Proceedings 10th NUMGE 2023
10th European Conference on Numerical Methods in Geotechnical Engineering
Zdravkovic L, Kontoe S, Taborda DMG, Tsiampousi A (eds)

© Authors: All rights reserved, 2023
https://doi.org/10.53243/NUMGE2023-434

From theory to practice - numerical modelling of geostructures on soft natural clays

M. Karstunen[1]

[1]*Department of Architecture and Civil Engineering, Chalmers University of Technology, Gothenburg, Sweden*

ABSTRACT: The growing need for accurate predictions on the Serviceability Limit State response of geostructures requires numerical methods that build upon models that are representative for the type of soils being studied. For many advanced geotechnical engineering applications on soft natural clays, constitutive models that incorporate features such as anisotropy, rate-dependency, and degradation of bonds (destructuration) are required. This paper summarises the experimental underpinning at element level, subsequent model development and the boundary value level validation of a family of models for soft natural clays, resulting in the Creep-SCLAY1S model. Moreover, the feasibility of using this model in real-world engineering problems, in terms of model calibration and initialisation, is discussed. The results show that Creep-SCLAY1S performs well for a wide range of engineering applications that span many rate-dependent loading and unloading paths. The boundary value level examples range from embankments on soft clays to deep excavations, with deep soil mixed columns or piles that stabilise the bottom of the excavation. Furthermore, for some outstanding issues with model initialisation in sloping ground and the challenges in accounting for installation effects, tentative solutions are elaborated. Finally, future developments and use of these type of phenomenological models is discussed.

Keywords: Constitutive modelling; soft clays; anisotropy; rate-dependency; numerical modelling

1 INTRODUCTION

Already for millennia, the human population has opted to live close to waterways, due to the good transport connections and favourable conditions for farming and subsequent industrialisation. Thus, from a geological point of view, many major cities are located in soft soil areas. Even though we have been successfully building on soft soils for thousands of years, it is costly. Furthermore, we are now facing unprecedented challenges due to globalisation, urbanisation and climate change.

We increasingly need to invest in infrastructure and facilities, as global markets rely on secure and reliable flow of people, goods, knowledge, and services. Thus, our roads and railways are facing increasing levels of utilisation paired with increasing axle loads. The rapid increase in the electrification of transportation, including the introduction of autonomous vehicles, comes with additional challenges in terms of use of space and demands for our ageing infrastructure. Those might include increased weights, separation of different transportation modes and adaptions to the required line of sight for autonomous driving.

As a result of the climate change, we already experience more extreme drying-wetting cycles, and a notable increase in the periods of intense precipitation and drought. Also, there is evidence of more erratic temperature fluctuations, affecting the surface layers in the ground. In the Nordic countries in particular, the increases in the annual number of freezing-thawing cycles (Tang et al., 2018) and fluctuations of pore pressures, have already caused a substantial increase in maintenance demands, as well as embankment and slope failures in unexpected locations.

The classic 1D consolidation analyses and 2D stability analyses using the concept of undrained shear strength in conjunction with Limit Equilibrium Method no longer suffice, as both the mobilised shear strength and the stiffness evolve due to the changed environmental conditions, mostly affecting the most critical shallow modes of failure. To understand what is happening, advanced physics-based numerical models are required, to enable forecasting for possible future climate scenarios.

As societies increasingly concentrate on cities and densely populated urban areas, we are forced to construct on soft soils that have inferior engineering properties, thus needing ground improvement and/or piles. Due to physical constraints, effective utilisation of the underground space is increasingly on the agenda. The underground construction potentially impacts the surroundings, leading to increased risks, both in the short and long term. The new constructions inevitably affect the existing structures and buildings, some with

high cultural and/or historic value. Thus, there are more demands on predictions and monitoring. Monitoring alone is insufficient, as the rigorous use of Observational Method (Peck, 1969), relies on predictions of the quantities that are being monitored. In case of excavations, this means (vertical and horizontal) displacements, stresses and pore pressure changes, the latter often affecting large areas. The problems we need to analyse are 3D, with time as the 4th dimension.

In an urban environment, the site characterisation needs to consider not only the geological history, but also account for the effects of anthropological loading history. For example, the installation effects of retaining structures, piles and ground improvement in soft soils are often of the same order of magnitude as the effects of the excavation and tunnelling themselves.

The deepening of waterways and canals, deposition of fills, as well as historic land and water use often cause significant background subsidence in cities built on soft soils, which cannot be ignored when planning and designing new developments. Fortunately, new sources of monitoring data that cover large areas, e.g. InSAR data from the European Ground Motion Service (Crosetto et al. 2020), can assist with this. The measurements on the surface of course need to be complemented with depth-integrated measurement of pore pressures and displacements.

All in all, future geotechnical design cannot be based on simplified semi-empirical methods, as we no longer have the conditions for which those methods have been calibrated for. Thus, we will increasingly need to rely on numerical modelling to understand, and predict, the performance of complex systems. A known challenge in geotechnical engineering is that we cannot control the direction of loading. The loading directions can be complex, as illustrated in Figure 1, where some typical total stress paths have been plotted in terms of mean stress (p) and deviatoric stress (q). Due to the low hydraulic conductivity and the viscous nature of natural soft clays, the emerging effective stress paths that control the stress-strain response are rate-dependent. The soil response for arbitrary loading paths thus needs to be generalised in a rate-dependent constitutive model, to accurately simulate the response of geostructures.

Both qualitatively and quantitatively, the results of any geotechnical numerical analysis depends on the constitutive model used to represent the soil behaviour (Potts, 2003). Furthermore, for natural soft soils, the results depend on the quality of soil sampling and testing used to derive the input data (Karlsson et al., 2016). Above all, the results rely on the ability of the geotechnical engineer in choosing a representative soil model, as well as deriving representative values for the relevant model parameters and (initial) state parameters.

Geotechnical literature and commercial finite element (FE) codes offer numerous, increasingly

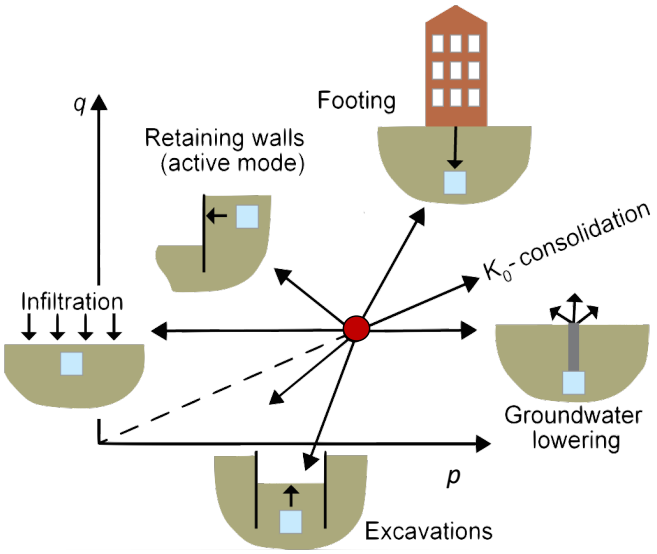


Figure 1. Typical loading path in terms of total stresses. Modified from Karstunen (2021).

complex, constitutive models. Many of these, however, have been developed via testing reconstituted soil samples, and only a few have been systematically validated against high-quality data on intact samples of natural soils.

The first part of the paper discusses the research in the last twenty years, by the author and her co-workers, that led to the development and validation of two advanced soil models, called SCLAY1S (Karstunen et al., 2005) and Creep-SCLAY1S (Sivasithamparam et al., 2015, Gras et al., 2018), aimed specifically for sensitive natural clays. The models have now reached such a level of maturity that they can be used in practical context as part of advanced FE analyses, as demonstrated for some cases by the Chalmers team in the second part of the paper.

2 EXPERIMENTAL EVIDENCE

The motivation for the development of the SCLAY1 family of models was the desire to have as simple constitutive model as possible, which yet would be able to capture the key features of the response of natural clays. The developments started with considering (large strain) anisotropy, bonding and destructuration, and later on rate-dependency and creep.

Firstly, a rate-invariant anisotropic model was developed (Wheeler et al. 1999, Wheeler et al. 2003) based on systematic radial probing of a natural clay from Espoo, Finland, called Otaniemi clay. The results demonstrated that we not only need to account for the initial anisotropy, but in addition, depending on the stress path and the irrecoverable straining along an effective stress path, the evolving anisotropy. Test-wise, the probing was challenging, with each drained triaxial test taking 2-3 months. The samples were first loaded along a given effective stress path well beyond yield to

change the initial anisotropy, and then unloaded along the same stress path. A reloading along another effective stress path that was significantly different from the first loading path, again well beyond yield, enabled to quantify the change in anisotropy, providing experimental evidence for the proposed rotational hardening law.

Figure 2 is demonstrating how the inclination of the yield surface, described by scalar α in triaxial stress space, is changing as a function of stress ratio η ($\eta = q/p'$). The data from Otaniemi clay has been complemented with data on both natural and reconstituted POKO (Porvoo-Koskenkylä) and Murro clays (data from Koskinen, 2014). Both α and η have been normalised by the stress ratio at critical state in triaxial compression (M) to combine the results for multiples clays. Also included as solid lines are the prediction by the rotational hardening law of the S-CLAY1 model (Wheeler et al, 2003).

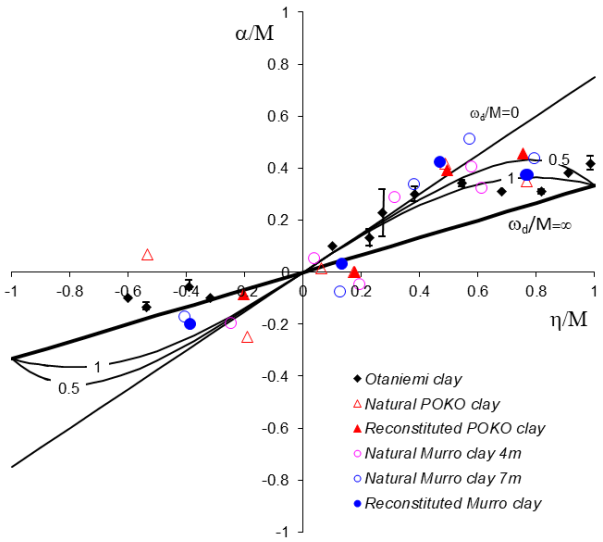


Figure 2. Evolution of anisotropy as a function of stress path (data from Koskinen 2014).

For typical values of the stress ratio at critical state (represented by model parameters ω_d/M between 0.5 to 1), SCLAY1 predicts maximum anisotropy for 1D (K_0) loading, and isotropy when the stress ratio η =0. The experimental results show some scatter, partly attributed to the fact that the loading was not always taken sufficiently beyond the yield to reach the target value predicted by the model for the given effective stress path, and partly to natural variation. Even the reconstituted samples may have variations as they had to be mixed from a wet natural clay in small batches. Furthermore, some of the samples were stored for a long time, given the limitations on test capacity, and the duration of testing, and thus may have suffered from some sample disturbance affecting the interpretation of yield.

The SCLAY1 model simulations in Wheeler et al. (1999, 2003) were however far from satisfactory,

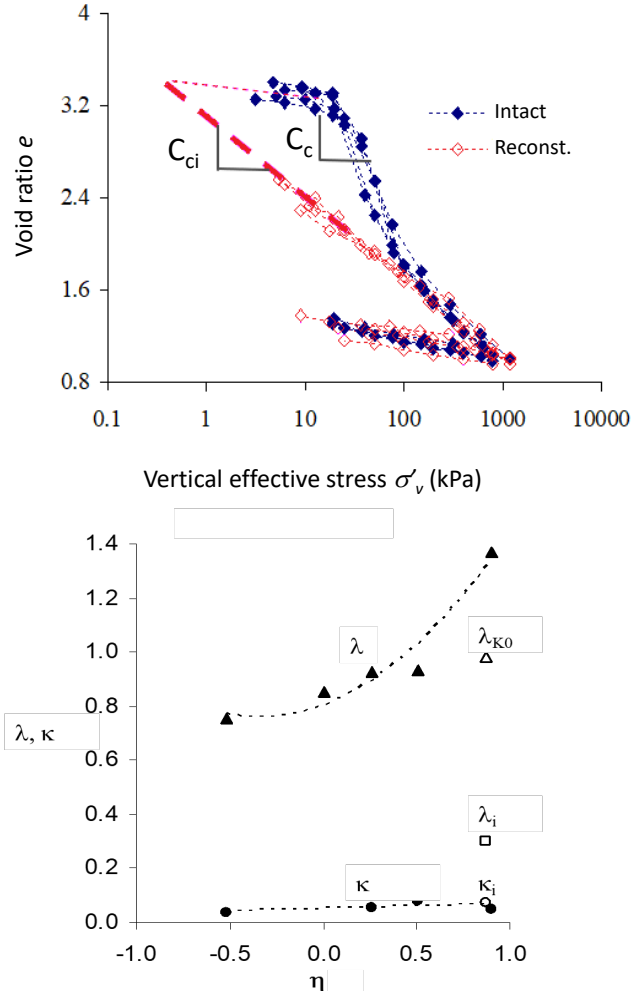


Figure 3. Compressibility of natural and reconstituted Vanttila clay. 1D compressibility (data from Yin et al., 2011, on top); Apparent compressibility in triaxial stress space (data from Koskinen, 2014, at the bottom).

because the model did not account for the effects of bonding and destructuration. Thus, Koskinen et al. (2002) enhanced the model by adding a state variable related to the initial amount of bonding, as well as destructuration law, following the idea of Gens and Nova (1993). The destructuration law in SCLAY1S (Koskinen et al 2002, Karstunen et al. 2005) enables to account for the significant difference in the compressibility of the same clay in its natural and reconstituted state, which is illustrated in Figure 3 for highly sensitive Vanttila clay from Espoo, Finland.

With a destructuration law, it also is possible to simulate the evolving compressibility of natural clays (Fig. 3 top) regardless of the effective stress level, as well as to account for the apparent stress-path dependency of the compressibility, see Fig. 3 (bottom), where λ is the apparent slope of normal compression line and κ the slope of the unloading (and recompression) line. The subscript i refers to the intrinsic values from an Incremental Loading (IL) oedometer test on reconstituted sample and λ_{K0} is the slope of the apparent virgin compression line

corresponding to the steepest part of the slope on an IL oedometer test on an intact sample of Vanttila clay. The dashed lines are the trend lines. The stress-path dependency of straining in natural clays was confirmed by drained stress probing of a medium sensitive Swedish clay in triaxial conditions with monitoring using X-ray Computed Tomography (XCT), see Birmpilis et al. (2022), providing unique 4D deformation data.

The apparent bonding in the natural sensitive clay is also reflected in the tendency of the soil to creep, as illustrated in Figure 4 for Vanttila clay, where the creep rate C_α is plotted against the effective vertical stress. The creep rates are the highest just around the apparent preconsolidation pressure. Before yield, and at high effective stress level, after all bonding has been erased, the creep rate of the intact samples is very similar to the creep rate of the reconstituted clay. Thus, the creep rate (or the modified creep index μ^* used in the following) can be considered as an intrinsic property.

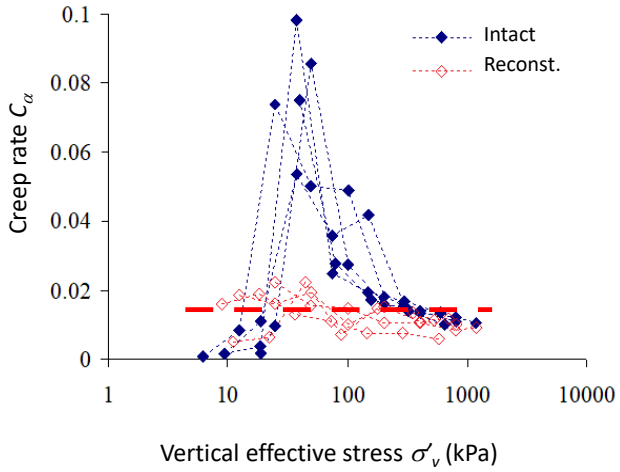


Figure 4. Creep rate of natural and reconstituted Vanttila clay. 1D compressibility (data from Yin et al., 2011).

3 CREEP-SCLAY1S MODEL

Creep-SCLAY1 model (Karstunen et al., 2013, Sivasithamparam et al. 2015), is an anisotropic creep model for soft clays developed in collaboration between Chalmers, Norwegian Geotechnical Institute and Plaxis BV. The model was subsequently extended, similarly to Karstunen et al. (2005), to be applicable for natural sensitive clays. This version, referred to as Creep-SCLAY1S, is available in e.g. Plaxis FE code.

The model is a hierarchical rate-dependent (creep) model. Similarly to its elastoplastic equivalent, SCLAY1S (Koskinen et al. 2002, Karstunen et al. 2005), model features, such as the evolution of anisotropy and the effect of bonding and destructuration, can be switched off by appropriate choice of model parameters. Associated flow rule is assumed for simplicity and numerical stability.

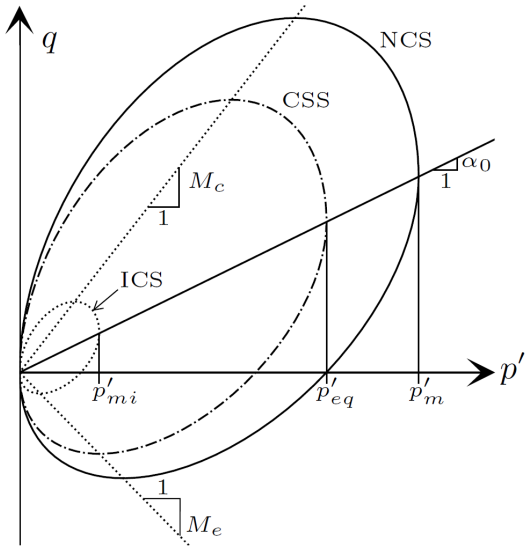


Figure 5. Creep S-CLAY1S model (after Gras et al., 2017).

The Normal Compression Surface of the Creep-SCLAY1S model (Fig. 5), which represents the boundary between small and large creep strains, is assumed to be initially anisotropic, similarly to the SCLAY1 model (Wheeler et al. 2003). This assumption is reasonable, given most natural clays have been deposited under K_0-condition typically in lacustrine or marine conditions. The expression in Equation (1) was first proposed by Dafalias (1986) based on thermodynamic considerations, and totally independently by Korhonen et al. (1987) based on experimental evidence. In the simplified case of triaxial space (Figure 5), the equation for NCS can be expressed as:

$$f_{NCS} = (q - p')^2 - \left(M(\theta)^2 - \alpha^2\right)\left[p'_m - p'\right]p' = 0 \quad (1)$$

where α is a state variable (a scalar only in this special case in triaxial stress space) related to the inclination of the yield surface, and M is the stress ratio at critical state. M is assumed dependent on Lode angle θ, enabling to give different values for M_c (critical state stress ratio in triaxial compression) and M_e (critical state stress ratio in triaxial extension), as measured for soft clays (see Sivasithamparam et al. (2015) for details). When extension tests are not available, the value for M_e can be estimated based on the friction angle at critical state corresponding to the Mohr Coulomb failure as:

$$M_e = \frac{6\sin\varphi_c'}{3+\sin\varphi_c'} \quad (2)$$

The current effectives stress state is represented by the Current Stress Surface (CSS). To account for bonding, as reflected by the sensitivity of the clay, and the additional resistance to yielding, an imaginary Intrinsic Compression Surface (ICS), inspired by Gens and Nova (1993), is adopted. The sizes of two surfaces (see Fig. 5) are related as follows:

$$p'_m = (1+\chi)p'_{mi} \tag{3}$$

where χ is a state parameter related to the sensitivity S_t (χ=S_t -1). The size of ICS is assumed to change as a function of incremental volumetric creep strains $\delta\varepsilon_p^c$:

$$\delta p'_{mi} = \frac{p'_{mi}\,\delta\varepsilon_p^c}{\lambda_i^* - \kappa^*} \tag{4}$$

where λ_i* is the modified intrinsic compression index, defined identically to the modified compression index λ*. λ_i* is derived from an oedometer test on reconstituted clay (or an oedometer test on natural clay at such a high strain level that all effects of any apparent bonding have been destroyed). κ* is the swelling index.

Simultaneously, when the size of ICS is changing according to Equation (4) due to irrecoverable volumetric creep strains, the apparent bonds in the clay, represented by state variable χ, are degrading according to the degradation law, which assumes that both volumetric $\delta\varepsilon_p^c$ and deviatoric $\delta\varepsilon_q^c$ creep strains have a role in bond degradation.:

$$\delta\chi = -a\chi\left[\ \left|\delta\varepsilon_p^c\right| + b\left|\delta\varepsilon_q^c\right|\right] \tag{5}$$

where a and b are model constants. The modulus sign implies that there is bond degradation regardless of the sign of the incremental creep strains.

The incremental creep strains are calculated using the concept of viscoplastic multiplier, proposed by Grimstad et al. (2010), which in the case of Creep-SCLAY1S results in the following expression for the incremental creep strains $\delta\varepsilon^c$:

$$\delta\varepsilon^c = \dot{\Lambda}\left(\frac{\partial f_{NCS}}{\partial\sigma'}\right) \tag{6}$$

$$\text{with}\quad \dot{\Lambda} = \frac{\mu_i^*}{\tau}\left(\frac{p'_{eq}}{p'_m}\right)^{\beta}\left(\frac{M_c^{\,2} - \alpha_{K0}^{\,2}}{M_c^{\,2} - \eta_{K0}^{\,2}}\right) \tag{7}$$

$$\text{and}\quad \beta = \frac{\lambda^*_i - \kappa^*}{\mu_i^*} \tag{8}$$

where the strain-rate related parameters are the intrinsic creep index μ_i*, and the reference time τ. The latter accounts of the loading rate used in an oedometer test (Leoni et al., 2008). Subscript *K0* refers to normally consolidated K_0 state.

The state variable α is used to represent, and keep track of, the evolution of the model surfaces as function of incremental creep strains, representing changes in anisotropy. As discussed in Wheeler et al., (2003) and Sivasithamparam et al., (2015), when generalising the model for solving problems with principal stress rotations in 2D, and in 3D, a deviatoric fabric tensor that can be defined analogously to the deviatoric stress tensor, needs to be used instead of scalar α.

In the simplified case of triaxial tests on samples cut from the soil in vertical direction, however, the rotational hardening law can be expressed in terms of the scalar α as:

$$\delta\alpha = \omega\left[\left(\frac{3\eta}{4} - \alpha\right)\langle\delta\varepsilon_p^c\rangle + \omega_d\left(\frac{\eta}{3} - \alpha\right)\left|\delta\varepsilon_q^c\right|\right] \tag{9}$$

where ω and ω_d are model constants related to the evolution of anisotropy. The McCauley brackets keep the predictions sensible on the left of critical state line. The modulus sign is needed to comply with the sign convention commonly used in triaxial testing, and disappears in the generalised form of the model.

4 ON MODEL CALIBRATION

From the outset, the Creep-SCLAY1S model has significantly more model parameters than the e.g. the Modified Cam Clay model (Roscoe & Burland, 1968). Indeed, adding any new feature (creep, anisotropy, bonding etc.) results in additional model parameters (constants), as well as additional state variables (that evolve with creep strains). The latter need to be properly initialised first, and subsequently updated throughout the analyses.

The number of parameters in not a major issue, given the values for most of the model constants and state parameters of Creep-SCLAY1S can be derived in a straight-forward manner based on a standard test programme. For example, the value for ω_d and the initial value of α can be theoretically derived based on the friction angle at critical state for normally consolidated or lightly overconsolidated clays (Wheeler at al. 2003). Similarly, the initial preconsolidation state, expressed either with overconsolidation ratio (*OCR*) or pre-overburden pressure (*POP*), is linked with the selected reference time τ. Thus, the differences in the strain-rates in the laboratory and on site are automatically accounted for by the model.

At the end, there are only three model constants (ω, a and b) that cannot be directly derived from standard tests, and thus need calibration with the help of element level simulations of laboratory tests. However, even these model constants have a clear physical meaning, as well as certain theoretical upper and lower bounds, as detailed in Gras et al. (2018). By simulating an IL oedometer test, the values for the destructuration parameters a and b can be calibrated, to simulate the oedometric stress-strain response at any stress level.

Usually results from Incrementally Loaded (IL) oedometer tests would first be used to derive values for the modified compression and swelling indices, preconsolidation pressure and creep index. It should,

however, be noted that λ_i* and μ_i* are both intrinsic values.

Constant Rate of Strain (CRS) oedometer tests are not ideal for the Creep-SCLAY1S model, as there is no creep phase with a constant load as needed for deriving μ_i*. The resulting value for preconsolidation pressure would also need to be corrected for strain rate, by adjusting either the value itself, or the reference time, which is not trivial (Leoni et al, 2008). Furthermore, as demonstrated by Muir Wood (2016), a CRS test is strictly speaking not even an element test.

For the strength parameters, anisotropically consolidated undrained triaxial tests are needed. A triaxial compression test (CAUC) enables estimating M_C. For problems that involve major unloading, excavation or installation effects, changes in anisotropy are expected. In those cases, a triaxial extension test (CAUE) is recommended, as it enables calibrating the value for ω that controls the rate of evolution of anisotropy. Drained triaxial tests could also be used, but they are notoriously difficult to perform on soft natural clay samples, as during very slow loading rates there may be excess pore pressures generated by creep strains.

Given Creep-SCLAY1S model is hierarchical, it is possible to swich features off, such as evolution of anisotropy and destructuration, based on what is relevant for the problem considered. In general, accounting for initial anisotropy improves the K_0-prediction significantly compared to the MCC model. Furthermore, problems that include significant rotation of principal stress axes are often associated with changes in anisotropy.

As for bonding and destructuration, whether that feature is necessary depends on the sensitivity of the clay and the problem solved. Any problems on sensitive clays where the strength mobilisation and potential strain softening is important, such as modelling the failure of an embankment or a slope, will benefit from incorporation of destructuration, and the same applies for modelling installation effects. Strain softening in general is known to result is numerical issues, such as severe mesh-dependency, and loss of converge. However, as highlighted by e.g. de Borst and Duretz (2020), viscoplastic models, such as Creep-SCLAY1S, benefit from some inherent regularisation. In contrast to other features, it is not possible to switch off the rate dependency.

Gras et al., (2017) discuss which parameters are most influential for a given type of loading. At boundary value level, the most sensitive parameter is the apparent preconsolidation pressure, given *NCS* is not only the boundary between small and large creep strains, but also the rate of creep is influenced by the equivalent preconsolidation pressure, as highlighted by Equation (6). Furthermore, as demonstrated by Tahershamsi & Dijkstra (2022), the relative importance of model parameters changes spatially with effective stress levels, and thus (consolidation) time.

Especially in urban settings, the preconsolidation pressure is affected not just by the geological history (sedimentation and erosion) and creep and cementation, but also by past construction processes, as illustrated in Figure 6. Theoretically, most natural soft clay deposits would be expected to be at least lightly overconsolidated because of creep and cementation, but there might also be changes due to anthropological effects. The best approach to calibrate the initial state of preconsolidation pressure is to run the problem with initial stresses only and calibrate the strain rates against measurements of background creep (e.g. from InSAR data).

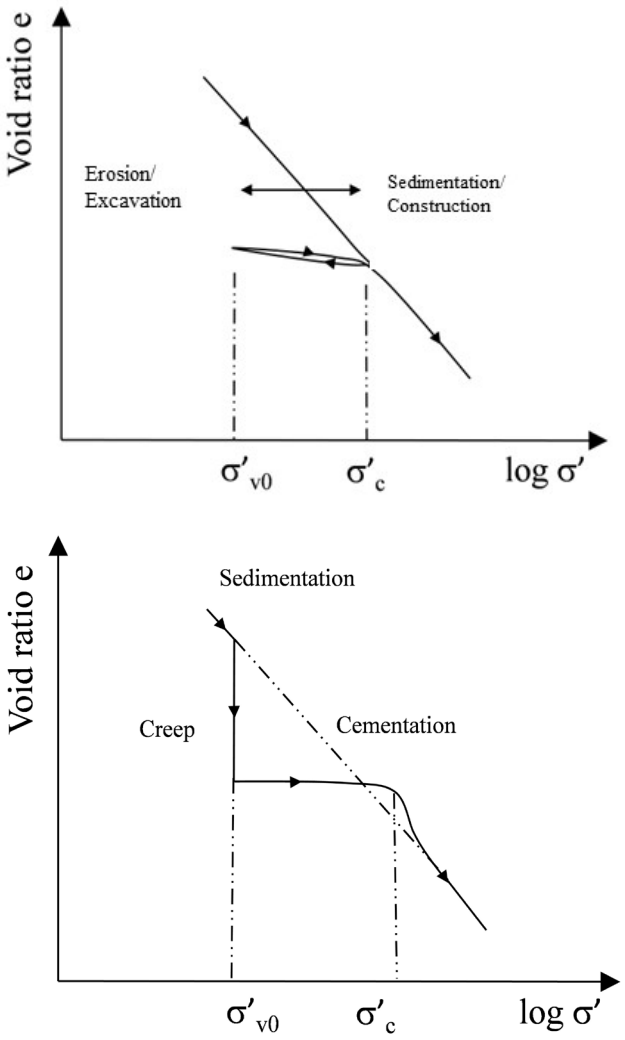


Figure 6. Processes affecting the apparent preconsolidation pressure: loading/unloading (top), creep and cementation (bottom).

5 EMBANKMENTS AND SLOPES

Embankments on soft soils are a classic geotechnical problem. In many cases the problem involves both loading and unloading, as surcharge loading is often

used to reduce future creep settlements. On sensitive clays, the geotechnical design of embankments is always controlled by deformations, rather than stability. Deformation analyses are in practice extremely error prone. International challenges, such as the settlement calculation competition for Haarajoki test embankment (Vepsäläinen et al. 1997) and the Ballina Embankment Challenge (Sloan 2018), demonstrate how poor both academics and practicing geotechnical engineers are in Class A predictions (Lambe 1973) of the time-dependent response of embankments on soft soils. (Class A predictions are predictions made before the construction of the embankment). The comparisons between professional engineers and Civil Engineering students by Karstunen et al. (2020), suggest that there is not much difference in the predictive ability between the two cohorts. However, when the parameters for the Creep-SCLAY1S model are properly calibrated, it is possible to make very good Class C predictions (Amavasai et al., 2018, 2022). The quality of the predictions namely significantly improves when a limited time series of field monitoring data is available. With field monitoring data from the early stages of consolidation, it is possible to reduce uncertainties, and thus improve the predictions for the future performance.

An example of Class C predictions for the Ballina test embankment (Amavasai et al., 2018) is given in Figure 7. As the embankment was built on vertical drains, the installation of the drains caused some disturbance. This effect was taken into account by using the method by Chai et al. (2001), which is computationally more effective than attempting to model the drains and the smear zone in 2D. Overall, it is possible to make reasonably good system-level predictions, once some time series of the system response are available, as confirmed by Liu et al. (2018). Equally good system-level predictions using Creep-SCLAY1S for the Haarajoki test embankment were made by Amavasai et al. (2022). More recently, simulations of the Onsøy test embankment (Berre 2014, 2018) that was brought to failure after three years of consolidation, demonstrate that once Creep-SCLAY1S has been calibrated to capture the response for the Serviceability Limit State, it is possible to predict both the height of the embankment at failure, as well as the time to failure (Karstunen, 2022).

Whilst using the Creep-SCLAY1S model for simulating the response of embankments, and the strength evolution under the embankments (Hernvall et al., 2023), is rather straight-forward, the application of an anisotropic model for the analyses of natural slopes is far more challenging. Namely, whilst K_0 conditions can be assumed as a starting point in simulating an embankment, this does not apply to a natural slope in its initial configuration. To initialise the anisotropy in an anisotropic model for a natural slope correctly, it is necessary to model the geological processes, such as the erosion by the river and the changes in groundwater level that have resulted to the geometry and hydrogeological conditions of the natural slope as it is now (Karstunen 2022, Sellin & Karstunen, 2023), see Figure 8. These processes result in major changes in anisotropy, affecting particularly the location where the eventual failure (by imposing gravity increase) is predicted to initiate. Sellin & Karstunen (2023) further demonstrate that the initial anisotropy and its evolution have a significant effect on the stability of a natural slope, both in terms of the mode of failure and the time to failure.

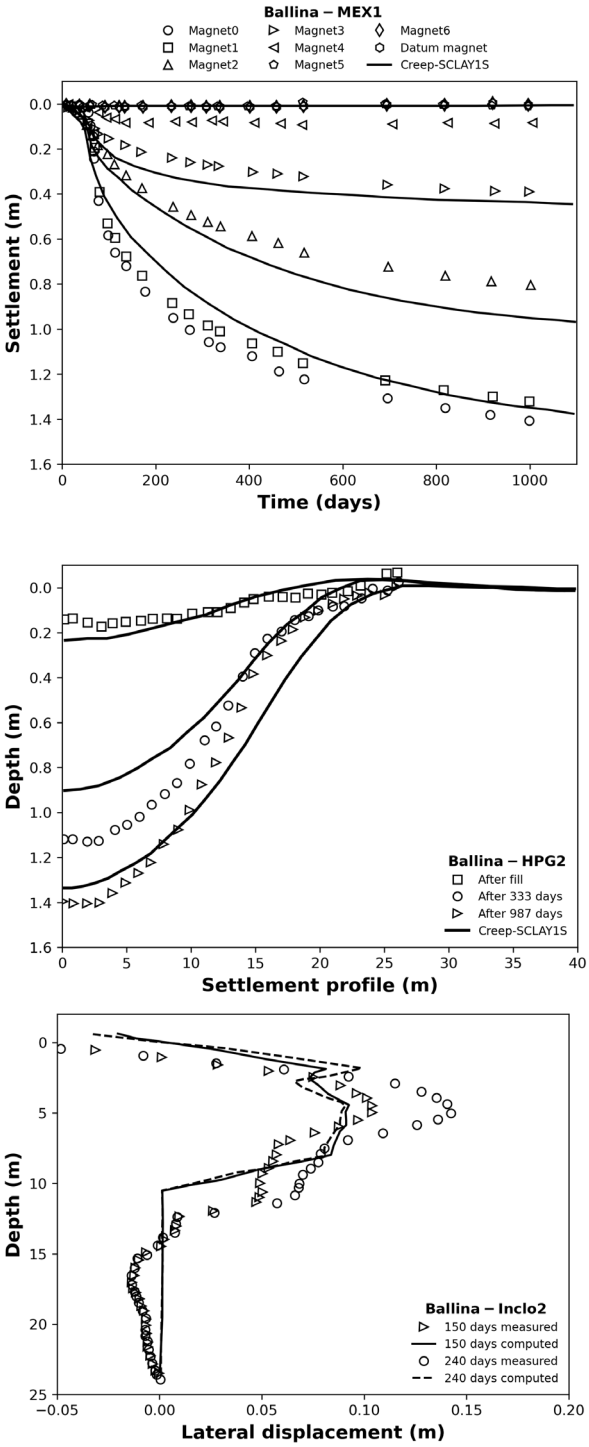


Figure 7. Ballina tests embankment. Simulations by Amavasai et al., (2018).

Due to the problems with excessive ground movements associated with sensitive clays, ground improvement with deep soil mixing method (DSM), typically using lime and cement as binders, has been used extensively in Scandinavia since 1970s. DSM has lower construction cost and a reduced cumulative energy consumption compared to piling methods (Zöhrer et al, 2010).

The main application of deep mixing in Sweden has been to limit displacements and vibrations under road and railway embankments. In some cases, it is enough to assume that the columns are wished-in-place. Simulations of Paimio test embankments on DSM columns by Abed & Karstunen (2023), exploiting the Volume Averaging Technique (Lee & Pande, 1998) as implemented by Vogler (2008), demonstrate that in the case of embankments, columns can be wished in place, as typically the embankments are constructed only after the excess pore pressures due to the installation have dissipated. In some cases, such as retaining structures, it is however also necessary to model the effects of the installation. A model that incorporates evolving anisotropy and destructuration, such as Creep-SCLAY1S, enables also to understand the processes associated with installation in natural clays.

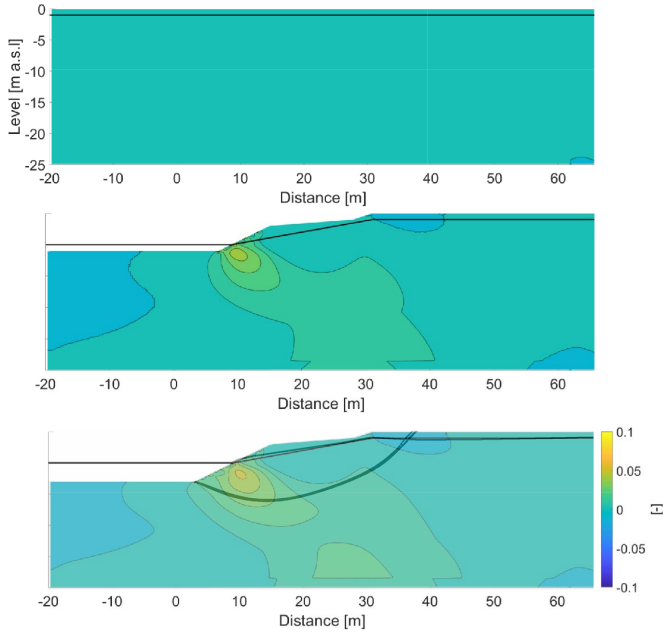


Figure 8. Evolution of anisotropy (the scalar value of α) from initial K_0 condition (top), to current in situ state (middle) and at failure (bottom).

6 SOIL-STRUCTURE INTERACTION PROBLEMS

The embedment of retaining walls, piles and ground improvement, such as DSM or stone columns, result in a soil-structure interaction problem. The design of embedded retaining structures in urban setting is largely dictated by the allowable ground movements. Multi-level propped retaining structures are often considered as small strain problems. However, in soft soils, that is not necessarily true. Namely, piles or other type of ground improvement are needed to control the movements, as well as to prevent failure due to basal heave. The installation of these elements within the excavation is an intrusive process, forcing the soft clay to fail, resulting in the generation of excess pore pressures, as well as time-dependent wall movements that can be of the same order of magnitude (or even larger) than those induced by the actual excavation.

Modelling installation effects is far from trivial. The installation of a single stone column could be modelled as a cavity expansion, as demonstrated by Castro & Karstunen (2010). The evolution of anisotropy and bond degradation play a major role in the subsequent performance of the structure (Castro et al. 2014, Sexton et al. 2016). However, cavity expansion is an oversimplification. During the installation of a displacement pile or a column, the soft soil around the object is heavily distorted, whilst constant volume conditions prevail. Only in the subsequent (pile) setup stage, consolidation and creep start to mobilise volume changes. The resulting changes in the state of the soil and the effective stress state around the pile determine the ultimate pile capacity (Karlsson et al. 2019). Modelling such a problem requires a numerical code capable of large deformations and a coupled hydromechanical framework (e.g. an Eulerian approach proposed in Dijkstra et al., 2011). Such an approach has been successfully used in combination with S-CLAY1S for modelling pile installation (Isaksson et al., 2023).

Simulations of instrumented excavations, such as Göta tunnel excavation in Gothenburg (Tornborg et al, 2021) and a permanent sheet pile wall in Uppsala (Tornborg at al., 2023), with the Creep-SCLAY1S model utilised the field monitoring data to account for the effects of pile driving (Göta tunnel) and the installation of DSM columns under the railway embankment (Uppsala), respectively. In these cases, the installation was simulated numerically as a volume expansion, resulting in changes in anisotropy and the amount of bonding. Göta tunnel simulations (Tornborg et al, 2021) considered exclusively the system response during the construction period, while in Uppsala, the main interest was the long-term system response, given four years of monitoring data was available (Tornborg at al., 2023). In both cases, a creep model was necessary due to the background creep deformations.

Whilst the use of DSM under embankments is common in Sweden, only rather recently, after successes elsewhere (Tanaka, 1993, O'Rourke and O'Donnell, 1997, Ou et al., 2008), overlapping DSM columns have been used in bottom support of braced excavations. As demonstrated by Ignat et al. (2016, 2020), DSM can be used to reduce vertical and lateral

movements, as well as stabilising the bottom of the excavation.

When DSM columns are installed, there are complex interactions between the columns and the surrounding soil (Shen et al., 2003). The installation of DSM columns on the passive side of a sheet pile wall, causes large soil displacements away from the wall, which need to be corrected for in the subsequent analyses (Fig. 9) to represent the system response. By incorporating these installation effects as a volume change (Bozkurt et al., 2023), changes in anisotropy are predicted (as illustrated in Fig. 9). Following this, the system response of the retaining structure during the subsequent excavation stages can be estimated with increased accuracy.

In Figure 10, the composite material, composing of DSM and the natural soft clay, was modelled using an isotropic soil model MNHard (Benz 2007, Benz et al. 2008). A more elegant alternative would be to develop an equivalent stress-strain matrix, tracking the response of both constituents exploiting the Volume Averaging Technique (VAT) (Lee & Pande. 1998), considering the equilibrium and kinematic conditions for this problem, which will differ from the case of embankment loading.

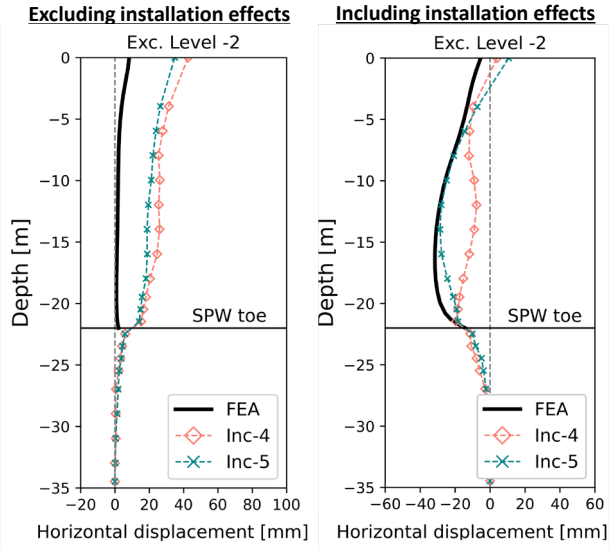


Figure 9. Model simulations of a deep excavation without (left) and with (right) installation effect after excavation to level -2 m. Positive values represent movements towards the excavation. Data from NCC AB.

7 CONCLUSIONS AND FUTURE OUTLOOK

The need to incorporate advanced soft soil features, i.e. anisotropy, destructuration and rate dependency, in the constitutive models for modelling natural sensitive clays was demonstrated, starting from observed behaviour at laboratory scale. The laboratory observations were the base for the formulation of the latest iteration of the SCLAY1 model family, the rate-dependent Creep-SCLAY1S model. Along the way, the importance of model calibration and initialisation were discussed, finishing with boundary value evaluation of the model performance for geostructures, such as embankments and retaining structures on soft clays. Some of the outstanding challenges in using these models for real-world engineering problems, most notably initialisation in the case of natural slopes as well as the installation effects, were discussed.

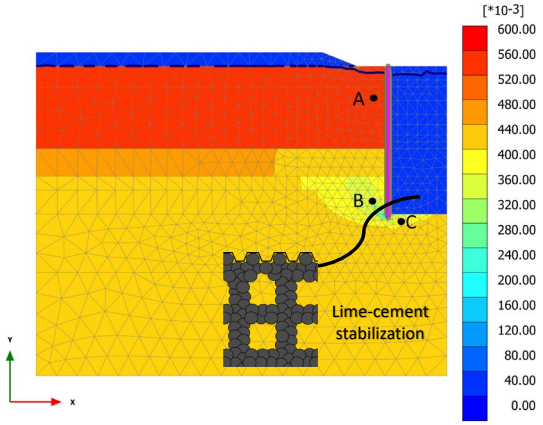


Figure 10. Predicted change in anisotropy a due to the installation of deep mixed columns on the passive side of the excavation.

Thanks to its viscoplastic formulation, Creep-SCLAY1S can be easily extended to incorporate additional mechanisms that impact the rate-dependent response of soft natural clays. The explicit modelling of accumulated strains from cyclic loading, is perhaps the most promising extension (Zuada Coelho et al., 2021). Another enhancement under development involves modelling the impact of climate change on the natural soft clay response, by incorporating partial saturation as well as thermal hardening and softening. During these developments, the hierarchical nature of the model will be preserved, to retain the model features that over time have been proven to be relevant both at element level and boundary value level.

Finally, although it sometimes appears as if an era of developing and using phenomenological models for soft soils comes to an end, nothing could be further from the truth. Only very recently experimental and modelling approaches at the clay-particle scale are gaining traction, enabling informing the hardening laws of the phenomenological models. Simultaneously, the advanced models, such as Creep-SCLAY1S, can be used to generate training data on low probability events (failure) for subsequent machine learning approaches.

8 ACKNOWLEDGEMENTS

I would like to thank all former and current colleagues, students, and collaborators for the interactions we have had over the years. The financial support by FORMAS and Vetenskapsrådet (Swedish research councils) and

Swedish Transport Administration (as part of BIG - Branchsamverkan i Grunden project), are greatly appreciated. The work is done as part of Digital Twin Cities Centre that is supported by Sweden's Innovation Agency VINNOVA.

9 REFERENCES

Abed, A., Karstunen, M. 2023. Using volume averaging technique for modelling deep mixing. *Numerical Analysis of Geomaterials, Symposium dedicated to the memory of Prof. G.N. Pande*. Assisi , Italy, May 10-12th, 2023.

Amavasai, A., Gras, J. P., Sivasithamparam, N., Karstunen, M., Dijkstra, J. 2022. Towards consistent numerical analyses of embankments on soft soils. *European Journal of Environmental and Civil Engineering*, 26(7), 2616-2634.

Amavasai, A., Sivasithamparam, N., Dijkstra, J., Karstunen, M. 2018. Consistent Class A & C predictions of the Ballina test embankment. *Computers and Geotechnics*, 93, 75-86.

Amavasai, A., Wood, T., Dijkstra, J. 2023. On the feasibility of data assimilation for uncertainty modelling in geotechnics. *Proceedings of NUMGE 2023 (10th European Conference on Numerical Methods in Geotechnical Engineering)*, London, UK, 26 - 28 June 2023.

Benz, T. (2007). *Small strain stiffness of soils and its numerical consequences*. PhD Thesis, University of Stuttgart, Germany.

Benz, T., Wehnert, M. & Vermeer, P.A. (2008). A Lode angle dependent formulation of the hardening soil model. *12th International Conference on Computer Methods and Advances in Geomechanics (IACMAG)*, pp. 653–660

Berre, T. 2014. Test fill on soft plastic marine clay at Onsøy, Norway. *Canadian Geotechnical Journal* 51 (1), 30–50.

Berre, T. 2018. Test fill brought to failure on soft plastic marine clay at Onsøy, Norway. *Canadian Geotechnical Journal* 55 (4), 563–576.

Birmpilis, G., Andò, E., Stamati, O., Hall, S. A., Gerolymatou, E., & Dijkstra, J. 2022. Experimental quantification of 3D deformations in sensitive clay during stress-probing. *Géotechnique*, 1-12.

de Borst, R., Duretz, T. 2020. On viscoplastic regularisation of strain-softening rocks and soils. *International Journal for Numerical and Analytical Methods in Geomechanics*, 44(6), 890-903.

Bozkurt, S., Abed, A., Karstunen, M. 2023. 2D & 3D numerical analyses of a deep excavation supported by LC columns. *Proceedings of NUMGE 2023 (10th European Conference on Numerical Methods in Geotechnical Engineering)*, Zdravković L, Kontoe S, Taborda DMG, Tsiampousi A (eds), London, UK, 26 - 28 June 2023.

Castro, J., Karstunen, M. 2010. Numerical simulations of stone column installation. *Canadian Geotechnical Journal*, 47(10), 1127-1138.

Castro, J., Karstunen, M., Sivasithamparam, N. 2014. Influence of stone column installation on settlement reduction. *Computers and Geotechnics*, 59, 87-97.

Chai, J. C., Shen, S. L., Miura, N., Bergado, D. T. 2001. Simple method of modeling PVD-improved subsoil. *Journal of Geotechnical and Geoenvironmental Engineering*, 127(11), 965-972.

Crosetto, Michele, et al. 2020. The evolution of wide-area DInSAR: From regional and national services to the European Ground Motion Service. *Remote Sensing* 12(12): 2043.

Dafalias, Y. F. 1986. An anisotropic critical state soil plasticity model. *Mechanics Research Communications*, 13(6), 341-347.

Dijkstra, J., Broere, W., Heeres, O. M. 2011. Numerical simulation of pile installation. *Computers and Geotechnics*, 38(5), 612-622.

Gens, A., Nova, R. 1993. Conceptual bases for a constitutive model for bonded soils and weak rocks. In: *Geomechanical Engineering of Hard Soils and Soft Rocks*. (Eds: A. Anagnostopoulos, F. Schlosser, N. Kaltesiotis, & R. Frank). , Vol. 1, pp. 485–494, A.A. Balkema, Rotterdam.

Gras, J.-P., Sivasithamparam, N., Karstunen, M., Dijkstra, J. 2018. Permissible range of model parameters for natural fine-grained materials. *Acta Geotechnica*, 13(2), 387-398.

Gras, J. P., Sivasithamparam, N., Karstunen, M., Dijkstra, J. 2017. Strategy for consistent model parameter calibration for soft soils using multi-objective optimisation. *Computers and Geotechnics* 90: 164-175.

Grimstad, G., Degado, S. A., Nordal, S., Karstunen, M. 2010. Modeling creep and rate effects in structured anisotropic soft clays. *Acta Geotechnica*, 5(1), 69-81.

Hernvall, H., Karlsson M., Karstunen, M. 2023. A numerical study of the time effects on the stability of a test embankment on sensitive clay. *Proceedings of NUMGE 2023 (10th European Conference on Numerical Methods in Geotechnical Engineering)*, London, UK, 26 - 28 June 2023.

Ignat, R., Baker, S., Karstunen, M., Liedberg, S., Larsson, S. 2020. Numerical analyses of an experimental excavation supported by panels of lime-cement columns. *Computers and Geotechnics* 118, 103296.

Ignat, R., Baker, S., Liedberg, S., Larsson, S. 2016. Behavior of braced excavation supported by panels of deep mixing columns. Canadian Geotechnical Journal 53, 1671–1687.

Isaksson, I., Karlsson, M., Dijkstra, J. 2023. Modelling pile installation in soft natural clays. *Proceedings of NUMGE 2023 (10th European Conference on Numerical Methods in Geotechnical Engineering)*, London, UK, 26 - 28 June 2023.

Karlsson, M., Emdal, A., Dijkstra, J. 2016. Consequences of sample disturbance when predicting long-term settlements in soft clay. *Canadian Geotechnical Journal* 53(12), 1965-1977.

Karlsson, M., Yannie, J., Dijkstra, J. 2019. Modeling aging of displacement piles in natural soft clay. *Journal of Geotechnical and Geoenvironmental Engineering* 145(10), 04019070.

Karstunen, M. 2021. From soft soil modelling to engineering application. *IOP Conference Series: Earth and Environmental Science* (Vol. 710, No. 1, p. 012002). IOP Publishing.

Karstunen, M. 2022. Advanced numerical models for geotechnical problems on soft clays. *10th Raúl J. Marsal Córdoba Lecture, XXXI Reunión Nacional de Ingeniería Geotécnica*, Guadalaraja, Mexico, November 16 - 19, 2022. ISBN 978-607-99185-7-6.

Karstunen, M., Sivasithamparam, N., Brinkgreve, R.B.J., Bonnier, P. G. (2013). Modelling rate-dependent

behaviour of structured clays. Proceedings of Installation Effects in Geotechnical Engineering, 43-50.

Karstunen, M., Birmpilis, G., Dijkstra J. 2020. Developing soft soil engineering skills using "Class B" and "Class C" predictions. Invited paper. *Proc. eGEE 2020: Online Conference on Geotechnical Engineering Education*, Athens, Greece, June 23-25, 2020. ISSMGE Online Library under TC306 conferences.

Karstunen, M., Krenn, H., Wheeler, S. J. Koskinen, M., Zentar, R. 2005. Effect of anisotropy and destructuration on the behaviour of Murro test embankment. *International Journal of Geomechanics* 5 (2): 87-97

Koskinen, M. 2014. *Plastic anisotropy and destructuration of soft Finnish clays*. PhD thesis. Aalto University, Espoo, Finland.

Koskinen, M., Karstunen, M., Wheeler, S.J. 2002. Modelling destructuration and anisotropy of a natural soft clay. *Proc. 5th European Conference on Numerical Methods in Geotechnical Engineering (NUMGE 2002)*, (Ed: P. Mestat), 11-20. Presses de l'ENPC, Paris.

Lambe, T.W. 1973. Predictions in soil engineering. *Géotechnique* 23(2), 149-202.

Lee, J. S., Pande, G. N. 1998. Analysis of stone-column reinforced foundations. *International Journal for Numerical and Analytical Methods in Geomechanics*, 22(12), 1001-1020.

Leoni, M., Karstunen, M., Vermeer, P. A. 2008. Anisotropic creep model for soft soils. *Géotechnique* 58(3), 215-226.

Liu, Z., Choi, J. C., Lacasse, S., Nadim, F. 2018. Uncertainty analyses of time-dependent behaviour of Ballina test embankment. *Computers and Geotechnics*, 93, 133-149.

Muir Wood, D. 2016. Analysis of consolidation with constant rate of displacement. *Canadian Geotechnical Journal* 53(5): 740-752.

O'Rourke, T.D., O'Donnell, C.J. 1997. Field behavior of excavation stabilized by deep soil mixing. *Journal of Geotechnical and Geoenvironmental Engineering*, 123(6), 516-524

Peck, R. B. 1969. Advantages and limitations of the observational method in applied soil mechanics. *Géotechnique*, 19(2), 171-187.

Potts, D. M. 2003. Numerical analysis: a virtual dream or practical reality? *Géotechnique*, 53(6), 535-573.

Roscoe, K.H. & Burland, J.B. 1968. On the generalized stress-strain behaviour of 'wet' clay, in: J. Heyman and F.A. Leckie, eds., *Engineering Plasticity* (Cambridge Univ. Press, Cambridge), 535-609.

Sellin C., Karstunen, M. 2023. Slope stability assessment in sensitive clay with an advanced constitutive model. *Proceedings of NUMGE 2023 (10th European Conference on Numerical Methods in Geotechnical Engineering)*, London, UK, 26 - 28 June 2023.

Sexton, B. G., McCabe, B. A., Karstunen, M., Sivasithamparam, N. 2016. Stone column settlement performance in structured anisotropic clays: the influence of creep. *Journal of Rock Mechanics and Geotechnical Engineering,* 8(5), 672-688.

Shen, S.L., Miura, N., Koga, H. 2003. Interaction mechanism between deep mixing column and surrounding clay during installation. *Canadian Geotechnical Journal* 40, 293–307.

Sivasithamparam, N., Karstunen, M., Bonnier, P. 2015. Modelling creep behaviour of anisotropic soft soils. *Computers and Geotechnics* 69, 46-57.

Sloan, S. 2018. Ballina Embankment Prediction Symposium (Editor). *Computers and Geotechnics*, 93, 1-280.

Tahershamsi, H., Dijkstra, J. 2022. Using experimental design to assess rate-dependent numerical models. *Soils and Foundations*, 62(6), 101244.

Tanaka, H. 1993. Behavior of braced excavations stabilized by deep mixing method. *Soils and Foundations* 33, 105–115.

Tang, A. M., Hughes, P. N., Dijkstra, T. A., Askarinejad, A., Brenčič, M., Cui, Y. J., ... Van Beek, V. 2018. Atmosphere–vegetation–soil interactions in a climate change context; impact of changing conditions on engineered transport infrastructure slopes in Europe. *Quarterly Journal of Engineering Geology and Hydrogeology*, 51(2), 156-168.

Tornborg, J., Karlsson, M., Karstunen, M. 2023. Permanent sheet pile wall in soft sensitive clay. Case study. *Journal of Geotechnical and Geoenvironmental Engineering*, 149(6), 05023003.

Tornborg, J., Karlsson, M., Kullingsjö, A., Karstunen, M. 2021. Modelling the construction and long-term response of Göta Tunnel. *Computers and Geotechnics*, 134, 104027.

Vepsäläinen, P.,, Lojander, M. , Koskinen, M. 1997. *Competition to calculate settlements at Haarajoki test embankment competition programme, competition materials*, FinnRA, Finland.

Vogler, U. 2009. *Numerical modelling of deep mixing with volume averaging technique*. Ph.D. thesis. The University of Strathclyde.

Wheeler, S. J., Karstunen, M., Näätänen, A. 1999. Anisotropic hardening model for normally consolidated soft clays. *Numerical Models in Geomechanics* (pp. 33-40). A.A. Balkema, Rotterdam.

Wheeler, S. J., Näätänen, A., Karstunen, M., Lojander, M. 2003. An anisotropic elastoplastic model for soft clays. *Canadian Geotechnical Journal*, 40(2), 403-418.

Yin, Z. Y., Karstunen, M., Chang, C. S., Koskinen, M., Lojander, M. 2011. Modeling time-dependent behavior of soft sensitive clay. *Journal of Geotechnical and Geoenvironmental Engineering*, 137(11), 1103-1113.

Zuada Coelho, B., Dijkstra, J., Karstunen, M. 2021. Viscoplastic cyclic degradation model for soft natural soils. *Computers and Geotechnics*, 135, 104176.

Zöhrer, A., Wehr, W. Stelte, M. 2010. Is ground engineering environmentally friendly? Ecological balance of foundation engineering methods. *Proc. 11th International EFFC-DFI Conference*, Session 3: Sustainability in the Foundation Industry.