Modelling rate-dependent behaviour of structured clays

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ABSTRACT: Due to the desire of reducing the embedded CO2 in construction and the pressure in public finances to get more value for money in big infrastructure projects, the demands for the accuracy of deformation predictions increase. Instead of piling, alternative environmentally friendly and cost effective solutions, such as preloading via surcharge, vertical drains and column methods, such as deep-mixing, are becoming increasingly attractive. Installation of piles and ground improvement into the ground will modify the state of the soil. This is sometimes beneficial, and sometimes detrimental, and so far this effect has been rarely taken into account. One reason for this is that the numerical techniques and the constitutive soil models have not been able to represent the changes in soil structure and state in a satisfactory manner. The aim of the European GEO-INSTALL project has been to develop numerical techniques that can be used to model installation effects in geotechnical engineering. A key part of this has been constitutive model development, and their robust implementation. The aim of this paper is to discuss some recently developed rate-dependent constitutive models for structured clays, which formed the basis for new developments, resulting in a new rate-dependent model able to represent the complex rate-dependent stress-strain behaviour of soft structured clays. The importance of modelling key features of soil behaviour in the context of rate-dependency are discussed in the light of experimental evidence, and demonstrated through a series of numerical benchmark simulations.

1 INTRODUCTION

Infrastructure construction of soft soils relies on representative predictions on long-term deformations, as often the serviceability considerations are controlling the design solution. This is particularly true for construction of embankments on soft soils. For example, in Sweden the earthwork and foundation costs are typically over 20% of the total construction cost (SGI 1995), and can form even a higher proportion than that in major infrastructure projects, due to the soft ground conditions. With the global drive to reduce the embedded CO2 in construction and the increasing challenges in both public and private finance, there is increasing need to consider design options that can be used as alternatives to rather CO2-heavy and relatively expensive solutions, such as piling. Hence, methods such as preloading, vertical drains, deep mixing and stone columns, are becoming increasingly attractive. This puts additional demands on the accuracy of deformation predictions. In addition, more and more construction occurs in densely populated urban areas on poor ground conditions and the effects on structures nearby is important. This creates new demands for soil modelling and numerical analyses, as due to the complexity of the materials involved, and the complexity of the actual problems, no analytical solutions exist.

Most natural soils, perhaps with the exception of extremely fibrous peats, can be considered as particulate materials, which when fully saturated consist of an assembly of soil particles surrounded by pore water. The strength and stiffness of the soils depends on the arrangement and packing of the soil particles, in particular the assembly of the intergranular contacts, as well as the presence of any apparent bonding between the particles. The latter results from natural bio-chemical processes such as the precipitation of calcites by bacteria in the soil and/or some complex geo-chemical possesses associated with sedimentation environment and subsequent geological and flow history.

Particularly fascinating is this aspect are the so-called sensitive clays, which in extreme cases are referred to as quick clays. Highly sensitive clays can be found in large areas of Northern Europe and North America (Rankka 2003), and they tend to change from rather stiff consistency to a flowing liquid gel when disturbed, forming a potential geohazard. Sensitive clays were formed when clay particles and other fine fractions were sedimented in cold and salty marine environment after the Pleistocene epoch. Due to the bi-polar electro-chemical charges associated with the clay minerals, in salty sedimentation environments the clay particles flocculate and form very open card-house structures (see e.g. Pusch 1970). Since their formation the clays have been consolidating and creeping under their self-weight. Due to leaching, the salt ions at the surface of the clay particles have dissolved. The possible leaching processes involve percolating water through the deposit, water seeping upwards through the deposit due to artesian pressure, and diffusion of salts towards zones with lower ion concentrations (Rankka et al. 2004). This way the initially stable structure of the clays has become meta-stable and sensitive.

When piles or any column-type of ground improvement are installed in the ground, there are large deformations in the soils, involving both shearing and the lateral expansion of the soil. Consequently, there are changes in the vertical and horizontal effective stresses, and the parameters associated with the state of the soil, such as void ratio, preconsolidation pressure, density, K_0 (coefficient of earth pressure at rest) etc. are modified. As demonstrated by Dijkstra et al. (2010, 2011) using photoelastic techniques, installation causes significant density changes and rotations of principal stresses. The former are of course beneficial, but effects of the latter on soil state are usually ignored. In granular media, pile driving may also cause particle crushing Lobo-Guerrero & Vallejo (2005) and breakage of the apparent bonding in the soil, which in turn may have a detrimental effect on the pile capacity.

In this paper, the focus is on structured natural clays, and modelling the changes in their stress-strain response due to loading and possible installation effects.

2 CONSTITUTIVE MODELLING OF STRUCTURED CLAYS

The response of clays is dominated by their microstructure, which has been studied with the help of reconstituted clays samples given the difficulties in micromechanical testing of intact natural clays. During irrecoverable straining for kaolin clay there is evidence on re-orientation of particles, and changes in particle contacts, at microstructural level (see e. g. Bai & Smart 1996, Hicher et al. 2000). This supports the macro-level evidence that the anisotropy of clays, when undergoing large strains, can evolve.

Based on experimental studies on natural Otaniemi clay, Wheeler et al. (2003) proposed an elastoplastic model, called S-CLAY1, with a rotational hardening law describing the changes in the inclination of the yield surface due to irrecoverable straining. The calibration of the parameters for the S-CLAY1 model is rather straightforward and the model has been thoroughly validated experimentally by Karstunen and her co-workers (Karstunen & Koskinen, 2004, 2008).

As shown by i.e., Burland (1990) & Leroueil & Vaughan (1990), when natural clays are undergoing large deformations, the initial apparent bonding is progressively lost, and at large strains, soil starts to behave as a fully destructured material. The degradation of bonding due to irrecoverable straining is referred to as destructuration (Leroueil et al. 1979). Most constitutive models that attempt to account for the effect of bonding and destructuration, utilize the conceptual ideas by Gens and Nova (1993). The same applies to the S-CLAY1S model (Karstunen et al. 2005), in which the sudden collapse of the sensitive clay structure upon yielding is represented using the concept of intrinsic yield surface (Gens and Nova 1993) in combination with the rotational hardening of the S-CLAY1 model (Wheeler et al. 2003). This was the first constitutive clay model that was able to represent the changes in the state of the structured clays in a simple comprehensive manner. The next stage was to incorporate rate-effects into the model.

Given it was thought that anisotropy and its evolution has a major effect on the soft clay response, the S-CLAY1 model was extended to account for creep and rate effects by Leoni at al. (2008) using the creep formulation by Vermeer and his co-workers (Vermeer et al. 1998, Vermeer & Neher 1999). The resulting model, called ACM (Anisotropic Creep model), has the advantage that the model parameters are easy to derive. Most importantly, the concept of reference time or time shift (see Leoni et al. 2008) enables, for the first time a systematic way to take into account the strain-rate in the tests that are used for defining the model input parameters. This is a very powerful feature of the model, which gives it a major advantage over the so-called overstress models, based on Perzyna's (1963) over-stress theory, proposed by e.g. Hinchberger & Gu (2009) and Karstunen & Yin (2010).

As discussed the in following, and pointed out by Grimstad at al. (2010) and Karstunen & Yin (2010), some of the key assumptions in the ACM model are, however, both inconsistent with experimental evidence and fundamentally wrong (see also the companion paper by Sivasithamparam et al. 2013). As a consequence, in practical context, the ACM model often ends up grossly overpredicting deformations in structured soft soils (see Karstunen et al. in press), when the values of soil constants are objectively selected. This affects the predictive ability of the model and prevents its use in practical context on structured soil deposits.

The work by Hinchberger & Gu (2009), Karstunen & Yin (2010) and Yin et al. (2011), demonstrates that for predicting certain features of natural clay behavior, such as tertiary creep and creep rupture, or indeed the phenomenon of progressive failure, it is also necessary to account for the effects of bonding and destructuration. In the following some features of the ACM model are highlighted, which necessitated the development of a new rate-dependent model Creep-SCLAY1 (for details see the companion paper Sivasithamparam et al. 2013). The importance of modelling key features of soil behaviour in the context of rate-dependency are discussed and demonstrated through a series of numerical benchmark simulations.

3 ACM AND CREEP-SCLAY1 MODELS

The creep formulation in the ACM model (Leoni et al. 2008) is based on the idea of a Normal Consolidation Surface (NCS), see Figure 1, which is treated as the contour of constant volumetric creep strain rate. For simplicity, the model is plotted in Figure 1 in triaxial stress space, in terms of mean effective stress *p'* and deviator stress *q*. The scalar α represents the current degree of anisotropy, *M* is the stress ratio at critical state and ψ is the dilatancy angle.

The volumetric creep strain rate is given by a simple power law as follows:

$$\dot{\varepsilon}_v^c = \frac{\mu}{\tau}\left(\frac{p'_{eq}}{p'_p}\right)^{\beta} \quad \text{with} \quad \beta = \frac{\lambda^* - \kappa^*}{\mu^*} \tag{1}$$

where μ^* is the modified creep index, λ^* in the modified compression index, κ^* is the modified swelling index and τ is the reference time (see Leoni et al. 2008 for details). The ratio in the brackets is an inverse of a generalised yield stress ratio. Hence, creep is occurring even within the overconsolidated region, when the current stress surface (CSS) in Figure 1, is smaller than the NCS.

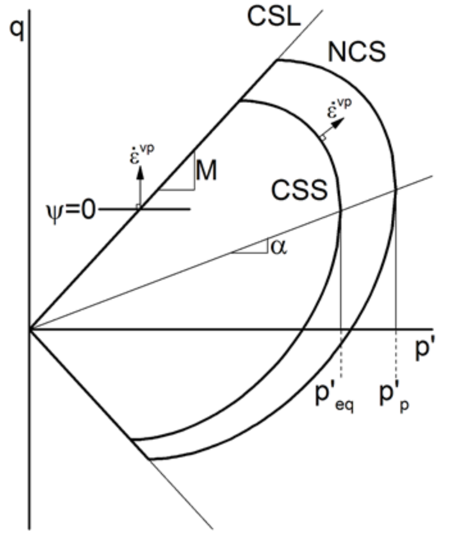

Figure 1. Current state surface (CSS) and normal consolidation surfaces (NCC) of the ACM model and the direction of visco-plastic strains (triaxial stress space).

In the ACM model, there is a separate failure surface on which zero dilatancy condition is imposed in order to comply with the condition of zero volumetric strain rate at critical state. Hence, the there is no "dry side" of the critical state. Because an associated flow is assumed on the CSS, there is actually a "jump" in the volumetric creep rate when approaching critical state (see Figure 1): the constant volumetric creep rate at the CSS surface suddenly reduces to zero (results of simulations showing this can be found in Sivasithamparam et al. 2013).

Most importantly, because the volumetric creep strain rate is assumed to be constant, large volumetric creep strains and very large deviatoric creep strains are predicted in the stress space between the K_0 line and critical state line. As shown in the simulations in this paper, this results in severe overprediction of deformations at boundary value problems. Because of these assumptions, quite significant apparent strain-softening is predicted in undrained simulations of shearing of normally consolidated or lightly overconsolidated samples. Furthermore, as shown by Sivasithamparam et al. (2013), the ACM model cannot model the isotach behaviour that is observed in soft clays caused by a step-change in strain rate. This is not satisfactory, and these insufficiencies inspired further model developments.

In the Creep-SCLAY1 model (Figure 2), instead of assuming constant volumetric creep strain rate, similarly to Grimstad et al. (2010) and Yin et al. (2011), it is assumed that the viscoplastic multiplier is constant along the CSS. The viscoplastic multiplier is defined in such a way that the model gives identical volumetric creep strain rate to Eq. (1) under oedometric loading (see Grimstad et al. (2010) for details) as:

$$\dot{\varepsilon}_v^c = \frac{\mu}{\tau}\left(\frac{p'_{eq}}{p'_p}\right)^{\beta}\left(\frac{M^2 - \alpha_{K_0^{NC}}^2}{M^2 - \eta_{K_0^{NC}}^2}\right) \tag{2}$$

where $\eta = q/p'$ is the stress ratio, and subscript K_0^{NC} refers to the normally consolidated K_0 stress path. The corresponding strain rate vectors are plotted in Figure 2. Due to the evolution of anisotropy, the Creep-SCLAY1 model predicts some minor apparent strain softening in undrained shearing in normally consolidated region. Because there is now a "dry side", with increasing strain rate it is also possible for the stress path to pass over the critical state line, which is in agreement with experimental evidence (see simulations in Sivasithamparam et al. 2013).

For finite element analyses, the model needs to be generalised. Instead of a scalar α, the anisotropy is described with a fabric tensor, and invariants can no longer be used (see Wheeler at al. 2003 for details). In order to account for Lode angle dependency, instead of the Drucker-Prager model that assumes constant *M*, the formulation of Sheng et al.

(2000) has been adapted, in order to have a smooth variation of the Lode angle of the fabric in the general stress space as the CSS and NCS rotate (see Figure 3). In order to incorporate the effect of bonding and destructuration in some of the simulations, an intrinsic surface has been added together with a destructuration law that is analogous with the S-CLAY1S model (Karstunen at al. 2005). In the following, this version of the model is referred to as the Creep-SCLAY1S model. It is basically a rate-dependent model that accounts for the changes in fabric arrangement and bonding, enabling a rather complete representation of the stress-strain behaviour of structured clays.

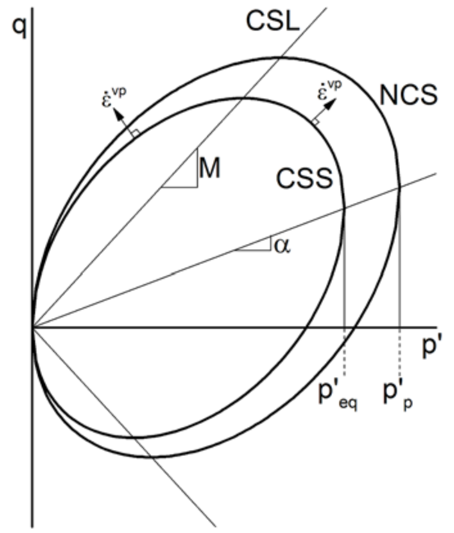

Figure 2. Current state surface (CSS) and normal consolidation surfaces (NCC) of the Creep-SCLAY1 model and the direction of viscoplastic strains (triaxial stress space).

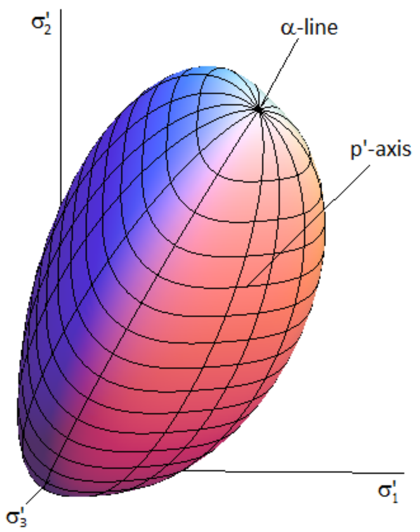

Figure 3. The Creep-SCLAY1 model (general stress space).

The usability of a constitutive model in a practical context relies on its robust implementation into the finite elements. The Creep-SCLAY1 model is implemented into the PLAXIS (Brinkgreve et al. 2010) finite element code as a user-defined soil model (UDSM) using a Modified Newton-Raphson scheme (Sivasithamparam 2012), whilst the ACM model is using a fully implicit Newton iteration (Leoni et al. 2008). In the following the model is applied to a typical benchmark problem, a simple embankment.

4 BENCHMARK SIMULATIONS

4.1 *FE model and model input*

The performance the ACM model and the Creep-SCLAY1 model are demonstrated in a benchmark problem using the 2D PLAXIS finite element code. An embankment constructed on soft soil is assumed to be 2 m high, with a width at the top of 10 m and the side slopes with a gradient of 1:2. The soft soil is assumed to have the properties of soft Bothkennar clay (Symposium 1992), which extends to a depth of 30m. The groundwater table assumed to be located at 1 m below the ground surface. At the surface there is a 1m depth over-consolidated dry crust. The problem has been analysed as a small strain simulation. The values for the model parameters for the soft clay layer are shown in Table 1. The embankment, assumed to be made of granular material, was modelled with a simple Mohr Coulomb model and the same applies to the dry crust layer (see Table 2 for material parameters). This problem is expected to be dominated by the soft soil response and is not sensitive to the embankment and crust parameters.

Table 1. Model parameters for the soft clay.

Parameter	e_0	OCR	K_0	α_0	χ_0
Soft clay	2.0	1.50	0.50	0.59	8

Parameter	γ [kN/m³]	κ^*	ν'	λ^*	M
Soft clay	16.5	6.67E-3	0.20	0.10	1.5

Parameter	λ^*_i	ω_d	ω	ζ	ζ_d
Soft clay	0.06	1.0	50	9	0.2

Layer	μ^*_i	μ^*	τ [days]
Soft clay	2.0E-3	5.07E-3	1

Table 2. Model parameters for the embankment and the dry crust.

Layer	γ [kN/m³]	E [kPa]	φ' [°]	c' [kPa]	ν'	ψ [°]
Embankment	20	40000	40	2	0.35	0
Dry crust	19	3000	30	6	0.20	0

The most difficult model parameter to determine is constant ω representing the evolution of anisotropy. Hence, for comparison the simulation is also run with a version of Creep-SCLAY1 model where the anisotropy has been fixed, i.e. only initial anisotropy is considered, with no evolution of anisotropy. This assumption was made e.g. by Bodas Freitas et al. (2011) in their rate-dependent model. Such a model would of course not be able to reproduce the stress-strain response in cases where there are major changes in the stress path directions, and in principle would not be able to reproduce the element level tests done by Karstunen & Koskinen (2008).

4.2 *Results*

The predicted vertical displacements by ACM and Creep-SCLAY 1 as function of time are plotted in Figure 4 and the corresponding settlement through at the end of construction and at the end of consolidation have been plotted in Figure 5. The settlement predicted by both models are huge (unrealistically large considering that the embankment is only 2 m high. The results clearly demonstrate that the problem is not a one-dimensional problem, as for K_0 consolidation the predictions by the two models would have been the same.

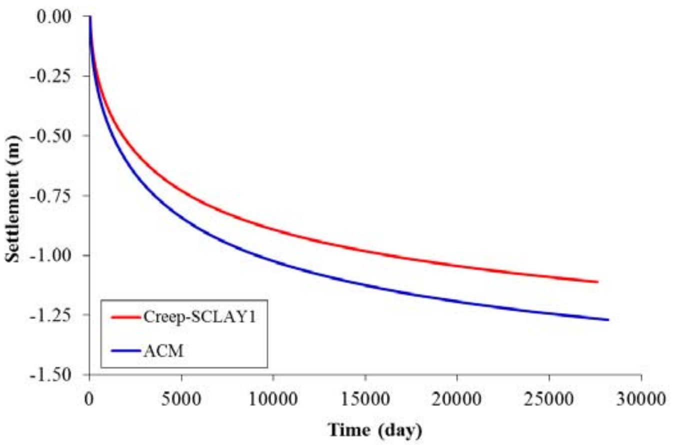

Figure 4. Predicted vertical displacements as a function of time by ACM and Creep-SCLAY1.

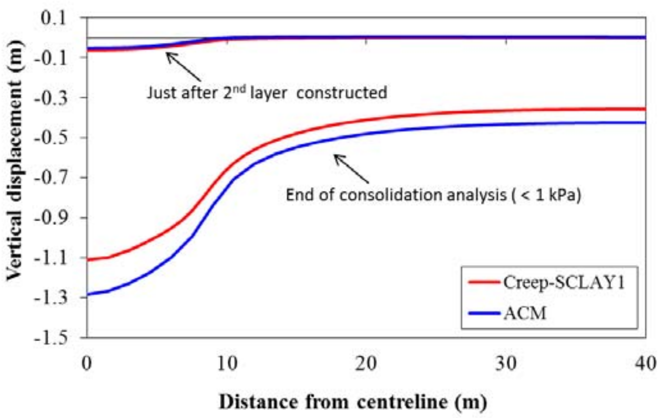

Figure 5. Predicted vertical settlement through by ACM and Creep-SCLAY1.

The introduction of the constant viscoplastic multiplier, is clearly improving the results, and this is even more apparent in Figure 6, where the lateral deformations under the toe of the embankment predicted by the two models have been plotted. The assumption in ACM model about CSS being the contour of constant volumetric creep rate and combining that with associated flow rule simply results in much higher shear strain prediction, and hence also larger horizontal deformations than predicted by Creep-SCLAY1. Both models predict notable deformations outside the loaded area (see Figure 5), which of course is not satisfactory.

In order to investigate what the effect of the evolution of anisotropy is, the benchmark problem has been re-analysed with Creep-SCLAY1 model with the evolution of anisotropy switched off ($\omega = 0$). As a consequence, no rotation of the CSS and NCS is allowed. The predictions of the vertical and horizontal deformations by this version of the model are high (Figures 7, 8 and 9), which demonstrates that the evolution of anisotropy dissipates plastic energy, so any predictions that ignore that are essentially over-conservative. Again, as would be expected, far too large deformations are triggered by the in situ stresses only.

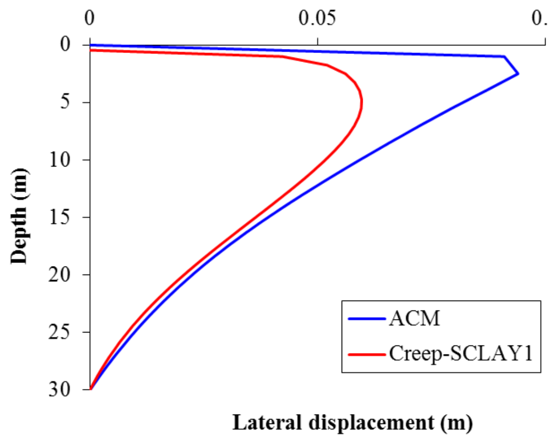

Figure 6. Predicted lateral displacements by ACM and Creep-SCLAY1 at the end of consolidation.

When considering installation effects, accounting for changes in anisotropy is extremely important. Castro & Karstunen (2010) and Castro et al. (2013) simulate stone column installation in Bothkennar clay by modelling the problem as a cavity expansion with S-CLAY1S model. Their results show that due to the installation for typical centre to centre spacings the soil fabric flips totally over. When the actual structure is then constructed, the fabric needs to rearrange once again. Preliminary analyses suggest that the resulting deformations might actually decrease up to 25%, all due to this dissipation of energy due to evolution of anisotropy.

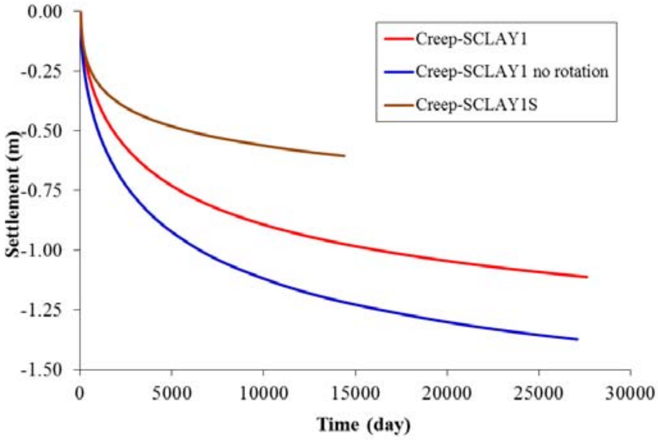

Figure 7. Predicted vertical displacements as a function of time by Creep-SCLAY1, Creep-SCLAY1 with no rotation and Creep-SCLAY1S.

One problem with creep models that do not account for the effect of bonding and destructuration is that both the compression index and the modified creep index are assumed to be constant. The experimental data on structured clays however clearly demonstrates that neither of these are constants for structured natural clays. Due to the gradual degradation of bonds, both the apparent compression index and the apparent creep index have maximum values just after the onset of yield (see e.g. data by Graham et al. 1983). Indeed, even their ratio (and consequently exponent β in Eq. 1) is not constant. As shown by Karstunen & Yin (2010) this type of behaviour can be represented with a rate-dependent model that accounts for the effect of bonding and its gradual destruction.

In order to explore the effect of bonding and destructuration at boundary value level, the benchmark was simulated with Creep-SCLAY1S model, i.e. an extension of the Creep-SCLAY1 model that accounts additionally for initial bonding and the subsequent degradation of bonding. The results have been included in Figures 7, 8 and 9, labelled Creep-SCLAY1S. The predicted vertical and horizontal deformations have reduced by nearly 45% and 55% respectively, and most importantly the unrealistic large deformations due to the in situ stresses only have virtually vanished.

CONCLUSIONS

Installation of piles and ground improvement into the ground will modify the state of the soil. This sometimes beneficial, and sometimes detrimental, and so far this effect has been rarely taken into account. One reason for this is that the numerical techniques and the constitutive soil models have not been able to represent the changes in soil structure and state in a satisfactory manner. The paper discussed some recently developed rate-dependent constitutive models for structured clays, culminating in a new rate-dependent model.

The numerical simulations show that the Creep-SCLAY1 model (Sivasithamparam et al. 2013), when extended to account for the effects of apparent bonding and destructuration, appears to have features that enable realistic predictions of rate-dependent stress-strain behaviour of soft structured clays. Furthermore, the results and the experimental evidence so far suggest that when modelling rate-dependent behaviour of structured clay, it is essential to account for both evolution of anisotropy and the effect of bonding and destructuration, in order to have a realistic rate-dependent constitutive model.

Creep and rate effects can be considered as microstructural reaction to stress increment (Pusch 2012). Although the micromechanics of structured clays is complex, and our sampling and experimental techniques have not yet developed to explore this fully, the simulations in this paper suggest that it is possible to account for these microstructural effects by advanced macroscopic constitutive models. These models have also the advantage that they enable us to improve our understanding of what might be happening, when piles and ground improvement elements are installed into soft structured clays.

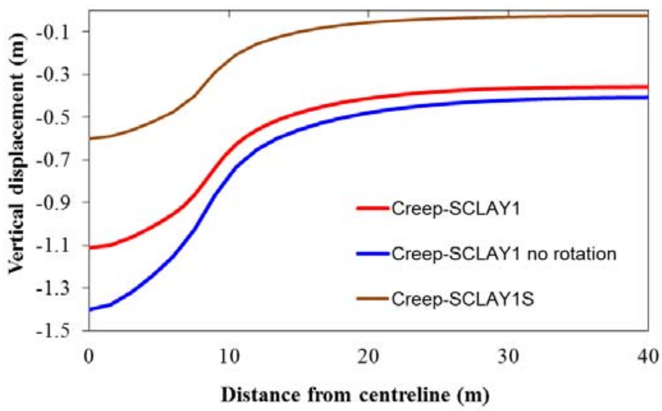

Figure 8. Predicted vertical settlement through by Creep-SCLAY1, Creep-SCLAY1 with no rotation and Creep-SCLAY1S.

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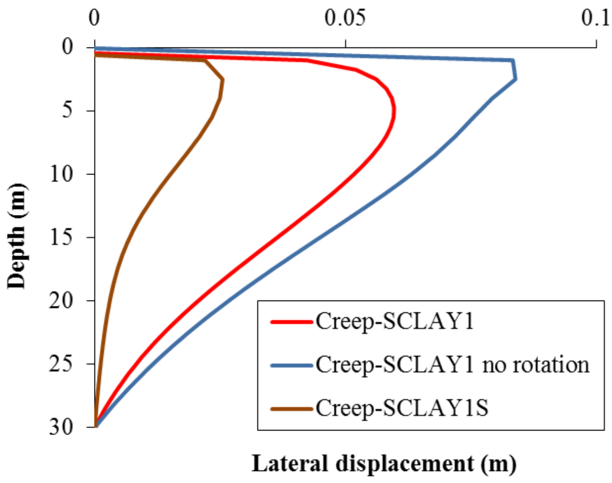

Figure 9. Predicted lateral displacements by Creep-SCLAY1, Creep-SCLAY1 with no rotation and Creep-SCLAY1S at the end of consolidation.

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