Evaluation of soil parameters for creep calculations of field cases

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ABSTRACT
Prediction of settlements in-situ are usually establish based on soil data extracted from laboratory tests. Hence it is crucial that the laboratory samples have the desired quality to give acceptable prediction of field cases. It is also vital that soil parameters are interpreted with special emphasis on the numerical model to be used. Hence, one needs to assess the underlying assumptions of a model while interpreting the model parameters. This important consideration should also take into account sample quality. In this study, key parameters governing creep are discussed in light of sample quality and are further illustrated from numerical analyses perspective. An application example is also presented based on a well-documented test fill from Sweden. The test fill, Väsby test fill, was designed and constructed by the Swedish Geotechnical Institute (SGI) in 1947 and is still monitored. Laboratory tests on samples extracted with 50 mm diameter Swedish sampler and 200 mm diameter Laval sampler are used as a basis for the study. These laboratory samples show a distinct difference in sample quality. Based on these samples, effect of sample disturbance on the pre-consolidation stress and soil destructurations are numerically illustrated in light of an isotache based creep model that is available in a finite element code Plaxis. A representative parameter data set is established for the two laboratory tests and then used for analysis of the Väsby test fill. This work shows that sample disturbance play a key role in settlement analysis of clay soils. However, with reasonable simplifications and understanding the implication of key parameters, a relatively simple creep model can capture the overall performance of a field case. The work also gives advice and recommendations as to how one should optimize use of available data along with an available numerical tool.

Keywords: Settlement, consolidation, creep, sample disturbance, parameter interpretation.

1 BACKGROUND AND DEFINITIONS

Consolidation and creep of clayey soils is an extensively studied topic in soil mechanics. Significant advancements have occurred since the formal inception of the classical theory of consolidation for soils (Terzaghi, 1923). The theory disregards time dependent deformations during consolidations. Thus it was already discovered as early as 1936 that the theory cannot capture observed field measurements due to existence of continued deformations after consolidation was finished (Buisman, 1936). The immediate modifications that followed were to divide the total deformations into the primary and secondary consolidation phases where the primary consolidation phase is computed according to the classical consolidation theory and a creep deformation is added afterwards as a secondary consolidation phase. However, shortcoming of such formulation was recognized by earlier researchers that indicated existence of creep during primary consolidation (e.g. e.g. Šuklje 1957; Bjerrum 1967; Janbu 1969). These assertions are substantiated with extensive long-term field measurements that clearly evidenced existence of creep during Primary consolidation (e.g. Larsson, 2003; Leroueil 2006).

In spite of the aforementioned points, the current practice is widely based on calculating settlement by disregarding creep during primary consolidation phase and adding it later if deemed necessary. The reason for still using such approaches could be the lasting effect of earlier works on consolidation which has an attribute of simplicity for teaching students. This is later carried out for use in the practice. However, it is important to benefit of existing research and level of understanding achieved through extensive works. This also calls for clarifying and re-defining fundamental concepts adhering to common terminologies. Therefore, the deformation of a saturated soil layer under loading can still be considered to consist of two successive phases, namely primary and secondary consolidation. During primary consolidation phase, soil deformation is accompanied by significant excess pore pressure and changes in effective stresses. Whereas during secondary consolidation, the soil continues to deform under approximately constant effective stresses. In a pragmatic sense this can simply be understood as primary consolidation consists of deformations due to effective stress increase and creep while secondary consolidation consists of creep deformation only. This also means that creep starts at the same time with the primary consolidation phase and is thus existent throughout the whole deformation phase. This work builds upon this fundamental understanding and definition of terminologies.

2 ON RATE DEPENDANCY OF NATURAL CLAYS

Soft clay deposits especially soft marine clays are characterized by their strong tendency to undergo significant creep deformation (high rate dependency), distructuration and anisotropy (see e.g. Šuklje 1957; Bjerrum 1967, Burland 1990). For detailed treatment of interplay between these features a reference is made to Grimstad et al. (2010) and Grimstad and Degago (2010). However, within the framework of this work an emphasis is given on elaborating the implication of rate dependency of natural clays in a simplified way and in relation to its practical aspects.

In practice deformations of such soft clay deposits is usually studied based on soil data obtained from laboratory tests. However, the time required to complete primary consolidation is significantly different for a laboratory specimen and an in-situ soil layer. For a laboratory specimen, the primary consolidation could last in the order of minutes; whereas, it could go on for several decades in thick soft clay deposits (Larsson & Mattson, 2003; Leroueil, 2006). The variations in consolidation periods give rise to different strain rates. The dependency of clays on strain rate implies that the resulting effective stress – strain relationship of clays would be a function of strain rate. An important implication of this is that the experienced pre-consolidation stress would be rate dependent. This, referred to as isotache concept (Šuklje 1957), is sketched in Figure 1 by considering two cases with similar incremental loadings up to end of primary (EOP) state. The concept depicted in Figure 1is meant to give a general principle, i.e. fast vs. slow consolidation duration can be understood as either laboratory condition vs. field condition; or a laboratory test conducted at fast rate versus one conducted at a slower test.

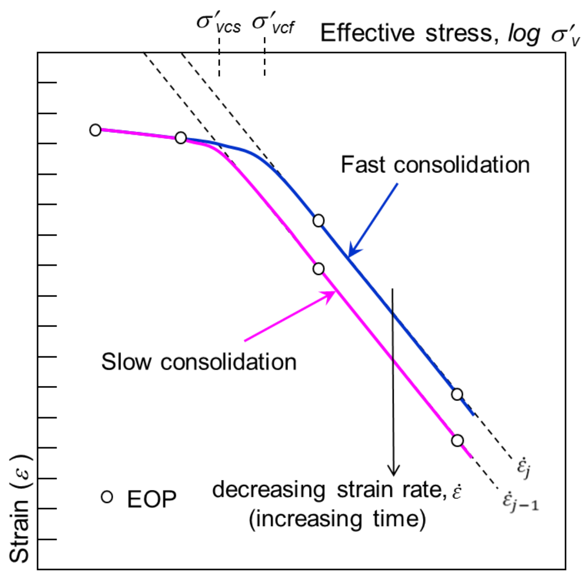

Figure 1. Sketch of effective stress vs. EOP strain relationships under similar load increment but different strain rates

3 ON SIGNIFICANCE OF SAMPLE QUALITY FOR CALCULATIONS

Numerical calculation of settlements of field conditions are normally based on soil parameters interpreted from laboratory tests.

Hence, it is crucial that the laboratory samples have the desired level of quality and be representative to give acceptable prediction of field performances.

In Scandinavia, one typically finds clays in normally consolidated state, with only an apparent over consolidation ratio (OCR) due to aging (Bjerrum 1967). Determination of this OCR is significantly affected by sample disturbance and its correct determination crucially lies on the sample quality (Leroueil & Kabbaj 1987, DeGroot et al. 2005). Different quality of the sample will also give different compressibility with respect to stress and time actions.

Sample disturbance is hardly avoidable in any soil sample extracted from in-situ. However, the degree of disturbance varies and ideally one aims to acquire a sample with highest quality. Still, it is common to encounter situations where field settlement analysis has to be based on samples of low quality. In such instances, it is important to study the implications of using parameters derived from samples of low quality. Hence, in this work such aspects are discussed from the numerical analyses perspective. Laboratory samples depicting effect of sample disturbance are used as a basis for various analyses. In addition, an application example is also presented based on a well-documented test fill from Sweden, i.e. the Väsby test fill (Larsson & Mattsson 2003). This work builds upon an earlier work by Degago & Grimstad (2014) with further extensions and enhancements along with significant elaboration.

4 SOME MODELLING ASPECTS OF NATURAL CLAYS

Constitutive models for clays that are used today are often based on the modified Cam-Clay model (MCCM) (Roscoe & Burland 1968). The MCCM was originally developed to model simple elasto-plastic behaviour of soils under triaxial stress-strain conditions. Various modifications and extensions are applied to the MCCM to account for various features of clays such as anisotropy, destructuration and creep. Typical modifications of the MCCM include:

(1) rotating the yield/reference surface to account for anisotropy (Dafalias 1986),
(2) accounting for an unstable structure by associating the yield/reference surface with a destructuration formulation proposed by Gens & Nova (1993) and
(3) modelling creep and rate dependency by controlling the size of the reference surface using concepts developed by, e.g. Šuklje (1957), Perzyna (1963) or Janbu (1963).

Common for the extensions to the MCCM is the increased complexity that these features bring with them and the need for extra soil parameters, which may require special laboratory tests. Hence, it is essential to consider the benefits of using advanced models compared to simple models. It is also vital to identify cases in which proper use of a simple model could give a better understanding of the problem than using a more advanced model. In this way, one can focus on certain selected aspects of soils and grasp the overall picture of the resulting soil responses. Accordingly, such approach has been used in this study. It is also important to consider that the current practice in the geotechnical consulting industry does not generally use advanced constitutive models that incorporate creep. The state-of-the practice is more inclined towards ignoring creep effect, dividing into primary consolidation settlement (without creep) and secondary consolidation settlements (only creep) or use of a simple isotropic or 1D creep model.

5 PROBLEM IDEALIZATION

Numerical simulation of a real case normally involves certain appropriate idealizations. Accordingly, when reducing a real problem into a numerical idealization that can readily be analysed, it is vital that soil parameters are interpreted with special emphasis on the numerical model and its underlying assumptions as well as the nature of the problem to be dealt with such as the expected stress range and associated time considerations.

This work focuses on settlement analyses of field cases with respect to sample quality.

The subject of the study is essentially on clays with significant potential to undergo creep deformations. In addition, such clays usually exhibit anisotropic behaviour and an unstable structure. For this demonstration, an elasto-viscoplastic soil model available in the FE code Plaxis is selected. This model is referred to as the Soft Soil Creep (SSC) model (Stolle 1999). The SSC is an extension of MCCM, by including the isotache concept (Šuklje 1957). In addition, the model is isotropic and does not take into account the effect of destructuration. However, for the test fill considered in this work, an isotropic formulation is considered sufficient and the effect of destructuration is investigated based on assessing effect of various combination of the input parameters. By keeping anisotropy and destructuration out of the picture, this work focuses on the creep parameters and their implications, based on the isotache formulation as adopted in the SSC model. It is worth mentioning that for problems that are not dominated by volumetric creep, the creep extension adopted in the SSC model is not preferable. Details of a different approach of extending a 1D creep model to 3D creep model is given in e.g. Grimstad et al. (2010).

One other aspect of idealization of settlement calculation is the often used assumption of small deformations. This might, in many cases, be of minor importance. However, when large settlements occur, neglecting the chance in buoyancy will lead to higher value of the calculated settlements as the effective stress change is not reduced in accordance with the increasing settlements. In Degago et al. (2011b) buoyancy and sample disturbance effects are shown to have significant influence on the calculated settlements of the Väsby test fill.

6 KEY PARAMETERS GOVERNING CREEP

To properly calculate the development of settlement and pore pressure histories, a fully coupled analysis with a creep constitutive model for the soil must be used. By observing creep behaviour, in laboratory and field tests, some mathematical equations that fits the observed data can be established. To start with, the isotache concept is quite appealing as it is simple and its parameters are easily determined from standard laboratory tests. Demonstration of how well laboratory tests can be simulated with use of an isotache type of model can be found in e.g. Degago et al. (2011a). Creep stain rate as a function of time, as defined in the isotache concept (Šuklje 1957), is given in Equation 1.

$$\dot{\varepsilon} = \frac{\mu^*}{t} \qquad (1)$$

where $\dot{\varepsilon}$ = the strain rate; μ^* = the modified creep parameter; and t = time.

The isotache concept furthermore uniquely relates effective stress state (p^{eq}), reference pre-consolidation stress (p_0^{eq}) and volumetric creep strain rate ($\dot{\varepsilon}_v^{vp}$). This relationship, as used in the SSC model, is given in Equation 2.

$$\dot{\varepsilon}_v^{vp} = \frac{\mu^*}{\tau} \cdot \left(\frac{p^{eq}}{p_0^{eq}} \right)^{\frac{\lambda^* - \kappa^*}{\mu^*}} \qquad (2)$$

where λ^* and κ^* = the modified compression indexes for virgin compression and recompression line respectively; τ = a reference time corresponding to the specified OCR or (p_o^{eq}/p^{eq}). Typical value of τ is one day as standard incremental oedometer tests are ran with one day increments.

As can be seen from Equation 2, in addition to μ^*, the ratio $(\lambda^* - \kappa^*)/\mu^*$ and OCR are very important for the calculated strain rate. Combining Equation 1 and Equation 2, one could find an expression for the age of the clay, t_{age}, based on OCR (for the same K_o^{NC}-line, K_o^{NC} is the coefficient of earth pressure under virgin loading) corresponding to a reference time τ, as

$$t_{age} = \tau \cdot OCR^{\frac{\lambda^* - \kappa^*}{\mu^*}} \qquad (3)$$

For a certain $(\lambda^* - \kappa^*)/\mu^*$, Equation 3 can be used to estimate either the age of a clay implied when an OCR corresponding to τ is known; or, an OCR corresponding to a certain reference time τ when the age of the clay, t_{age}, is known. This implies that one must assume that the ratio $(\lambda^* - \kappa^*)/\mu^*$ is constant in time.

For example, Waterman & Broere (2005) suggests values of $(\lambda^* - \kappa^*)/\mu^*$ to be in the range 5 (“for soils with considerable amount of creep”) to 25 (“for soils with little creep”). This implies that a standard one day incremental oedometer test conducted on a sample taken from a 500 year old clay would give OCR = 1.6 or 11.3 for $(\lambda^* - \kappa^*)/\mu^* = 25$ or 5 respectively. Accordingly, soils with considerable amount of creep are expected to show OCR as high as 11 which seem unlikely. A more practical range for $(\lambda^* - \kappa^*)/\mu^*$ would be 15–50. Furthermore Waterman & Broere (2005) suggest that for $(\lambda^* - \kappa^*)/\mu^* > 25$ that creep could be ignored. This might be correct in some cases where the applied stress increase gives a final stress situation well above the initial pre-consolidation stress. However, in many cases the stress increase is rather moderate and the new situation is around the pre-consolidation stress, making the contribution from creep more significant, regardless of the ratio. For OCR = 1.0 the creep rate becomes independent of the ratio and the actual value for μ^* is more important (see Eq. 2).

In situations where the main interest is the final settlement after a certain period of time, a time independent elasto-plastic model can also be used to calculate long-term settlements. This is done by selecting a single isotache that in average meets the final expected combination of strain and stress. An isotache selected in this way would typically yield a lower OCR and lower λ^*. Equation 4 shows the corrected over consolidation ratio, OCR_{corr}, which should be used in an elasto-plastic analysis when the true undisturbed OCR is known.

$$OCR_{corr} = \left(\frac{\tau}{t_{age}}\right)^{\frac{\mu^*}{\lambda^* - \kappa^*}} \cdot OCR \qquad (4)$$

This principle is sketched in Figure 2 with an attempt to further clarification. The elasto-plastic approach would be the path A-B-D with the elastic part shown by path A-B and the plastic part given by B-C. The path A-C-D gives an elasto-viscoplastic modelling aspect where the elastic part is given by path A-C and the creep part is given by path C-D. The path followed by A-C-D and A-B-D will give similar final settlement.

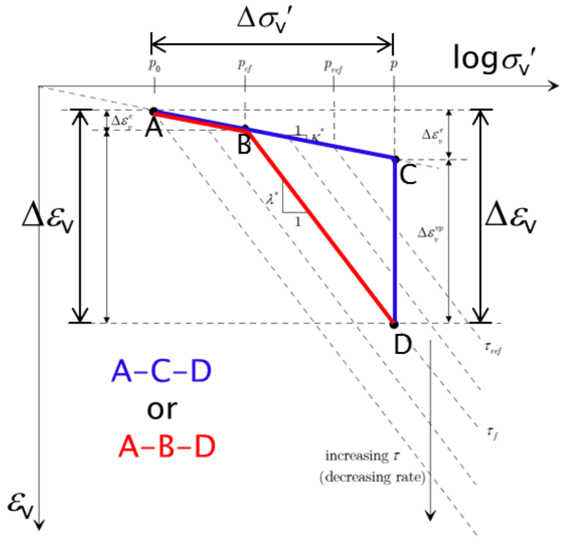

Figure 2. A case of an isotoche formulation and an elastoplastic giving similar final settlements.

The concept sketched in Figure 2 is numerically illustrated for Väsby test fill where exclusion of creep and buoyancy effects gave excellent settlement prediction, as shown in

Figure 3, by adopting models that disregarded creep, i.e. ILLICON and a time independent elasto-plastic model (the Soft Soil model in Plaxis). A detailed treatment of the case of using low OCR (OCR_{corr}) with an elasto-plastic model to counterbalance effect of creep can be found in Degago (2011).

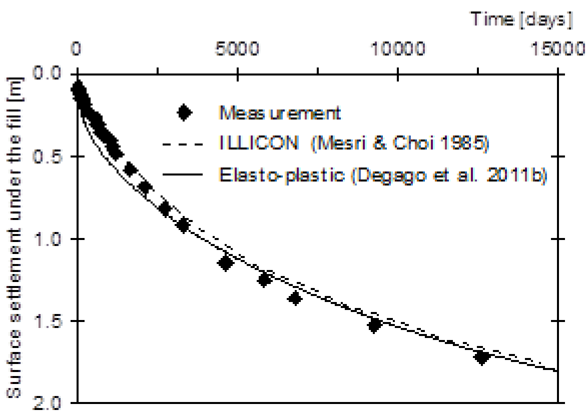

Figure 3. Settlement analyses of the Väsby test fill by ignoring creep and buoyancy effects (after Degago et al. 2011b).

By coincidence such approach, Equation 4 or Figure 2, changes soil parameters in a similar way typical to sample disturbance. This is one of *the main reasons for explaining why some researchers (e.g. Mesri & Choi 1985) have been successful in predicting long-term settlements (*

Figure 3) by disregarding creep contribution despite the soil showing a significant effect of creep (Leroueil & Kabbaj 1987, Degago et al. 2011b).

7 THE VÄSBY TEST FILL

The Väsby test fill was designed and constructed by the Swedish Geotechnical Institute (SGI) in 1947. The test fill has been monitored with extensive instrumentations and there exists a detailed documentation of measurements (Larsson & Mattsson 2003). The Väsby test fill consists of a 30 m x 30 m square and 2.5 m high gravel fill constructed within 25 days. The applied total stress due to the fill was 40.6 kPa. The test fill site consists of soft sediments of glacial and post-glacial origin. The soft soil layer under the fill is 14 m thick. The ground water table, which is located at an average depth of 1 m beneath the original terrain, has a hydrostatic pressure distribution. Surface and sub-surface settlements are continuously measured since the construction of the fill.

In order to investigate the effect of sample disturbance, Leroueil & Kabbaj (1987) conducted incremental oedometer tests on the Väsby clay. They took samples with 200 mm Laval sampler and compared it with results from the 50 mm Swedish sampler (conducted in 1967). Even though the samples were extracted from a similar depth of ca. 4.2 m, they show significant effect of sample disturbance. A distinct feature of sample disturbance was that it resulted in lower OCR and λ^*. In Figure 4, the two different laboratory curves are compared. The OCR for the 50 mm is estimated to be 1.1 and for the 200 mm to be 1.6.

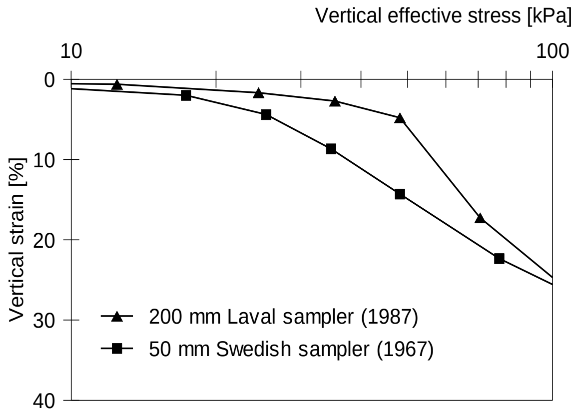

Figure 4. Laboratory incremental oedeometer results on samples taken at 4.2 m depth using two different samplers (after Leroueil & Kabbaj 1987).

8 NUMERICAL ILLUSTRATIONS

Some of the creep aspects highlighted in earlier sections are numerically illustrated based on various parameter sets. These data sets are mainly established as variations of parameters interpreted from oedometer tests of Väsby clay (Figure 4) and are given in Table 1 & 2. One of the soil parameters that can easily be affected by sample disturbance is the OCR. Hence, a distinct variation among the data sets is their OCR and this is used to systematically group the data sets as presented in the two tables. Data set 1, 2 and 3 are characterized by low OCR (= 1.1) as interpreted from the Swedish 50 mm tube sampler, and set 4 and 5 are characterized by high OCR (= 1.6) as interpreted from the Laval 200 mm block sampler. In set 6, an even higher OCR (2.18) is considered based on clay age considerations. A constant average permeability of 4.0e-5 m/day is adopted for all data sets.

Table 1. Data sets with low OCR (OCR = 1.10).

Parameter, Symbol [unit]	Analysis sets		
	Set 1	Set 2	Set 3
κ^* [-]	0.030	0.030	0.030
λ^* [-]	0.191	0.191	0.357
μ^* [-]	0.011	0.021	0.011
OCR [-]	1.10	1.10	1.10
$(\lambda^* - \kappa^*)/\mu^*$[-]	14.6	7.7	29.7
$\dot{\varepsilon}$ [%yr^{-1}]	99.51	369.1	23.62
t_{age} [yr]	0.011	0.006	0.047

where κ^* is the modified swelling index; λ^* is Modified compression index; μ^* is Modified creep index; $\dot{\varepsilon}$ is initial creep rate; t_{age} is the implied " Age of clay"

Table 2. Data sets with high OCR (OCR = 1.60 and 2.18).

Parameter, Symbol [unit]	Analysis sets		
	Set 4	Set 5	Set 6
κ^* [-]	0.030	0.030	0.030
λ^* [-]	0.357	0.571	0.357
μ^* [-]	0.021	0.021	0.021
OCR [-]	1.60	1.60	2.18
$(\lambda^* - \kappa^*)/\mu^*$[-]	15.6	25.8	15.6
$\dot{\varepsilon}$ [%yr^{-1}]	508.2	0.004	0.004
t_{age} [yr]	4.13	497	500

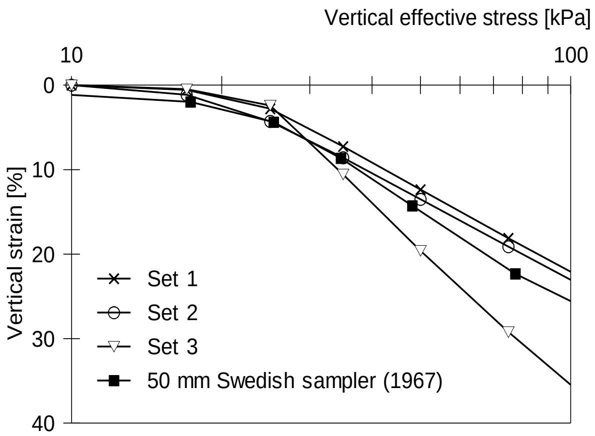

Figure 5. Oedometer simulation using data set 1, 2 and 3 (Table 1) with the test result obtained from 50 mm tube sampler.

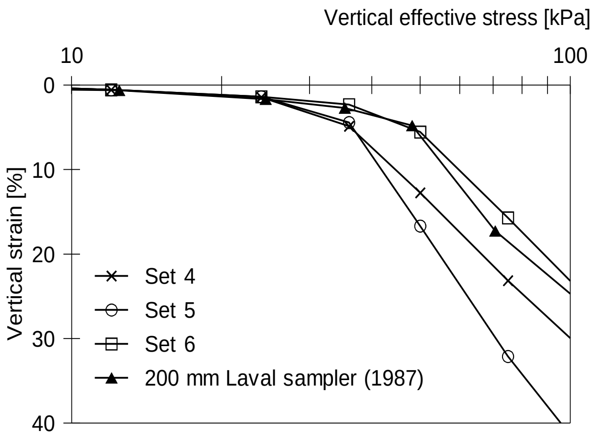

Figure 6. Oedometer simulation using data set 4, 5 and 6 (Table 2) with test result obtained from 200 mm Laval sampler.

8.1 Simulation of laboratory tests

In Figure *5*, results from laboratory oedometer test simulations corresponding to set 1, 2 and 3, as given in Table 1, are presented along with the experimental results obtained from the 50 mm Swedish sampler. Correspondingly, Figure *6* gives numerical results of set 4, 5 and 6 (Table 2) along with the test data obtained from the 200 mm Laval sampler. The maximum possible stress increase that can be obtained in the field is about 40 kPa, which means that the results above 70 kPa are of little interest. Furthermore, due to buoyancy and stress distribution with depth, this value will be reduced to less than 50 kPa (Degago et al. 2011b). This must be kept in mind while evaluating laboratory simulation results with respect to laboratory measurements (Figure *5*-Figure *6*). Some researchers have reported constant μ^*/λ^* ratios for wide range of soft clays (Mesri & Godlewski 1977). However, for models that assume a linear relationship between strain and log stress (constant λ^*), like SSC model, this will imply a constant μ^*. In reality both λ^* and μ^* are stress dependent (due to destructuration effect) and the ratio must be evaluated over the actual stress range. Hence, a constant value is not expected unless the changes are small. Therefore adjustments to the μ^*/λ^* ratio can be justified.

For set 1, λ^* was interpreted from the 50 mm test data and $\mu^* = 0.06\lambda^*$ was used from literature (Mesri & Choi 1985). Set 2 and set 3 are, respectively, μ^* and λ^* variations of set 1. Accordingly, set 2 fits the 50 mm test data best with set 1 slightly underpredicting and set 3 significantly overpredicting deformations as observed in the test (Figure *5*). It is worthwhile to notice that set 2 has a higher creep potential (higher μ^*) while set 3 is significantly softer (higher λ^*). However, $(\lambda^* - \kappa^*)/\mu^*$ is higher for set 3 (with a lower μ^*), implying that creep has relatively less importance.

For set 4, λ^* was interpreted from the 200 mm test data and $\mu^* = 0.06$ was adopted. Set 5 and set 6 are variations of set 4. The λ^* value for set 5 is chosen such that for an OCR of 1.6, a clay age of $t_{age} \approx 500$ years is obtained (Eq. 3). In set 6 the OCR is increased such that the age of the clay is t_{age} = 500 years. From Figure *6*, set 6 underpredicts the deformations, while both set 4 and 5 overpredict deformation with set 5 giving the highest overprediction of the 200 mm test data.

8.2 Simulations of the Väsby fill

The soil parameter data sets given in Table 1 & 2 are adopted in the simulations of the Väsby fill. An axisymmetric model consisting of a very fine mesh was used in the analysis. In addition, an updated-mesh and updated-water pressure procedure were adopted to account for the effect of large deformations and buoyancy effect respectively. The computed vertical deformation patters are shown in Figur xx by selecting results for parameter set 2 and 5. Figure 8 gives 57 years of measured surface settlements, below the centre of the fill, along with the corresponding numerical analysis results.

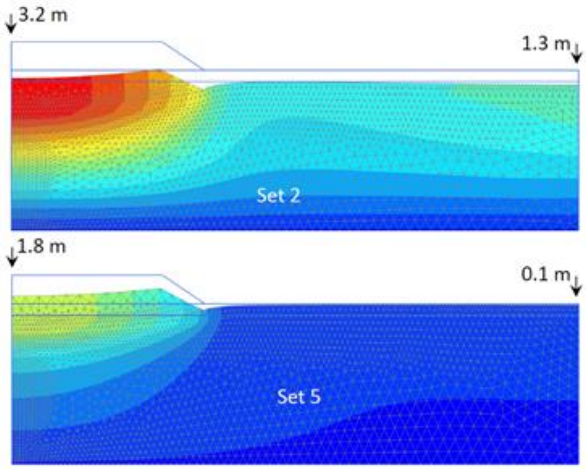

Figure 7. Vertical deformation patterns for two parameter sets inreference to initial geoemtry (arros indicate vertical deformations at 50 m a distance apart)

From data sets with low OCR (Table 1), it is interesting to note that parameter set 1 and 3 only slightly overpredicted the measured settlements despite their low OCR values. Set 2 significantly overpredicted the settlements due to the implied highest initial creep rate of all the other data sets. Set 3 gave the best field prediction even though it gave significantly softer response in the oedometer simulation (
Figure 5). This is because the initial creep rate for set 3 is lower than for set 1 and 2. The ratio $(\lambda^* - \kappa^*)/\mu^*$ is highest for set 3 which means that creep is less important in the far field deformations and for situations where the pre-consolidation stress is exceeded. From data sets with high OCR (Table 2), set 4 also gave excellent prediction while 5 and 6 under predicts the surface settlements. The underprediction by set 6 is small compared to the significantly stiffer response observed in the corresponding oedometer simulations (
Figure 6).

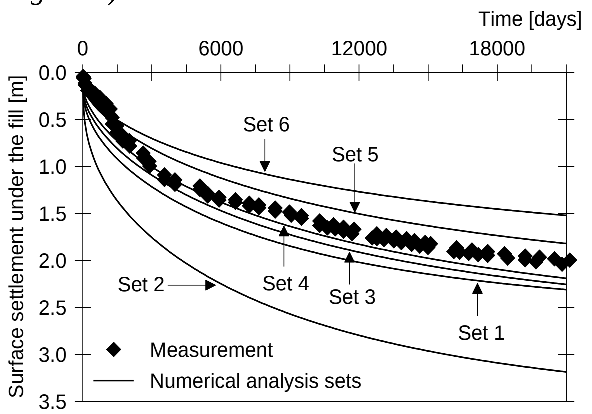

Figure 8. Measured and simulated surface settlement histories below the centre of the fill

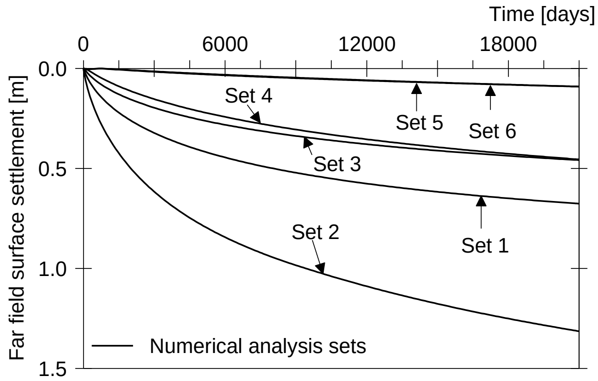

Figure 9. Simulated far field surface settlement histories

It is also important to evaluate far field settlements as it provides a benchmark for controlling analyses results and parameters adopted for the analysis. This would also help to evaluate the contribution of the settlement that is coming from the actual fill load and settlement resulting due to pure creep. Hence, far field settlement, indirectly, reveals how much part of the total settlement under the fill is actually due to the fill. In reality, the far field settlements are relatively insignificant.

Prediction of such far field settlements can only realistically be achieved if age considerations are taken into account in the selection of the parameters. The far field surface settlements, 50 m from the centreline, as implied by the analyses sets are given in Figure 9. Set 1, 2, 3 and 4 gave significant far field settlements with set 2 giving the highest settlement. This means that a significant part of the settlement obtained using these sets (Figure 8) is actually not related to the fill. Set 5 and 6 gave the lowest and most realistic far field settlements that represent a case of pure creep settlement in a flat natural terrain (Figure 9). This was achieved due to the reasonable age considerations implied by the data sets 5 and 6. Overall, based on Figure 8 & Figure 9, parameter set 5 gave the best surface settlements predictions.

9 FINAL REMARKS AND CONCLUSIONS

Good prediction of long-term settlements of embankments heavily relies on laboratory test results from good sample quality. Hence, it is highly essential that soil parameters for creep analysis are interpreted from samples of high quality. However, in reality, sample disturbance is virtually unavoidable and laboratory samples suffer to a varying degree of sample disturbance.

In this work, the effect of sample quality, by selecting different data sets, for the SSC model, was studied based on laboratory and field measurements. The calculations show that good back calculations of the surface settlement at the centre of the embankment can be made with various sets of parameters. However, the far field settlement can only be realistic when the age of the clay is taken into consideration. A correct combination of the $(\lambda^* - \kappa^*)/\mu^*$ ratio and OCR is important for an overall sound creep settlement analyses. This means that for realistic ratios of $(\lambda^* - \kappa^*)/\mu^*$, valid for the experienced (actual) stress range, satisfactory far field settlement predictions can be justified at the same time as the settlement below a fill is satisfactory predicted.

It is vital to understand the role of parameters using simple models before resorting to advanced models. An example of such advanced model that includes creep, anisotropy and destructuration is the n-SAC model (Grimstad & Degago 2010). Such a model puts even more demand on the sample quality for calibration of input parameters. In fact, with proper use, a simple model like the SSC model can give successful predictions and give the user control on certain key input parameters. However, to further improve predictions additional aspects of natural clays such as anisotropy and destructuration should be accounted for.

10 OUTLOOK

In order to enhance models for settlement analyses, a series of research projects (e.g. the CREEP and Geofuture projects) have recently been initiated in Norway. An EU CREEP project led by the Norwegian University of Science and Technology (NTNU) has been active for the last four years (2011-2015). An RCN GeoFuture project that is led by the Norwegian Geotechnical Institute (NGI) is still active. The GeoFuture project is financed by the Research Council of Norway (RCN) and the industry in Norway. NTNU and the Norwegian Public Roads Administration are working among others work on this task.

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