Statens vegvesen

FOU- prosjekt Parameterbestemmelser for siltige materi-aler. Delrapport C). Samlerapport

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Delrapport C). Samlerapport

Sammendrag

The results from the site investigation and from the laboratory analyses are analysed and discussed in this report. Conclusions from the results are made and some recommendations for further work are also made.
The results confirm that the site is underlain by a relatively uniform silty material, which makes it a very good research site. From the field work analysis of the data confirms the usefulness of the CPTU in material characterisation and determination of soil parameters in silty material. There is an inconsistency between the field vane and fall cone remoulded strength values. This warrants more research. As regards the laboratory testing a particular feature of the soils behaviour was that, although the behaviour of the material was like silt or sand, the resulting parameters were closer to that of clay. It is difficult to interpret s_u in silty material and some recommendations, based on a limiting strain criteria, were developed in this work. Not unexpectedly the 54 mm plastic specimens were found to be the worst of those recovered. Surprisingly the quality of the 76 mm steel and 54 mm steel were very similar. The reason for this is not completely clear but is likely to be due to some small details in the sampling procedure, which warrants further work. A special feature of the material behaviour is that is much more susceptible to disturbance during specimen preparation for testing than clay material and this also requires future attention. There seems to be some promise in the use of shear wave velocity measurements to quantify sample quality.

Note: *This report is to be read in conjunction with the laboratory testing report of February 2005. (Statens Vegvesen, Region Vest, Teknologiavdelingen, Internal Rapport, Nr. 2382, dated 05.02.2005). Figures given in the laboratory testing report are not repeated here.*

Emneord:	Basic parameters, CPTU, Index tests, Oedometer tests, Triaxial tests, shear wave velocity measurements

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TABLE OF CONTEST.

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1. INTRODUCTION.

The Norwegian Public Road Administration, Technological Department Region West, have preformed geotechnical site investigations (in situ measurements and sampling) and geotechnical recommendations for a period of 30 years. In recent years, for new road projects, for example soft silty soils were discovered in some areas of the region while sampling on new roads project. For examples were soft silty soils found at E39 Stigedalen in Nordfjordeid, Stedje in Sogn and Fjordane and several of areas in Os county.

Only a limited amount of research papers and knowledge on which site investigations methods and sampling procedures gives the "best" reliable parameters to use in a geotechnical engineering recommendations is available in Norway. This is due to the fact that most geotechnical research in Norway for the last decades have been concentrated on clay and quick clay. The Technological Department experience with performing sampling in a soft silty soils is that it is difficult to get undisturbed samples to use in a geotechnical engineering recommendation which involves evaluation of the material strength and deformation parameters. This is due to the fact that the soft silty soil found in the region is often very sensitive and wet, which leads to loss of sample material and sample disturbances.

To increase the knowledge about the material behaviour of silt the Technological Department has preformed in situ measurements, sampling and laboratory tests with material from Skeisleira in Os county. The project is financed with FoU- foundings from Research Department at the Norwegian Public Road Administration and the project is done in a co-operation with University College Dublin (UCD), The Norwegian University of Science and Technology (NTNU) in Trondheim and Vegdirektoratet.

In this report the results from the site investigation and the laboratory are analysed and discussed. This report is to be read in conjunction with the Internal Rapport, Nr. 2382 of 5. February 2005 (laboratory testing report). *Figures given in the laboratory testing report are not repeated in this report.* The site investigation preformed in this project involves total sounding method, field vane, cone penetration with pore pressure measurements, T-bar measurements and sampling with several different sample tubes, Statens Vegvesen (2005a). The laboratory work preformed in this project involves routine laboratory tests, maintained load (ML) oedometer tests, constant rate of strain (CRS) oedometer tests, anisotropically consolidated undrained triaxial tests (CAUC) and shear wave velocity measurement by bender elements, Statens Vegvesen (2005b).

The laboratory work described in this project was carried out mostly at the University College Dublin and at The Norwegian Public Road Administration, Technological Department Region vest (Bergen) in the period October 2004 to July 2005.

2. BASIC PARAMETERS.

2. 1. Bulk density and water content

These data are presented in Figs. 1 and 2 of the lab test report. On Fig. 1, data from basic index testing obtained immediately after sample extrusion are given. In general the values are consistent with depth and show only a slight scatter. Average values of water content and bulk density are shown on the figures and are 32% and 2 Mg/m^3 respectively. There is some evidence of slightly higher water content values and corresponding lower bulk density values at about 4.5 m.

Data on Fig. 2 were obtained from oedometer and triaxial test at UCD. It can be seen that there is more significant scatter than is evident on Fig. 1, though the average values are about the same. This increased scatter probably reflects additional disturbance on transportation of the sub-samples to Dublin and on building the specimens into the triaxial and oedometer equipment. Every possible attempt was made to avoid any additional disturbance for example by using a thin piano wire to trim the samples.

2. 2. Sampling induced densification

In silty material it is possible that partial drainage during sample tube insertion will densify the material and cause a corresponding reduction in water content. Any such densification will vary with the details of the samplers. These are summarised on Table A.

Characteristic	ϕ = 54 mm, composite	ϕ = 54 mm, steel	ϕ = 76 mm, steel
Inner diameter (mm)	*54.5*	*54.5*	*75.6*
Outer diameter (mm)	*65*	*57*	*80*
Sampling length (mm)	*800*	*800*	*800*
Cutting edge angle (deg.)	*5*	*10*	*9*
Area ratio (%)	*44*	*9 - 11*	*11.4*
Inside clearance ratio (%)	*0.6*	*0 - 0.9*	*0*
Outside clearance ratio (%)	*0*	*0*	*0*

Table A. Summary of sampler characteristics

Another importance difference to be considered is that both sets of 54 mm samples were taken using the full displacement technique without the use of water. With the 76 mm samples water was used to aid drilling and before extracting the samples water was injected via one tube on the sampler perimeter to minimise the suction at the base of the sample.

Inspection of Table A would suggest that, given its characteristics, the 54 mm composite sampler is likely to cause most densification. This possibility is investigated on Figure A. It can be seen that the average values for the three different sample types and from the different measuring techniques are more or less identical. There is some evidence that the 54 mm steel samples are less dense and have higher water content. However insufficient evidence exists to confirm whether this is due to sampling effects or simple natural material variability.

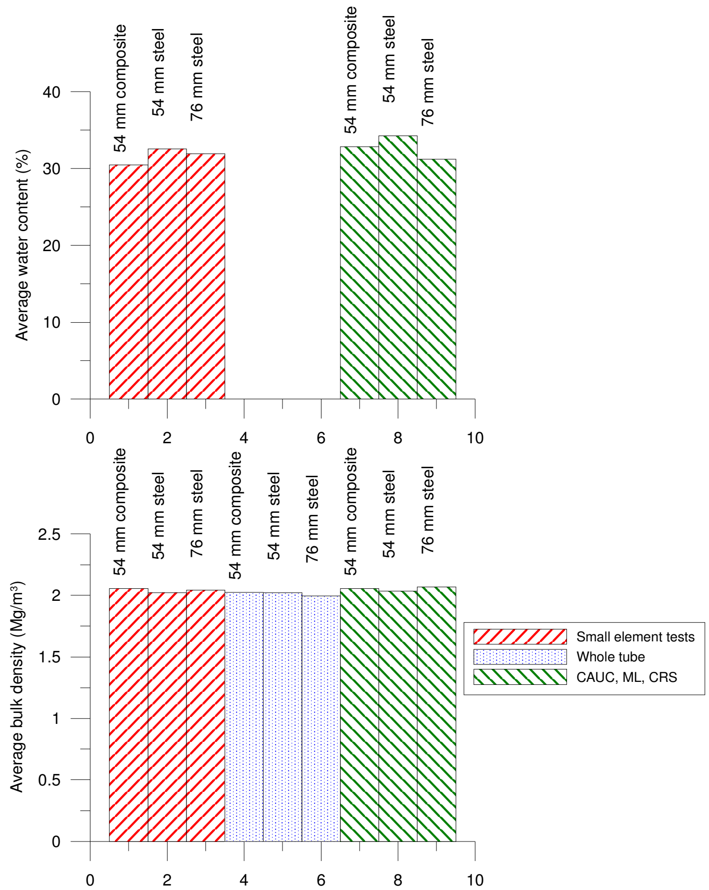

Figure A. Sampling induced densification

2. 3. Particle density

Average value of particle density (Fig. 3 of lab test report) is 2.94 Mg/m^3 and there is little scatter in the available data. This value is unusually high and may reflect the presence of heavy metals in the form of oxides in the material (Head, 1980). However the accuracy of the test can be checked by using the value, together with measured bulk density and water content, to determine the degree of saturation. This has been done for the triaxial and oedometer tests (see Tables 1, 2 and 3 of the lab test report) and in general a degree of saturation close to 100% is obtained, confirming the measurement is correct. These high value warrant further investigation by X-ray diffraction, scanning electron microscope or similar.

2. 4. Salt content

Salt content of the pore water in the material (Fig. 4 of the lab test report) is very low. This confirms that the material has been leached by fresh water flow subsequent to deposition in a shallow marine environment (Bondevik and Mangerud, 2002)

2. 5. Organic content

Organic content was measured using the loss on ignition at 440°C test at both Statens Vegvesen laboratories in Bergen and at UCD (see Fig. 5 of lab test report). Similar values, all generally less than 2%, were obtained. Note that this not necessarily mean that 2% of the material was organic as this test is more a reflection of the total carbon content of the material. There was only occasional visible organic zones, e.g. test ML54(S)-2 at 7.4 m.

2. 6. Particle size distribution

Particle size distribution curves are shown on Fig. 6 of the lab test report and the distribution of the percentage clay and silt with depth is shown on Fig. 7. Average clay and silt contents are 16% and 77% respectively.

According to NGF (1982), if more than 15% of the material (by mass) is of clay size (< 2 μm) then the main term used to describe it should be "CLAY", As can be seen from Figs. 6 and 7 of the lab test report 12 of the 19 tests have clay content greater than or equal to 15%. Clearly the plot shows the material is borderline between silty CLAY and clayey SILT.

The British Standard, BS5930 (1999) contains no such definitive guidance but says that such borderline material should be termed CALY / SILT unless other supporting laboratory tests (such as plasticity tests, see next section) are available. BS5930 (1999) suggests that the description of the material should be consistent with its engineering behaviour.

2. 7. Plasticity

Plasticity data, shown on Fig. 8 of lab test report, shows the material has average liquid limit (w_L) and plasticity index (I_p) of 34.3% and 12.6% respectively. The standard plasticity chart shows the material to fall on or just above the A-line. According to both NGF (1982) and to BS5930 (1999), based on these data, the material should be described as CLAY of low to intermediate plasticity.

Also shown for comparison on the plasticity chart is data from the well known NGI research site at Onsøy near Fredrikstad in Southern Norway (Lunne et al., 2003). The Onsøy clay has w_L =65%, I_p = 40% and is described as clay of high plasticity

Average liquidity index (LI) for the Os soils is about 0.8. Values greater than 1.0 would normally indicate a sensitive material with an open structure.

3. 0. MATERIAL CLASSIFICATION FROM CPTU DATA

It is also possible to classify the material from the CPTU data. As has been detailed in the field testing report (Statens Vegvesen, 2005a) separate CPTU tests were carried out using equipment from Statens Vegvesen Hordaland and Rogaland. Conventional q_t, R_f and u_2 data are shown on Figure B. Measured cone resistance (q_c) has been corrected for the out of balance pore pressure effects to give q_t. However no such correction has been applied to R_f in this case as the pore pressure above the friction sleeve (u_3) is not known.

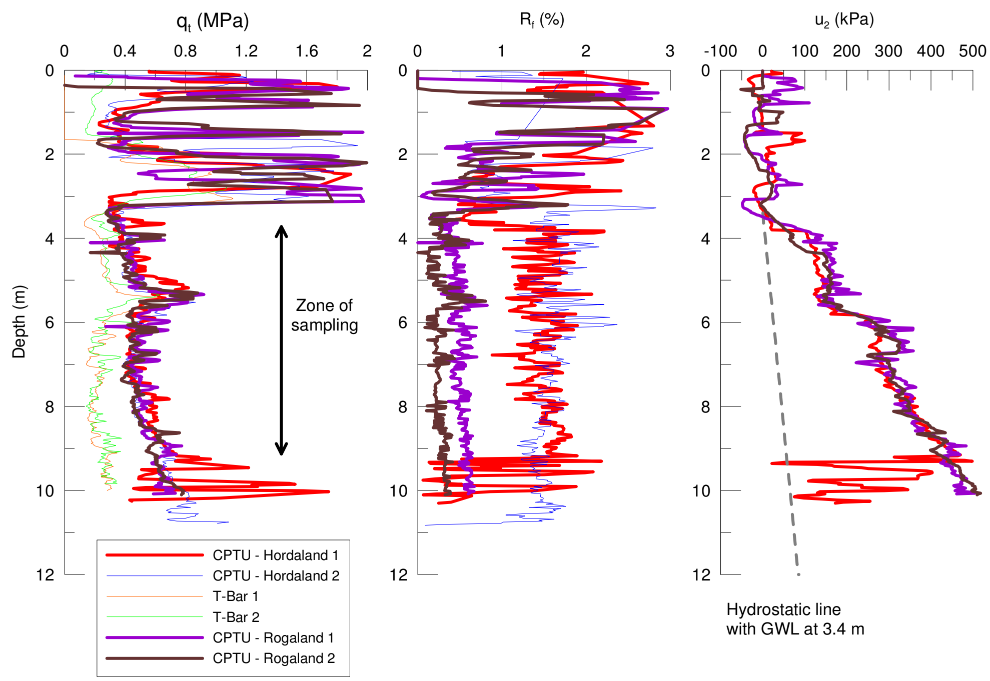

Figure B. CPTU q_t, R_f and u_2

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 suggesting the material is behaving in a clay like manner, with little dissipation of excess pore pressure occurring during penetration. For q_t and u_2 the Hordaland and Rogaland equipment give very similar results.

However there is a significant difference between the sleeve friction (f_s) and hence the friction ratio (R_f) data for the two sets of equipment. The Hordaland equipment gives R_f of about 1.5% in the sampling zone whereas the Rogaland equipment gives values less than 0.5%. During the fieldwork it was thought the Rogaland friction sleeve may have suffered some damage. Attempts were made to correct the f_s data for pore pressure effects according to Lunne et al. (1997a) and Sandven (1990) but these did not make any significant difference to the results. For the purposes of this report the only the Hordaland f_s data will be considered.

3. 1. Standard classification chart

In the standard chart of Robertson et al. (1986) (also reproduced in Lunne et al., 1997a) use is made of q_t, R_f and the pore pressure parameter B_q for the purposes of soil classification, see Figure C. This plot suggests that the soil can be sub-divided into two zones:

- above 6 m where $B_q \approx 0.2$ and
- below 6 m where $B_q \approx 0.6$.

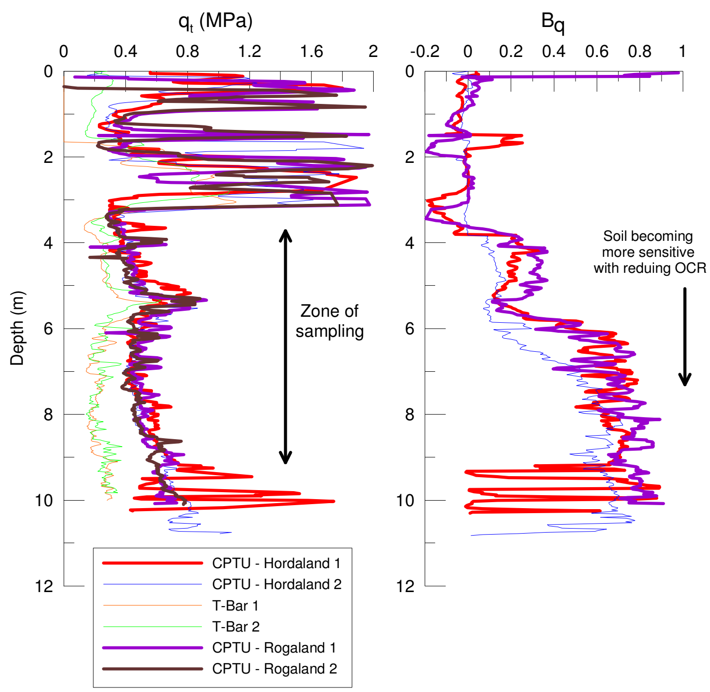

Figure C. CPTU q_t and B_q

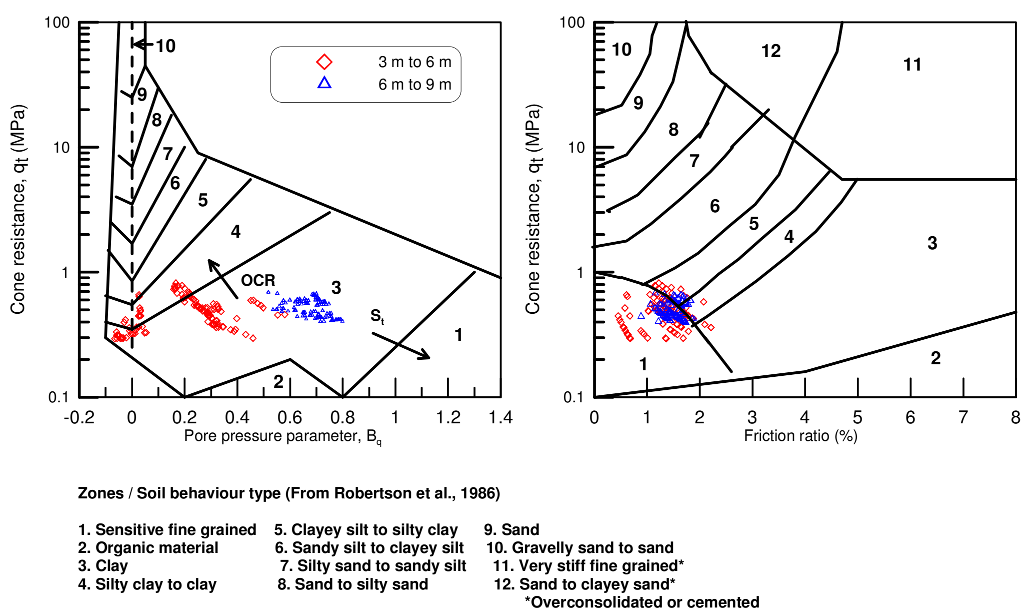

Figure D. Robertson et al. (1986) classification chart

The data are subsequently sub-divided into two zones and plotted on the Robertson et al. (1986) chart on Figure D. According to the chart the soil would be classified as “clay” or “silty clay to clay” above 6 m and “clay” blow 6 m. OCR decreases and sensitivity increases with depth. The R_f data are less useful than the B_q data for the purposes of distinguishing between the two layers.

This chart illustrates the importance of the pore pressure measurement in classifying silty soils. The discontinuity in the B_q profiles coincides with the zone of high q_t. In addition 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”.

3. 2. Other classification charts

Attempts have also been made to see if any other classification schemes would be useful in distinguishing between the two layers. For example normalising the data to allow for overburden effects (Q_t, F_r and B_q) as suggested by Robertson (1990), using the cone resistance number (N_m, Sandven, 1990) or using a plot of Q_t(1-Bq) against F_r as suggested by Jeffries and Davies (1991) was not helpful.

The data seems to fall outside the typical limits suggested by Senneset and Janbu (1985) as shown on Figure E.

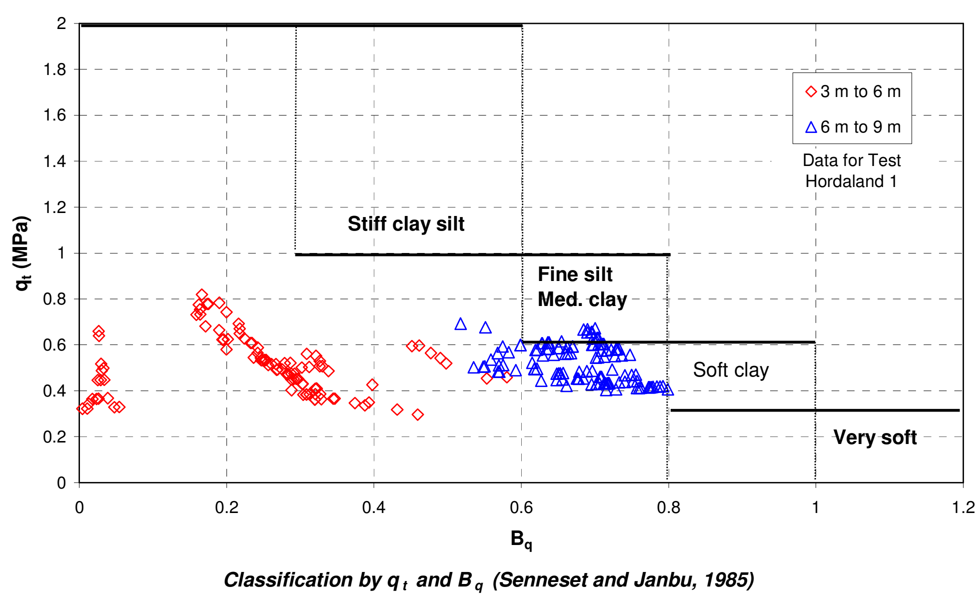

Figure E. Classification chart by Senneset and Janbu (1985)

Some researchers have suggested that these charts (e.g. q_t and B_q) involve plotting parameters which are a function of q_t on both axes and that instead two separate parameters should be plotted on each axis. Such a chart was proposed by Fellenius and Eslami (2000). Data from this site are plotted in this form on Figure F. Although it shows a clear distinction between the data above and below 6 m it is not particularly helpful in classifying the material.

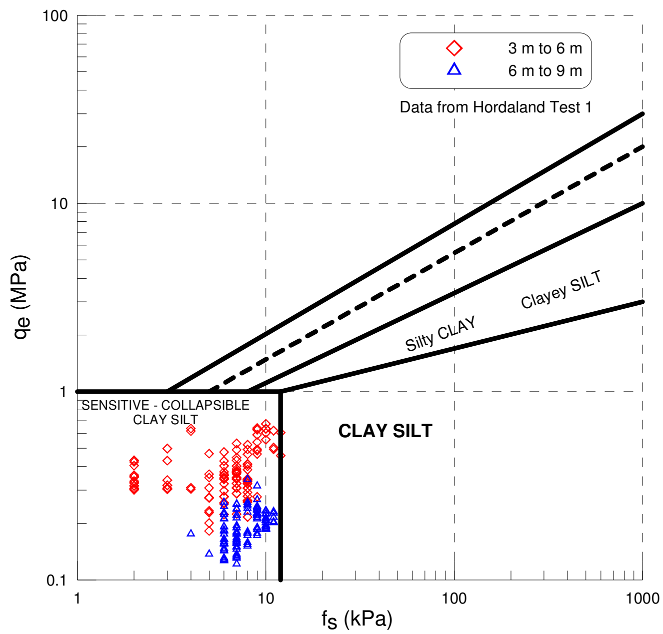

Figure F. Classification chart by Fellenius and Eslami (2000)

<u>Conclusion</u>

It is tentatively suggested that the soils in the sampling zone can be sub-divided into two layers, i.e. a more silty soil above about 6 m and a more clayey soil below 6 m. This is consistent with the plasticity data (Fig. 8 lab test report) where it can be seen that the sample above 6 m has the lowest value of I_p. The pore pressure parameter B_q is very useful for the purposes of soil classification.

4. 0. UNDRAINED SHEAR STRENGTH AND SENSITIVITY FROM INDEX TESTS

4. 1. Intact strength

Undrained shear strength values (s_u) from index tests are shown on Fig. 9 of the lab test report. 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 total sounding pushing force and CPTU q_t values in this zone, see Figures B and C. According to BS5930 (1999) the consistency of the material should be described generally as "soft". There is no significant difference between the results from the two types of test.

In general these values are in excess of the typical $0.3\sigma'_{v0}$ quoted for normally consolidated clays (Hight et al., 1987), suggesting that at least above 6 m to 7 m depth, that the material is lightly overconsolidated.

Results of in situ field vane tests are also shown on Fig. 9 of the lab test report. Tests were carried out on two occasions on the site and in the 2004 investigation two techniques (hand and mechanical torque) were used. 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. The field vane tests also confirm that below 6 m to 7 m the material is normally consolidated.

4. 2. Remoulded shear strength

Values of remoulded shear strength (s_{ur}) from index and field vane tests are shown on Fig. 10 of the lab test report. From the index tests it can be seen that the remoulded strength decreases below about 6 m. This finding is consistent with the interpretation of the CPTU discussed above that the soil in the sampling zone can be sub-divided into two layers, with that below 6 m being more clayey and more sensitive.

There is a significant difference between s_{ur} from the two sets of tests. Index tests s_{ur} values are very low, particularly below 6 m depth, being typically 0.1 kPa to 0.2 kPa. On the other hand field vane tests show values close to 1 kPa. Inspection of and experience with handling the material (see Figure G) suggest that it is more quick than suggested by the field vane. The reason for the difference is not clear but is likely to be due to a combination of influence factors such as:

- shearing mode different,
- possible greater degree of remoulding in the laboratory index tests compared to that in the field with the field vane,
- overburden stress in the field tests.

Figure G. Observed sensitivity of material

The field vane s_{ur} values found here are similar to those from the Statens Vegvesen Vinnesparsellen site, which is also underlain by silty material (Unpublished information from G. Gudjonsson).

However the opposite effect is found for some other soils, e.g. Onsøy clay where the field vane gave lower s_{ur} values than the index tests (Lunne et al., 2003).

Other researchers (summarised in Lunne et al., 1997a) have suggested that the sleeve friction, f_s, provides a direct measurement of s_{ur}. Data for this site are shown on Figure H (Hordaland rig data only). The values of f_s are significantly higher than s_{ur}. However the likely existence of two layers within the sampling zone (above and below 6 m) is apparent from the figure.

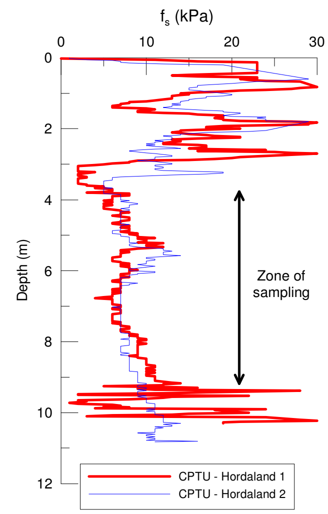

Figure H. CPTU fs

4. 3. Sensitivity

Sensitivity (S_t) values from both the index tests and the field vane are shown on Fig. 10 of the lab test report. Consistent with the both the measured remoulded shear strength values and the CPTU data as discussed above, the index tests suggests much higher sensitivity, with values being of the order of 70 above 6 m and up to 350 below 6 m. According to NGF (1982) a material can be classified as quick" if $S_t \geq 30$ and $s_{ur} \leq 0.5$ kPa. This 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". Observations of the behaviour of the material in the lab, see Figure G, are also indicative of high sensitivity values and suggests that the field vane values of s_{ur} are unrealistically high.

5. 0. OEDOMETER TEST RESULTS

Detailed test results for maintained load oedometer tests (ML) are given on Figures 12 to 26 of the lab test report and are summarised on Table 1 of that report. Similarly results of the CRS tests are given on Figures 27 to 45 and are summarised on Table 2.

5. 1. Sample quality assessment

Prior to any discussion on the test results and the derived parameters it is necessary first to make an assessment of the sample quality. Normally this is done by studying the normalised volume change ($\Delta V/V_0$) or the normalised void ratio change ($\Delta e/e_0$) recorded on loading the sample back to in situ vertical effective stress (Andresen and Kolstad, 1979 and Lunne et al., 1997). These techniques were developed for marine clays with OCR in the range 1 to 4 and it is not clear whether they are applicable to the silty soils under study here. However they will be used in the absence of any other simple technique and these data are shown on Figure I.

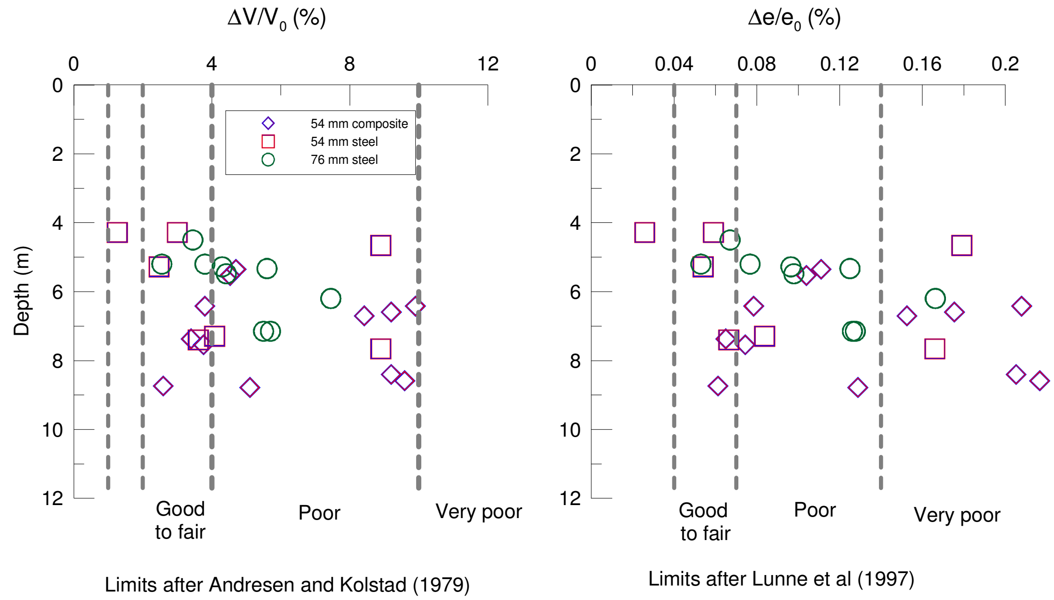

Figure I. Oedometer tests – sample quality assessment

According to the $\Delta V/V_0$ assessment approach approximately 25% of the samples can be categorised as "good to fair" but the majority fall in the "poor" category. The alternative $\Delta e/e_0$ criterion seems to be stricter with few of the samples in the "good to fair" criterion and most in the "poor" or "very poor" categories. On average the 54 mm steel samples are best with $\Delta V/V_0$ and $\Delta e/e_0$ equal to 4.6% and 0.091 respectively, followed closely by the 76 mm steel samples (4.8%, 0.104) and the 54 mm composite samples are the worst (6.2%, 0.132).

As was pointed out above, the silty nature of the material meant that during building in of the specimens a thin film of water and water "charged" material developed at the surface. Thus high strains were often required to bring the samples back to in situ vertical effective stress. Therefore even if high strains were recorded up to this stage, and the specimen was classified as "poor", it is not certain that the subsequent behaviour (and hence the derived parameters m etc.) would reflect this characterisation. Similarly if poor sampling densified the material it could have low $\Delta V/V_0$ and $\Delta e/e_0$ values and subsequently be classified as a good specimen. In conclusion it is felt that the techniques used to quantify sample quality here may be inappropriate and may not reflect the true specimen quality.

On Figure I it is clear that the data fall into two separate groups, those with $\Delta V/V_0$ less than and greater than about 6%. The latter group consist of those specimens which exhibited this "charged" dilatant behaviour. In an assessment of sample quality than it may be more appropriate not to consider those specimens. A similar conclusion would then be made with the quality of the 54 mm steel and 76 mm steel samples similar and the 54 mm composite worst.

5. 2. Stress / strain behaviour

A summary of test results for depth interval 4 m to 4.8 m, 5 m to 5.8 m, 6 m to 6.8 m, 7 m to 7.8 m and 8 m to 8.8 m are given on Figures 1 to 5 of this report. In general the colours red, blue and black are used to represent the 54 mm plastic, 54 mm steel and 76 mm samples respectively. Data are presented on stress – strain (σ - ε), logσ - ε, and σ - M (constrained modulus) form.

The following trends can be observed in the plots:

- The σ - ε or logσ - ε plots are of rounded nature and there is little evidence of a clear yield point (or preconsolidation pressure, p'_c).
- These curves are more indicative of the behaviour of sand or silt rather than clay as can be seen with reference to Figure J from Janbu (1985).
- There is no clear difference between the results from the various sample types.
- Occasionally very significant strain occurs at low stress, e.g. Test CRS 19 (Fig. 1), many of the tests on Fig 3, Test CRS 18, Fig. 4 etc. This high strain occurs due to a compressible "water charged" layer which develops at the top of the specimen during preparation.
- A number of the specimens in the 7 m to 7.8 m depth interval (Fig. 4) show some evidence of a p'_c value.

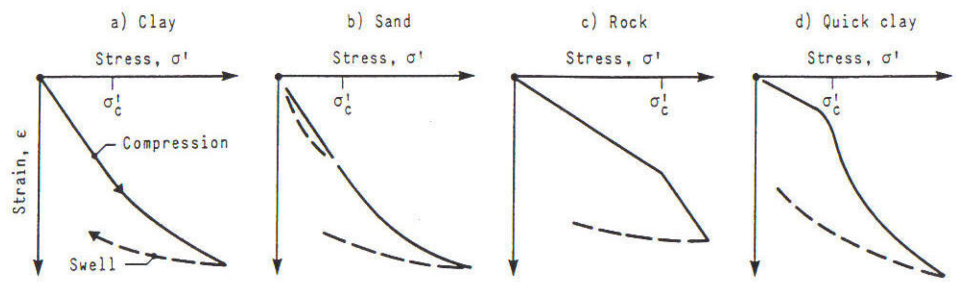

Figure J. Expected 1 D σ - ε behaviour (Janbu, 1985)

5. 3. Stress / constrained modulus relationship

All of the test results show a gradual increase in M with increasing stress. Again this is more indicative of sand or silt behaviour rather than that of clay, see Figure K. There is no evidence (except perhaps for the 7 m to 7.8 m tests on Fig. 4) of constant M at low stress or a rapid reduction in M as p'_c is approached, as would be expected for sensitive clay.

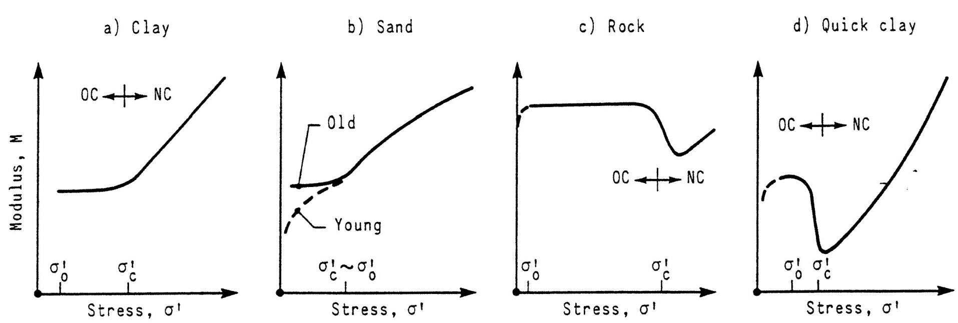

Figure K. Expected 1 D σ - M relationships (Janbu, 1985)

5. 4. Preconsolidation pressure p'_c and OCR

Although through some subjective guesswork it is possible to estimate p'_c from a good number of the tests it is felt that such an exercise is not worthwhile given the uncertain nature of the results. It is possible only to determine p'_c and OCR relatively reliably from two tests, namely ML54(P)-3 and ML54(S)-3. Both these samples are from the 7m to 7.8 m depth where the behaviour of the material is closer to clay than that above 6 m. The Casagrande construction technique suggests the samples have p'_c of 175 kPa and 113 kPa with a corresponding OCR of 2.25 and 1.48. Given the rounded nature of the cures, these values should be treated with caution.

It is also possible to estimate OCR from the undrained shear strength data. Following a comprehensive literature review Chandler (1987) concluded that on average:

$$\frac{s_{u-vane}}{\sigma`_v} = 0.25(OCR)^{0.95} \qquad (1)$$

With reference to Fig 9 of the lab test report the following can be determined (Table B):

Depth range (m)	s_u (kPa)	s_u / σ'_{v0}	OCR
3.5 – 4.5	*40*	*0.8*	*3.4*
4.5 – 5.5	*30*	*0.6*	*2.5*
5.5 – 6.5	*25*	*0.45*	*1.9*
6.5 – 7.5	*30*	*0.35*	*1.4*
7.5 – 11.5	*18*	*0.25*	*1.0*

Table B. OCR as derived from field vane (Chandler, 1987)

These data suggest that above about 7.5 m the material has been lightly overconsolidated. The reason for this is likely to be due to a combination of factors, e.g. excavation for a nearby road construction in 2002, a fluctuating water table and due to possible glacier re-advance (Bondevik, and Mangerud, 2002).

This finding is also consistent with the CPTU data. Sandven (1990) and Lunne et al. (1997a) suggested that if the q_t profile exceeds that of a normally consolidated clay (corresponding to 2.5 to $5\sigma'_{v0}$) then the material can be considered lightly overconsolidated (see Figures B and C).

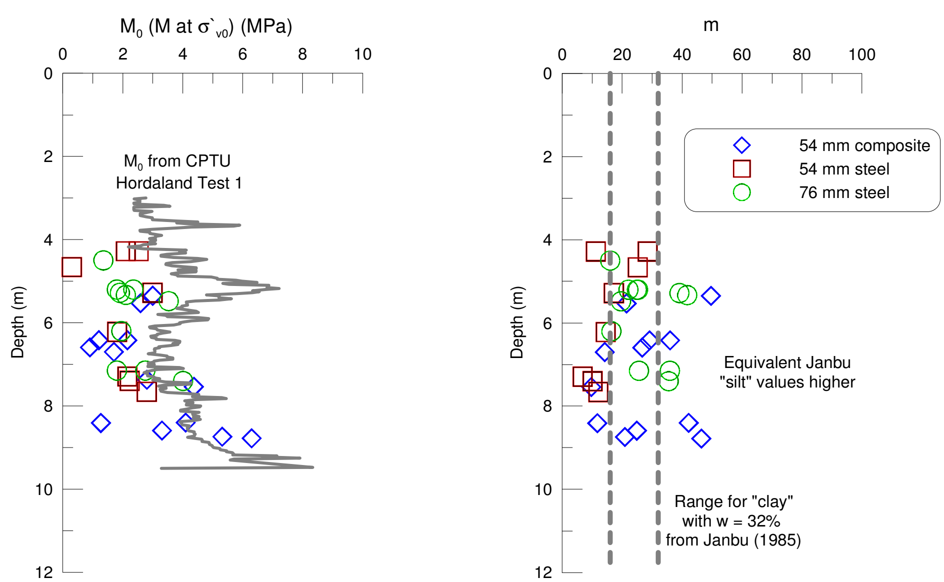

Figure L. M_0 and m

5. 5. M_0 (i.e. M at σ'_{v0})

Values of M_0 (i.e. the constrained modulus at in situ effective stress) are plotted on Figure L. There is no clear difference between the results from the different sampler types with M_0 being on average 2 MPa to about 7.5 m depth. Below 7.5 m (where most of the tests were on 54 mm composite samples), M_0 appear higher and to be about 4 MPa on average.

M_0 can also be determined form CPTU data from the equation (Lunne et al., 1997a)

$$M_0 = \alpha_i q_{net} \qquad (2)$$

where α_i is in the range 5 to 15 for most clays. On the profile shown on Figure L, α_i has been assumed to equal 10. It can be seen that the agreement between the lab and field data is reasonable.

5. 6. Modulus number (m)

Modulus number (m) values, also plotted on Figure L, are lowest for the 54 mm steel samples (average = 15.7) and about equal for the 54 mm plastic and 76 mm steel samples (average = 27.7 and 28). The reason for the 54 mm steel samples showing lower values is not clear. In any event all of the values are generally within the range suggested by Janbu (1985) for "clay" with average water content = 32%. The equivalent "silt" value for a material of water content 32% would be about 50 according to Janbu (1985).

Sandven (2003) reports m value in the range 30 to 50 for the well research Halsen silt from Sjørdal near Trondheim. At this site average water content is about 20%.

Stjordal (2005) reports that oedometer tests in silty material are best interpreted with the use of the Janbu (1985 and others) parameter a =0.25.

5. 6. Coefficient of consolidation (c_v)

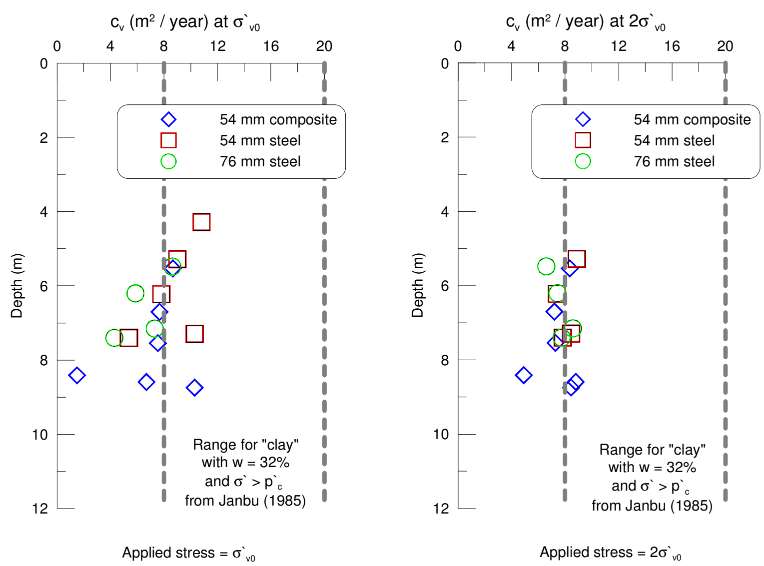

Figure M. c_v

Coefficient of consolidation (c_v) values for each load increment of the maintained load oedometer tests are plotted on the individual test result sheets (Figs 12 to 26 of lab test report). These value were calculated by the conventional root time interpretation according to Taylor (1942). Values for the load increments corresponding to σ'_{v0} and $2\sigma'_{v0}$ are plotted on Figure J. It can be seen that:

- there is no difference between the results from the different sampler types,
- values at σ'_{v0} and $2\sigma'_{v0}$ are more or less the same,
- the average value is about 8 m^2 / year,
- this corresponds to the lower bound for Scandinavian "clays" with water content of about 32% according to Janbu (1985),
- according to Janbu (1985) values for silts should be higher than this,
- there is no evidence of significant preconsolidation in the data.

5. 7. Creep

Values of creep resistance, r_s, against applied stress are plotted on Figure N. These values were determined from the ML oedometer tests for load increments where the load had to be maintained either overnight or over a weekend. There is no clear relationship between r_s and stress. Instead the points seem to group around two values of about 150 and 250. Additional plots (not included) of r_s against moisture content and sample quality indicator ($\Delta e/e_0$) similarly show no clear relationship. These relatively low values may reflect the small organic content in the material.

According to Janbu (1985) for "medium clay", with water content of about 32% and at "low" stress, r_s should be approximately 250. As for m and c_v this suggests that the Os oedoemter test parameters are much closer to those of a clay rather than silt, despite the stress – strain curves being characteristic of silt. As for the c_v data there is no evidence of preconsolidation.

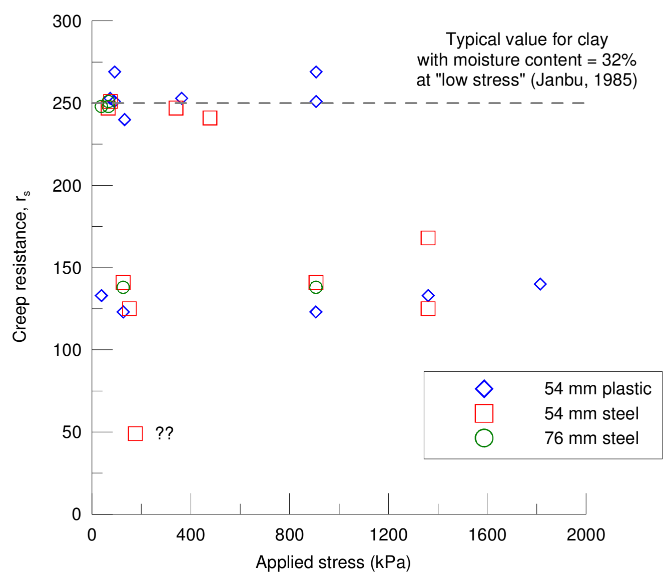

Figure N. Creep resistance, r_s

6. 0. TRIAXIAL TESTS RESULTS

Detailed test results for the CAUC triaxial tests are given on Figures 46a to 56b of the lab test report (note two figures a plus b per test) and are summarised on Table 3 of that report.

6. 1. Sample quality assessment

As for the oedometer tests it is initially necessary to make an assessment of the sample quality using the techniques of Kleven et al. (1986) and Lunne et al. (1997) as shown on Figure O.

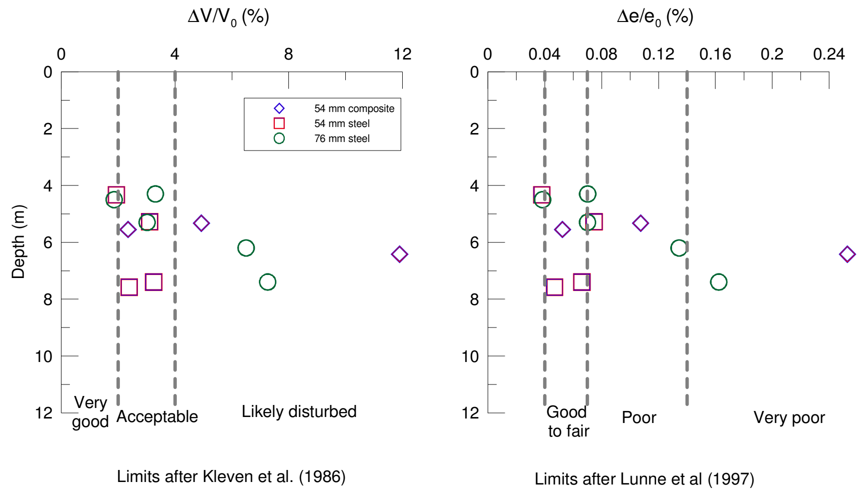

Figure O. Triaxial tests – sample quality assessment

From the figure the following observations can be made:

- The two criteria give a similar assessment
- The 54 mm plastic sample quality is very variable and on average these samples are the worst.
- The 76 mm steel data fall into two groups, those of "good to fair" quality and "poor to very poor" quality. The reason for this will be discussed below.
- The quality of the 54 mm steel samples is similar to the first group of 76 mm samples.

As for the oedometer tests the data appears to plot in two zones. Those tests with $\Delta V/V_0 > 6\%$ were clearly disturbed during specimen preparation. Inspection of Table 3 of the lab test report shows that, of the 18 tests which were commenced, only 11 were completed. This was due to loss of coherence and slumping of the specimens during preparation. All of the specimens were trimmed from 54 mm or 76 mm to 50 mm (with the exception of two). It was difficult to prepare all of the specimens. Local dilation occurred as the piano wire was drawn through the sample and the material tended to "stick" to the wire and not come away cleanly as it would for say a marine clay such as Onsøy. Even after placing the specimen in the triaxial apparatus it was still very sensitive to disturbance.

All of the specimens which slumpled were from the sensitive clayey zone below 6 m. There was a particular problem with the 54 mm plastic and the 76 mm samples. For the 76 mm samples the problem was more acute as the parent sample was larger and therefore required more trimming. Two of the 76 mm tests (76-4 and 76-5) were carried out at 76 mm diameter. However the result were variable. This is the reason for the grouping in the results for the 76 mm samples. Those which behave well during preparation are classified as "good to fair", those which did not are classified as "poor".

In conclusion it is felt that:

- overall the quality of the 76 mm and 54 mm steel samples is similar,
- it is not clear whether it is advantageous or not to trim the specimens or work at the sampled diameter (requires further study)
- Some better technique, other than simple piano wire trimming in a soil lathe, needs to be developed for preparation of the samples below 6 m. A possibility is the technique described by Landva (1964).

6. 2. Stress – strain behaviour

In order to gain an understanding of the expected behaviour of silt in triaxial testing, reference is made to Figure P from Ovando-Shelley (1986). Tests were carried out on loose (porosity = 42.6%), medium dense (39.7%) and dense silt (37.7%). From the stress paths it can be seen that initially the loose and medium dense silts contract before exhibiting dilatant behaviour as they near failure. The dense silt dilates throughout shearing. All the materials show an increasing deviator stress with increasing axial strain, the slope of this line increasing with increasing density.

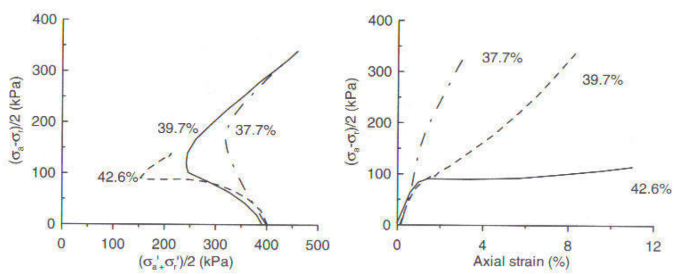

Figure P. Stress – strain behaviour and stress paths in silt (Ovando – Shelley, 1986)

6. 3. Stress paths

All of the CAUC test results from depths 4.4 m, 5.4 m and 7.4 m are summarised on Figures 6, 7 and 8 of this report. Some observations can be made as follows:

- All of the tests from 4.4 m and 5.4 m and one of those form 7.4 m exhibit dilatant behaviour from the start of shearing.
- These test results generally resemble those of the dense silt on Figure P, despite the CPTU and other data suggesting the material is loose.
- Two of the tests from 7.4 m exhibit contractant behaviour consistent with the earlier finding that the material from this zone is more "clayey".
- The results form 5.4 m are particularly interesting as they show that specimens form each of the 3 sample types behave in a very similar manner, at least up to strains of about 6%.
- The results from 4.4 m suggest there may be some advantage in trimming the 76 mm samples before testing although insufficient data is available to make any definitive conclusion here.
- The results from 7.4 m suggest the 76 mm steel sample quality is similar to that of the 54 mm steel sample despite it having a much higher $\Delta V/V_0$ value (7.26% compared to 2.39%).

This gives further credence to the point developed above that the normal sample quality assessment procedures may not apply to silty material.

6. 4. Undrained shear strength (s_u)

It is not clear how to interpret of s_u from triaxial tests on silt which exhibits dilatant behaviour as discussed above. It is clearly not appropriate to adopt the same technique as for clays where the simple peak value is taken. Here the applied shear stress can increase constantly with increasing strain. Some possibilities for the determination of s_u for silt are as follows (Brandon et al., 2005)

1. Simple peak deviator stress regardless of strain (conventional approach)
2. Shear stress at some limiting strain
3. A = 0 or Δu = 0
4. Reaching Mohr - Coulomb line
5. Peak principle stress ratio (σ'_1/σ'_3)
6. Peak pore pressure

For the purposes of this study the first 4 criteria will be used. For criterion 2 the strength will be taken at 2% strain.

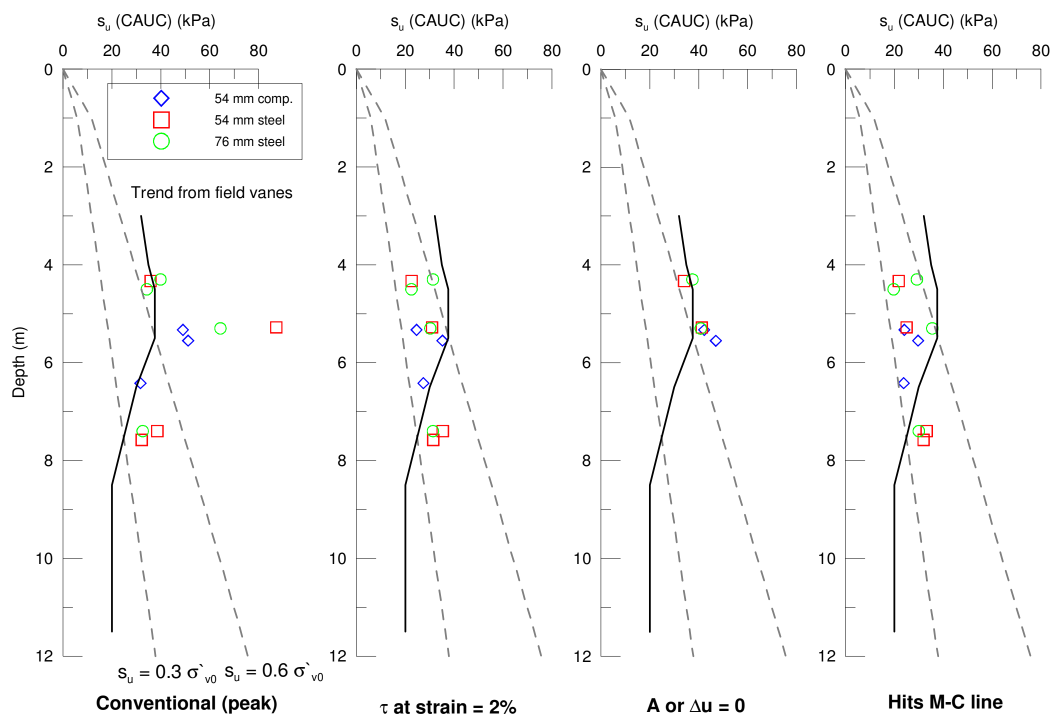

Figure Q. s_u from lab testing using various interpretation methods

s_u obtained form the 4 methods are shown on Figure Q. The trend from the field vane tests are shown for guidance. It would expected that the lab values would be higher than those from the field vane. Chandler (1987) suggested that for material with I_p = 13%, vane strength will be about 70% of lab CAUC strength.

For the conventional approach many of the results seem reasonable but there a clearly a few spurious high results due to method of interpretation. The other three methods give no unusually high results. However it seems that Methods 2 and 4 seem to yield somewhat conservative values of s_u.

Brandon et al. (2005) recommended the use of Method 3 for silts and it can be seen that here also it seems to give sensible output. Unfortunately not all of the test showed a situation where A or Δu became zero, hence there are fewer points. However this criterion seems useful and warrants more work. It is possible that use of this method may be associated with very high axial strains therefore some care is needed in its implementation.

s_u values derived from CPTU and T-bar tests are shown on Figure R. The same scales are used on Figures Q and R for comparison purposes. s_u was derived from the pore pressure response ($N_{\Delta u}$), the net cone resistance (N_{kt}) and the effective cone resistance (N_{ke}) as well as the net T-bar resistance. Maximum and minimum factors for a sensitive clay with OCR of about 2 were obtained form Karlsrud et al. (2005) and are summarised on Table C. Note that these factors are intended to yield CAUC s_u values corresponding to high quality samples.

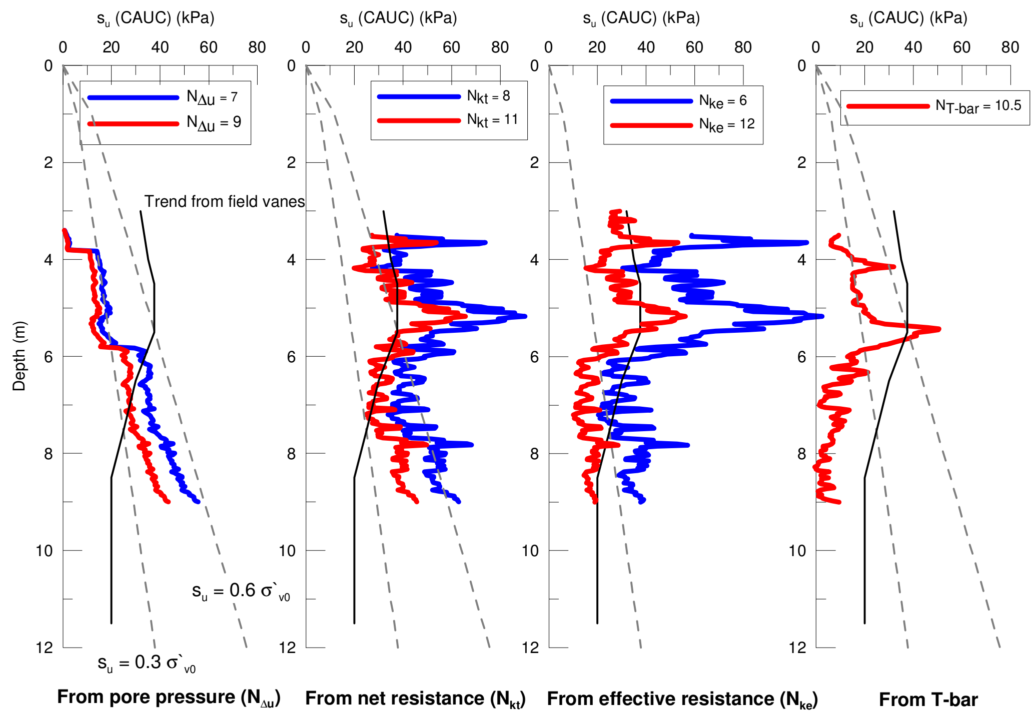

Figure R. s_u derived from CPTU and T-bar tests

Table C. Appropriate CPTU and T-bar strength factors (Karlsrud et al., 2005)*

Factor	$N_{\Delta u}$	N_{kt}	N_{ke}	$N_{T\text{-}bar}$
Minimum	*7*	*8*	*6*	*10.5*
Maximum	*9*	*11*	*12*	*10.5*

* except for N_{ke}. Min form Karlsrud et al. (2005). Max. from Stjordal (2005)

The following comments can be made:

- The conventional N_{kt} approach seems to work well for this material.
- As does the $N_{\Delta u}$ approach but only in the more clayey material below 6 m. Above 6 m unreasonably low values of s_u are obtained
- The effective resistance (N_{ke}) approach also works well once the maximum N_{ke} of 12 is used. This value was obtained from local experience of silty soils in the Bergen area by Stjordal (2005).
- T-bar values are very low and this is probably due to factor of 10.5 being inappropriate for high sensitivity material.

6. 5. Other parameters from triaxial tests

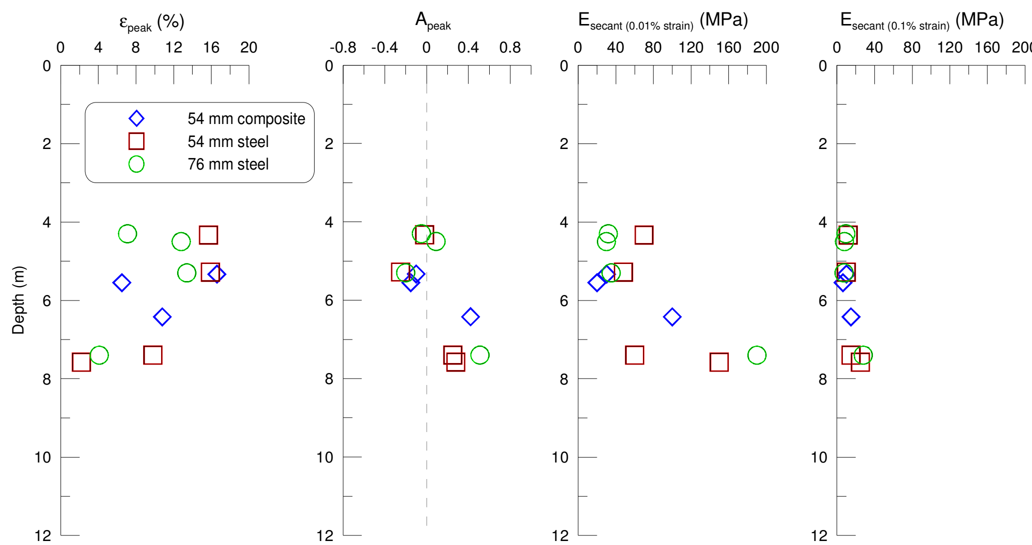

Figure S. Other parameters from triaxial tests

Other parameters derived from the triaxial test results are shown on Figure S. Some comments can be made as follows:

- ε_{peak} more or less the same for all samplers, drops with depth as soil becomes more clayey
- A_{peak} two groups less than zero for silty soil above 6 m and greater than zero for clayey soil below 6 m. No clear difference between sampler types.
- E much the same for all sampler types, increase with depth. The material shows significant non linearity of stiffness

6. 6. Effective stress strength parameters

Inspection of Figures 6 to 8 at the rear of this report shows that the effective stress parameters are more or less the same for all sampler types. Typically $\phi' = 35°$ and c' (or a') = 0.

These values can be compared to those which would be obtained form CPTU results using the technique of Senneset et al. (1988), which is reproduced in Lunne et al. (1997b). Taking the chart corresponding to lightly overconsolidated silts ($\beta = 0°$)

- For the 3 m to 6 m depth interval, N_m is typically in the range 8 to 12, $B_q = 0.25$, $\tan\phi' = 0.65$, $\phi' = 33°$
- Below 6 m, N_m is typically in the range 4 to 9, $B_q = 0.65$, $\tan\phi' = 0.7$, $\phi' = 35°$

The CPTU values are consistent with the lab data suggesting that the amount of densification caused by sampling may be small.

6. 7. Shear wave velocity measurements

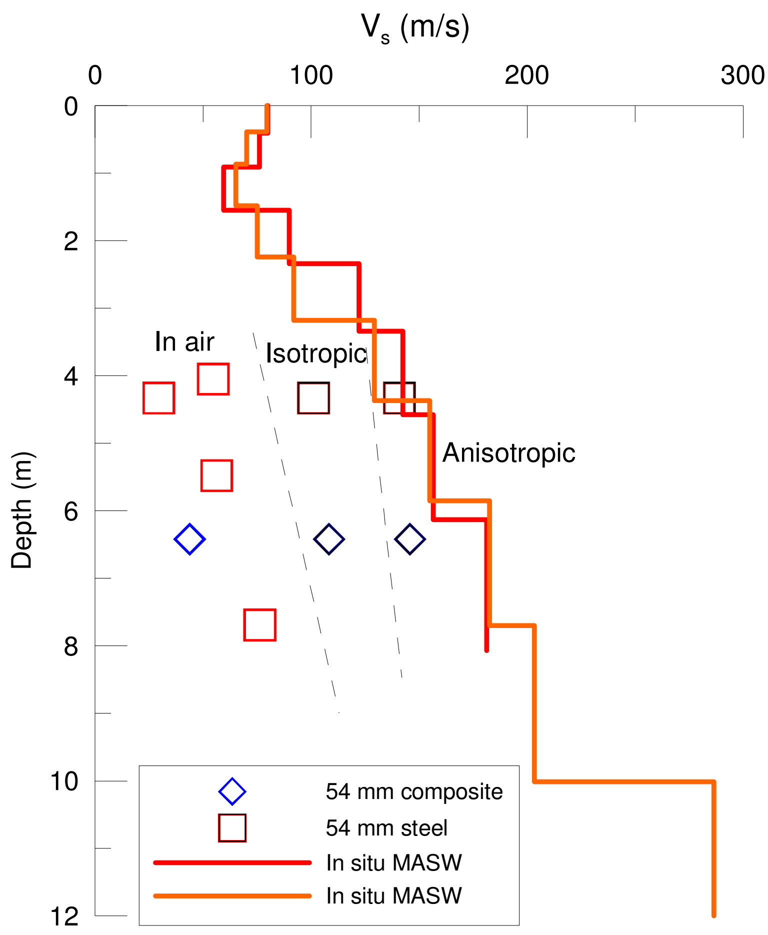

Figure T. Results of lab and in situ shear wave velocity measurements

Results of laboratory and field shear wave velocity measurement are shown on Figure T. For the lab results values "in air" immediately after extrusion, after isotropic consolidation and anisotropic consolidation are given. In situ values were obtained using the MASW surface wave technique. Several profiles of the site were tested and the values obtained were very consistent. For a full discussion on these techniques the reader is referred to Donohue (2005)

Donohue (2005) found that the "in air" values were the best for the purposes of assessing sample quality. Relatively few data are available here but the trend suggests the 54 mm steel samples are of higher quality than the 54 mm composite ones. The V_s values measured in the lab after anisotropic consolidation are very similar to those measured in situ, again suggesting that sampling induced densification may be small.

7. 0. CONCLUSIONS

1. Material relatively uniform with approximately constant water content and bulk density with depth
2. Unusually high particle density.
3. Clear evidence of fresh water leaching.
4. Soils in the sampling zone can be sub-divided into two layers, i.e. a more silty "high sensitivity" soil above about 6 m and a more clayey "quick" soil below 6 m.
5. The pore pressure parameter B_q is very useful for the purposes of soil classification. Penetration tests in silty material should always include pore pressure measurement.
6. Discrepancy between s_{ur} from field vane and index tests. Field vane values unreasonably high.
7. For both the oedometer and triaxial tests the normal techniques used to quantify sample quality here may be inappropriate and may not reflect the true specimen quality.
8. Material lightly overconsolidated above about 7.5 m
9. Oedometer stress – strain and stress - M curves exhibit "silt" behaviour but parameters m, c_v and r_s closer to those of "clay" rather than silt.
10. In triaxial tests 54 mm steel and 76 mm sample quality similar but 54 mm plastic worst.
11. Many of the samples show triaxial behaviour similar to that of a dense silt, where CPTU, total sounding and other data confirm it to be loose.
12. The conventional approach of obtaining s_u from triaxial tests is unsuitable for dilatant silts. The method involving taking strength where A or Δu becomes zero seems sensible and warrants more research.
13. The conventional N_{kt} approach of obtaining s_u from CPTU seems to work well for this material. Other techniques also give reliable results.
14. T-bar derived s_u values are very low probably due to the non appropriate use of normal factors to this high sensitivity.
15. Lab and CPTU data give similar ϕ' values in the range 33° to 35°. Similarly lab and in situ V_s values are similar. Both findings suggest sampling induced densification may be small.
16. Comparison of lab and in situ V_s values may be a valuable tool to help assess sample quality.

8. 0. RECOMMENDATIONS

1. Carry out X-ray diffraction and scanning electron microscope studies to examine microstructure of material, check for reason for high particle density etc.
2. Seek alternative methods for determining sample quality of silty material (e.g. comparison of laboratory shear wave velocity by bender elements and in situ measurements)
3. Investigate discrepancy in f_s values from two CPTU rigs. Importance in classification of material.
4. Investigate further by additional sampling the reason for the lower than expected quality of the 76 mm samples. Consider using full displacement sampling, without water and with tubes with sharpened cutting edge angles.
5. Work need to be carried out to develop a better technique for specimen preparation for triaxial testing and to investigate whether the specimens should be trimmed or tested at the sample diameter.

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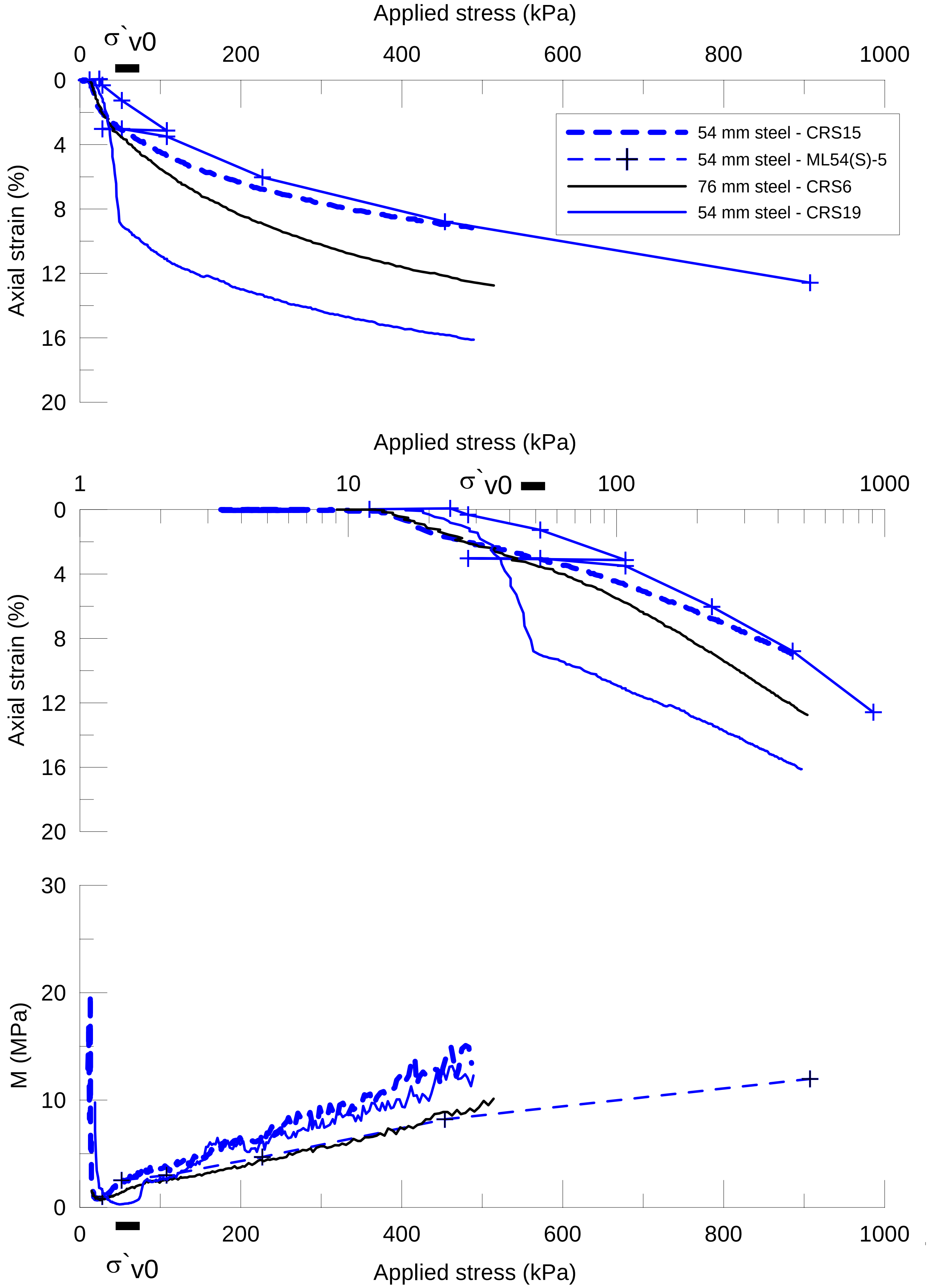
Special thanks to the families Long and Cosgrave for taking care of the "iceman" while he stayed in Ireland during some short period while working on the project.

REFERENCES

1. Statens Vegvesen (2005a). “Parameterbestemmelser for siltige materialer, Delrapport A, Feltundersøkelser”. Region Vest, Teknologiavdelingen, Internal Rapport, Nr. 2381, dated 05.02.2005.
2. Statens Vegvesen (2005b). “Parameterbestemmelser for siltige materialer, Delrapport B, Laboratorieundersøkelser”. Region Vest, Teknologiavdelingen, Internal Rapport, Nr. 2382, dated 05.02.2005.
3. Andresen, A. and Kolstad, P. (1979). “The NGI 54 mm sampler for undisturbed sampling of clays and representative sampling of coarser materials”. Proc. Int. Sym. SOA Current Practice of Soil Sampling, Singapore, pp 13 - 21.
4. Bondevik, S. and Mangerud, J. (2002). “A calendar age estimate of very late Younger Dryas ice sheet maximum in western Norway”. Quaternary Science Reviews, 21, pp 1661 – 1676.
5. Brandon, T.L., Rose, A.T. and Duncan, J.M. (2005). “Drained and undrained strength interpretation for low plasticity silts”. Submitted for publication in ASCE Journal of Geotechnical Engineering.
6. BS5930 (1999) “Code of practice for site investigations”. British Standard Institute. (Revised and republished in 1999. Originally issued in 1981)
7. Chandler, R.J. (1987). “The in-situ measurement of the undrained shear strength of clays using the field vane”. Proc. Int. Sym. on Lab. and Field Vane Testing, Tampa, Florida, ASTM STP 1014, AF Richards ed, pp 13 - 44.
8. Donohue, S. (2005). “Soil sample quality from suction and shear wave velocity measurements”. PhD Thesis, University College Dublin, September
9. Fellenius, B.H. and Eslami, A. (2000). “Soil profiles interpreted from CPTU data”. Year 2000 Geotechnics. Geotechnical Engineering Conference. Asian Institute of Technology. Bangkok, Thailand, November.
10. Head, K.H. (1980). Manual of soil laboratory testing, Volume 1, Soil classification and compaction tests”. Pentech Press, ISBN 0 7273 1304 5.
11. Hight, D.W., Jardine, R.J. and Gens, A. (1987) “The behaviour of soft clays”. In Embankments on Soft Clays, Special Publication Bulletin of the Public Works Research Centre, Ministry of the Environment, Physical Planning and Public Works, Athens, Greece.
12. Janbu, N. (1985). “Soil models in offshore engineering”. 25th Rankine Lecture, Geotechnique, 35, No. 3, pp 241 – 281.
13. Jeffries, M.G. and Davies, M.P. (1991). Discussion on “Soil classification by the cone penetration test”. Can Geo. Jnl., 28(1), 173 – 176.
14. Karlsrud, K., Lunne, T., Kort, D.A. and Strandvik, S. (2005). ”CPTU correlations for clays”. 16th ICSMGE, Osaka, September.
15. Kleven, A., Lacasse, S. and Andersen, K.H. (1986). “Foundation engineering criteria for gravity platforms”. Soil parameters for offshore foundation design. Part II, NGI Report 40013-34, April.
16. Landva, A. (1964). “Equipment for cutting and mounting undisturbed specimens of clay in testing devices”. NGI Publication No. 56, pp 1 - 5.
17. Lunne, T., Robertson, P.K. and Powell, J.J.M. (1997a). “Cone penetration testing in geotechnical practice”. Blackie Academic and Professional, ISBN 0 751 40393 8.
18. Lunne, T., Berre, T. and Strandvik, S. (1997). “Sample disturbance effects in soft low plasticity Norwegian clay”. Proc. Sym. on Recent Developments in Soil and Pavement Mechanics, Rio de Janeiro, June 1997, pp 81 -92. Published by Balkema.
19. Lunne, T., Long, M and Forsberg, C.F. (2003). “Characterisation and engineering properties of Onsøy clay”. Proceedings International Workshop on Characterisation and Engineering Properties of Natural Soils (“Natural Soils 2002”). NUS Singapore, December. Eds. Tan, T.S. et al. Published by Balkema, Vol. 1, pp 395 - 428.

20. Norges Geoteknisk Forening, NGF (1982). “Veiledning for symboler og definisjoner I geoteknikk, presentasjon av geotekniske undersøkelser”. Melding Nr. 2.
21. Ovando-Shelley, E. (1986). “Stress – strain behaviour of granular soils tested in the triaxial cell”. PhD thesis University of London, Imperial College.
22. Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J. (1986). “Use of piezometer cone data”. Proc. ASCE Speciality Conference In Situ ’86: Use of In Situ Tests in Geotechnical Engineering, Blacksburg, pp 1263 – 1280.
23. Robertson, P.K. (1990). “Soil classification using the cone penetration test”. Can Geo. Jnl., 27(1), pp 151 – 158.
24. Sandven, R. (1990). “Strength and deformation properties of fine grained soils obtained from piezocone tests”. PhD Thesis, NTH Trondheim.
25. Sandven, R. (2003). “Geotechnical properties of a natural silt deposit obtained from field and laboratory tests”. Proceedings International Workshop on Characterisation and Engineering Properties of Natural Soils (“Natural Soils 2002”). NUS Singapore, December. Eds. Tan, T.S. et al. Published by Balkema, Vol. 2, pp 1237 - 1278.
26. Schnaid, F., Lehane, B.M. and Fahey, M. (2004). “In situ test characterisation of unusual geomaterials”. Proc 2$^{nd.}$ Int. Conf on Geotechnical Site Characterisation, ISC’2, Porto, September, Vol. 1, pp 49 – 74. Published by MillPress.
27. Senneset, K. and Janbu, N. (1985). “Shear strength parameters obtained from static cone penetration tests”. ASTM Special Technical Publication, STP 883, pp 41 – 54.
28. Senneset, K., Sandven, R., Lunne, T., By, T., and Amundsen, T. (1988). “Piezocone tests in silty soils”. ISOPT-1, Orlando, 2, pp 955 – 966, Balkema.
29. Stjordal, A. (20059. Personal communication with Arne Stjordal of MultiConsult, Bergen.
30. Taylor, D.W. (1942). “Research on consolidation of clays” MIT Dept. of Civil Engineering, Serial No. 82.



FoU Project - Os Site - 2004 Site Investigation

Report No. 2005 - G2

Figure No. 1

Comparison of oedometer tests 4 to 4.8 m

on 76 mm steel, 54 mm plastic and 54 mm steel specimens

$\sigma`_{v0}$ = 50 kPa

Drawn by GC

Date 2005-10-12

Checked ML

Approved

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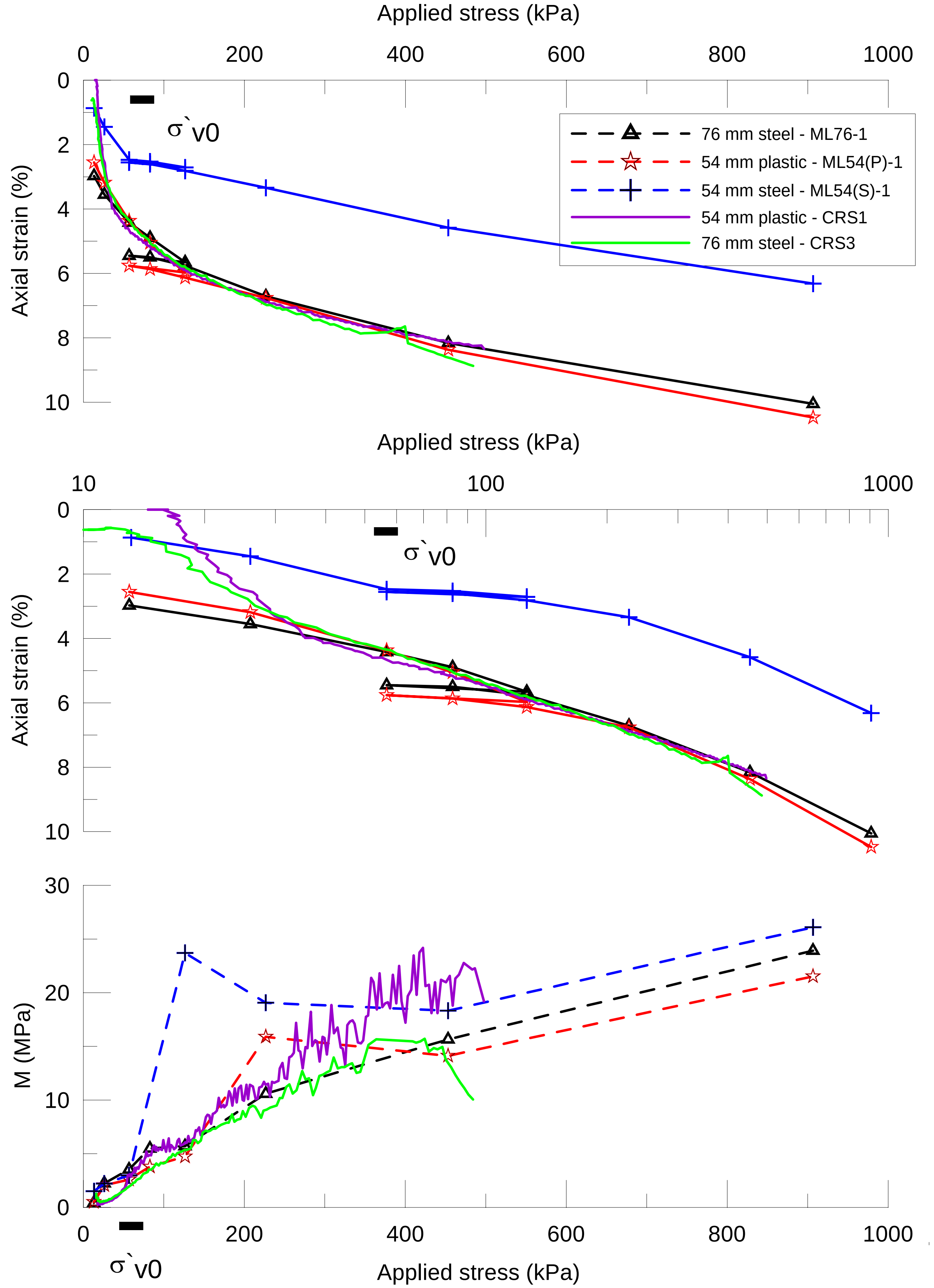

FoU Project - Os Site - 2004 Site Investigation	Report No. 2005 - G2	Figure No. 2
Comparison of oedometer tests 5 to 5.8 m on 76 mm steel, 54 mm plastic and 54 mm steel specimens $\sigma`_{v0}$ = 58 kPa	Drawn by GC	Date 2005-10-12
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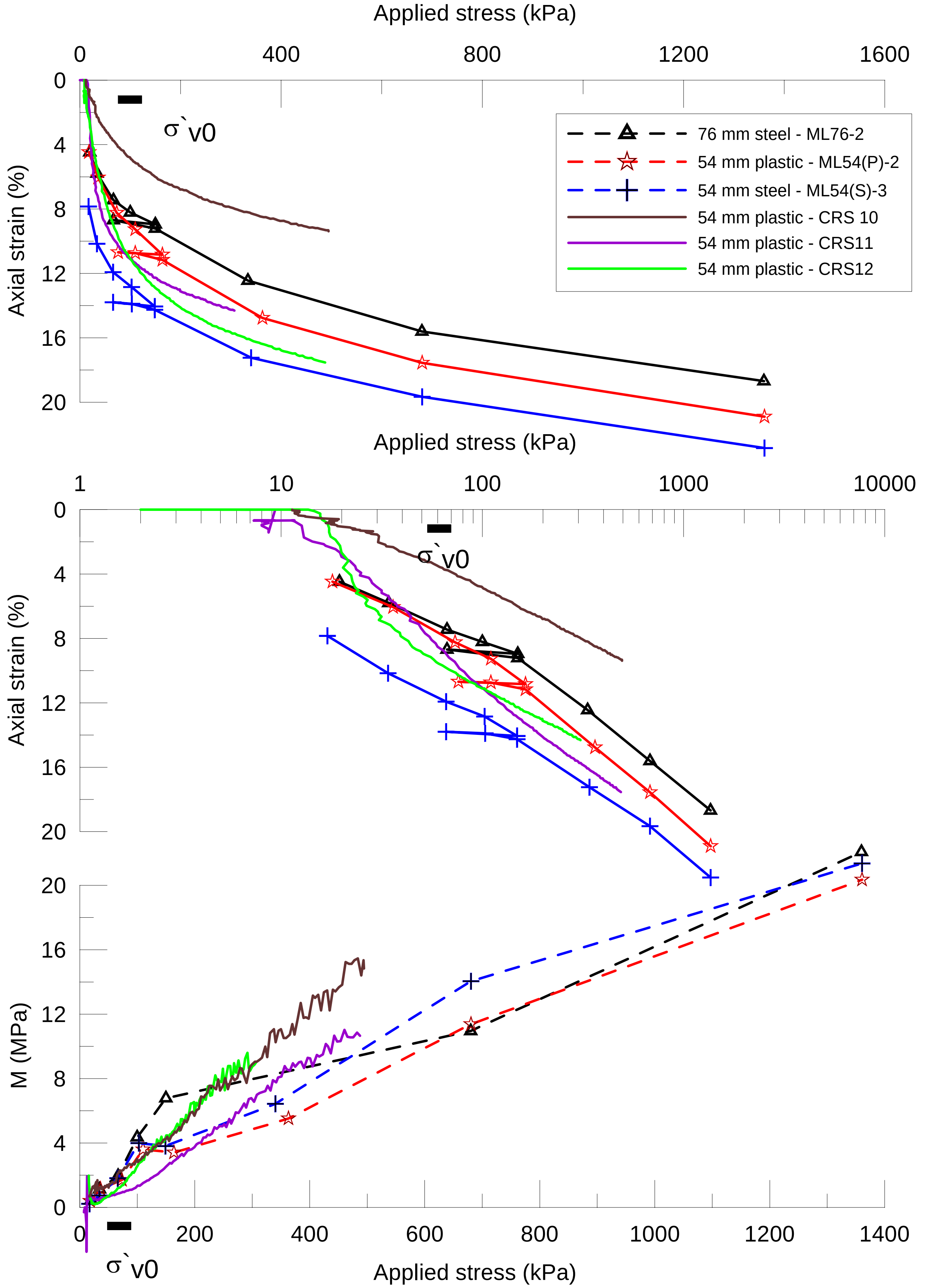

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FoU Project - Os Site - 2004 Site Investigation

Report No. 2005 - G2

Figure No. 3

Comparison of oedometer tests 6 to 6.8 m
on 76 mm steel, 54 mm plastic and 54 mm steel specimens
$\sigma`_{v0}$ = 70 kPa

Drawn by GC

Date 2005-10-12

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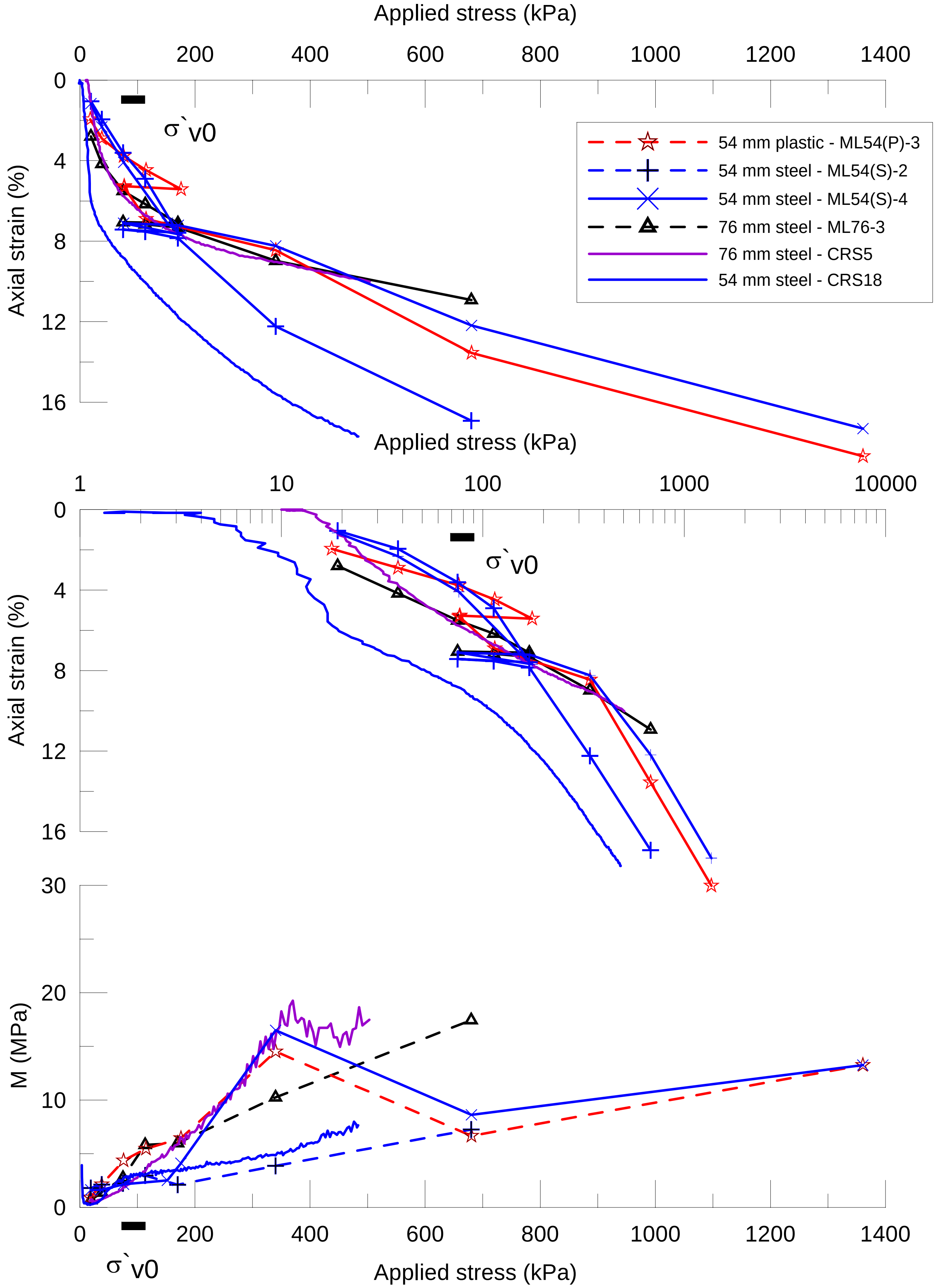

FoU Project - Os Site - 2004 Site Investigation

Report No.	Figure No.
2005 - G2	4
Drawn by GC	Date 2005-10-12
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Comparison of oedometer tests 7 to 7.8 m
on 76 mm steel, 54 mm plastic and 54 mm steel specimens
$\sigma`_{v0}$ = 75 kPa

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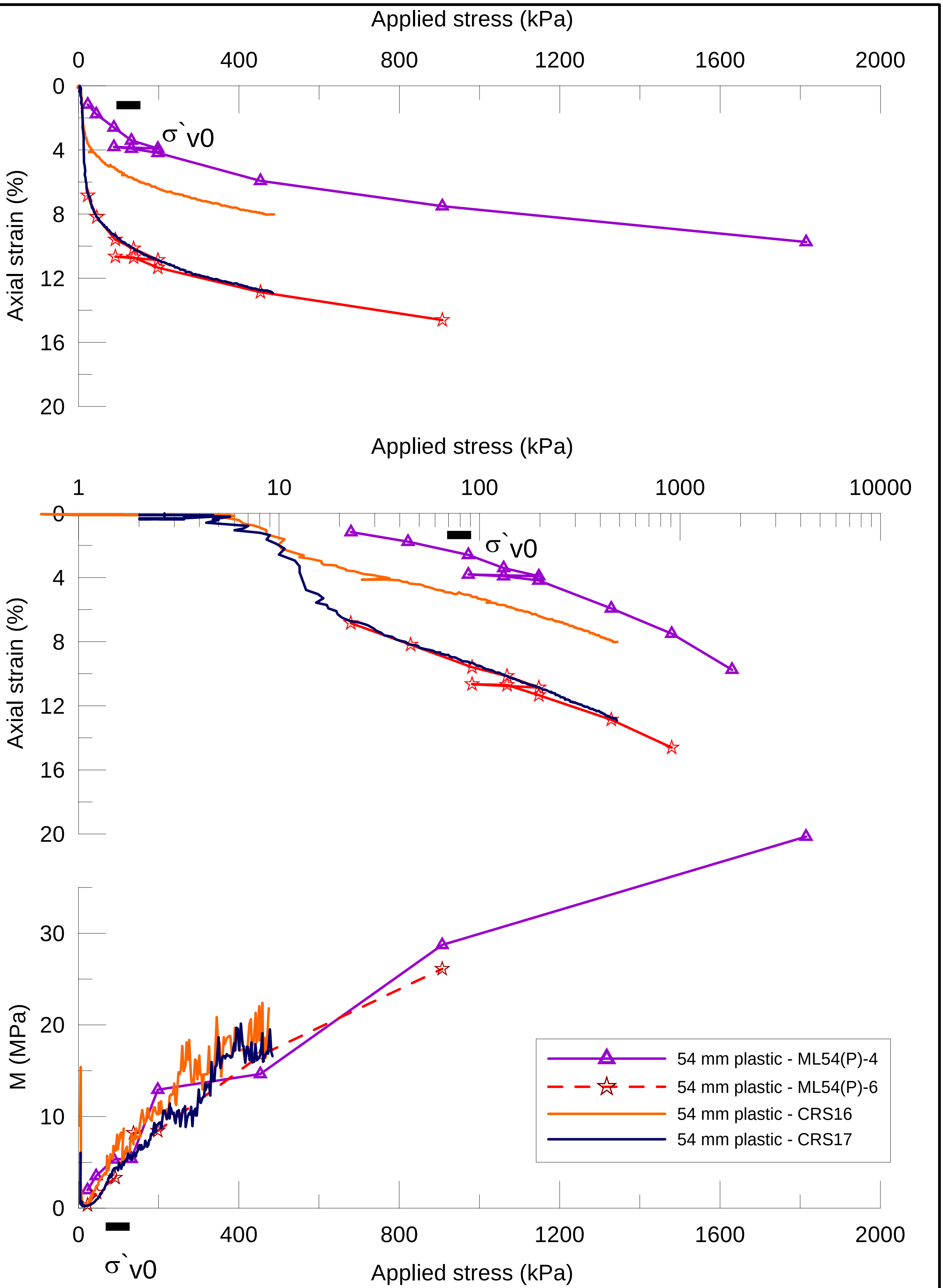

FoU Project - Os Site - 2004 Site Investigation	Report No. 2005 - G2	Figure No. 5
Comparison of oedometer tests 8 to 8.8 m on 76 mm steel, 54 mm plastic and 54 mm steel specimens $\sigma`_{v0}$ = 88 kPa	Drawn by GC	Date 2005-10-12
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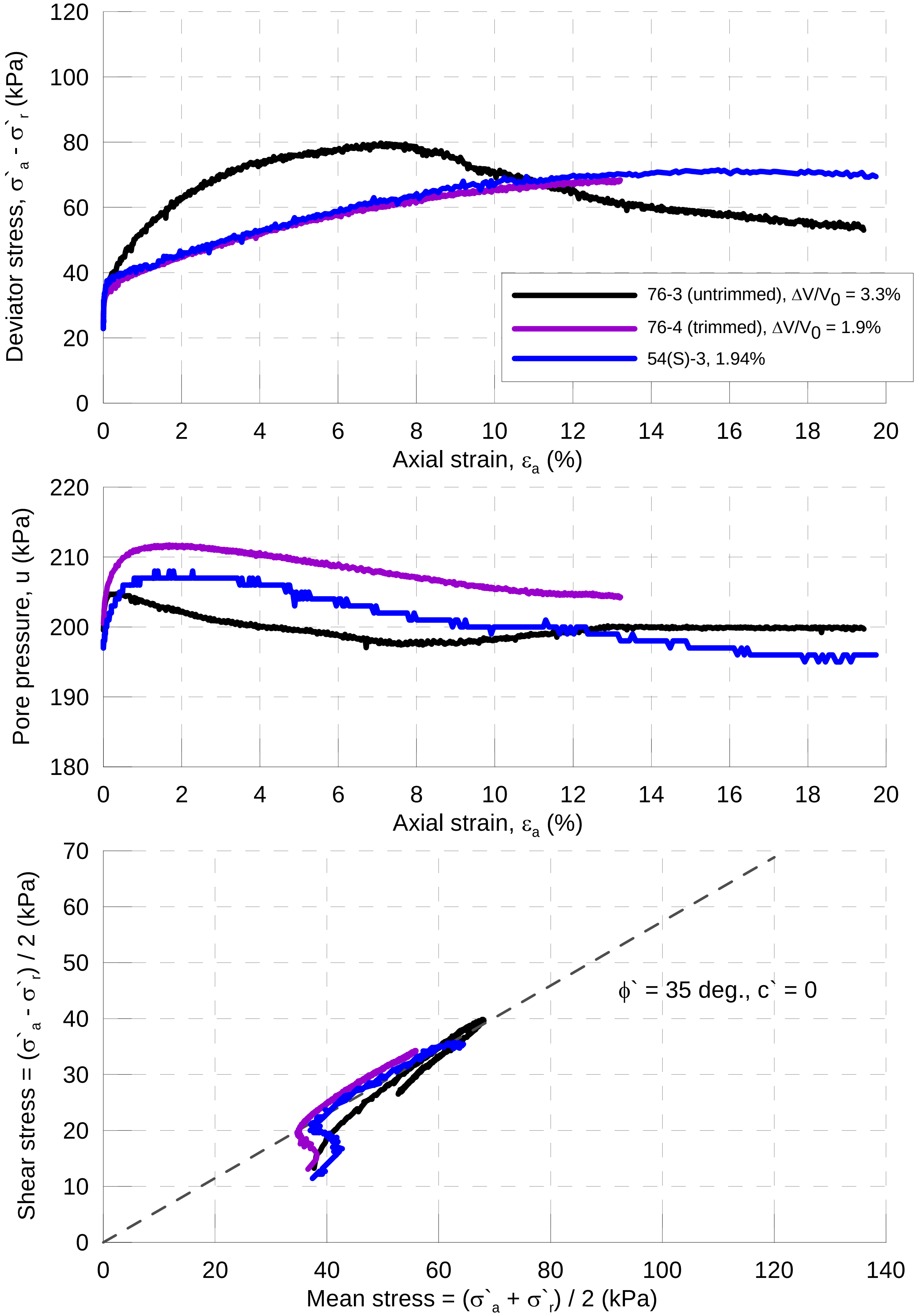

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FoU Project - Os Site - 2004 Site Investigation	Report No. 2005 - G2	Figure No. 6
Comparison of all CAUC tests at about 4.4 m	Drawn by GC	Date 2005-10-10
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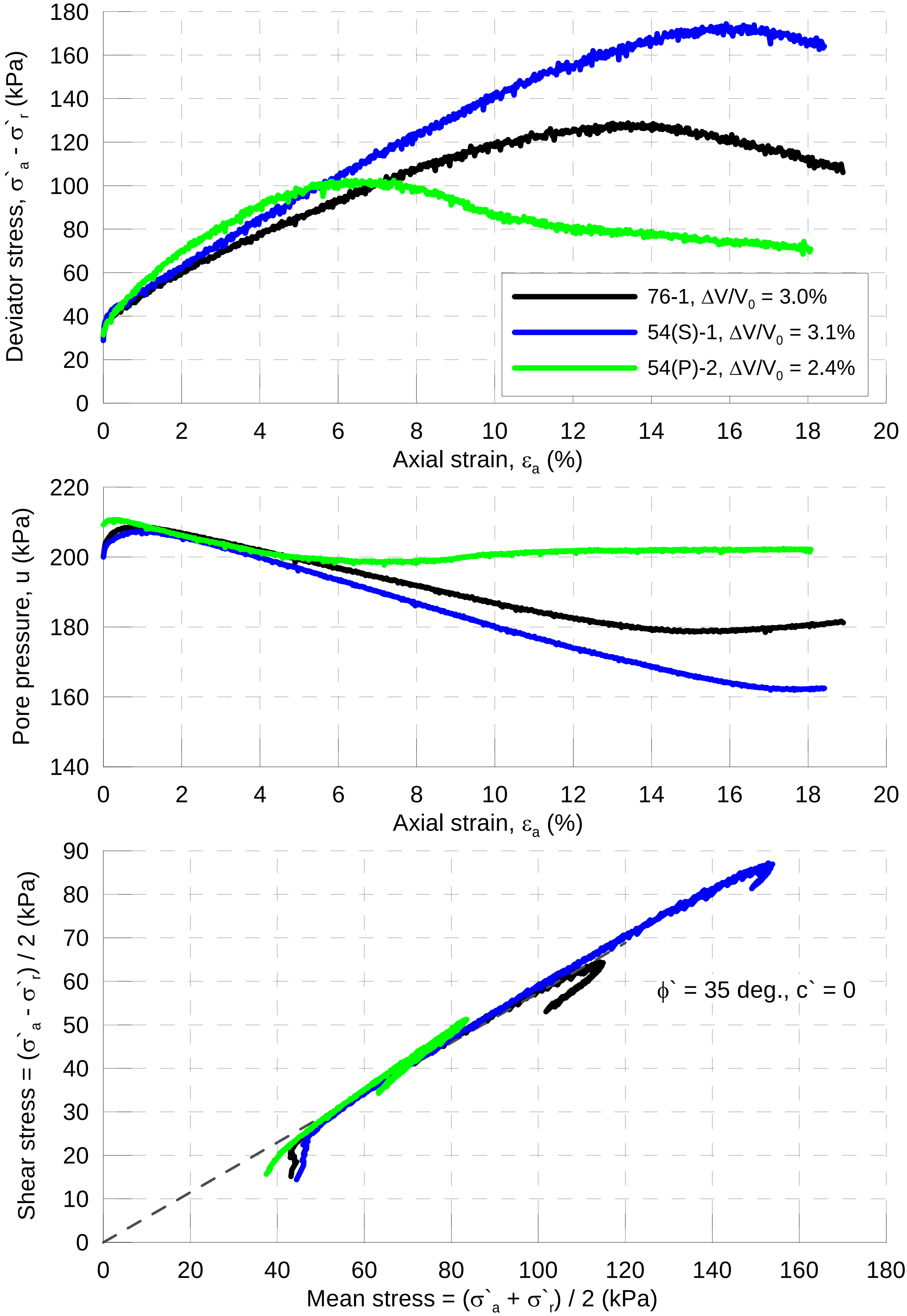
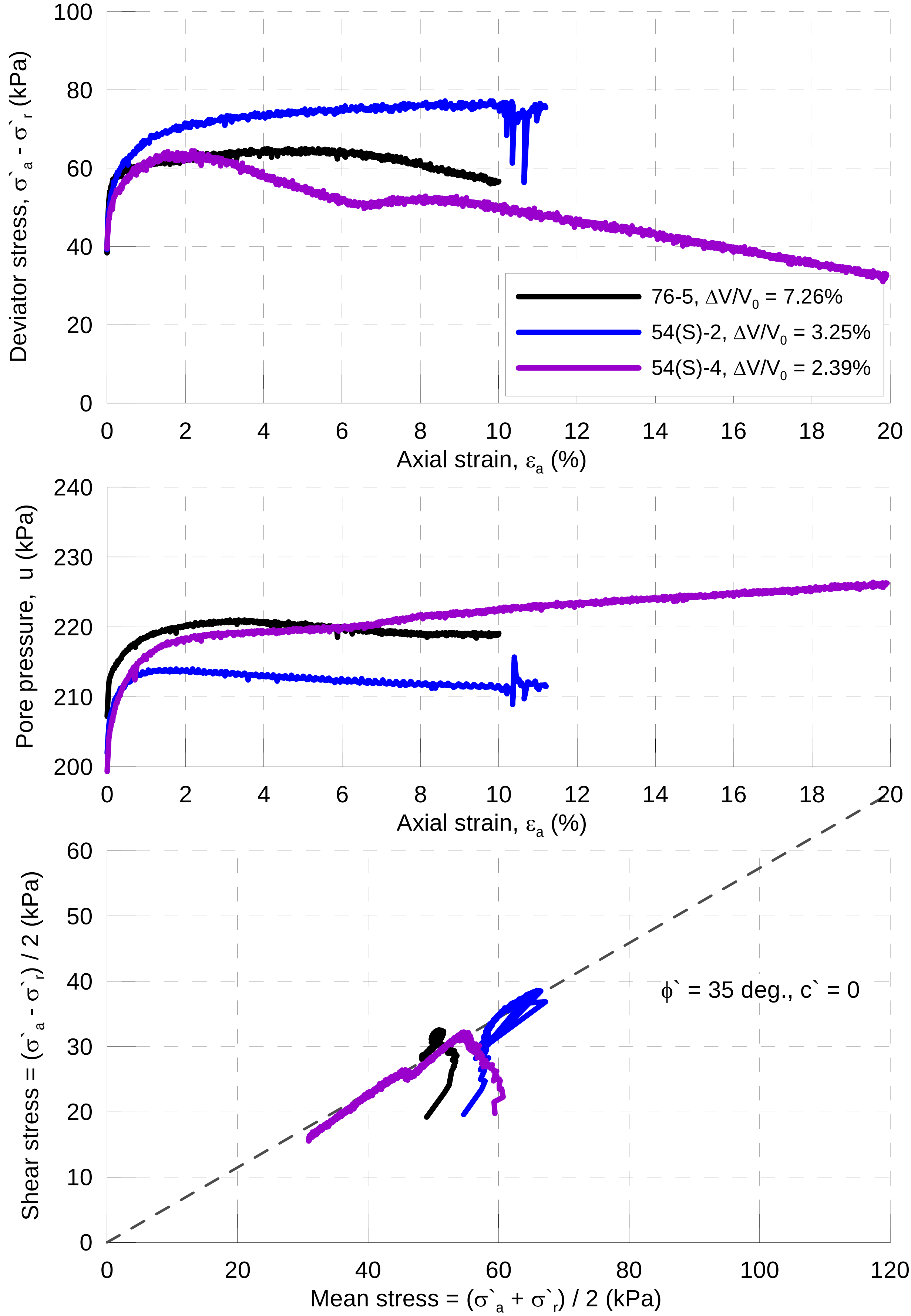
	Approved	



FoU Project - Os Site - 2004 Site Investigation	Report No. 2005 - G2	Figure No. 7
Comparison of all CAUC tests at about 5.4 m	Drawn by GC	Date 2005-10-10
	Checked ML	
	Approved	

D(AF)\FoU\AllCAUC5.4m.grf



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FoU Project - Os Site - 2004 Site Investigation	Report No. 2005 - G2	Figure No. 8
Comparison of all CAUC tests at about 7.4 m	Drawn by GC	Date 2005-10-10
	Checked ML	
	Approved	

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