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From soft soil modelling to engineering application

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Abstract. Soft soil engineering is still a challenge, in particular when we increasingly need to construct on densely populated urban areas next to existing structures, and need to deal with the effects of climate change. The paper discusses the systematic research by the author and her co-workers in the last 20 years that has resulted in the development and validation of advanced soil models, specifically geared for Scandinavian soft soil conditions. The culmination of this research is Creep-SCLAY1S, a rate dependent anisotropic model, which is backed by hierarchical development and systematic validation. Some recent examples of engineering applications where the model was used are are highlighted, in addition to the parameter determination and calibration.

1. Introduction

People have been successfully building on soft soils for thousands of years. However, we are now facing new challenges. As a consequence of the global trend of urbanisation, people are moving in mass to cities. Thus, more and more construction is taking place in densely populated urban areas, increasingly on poor quality marginal soils. The new constructions interact with existing structures and buildings, some of which may have high cultural and historic value. The construction activities can also negatively affect the infrastructure and facilities that the modern society relies on, both in short- and long-term. Reliable transport links within cities and in between the cities are a necessity, as the driver for prosperity is the secure and reliable flow of people, goods and services. Consequently, our roads and railways are facing increasing level of utilisation, and there is a pressure to allow ever higher higher axle loads. In urban context, the limitations on the physical space causes congestion, stress and pollution. To mitigate these effects, we increasingly plan to construct underground.

A new era, with increased environmental pressure and intensified usage of the urban areas and their infrastructure is starting. The forthcoming introduction of autonomous vehicles and the rapid increase in electric vehicles will introduce new challenges for the physical infrastructure in terms of e.g. use of space, view of sight, separation of transportation modes and fire security. As a result of the climate change, we are expected to experience an increase in the periods of intense precipitation and drought, temperature fluctuations as well as more extreme drying-wetting cycles. In particular in the Nordic countries, increases in the annual number of freezing-thawing cycles [1] and soil cracking due to drying, already cause accumulating problems with maintenance. An increase in the annual average temperature will also permanently affect the underground. As a consequence of all above, we no longer can base geotechnical design on simplified semi-empirical methods which have worked until now, as we no longer have the

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conditions for which we have experience for. Thus, We need to rely increasingly on advanced numerical modelling that enable forecasting for future, with model based on the principles of relevant physics.

The geotechnical design must consider both the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS). Increasingly, especially in the urban areas, the design is controlled by SLS considerations. This is particularly true when constructing on soft soils. In SLS design, 1D analyses are no longer sufficient. It is necessary to make accurate predictions for both the short term and long-term response of geotechnical structures, integrating rigorous Observational Method [2] during construction period with strategic infrastructure asset management during the use [3].

Both qualitatively and quantitatively, the results of any geotechnical numerical analyses depend on the constitutive model used to represent the soil behaviour, as well as the quality of soil sampling and testing from which the input has been derived. Above all, the results rely on the experience and the ability of the geotechnical engineer in choosing a representative soil model and deriving representative values for the relevant state and model parameters. Geotechnical numerical analyses are often performed using commercial finite element (FE) codes that offer a number of constitutive models, historically developed on testing laboratory-made soil samples. Thus, many of the models have not been systematically validated against data from natural soils, and are thus unlikely to give accurate predictions.

The paper discusses the systematic research by the author and her co-workers in the last 20 years that has resulted in the development and validation of advanced soil models, specifically for Scandinavian soft soil conditions. The models have now reached the level of maturity that enables their use in practical context, as part of advanced finite element analyses. Some examples are highlighted, as well as discussion about parameter determination, model calibration and future challenges.

2. Anisotropic creep model for soft clays

The natural clays in Scandinavia were mostly formed during and after the last Ice Age. Following the subsequent retreat and melting of the glaciers, large volumes of fine-grained sediments were depositing at high rates, forming thick deposits of soft clays. These natural clays exhibit rather complex response: they are anisotropic with regards of strength, stiffness, yielding and hydraulic conductivity [4, 5]. Due to their high compressibility, the anisotropy may change due to geological, environmental and anthropogenic processes. The soft clay deposits that have been exposed to isostatic uplift and leaching are often sensitive, which makes the material stronger than expected, but yet meta-stable and brittle [6, 7]. Furthermore, the response is strain-rate dependent [8]. The strain-rates in laboratory and field differ by orders of magnitude [9], making the mapping from element scale in the laboratory to the field scale, and indeed the parameter determination, non-trivial, unless a rate dependent model is adopted.

In geotechnical design, we often cannot control the loads, and even less the stress path that is emerging from the coupled hydro-mechanical response of the material of natural origin. This is particularly true in underground construction in urban setting, as illustrated in Figure 1. Some of the total stress paths related to different types of construction have been plotted in terms of mean stress p and deviatoric stress q. As we cannot do testing in every project for all these stress paths, we need a representative constitutive model. A constitutive model is simply a mathematical formulation that enables predictions for the soil response under any arbitrary stress path, based on a single set of model parameters we have derived from a series of laboratory tests. The model parameters are kept constant, regardless of the stress-path (imposed or emerging), and only the state variables associated with the stresses and the soil state (e.g. preconsolidation pressure, void ratio etc.) can change during the analyses. For deep excavations in soft soils constructed using Observational Method [2], we need to be able to predict the wall movements and the vertical settlements behind the wall as a function of time, to set up trigger levels for monitoring (see Fig. 2). Without predictions to compare with, the monitoring gives us no understanding of the performance of the wall. In urban areas on soft soils, we often have ongoing background settlements that also need to be accounted for. A rate-dependent constitutive model offers a generalised stress-strain rate relationship, i.e. what are the incremental strains caused by changes in effective stresses at a given rate of loading.

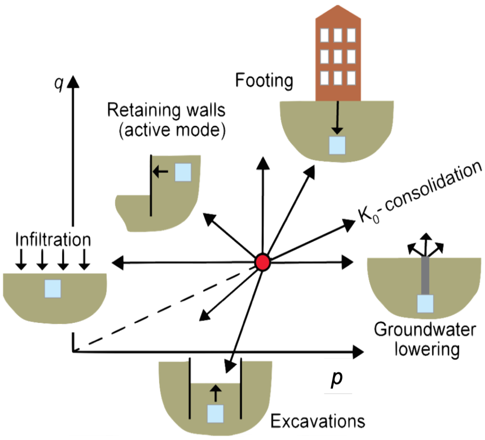


Figure 1: Typical loading paths.

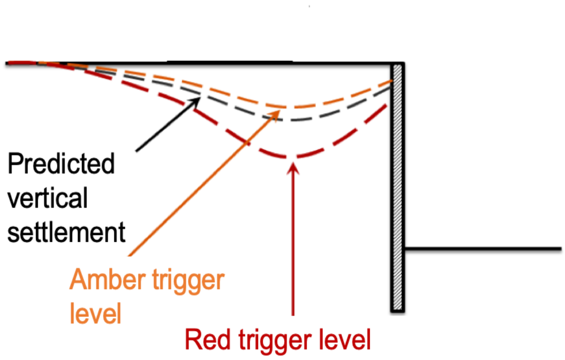


Figure 2: Predications and trigger levels.

Creep-SCLAY1S [10, 11] is a rate-dependent elasto-visco-plastic constitutive model developed for Scandinavian soft clays. The model has its basis in the Modified Cam Clay model [12]. The key assumption in Creep-SCLAY1S is that there is no purely elastic domain. Thus, viscoplastic (creep) deformations occur at all stress states due to the particulate nature of the material, but become negligible with the overconsolidation of the clay. The total strain rate is expressed as a combination of the elastic and viscoplastic component:

$$\delta\epsilon_v = \delta\epsilon_v^e + \delta\epsilon_v^c \tag{1}$$

$$\delta\epsilon_d = \delta\epsilon_d^e + \delta\epsilon_d^c \tag{2}$$

where $\delta\epsilon_v^c$ and $\delta\epsilon_d^c$ are the volumetric and deviatoric creep strain increments, respectively. The Normal Compression Surface (NCS) that represents the boundary between the small and large irrecoverable creep strains is initially fixed in the time domain by a reference time τ [13]. The reference time τ links the loading rate used in laboratory tests with the respective apparent preconsolidation pressure, thus defining the size of NCS p'_m (Fig. 3). Initially, for most natural soils NCS is inclined, due to the initial anisotropy associated with the 1D loading in the past. The formulation adopted for NCS in Creep-SCLAY1S is a sheared ellipse [14]. When looking at the model in the triaxial space (Fig. 3), the equation for NCS can be expressed as:

$$p'_m = p' + \frac{(q - \alpha p')^2}{(M(\theta)^2 - \alpha^2)\ p'} \tag{3}$$

where p' is the mean effective stress, q is the deviatoric stress, α is a state variable related to the inclination of the yield surface and p'_m relates to the size of NCS (thus the apparent preconsolidation pressure). Note that α is scalar only in the special case when the main axis of anisotropy coincides with the principal stress axis, as would be the case in triaxial testing of vertical samples from a vertically consolidated deposit. Hence, in a multi-dimensional finite element implementation the scalar α needs to be replaced with a fabric tensor and deviatoric stress q needs to be replaced with the deviatoric stress tensor [15]. M is the stress ratio at critical state. M is assumed to depend on Lode angle θ (see [10] for details), enabling to account for the differences of M_c (critical state stress ratio in triaxial compression) and M_e (critical state stress ratio in triaxial extension) measured for natural soft soils.

The anisotropy is assumed to change due to the (irrecoverable) creep strains. The rotational hardening law was developed based on drained triaxial probing of natural Otaniemi clay [15], and systematically validated against several Finnish clays [16]. Thus, NCS is assumed to rotate as a function of volumetric and deviatoric creep strains as originally proposed in [17], analogously to the elasto-plastic S-CLAY1 model [15]:

$$\delta\alpha = \omega\left(\left[\frac{3\eta}{4} - \alpha\right]\langle\delta\epsilon_v^c\rangle \ + \omega_d\left[\frac{\eta}{3} - \alpha\right]|\delta\epsilon_d^c|\right) \tag{4}$$

where η is the stress ratio (q/p') and ω and ω_d are model constants related to the evolution of anisotropy. The value for ω_d is unique for a given soil, and therefore can, similarly to the initial value of α, be theoretically derived based on the assumed value K_0^{nc} (the coefficient of earth pressure at normally consolidated state) for clays that are normally or lightly overconsolidated [15]. The McCauley brackets around $\delta\epsilon_v^c$ are simply used to keep the predictions qualitatively sensible on the left side of critical state line, when the volumetric creep strain increment is negative [15]. The modulus sign around $\delta\epsilon_d^c$ is only needed due to the common sign convention in triaxial testing, and not needed in the generalised form of the model.

To account for the sensitivity of natural clays, and the resulting additional resistance to yielding and failure, an imaginary Intrinsic Compression Surface (ICS, see Fig. 3) is introduced [18], following the ideas in [19]. The two surfaces are assumed to be related as follows:

$$p'_m = p'_{mi}(1+\chi) \tag{5}$$

where p'_{mi} is the size of ICS (see Fig. 3) and χ is a state variable related to the sensitivity S_t (χ=S_t-1) of the clay. It is assumed that the size of ICS is increasing as a function of volumetric creep strains as follows:

$$\delta p'_{mi} = \frac{p'_{mi}}{\lambda_i^* - \kappa^*}\delta\epsilon_v^c \tag{6}$$

where λ_i^* is the modified intrinsic compression index, representing the compressibility of the soil once all bonds have been degraded, and κ^* is the modified swelling index. Simultaneously, as the

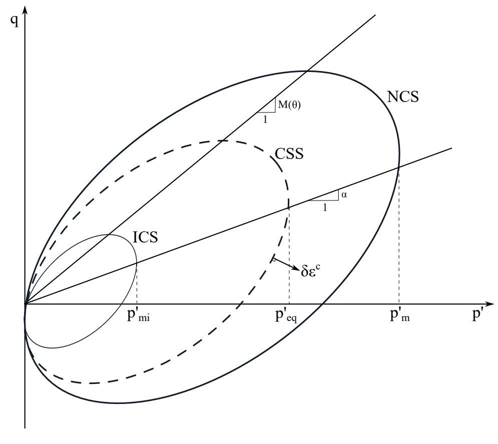


Figure 3: Normal compression surface (NCS) and intrinsic compression surface (ICS) for Creep-SCLAY1S. Current stress surface (CSS) maps the current stress state to the hydrostatic axis.

size of ICS is changing according to Eq. 6 due to volumetric creep strains, the apparent bonds in the clay, represented by state variable χ are degrading according to the following degradation law [18]:

$$\delta\chi = -a\chi\left(|\delta\epsilon_v^c| + b\,|\delta\epsilon_d^c|\right) \tag{7}$$

where a and b are model parameters related to the bond degradation. The modulus sign is again needed around $\delta\epsilon_d^c$ due to the common sign convention in triaxial testing, and disappears in the generalised form of the model. The incremental creep strains in Eqs. (4), (6) and (7) are calculated using the concept of a viscoplastic multiplier $\dot{\Lambda}$ [20] that in the case of Creep-SCLAY1S results in the following expression [10]:

$$\delta\epsilon_v^c = \dot{\Lambda}\frac{\partial p'_m}{\partial p'} \tag{8}$$

$$\delta\epsilon_d^c = \dot{\Lambda}\frac{\partial p'_m}{\partial q} \tag{9}$$

$$\dot{\Lambda} = \frac{\mu_i^*}{\tau}\left(\frac{p'_{eq}}{(1+\chi)p'_{mi}}\right)^{\beta}\frac{M_c^2 - \alpha_{K_0}^2}{M_c^2 - \eta_{K_0}^2} \tag{10}$$

$$\beta = \frac{\lambda_i^* - \kappa^*}{\mu_i^*} \tag{11}$$

where p'_{eq} is the equivalent current effective stress (see Fig. 3) and μ_i^* is the intrinsic creep index. Subscript K_0 in Eq. (10) refers to the earth pressure at rest in normally consolidated state, and should not be confused with the *in situ* K_0. Consequently, the magnitude of the creep strains depends on the proximity of the current (effective) stress state to NCS. An associated flow rule is assumed for the sake of simplicity, and has been confirmed to be valid experimentally [16].

In contrast to the equivalent elasto-plastic model [18], and the classic Perzyna type [21] elasto-visco-plastic models (e.g. [22]), the Creep-SCLAY1S model does not have a purely elastic region. No consistency condition is imposed, and thus it is possible to have effective stress states outside NCS, resulting in high creep rates. Given the apparent preconsolidation pressure and creep rate of sensitive clays are temperature-dependent [23], it is important that the laboratory testing for parameter derivation is done at relevant (constant) temperature.

One of the attractive features of the model is its hierarchical structure: various features, such as the evolution of anisotropy, initial anisotropy and bonding and destructuration, can simply be switched off by suitable parameter input. Thus, in addition to providing a tool for forecasting the response of geostructures, the Creep-SCLAY1S model can be used for understanding the importance of the various features on the predicted rate-dependent response of a clay.

The generalised version of the Creep-SCLAY1S model is implemented as a user-defined model [10] for the Plaxis finite element code in collaboration between Chalmers University of Technology, Norwegian Geotechnical Institute and Plaxis bv, using the implicit algorithm in [24], and was later on implemented in Tochnog Professional.

3. Parameter determination and model calibration

The first part of setting up a numerical model for a boundary value level analyses of a geotechnical problem is to divide the soil deposit into representative layers, for the subsequent derivation of the model parameters. This is usually done based on the *in situ* water content and CPT profile, and can be confirmed once the rest of the laboratory data is available. Fig.4 shows the incrementally loaded (IL) oedometer test data for Haarajoki test embankment [25], compiled for the analyses with Creep-SCLAY1S model in [26]. Only by combining the results for comparison, one can appreciate the scatter due to possible sample disturbance as well as the natural variation within a layer, which in this case applies mainly to the dry crust (layer 1) and layer 4.

Once the layering has been decided upon, it is time to derive the model parameters. There is a myth that the more complex the model, the more difficult it is to derive the model parameters. This is, however, largely untrue, if the model is able to represent the relevant features of the soil response. Provided the parameters of a constitutive model have a physical meaning, and the relevant standard laboratory tests are available, the determination of the values of the parameters is relatively easy. Furthermore, for the Creep-SCLAY1S model, there is a well-established physical and practical range for the parameters [11].

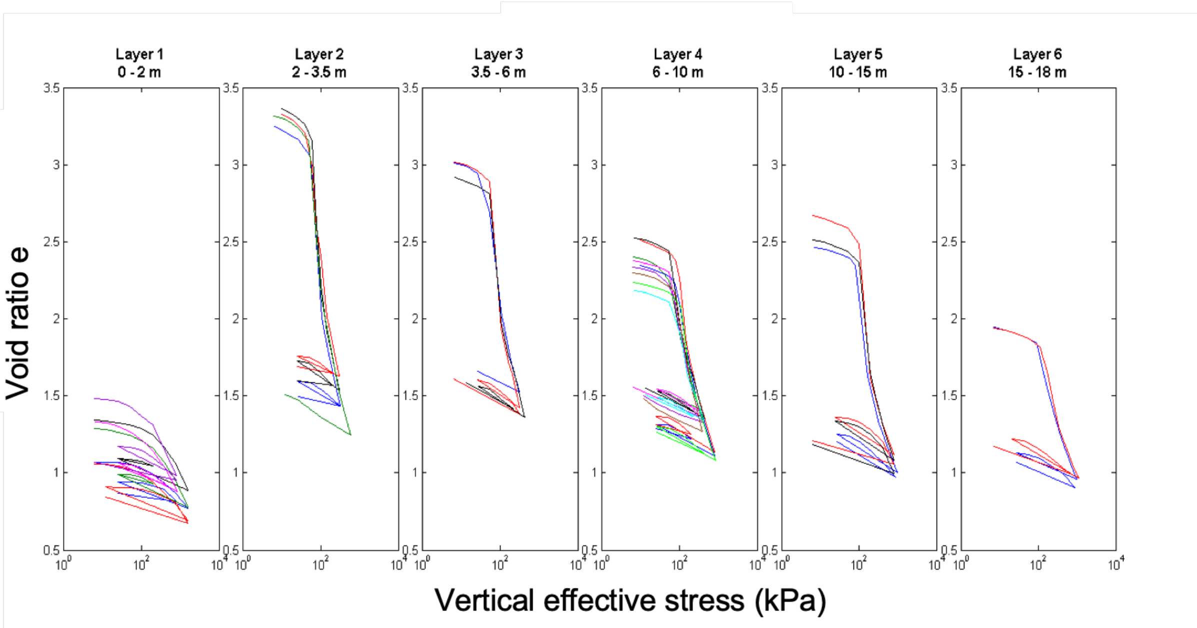


Figure 4: Division of the deposit in individual layers for Haarajoki embankment.

Firstly, the rate-independent intrinsic parameters, such as the stress ratios in critical state (M_c and M_e are easy to derive from anisotropically consolidated undrained triaxial tests in compression (CAUC). For excavation problems, an additional triaxial test with shearing in extension is preferred (CAUE), but if those are not available, the values for M_e can be calculated from M_c by simply assuming equal friction angle in compression and extension. M_c value is also used in estimating the initial anisotropy α_0 and ω_d, the parameter relating to the rotation of NCS [10, 11, 15]. It should be noted that λ_i can be derived based on oedometer tests on natural clay samples, provided the test is continued to such high stress levels that all bonds are destroyed (see Fig. 5 for an example on Vanttila clay). The swelling index κ^* and the intrinsic creep index μ_i^* can can also be derived based on the IL tests on natural clay samples. As seen later on for the Ballina test embankment [30], based on CRS tests alone, it is not possible to get good predictions with a creep model. A CRS test is, however, useful for planning the steps in IL test, or when performed at different strain rates, for studying the rate-dependency.

The parameter with most influence on the predictions by a creep model is the apparent preconsolidation pressure σ'_c. The value is affected by geological and anthropological processes, and is thus extremely site-specific. Even though the compressibility is modelled with a semi-logarithmic relationship in Creep-SCLAY1S, the interpretation for σ'_c is best done in the linear scale, because changes in anisotropy may obscure yield in the semi-logarithmic scale, as demonstrated in [31]. Furthermore, the value interpreted from laboratory results can be severely affected by sample disturbance, which can have massive impact on the subsequent predictions [32]. For this reason, even though the process of parameter determination can largely be automated [26], some engineering judgement remains essential, and sensitivity studies are recommended.

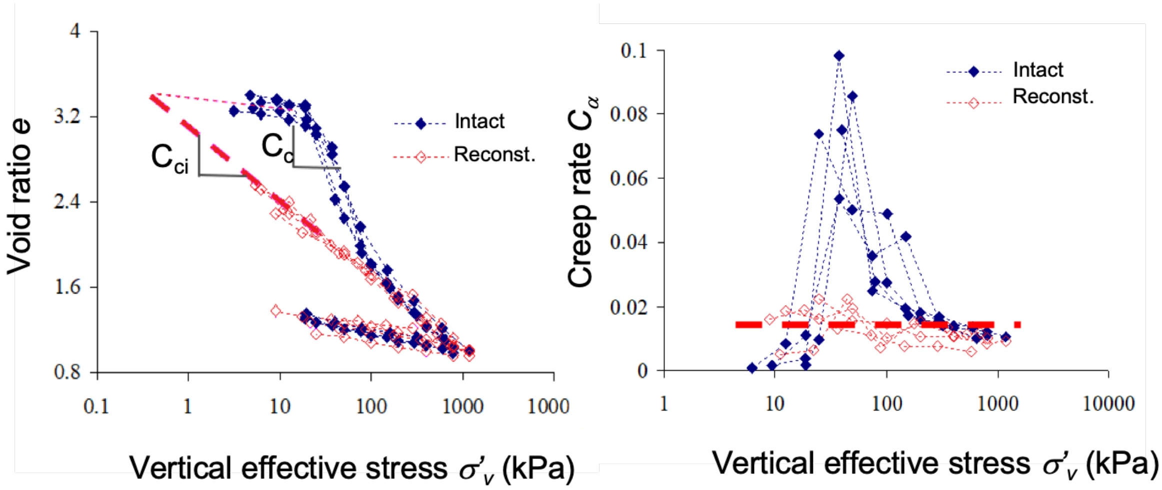


Figure 5: Compression and creep of Vanttila clay. Data from [28].

Creep-SCLAY1S has some model parameters that cannot be directly measured from standard tests, namely parameters a and b relating to destructuration and parameter ω relating to the rate of rotation of NCS. However, given the range for these parameters is known [11], they can be calibrated using a strain driver with the model implemented. Furthermore, by using clever multi-objective optimisation [29], it is possible to arrive to a consistent and unique set of model parameters based on multiple loading paths. Systematic probing with a strain driver also helps to investigate which parameters are most important to a given stress path.

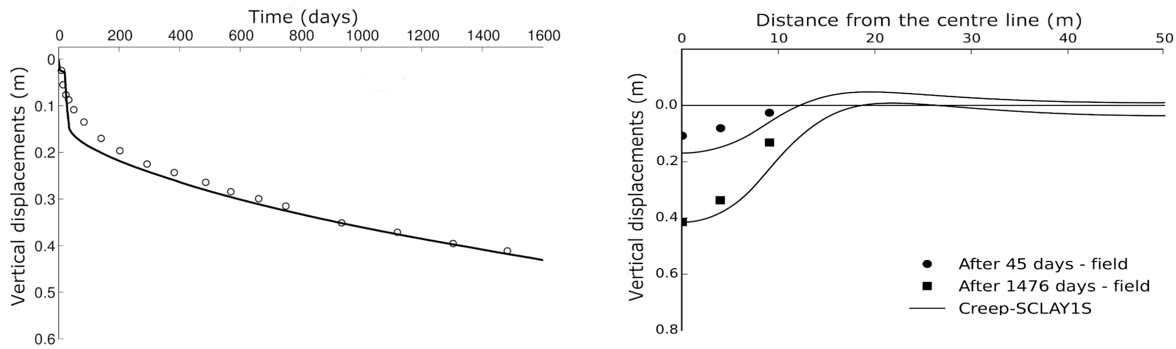
IL and CAUC tests for each soil layer provide sufficient data for derivation and model calibration to arrive at all model parameters for Creep-SCLAY1S. Having additionally CAUE tests is recommended for excavation and unloading problems, with many points in the domain that experience unloading. As loading in extension typically involves major changes in the anisotropy, CAUE tests also enable fine-tuning the parameter ω relating to the rotation of NCS. CRS tests are also useful in planning IL tests, and in addition they can be used for validation of the calibrated parameter set, as fundamentally CRS test is a boundary value problem [33]. In the following, the application of Creep-SCLAY1S to modelling geotechnical problems in the field scale is discussed.

4. Model application to field-scale problems

4.1. Embankments on soft soils

Critical state models, such as Modified Cam Clay [12], are often used to simulate the time-dependent consolidation of embankments on soft soils. However, as the finite element simulations by the author made in early 90's on Paimio test embankments in Finland [34] demonstrate, an isotropic model often over-predicts the horizontal movements. Adding anisotropy, as well as bonding and destructuration, improves the predictions substantially [18], but in the sensitive clays found in Scandinavia, rate-dependency cannot be ignored. The predictions for the test embankment in Murro were significantly improved by adopting a rate-dependent model [18, 22].

Consequently, the Haarajoki test embankment in Finland was re-analysed [26] with Creep-SCLAY1S by deriving the model parameters in a consistent manner from the raw laboratory data. The predictions for the vertical displacements as function of time are presented in Fig. 6 for the section of the embankment that has no vertical drains. The match between the predictions and measurements is very good. Analyses were also done for the vertically drained section, by mapping the 3D problem into 2D using the simple mapping proposed in [36], and the results are equally good (see [26] for full results). The latter simulations, however, require assumptions on the smear zone caused by the installation of the drains. Given the uncertainties in the hydraulic conductivity in the smear zone, and the extent of the smear zone, there is no need adopt the more complicated mapping techniques, such as those used in [27] for Haarajoki.



(a) Predicted (line) and measured (markers) settlements at centre line.

(b) Settlement profile in cross-section for two instances of time.

Figure 6: Predicted settlements for Haarajoki test embankment (after [26]).

The application of the model is easier, when all experimental results needed are available at the time of parameter calibration. For Class A predictions (made before the construction) of the embankment used in Ballina embankment challenge [30], the author and her co-workers had to initially rely on CRS tests results [35] that is not ideal for the creep model. In particular, CRS tests do not always take samples to sufficiently high stress levels for the destructuration to be complete. Thus, in addition to the assumptions related to smear effects, there were uncertainties related to the correction of the CRS preconsolidation pressure for rate effects, and the intrinsic compression and creep index. So, even though the element level model calibrations appear to be satisfactorily (see 7 for one of the layers), our Class A predictions [35] severely underpredict the settlements as shown in Fig. 9 with a gray shaded area. The Class C predictions, also shown, were simply done by exploiting IL oedometer test results that came available much later, as well as the existing field measurements for re-assessing the hydraulic conductivity in the vertically drained area. Fig. 8 shows that this improved the predictions significantly. The settlements as function of depth and time, as well as the horizontal movements, were satisfactorily predicted [35] without changing any other model parameters.

Based on our experience so far, Creep-SCLAY1S model gives very satisfactory results for predicting the deformations and pore pressures under embankment loading as a function of time, provided the necessary high quality laboratory results are available for the calibration of the model parameters. In addition to deformation predictions, the model can be used to assess how the apparent stability (undrained strength) is changing in time due to consolidation, creep and changes in the groundwater level. This helps the design of staged construction and/or surcharge loading as well as assessing the possibility for increasing the axle loads several decades after construction. Unfortunately, there are not many tests embankments in the world that combine decent site characterisation with multi-dimensional monitoring over long periods of time. Assessing the changes in the stiffness and hydraulic conductivity due to the installation of vertical drains, introduces a fair amount of additional subjectivity, and thus requires a time-series of reliable field measurements.

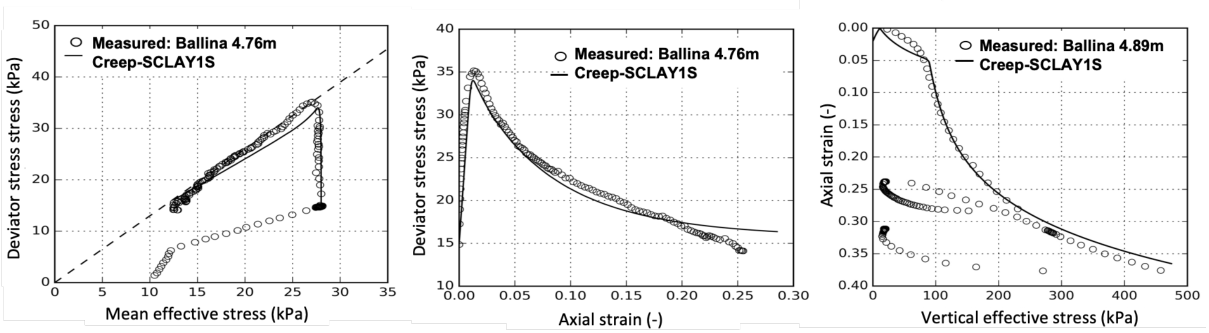


Figure 7: Calibration of Creep-SCLAY1S model against laboratory data for Ballina clay.

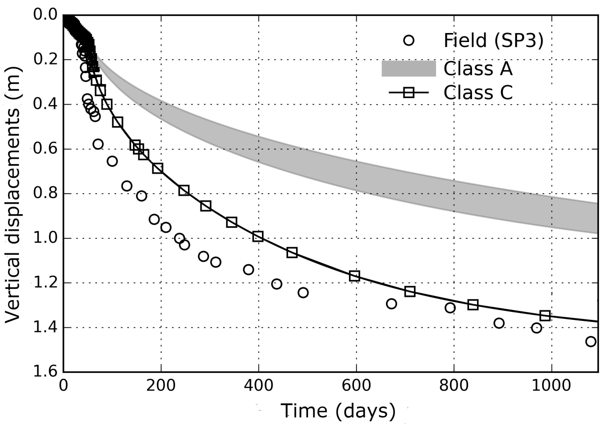


Figure 8: Ballina test embankment (data and simulations after [35]).

New road and railway embankments, however, have often much stricter criteria for differential settlements than old embankments. Consequently, in addition to vertical drains, deep mixed columns, stone columns and embankment piles are used. The installation of ground improvement and piles in sensitive soils causes substantial disturbance, excess pore pressures and the loss and recovery of strength and stiffness. The ability to modelling installation effects is becoming increasingly important when more and more construction is occurring next to existing structures, as discussed in the following.

4.2. Excavations and deep foundations in soft soil

So far Creep-SCLAY1S has mainly been applied to quasi-static loading problems, such as embankments on soft soils. However, with urban densification, there is an increasing need to construct ever taller buildings, with ever deeper basements. There is also a desire to move cars and public transport underground in order to reduce traffic noise and make effective use of the limited space. Underground construction in soft soils is not trivial in cities like Gothenburg with very deep clay deposits (over 100 m deep), in the areas where there are ambitious redevelopment plans. Due to the geological and anthropological history, the background creep settlements in the city centre of Gothenburg are as high as 30-40 mm/year in some local hot-spots (often rather recently filled areas). Most areas in the city continue to settle at a rate of around 1-5 mm/year based on results from remote sensing, which is still relevant from infrastructure design point of view: for road and railway tunnels and bridges the design life is typically 120 years. Thus, when designing foundations and tunnels the background creep settlements need to be taken into account, considering the effect during construction and long-term during operation.

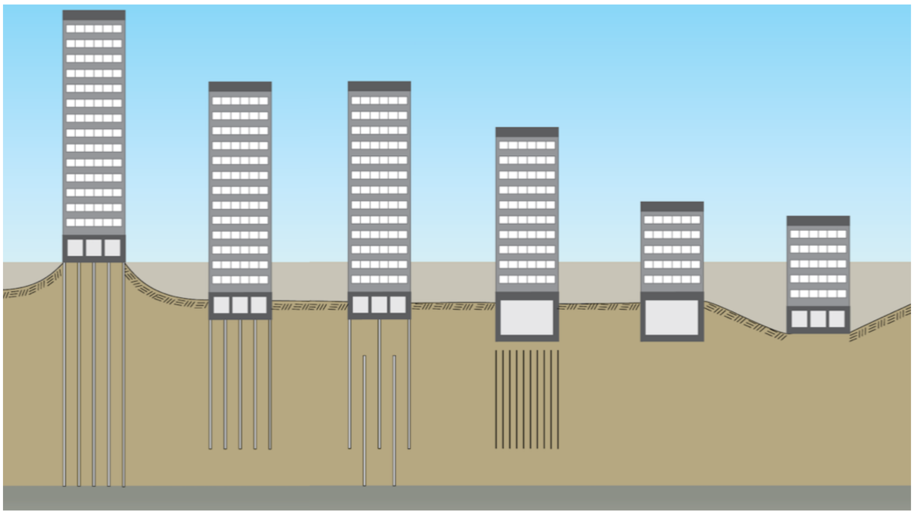


Figure 9: Overview of foundation solutions for deep subsiding soft soil deposits.

For buildings, end bearing piles under buildings in subsiding environment will simply result in emerging buildings, with problems with utilities, and excessive pile loads due to the negative skin friction (Fig. 9). Thus, alternatives such as floating pile foundations, overlapping piles that 'convert' the negative skin friction into a positive effect, as well as rigid inclusions offer attractive alternative solutions. In order to 'tune' the superstructures to settle at the same rate as the surrounding clay, the installation effects need to be accounted for. As shown in Fig. 5, the creep rates are much higher for the intact clay than the same clay in the reconstituted (disturbed) state. It thus is not sufficient to assume piles to be wished-in-place. It is possible to model installation effects with a model that accounts for the effects of bonding and bond degradation.

One of the first attempts to model installation effects in Gothenburg clay with Creep-SCLAY1S, relied largely on experience from previous projects [37], which is only feasible when a very experienced engineer is using a lot of time in digging into old case records with measurements and laboratory data. Via numerical modelling, it is possible to understand the installation effects due to e.g. cavity expansion, as long as the constitutive model has the relevant state parameters to describe the soil degradation and anisotropy. Examples for modelling stone column installation with cavity expansion can be found in [38, 39]. The simulations in [38] show that significant changes in anisotropy, and degradation of bonds in sensitive soils occur as a results of the installation, and furthermore, how it affects the subsequent response when the soil is subsequently loaded [39]. The installation of displacement piles, however, involves large deformations and shearing, in addition to volume expansion. Recently, Creep-SCLAY1S was used in modelling pile installation [40], by combining it with the Strain Path Method [41] with very promising results.

When constructing tunnels and deep excavations in soft clays, the main concerns are the stability against bottom uplift and the effects of the excavation to the surrounding structures, both in the short-terms and the long-term. Furthermore, ability to predict the increase in vertical pressures under the bottom slab is important for structural design. Recently, the construction of Göta tunnel in Gothenburg was modelled with Creep-SCLAY1S in [42]. This involved simulating the time-line of the construction sequence in detail, by studying the workbooks from the actual construction, all the way to the full life-time of the tunnel. Thus, both the short-term (construction time) and long-term response of the tunnel (after tunnel construction and backfill) that opened for traffic in 2006 was numerically simulated. Some of the results are presented in this conference [43]. Successful, yet unpublished, simulations were also performed for the Marieholm tunnel in Gothenburg, which is a 500 m long tunnel, involving a 300 m long immersed tube tunnel jointed to a 100 m long cut-and-cover tunnel. The latter section, which we simulated with 2D finite element analyses, was used as dry dock in which the tunnel elements, each with a length of 100 m, were constructed before they were lowered at the final location into the excavated river bed. Filling and emptying the dry dock involved large loading-unloading loops. Most recently, the model has been used by NCC in simulating the excavation of the West Link railway tunnel at the Central station in Gothenburg.

The advantage of CREEP-SCLAY1S over the simpler constitutive models used in industry is that one model with one set of model parameters predicts, both the short and long-term response. The simulations show that by calibrating the model carefully, as was done for Haarajoki [26], Ballina [35], very good predictions are obtained even for an unloading problem, such as Göta tunnel [42, 43], even though the model does not incorporate small strain stiffness [44]. As shown in [37], the small strain stiffness for sensitive clay more complex compared to stiff clays, such as London clay [45]. As yet, we simply do not have enough data on the small strain stiffness of sensitive clays to justify the inclusion of this feature in the model.

5. Future challenges

The increasing challenges resulting from building in the urban environment, climate change, infrastructure demands, and foundations for renewable energy require new model features that depart from quasi-static hydro-mechanical loading only. The hierarchical nature of Creep-SCLAY1S makes model extensions possible, whilst preserving the features developed in the last two decades. Currently, additions that include thermal effects and cyclic accumulation are under development. Furthermore, compatibility of the model for fully coupled large deformation analyses is explored for modelling pile installation and its effects to the surrounding structures.

Modelling soil-structure-interaction remains challenging. This involves both the interaction between soft soils and stiff walls and/or piles, and the modelling of ground improvement, such as deep-mixed columns and stone columns. In a new project starting in 2020 the Creep-SCLAY1S will be combined with Volume Averaging Technique (VAT) [46, 47, 48] to arrive at an efficient numerical method for modelling of deep mixing columns in sensitive clays. The idea is to consider both axial (e.g. embankment) and lateral loading (passive zone of retaining walls), thus reducing the need for 3D finite element analyses, such as those in [49].

Although large improvements on systematic calibration and parameter optimisation have been made [29, 11], the methods are currently further extended to the boundary value level, so that other essential mechanisms (generation and dissipation of pore water pressures, inertial effects) can be included [50]. The last remaining challenge for successful user adoption is the overlooked problem of model initialisation. All state based models, including Modified Cam clay [12] and their derivatives suffer from the difficulties in initialising the initial stress and state variables in complex settings that deviate from green field conditions with horizontal soil layers. A great example of this problem encountered for natural slopes is presented in [51]. Model simulations of the geological processes that most likely have formed the natural slopes in sensitive clay can help us in understanding where to take representative samples for laboratory testing and also will enable us to explain the laboratory results.

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