico.

Bishop A. W. (1966), The strength of soils as engineering materials. Rankine Lecture, Geotechnique, 16 (2), 91 – 130

Bjerrum, L. (1961). The effective shear strength parameters of sensitive clays. ISSMGE, 5 Paris, 1961, Proceedings Vol. 1, pp 23-28. (NGI pub 45)

Bjerrum, L. (1973). Problems of soil mechanics and construction on soft clays. State-of-the-art report. Proc. of the 8th ICSMFE, Moscow.

Brooker, E.W. and H.O. Ireland (1965). Earth Pressure at Rest Related to Stress History. Canadian Geotechnical Journal, Vol. 2 (1), 1-15

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

DeGroot, D. J., S. E. Poirier and M. M. Landon (2005). Sample disturbance-Soft clays. Studia Geotechnica et Mechanica, Vol. 27(3-4), 91-105.

DNV (2012). Recommended Practice DNV-RP-C207- Statistical Representation of Soil Data.

Fellenius W. (1922). Statens Jarnvagars Geoteknisk Kommission, Stockholm

Gylland A, Thakur V, Emdal A (2016), Extended interpretation basis for the vane shear test. 17th Nordic Geotechnical Meeting, Island.

Gylland, A.S, H.P Jostad, S. Nordal,and A Emdal (2013) Micro-level investigation of the in situ shear vane failure geometry in sensitive clay. Geotechnique. 63(14),1264-1270.

Holtz, R.D. and W.D. Kovacs, W.D. (1981). “An Introduction to Geotechnical Engineering,” Prentice-Hall, Inc., New Jersey, 733 pp.

Janbu N (1967) Grunnlag i geoteknikk. Tapir

Janbu, N. (1963). Soil compressibility as determined by oedometer and triaxial tests. In Proceedings of the third European Conference Soil Mechanics, Wiesbadem. Vol. 1, pp. 19–25.

Karlsrud K and Hernandez-Martinez F G (2013). Strength and deformation properties of Norwegian clays from laboratory tests on high quality block samples. Canadian Geotechnical Journal, 50(12): 1273-1293.

Karlsrud K., Lunne T., Kort D.A., Strandvik S. (2005). CPTU correlations for clays. In: Proc. of 16th ICSMGE, Osaka. Millpress, Rotterdam: 693-702

La Rochelle, P. and G. Lefebvre (1970). Sampling disturbance in Champlain clays. Proc. of the Symp. on Sampling of soil and rock, ASTM.

Lacasse S (2016). Keynote lecture on undrained shear strength of soils. 17th Nordic Geotechnical Meeting, Island.

Ladd CC, Foot R. (1974). New design procedure for stability of soft clays. Journal of the Geotechnical Engineering Division, ASCE 100(7), 763-786.

Ladd, C.C. and DeGroot, D.J. (2003). "Recommended Practice for Soft Ground Site Characterization." The Arthur Casagrande Lecture, Proceedings of the 12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering, Boston, MA, Vol. 1, pp. 3-57.

Lambe T W (1960). A mechanistic picture of shear strength in clay. ASCE Research conference on shear strength of cohesive soils, 555-580.

Lefebvre, G. and C. Poulin (1979). A new method of sampling in sensitive clay. Can. Geotech. J. 16(1), 226-233.

Leroueil, S. and D. W. Hight (2003). Behaviour and properties of natural soils and soft rocks. Characterisation and Engineering Properties of Natural Soils.

Leroueil, S. and P. R. Vaughan (1990). The general and congruent effects of structure in natural soils and weak rocks. Géotechnique 40(3), 467-488.

Leroueil, S., M. Roy, P. L. Rochelle and F. A. Tavenas (1979). Behavior of Destructured Natural Clays. J. Geotech. Eng. Div. 105(6), 759-778.

Lunne, T. & Andersen, K.H. 2007. Soft clay shear strength parameters for deepwater geotechnical design. Proc., 6th OSIG, SUT, London, UK, 151–176.

Lunne, T., Robertson, P. K. and Powell, J. J. M. (1997): Cone Penetration Testing in Geotechnical Practice. Blackie Academic & Professional.

Lunne, T., Robertson, P.K. & Powell, J.J.M. 2002. Cone penetration testing in geotechnical practice. Spon Press.

Lunne, T., T. Berre and S. Strandvik (1997). Sample disturbance effects in soft low plastic Norwegian clay. Proc. of the Symp. on Recent Develop. in Soil and Pavement Mech., Rio de Janeiro.

Nagaraj, T. S., B. R. S. Murthy, A. Vatsala and R. C. Joshi (1990). Analysis of Compressibility of Sensitive Soils. J. Geotech. Eng. 116(1), 105-118.

Nagaraj, T. S., N. Miura, S. G. Chung and K. N. Prasad (2003). Analysis and assessment of sampling disturbance of soft sensitive clays. Géotechnique 53(7), 679-683.

NIFS (2014). www.naturfare.no

NGF Seminar (2010); NGM seminar NGF seminar om bruk og misbruk av CPTU og vingeboring. CPTU Seminar 24-25.8.2010.

NIFS-rapport 14/2014. En omforent anbefaling for bruk av anisotropifaktorer i prosjektering i norske leirer.

NIFS-rapport 14/2014. En omforent anbefaling for bruk av anisotropifaktorer i prosjektering i norske leirer.

Norsk Standard NS-EN 1997-1:2004+NA: 2008. Eurokode 7: Geoteknisk prosjektering Del 1: Allmenne regler.

Norsk Standard NS-EN 1997-1:2004+NA: 2008. Eurokode 7: Geoteknisk prosjektering Del 1: Allmenne regler.

Rolf Sandven, A. Vik, S. Rønning, E.Tørum (2012). Detektering av kvikkleire fra ulike sonderingsmetoder. NIFS rapport nr. 46-2012. Utgitt av NVE.

Sallfors, G. (1975) Preconsolidation Pressure of Soft High Plastic Clays, Ph.D. thesis, Chalmers University of Technology, Gothenburg, Sweden.

Sandven Rolf, A. Vik, S. Rønning, E.Tørum (2012). Detektering av kvikkleire fra ulike sonderingsmetoder. NIFS rapport nr. 46-2012. Utgitt av NVE.

SGI (2007): Skjuvhållfasthet–utvärdering i kohesionsjord. Information 3.

Skempton, A.W. (1948a). “The ϕ= 0 Analysis of Stability and Its Theoretical Basis”, Proc. 2n Intern. Conf. on Soil Mechanics and Foundation Engineering, 1, 72-78.

Tanaka, H. (2000). Sample quality of cohesive soils : Lessons from three sites, Ariake, Bothkennar and Drammen. Soils and foundations 20(4), 57-74

Tavenas, F. and S. Leroueil (1987). State-of-the-art on laboratory and in-situ stress-strain-time behavior of soft clays. Proceedings of the International Symposium on Geotechnical Engineering of Soft Soils, Mexico City, 1–46.

Tavenas, F. and S. Leroueil (1987). State-of-the-art on laboratory and in-situ stress-strain-time behavior of soft clays. Proceedings of the International Symposium on Geotechnical Engineering of Soft Soils, Mexico City, 1–46.

Tergazhi K (1943). Theoritical Soil Mechanics. John Wiley, New York.