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Title of paper: Engineering characterisation of Norwegian glaciomarine silt

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Title: Engineering characterisation of Norwegian glaciomarine silt

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ABSTRACT: Guidance is provided for geotechnical engineers designing civil engineering works in silty soils based on a detailed characterisation of a glaciomarine silt from Os in western Norway. It was found that these soils are susceptible to disturbance by good quality fixed piston tube sampling and care needs to be taken when using laboratory derived design parameters, particularly for consolidation and shear strength properties. A technique for assessing sample disturbance using shear wave velocity and suction measurements proved promising. Conventional techniques for determining soil strength from triaxial tests in silt are inappropriate due to the dilational nature of the material and more reliable and logical strength estimates can be made from a limiting strain criterion. Field vane data should be used with caution as measured strength, particularly remoulded values, may be high and it seems more reliable parameters can be derived from CPTU tests. One dimensional consolidation and creep of these silts can be modelled successfully by the well-known Janbu formulation. The behaviour of the Os silts does not fit easily into classical soil mechanics and published frameworks for soft soils. It seems the material is of “transitional” type and this work adds to the database of such soils which includes other natural silts and gap graded soils. For future work it is recommended that larger sample tubes (say 75 mm) with a very sharp cutting edge should be used in parallel with in situ CPTU testing.

1. Introduction

Much geotechnical research has been conducted on clay and sand soils. Little information exists in the literature on the engineering properties of “intermediate” silty soils and how these are related to their geological origin. The reasons for this are probably related to the difficulties in sampling the material and subsequently preparing it for laboratory testing. As a

result much of the work that has been carried out is on reconstituted material. Some examples of research on “undisturbed” material have been reported by Schultze and Horn (1965), Börgesson (1981), Skúlason (1996), Høeg et al. (2000), Sandven (2003) and Long (2007) and these will be referred to in later sections.

In particular there is little guidance available for practicing engineers on how to choose design parameters, especially for problems related to 1D compression and strength. Construction of new highways in coastal areas of Norway, for example near the major cities of Bergen and Stavanger, frequently involves crossing sites underlain by silty soils.

The purpose of the work described here is to characterise typical Norwegian glaciomarine silt in detail, with a view to developing guidelines for practicing engineers working with these soils and other similar soils worldwide. A particular objective was to examine the influence of sampling disturbance on the measured soil properties. This was achieved by trying out a number of different soil samplers at the site. A separate focus was to examine the applicability of two well know in situ investigation techniques namely the vane test and cone penetration test, as well as the more recently develop T-bar test, to see if these formed a reliable alternative to sampling and laboratory testing.

The behaviour of the Os silt will be compared to that of other similar soils and an assessment of how the material fits into published frameworks for soft soils will be made.

2. The site

2.1 General

The location of the site in Norway is near Os in the Bergen area and the detailed test locations are shown on Figures 1a to 1c respectively. It is a triangular plot of relatively flat land with the longest side of the site being about 80 m and an elevation above sea level of 27.2 m. Some road improvement works had been carried out adjacent to the site and it was

chosen for this research because it was known to be underlain by a relatively thick deposit of silty soils, which were characteristic of those found in the area.

2.2 *Background geology*

Bondevik and Mangerud (2002) describe the geological origin of the silt deposits in the area. The silt was deposited at a time when the ice sheet reached and remained in its most southerly position between 11,600 and 11,700 years before present. Silty deposits were accumulated in a shallow sea adjacent to the glacier front. In general these deposits infilled natural topographical depressions forming relatively horizontal terraces. Outcrops of the material in the area show horizontal laminated silt and fine sand.

2.3 *Previous investigations*

Some ground investigation was carried out by the Norwegian Public Roads Administration (Statens vegvesen, 2002) as part of the road improvement scheme. This work comprised a series of “total sounding” tests (rotary percussive boring described by Rygg and Andresen, 1988), which were used to define the general stratigraphy of the area together with one borehole from which four conventional 54 mm diameter composite samples (see below for details) were retrieved.

Phase 1 of this research project was carried out in 2004 / 2005 (Gudjonsson, 2005, Long and Gudjonsson, (2005a and 2005b). This comprised:

- Further total sounding tests
- Drilling and sampling from 3 boreholes
- Four CPTU tests (by two different operators)
- Two sets of in situ vane tests
- Two T-bar tests

The sampling work was largely used for basic material identification. In order to study the effects of sampling on the Os silt a second phase of field work was carried out in October

2006 and the result of testing of these (Phase 2) samples together with the results of the in situ tests from Phase 1 are the subject of this paper.

3. Drilling, sampling and in situ testing techniques

3.1 General

The location of all the tests under consideration is shown on Figure 1c. It can be seen that all of the work was carried out in an area of about 8 m x 8 m, thus ensuring natural material variability effects were minimised.

3.2 Drilling and sampling

For the Phase 2 works, sampling was by means of standard Norwegian 54 mm steel and 54 mm composite samplers. The 54 mm steel sampler was developed and designed by NGI (Norwegian Geotechnical Institute) in the 1950's and 1960's (Andresen and Kolstad, 1979, Andresen, 1981). The pertinent dimensions of the sampler are summarised on Table 1. Following pre-augering through the crust, sampling is carried out by the displacement method. The sampler (with the piston in front of the sample tube) is pushed down to the desired depth without further pre-augering. During sampling the inner rods and the piston are fixed in a locked position, and the outer rods are pushed down at a constant rate. After withdrawal of the sampler, the sample is sealed at the top by leaving the piston in place when the cylinder is disconnected from the sampler. Further details are given in Andresen and Kolstad (1979) and Andresen (1981).

The NGI 54 mm composite sampler was developed at the end of the 1970's and is now the most common sampler used in Norway (Table 1). The sampler is a composite piston sampler using plastic inner liners to prevent corrosion and to avoid practical difficulties with the production of steel cylinders. The 54 mm composite sampler has a relative poor area ratio whereas the 54 mm steel sampler has a comparatively blunt cutting edge angle.

3.3 *In situ* vane testing

A standard Geonor H10 vane of dimensions 65 mm x 130 mm was used. Two series of tests were carried out, each in a dedicated borehole, with mechanical and hand torque devices respectively. Testing was to BS5930 (1999). Rotation of the vane was at about 12° / minute. The vane was rotated about 25 times prior to determining the remoulded shear strength.

3.4 *CPTU and T-bar testing*

Two sets of CPTU (cone penetration tests with pore pressure readings) tests were also carried out, with an ENVI memocone and a Geotech cone respectively. All testing was carried out according to ISSMGE (1999) using standard 10 cm² cones, which were penetrated at the rate of 2 cm/sec.

In soft soils the need to correct the CPTU cone resistance for pore pressure effects (Lunne et al., 1997a) can be very significant. Hence use has recently been made of a T-bar, which has a bearing area 10 times, that of the cone (e.g. see Long and Gudjonsson, 2004). Also, theoretically, one of the attractions of the T-bar over the cone is the existence of closely bracketed plasticity solutions for plane strain flow around a cylinder. In T-bar penetration tests the same equipment is used as for CPTU, except that the conical tip is removed just below the load cell and friction sleeve and replaced by a steel bar, thus forming a T shaped penetration device. The diameter of the T-bar was 40 mm and the length was 250 mm. It was roughened by light sandblasting.

4. Laboratory testing

4.1 *General*

The principal means of studying the difference in behaviour of the material from the different samplers was by means of anisotropically consolidated undrained compression (CAUC) triaxial tests in which the specimens are anisotropically consolidated to the best estimate of the *in situ* stress. A limited number of maintained staged load (MSL) and constant

rate of strain (CRS) oedometer tests were also performed to study 1D compression parameters.

4.2 Triaxial tests

A significant challenge when working with soft silty soils is that the specimens tend to slump under their self-weight during the sample freestanding period prior to filling the triaxial cell with water. This is because they have very low initial residual effective stress (suction). For this work, in order to avoid minimise handling and provide some support to the specimens at all times, they were extruded directly into the rubber membrane / membrane stretcher using a device very similar to that described by Wijewickreme and Sanin (2006). Otherwise the procedures used were broadly those adopted as standard by the Norwegian Geotechnical Institute (NGI), as described by Berre (1982). Specimens of diameter as sampled (5.4 cm) were trimmed to a height of 12 cm. Pore pressure was measured conventionally at the base of the specimen and strain were measured external to the cell using an LVDT. The cell pressure was progressively increased from 25 kPa to 50kPa to 100 kPa and 200 kPa so as the initial specimen effective stress (or suction), u_r , could be estimated. Following a saturation check (B always ≥ 0.95) some isotropic consolidation was carried out at an effective cell pressure of $0.5 \sigma'_{h0}$ (in situ horizontal effective stress) before slowly applying the in situ stress. In the absence of measurements the coefficient of earth pressure at rest (K_0) was assumed to be equal to 0.5, by correlations with plasticity and stress history, from Brooker and Ireland (1965). The final consolidation stresses are kept constant until the rate of volumetric strain is less than 0.0001% per minute. Shearing was carried out using mechanical strain control at the relatively slow rate of 18% per day. Corrections were applied for the restraining effects of the membrane and filter paper.

4.3 Oedometer tests

In the case of both the MSL and CRS tests specimens were extruded directly into 50 mm diameter lubricated oedometer rings. Drainage was two-way for MSL tests and one-way for CRS tests as pore pressures were measured at the specimen base. For the MSL tests the initial load increments was to $0.25\sigma'_{v0}$ (the in situ vertical effective stress) and subsequently a load increment ratio of two was used. Loading was maintained until approximately the end of primary compression. CRS tests were carried in a ring mounted in a normal triaxial cell. Displacements were measured external to the cell using an LVDT and strain rate was controlled mechanically at an axial strain rate of 3% per hour, which resulted in excess pore pressures of between 5% and 7% of the applied total stress. Otherwise the procedures used were again broadly those adopted as standard by NGI (Sandbækken et al., 1986).

4.4 Bender element tests

Measurement of in-situ shear wave velocity was performed using the Multichannel Analysis of Surface Waves (MASW) technique (Donohue, 2005, Long and Donohue, 2007). In the lab measurements of shear wave velocity were performed using bender elements. The bender element system involved V_{vh} (vertically propagating, horizontally polarised shear wave) measurements on unconfined samples of 54 mm diameter and about 50 mm high. These specimens were independent of but from a depth immediately adjacent to those used for the CAUC triaxial testing, thereby enabling a direct comparison with parameters determined in each test. An arbitrary function generator (Tecstar FGA 2030) was used for signal generation in the bender element system and detection and observation of the resultant output was made using a portable digital storage oscilloscope (DSO), a Picoscope ADC 212/100. A sinusoidal input wave was used and the time of first arrival was chosen visually by the user. The frequency of the input wave was varied between 1 kHz and 10 kHz and the V_s value quoted corresponds to a d/λ (travel distance over wavelength) ratio of ≥ 4 . Sanches-

Salinero et al. (1986) proposed the use of this criterion so as to control the degree of attenuation of the received trace due to geometric damping.

5. Site characterisation

5.1 Ground conditions

Visual inspection of recovered samples confirms that the Os silt comprises two distinct soft strata as can be seen on Figure 2. Beneath a desiccated crust of some 3 m thickness a soft to firm grey silt was noted. Below about 6 m the material became more clayey and could be described as clayey silt. The water table is located at about 3.5 m, towards the base of the desiccated crust.

The stratigraphy of the site is also confirmed by the results of the four CPTU tests, see Figure 2. In the zone of sampling it can be seen that the material is relatively uniform with corrected cone resistance (q_t) increasing from about 0.35 MPa at 3.5 m to 0.6 MPa at 9 m. There is a zone of higher consistency at about 5.5 m where q_t is about 0.7 MPa. Generated pore pressure values (u_2) are well in excess of the “hydrostatic” line with little dissipation of excess pore pressure occurring during penetration. For q_t and u_2 the Geotech and Envi equipment give very similar results.

However there is a significant difference between the sleeve friction (f_s) and hence friction ratio (R_f , Lunne et al., 1997a) for the two sets of equipment. The Geotech equipment gives R_f of about 1.5% in the sampling zone whereas the Envi equipment gives values less than 0.5%. According to ISSMGE (1999) the required minimum accuracy for f_s in a Class 2 CPTU test is 15 kPa. Here the measured f_s values are typically in the range 3 to 8 kPa, i.e. less than half the required accuracy. Thus the variation in R_f is not surprising but has very important implications if CPTU f_s data is being used to classify soil, using a classification chart such as the well-known one of Robertson et al. (1986).

The profile of pore pressure parameter (B_q , see Lunne et al., 1997a) clearly distinguishes between the two zones with $B_q \approx 0.2$ above 6 m and ≈ 0.6 below 6 m. Robertson et al. (1986), Schnaid et al. (2004) and other have suggested that if B_q is between about 0.3 and 0.5 the cone penetration is through “mainly undrained silty soils”, whereas if B_q is greater than 0.5 penetration is through “mainly undrained clayey soils”.

T-bar penetration resistance values are less than those of the CPTU. The difference is greatest in the lower more clayey material where the resistance is relatively constant at about 0.25 MPa.

5.2 Basic material parameters

Basic material properties are shown on Fig. 3. Despite some scatter in the data, values of water content (w) and bulk density (ρ), from triaxial, oedometer and bender element tests, are relatively constant with depth. Average values for w and ρ are about 33.2% and 2.0 Mg/m³ respectively. Though there is no difference in the average result, the scatter for the oedometer data is somewhat greater and this may reflect some disturbance caused by pushing a 50 mm oedometer ring into a 54 mm specimen. The scatter in the data is consistent with its glaciomarine geological origin.

Sensitivity (S_t) values from both the fall cone index test and the field vane are also shown on Figure 3. Values from the index tests suggests much higher sensitivity, with values being of the order of 70 above 6 m and up to 300 below 6 m. According to NGF (1982) a material can be classified as “high sensitivity” if $S_t \geq 30$ and “quick” if $S_t \geq 30$ and remoulded shear strength (s_{ur}) ≤ 0.5 kPa. The latter is the case for the material below 6 m but that above 6 m has s_{ur} generally greater than 0.5 kPa and can therefore be classified as “high sensitivity”. When the material is remoulded by hand in the laboratory for the fall cone index test it changes easily into a liquid state indicative of high sensitivity or quick material and this suggests that the field vane values of s_{ur} are unrealistically high. It is possible that either 25

rotations of the field vane in silty soils are insufficient to completely remould the material or that partial drainage during remoulding may have resulted in relatively high field s_{ur} values.

5.3 Particle size distribution

Particle size distribution curves are shown on Figure 4a and the distribution of the various constituents with depth is shown on Figure 4b. These were hydrometer tests carried out on triaxial test specimens following the triaxial test. Consistent with the physical appearance of the material and the results of the CPTU tests, there is a clear difference between data from above and below 6 m. Above 6 m clay content ($< 2 \mu\text{m}$) is very low being generally less than 5%. Below 6 m there is on average about 12% clay and it never exceeds 15%. All the remaining particles are made up of those of silt size. According to NGF (1982), if less than 15% of the material (by mass) is of clay size then the main term used to describe it should be “SILT”. There is no difference in the results from the 54 mm steel and 54 mm composite samples.

5.4 Plasticity

Atterberg limit tests demonstrate the material has average liquid limit (w_L) and plasticity index (I_p) of 33% and 12% respectively. The data falls on or just above the A-line. Average liquidity index (I_L) is about 0.93.

5.5 Other fundamental material properties

Other fundamental material properties are summarised on Table 2. Average value of particle density is relatively high at 2.94 Mg/m^3 , but not unusual for this area of Norway. Unfortunately no detailed study of the mineralogy or microfabric of the Os silt was carried out. However the bedrock close to the study area comprise Upper Ordovician–Lower Silurian rocks and specifically consisting mainly of phyllites, but including carbonate and graphite-rich schist (Bondevik and Mangerud, 2002, Ingdahl, 1989) and therefore the particles are likely to be relatively incompressible and are unlikely to crush even at high stresses.

Salt content of the pore fluid is very low. This is consistent with the nature of a highly sensitive or quick material and confirms that the material has been leached by fresh water flow subsequent to deposition (Bondevik and Mangerud, 2002).

5.6. *Sampling induced densification*

In silty material it is possible that partial drainage during sample tube insertion will densify the material (i.e. increase ρ) and cause a corresponding reduction in w . An examination of the data presented on Figure 3 suggests that specimens obtained from the 54 mm composite sample tubes have slightly lower w and higher ρ than the 54 mm steel samples. For example, for the Bender element tests, average values for the composite and steel samples are 31.4% / 2.04 Mg/m³ and 32.6% / 2.0 Mg/m³ respectively. However insufficient evidence exists to confirm whether this is due to sampling effects or simple natural material variability.

6. **Sample quality assessment**

Prior to consideration of triaxial and oedometer test results it is first necessary to examine the sample quality. In Norway and elsewhere sample quality is typically assessed by determining the normalised volume change ($\epsilon_{v0} = \Delta V/V_0$) during loading back to the in situ stress or alternatively the normalised void ratio change ($\Delta e/e_0$) to the same stress (Andresen and Kolstad, 1979, Lunne et al., 1997b). Data for this work is shown on Figure 5, together with some quality criteria proposed by Lunne et al. (1997b) and the following can be noted:

- the upper silt layer specimens are generally classified as “good to fair”,
- the specimens from the lower clayey silt fall in the “poor” or “very poor” zones
- there is a general reduction in specimen quality with depth,
- there is no significant difference between the oedometer and triaxial test results,
- the 54 mm composite and 54 mm steel samples are of similar quality (average $\Delta e/e_0$ for the 54 mm composite samples is marginally lower).

As it is possible that the material was densified during the tube sampling and thus the ε_{v0} or $\Delta e/e_0$ criteria may be lower than for an undisturbed sample and therefore may not accurately reflect the sample quality, an alternative technique based on shear wave velocity and suction measurements was assessed (see e.g. Hight and Leroueil, 2003, Donohue, 2005 and Donohue and Long, 2007).

Suction data (u_r) for the triaxial test samples and bender element shear wave velocity measurements (V_s) on adjacent samples are shown on Figure 6. The suction values are compared to $1/5 \sigma_{v0}'$ as Tanaka et al. (1996) have suggested that for high quality samples of normally to slightly overconsolidated clay u_r is approximately $1/5 \sigma_{v0}'$ to $1/6 \sigma_{v0}'$. Recently, Tanaka & Tanaka (2006) and Tanaka (2008) suggested that suction does not have a consistent relationship with in-situ vertical effective stress and that this hypothesis needs to be treated cautiously.

It can be seen that the upper silt material has u_r values close to $1/5 \sigma_{v0}'$ whereas the u_r values for the lower clayey silt are well below the $1/5 \sigma_{v0}'$ line. Again there seems to be a reduction in the sample quality with depth and there is no clear difference in quality between the 54 mm composite and 54 mm steel samples.

Uncorrected shear wave velocity values on unconfined samples are compared to in situ values determined using the MASW surface wave technique (Long and Donohue, 2007) and to values on remoulded samples (at the same water content and density as the intact specimens). No correction has been made for effective stress as previous experience has shown that uncorrected unconfined values give the best indication of sample disturbance. It can be seen that all of the values are relatively low compared to the in situ data and are much closer to the remoulded values. Average V_s for the 54 mm steel samples is about 50.1 m/s compared to 38.6 m/s for the 54 mm composite samples, suggesting the steel samples are less disturbed.

Donohue (2005) and Donohue and Long (2007, 2008, 2009) have suggested that the V_s and u_r values can be combined to provide an assessment of sample quality. Two normalised parameters, L_{vs} (loss of velocity) and L_u (loss of suction) were used to evaluate disturbance where:

$$L_{vs} = \frac{V_s^{insitu} - V_s}{V_s^{insitu} - V_s^{remoulded}} \quad (1)$$

$$L_u = \frac{0.2\sigma'_{v0} - u_r}{0.2\sigma'_{v0}} \quad (2)$$

Theoretically use should be made of perfect sampling stress, σ_{ps}' (Ladd and Lambe, 1963), or σ_{v0}' instead of $0.2 \sigma_{v0}'$. This would make determination of sample quality difficult for this site as the differences in L_u would be very small. It may be more relevant to sites where higher suctions are measured.

Data for this site are shown on Figure 7 together with some tentative quality criteria. Two of the samples, both from the 54mm steel sampler fall in the “good to fair” category. All of the other data points fall in the tentative “poor” category.

7. Behaviour in triaxial tests

7.1 Stress – strain behaviour

Results of CAUC triaxial tests for the upper silt, lower clayey silt 7 m and 9 m samples and lower clay silt 8 m and 10 m samples are given on Figures 8a to 8c respectively. In each case pairs of test results are presented, i.e. tests from the two different samplers at the same depth. Data is presented in the form of shear stress (t') [$t' = (\sigma_a' - \sigma_r') / 2$] versus axial strain (ϵ_a) and as a mean stress / shear stress (s', t') stress path plot [$s' = (\sigma_a' + \sigma_r') / 2$].

For the upper silt material (Fig. 8a) all tests show dilatant behaviour with shear stress increasing with increasing ϵ_a at least up to axial strains of 10%. For the most part the stress paths dilate strongly and form a clear failure line. There is no clear difference between the results for the two different sampler types.

Results for the lower clayey silt are less consistent. The 10 m samples, which have the highest clay content of about 15%, both show contractant behaviour with pronounced strain softening post peak. However for the other 3 pairs of tests one specimen shows dilatant behaviour and the other shows contractant. There is no consistent pattern relating the behaviour to the sample type or clay content (which in any case is more or less the same for each pair). All of the stress paths form a clear failure line.

When examining the stress – strain behaviour of the material knowledge of the overconsolidation ratio is important. Here this is difficult to assess reliably from the oedometer curves (see Figure 12 below) due to a combination of sample disturbance and the silty nature of the material. A best estimate from the oedometer tests and from the CPTU results (Lunne et al., 1997a suggest that if q_t is of the order of $2.5\sigma_{v0}'$ to $5\sigma_{v0}'$ then the material is probably lightly overconsolidated) is that the material is normally to lightly overconsolidated.

A preliminary pre-test assessment of the likely behaviour of the material can be made from the initial voids ratio (e_0). For tests where e_0 is greater than 0.9 or less than 0.8, the behaviour was strongly contractant or strongly dilatant respectively. For e_0 between 0.8 and 0.9, the specimens tended to initially contract and then subsequently dilate strongly. As the clay content in each pair of specimens is more or less equal, any inconsistent behaviour is due to a combination of natural material variability and disturbance effects either during sampling or during building the specimens into the triaxial cell. Nonetheless the practical implication of these results is that care needs to be exercised when choosing design parameters for engineering works as will be discussed below.

7.2 Pore pressure response

Pore pressure response for the 54 mm composite samples is shown on Figure 9. The 54 mm steel samples showed almost identical behaviour. Despite some noise in the data, it can

be seen that there is a clear difference between the response of the upper silt and lower clayey silt. The upper silt behaves in a dilatant manner with pore pressure change reducing below zero for the 5m test. For a particular strain in the clayey silt the change pore pressure increases with increasing depth of the specimen, probably reflecting the gradual increase in clay content with depth.

7.3 Effective stress strength parameters

Effective stress strength parameters (ϕ' , c') are required for long term stability analyses. Current practice is to obtain these parameters from triaxial testing. Usually a generous safety factor is applied. As can be seen from Figures 8a to 8c the material under investigation shows a large strain friction angle, ϕ' , value of about 35° , perhaps not inconsistent with the silty nature of the material.

Others have reported similar results, e.g. Schultze and Horn (1965) for German silt ($\phi' = 36^\circ$), Börgesson (1981) for silt from northern Sweden (ϕ' up to 40°), Skúlason (1996) for Icelandic silt ($\phi' = 40^\circ$ and greater), Høeg et al. (2000) who found ϕ' of about 37° also for Swedish silt and Long (2007) who found ϕ' values in excess of 40° for two Irish silts. Although reasonably high friction angles would be expected for these materials, as they have low clay mineral content, some of the quoted values seem somewhat higher than would normally be expected for loose silty material and there must be at least some suspicion of sampling induced densification.

Friction angle values can also be estimated from CPTU data (Senneset et al., 1988 reproduced in Lunne et al., 1997a) by comparing the bearing capacity number ($N_m = q_{net}/\sigma'_{v0}$) with pore pressure parameter B_q . Taking the chart corresponding to lightly overconsolidated silts (angle of plastification, β , assumed to be 0°):

- For the upper silt, above 6 m, N_m is typically 10.5, $B_q = 0.2$, $\tan \phi' = 0.62$, $\phi' = 32^\circ$
- Below 6 in the clayey silt, N_m is typically 7.5, $B_q = 0.6$, $\tan \phi' = 0.77$, $\phi' = 38^\circ$

These values are consistent with the triaxial test results. In practice, in order to provide sufficient safety factor and to minimise strains, a safety factor typically 1.3 on $\tan \phi'$ is applied. In this case this would result in a design value of about 28° , which corresponds to strains of 0.5% to 1.0%, which are perhaps reasonable, if near the high side, of allowable working values.

7.4 Undrained shear strength (s_u)

Several researchers (e.g. Senneset et al., 1982 and Sandven, 2003) have noted that use of s_u values are inappropriate for soils where $B_q < 0.4$ as the response of the material under load will be at least partially drained. This criterion applies to material coarser than clayey silt. As an alternative they suggest using an effective stress approach. Although B_q values are indeed low for the upper silt, some discussion on s_u is necessary as this parameter is used frequently by practicing engineers both directly and in correlations.

Undrained shear strength values (s_u) from index tests are shown on Fig. 10a. These mostly comprised fall cone tests and there are a small number of unconfined compression tests. Values of s_u increase from 20 kPa at 3.5 m to a maximum of 50 kPa at 5.5 m and then decrease again to about 20 kPa at 8.5 m. The slightly higher values around 5.5 m are consistent with higher CPTU q_t values in this zone, see Figure 2. There is no significant difference between the results from the two types of test and it is possible that the results are influenced by partial drainage in the silt.

Results of in situ field vane tests are shown on Fig. 10b. There is little difference between the results of the different tests and it can be seen that the s_u profile is very similar to that from the index test both in magnitude and shape. Above 6 m, these values are in excess of the typical $0.3\sigma_{v0}'$ quoted for normally consolidated clays (Hight et al., 1987), suggesting that either the material is lightly overconsolidated or that, despite the tests being carried out

relatively rapidly at about 12° / minute, they are partially drained. In the clayey silt, below 6 m, s_u values fall below the $0.3\sigma_{v0}'$ relationship.

Both sets of CAUC s_u values (simple peak), as shown on Figure 10c, are similar to the index tests and field vane data in the upper silt. However in the lower clayey silt s_u is significantly higher and is approximately equal to $0.3\sigma_{v0}'$. It is not clear how to interpret of s_u from triaxial tests on silt due to their tendency to dilate during shear. Any assessment of the dilational component of the undrained shear strength of silts needs to account for possible effects of densification due to reconsolidation and sampling. Establishing a consistent failure criterion for use in practical application for silts (e.g. in foundation design or in the short term stability of cuts and embankments) can be very difficult. This issue has been studied in detail by Brandon et al. (2006) for silts from the Lower Mississippi valley. Some possibilities for the determination of s_u for silt are as follows:

1. Simple peak deviator stress regardless of strain (conventional approach)
2. Shear stress at some limiting strain
3. Pore pressure parameter $A = 0$ or $\Delta u = 0$
4. Reaching Mohr - Coulomb line
5. Peak principle stress ratio (σ_1'/σ_3')
6. Peak pore pressure

There is little guidance in the literature as to which criterion is most appropriate. Börgesson (1981) used criterion 2 with a limiting strain of 10%. Wang and Vivatrat (1982) and Fleming and Duncan (1990) also advocated use of this criterion with limiting strains of 5% to 15%. Long (2007) recommended adopting criteria 2 (at 2% strain) or 6. Stark et al. (1992) used both criteria 1 and 6. Brandon et al. (2006) suggested that criterion 1 produced too much scatter and found that relating s_u to changes in pore pressure provided a useful and practical framework. They found that taking s_u at the point when $A = 0$ gave reasonable

values of s_u and reduced the scatter in the results compared to other methods. The basis of this approach is that reliance is not placed on strength that results from negative changes in pore pressure.

In this case, as can be seen from Fig. 9, criterion 3 does not generally apply to the Os silt. In eight of the twelve tests considered, peak pore pressure occurs at about 2% strain, so therefore criterion 2 and criterion 6 are co-incident if 2% strain is selected. For the purposes of this study criteria 1, 2 (at 2% strain) and 4 are compared and the results are presented on Figure 11. Lines representing $0.3\sigma_{v0}'$ and $0.5\sigma_{v0}'$ and the trend from CPTU are also shown for guidance. Overall there is not a large difference between the results from the different methods. It can be seen however that the use of criteria 2 or 4 removes the unusually high data points and results in less scatter of the data. The resulting values are consistently just below the $0.3\sigma_{v0}'$ line. Criterion 4 gives slightly higher values than criterion 2. Again there is no difference between the results for the two sampler types. From the point of view of engineering practice use of these 2 criteria is logical, as it will result in strains around engineering structures being limited to 2% and ensure no dilation (i.e. negative pore pressures) is permitted.

Undrained shear strength can also be derived from CPTU data (Lunne et al, 1997a) by means of the bearing capacity factors relating to corrected cone resistance (N_{kt}), excess pore pressure ($N_{\Delta u}$) or effective cone resistance (N_{ke}). These were chosen to be 11, 9, and 6 respectively after Lunne et al. (1997a) and Karlsrud et al. (2005). Results from the $N_{\Delta u}$ approach seem unreliable especially in the upper silt. Data from the N_{kt} or N_{ke} approaches are more encouraging. Of the two perhaps that use of N_{kt} is most reliable as determination of the effective cone resistance (q_e) is required for the N_{ke} approach. For soft clays and loose silts the q_e values will be very small thus making the technique less reliable.

8. Behaviour in 1d compression

8.1 General note

In current practice in Ireland, the UK and elsewhere use is generally made of Terzaghi's theory of 1D consolidation. This involves obtaining primary consolidation properties from plots of oedometer log effective stress (σ'_v) versus strain (ε) or void ratio (e) and creep properties from plots of ε or e against log time. Whereas this theory may work well for uniform soft clays, in the author's experience it is difficult to apply to silty materials. This is due to the non-linearity of the resulting curves and to the artificial separation of primary consolidation and creep effects.

Although conventional curves will also be presented here, use is made in general of Janbu's (1985) theory for primary and secondary settlements, in which the stress induced primary consolidation is calculated with an effective stress dependent tangent modulus, and the time dependent secondary consolidation is determined using the "time – resistance" concept. In this theory it is not necessary to separate primary and secondary consolidation phases because in practice creep takes place in all parts of the process.

8.2 Behaviour in 1D compression

A comparison of MSL oedometer test log σ'_v versus e and σ'_v versus constrained modulus ($M = \Delta\sigma'_v/\Delta\varepsilon$) curves, for samples of the upper silt from between 5.3 m and 5.6 m and the lower clayey silt from between 6.2 m and 10.2 m are shown on Figures 12a and 12b respectively. Note some additional Phase 1 tests from a 76 mm steel sampler (identical otherwise to the 54 mm steel sampler has been included. There is a distinct difference in behaviour for the two zones. For the upper silt the classical log σ'_v versus e curves are flat and may indicate some densification of the material (Long, 2007). It is impossible to determine the yield or preconsolidation stress. Other researchers have shown that a flattening of the log σ'_v versus e is characteristic of transitional soils (e.g. Martins et al., 2001). For the lower

clayey silt the curves are somewhat more rounded and some rough estimate of the yield stress is possible. There is no difference between the test results for the three samplers.

According to Janbu (1985) silt or sand material will show a gradual increasing M with increasing σ_v' as the particles are compressed tighter together. He suggested the material could be characterised by a power function as follows:

$$M = mp_a \left(\frac{\sigma_v'}{p_a} \right)^{1-a} \quad (3)$$

where:

m = modulus number

p_a = reference stress = 100 kPa

a = exponent = 0.25 for silt

As can be seen from Figure 12a this model can be used successfully to characterise the behaviour of the upper Os silt. For the case shown if the modulus number m is chosen to be about 70, the resulting model curve fits the measured data reasonably well. Janbu's model has also been successfully applied elsewhere, for example to Icelandic silts by Skúlason (1996) and to Irish silts by Long (2007). However data from the lower clay silt (Figure 12b) indicates that the material behaves like a clay with:

$$M = m\sigma_v' \quad (4)$$

and if $m = 16$ (Janbu, 1985) is chosen the model again fits the data quite well.

8.3 1D Compression parameters

Values of M_0 (i.e. M at in situ vertical effective stress σ_{v0}') are shown on Figure 13a and are between 2 MPa and 3.5 MPa. M_0 can also be determined from CPTU data from the equation (Lunne et al., 1997a):

$$M_0 = \alpha_i q_{net} \quad (5)$$

Lunne et al (1997a), Senneset et al. (1988) and Sandven (2003) suggest α_i is in the range 2 to 10 for silty soils. Choosing $\alpha_i = 5$ gives a profile of M_0 almost exactly the same as the values derived from the oedometer tests.

Janbu (1985) found that there was a strong relationship between initial water content and modulus number for Norwegian marine clays and silts. Data for this site together with Janbu's trendlines, for water content in the range 26% to 70%, are shown on Figure 13b. It can be seen that the data fit well within the typical limits suggested by Janbu.

Only a limited number of load increments were maintained for sufficient time to permit Janbu's (1985) creep number r_s to be determined. However for the Os silts r_s varied between 140 and 250, with an average of about 210. As for m this value is at the lower bound of those suggested by Janbu for clay with water content of about 30%. Janbu (1985) also suggested r_s will increase with increasing stress but no such clear trend was found here.

9. Discussion with respect to sampling disturbance

A primary objective of this work was to assess the effects of sample disturbance on the engineering behaviour of the material. It had been expected that the 54 mm steel samples would be of higher quality due to the relatively poor cutting edge geometry (area ratio) of the 54 mm composite sampler. Although it is tempting to conclude that the 54 mm steel samples were indeed of better quality, as they displayed less effects of densification (slightly lower density and slightly higher water content) and the average V_s was some 33% higher, all other parameters point to the quality of both sets of samples being more or less identical. It is possible that the sharp cutting edge used with the 54 mm composite sampler compensated for the relatively poor area ratio.

For the upper silt the specimens can be classified as "good to fair". Despite the consistency of the data, and the good correlation between field and lab s_u and between lab and CPTU derived ϕ' and M , the lower clayey silt samples are generally classified as "poor". As

these classification criteria which were originally developed for clay it would lead one to at least question whether these specimens were really “poor”. Although there does seem to be some promise in assessing sample disturbance using shear wave velocity and suction measurements, a conclusion cannot be made to this point without parallel high quality samples such as large blocks (e.g. using the Sherbrooke block sampler, Lefebvre and Poulin, 1979). Extracting such samples would be problematic and very expensive.

As block sampling will certainly be prohibitive in routine investigations, there is clearly a need to improve sample quality in silty material. For future work it is recommended that larger sample tubes (say 75 mm) with a very sharp cutting edge should be used. Also in order to address the issue of reducing sample quality with depth consideration should be given to pre-drilling a borehole to just above the sampling location and to supporting the borehole with drilling mud. In future investigation sampling should always be done in parallel with in situ CPTU testing

10. Comparison with other silts

10.1 1D compression behaviour

It is important to place the results of this work in a broader context by comparing these data to that of other silts and to published frameworks for soft soils. Data for one dimensional compression tests for the Os silt, shown on Figures 12a and 12b, show that it is unlikely that the results reach a unique Normal Compression Line (NCL). Unfortunately the testing arrangements used limited the maximum stress to about 1800 kPa. However a reasonable interpretation of the data would be that the compression curves remain parallel with little tendency to converge. The same conclusion can be made for both the upper silt and lower clayey silt.

These data are in contrast to tests on silty clay from the nearby E39 Jektevik – Sandvikvåg site, see Figure 12c. This material has the same average water content as the Os silt ($\cong 33\%$)

but has average clay content of some 38% and sand content of 5%. Here the results show a reasonably clear convergence on a unique NCL, albeit at very high stresses.

Similar behaviour to that of the Os silt has been found elsewhere. Nocilla et al. (2006) report results for an Italian silt, from the Po River, where there is no evidence of convergence of the compression curves even at high stresses, of the order of 14MPa, and low void ratios. The non-uniqueness of the NCL has also been found for a range of other soils, which have been called “transitional soils” by Nocilla et al. (2006). These soils consistently have an intermediate grading between sand and clay. Examples include a gap-graded residual sandstone from Brazil (Martins et al., 2001; Ferreira & Bica, 2006).

Perhaps the most well studied silt materials in world are those from the area of the Venice Lagoon. Biscontin et al. (2007) and Cola & Simonini (2002) have shown that these silts also do not reach a unique NCL, but their compression curves remain parallel with little tendency to converge. Here this behaviour is likely to have been due to the highly heterogeneous and complex nature of the material, rather than it being a “transitional soil”

10.2 Published frameworks for soft soils

It is also possible to compare the behaviour of the soils within other published frameworks. For example Mitchell and Soga (2005) proposed the relationships between sensitivity, liquidity index (I_L) and vertical effective stress for normally consolidated to moderately overconsolidated clays as shown on Figure 14. Here the S_t values were based on fall cone tests. Data for the Os silt are superimposed on the Mitchell and Soga (2005) plot. It can be seen that the data are inconsistent with the suggested relationship, which would predict that the material should have a much lower S_t than actually measured. The average value of I_L of 0.93 seems rather low for so high sensitivity. According to Skempton & Northey (1952), it would be expected that, at depths less than 10m, very high sensitivities occur in soils of liquidity index well above unity.

In contrast data for four other sites, in the same area of Norway close to Bergen, are in general consistent with the framework proposed by Mitchell and Soga (2005). These materials are from:

- E39 Jektevik – Sandvikvåg; $w = 33\%$, clay content = 38%, sand content = 5%
- Jondalstunnel; $w = 29\%$, clay = 14%, sand = 26%
- Statoil Forus Vest; $w = 32\%$, clay = 10%, sand = 18%
- E18 Foruskrysset $w = 30\%$, clay = 18%, sand = 8%

The most significant difference between the Os silt and these materials is the greater clay content.

11. Summary and conclusions

The main objective of this work was to provide guidance for geotechnical engineers designing civil engineering works in silty soils. It was achieved by detailed characterisation of a silt site at Os in western Norway. Some findings are as follows:

1. CPTU can be very useful for classification of silty soils. CPTU data gave reliable estimates of effective friction angle, undrained shear strength and constrained modulus, M . However care needs to be taken with sleeve friction, f_s , as values recorded can be below equipment accuracy.
2. Field vane and index tests gave inconsistent remoulded shear strength and hence sensitivity values. It is possible that 25 rotations of the field vane in silty soils are insufficient to completely remould the material.
3. At this site the sample quality was often classified as “poor” according to classification systems derived for clays. Unfortunately no comparative high quality samples (e.g. blocks) were available. Nevertheless for future work it is recommended that larger sample tubes (say 75 mm) with a very sharp cutting edge should be used. Also in order to address the issue of reducing sample quality with depth consideration should be given

to pre-drilling a borehole to just above the sampling location and to supporting the borehole with drilling mud.

4. Triaxial test results for the upper Os silt (0% clay) gave consistent dilative response. However response for the lower clayey Os silt (up to 15% clay) was inconsistent being sometimes contractive and sometimes dilative independent of sample type or clay content but could generally be anticipated from the initial void ratio. Care needs to be taken when choosing design parameters based on laboratory tests on silty soils.
5. Interpretation of triaxial tests for undrained shear strength (s_u) using the traditional approach can give unrealistically high values due to the dilatant nature of the material. A more reliable and logical approach is to take s_u at a limiting strain of about 2% or at peak pore pressure.
6. The well-known Janbu model for 1D consolidation and creep can be used successfully to characterise the behaviour of both the upper silt and lower clayey silt. Modulus number m can be found reliably by correlation with water content.
7. The behaviour of the Os silts does not fit easily into classical soil mechanics and published frameworks for soft soils. It seems the material is of “transitional” type and this work adds to the database of such soils which includes other natural silts and gap graded soils. Further research is required to determine how widespread the patterns of behaviour seen here are and to identify a new framework for such soils.

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Table 1.

Summary of dimensions and features of samplers.

Sampler	L (mm)	D _w (mm)	t (mm)	C _a (%)	D _w /t	C _i (%)	α (deg.)
54 mm composite	800	65	5.5	44	12	0.6	5
54 mm steel	800	56.5	1.25	9.5	45	0.9	14.3*

L = length, D_w = outside diameter, D_e = inside diameter of cutting shoe, D_s = inside diameter of sampling tube, t = wall thickness, C_a = area ratio = $(D_w^2 - D_e^2)/D_e^2$, C_i = inside clearance ratio clearance = $(D_s - D_e)/D_e$, α = cutting edge angle.

* For 54 mm steel sampler α varied between 13° and 16° with an average of about 14.3°

Table 2.

Summary of fundamental material properties

Parameter	Symbol	Units	Value
Water content	w	%	33.2
Bulk density	ρ	Mg/m ³	2
Sensitivity	S _t		
above 6m			70
below 6 m			300
Clay content			
above 6 m		%	3
below 6 m		%	12
Liquid limit	w _L	%	34.3
Plasticity index	I _p	%	12.6
Specific gravity	G _s		2.94
Organic content		%	<2
Salt content of pore fluid		g/l	1

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LIST OF FIGURES

Fig. No.	Caption	File Name*
1	Site location (a) in Norway (b) near Bergen (c) detailed test locations	DELL/papers/OsSilt/Norge.jpg DELL/labtests/FoUOs/TestLocations.jpg DELL/labtests/FoUOs/TestLocations.grf
2	Results of CPTU and T-bar tests	DELL/labtests/FoUOs/CPTUTBaru2Bq.grf
3	Basic material properties	DELL/labtests/FoUOs/Phase2/Basics.grf
4	Particle size distribution	DELL/labtests/FoUOs/Phase2/psdcurvesandwidthdepth.grf
5	Quality of triaxial and oedometer samples using NGI criteria	DELL/labtests/FoUOs/Phase2/Triaxandoedomquality.grf
6	Shear wave velocity and suction measurements	DELL/labtests/FoUOs/Phase2/Vsur.grf
7	Sample quality from shear wave velocity and suction measurements	DELL/labtests/FoUOs/Phase2/QualfromVsur.grf
8	Triaxial tests results (a) upper silt, (b) lower clayey silt 7 m and 9 m samples and (c) lower clayey silt 8 m and 10 m samples	DELL/labtests/FoUOs/Phase2/alltriaxuppersilt.grf Alltriaxlowerclayeysilt.grf Alltriaxlowerclayeysilt2.grf
9	Pore pressure response in triaxial tests – 54mm composite samples	DELL/labtests/FoUOs/Phase2/triaxppw.grf
10	Undrained shear strength s_u from (a) index tests, (b) field vane and (c) CAUC triaxial tests (simple peak)	DELL/labtests/FoUOs/Phase2/su.grf
11	Undrained shear strength - s_u from CAUC triaxial tests using different criteria and from CPTU tests	DELL/labtests/FoUOs/Phase2/sudiffapproachPh2+22040013.xls
12	Typical oedometer test results (a) upper Os silt, (b) lower Os clayey clayey silt and (c) E39 silt	DELL/labtests/FoUOs/Phase2/Revisedoedsuppersilt+revisedoedslowerclaysilt+e39oeds.grf
13	(a) Oedometer and CPTU M_0 values and (b) Relationship between m and water content	DELL/labtests/FoUOs/Phase2/mandM0revised.grf
14	Behaviour of silts in Bergen area in relation to framework of Mitchell and Soga (2005)	DELL/labtests/FoUOs/Phase2/MandSframewor.grf

Fig. 1. Site location (a) In Norway (b) Bergen area (c) Detailed test locations

Fig. 2. Results of CPTU and T-bar tests

Fig. 3. Basic material properties

Fig. 4. Particle size distribution

Fig. 5. Quality of triaxial and oedometer samples using NGI criteria

Fig 6. Shear wave velocity and suction measurements

Fig 7. Sample quality from shear wave velocity and suction measurements

Fig 8. Triaxial test results (a) upper silt, (b) lower clayey silt 7 m and 9 m samples and (c) lower clayey silt 8 m and 10 m samples

Fig 9. Pore pressure response in triaxial tests – 54 mm composite samples only

Fig 10. Undrained shear strength - s_u from (a) index tests, (b) field vane and (c) CAUC triaxial tests (simple peak)

Fig. 11. Undrained shear strength - s_u from CAUC triaxial tests using different criteria and from CPTU tests

Fig. 12. Typical oedometer test results (a) upper Os silt, (b) lower Os clayey silt and (c) E39 Jektevik silt

Fig. 13. (a) Oedometer and CPTU M_0 values and (b) Relationship between m and water content

Fig. 14. Behaviour of silts in Bergen area in relation to framework of Soga and Mitchell (2005)