Consequences of sample disturbance for predicting long-term settlements in soft clay

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<th>Journal:</th>
<th>Canadian Geotechnical Journal</th>
</tr>
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<td>Manuscript ID</td>
<td>cgj-2016-0129.R2</td>
</tr>
<tr>
<td>Manuscript Type:</td>
<td>Article</td>
</tr>
<tr>
<td>Date Submitted by the Author:</td>
<td>28-Jul-2016</td>
</tr>
<tr>
<td>Complete List of Authors:</td>
<td>Karlsson, Mats; Chalmers University of Technology, Civil and Environmental Engineering</td>
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<td>Emdal, Arnfinn; The Norwegian University of Science and Technology, Civil and Transport Engineering</td>
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<td>Dijkstra, Jelke; Chalmers University of Technology, Department of Civil and Environmental Engineering</td>
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<tr>
<td>Keyword:</td>
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Note: The following files were submitted by the author for peer review, but cannot be converted to PDF. You must view these files (e.g. movies) online.

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Consequences of sample disturbance for predicting long-term settlements in soft clay

Mats Karlsson, Arnfinn Emdal & Jelke Dijkstra

July 30, 2016

Abstract

An approach for assessing the effects of sample quality is presented. Soil samples were taken using a 50 mm Swedish STII piston sampler and the NTNU mini-block sampler from a soft clay test site. Differences in laboratory test results are identified for several stress paths, assisted by simulations made using an advanced constitutive model. Hitherto such comparisons have focused on differences in basic engineering properties such as strength and stiffness. The effect of choosing alternative model parameters from piston and block samples is demonstrated through the analysis of the long-term settlement of an embankment. The simulations show that substantially larger settlements and lateral displacements are predicted using parameters obtained from the piston samples. Furthermore, the magnitude of the differences is
larger than expected. This demonstrates that for this application relatively small differences in the assessed sample quality, using traditional laboratory data interpretation methods, are amplified when applied to a prototype boundary value problem. It is suggested that a little more care in sampling and testing can result in large cost savings as a result of the more reliable model parameters that can be extracted, particularly when the improved sampling is combined with the use of an advanced constitutive model.

Key words: sample disturbance, soft soils, numerical modelling, long-term predictions.

Introduction

The effect of soil sample disturbance have been widely studied and various new types of samplers have been designed to reduce the disturbance. In parallel, special laboratory tests have been devised to simulate the effects of sample disturbance and metrics have been introduced to assess sample quality from comparison of subsequent non-destructive and destructive tests (Lefebvre and Poulin 1979; Rochelle et al. 1981; Baligh et al. 1987; Hight 1993; Tanaka et al. 1996; Lunne et al. 1997; Tanaka et al. 2002; Santagata and Germaine 2002; Poirier et al. 2005; Long 2006; Lunne et al. 2006; Landon et al. 2007; Löfroth 2012; Chung et al. 2014). The majority of the research so far however stops after quantifying the effect of sample disturbance and its effect
on basic soil properties, such as (small-strain) stiffness, (undrained) shear strength and/or pre-consolidation pressure. Although it may be assumed that a higher peak strength or stiffness obtained from a less disturbed sample would be desirable in geotechnical application, much more information about the soil is obtained from consideration of the full stress-strain response. This is especially true for long-term predictions of deformations in rate dependent sensitive soft soils, where predictions with simplified methods tend to be somewhat inadequate. The application of advanced constitutive models which are more able to provide realistic simulations of a wide range of observed soil behaviours reveals even more starkly the benefits to be obtained from improved soil sampling.

In this paper we present one of the first holistic studies of the effects of sample quality on results of laboratory tests and associated predictions of long-term deformations. The quality of block samples taken with a mini-block sampler will be compared with samples taken with the Swedish industry standard 50 mm STII piston sampler. The comparison of sample quality will be based not only upon volume changes recorded during re-loading to in-situ effective stress, but also on predictions of long-term settlement response of an embankment made using the data from either the STII or mini-block sampler. This settlement analysis requires evaluation of model parameters and the execution of an advanced numerical analysis incorporating effects of consolidation and creep at boundary value level.
Experimental methodology

Two distinct methods for soil sampling have been used. The first is the Swedish 50 mm diameter piston sampler and the second is the high quality mini-block sampler developed by the Norwegian University of Science and Technology (NTNU) as a downsized version of the Sherbrooke block sampler (Lefebvre and Poulin 1979; Emdal et al. 2016). Soft soil samples with both samplers have been taken up to 10 meters depth at the Utby test site. A team of Swedish technicians performed the Swedish sampling and the field crew from NTNU performed the mini-block sampling. The samples were then transported to Chalmers University of Technology, stored at 7 °C and tested within two weeks after sampling at the climatised laboratory facilities at Chalmers.

The 50 mm STII Swedish piston sampler is described in detail in SGF (2009). The key dimensions are summarised Table 1. The main advantage of this sampler is that after extrusion from the tube the sample can be directly tested without additional trimming. The NTNU mini-block sampler, produces samples with a diameter of 165 mm and height of approximately 300 mm. The sampling procedure described in Karlsson et al. (2015) and Emdal et al. (2016) has several advantages compared to the full-sized Sherbrooke sampler such as: full compatibility with standard boring rigs without the need for construction of ramps or other support structures (which often leads to faster turnaround of sampling); and easier/better controlled sampling and sample
handling resulting from the smaller size and weight. Perhaps the most striking difference between the procedure for the Sherbrooke sampler compared to the STII sampler is that after sampling the sample is not covered with paraffin wax, so that a larger mechanical unloading of the sample can be expected (though suction will retain some of the effective stress). Compared to the piston sampler, an additional step is evidently required in sample preparation: the sample has to be trimmed to the same size as those obtained from the piston sampler. The data presented here for the STII samples are probably superior to those normally obtained from STII sampling since we took more than usual care with sample transport, sample storage and laboratory testing.

Table 1: Dimensions of STII sampler

<table>
<thead>
<tr>
<th>Sampler</th>
<th>$D_1$ (mm)</th>
<th>$D_2$ (mm)</th>
<th>$D_3$ (mm)</th>
<th>$t$ (mm)</th>
<th>Angle ($^\circ$)</th>
<th>$D_2/t$ (-)</th>
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<td>54</td>
<td>50.2</td>
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Test Site

The test site is located in Utby (WGS84DD 57.73650, 12.07117) East of the centre of Gothenburg, and is part of a research project on energy piles. A rather homogeneous clay deposit is found for depths from 6 to 10 metres which for all STII samples would classify as a high plasticity clay, CH, with high sensitivity following the British Standard 5930:2015. All samples, however, are close to the silt threshold and one block from a depth of 6 metres would be
classified as a high plasticity silt, MH. Table 2 presents the relevant properties as determined from the classification tests. The mini-block samples show systematically higher values of the plastic limit, wp, most probably because these mini-block samples are better sealed. Additional tests on the chemical composition of the pore water (using the ion chromatograph Dionex ICS-900: Table 3) and identification of metals (using the iCAP Q ICP-MS: Figure 1). The complexity of the natural pore water can be related to the clay chemistry, the marine deposition environment, and to recent contamination from a nearby upstream scrapyard. The electrical conductivity of the pore water ranges from about 1250 to 1400 mS/m, (measured, using WTW Multi 350i Multimeter, for different depths at 22 ºC).

**Laboratory testing**

All laboratory tests have been performed at Chalmers in a climate controlled geotechnical laboratory with a constant temperature of 7ºC. The laboratory tests included all relevant element tests to assess the initial volume change during re-consolidation of the samples to the field effective stress conditions, and the subsequent mechanical response. All tests were performed within two weeks after sampling to reduce additional sample degradation, L’Heureux and Kim (2012).

A sample disturbance factor can be determined from the void ratio change Δe normalised with the in-situ void ratio e₀ as determined from the water
Table 2: Soil characterisation: wet density $\rho$; water content $w$ at Plastic limit, Natural content and Liquid limit; Plasticity Index; Sensitivity $S_t$ and shear strength $\tau_{fu}$ from fall cone tests

<table>
<thead>
<tr>
<th>Depth $(m)$</th>
<th>Sampler</th>
<th>$\rho$ $(t/m^3)$</th>
<th>$w_p$ (%)</th>
<th>$w_N$ (%)</th>
<th>$w_L$ (%)</th>
<th>$PI$ (%)</th>
<th>$S_t$ (-)</th>
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<td>70</td>
<td>60</td>
<td>35</td>
<td>5</td>
<td>3</td>
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</table>

Table 3: Chemical analyses of the soil (milli Molar=mM).

<table>
<thead>
<tr>
<th>Depth $(m)$</th>
<th>Li $(mM)$</th>
<th>Na $(mM)$</th>
<th>NH$_4^+$ $(mM)$</th>
<th>K $(mM)$</th>
<th>Mg $(mM)$</th>
<th>Mn $(mM)$</th>
<th>Ca $(mM)$</th>
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<td>0.0497</td>
<td>0.06</td>
<td>n.a.</td>
<td>0.0086</td>
</tr>
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</table>

Content of the sample (remoulding of the sample will not significantly affect the water content of the sample, which is consequently an ideal measure for
normalisation of void ratio for saturated samples). This sample disturbance factor

\[ sdf = \frac{\Delta e}{e_0} \]  

was first proposed at the Norwegian Geotechnical Institute by Lunne et al. (1997). It is more sensitive than consideration of the volume change alone (for example using the volumetric strain \( \varepsilon_v \)). The vertical effective in-situ stress

\[ \sigma' \]
\( \sigma_{v0}' \) is used as reference stress level in order to maintain a constant reference that can be consistently identified in all test types. The void ratio change \( \Delta e \) is calculated from the volumetric strain increments during consolidation to \( \sigma_{v0}' \) in triaxial and 1D compression tests, including constant rate of strain and incremental loading oedometer tests, a series of undrained and drained triaxial tests in compression and extension have been performed. The aim of the testing programme was to acquire the stiffness (degradation) and the creep properties of the soil in order to be able to obtain reliable parameters for the MAC-s constitutive model by Olsson (2013).

The standard laboratory equipment available at Chalmers, as described in Olsson (2013) has been used for the majority of the tests. The incremental loading oedometer tests were performed with drainage at top and bottom, sample diameter \( D = 50 \) mm, sample height \( h = 20 \) mm. The duration of each loading stage varies between 1-3 days, depending on the magnitude of the stress increment and the monitored slope in the strain – time curve. The sample contained in a teflon ring and manually loaded using dead weight. Automated logging of vertical displacement used a progressively increasing time interval. Constant Rate of Strain (CRS) tests were performed on samples of diameter 50 mm (except one of the block samples which was cut in a 40 mm ring). The sample height was \( h = 20 \) mm. The loading rate of 0.0096 mm/min was applied with an electromechanical actuated load frame. The automated logging of vertical displacement and load were performed with a 120 s interval.
All triaxial tests were performed on samples of diameter $D = 50$ mm and height $h = 100$ mm in a GDS Bishop Wesley cell modified for soft soil testing. Some of the tests used upgraded DC-servo controlled syringe pumps rather than those supplied by GDS. Back pressure was applied at the top of the sample, the volume change required to preserve constant back pressure during consolidation was used in the subsequent analysis of the sample disturbance. In all tests, filter paper side drains were used to speed consolidation. All tests were consolidated to the estimated in-situ effective vertical stress using a value of $K_0 = 0.5 - 0.6$. A series of undrained and drained anisotropically consolidated compression and extension tests were conducted (CAUC, CADC & CAUE). The compression tests were displacement controlled, at displacement rates of 0.01 mm/min (undrained) and 0.001 mm/min (drained), whereas the extension test was stress controlled, increasing cell pressure at 0.83 kPa/min while maintaining constant axial stress. The extension tests required a suction cap on top of the sample. In the test on the block sample from a depth of 7 m the suction cap did not maintain proper contact with the sample during shearing, with consequent inaccuracies in the determination of initial stiffness and volume change.

Test Programme

The laboratory tests performed on the STII and mini-block samples are summarized in Table 4. The main focus of the experimental programme was
the direct comparison of the merits of the two sampling techniques using an advanced constitutive model for soft soils to simulate response of laboratory elements and of a prototype boundary value problem. The number of tests of each type that could be performed in order to provide sufficient data for model calibration was limited by the time constraint of testing within two weeks of sample extraction.

Table 4: Test overview

<table>
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<tr>
<th>Depth (m)</th>
<th>sampler</th>
<th>CRS</th>
<th>IL</th>
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Experimental results

Sample quality criterion

Figure 2 shows the change in void ratio $\Delta e$ during re-consolidation to the in-situ stress level, normalised by the in-situ void ratio $e_0$ (as determined from the water content). The sample quality boundaries proposed by Lunne et al. (1997), based mainly upon experience with Norwegian clays are indicated. The majority of the measured index properties for the Utby clay are close to those quoted for the Onsoy clay, see Lunne et al. (1997) and Lunne et al. (2006), though the natural water content and sensitivity are slightly larger. The quality of the Utby clay samples is reckoned to be acceptable according to this criterion. Swelling of the sample during re-consolidation was observed for two triaxial tests on the mini-block samples and is plotted as a negative value in Figure 2. The values observed in CRS tests are not directly equivalent to ‘re-consolidation’ strains since there is no load control. These values are, however, plotted for comparison. The Norwegian quality bounds indicate that the majority of the Utby samples are of ‘Excellent to very good’ quality. Clearly, the Lunne quality factor is most discerning for incremental loading (IL) oedometer tests, where large differences between the STII and mini-block samples are found even though the reconsolidation schemes are identical.
1D compression testing

The compression response for the CRS and IL tests on STII and mini-block samples is presented in Figures 3a – 3d & Figures 4a – 4c. The constrained modulus $M$ is also plotted for the CRS tests in Figures 3a – 3d to highlight the large differences in the (initial) stiffness. Results for one-dimensional compression of three remoulded samples are presented in Figure 4d for comparison. The values of creep index $C_\alpha$ deduced from the tests in Figures 4a – 4c are presented in Figures 5a – 5d and typical time–strain curves for samples from a depth of 10 m depth are presented in 6a – 6b. The $C_\alpha$ value is evaluated from the tail of the data of each stage of the

![Figure 2: Classification of sample disturbance.](image-url)
tests where a linear trend in the log $t - \varepsilon$ plot is observed.

Figure 3: Comparison of load – compression response for constant rate of strain 1D compression tests on STII and block samples for several sampling depths.
The CRS results indicate that large differences are obtained for the initial stiffness response. The apparent values of preconsolidation pressure, corresponding to the change of slope of the curve load:void ratio, is systematically higher for the mini-block samples. The values of constrained modulus are roughly 50% higher values of $M$ over the relevant strain range of $0 - 2.5\%$ axial strain for all block samples.

For larger axial strains the modulus curves seems to coincide for all samples. The hydraulic conductivity, $K$, evaluated from the CRS was estimated to be about $1 \cdot 10^{-9} \text{ m/s}$ for all the CRS tests at the in-situ effective stress. The conductivity falls with increasing strains – reduction of void ratio following a relationship: $\beta = 3$, where $\beta = \frac{\Delta \log(K)}{\Delta \varepsilon}$.

The IL data support all these conclusions. The data show more pronounced differences than might be expected from the Lunne metric, in Figure 2. However, the interpretation of the results to deduce values of pre-consolidation pressure is somewhat difficult for the STII data. The deduced values of creep index, $C_{\alpha}$, indicate that the drop in creep rate after reaching the pre-consolidation pressure is more pronounced for the mini block samples than for the STII samples. However, the values of creep index for these ‘undisturbed’ samples are still about double the values obtained from the tests on the reconstituted material. These should indicate the intrinsic material properties. We deduce that some structure is remaining after application of the last load step.
Figure 4: Comparison of load – compression response for IL oedometer 1D compression tests on STII and block samples for several sampling depths. Three remoulded samples are included in (d).
Figure 5: Comparison of evaluated creep index $C_\alpha$ as a function of normalised stress level for IL oedometer 1D compression tests on STII and block samples for several sampling depths. Also including results from three remoulded samples.
Figure 6: Time - strain curves for IL oedometer 1D compression tests on STII and block samples for 10 m depth.

**Triaxial testing**

Three sets of plots are presented for the triaxial tests. The first, Figures 7a – 7d, present the deviatoric stress $q = \sigma'_1 - \sigma'_3$ evolution as a function of the axial strain for the (un)drained compression and extension tests on the STII and mini-block samples. The effective stress paths $p' - q$, where $p' = (\sigma'_1 + 2\sigma'_3)/3$ are plotted in Figures 8a – 8d.
Figure 7: Comparison of strain – stress response from triaxial tests on STII and block samples for several sampling depths.
Figure 8: Comparison of stress paths \( p' = \frac{\sigma_1' + 2\sigma_3'}{3} \); \( q = \sigma_1' - \sigma_3' \) from triaxial tests on STII and block samples for several sampling depths.
As expected the largest differences are found for the initial and unload- 
ing/reloading stiffnesses and the peak strength in the range of 0 – 2% axial 
strain. Up to 50% higher (undrained) tangent stiffnesses for the mini-block 
samples are obtained in triaxial compression (Figs. 7a & 7b ). This is 
consistent both with the 1D compression data already presented and with 
previous work on sample quality. The undrained extension tests show only 
small differences between the STII and mini-block samples because the soil 
finds the applied stress path so unexpected that a significant destruction 
of the in-situ fabric of the soil occurs. Therefore, the damage to soil fabric 
during sampling of the lower quality sample, potentially the STII sample, will 
not affect the evolution of stiffness so much.

The effective stress paths for test specimens prepared from the mini- 
block samples (Figs. 8a – 8d ) show a sharper peak in deviator stress, 
but, surprisingly, show similar incremental effective stress response (same 
inclination of the effective stress path) and rather similar eventual undrained 
strength. The drained tests which incorporate an unloading/reloading loop 
(Figs. 7b & 8b) show similar results for the two sampling techniques. However, 
the the STII sample shows a larger reduction of stiffness compared to the 
block sample. Before the unloading-reloading loop the excess pore pressure 
in the drained sample is less than 1 kPa. However, after there loading the 
excess pore pressure increases and is about 6 kPa by the end of the tests. 
The decrease in hydraulic conductivity is larger than anticipated, especially 
for the mini-block sample, leading to undissipated, though small, excess pore
pressures.

Numerical analysis

Constitutive model and parameter set

The constitutive model used for the subsoil in simulations of the long-term settlements of an embankment is the Modified Anisotropic Creep model with structure (MAC-s) described in Appendix A. The MAC-s model Olsson 2013 is based on the CREEP-SCLAY1S model Karstunen et al. (2005) and the n-SAC model Grimstad and Degago 2010. The main difference between the MAC-s and CREEP-SCLAY1S models lies in the greater flexibility provided for the formulation of the normal compression surface.

Parameters for the MAC-s model have been selected from simulations of the laboratory tests on specimens prepared using both of the sampling methods. Table 5 presents parameters that are independent of the sampling method; Tables 6 and 7 present the remaining parameters evaluated from the mini-block and STII piston samples, respectively.

The procedure for evaluating the model parameters is as follows:

1. First the swelling index, $\kappa^*$, and the intrinsic modified compression index, $\lambda_i^*$, are determined from CRS tests using $\ln p' - \epsilon_v$ plot, where $\epsilon_v$ is the volumetric strain.
2. Secondly the intrinsic creep parameter, $r_{si}$, is evaluated from the incremental loading oedometer tests, either from the ‘undisturbed samples’ at very large strains or from tests on remoulded samples.

3. The pre-consolidation pressure is determined from CRS and the undrained compression triaxial tests. Iterations may then be required in order to improve the match with the model simulations as is shown in Figures 9a – 9b. The deduced values for the vertical overconsolidation ratio OCR are given in Tables 6 & 7.

4. The slopes of the critical state line in compression and extension, $M_c$ and $M_e$, are evaluated from the undrained triaxial test data and improved through model simulations with overall test response Figure 10a – 10b and Figure 11a – 11b.

5. The initial inclination of the normal compression surface, $\alpha_0$, and the rotational hardening parameter, $\omega_d$, are determined using the approach described by Olsson (2013) and Karstunen et al. (2005), these parameters are related to $K_{nc}^0$ and $M_c$.

6. The bonding parameter, $\chi_0$, describes the amount of structure in the material and is evaluated from the CRS test and verified against the triaxial compression test. The additional parameters $\xi_v$ and $\xi_d$ are control the rate of degradation.

7. The parameter $m$ controls the shape, slenderness, of the normal com-
pression surface. For normally consolidated clays a value of $m = 0.4$ is recommended, see Olsson (2013).

This procedure results in a very satisfactory fit to experimental data from both sampling techniques, as shown in Figs. 9a – 11b.

Table 5: General model parameters for MAC-s model valid for both sampling methods.

<table>
<thead>
<tr>
<th>Depth $(m)$</th>
<th>$\rho$ $(t/m^3)$</th>
<th>$\nu_{ur}$ (-)</th>
<th>$\lambda_i^*$ (-)</th>
<th>$r_{si}$ (-)</th>
<th>$\chi_0$ (-)</th>
<th>$\omega_v$ (-)</th>
<th>$\tau_{ref}$ $(day)$</th>
<th>$\xi_v$ (-)</th>
<th>$\xi_d$ (-)</th>
<th>$m$ (-)</th>
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<td>200</td>
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<td>8.5</td>
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<td>0.4</td>
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Table 6: Evaluated model parameters for MAC-s model valid for mini-block sampling method.

<table>
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<tr>
<th>Depth $(m)$</th>
<th>$M_e^{Block}$ (-)</th>
<th>$M_o^{Block}$ (-)</th>
<th>$\kappa^{Block}$ (-)</th>
<th>OCR$^{Block}$ $(kPa)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 - 5</td>
<td>1.60</td>
<td>1.1</td>
<td>0.015</td>
<td>1.45</td>
</tr>
<tr>
<td>6 - 40</td>
<td>1.60</td>
<td>1.1</td>
<td>0.015</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Table 7: Evaluated model parameters for MAC-s model valid for STII piston sampling method.

<table>
<thead>
<tr>
<th>Depth $(m)$</th>
<th>$M_e^{STII}$ (-)</th>
<th>$M_o^{STII}$ (-)</th>
<th>$\kappa^{STII}$ (-)</th>
<th>OCR$^{STII}$ (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 - 5</td>
<td>1.55</td>
<td>1.0</td>
<td>0.02</td>
<td>1.25</td>
</tr>
<tr>
<td>6 - 40</td>
<td>1.55</td>
<td>1.0</td>
<td>0.02</td>
<td>1.3</td>
</tr>
</tbody>
</table>
Figure 9: Stress-strain curves of simulations compared with laboratory results of CRS tests at 8m for both mini-block and STII sampler.

Figure 10: Simulation compared with laboratory results of $K_0$-consolidated undrained triaxial test from 6m in terms of stress path for both mini-block and STII sampler.
Figure 11: Simulation compared with laboratory results of $K_0$-consolidated undrained triaxial test from 6m in terms of stress-strain curves for both mini-block and STII sampler

Case study of model comparison

A plane strain finite element model of an idealised prototype has been created using PLAXIS 2D AE. This idealised case is used to demonstrate the effect of different sampling techniques on the predicted response and consists of an embankment 2 m high, 12 m wide, and with 1:3 side slope constructed on a 40 m thick clay deposit. The top 2 m forms a dry crust and the ground water surface is located at this depth. The finite element (FE) model of the clay, shown in Figure 12, has depth 40 m and width 100 m. The finite element model consists of 714 15-noded elements. The boundary conditions for the sides are set to restrict horizontal displacements whilst at the bottom the vertical displacements are fixed. No drainage is permitted across the
centre line of the embankment, which is an axis of symmetry, but all other boundaries permit free drainage.

Model parameters

The foundation soil is divided into three layers (Figure 12). The model parameters for each clay layer are shown in Tables 5 – 7. The initial stress state is computed assuming an earth pressure coefficient, $K_0 = 0.6$, as found from $K_0$-triaxial tests performed on a typical Gothenburg clay (Olsson 2013). The initial hydraulic conductivity for the clay layers has been set to $1 \cdot 10^{-9}$ m/s for the top 10 m and $8 \cdot 10^{-10}$ m/s for the deeper layers, evaluated for the in-situ stress levels using the CRS oedometer test data. (The differences between hydraulic conductivities determined from STII and mini-block samples are reckoned to be negligible). The dry crust and the embankment are modelled using the Mohr-Coulomb material model with model parameters, typical for a Swedish embankment, shown in Table 8, where $E'$ is the Young’s modulus and $\nu'$ is the Poisson’s ratio. The ground water surface is located 2m below the surface, just under the dry crust, and equilibrium pore pressures are assumed to be hydrostatic with depth.

<table>
<thead>
<tr>
<th>Material</th>
<th>$\rho$ (kg/m$^3$)</th>
<th>$E'$ (MPa)</th>
<th>$\nu'$</th>
<th>$K_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embankment</td>
<td>2.0</td>
<td>25</td>
<td>0.3</td>
<td>1</td>
</tr>
<tr>
<td>Dry crust</td>
<td>1.8</td>
<td>7</td>
<td>0.3</td>
<td>0.7</td>
</tr>
</tbody>
</table>

Table 8: Model parameters for the dry crust and the embankment
Results

The results from the simulations are presented in Figure 13 to Figure 17. All horizontal and vertical dimensions have been normalised with the embankment width $W_{emb}$ and height $H_{emb}$, respectively, and displacement components have been normalised with the embankment height $H_{emb}$. In Figure 13 the predicted displacement with time is shown. After 100 years the model using parameters from STII piston predicts about 50% more vertical displacement at the ground surface than the model using parameters from mini-block samples, after 10 years the difference is already $> 25\%$.

It is clear that the STII piston parameters give larger vertical displacements with depth, especially in the upper half of the clay (Figure 14). The normalised horizontal displacements under the toe of the embankment after 100 years of

Figure 12: Problem geometry: soil layers and mesh shown
Figure 13: Normalised vertical displacement w.r.t. embankment height w.r.t. time from the FE simulations at the centre line for both samplers i.e. mini-block and STII piston.

consolidation, presented in Fig. 15, show similar large differences between the two predictions. It is somewhat surprising that even in case of the use of an advanced model that properly captures the laboratory data over a large range the differences are so significant.

The STII parameters indicate a greater area of influence of the vertical displacement in the clay beyond the toe of the embankment (Figure 16). The difference decreases with increasing distance from the embankment and displacements start to coincide at approximately two embankment widths from the centre line. The excess pore pressure response, plotted against time at 10 m depth in Figure 17, shows, as expected, a much slower dissipation of pore pressures for STII parameters than for block sample parameters: for which both OCR and the elastic stiffness are higher.
Figure 14: Normalised vertical displacement w.r.t. embankment height after 100 years from the FE simulations for both samplers i.e. mini-block and STII piston. (CL = at centre line)
Figure 15: Normalised horizontal displacement w.r.t. embankment height at the toe after 100 years from the FE simulations for both samplers i.e. mini-block and STII piston.

Figure 16: Normalised vertical displacement w.r.t. embankment height at ground surface and 6 m depth after 100 years from the FE simulations for both samplers i.e. mini-block and STII piston.
Figure 17: Excess pore pressure at 10 m depth w.r.t. time from the FE simulations at the centre line for both samplers i.e. mini-block and STII piston.

Conclusions

An extensive comparison of the performance of the Swedish 50 mm STII piston sampler and the NTNU mini-block sampler (a miniaturisation of Sherbrooke sampler) has been presented for a Swedish high plasticity soft clay with high sensitivity ($S_t = 20 – 40$). A new holistic approach that incorporates data evaluation across a large range of the non-linear stress-strain response and its effect on prototype geotechnical performance has been presented. Such an approach is a valuable extension to traditional interpretations of sample disturbance, which tend to be limited to comparison only of some key features of the results of element tests, and do not asses the implications of sample quality for the determination of parameters for advanced constitutive models which might be used for simulation and prediction of prototype boundary
value problems.

A traditional interpretation shows that only the results from incremental loading oedometer tests on STII and mini-block samples reflected the Lunne quality metric based on void ratio change during reconsolidation. All data from triaxial tests indicated apparently excellent to very good sample quality even if large differences were observed in the subsequent testing. Even the 1D compression data from the block samples indicated consistently larger values of OCR (> 20%), constrained modulus $M$ (> 50%) as well as more pronounced peak strength and faster decay of creep rate. The triaxial data show a large difference for initial and unloading/reloading stiffness and peak strength, and the block samples indicated higher stiffnesses and strengths than the piston samples. In triaxial extension tests the differences are less pronounced, probably as a result of the significant rearrangement of soil fabric which is encouraged by the extension stress path.

Further interpretation of the test data using the MAC-s constitutive model resulted in two distinct data sets, with model parameters calibrated on either the Swedish piston or the mini-block samples. The model was able to provide an excellent simulation of the laboratory observations for both groups of samples. The effects of the different choices of parameters on the predicted long-term settlements of an embankment on soft soil were used to demonstrate the large differences in the predicted displacements: differences in vertical component at the surface of the clay up to 50% after 100 years; and differences in horizontal component at the toe up to 300% after
100 years. The magnitudes of these differences are larger than might have been
anticipated solely from general comparison of the laboratory experimental
data. Predicted rates of settlement are also very different with the block
sample parameters leading to shorter consolidation times as a result of higher
deduced values of overconsolidation ratio and stiffness. This application
illustrates how relatively small differences in sample quality assessed using
traditional methods and traditional methods of interpretation of laboratory
data may assume a much greater importance when used to calibrate advanced
constitutive models which are to be applied in prototype geotechnical design.
The potential benefit to be obtained from improved predictions of performance
may easily justify the cost of higher quality sampling of the soils.

Acknowledgements

The financial support from Trafikverket in the framework Branschsamverkan
i Grunden and EC/FP7 CREEP PIAG-GA-2011-286397 is greatly acknowled-
ged. Finally, the Authors would acknowledge the support of NTNU in
providing the mini-block sampler and the technicians and NCC for the test site.
JD is supported by a Fellowship grant EC/FP7-MC-IEF 623613 EMPIRC.
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Appendix A

Modified Anisotropic Creep model with structure (MAC-s)

Elastic parameters

The MAC-s model uses Modified Cam-Clay type of elastic model, i.e. the tangent bulk modulus is defined according to eq. (A.1).

\[ K' = \frac{p'}{\kappa} \]  \hspace{1cm} (A.1)

where \( \kappa \) is the slope of swelling line and \( p' \) is the mean effective stress. The shear modulus, \( G \), is calculated from the bulk modulus by assuming a constant Poisson’s ratio, \( \nu \), eq. (A.2).

\[ G = \frac{3 (1 - 2\nu)}{2 (1 + \nu')} \cdot K' \]  \hspace{1cm} (A.2)

Normal compression and plastic potential surface

The normal compression surface, \( F \), in general stress space is defined according to eq. (A.3).

\[ F = \frac{1}{2} (s)^T (s) - p'^2 \cdot \left( 1 - \left( \frac{p'}{p_c} \right)^m \right) \cdot \left( M^2 - \frac{1}{2} \alpha_d^T \alpha_d \right) \]  \hspace{1cm} (A.3)

where \( s = \sigma_d - p' \alpha_d \) and the exponent \( m \) in eq. A.3 controls the shape of normal compression surface. Using the equivalent mean stress, \( p_{eq} \) (i.e. \( p_c \) in
eq. (A.3) will go on the right hand side), the normal compression surface is formulated according to eq. (A.4).

\[
p_{eq} = \frac{p'}{\left[1 - \frac{\frac{1}{2}(\sigma_d - p' \cdot \alpha_d)^T \sigma_d - p' \cdot \alpha_d}{p'^2 (M^2 - \frac{3}{2} \{\alpha_d\}^T \{\alpha_d\})}\right]^{\frac{1}{m}}}
\]  

(A.4)

The plastic potential that is used for this model has the same shape as the CREEP-SCLAY1S model, i.e. the rotated ellipsoid according to eq. (A.5).

\[
p_{eq}^P = p' + \frac{3}{2} \frac{\{\sigma_d - p' \cdot \alpha_d\}^T \{\sigma_d - p' \cdot \alpha_d\}}{p' (M^2 - \frac{3}{2} \{\alpha_d\}^T \{\alpha_d\})}
\]  

(A.5)

A visualisation of the new normal compression surface, NCS, and the plastic potential surface in \(p'\)-q stress space together with the influence of different \(\alpha_0\) and \(m\) values is displayed in Figure A.1.

The MAC-s model makes use of the visco-plastic multiplier see eq. (A.6)

\[
\dot{\lambda} = \frac{1}{r_{si} \cdot \tau} \cdot \left(\frac{p_{eq}^{eq}}{1 + \chi} \cdot p'_{ci}\right)^{r_{si} \cdot (\lambda^t - \kappa^t)} \cdot \frac{M_c^2 - \alpha_0^2}{M_c^2 - \eta_0^2}
\]  

(A.6)

where \(M_c\) is the critical state line in compression and \(\alpha_0\), is the rotation of the potential surface in K0NC loading and \(\eta_0\) is the stress ratio corresponding to \(K_0^nc\) loading path. The MAC-s model also incorporates structure or bonding, as described for the S-CLAY1S model.

**Hardenings laws**

The MAC-s model incorporates three different hardening laws. The hardening laws are similar to the CREEP-SCLAY1S model. The first hardening law
Figure A.1: (a) the NCS (solid line) of the MAC-s model with $m = 0.4$ compared with plastic potential surface (dotted line) in p’-q stress space together with the failure criteria and the $\alpha_0$-line. (b) the effect of $m$ on shape of NCS.

controls the change in the size of the intrinsic reference surface with respect to the volumetric creep strains ($\varepsilon^c_v$), similar to that of Modified Cam-Clay, according to eq.(A.7)

$$p_{ci} = p_{ci0} \exp \left( \frac{\varepsilon^c_v}{\lambda^*_i - \kappa^*} \right)$$

(A.7)

where $\lambda^*_i$ and $\kappa^*$ are the modified compression index and modified swelling index respectively, evaluated from a stress-strain plot. The parameter $p_{ci0}$ is the initial pre-consolidation pressure at the hydrostatic axis and the subscript “i” denotes the intrinsic parameter.

The second hardening formulates the rotation of the reference surface, i.e.
the evolution of the anisotropy, due to creep strains similarly to eq. (A.8).

\[ d\alpha_d = \left(\omega_v \left[\frac{3\eta}{4} - \alpha_d\right] \cdot \langle d\varepsilon_v^c \rangle + \omega_d^* \left[\frac{\eta}{3} - \alpha_d\right] \cdot d\varepsilon_d^c \right) \]  

(A.8)

where \( \eta \) the generalised stress ratio, defined as \( \eta = \sigma_d/p' \), and \( \varepsilon_d^c \) is the deviatoric creep strain.

The third hardening law describes the degradation of bonding in the soil and is defined by an intrinsic reference surface. The intrinsic reference surface has the same shape and inclination, but differs in size with respect to the normal compression surface. The size of the intrinsic reference surface is related to the normal compression surface by a parameter \( \chi \) which determines the current degree of bonding according to eq. (A.9).

\[ p_c = (1 + \chi) \cdot p_{ci} \]  

(A.9)

where \( p_{ci} \) is the intrinsic pre-consolidation pressure at the hydrostatic axis.

The degradation of bonding is associated with volumetric and deviatoric creep strains and formulated according to eq. (A.10).

\[ d\chi = -\chi \left(\xi_v |d\varepsilon_v^c| + \xi_d |d\varepsilon_d^c|\right) \]  

(A.10)

where \( \xi_v \) and \( \xi_d \) are additional model constants controlling the rate of degradation.