Characterisation of Quick Clay at Dragvoll, Trondheim, Norway

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ABSTRACT: A detailed characterisation of the quick clay underlying the NTNU research site at Dragvoll, Trondheim is presented. The objective of the work is to provide guidance on quick clay parameters to engineers and researchers working with similar clays in Scandinavia and North America. Dragvoll clay is characterised by its high sensitivity and is quick at relatively shallow depth. The material exhibits low undrained shear strength and high compressibility except over a shallow overconsolidated zone. Its properties are similar to other quick clays in the area and are consistent with well know correlations for Norwegian clays. A combination of simple index tests such as water content and Atterberg limits and CPTU testing proved very useful in characterising the material. The material is very sensitive to sampling and laboratory handling. Results of standard and non standard tests, such as piezoball testing, are presented.

1. INTRODUCTION

Deposits of marine clay, which have been leached of their salt content, and thus have high sensitivity, are found over large areas of Norway, Sweden and Canada. These deposits pose many difficulties for engineers working in such areas. In addition landslides caused by both natural and man induced factors frequently occur. Recent examples in Norway of such slides include those at Kattmarka (2009), Lyngen (2010) and Esp (2012). A photograph of the Kattmarka slide is shown on Figure 1.

The importance of research into such sites is shown on Figure 2. This illustrates the many quick clay slides which have occurred in the Trondheim area (Sveian et al., 2006).

Research into the engineering properties and behaviour of these deposits has been undertaken at the Geotechnics Division of the Norwegian University of Science and Technology (NTNU formerly NTH) for many years. This work has included the establishment of a number of research sites where a detailed characterisation of the site is made and different ground investigation and foundation techniques can be tried out. One such site is located at Dragvoll. The objective of this paper is to present a detailed characterisation of the soils at Dragvoll based on the results of routine and advanced laboratory and field testing. Subsequently results for some more unusual techniques such as the piezoball and trial on small scale lime cement columns will be detailed. It is intended that the results presented will form a useful reference to engineers working on such soils.

Figure 1 Quick clay slide at Kattmarka, Namsos, Norway (2009)

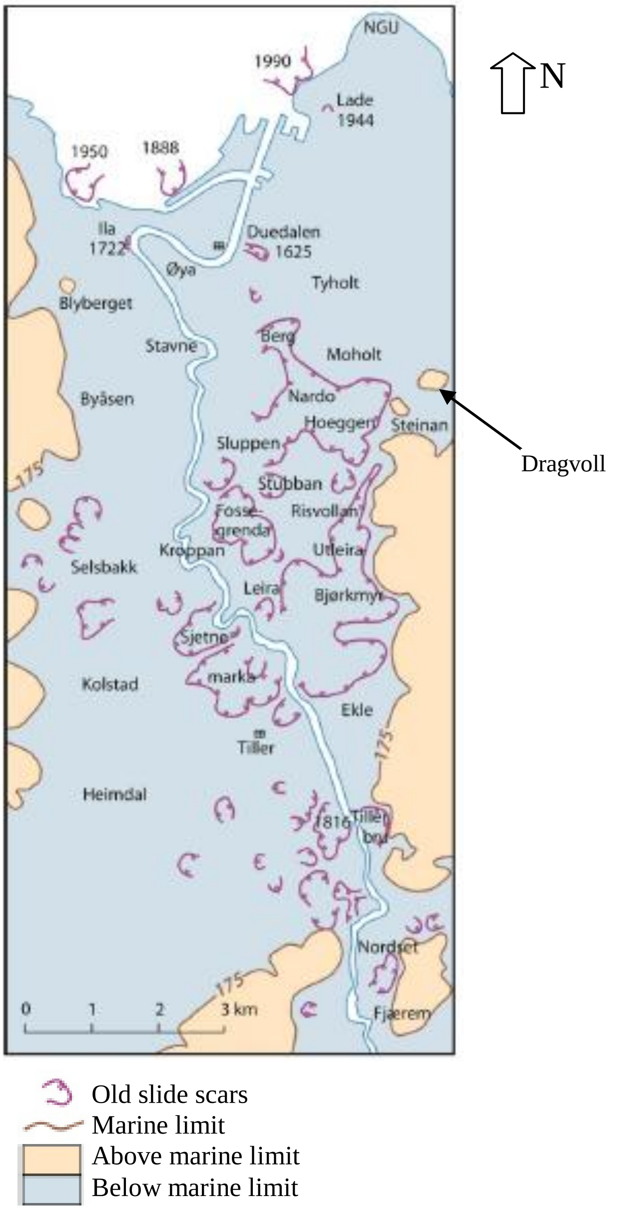


Figure 2 Quick clay slides in Trondheim area, Sveian et al. (2006)

2. DRAGVOLL RESEARCH SITE

2.1 Location, topography and present and past use

The Dragvoll site is located, east of central Trondheim, some 9 km north east of the well known NTNU quick clay research site at Kvenild, in Tiller. Data from Tiller has been used in many publications, e.g. by Sandven (1990), Sandven et al. (2004) and Lunne et al. (2006). It will be demonstrated later in the paper that the two sites have similar geotechnical characteristics.

The site location in Norway and in relation to Trondheim harbour and City Centre as well as the Kvenild research site is shown on Figures 3a and 3b respectively. A detailed site layout showing the location of previous tests, and the NTNU university campus is shown on Figure 4.

A view of the site, from the south west, is shown on Figure 5. It can be seen that the site slopes slightly to the south and surface water is drained into the small creek running southwest to northeast, some 150 m from the test area. The elevation of the test area is some +159 m above current sea level. It is currently used as farmland and mostly for grass production.

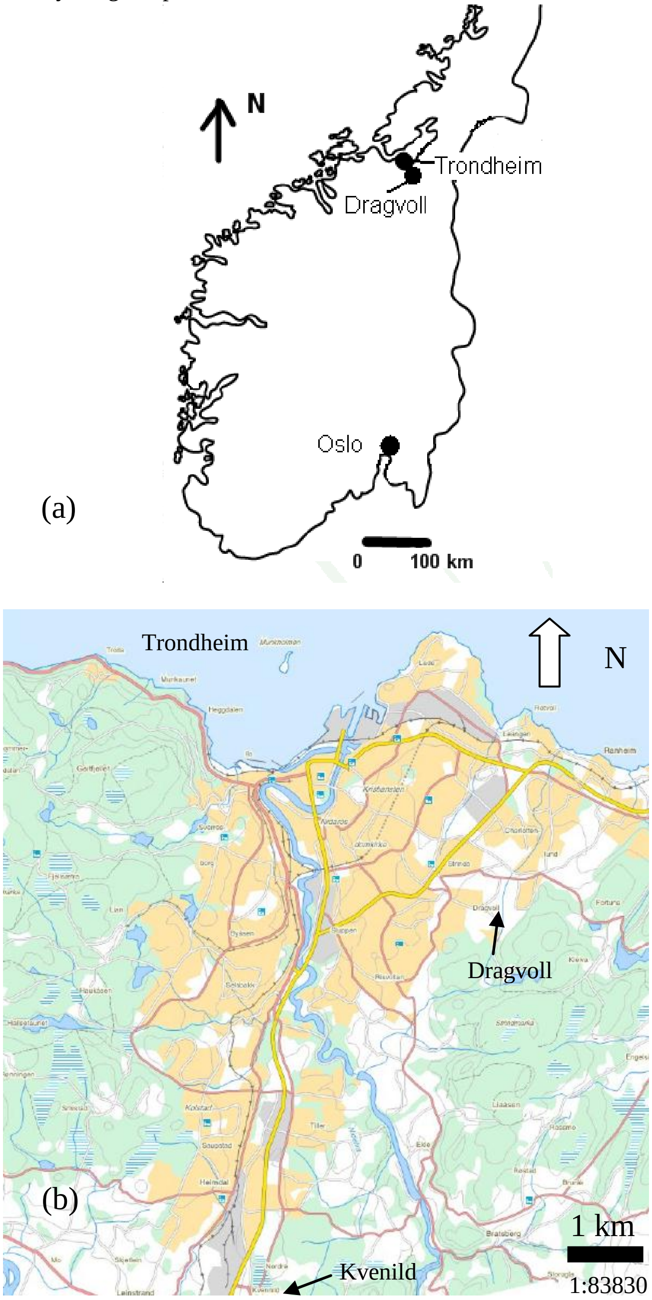


Figure 3 (a) Dragvoll site in Norway and (b) Trondheim site locations

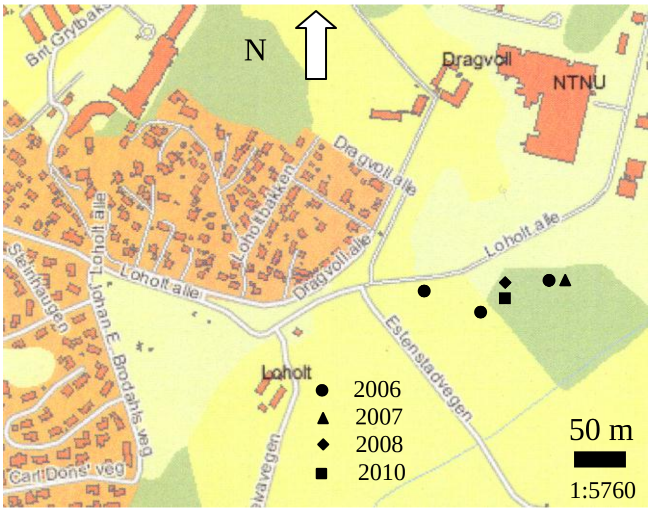


Figure 4 Test locations at Dragvoll site (Note 1968-69 testing throughout area. Nearest borehole 50 m west of 2010 tests)

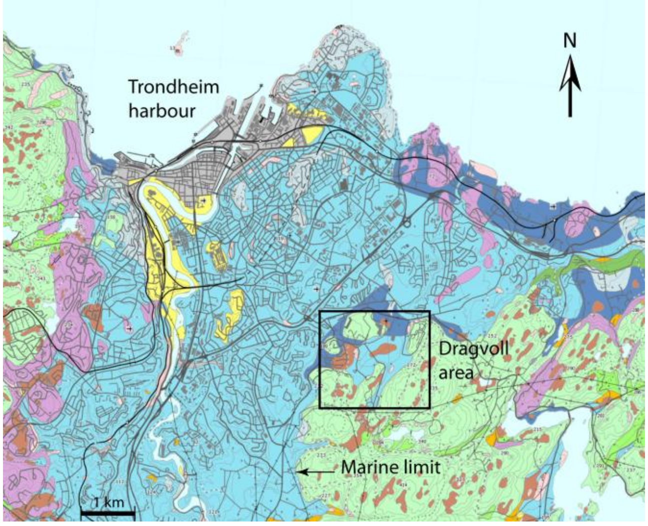
2.2 Test locations

A summary of the main investigations carried out to date at the Dragvoll site is given on Table 1. The locations at which the testing was carried out is shown on Figure 4. All of the 2006 – 2010 testing was undertaken within an area of some 120 m x 50 m.

Figure 5 View of Dragvoll site from the south west

Table 1 Summary of main investigations at Dragvoll site

Date	Description
1968-1969	Conventional investigation for proposed development of the area as an extension to the university campus. Work done by consulting company O. Kummeneje. Included borings with sampling (NGI / Geonor 54 mm steel sample tubes), rotary pressure soundings, in situ vane tests and pore pressure measurements. Several of these boreholes close to those done for research purposes later. For details, see Kummeneje (1969)
2006	Testing by NTNU mostly as a student exercises. Included drilling and sampling with NGI / Geonor 54 mm steel sample tubes, rotary pressure sounding, field vane testing and CPTU tests.
2007	As 2006
2008	As 2006
2010	CPTU testing by NTNU and piezoball testing using University of Western Australia (UWA) piezoball. Some limited sampling with NGI / Geonor 54 mm steel sampler.



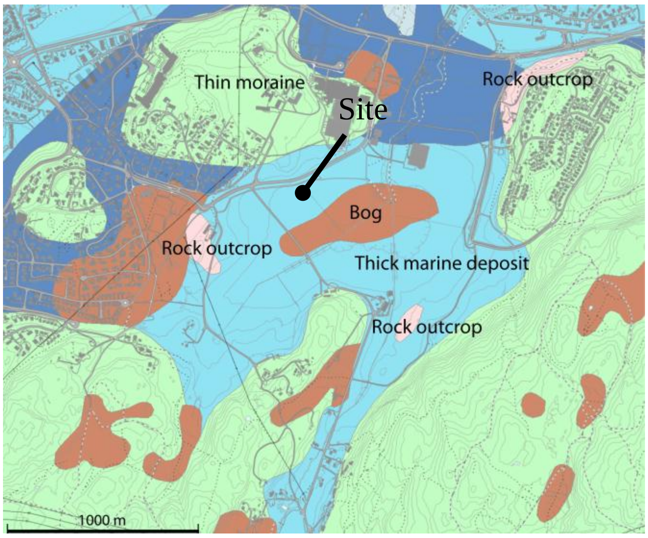


Figure 6 Geological maps of area (www.ngu.no) (a) general map showing Trondheim area and (b) detailed map of site location

2.3 Background geology

2.3.1 General

The background geology of the site area is shown on Figure 6. In Figure 6a the overall geology of the Trondheim area is shown and in Figure 6b the focus is on the Dragvoll location. It can be seen that the site is located close to the marine limit and is underlain by thick marine deposits. The marine deposits in the Dragvoll area are isolated from the main body of deposits in Trondheim by outcrops of rock and moraine and it is clear that deposition in this area was into a localised depression or "bowl". Further details of the geological history of the area can be found in Reite et al. (1982) and Reite (1995).

2.3.2 Post depositional processes and timing

The Fennoscandian ice sheet covered the Scandinavian region with an ice sheet of up to 3000 m thickness. Melting of this sheet caused isostatic land heave that again caused large marine deposits to rise above sea level. The highest sea level attained immediately prior to the disappearance of the glaciers is often referred to as the marine limit, as shown on the map in Figure 6. In the Trondheim area the marine limit is some +175 m above current sea level, as illustrated in Figure 2.

As the Dragvoll / Stokkanmyra area is elevated about +159 meter above current sea level, this implies that the sediments has been subjected to rather shallow waters (15 m and less) after sedimentation, i.e. the area emerged from the salt water environment fairly rapidly. Hafsten & Mack (1990) show that the area 11500 year BP was part of the fjord landscape whereas 1000 years later the isostatic land heave caused by the ongoing deglaciation had elevated the area so that it became a bay of brackish water limited by the outcropping rock and morraine to the north.

Further land heave changed the bay into a lake, slowly developing into a swamp, thereafter to a bog area that there are still remains of today. The area is now cultivated into farmland with still some very wet spots. These are now drained into the creek Stokkanbekken running to the northeast through the area.

In the central parts of Stokkanmyra sediment depths up to 50m thick can be found (Hafsten and Mack, 1990).

2.3.3 Stress history

From the geological history of the area, no exceptional loading events are known; only normal sedimentation processes. Once above sea level, groundwater fluctuations may have induced some changes in stress history. It would seem very likely that small variations in climate and temperature, i.e. cycles of warm and cold periods, caused temporary lowering of the groundwater table.

Groundwater level is very close to ground surface level and was, about 1 m below ground during the (summer) 2010 investigations. Preconsolidation stress (p_c') (sometimes called yield stress) in 1D oedometer tests has been estimated from oedometer tests which were carried out during the 2006, 2007, 2008 and 2010 investigations. The Janbu (1963) procedure was used.

Values of p_c' are shown on Figure 7a. Above 6 m the p_c' values are well in excess of the insitu vertical effective stress (σ_{v0}') indicating overconsolidated conditions. Below 6 m values are closer to normally consolidated conditions but these data are also likely to have been effected by sample disturbance effects.

As there is no reason for this apparent overconsolidation from the geological history of the site, it must be concluded that it has been caused primarily by natural fluctuations in ground water level and also possibly by creep induced delayed consolidation effects (Bjerrum, 1967). This delayed consolidation or creep introduces additional inter particle bonding and is sometimes referred to as "ageing".

3. MATERIAL COMPOSITION

3.1 Grain size distribution

Particle size distribution curves (for material between 3.8 m and 5.8 m only) are shown on Figure 8a. The tests suggest the material is very consistent, with average clay content of 39% and the remainder of the material being made up of approximately equal percentages of fine, medium and coarse silt. The variation in the percentage of clay with depth is shown on Figure 8b. There does seem to be some tendency for a decrease in clay content with depth towards an average of 30% at 10 m. According to standard Norwegian practice, (NGF, 1982), based on particle size distribution, the material would be termed CLAY.

3.2 Form of clay fraction

Particle density (sometimes called specific gravity) varies between 2.79 Mg/m^3 to 2.88 Mg/m^3 with an average of 2.83 Mg/m^3. These relatively high values are not unusual for Norwegian clays.

Results of some X-ray diffraction analyses of Dragvoll clay (samples from about 2.1 m depth) are summarised on Table 2. It can be seen that the material is dominated by the clay minerals chlorite and muscovite and to a lesser extent by quartz. These values are characteristic for Norwegian marine clays and are consistent with the parent geology.

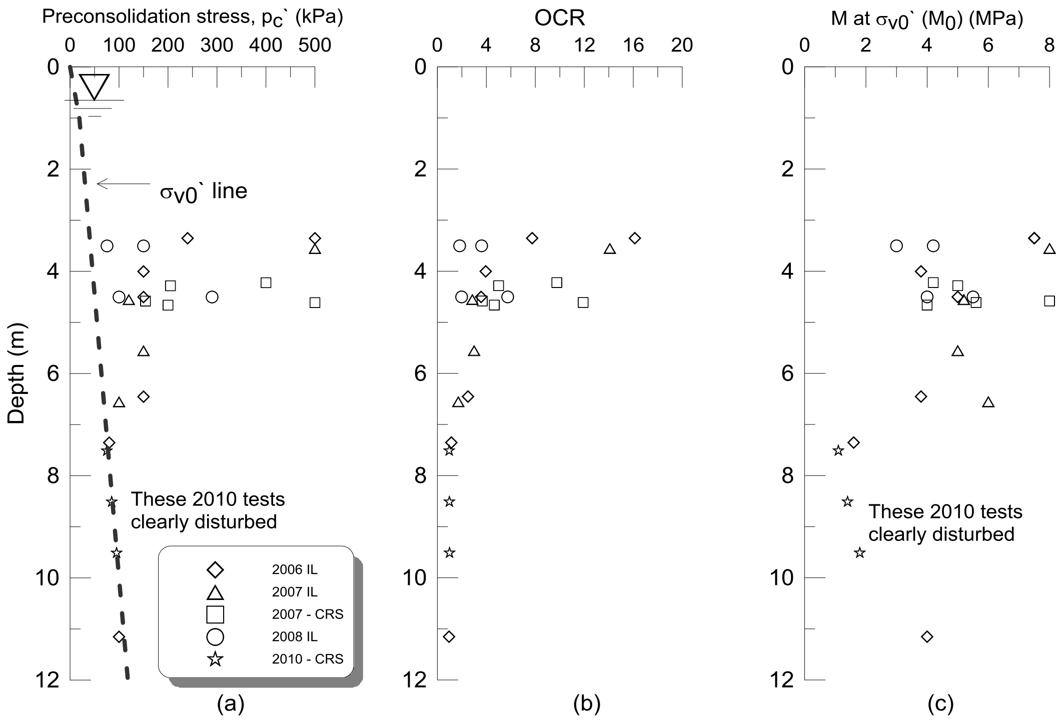


Figure 7 Oedometer test results (a) pre-consolidation stress, (b) over consolidation ratio and (c) constrained modulus at in situ effective stress

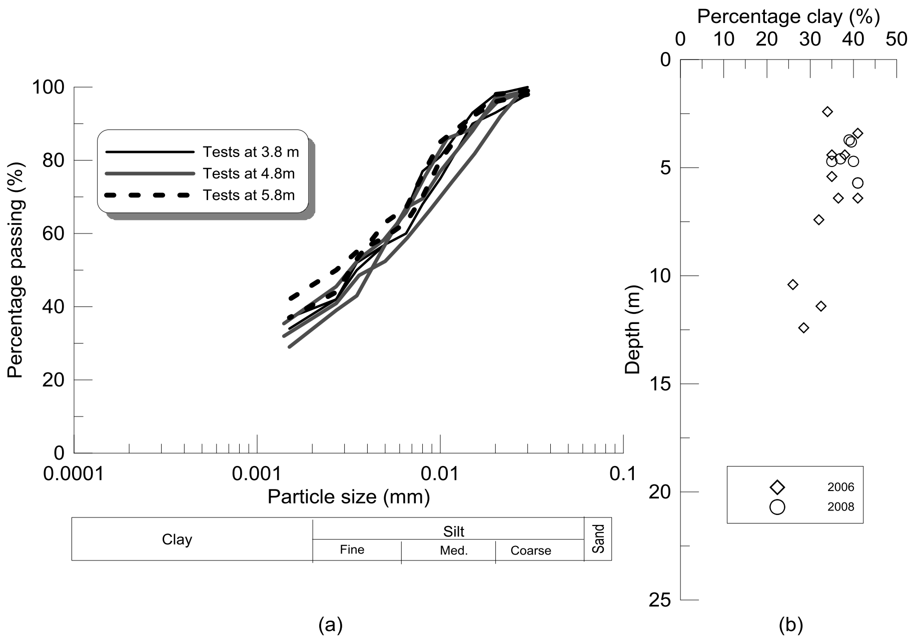


Figure 8 Grain size distribution (a) particle size distribution curves and (b) variation in clay content with depth

Table 2 Summary of X-ray diffraction analyses Dragvoll clay

Sample	2.05 m	2.15 m
Mineral	%	%
Chlorite	36	32
Muscovite	32	36
Biotite	5	2
Quartz	15	15
K-feltspar	6	7
Amphibole	4	5
Plagioclase	3	2
Calcite	< 1	< 1
Total	101	99

3.3 Pore water chemistry

Salt content values in the non quick zone, shown on Figure 21c, vary generally between 8.5 g/l and 0.5 g/l with a few of the values below 6 m being 0.4 g/l. The influence of the percolation of fresh water can be clearly seen with values well below that of the original depositional marine water (30 g/l).

4. STATE AND INDEX PARAMETERS

4.1 Water content and degree of saturation

A plot of water content (w) versus depth is shown on Figure 9a. Except for a region above 5 m, water content is generally greater than 30%, with an average of 32% to 33%. According to Janbu (1970) w of about 30% corresponds to the division between normally to lightly overconsolidated clay and moderately overconsolidated clay. There is a slight tendency for an increase in water content with depth. Except near to the surface the degree of saturation of the clay is 100%.

4.2 Atterberg limits

Plasticity index (I_p) (shown on Figure 9c) is mostly less than 10%, i.e. indicating low plasticity (Janbu, 1970), except for a zone above 5 m. The Atterberg limits chart, shown on Figure 10, confirms the consistency of the material and shows the result to lie just above the A line in the zone "CL" corresponding to clay of low plasticity.

4.3 Density and void ratio

Bulk density (ρ_b) (Figure 9b) decrease slightly, in tandem with the water content trend, from about 2 Mg/m^3 above 5 m to 1.9 Mg/m^3 on average with depth. Initial void ratio (e_0) values vary between about 0.7 and 1.0 with an average of about 0.8.

4.4 Liquidity and void index

Liquidity index, which is defined as:

$$I_L = \frac{w - w_p}{I_p} \qquad (1)$$

can be a very useful indicator of the stress state of the material. Note that w_p = plastic limit. Values for the Dragvoll site, shown on Figure 11, are generally one or less than one over the upper overconsolidated zone and exceed one in the normally / lightly overconsolidated zone below about 6 m.

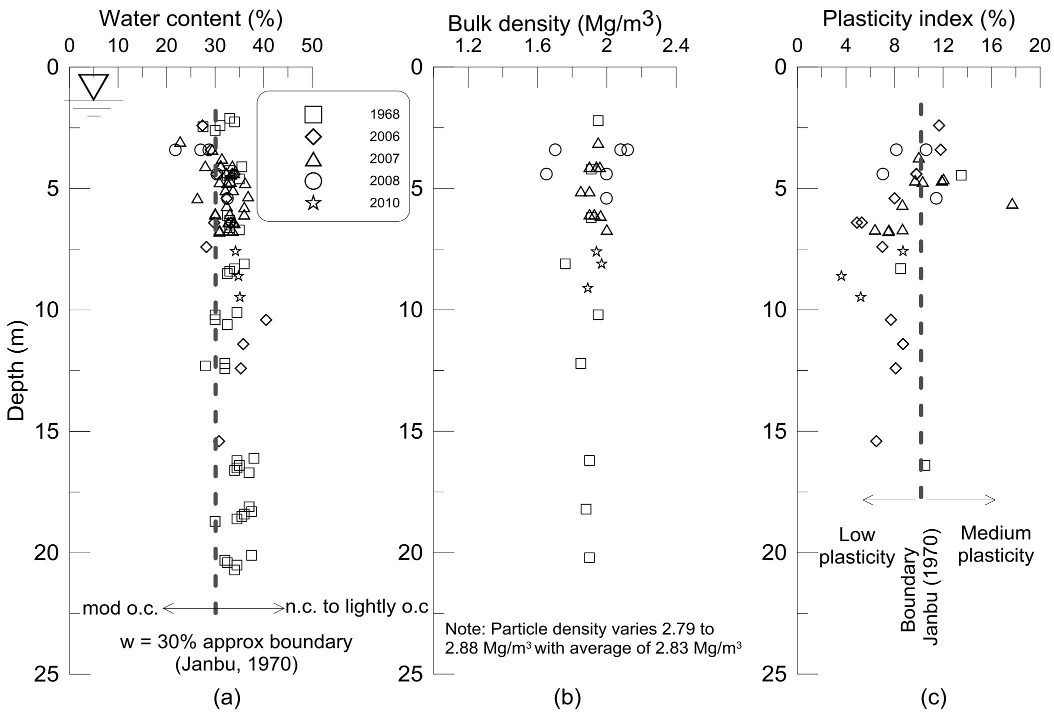


Figure 9 (a) water content, (b) bulk density and (c) plasticity index

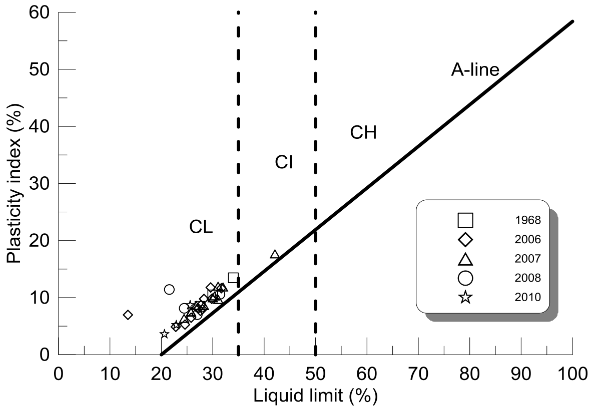


Figure 10 Plasticity (“A” line) chart

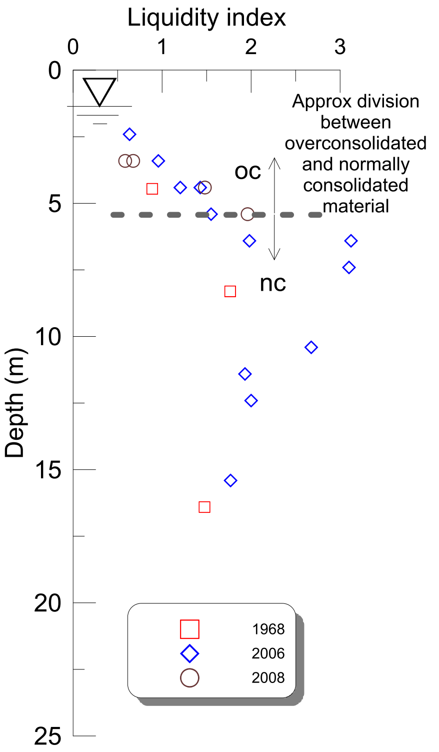


Figure 11 Liquidity index

Burland’s (1990) in situ void index (I_{v0}) values can be determined from e_0 as follows:

$$I_{v0} = \frac{e_0 - e^*_{100}}{C^*_c} \tag{2}$$

where:
e^*_{100} = void ratio on Burland’s intrinsic compression line (ICL) for σ_v' =100 kPa
C_c^* = intrinsic compression index

As no intrinsic compression tests are available for Dragvoll clay, the e^*_{100} and C_c^* values have been chosen to equal 0.51 and 0.135 respectively after the correlations published by Burland (1990).

5. STRUCTURE

5.1 Macrofabric

Dragvoll clay is essentially homogenous grey silty clay. There is no evidence (on the macro scale) of laminations or fissures. Over the top 3 m or so there is evidence of organic material in the form of rootlets. There are also some gravel fragments up to 20 mm in diameter.

5.2 Microfabric

Some comments on the microstructure of the material can be made based on the behaviour of the material in 1D compression tests within the framework proposed by Burland (1990).

No specific tests have been done to measure the ICL and SCL (sedimentation compression line) of Dragvoll clay so Burland’s published relationships are used. Two typical test results for Dragvoll clay are shown on Figure 12. The tests were carried out on 54 mm diameter NGI / Geonor steel piston tube samples from about 3.5 m and 8.15 m depth. These samples were chosen to represent the over consolidated and lightly over consolidated zones respectively.

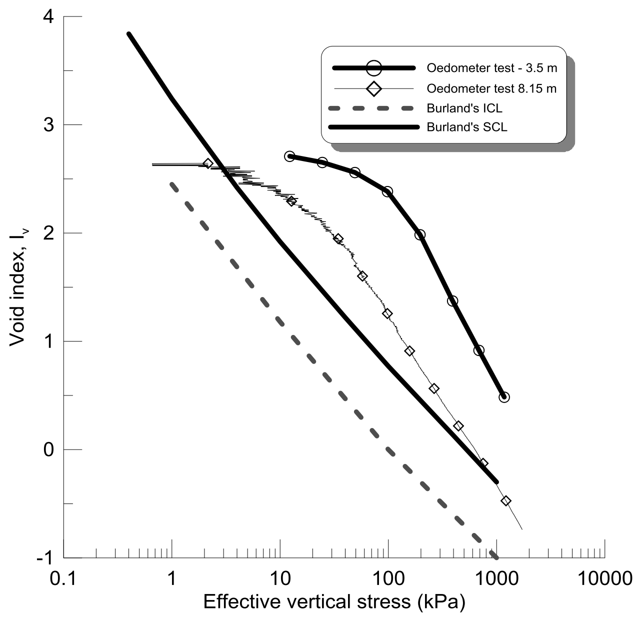


Figure 12 1D compression data from samples from 3.5 m and 8.15 m plotted in form suggested by Burland (1990)

Both test curves initially remain horizontal and then cross the SCL before plunging steeply after yield. The 3.5 m test is able to retain its structure for a much greater stress interval than the 8.15 m sample. Both curves trend towards but only the 8.15 m one reaches the SCL. Neither reaches the ICL even at high stress suggesting that the oedometer test does not impart sufficient mechanical energy to break down the natural fabric and bonding of the material completely.

The results are indicative of a structured material, which has been deposited slowly in still water leading to an open random fabric with high values of void index.

No scanning electron microscope photographs or similar are available for Dragvoll clay.

6. ENGINEERING PROPERTIES

6.1 Yield stress (or apparent pre-consolidation stress)

Five sets of oedometer data are available for the site:

- 2006 incremental load
- 2007 incremental load
- 2007 constant rate of strain (CRS)
- 2008 incremental load
- 2010 CRS

Note that all of the samples were retrieved using the NGI / Geonor 54 mm diameter fixed piston sampler with steel tubes, see Andresen and Kolstad (1979). The first four investigations are generally from shallow depth, i.e. less than 6 m.

Sample quality for the 2006, 2007 and 2008 tests is good with strain (ε) at σ_{v0}' being generally less than 4% and often about 2%. The sample quality for the 2010 deeper samples is unfortunately poor with ε at σ_{v0}' being between 9.3% and 12.3%.

A discussion on the determination of the 1D yield stress (p_c') was given above in Section 2.3.4 on stress history. Derived values of p_c' are shown on Figure 6a. It is very likely that the results for the deeper tests below 7 m, especially those from the 2010 investigation have been affected by sample disturbance effects.

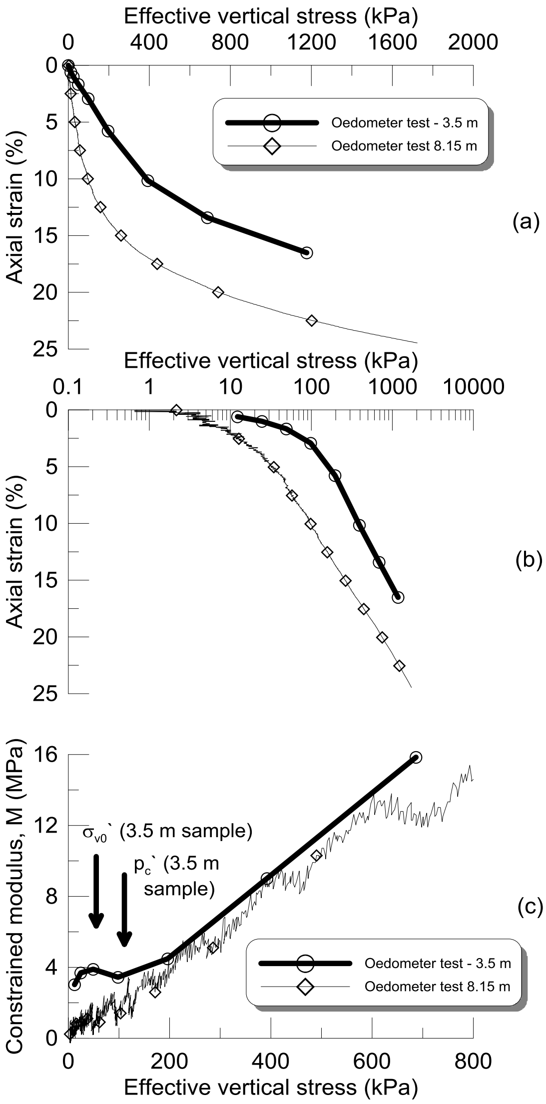


Figure 13 1D compression data for sample from 3.5 m plotted in (a) σ_v' versus ε (b) log σ_v' versus ε and (c) σ_v' versus M formats

6.2 OCR

Over consolidation ratio (OCR) values are shown on Figure 7b and can be seen to decrease from about 16 near 3 m to 1 below 6 m. However as discussed above these low values are likely to be at least partly due to sample disturbance effects.

6.3 Relationship between stiffness and strain – 1D consolidation tests

Some typical 1D oedometer tests (same as those shown on Figure 12) are given on Figure 13. Data is plotted in the traditional σ_v' versus strain (ε), log σ_v' versus ε and in σ_v' versus constrained modulus M (=Δσ'/Δε) formats. It can be clearly seen that the quality of the 3.5 m sample is higher than that from 8.15 m. The log σ_v' versus ε plot shows typical behaviour for clays with a stiffer portion before the clearly defined pre-consolidation stress at about 100 kPa followed by a much softer response post yield.

Constrained modulus values, M, follow the classical pattern for Norwegian clays (Janbu, 1963) of a decrease on loading, followed by a significant increase during reloading to a maximum value of about 4 MPa. This value then drops to a minimum at the pre-consolidation stress and then increases approximately linearly with stress in the normally consolidated zone. The 8.15 m sample shows very similar behaviour to that at 3.5 m in the normally consolidated zone.

6.4 Stiffness – constrained modulus M

Values of and M_0, i.e. M at σ_{v0}', are shown on Figure 7c. M_0 values are scattered but are on average 5 MPa above 6 m and lower below this. The 2010 deeper values are very low, probably due to the poor sample quality as discussed above.

6.5 Compressibility in the normally consolidated zone

Stiffness in the normally consolidated zone can be expressed by the modulus number m which is the slope of the σ_v' versus M plot after p_c' (Janbu, 1963). Values of m versus depth are shown on Figure 14a. And can be seen to be relatively scattered but generally average about 16 which is on the lower bound suggested by (Janbu, 1985) for clay with average water content of 32.5%.

6.6 Coefficient of consolidation

The coefficient of consolidation at in situ vertical effective stress (c_v at σ_{v0}') was determined from the oedometer tests are shown on Figure 14b. The values are generally in excess of the range suggested by Janbu (1985) (for material with an average water content of 32.5%) above 6 m, again suggesting the material is overconsolidated and fall to values less than 8 m^2 / yr below this depth. Again there are indications that the samples from the 2010 works are disturbed.

In addition the coefficient of consolidation in the horizontal direction (c_h) has been established from the results of piezocone (CPTU) dissipation tests (Bihs et al., 2012), see Figure 15. The approach from Houlsby & Teh (1988) was used by assuming a soft soil with a rigidity index of about 30 and determining the c_h at 50 % dissipation.

As one can see from Figure 16 the c_h values are close to the NC c_v from the laboratory investigations. In general the two different interpretation methods used compare well with each other. The method established by Senneset et al. (1982) was also applied using the rate approach with "soft" rate factor at 50 % dissipation. Dissipation tests have been carried out in the soft clay zone at 6.5 and 10.89 m. The results are presented in Figure 15 in the form of a logarithmic plot of the normalised pore pressure U against time t. U is defined as:

$$U = \frac{u_2 - u_0}{u_2} \tag{3}$$

All tests were carried out until 50 % of the excess pore pressure is dissipated. The tests show a steady dissipation of pore pressure down to the hydrostatic level.

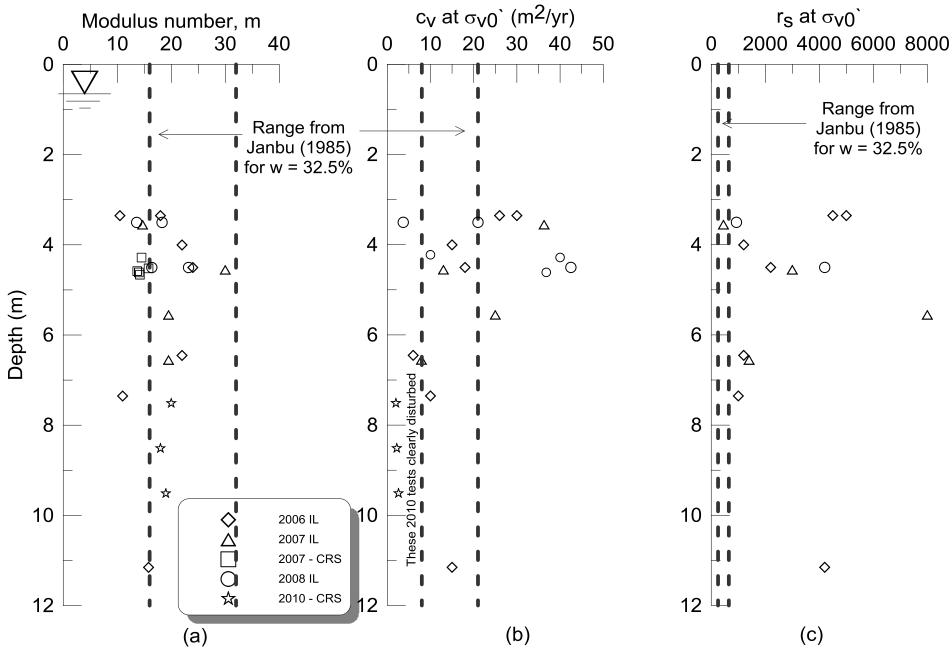


Figure 14 1D compression parameters (a) modulus number, (b) coefficient of consolidation and (c) creep number

6.7 Creep

Creep number values at in situ vertical effective stress, i.e. r_s at σ_{v0}', shown on Figure 14c are also somewhat scattered but suggest the material is relatively highly resistant to creep, i.e. is over consolidated.

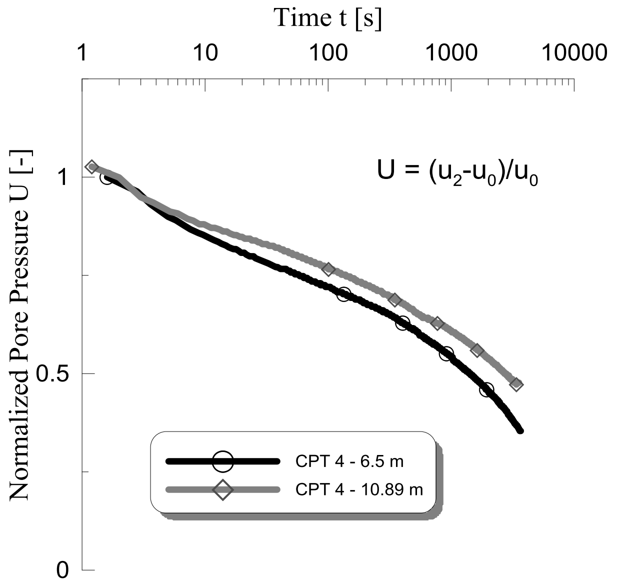


Figure 15 CPTU dissipation test results

6.8 Relationship between stiffness and strain – triaxial tests

The results of a typical CIU triaxial test, in the form of secant Young’s modulus (E_v') against strain, is shown on Figure 17. The sample was from 8.4 m and again was taken using the NGI / Geonor 54 mm steel sampler. It was consolidated isotropically to approximately the mean in situ stress of 60 kPa. The volume change during consolidation was 3.55% indicating the sample quality is reasonably good.

As an external displacement gauge only was used it is not possible to resolve the data to strain less than about 0.5%. It can be seen that the E_v' drops rapidly with increasing strain in the classical fashion of the S-shaped curve.

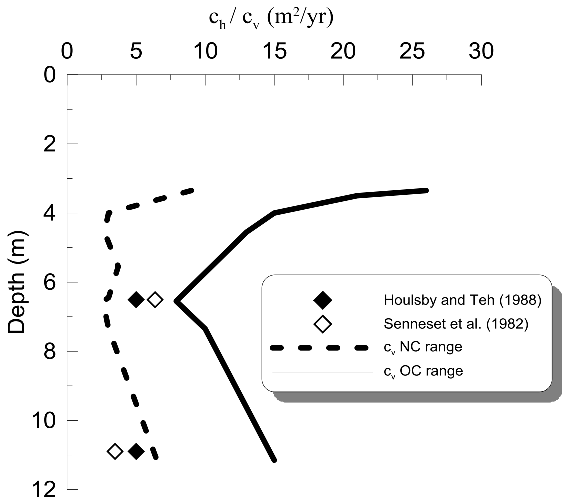


Figure 16 Coefficient of consolidation from CPTU dissipation tests

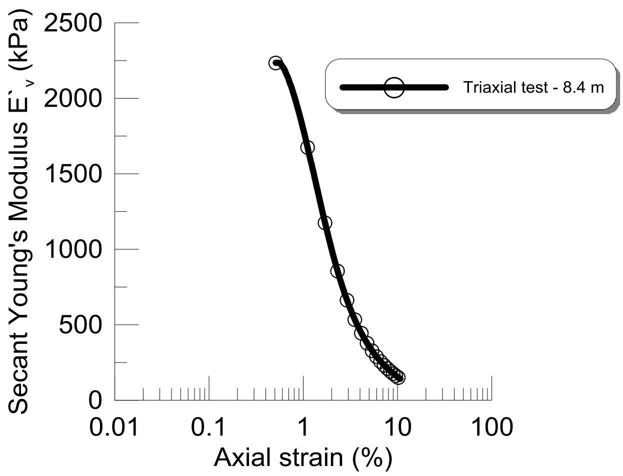


Figure 17 Stiffness from triaxial testing (CIU test on sample from 8.4 m)

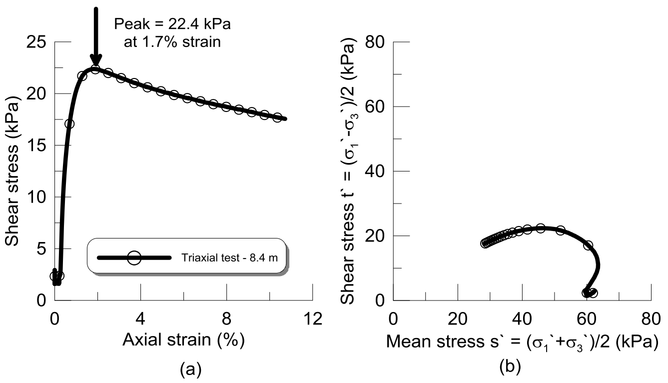


Figure 18 Stress versus strain and stress path for typical triaxial test (CIU test on sample from 8.4)

6.9 Laboratory stress / strain and stress paths

Shear stress (τ) versus strain (ε) and a stress path plot for a typical triaxial test from 8.4 m (same as reported above) are shown on Figure 18. The stress path is in s' versus t' format where:

$$s` = \frac{\sigma_1` + \sigma_3`}{2} \tag{4}$$

$$t` = \frac{\sigma_1` - \sigma_3`}{2} \tag{5}$$

The τ versus ε plot is characteristic for Norwegian soft clays with peak τ (i.e. undrained shear strength s_u) occurring at relatively low strain followed by gradual strain softening. The stress path plot initially exhibits some sample structure with the curve forming a line with slope 1/3 before contracting (bending to the left) in the classical pattern of a lightly overconsolidated clay, and subsequently forming a clear failure line. Oedometer test data (Fig. 7) from this depth had also suggested that the material was close to normally consolidated conditions.

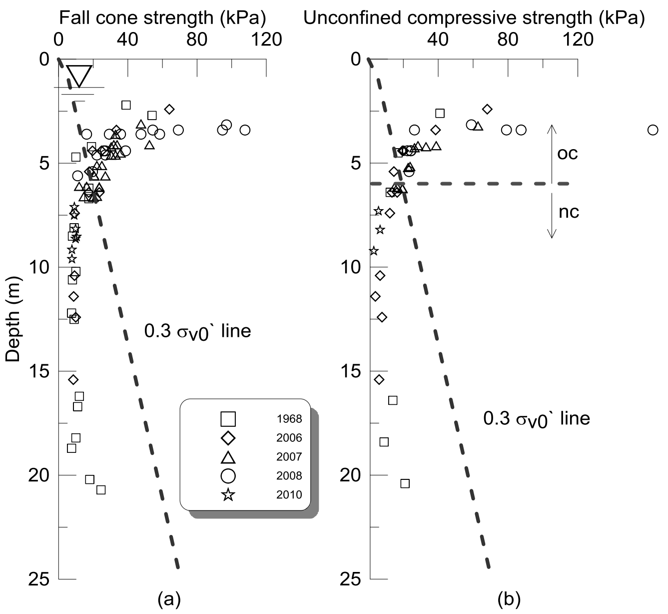


Figure 19 Laboratory index strength tests

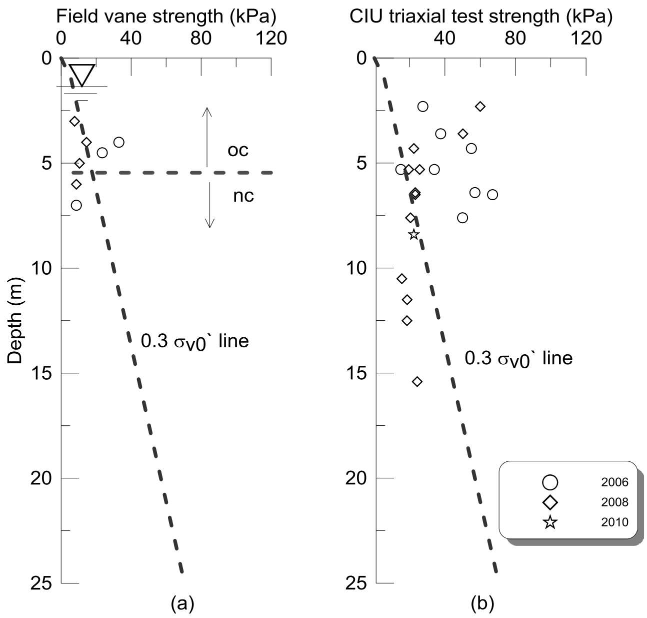


Figure 20 Undrained shear strength from (a) Field vane testing and (b) Laboratory CIU triaxial tests

6.10 Undrained shear strength from laboratory testing

6.10.1 Index testing

Index shear strength tests from fall cone and unconfined compression tests are shown on Figure 19. Both sets of data show the same trend. Above about 6 m, s_u values are relatively high, well above the $0.3\sigma_{v0}'$ line, indicating the material is overconsolidated. Below 6 m there is a significant fall off in the s_u values. They fall well below the $0.3\sigma_{v0}'$ line indicating the material is normally consolidated. These low values may also reflect sample disturbance effects and stress relief effects.

6.10.2 Triaxial testing

Undrained shear strength from CIU triaxial tests is shown on Figure 20b. This test data needs to be treated with caution due to the selection of the consolidation stress. In the 2006 tests the specimens were generally consolidated to a stress slightly less than the in situ value while the opposite is true of the 2008 tests. Nonetheless the results confirm the findings detailed above.

6.11 In situ undrained shear strength – field vane test

Undrained shear strength values from field vane (5.5 cm x 11 cm blade) tests are shown on Figure 20a. Although the data is limited the field vane results confirm the findings from the index test results above (Figure 19) with the material above 5 m showing clear indications of being over consolidated while that below 5 m is normally consolidated.

6.12 Remoulded shear strength and sensitivity

According to (NGF, 1982) a material is quick if S_t values are greater than 30 and the remoulded shear strength data are less than 0.5 kPa. Fall cone data shown on Figure 21 show clearly that above about 6 m the material is not quick but that below 6 m and above 16m there are significant zones of "quick" clay. Physical inspection of the behaviour of the 2010 samples confirms that this material is highly sensitive and remoulds to a liquid easily on agitation.

Field vane remoulded shear strength and sensitivity values are shown on Figure 22. These have been separated from the fall cone tests as recent work shows that due to the different mode of shearing the values are not directly comparable. None of the test indicates "quick" conditions. However the maximum depth at which a test was carried out was 7 m.

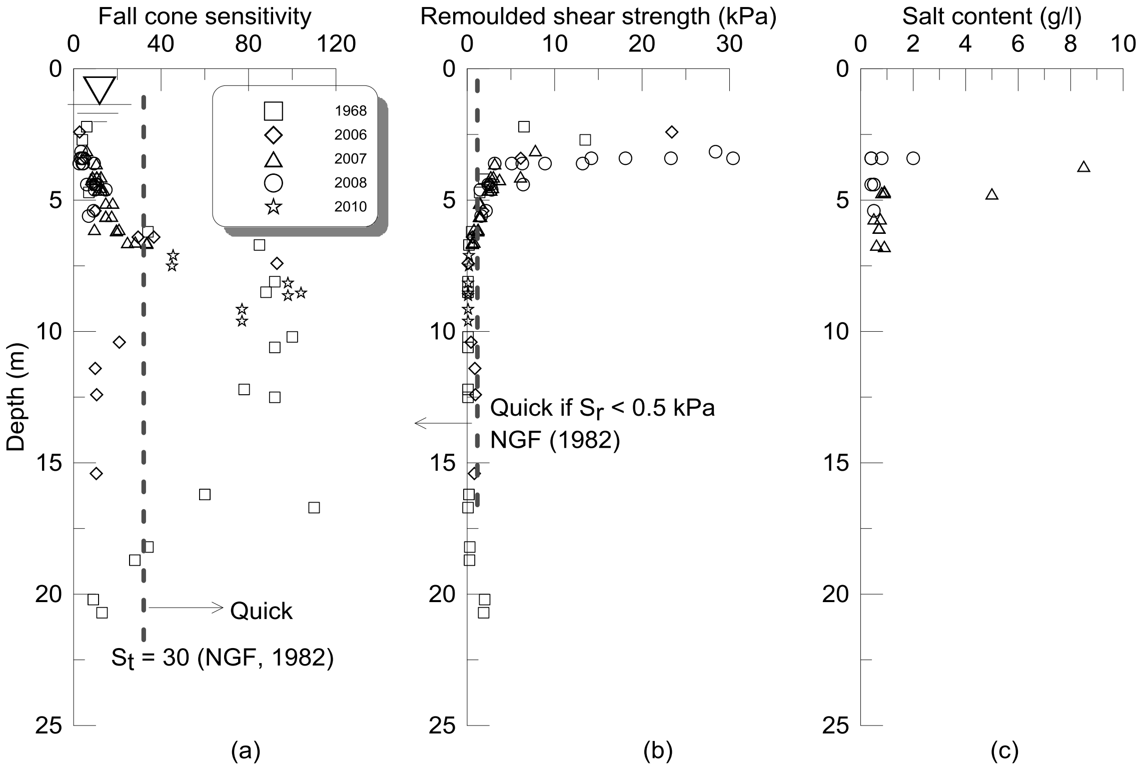


Figure 21 Laboratory fall cone tests showing (a) sensitivity, (b) remoulded shear strength and (c) salt content

6.13 Fin situ strength – cone penetration testing

Two sets of CPTU data are available for the site from 2007 and 2010. The latter set of tests are reported in detail by Bihs et al. (2012). Both sets of tests were carried out using a Geotech CPTU. For the 2010 tests the data was transmitted acoustically along the rods.

The 2007 data is shown on Figure 23 in the form of corrected cone resistance (q_t) and pore pressure generated (u_2) with depth. No sleeve friction (f_s) readings are available.These profiles are useful as they give data up to 25 m depth.

The 6/11/07 q_t profile is considered unreliable below about 5 m because of a problem with the cable. The 1/9/07 q_t data looks reasonable and shows high q_t values over the upper 3 m to 4 m before decreasing to a minimum between 5 m and 6 m and then increasing steadily with depth. These data clearly indicate, as has been discussed above, that above about 6m the material is overconsolidated and that below this depth is normally consolidated. This conclusion is based on the empirical correlation of Lunne et al. (1997), who suggested that the material will be normally consolidated if q_t is in the range 2.5 σ_{v0}' to 5.0σ_{v0}'.

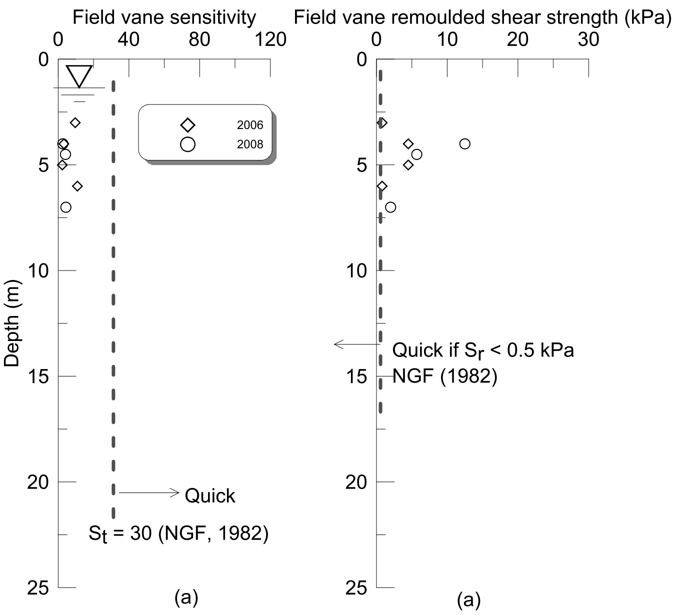


Figure 22 Field vane (a) sensitivity, (b) remoulded shear strength

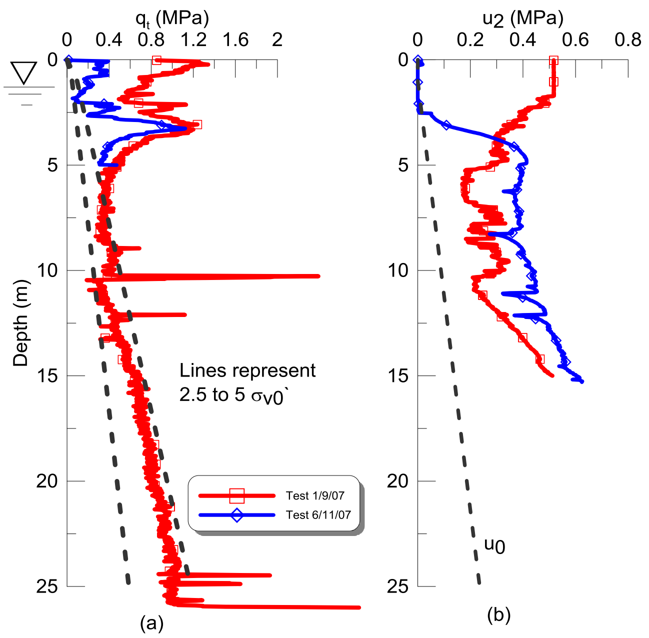


Figure 23 CPTU tests from 2007 (a) q_t and (b) u_2

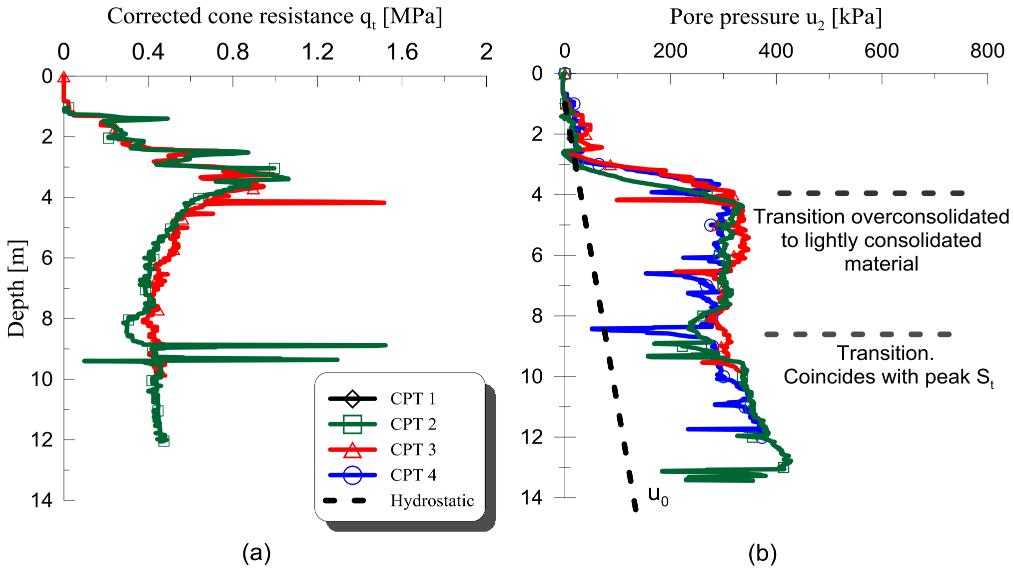


Figure 24 CPTU tests from 2010 (a) q_t and (b) u_2

Both sets of u_2 data look unreliable with perhaps that of 6/11/07 being the best. In this latter case u_2 values are close to zero above the water table and then build up rapidly to values well in excess of u_0 within the saturated clay. Comparison with the in situ pore pressure (u_0) values suggest that the material behaves in an undrained manner during CPTU penetration.

The 2010 CPTU results are presented in Figure 24. The data shows good agreement between the four different tests carried out. Pore pressure response from CPT 1 was poor indicating saturation problems so this data was disregarded. The 2010 data are very similar to those measured in 2007 and also confirm the results from the laboratory investigation. Above 4 m pore pressures are close to the hydrostatic line and cone resistance values are around 1 MPa. Below 4 m q_t decreases and u_2 increase rapidly.

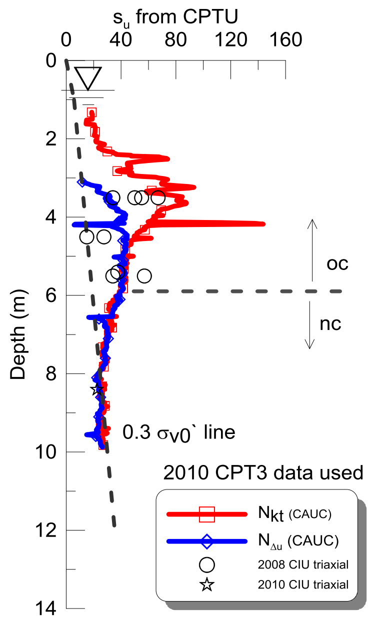


Figure 25 Undrained shear strength from CPTU tests

The pore pressure profile in particular, indicated another clear boundary at about 9 m. This boundary coincides with the location of peak S_t. This finding warrants further investigation

Undrained shear strength (s_u) for clay soils is often determined using CPTU results. A series of empirical bearing capacity factors

e.g. N_{kt} and $N_{\Delta u}$ (Lunne et al., 1997) have been used for this purpose, e.g.:

$$s_u = \frac{q_{net}}{N_{kt}} = \frac{q_t - \sigma_{v0}}{N_{kt}} \qquad (6)$$

$$s_u = \frac{\Delta u}{N_{\Delta u}} \qquad (7)$$

The main issue, of course, involves the choice of suitable N_{kt} and $N_{\Delta u}$ values. Fortunately Karlsrud et al. (2005) have developed a series of such factors for Norwegian clays based on CAUC (anisotropically consolidated undrained triaxial tests) on high quality Sherbrooke block samples and on high quality CPTU testing. For Dragvoll clay Karlsrud et al.'s work would suggest N_{kt} and $N_{\Delta u}$ values of 10 and 6 respectively. The corresponding s_u profiles are plotted on Figure 25 and would appear to give very reasonable results showing a high strength overconsolidated zone above about 6 m and a normally consolidated zone below this.

Triaxial test results (see Section 6.10.2) are also superimposed on this figure and the results are in broad agreement with the CPTU derived values.

6.14 In situ strength – piezoball testing

Ball probes are becoming increasingly popular for characterising soft sediments, particularly in offshore environments. Ball probes are often used for deep profiling as they can fit inside casing used for offshore works. In these tests the cone end is removed and is replaced by a ball, typically of diameter 113 mm (i.e. area = 100 cm^2, 10 times at of a conventional cone).

Given the known reliability of pore pressure measurements in soft clays, pore pressure sensors have been added to the ball (Kelleher and Randolph, 2005), (Peuchen et al., 2005) and (Boylan et al., 2007). The balls used were developed by Benthic Geotech, Fugro and Lankelma respectively.

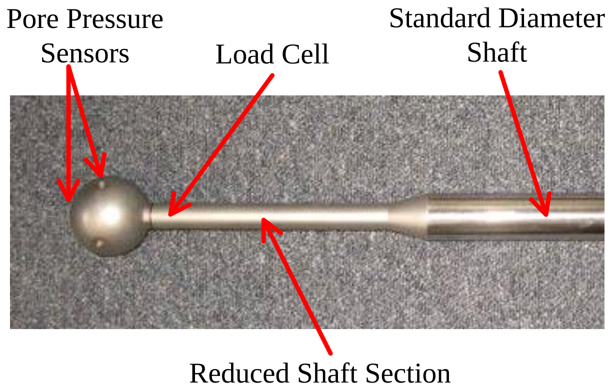


Figure 26 Piezoball developed by University of Western Australia (UWA)

Some balls have recently been introduced which permit the measurement of pore pressure at several locations. The UWA piezoball, described by Boylan et al. (2011), and shown on Figure 26, is 60 mm in diameter has 4 small sensors around the equator of the ball and one at its tip. This ball has been used on Dragvoll clay. Details of this work are given elsewhere (Boylan et al., 2012). However some sample data are shown on Figure 27 below. The data shown is the corrected ball resistance q_{ball} and the tip and equator pore pressures. Corrected ball resistance is normally calculated using (Randolph, 2004):

$$q_{Ball} = q_m - [\sigma_{vo}\text{-}(1-a)u_0]\frac{A_s}{A_p} \qquad (8)$$

where:
A = net area ratio of the cone
A_s/A_p is the ratio of shaft area to the bearing area of the probe, which in this case is 0.1

Randolph et al. (2007) suggested that this equation is not quiet correct and a more precise solution would involve u_2 and σ_v' (parameters which are not always available)

A similar picture to that shown by the CPTU emerges. Not surprisingly the tip pore pressures are greater than those at the equator. The data also seems to indicate a boundary within the normally / lightly over consolidated zone coinciding with the area of highest sensitivity.

Undrained shear strength can be determined for the Ball by:

$$s_u = \frac{q_{Ball}}{N_{Ball}} \qquad (9)$$

where N_{Ball} is the bearing capacity factors for the Ball. Plasticity solutions based on simplified assumptions of soil behaviour have been frequently used to determine N_{Ball}, see for example Randolph (2004). Typically a value of 12 is used. However this factor would lead to very low values of s_u for Dragvoll clay and further work is required to establish the relationship between q_{ball} and s_u for this material, see Boylan et al. (2012)

6.15 Drained shear strength

Triaxial tests show effective cohesion c' and tanϕ' (effective friction angle) were typically 6 kPa and 0.5 (i.e. ϕ'≈ 27°) respectively.

6.16 Comparison between Dragvoll and Tiller sites

The data presented above confirm that the clay underlying the Kvenild (Tiller) and Dragvoll sites are relatively very similar. This can be seen from the summary of index test data on Table 3 and from the results of some CPTU tests undertaken at Tiller in 2010, see Bihs et al. (2012) as shown on Figure 28. The q_t and u_2 data are very similar. There is also an upper over consolidated zone at Tiller and both profiles show a similar increase in q_t with depth. The Tiller data does however show a greater rate of increase in u_2 with depth.

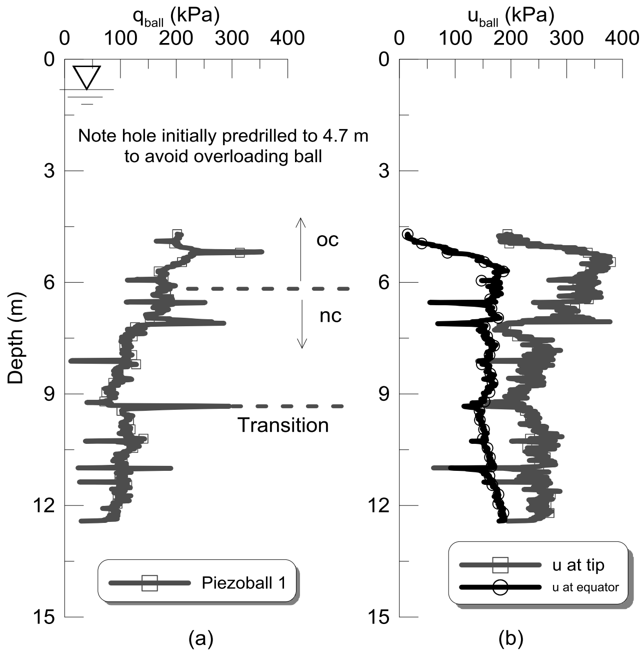


Figure 27 Sample piezoball results

Table 3. Average index test results Kvenild / Dragvoll

Parameter	w	ρ_b	I_p	clay
Unit	%	Mg/m3	%	%
Kvenild	36	1.9	5	38
Dargvoll	32.5	1.95	8	39

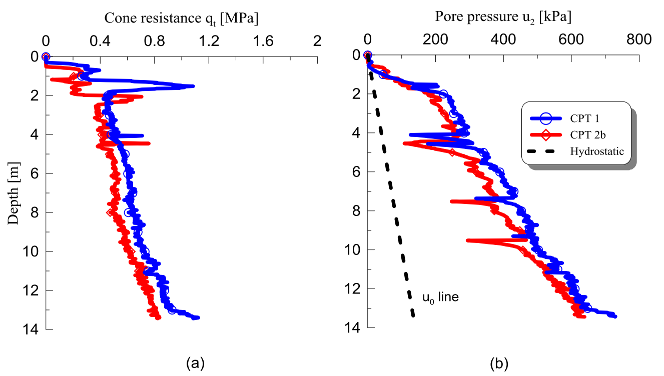


Figure 28 CPTU tests from Tiller site from 2007 (a) q_t and (b) u_2

7. ENGINEERING PROBLEMS AND CASE HISTORIES

7.1 Landslides in quick clay

Figure 2 illustrates the importance and significance of quick clay landslides in the Torndheim area. The Dragvoll material has been show to be typical of that encountered in the area and thus forms a very good research materials for studying the properties of quick clay and quick clay landslide mechanisms.

7.2 Building to foundations

Because the natures of the soft clays, major building foundations in the area have largely been piled, for example those of the NTNU sports hall, which is located just east of the study area, see Figure 3.

7.3 Model scale lime cement columns

Aunaas (2007) reports on a series of in situ tests carried out on small scale lime cement columns at the Dragvoll site (close to the small creek south of the main test area)

8. CONCLUSIONS

1. This paper has detailed the characteristics and engineering properties of Dragvoll clay, a thick deposit of highly sensitive and sometimes quick marine clay. A variety of in situ devices and different laboratory tests have been used to investigate its properties. As well as its high sensitivity, the clay is characterised by its low strength and high compressibility. There is a relatively shallow overconsolidated zone towards the top of the stratum.
2. The Dragvoll clay is typical of quick clays encountered in the Trondheim area and thus forms a good reference / research material
3. The material is very sensitive to sampling and laboratory handling.
4. In order to fully understand the properties and behavior of the material some high quality block sampling and associated laboratory testing should be undertaken as a priority.

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