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PLAXIS The Creep-SCLAY1S model 2018

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TABLE OF CONTENTS

1 Introduction 4

2 The Creep-SCLAY1S constitutive model 5
2.1 Strain decomposition 5
2.2 Elastic response 5
2.3 Flow rule 6
2.4 Yield surface 7
2.5 Hardening rules 11
2.6 Dependency of M from the Lode angle 12

3 Parameter determination 14
3.1 Initial state variables 14
3.2 Isotropic stiffness parameters 15
3.3 Creep parameters 16
3.4 Critical state parameters 16
3.5 Anisotropy parameters 17
3.6 Bonding parameters 19
3.7 Summary of model parameters 20
3.8 Output state parameters 20

4 Laboratory tests simulations 21
4.1 Rate dependency 21
4.2 Bonding and destructuration 24

5 Practical Applications 28
5.1 One-dimensional consolidation: comparison with soft soil and soft soil creep 28
5.2 Embankment on soft soil 31

6 Conclusions 35

7 References 36

1 INTRODUCTION

The Creep-SCLAY1S model is capable of simulating the anisotropic and rate-dependent behaviour of soft soils. The mathematical formulation also includes the possibility to model bonding and destructuration. Many constitutive models have been proposed in literature to describe the behaviour of structured natural clays. Most of the rate-dependent models were developed based on the Perzyna's (Perzyna (1963) and Perzyna (1966)) overstress theory (e.g. Hinchberger & Rowe (2005) and Zhou, Yin, Zhu & Cheng (2005)), with the disadvantage of a difficult parameter calibration process (Karstunen & Yin (2010)). Another approach consists in using empirical formulations, as for example the Soft Soil Creep model available in the PLAXIS standard library. The Creep-SCLAY1S can be considered a special kind of extended overstress model derived from an empirical equation (Sivasithamparam, Karstunen & Bonnier (2015)).

An overview of the developments that have lead to the implementation of the Creep-SCLAY1S model is described here. Based on the observations of the re-orientation of clay particles due to irreversible strains, the changes in the inclination of the yield surface have been introduced through a rotational hardening law in the elastoplastic model S-CLAY1 (Wheeler, Näätänen, Karstunen & Lojander (2003)). Later, the model has been extended to include destructuration. In fact, as shown by Leroueil & Vaughan (1990), natural clays are characterized by an initial bonding which is progressively lost for increasing deformations until the soil behaves as a destructured material at large strains. The Creep-SCLAY1S model (Koskinen, Karstunen & Wheeler (2002)) is an extension of the S-CLAY1 model and uses the intrinsic yield surface (Gens & Nova (1993)) to model the sudden collapse of the sensitive clay structure upon yielding (Karstunen, Sivasithamparam, Brinkgreve & Bonnier (2013)).

To include creep and rate effects, the Anisotropic Creep Model has been developed by Leoni, Karstunen & Vermeer (2008) based on the S-CLAY1 model. The model accounts for the initial anisotropy and its modification due to irrecoverable strains, and uses rotated ellipses that evolve with the volumetric creep strains. Even though the ACM introduces some features (as e.g. the reference time) that represent a major advantage compared to the overstress models, some of its key assumptions are inconsistent with experimental evidence.

The Creep-SCLAY1S model overcomes the limitations of the ACM model, as it will be illustrated in the next chapter, and includes the effect of destructuration, as in the S-CLAY1S model.

The model is implemented as a user-defined soil model (UDSM) in PLAXIS. Before starting PLAXIS, the *creep-sclay1s.dll* and the *creep-sclay1s64.dll* have to be placed in the *udsm* sub-folder contained in the PLAXIS installation folder. When creating the new material data set, the *User-defined* option should be selected through the *Material model* combo box in the *General* tabsheet of the *Material sets* window. In the *Parameters* tabsheet, the *creep-sclay1s64.dll* should be selected as the *DLL file* from the drop-down menu. The *Creep-SCLAY1S* is used as the *Model in DLL*.

This document consists of five sections (including the current Chapter 1):

- the formulation of the constitutive model is described in Chapter 2;
- the determination of the model parameters is discussed in Chapter 3;

- some laboratory tests are simulated to show the main features of the model and the variation of some model parameters in Chapter 4;
- application examples are presented in Chapter 5;
- conclusions are drawn in Chapter 6.

2 THE CREEP-SCLAY1S CONSTITUTIVE MODEL

The Creep-SCLAY1S model (Sivasithamparam (2012)) has been implemented as a user-defined soil model in PLAXIS. The model is developed to simulate the anisotropic and rate-dependent behaviour of structured soils. For the sake of simplicity, the mathematical formulation of the model is presented here in triaxial space, while the model has been implemented as a full 3D model. For the triaxial space, the mean effective stress p' and the deviatoric stress q are expressed by Eq. (2.1) and Eq. (2.2), respectively:

$$p' = \frac{(\sigma'_a + 2\sigma'_r)}{3} \tag{2.1}$$

$$q = \sigma'_a - \sigma'_r \tag{2.2}$$

and the volumetric and deviatoric strain components are given by Eq. (2.3) and Eq. (2.4), respectively:

$$\varepsilon_v = \varepsilon_a + 2\varepsilon_r \tag{2.3}$$

$$\varepsilon_q = \frac{2}{3}(\varepsilon_a - \varepsilon_r) \tag{2.4}$$

The subscripts a and r refer to the axial and radial direction in the triaxial stress space.

2.1 STRAIN DECOMPOSITION

As in the classical elasto-plasticity theory, the total strains are expressed as a combination of elastic and inelastic components. In this formulation, the inelastic component is assumed to be purely viscous. The elastic and creep parts of the strain in the model are combined with an additive law for both the volumetric and the deviatoric strain:

$$d\varepsilon_v = d\varepsilon_v^e + d\varepsilon_v^c \tag{2.5}$$

$$d\varepsilon_q = d\varepsilon_q^e + d\varepsilon_q^c \tag{2.6}$$

where the $d\varepsilon$ implies differentiation with respect to time (rate), while the superscript e and c refer to the elastic and creep component, respectively.

2.2 ELASTIC RESPONSE

The Creep-SCLAY1S model assumes that there is no purely elastic domain, in contrast to the classic overstress theory, and allows for creep within the Normal Consolidation Surface (NCS). This means that even small increases of stresses are likely to cause yielding. Therefore, irreversible deformations are dominating in many problems of practical interest, while elastic strains are mainly observed during unloading and recompression. The elastic behaviour is assumed to be isotropic with the elastic strain

components given by Eq. (2.7) and Eq. (2.8):

$$d\varepsilon_v^e = \frac{dp'}{K} \tag{2.7}$$

$$d\varepsilon_q^e = \frac{dq}{3G} \tag{2.8}$$

The stiffness is expressed by the stress-dependent elastic bulk modulus K and elastic shear modulus G:

$$K = \frac{p'}{\kappa^*} \tag{2.9}$$

$$G = \frac{3p'}{2\kappa^*}\frac{1-2\upsilon'}{1+\upsilon'} \tag{2.10}$$

The stiffness moduli depend on the Poisson's ratio υ' and the modified swelling index κ^*, determined as the slope of the elastic swelling line in the $\varepsilon_v - ln(p')$ plane. More information on the swelling index can be found in Section 3.2 of Chapter 3.

The model cannot simulate the non-linearity of the small strain stiffness and, therefore, it is not appropriate for dynamic applications. This is acceptable since the model is intended to simulate the behaviour of soft clays in many practical problems, where the primary requirement for accurate numerical analysis is the possibility to successfully model the onset of large plastic strains and the subsequent pattern of plastic straining (Wheeler, Näätänen, Karstunen & Lojander (2003)).

2.3 FLOW RULE

The Creep-SCLAY1S model assumes an associated flow rule (with irreversible strain increments perpendicular to the yield surface). Experimental evidence suggests that this is a reasonable assumption for natural clays when combined with an inclined yield surface (Wheeler, Näätänen, Karstunen & Lojander (2003)) and a rotational hardening law (Eq. (2.18)). The creep strain rates are not constant but are calculated as:

$$d\varepsilon_v^c = d\Lambda\frac{\partial p'_{eq}}{\partial p'} \tag{2.11}$$

$$d\varepsilon_q^c = d\Lambda\frac{\partial p'_{eq}}{\partial q} \tag{2.12}$$

where $d\Lambda$ is the viscoplastic multiplier that is constant along the current state surface. It is expressed by the following formula:

$$d\Lambda = \frac{\mu^*}{\tau}\left(\frac{p'_{eq}}{p'_p}\right)^{\beta}\left(\frac{M(\theta_\alpha)^2 - \alpha^2_{K_0^{NC}}}{M(\theta_\alpha)^2 - \eta^2_{K_0^{NC}}}\right) \tag{2.13}$$

where μ^* is the modified creep index, τ is the reference time, p'_{eq}/p'_p represents the inverse of the overconsolidation ratio OCR, β is the creep exponent, $\alpha_{K_0^{NC}}$ defines the inclination of the ellipses in the normally consolidated state and $\eta_{K_0^{NC}}$ the value of the

stress ratio in K_0^{NC} loading. The ratio $\left(\frac{M(\theta_\alpha)^2 - \alpha^2_{K_0^{NC}}}{M(\theta_\alpha)^2 - \eta^2_{K_0^{NC}}}\right)$ ensures that, under oedometer conditions, the resulting creep strain corresponds to the measured volumetric creep strain rate. The modified creep index μ^* depends on the one-dimensional secondary compression index C_α as shown in Section 3.3 of Chapter 3. The reference time τ is equal to 1 (i.e. 1 day) if the normal consolidation surface is derived from a 24 h oedometer test or when modelling a boundary value problem where the OCR value has been determined based on a 24 h oedometer test. The creep exponent β is defined as:

$$\beta = \frac{\lambda^* - \kappa^*}{\mu^*} \tag{2.14}$$

where λ^* is the modified compression index.

It is important to notice that the rate of the plastic multiplier $d\Lambda$ has been derived based on an empirical equation, rather than a consistency rule. For this reason a stress state outside the normal consolidation surface is allowed.

2.4 YIELD SURFACE

The creep formulation of the Creep-SCLAY1S model is based on the definition of two surfaces: the Normal Consolidation Surface (NCS) and the Current State Surface (CSS) (Figure 2.1). The first one represents the boundary between small and large creep strains, the second one represents the current state of effective stress. They both have the shape of a sheared ellipse in the $p' - q$ plane, defined by Eq. (2.15):

$$p'_m = p' + \frac{(q - \alpha p')^2}{\left(M(\theta_\alpha)^2 - \alpha^2\right) p'} \tag{2.15}$$

where p'_m defines the size of the curve, i.e. the intersection of the vertical tangent to the ellipse with the p' axis, and it is equal to p'_{eq} for CSS and p'_p for NCS, $M(\theta_\alpha)$ is the stress ratio at critical state (dependent on the modified Lode angle θ_α as described in Section 2.6), α is a scalar quantity that describes the inclination of the surfaces as a measure of the plastic anisotropy of the soil. When p'_{eq} is equal to p'_p (vertical preconsolidation pressure), the current stress lies on the normal consolidation surface. When α is equal to zero, the behaviour is isotropic and Eq. (2.15) corresponds to the Modified Cam Clay yield curve. Experimental evidence shows that soft clays are characterized by a certain degree of anisotropy (Leroueil & Vaughan (1990)), where the maximum mean effective yield stress does not lie on the mean effective stress axis p' but it occurs at the presence of deviatoric stresses (Figure 2.2).

In addition to the real yield surface for the natural material with bonding, another surface identified as "intrinsic yield surface" is introduced, based on the general framework proposed by Gens and Nova (1993) to incorporate bonding and destructuration within elasto-plastic constitutive models (Figure 2.3). The surface is of the same shape and orientation as the real yield surface for the natural bonded soil, but is smaller in size. The size of the intrinsic yield surface is specified by the parameter p'_{mi}:

$$p'_m = (1 + \chi) p'_{mi} \tag{2.16}$$

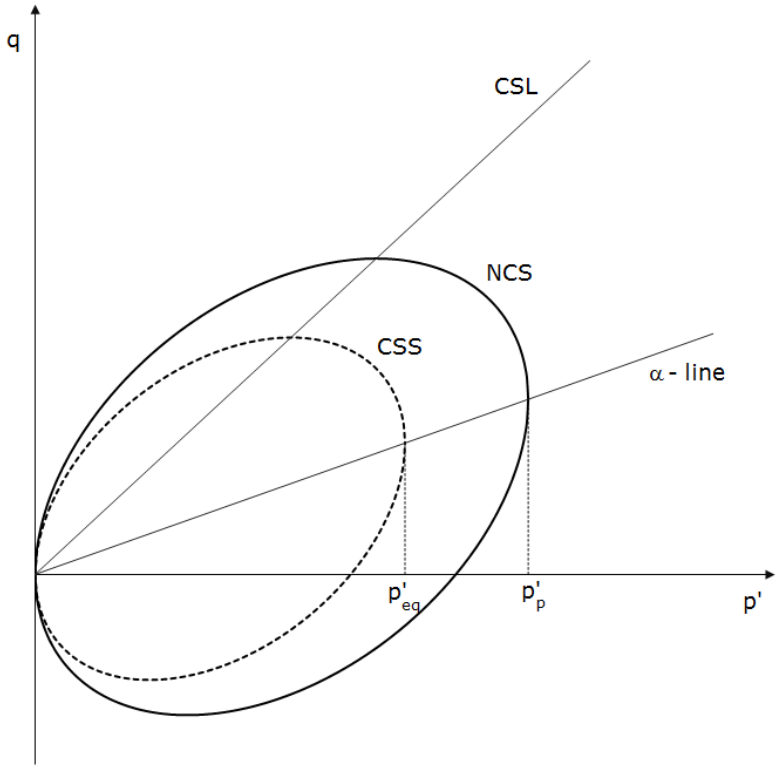

Figure 2.1 Current Stress Surface (CSS) and Normal Consolidation Surface (NCS) of the Creep-SCLAY1S model in the triaxial stress space.

where p'_m defines the size of the yield surface, χ is the amount of bonding and p'_{mi} is the size of the intrinsic yield surface given by the intersection of the vertical tangent to the ellipse with the p' axis. In the Creep-SCLAY1S model p'_m corresponds to p'_p and the ratio (or difference) between p'_p and p'_{mi} is a measure of the bonding effect. The intrinsic yield surface corresponds to the normal consolidation surface in the case of no bonding.

Structured clays are characterized by an initially stiffer behaviour compared to the corresponding normally consolidated state. When the preconsolidation stress is reached, the destructuration process starts and a more or less rapid decrease of the void ratio is observed. When the material has completely lost bonding, it behaves as a normally consolidated clay.

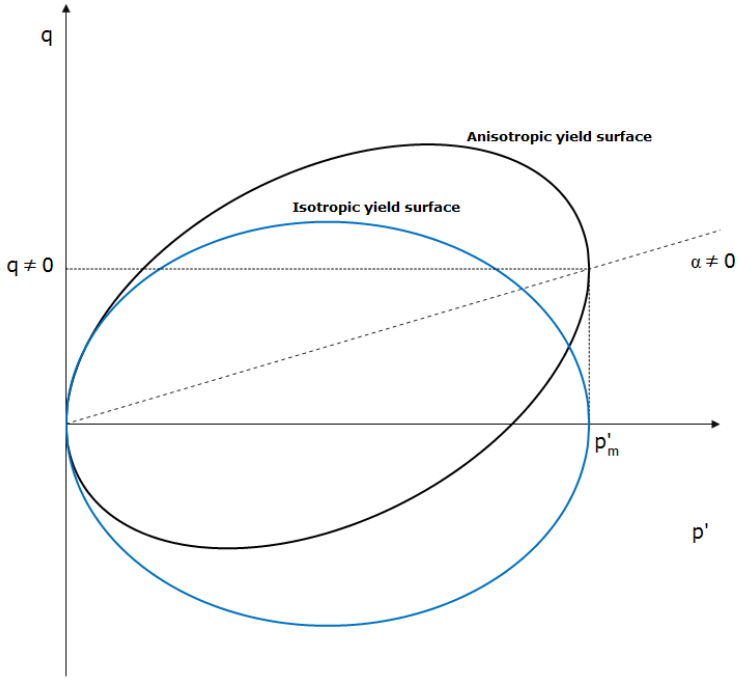

Figure 2.2 Comparison between the isotropic and anisotropic yield surface in the triaxial stress space.

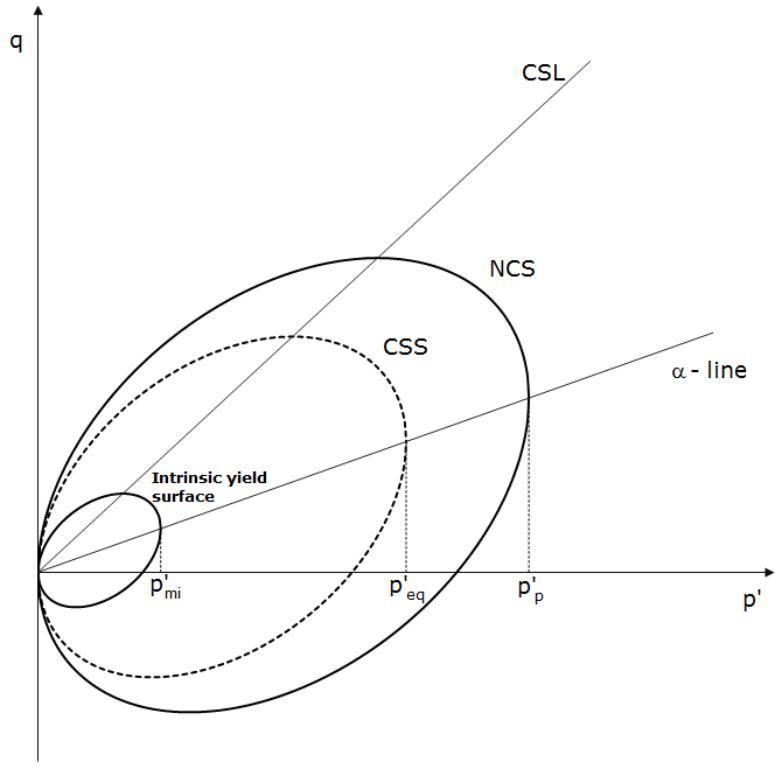

Figure 2.3 Intrinsic yield surface, Current Stress Surface (CSS) and Normal Consolidation Surface (NCS) of the Creep-SCLAY1S model in the triaxial stress space.

2.5 HARDENING RULES

The Creep-SCLAY1S model is characterized by three hardening rules. The size of the normal consolidation surface evolves with the volumetric creep strains ε_v^c according to the hardening law:

$$p'_p = p'_{p0} exp\left(\frac{\varepsilon_v^c}{\lambda^* - \kappa^*}\right) \tag{2.17}$$

where p'_{p0} is the initial isotropic preconsolidation pressure, $\lambda^* = \lambda/(1 + e_0)$ and $\kappa^* = \kappa/(1 + e_0)$ are the modified compression and swelling index, respectively, and e_0 is the initial void ratio. The increase in size of the yield curve is due to rearrangement of the particles to a denser packing arrangement, as represented by irreversible volumetric strains.

The rotational hardening law (Eq. (2.18)) allows to model the evolution of anisotropy due to irreversible strains. It describes the changes in the orientation of the normal consolidation surface with the volumetric and deviatoric creep strains:

$$d\alpha = \omega\left(\left[\frac{3\eta}{4} - \alpha\right]\left\langle d\varepsilon_v^c\right\rangle + \omega_d\left[\frac{\eta}{3} - \alpha\right]\left|d\varepsilon_q^c\right|\right) \tag{2.18}$$

where ω and ω_d are two soil constants, η is the stress ratio q/p', $\langle\cdot\rangle$ are Macaulay brackets around the increment of the creep volumetric strain $d\varepsilon_v^c$ and $\left|d\varepsilon_q^c\right|$ is the norm (absolute value) of the increment of the creep deviatoric strain. The Macaulay brackets are used to indicate that $\left\langle d\varepsilon_v^c\right\rangle = d\varepsilon_v^c$ when $d\varepsilon_v^c > 0$ and $\left\langle d\varepsilon_v^c\right\rangle = 0$ when $d\varepsilon_v^c < 0$. This ensures the possibility for the stress path to cross the critical state line ($\eta > M$), predicting swelling on the dry side of critical state, in agreement with experimental evidence (Sivasithamparam, Karstunen, Brinkgreve & Bonnier (2013)) and overcoming one of the limitations of the anisotropic creep model by Leoni, Karstunen & Vermeer (2008). The soil constant ω controls the absolute rate at which the normal consolidation surface rotates with viscous straining towards its current target value of α. The soil constant ω_d controls the relative effectiveness of creep shear strains and creep volumetric strains in determining the overall current target value for α (Wheeler, Näätänen, Karstunen & Lojander (2003)). The limit case of ω_d equal to zero in the case of loading at constant η results in α asymptotically approaching a target value of $3\eta/4$, with the difference $(3\eta/4 - \alpha)$ decaying at an exponential rate with creep volumetric strains. On the contrary, for ω_d that tends to infinity, α approaches asymptotically a target value of $\eta/3$, with increasing creep deviatoric strains. In general, for a real soil, the target value of the orientation α is a value between $\eta/3$ and $3\eta/4$ and tends to be closer to the lower limit for stress paths where the creep deviatoric strains are dominating (i.e. low values of stress ratio η), and vice versa, it is closer to the upper limit for larger values of η, where the creep volumetric strains are dominant. Eq. (2.18) was developed based on a comprehensive series of tests on Otaniemi clay (Näätänen, Lojander, Wheeler & Karstunen (1999)) and subsequently verified on a more general scale through tests on other clays.

According to the rotational hardening rule, the anisotropy angle α reduces with increasing plastic strains. This modification can be observed in the evolution of the shape of the yield surface.

The degradation of bonding is described through the following hardening law:

$$d\chi = -\xi\chi\left(\left|d\varepsilon_v^c\right| + \xi_d\left|d\varepsilon_q^c\right|\right) \tag{2.19}$$

where both the creep volumetric strain and the creep deviatoric strain tend to reduce the bonding parameter χ to zero. The soil constants ξ and ξ_d control the absolute rate of destructuration and the relative effectiveness of creep deviatoric and volumetric strains in destroying the bonding (Koskinen, Karstunen & Wheeler (2002)). Due to debonding, the void ratio decreases rapidly.

2.6 DEPENDENCY OF M FROM THE LODE ANGLE

In the Creep-SCLAY1S model the stress ratio at critical state M is not constant in the deviatoric plane. Its formulation describes a smooth critical state surface similar to the Matsuoka and Nakai failure surface (Matsuoka & Nakai (1974)). Since p'_{eq} is defined on the anisotropic line (α-line), M depends on the modified Lode angle θ_α:

$$M(\theta_\alpha) = M_c\left(\frac{2m^4}{1 + m^4 + (1 - m^4)sin3\theta_\alpha}\right)^{\frac{1}{4}} \tag{2.20}$$

where $m = M_e/M_c$ with M_e being the value of M in triaxial extension test ($\theta_\alpha = 30°$) and M_c the value of M in triaxial compression test ($\theta_\alpha = -30°$). When M_e is equal to M_c, the failure surface equals the Drucker-Prager failure surface in the π-plane (as for the Modified Cam-Clay model). The modified Lode angle can be defined as:

$$sin(3\theta_\alpha) = -\left[\frac{3\sqrt{3}}{2}\frac{(J_3)_\alpha}{(J_2)_\alpha^{3/2}}\right] \tag{2.21}$$

where $(J_2)_\alpha$ and $(J_3)_\alpha$ are the second and third invariants of the modified stress deviator $q - \alpha p'$, which compares the stress state with the α-line (Sivasithamparam, Karstunen & Bonnier (2015)). It should be noted that the value of m should be greater than 0.6 to preserve physically realistic convex failure surface as shown in Figure 2.4 (Gras, Sivasithamparam, Karstunen & Dijkstra (2017)).

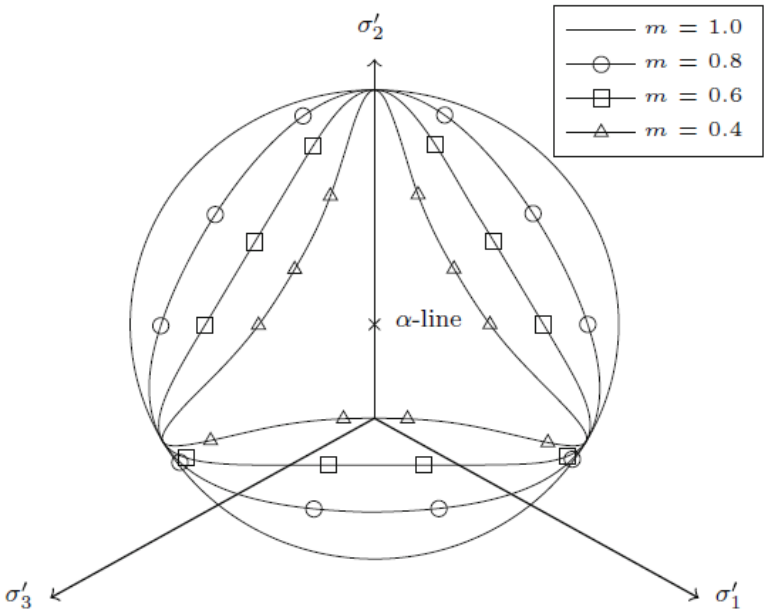

Figure 2.4 Failure surfaces in the deviatoric plane (from Gras, Sivasithamparam, Karstunen & Dijkstra (2017).

3 PARAMETER DETERMINATION

The parameters required for the Creep-SCLAY1S model are summarized in Table 3.1. Most of the parameters can be determined through laboratory tests commonly performed in engineering practice. In some cases, formulas have been proposed to determine the value or to define a lower and upper bound for some parameters.

Table 3.1 Model parameters.

Parameter type	Symbol	Description	Unit
Initial state variables	e_0	Initial void ratio	-
	POP	Pre-overburden pressure	-
	OCR	Overconsolidation ratio	-
	α_0	Initial anisotropy	-
	χ_0	Initial amount of bonding	-
Isotropic stiffness parameters	λ_i^*	Modified intrinsic compression index	-
	κ^*	Modified swelling index	-
	υ'	Poisson'ratio	-
Creep parameters	μ_i^*	Modified intrinsic creep index	-
	τ	Reference time (usually 1 day)	day
Critical state parameters	M_c	Slope of critical state line in compression	-
	M_e	Slope of critical state line in extension	-
Anisotropy parameters	ω	Absolute effectiveness of rotational hardening	-
	ω_d	Relative deviatoric effectiveness of rotational hardening	-
Bonding parameters	ξ	Absolute rate of destructuration	-
	ξ_d	Relative deviatoric rate of destructuration	-
Other parameter	K_0^{NC}	Coefficient of lateral stress in normal consolidation	-

3.1 INITIAL STATE VARIABLES

The initial condition of the soil is defined through the following state variables:

- the initial void ratio e_0;
- the pre-overburden pressure *POP* and the overconsolidation ratio *OCR*;
- the initial inclination α_0 of the surfaces NCS, CSS and intrinsic yield surface (see Section 3.5);
- the initial amount of bonding χ_0 (see Section 3.6).

The preconsolidation pressure is included in the model through the input value of *POP* (Eq. (3.1)) or *OCR* (Eq. (3.2)), and it defines the size of the normal consolidation surface NCS.

The value of the preconsolidation pressure depends on the sedimentation history, natural processes such as erosion and deposition and human actions (e.g. loads from existing structures). It can be determined through an oedometer test on undisturbed samples. By defining the *POP* and/or *OCR* value, the preconsolidation pressure is considered to vary with depth:

$$POP = \sigma'_{vp} - \sigma'_{v0} \tag{3.1}$$

$$OCR = \frac{\sigma'_{vp}}{\sigma'_{v0}} \tag{3.2}$$

where σ'_{v0} is the initial in situ vertical effective stress.

Since the NCS surface represents the boundary of small and large irrecoverable strains observed in a $e - log(\sigma'_1)$ plane, the model is very sensitive to the chosen value for *POP* and *OCR* (and consequently, p'_p).

The value of the initial state variables is updated during the analysis.

3.2 ISOTROPIC STIFFNESS PARAMETERS

The intrinsic value of the modified compression index, λ_i^*, is different from the modified compression index λ^*, i.e. the slope of the compression line in the $\varepsilon_v - ln(p')$ plane. The intrinsic value should be determined from isotropic compression tests on reconstituted soil samples or for undisturbed samples at very high stresses (Figure 3.1), assuming that all the bonds are completely lost at high stresses.

The modified swelling index κ^* is the slope of the unloading/reloading curve in the $\varepsilon_v - ln(p')$ plane. In the case of sensitive clays, the value of κ^* may vary for different unloading/reloading path: in those cases, it is suggested to use its largest value, in order to be on the conservative side.

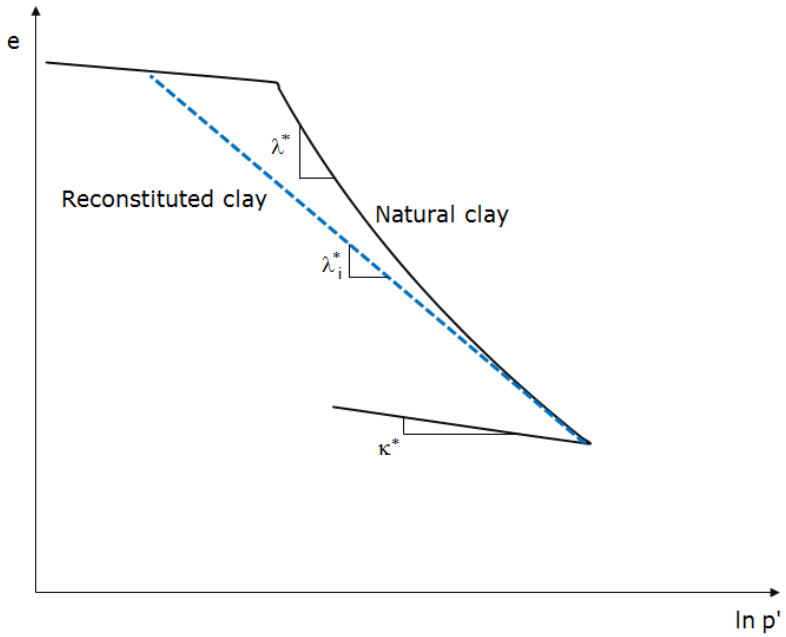

Figure 3.1 Behaviour of natural and reconstituted clay.

Alternatively, both λ_i^* and κ^* can be determined from the one-dimensional intrinsic compression index C_{ci} and the one-dimensional swelling index C_s, respectively, i.e. the slope of the intrinsic compression line and the slope of the unloading/reloading line, respectively, in the $e - log(\sigma'_1)$ plane assuming K_0 equal to 1 and υ' equal to 0.2.

$$\lambda_i^* = \frac{C_{ci}}{ln10(1 + e_0)} \tag{3.3}$$

$$\kappa^* \approx \frac{2C_s}{ln10(1+e_0)} \tag{3.4}$$

The formula that links the swelling index κ^* to the one-dimensional swelling index C_s is not exact because the ratio of horizontal and vertical stresses changes during one-dimensional unloading. For this approximation it is assumed that the average stress state during unloading is isotropic (i.e. the horizontal stress is equal to the vertical stress).

Usually, the range of the ratio λ_i^* / κ^* is between 2.5 and 7.

The value of the Poisson'ratio υ' for elastic unloading/reloading is constant and approximately equal to 0.15/0.2.

3.3 CREEP PARAMETERS

The creep parameters μ^* and τ are used to determine the plastic multiplier (as shown in Eq. (2.13) in Section 2.3) that defines the creep strain rates.

The modified creep index μ^* is measured in the $\varepsilon_v - ln(t)$ plane. It is an intrinsic material property (also indicated as μ_i^*), which means that it should be calculated based on tests where the structure is completely erased, e.g under very high stress values or on remoulded samples. The value of μ_i^* represents the limit value of the slope of the curve in the $\varepsilon_v - ln(t)$ plane, with increasing t and can be determined from oedometer test by measuring the value sufficiently long time after the last load step.

The modified creep index μ_i^* can also be derived from the intrinsic secondary compression index $C_{\alpha i}$:

$$\mu_i^* = \frac{C_{\alpha i}}{ln10(1+e_0)} \tag{3.5}$$

The reference time τ is the duration of the load step in the oedometer test used to obtain the initial preconsolidation pressure. It is expressed in days and is equal to 1, in the case of a 24h oedometer test (Leoni, Karstunen & Vermeer (2008)).

The values of μ_i^* and τ should be derived using the same unit time.

3.4 CRITICAL STATE PARAMETERS

The inclination of the critical state line in triaxial compression and triaxial extension is given by the parameters M_c and M_e, respectively. The values can be determined based on the value of the critical state friction angle φ'_{cv}:

$$M_c = \frac{6sin\varphi'_{cv}}{(3 - sin\varphi'_{cv})} \tag{3.6}$$

$$M_e = \frac{6sin\varphi'_{cv}}{(3 + sin\varphi'_{cv})} \tag{3.7}$$

3.5 ANISOTROPY PARAMETERS

The anisotropy parameters are the initial inclination of the surfaces α_0, the relative effectiveness of creep strains in rotational hardening ω_d and the absolute effectiveness of rotational hardening ω, as explained in Eq. (2.18) of Section 2.5.

The initial inclination α_0 can be calculated assuming that the soil has been subjected to one-dimensional consolidation. Therefore, the inclination of the yield curve corresponds to that produced by a one-dimensional K_0^{NC} consolidation, assuming an associated flow rule and considering that the load ratio between deviatoric and volumetric plastic strain rates is approximately $2/3$. The corresponding formula (Eq. (3.8)) is proposed by Wheeler, Näätänen, Karstunen & Lojander (2003):

$$\alpha_0 = \alpha_0^{NC} = \frac{\eta_{K_0^{NC}}^2 + 3\eta_{K_0^{NC}} - M_c^2}{3} \tag{3.8}$$

where M_c is the slope of the critical state line in compression measured from drained or undrained triaxial tests (Eq. (3.6)) and $\eta_{K_0^{NC}}$ is the value of the stress ratio q/p' corresponding to a normally consolidated value of K_0^{NC}. It can be calculated as shown in Eq. (3.9):

$$\eta_{K_0^{NC}} = \frac{3\left(1 - K_0^{NC}\right)}{\left(1 + 2K_0^{NC}\right)} \tag{3.9}$$

An estimate of K_0 for normally consolidated soils is provided by Jaky's formula (Jaky (1944)):

$$K_0^{NC} \approx 1 - sin\varphi'_{cv} \tag{3.10}$$

The relative effectiveness of rotational hardening due to deviatoric straining, ω_d, can be calculated as proposed by Wheeler, Näätänen, Karstunen & Lojander (2003):

$$\omega_d = \frac{3\left(4M_c^2 - 4\eta_{K_0^{NC}}^2 - 3\eta_{K_0^{NC}}\right)}{8\left(\eta_{K_0^{NC}}^2 - M_c^2 + 2\eta_{K_0^{NC}}\right)} \tag{3.11}$$

Alternatively or, after a first estimation of α_0 and ω_d from analytical solutions (Eq. (3.8) and Eq. (3.11)), the initial inclination can be calibrated based on CAUC (Consolidated Anisotropic Undrained Compression) and CAUE (Consolidated Anisotropic Undrained Extension) triaxial tests on undisturbed soil, starting from an anisotropic cell pressure with stress ratio η close to α_0 and applying a cell pressure such that p'_{eq} is lower than the preconsolidation pressure p'_p. This allows to determine the inclination of the critical state line in compression and extension and to draw the yield points (and consequently, the yield surface and its inclination α_0).

The parameter ω controls the absolute rate at which the normal consolidation surface (NCS) rotates with plastic straining. It can be calibrated based on experimental data but a first estimation can be made through the following procedure (Eq. (3.16)). Experimental evidence shows that the initial anisotropy is erased when applying isotropic loading up to a pressure that is two or three times larger than the preconsolidation pressure

(Anandarajah, Kuganenthira & Zhao (1996)).

The ratio between the deviatoric and the volumetric strain rates is given by the flow rule:

$$\frac{\left|d\varepsilon_q^c\right|}{d\varepsilon_v^c} = \frac{2\alpha_0}{M(\theta)^2} = \frac{2\alpha_0}{M_e^2} \tag{3.12}$$

where $M(\theta) = M_e$ because, when isotropic compression is applied to a natural material with an in situ normally consolidated history, the isotropic stress path is below the anisotropy line (Wheeler, Näätänen, Karstunen & Lojander (2003)). Additionally, starting from a normally consolidated state, a subsequent isotropic compression results into a positive increment of the volumetric strain and a negative increment of the deviatoric strain, so the absolute value of $d\varepsilon_q^c$ is considered (Gras, Sivasithamparam, Karstunen & Dijkstra (2017)). During isotropic loading, q is equal to zero and the rotational hardening rule becomes:

$$d\alpha = -\alpha_0\omega\left(d\varepsilon_v^c + \omega_d\left|d\varepsilon_q^c\right|\right) \tag{3.13}$$

By substituting Eq. (3.12) in Eq. (3.13), it becomes:

$$-\omega d\varepsilon_v^c = \frac{M_e^2 d\alpha}{M_e^2\alpha_0 - 2\alpha_0^2\omega_d} \tag{3.14}$$

Eq. (3.14) can be integrated in the range between α_0 and $\alpha_1 = \alpha_0 + \Delta\alpha$ to obtain:

$$\omega\Delta\varepsilon_v = ln\left(\frac{\alpha_0/\alpha_1 - 2\alpha_0\omega_d}{M_e^2 - 2\alpha_0\omega_d}\right) \tag{3.15}$$

With $\omega\Delta\varepsilon_v = \omega(\lambda^* - \kappa^*)ln(p'/p'_p)$, for materials without structure, Eq. (3.15) becomes:

$$\omega = \frac{1}{(\lambda^* - \kappa^*)ln\left(p'/p'_p\right)} ln\left(\frac{\alpha_0/\alpha_1 + 2\alpha_0\omega_d}{M_e^2 + 2\alpha_0\omega_d}\right) \tag{3.16}$$

Eq. (3.16) has been recently updated by Gras, Sivasithamparam, Karstunen & Dijkstra (2017) and differs from the one proposed by Leoni, Karstunen & Vermeer (2008) in the positive signs in the logarithmic argument. This avoids undetermined values for ω (Sivasithamparam, Karstunen & Bonnier (2015)). The ratio α_0/α_1 can be approximated to 10, assuming that anisotropy is erased when α decreases to $1/10th$ of its initial value. The ratio $ln\left(p'/p'_p\right)$ is around 2, considering that most of the anisotropy is erased when loading up to two (or three) times the preconsolidation pressure.

A lower and upper bound of ω is suggested considering that M_e, α_0 and ω_d can be calculated as a function of M_c, which ranges from 0.8 to 1.6 (as reported in literature):

$$\frac{1.5}{\lambda^* - \kappa^*} \leq \omega \leq \frac{4.2}{\lambda^* - \kappa^*} \tag{3.17}$$

In the case of structured clays, the variation of creep volumetric strains is also related to

the degree of bonding:

$$\omega \Delta \varepsilon_v = (\lambda_i^* - \kappa^*) ln \left(\frac{\frac{p'}{p'_p}(1+\chi_0)}{1+\frac{\chi_0}{\chi_1}} \right) \tag{3.18}$$

where χ_1 is the final amount of bonding.

A lower and upper bound of ω for materials with an initial amount of bonding χ_0 are given by:

$$0 < \omega \leq \frac{2.9}{\lambda_i^* - \kappa^* ln \left(\frac{2(1+\chi_0)}{1+\frac{\chi_0}{2}} \right)} \tag{3.19}$$

For all the details on the proposed limit values it is suggested to refer to Gras, Sivasithamparam, Karstunen & Dijkstra (2017).

3.6 BONDING PARAMETERS

The initial degree of bonding can be approximately calculated based on the sensitivity index of the clay S_t.

$$\chi_0 \approx S_t - 1 \tag{3.20}$$

where S_t is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength. It can be determined from fall cone test on natural and remoulded samples.

The absolute rate of bonding ξ can be determined by curve fitting in the $\varepsilon_v - ln(p')$ plane for drained consolidated tests at a stress ratio equal to the initial anisotropy α_0 such that the deviatoric creep strains are null and the variation in the amount of bonding can be written as:

$$d\chi = -\xi\chi\left(\left|d\varepsilon_v^c\right| + \xi_d\left|d\varepsilon_q^c\right|\right) \approx -\xi\chi d\varepsilon_v^c \tag{3.21}$$

If ξ is equal to zero, there is no destructuration, while for high values of ξ the debonding is very rapid.

Gras, Sivasithamparam, Karstunen & Dijkstra (2017) proposed limit values for ξ equal to:

$$\xi \geq \frac{ln2}{\left[ln(2+2\chi_0) - ln\left(1+\frac{\chi_0}{2}\right)\right](1+\xi_d)(\lambda_i^* - \kappa^*)} \tag{3.22}$$

$$\xi \leq \frac{1+\chi_0}{\chi_0(\lambda_i^* - \kappa^*)\left(1+2\xi_d\frac{\alpha_{K_0^{NC}}}{M_e^2}\right)} \tag{3.23}$$

The relative rate of bonding due to deviatoric creep, ξ_d, can be determined by curve

fitting, after having defined χ_0 and ξ. In soft clays it is reasonable to assume that deviatoric creep strains have less or equal influence as the volumetric creep strains on the destructuration process (Gras, Sivasithamparam, Karstunen & Dijkstra (2017)):

$$0 \leq \xi_d \leq 1 \tag{3.24}$$

A typical value that has been calibrated for various Scandinavian and Scottish clays is between 0.2 and 0.4.

3.7 SUMMARY OF MODEL PARAMETERS

A summary of the model parameters and the tests or formulas that can be used to determine them is presented in Table 3.2.

Table 3.2 Model parameters determination.

Parameter	Parameter determination
e_0	Oedometer test
POP	Oedometer / CAUD triaxial test
OCR	Oedometer / CAUD triaxial test
α_0	Eq. (3.8), depends on K_0^{NC}
χ_0	Fall cone test / Shear vane test
λ_i^*	Isotropic compression / Oedometer / CRS test
κ^*	Isotropic compression / Oedometer /CRS test (with unloading/reloading path)
υ'	Oedometer test
μ_i^*	Oedometer test
τ	Oedometer test
M_c	CADC / CAUC triaxial test
M_e	CADE / CAUE triaxial test
ω	To be optimized, Eq. (3.17) or Eq. (3.19)
ω_d	Eq. (3.16), depends on K_0^{NC}
ξ	To be optimized with curve fitting triaxial compression test, Eq. (3.22) and Eq. (3.23)
ξ_d	To be optimized with curve fitting oedometer test, Eq. (3.24)
K_0^{NC}	Triaxial test

3.8 OUTPUT STATE PARAMETERS

In addition to the traditional output quantities available for all material models, it is possible to visualize 17 state parameters when performing a finite element calculation with the Creep-SCLAY1S model:

- State variable 1 to State variable 6: rotation of the yield surface in the general stress space, α_{xx}, α_{yy}, α_{zz}, α_{xy}, α_{yz} and α_{zx};
- State variable 7: scalar value of the rotation of the yield surface, α;
- State variable 8: size of the yield surface of the reconstituted soil, p'_{mi};
- State variable 9: amount of bonding, χ;
- State variable 10: size of the yield surface, p'_m;
- State variable 11: current void ratio;

- State variable 12: flag to indicate that the initialization has been done, equal to 123;
- State variable 13: size of the yield surface (NCS), p'_p;
- State variable 14: mean stress, p';
- State variable 15: overconsolidation ratio, p'_p/p'_{eq};
- State variable 16: derivative of p'_{eq} with respect to p';
- State variable 17: deviatoric strain size.

4 LABORATORY TESTS SIMULATIONS

The main features of the Creep-SCLAY1S model are the rate dependency, the anisotropy, the bonding and the destructuration process during creep straining. Several laboratory tests are simulated by means of the PLAXIS SoilTest facility or by reproducing a finite element model in PLAXIS 2D in order to highlight the model features.

4.1 RATE DEPENDENCY

Several undrained triaxial compression tests are performed on a material data set that corresponds to the Bothkenar clay. The effect of bonding are not considered in this test. The model parameters are summarized in Table 4.1. The triaxial compression test is performed in undrained anisotropic conditions, where K_0 is equal to 0.5 and the initial effective stress $|\sigma'_3|$ is set to 100 kPa. Different strain rate from 1 to 100 are applied by varying the applied strain ε_1 and the duration. The results are shown in the $\varepsilon_1 - q$ plane (Figure 4.1) and in the $p' - q$ plane (Figure 4.2). The model is capable to predict the rate effect dependence, showing that changes in strain rate can determine major changes in the undrained peak strength with higher peaks for increasing strain rates.

Table 4.1 Bothkenar clay parameters.

Parameter	Symbol	Value	Unit
Modified swelling index	κ^*	0.0067	-
Poisson'ratio	υ'	0.2	-
Modified intrinsic compression index	λ_i^*	0.1	-
Slope of critical state line in compression	M_c	1.5	-
Slope of critical state line in extension	M_e	1.0	-
Absolute effectiveness of rotational hardening	ω	50.0	-
Relative deviatoric effectiveness of rotational hardening	ω_d	1.0	-
Absolute rate of destructuration	ξ	0.0	-
Relative deviatoric rate of destructuration	ξ_d	0.0	-
Overconsolidation ratio	OCR	1.0	-
Pre-overburden pressure	POP	0.0	-
Initial void ratio	e_0	1.5	-
Initial anisotropy	α_0	0.59	-
Initial amount of bonding	χ_0	0.0	-
Reference time	τ	1.0	day
Modified intrinsic creep index	μ_i^*	0.00507	-
Coefficient of lateral stress in normal consolidation	K_0^{NC}	0.5	-

In Graham, Crooks & Bell (1983) the results of a series of test that have been performed on reconstituted normally consolidated Belfast clay are shown.The tests are based on a technique from Richardson & Whitman (1963) in which the strain rate applied to a sample is step-changed during the test. Each strain rate is applied long enough to establish the stress-strain relationship for that stage. The change of strain rate causes a jump of the stress-strain path to the original curve defined by the corresponding constant strain rate. This unique relationship between the stress-strain curve and the strain rate is known as isotach behaviour and it has been observed in most undisturbed and remoulded soft clays and intact natural stiff clays. The Creep-SCLAY1S is capable of simulating the isotach behaviour as shown in Figure 4.3 and Figure 4.4, where an undrained compression triaxial test has been performed at different strain rates (0.02% per day, 0.2% per day, 2% per day).

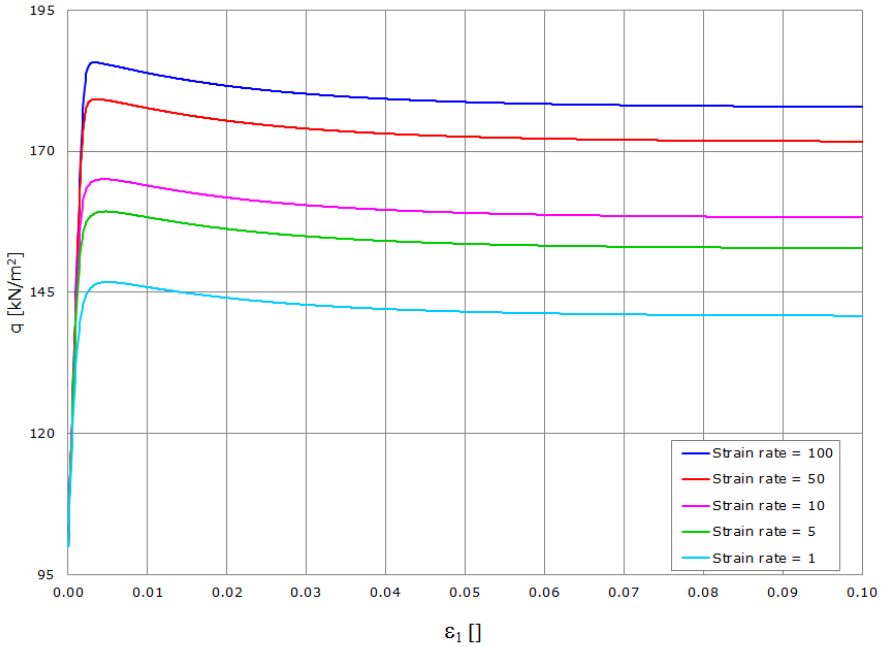

Figure 4.1 Results of undrained anisotropic triaxial compression tests at different strain rates in the $\varepsilon_1 - q$ plane.

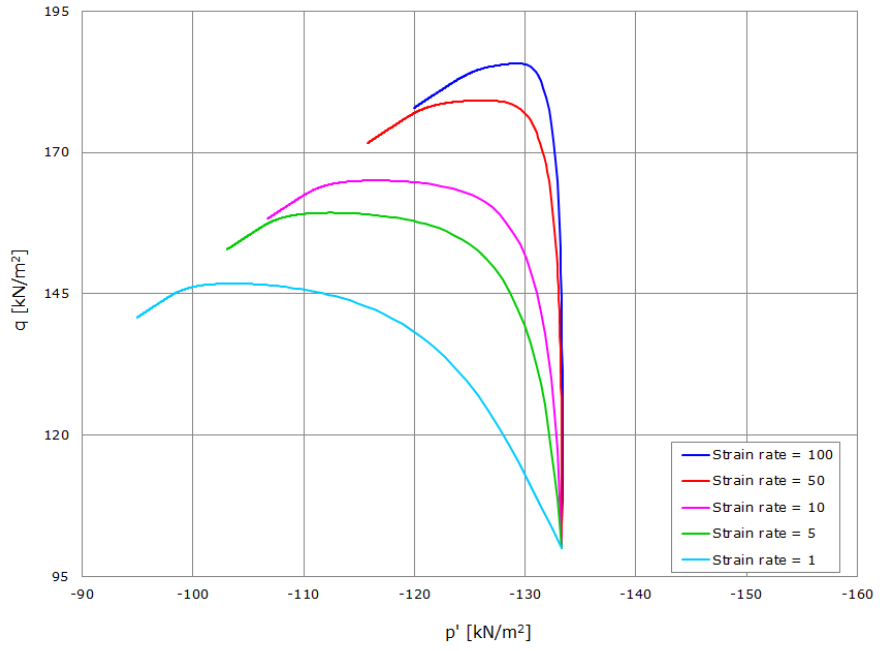

Figure 4.2 Results of undrained anisotropic triaxial compression tests at different strain rates in the $p' - q$ plane.

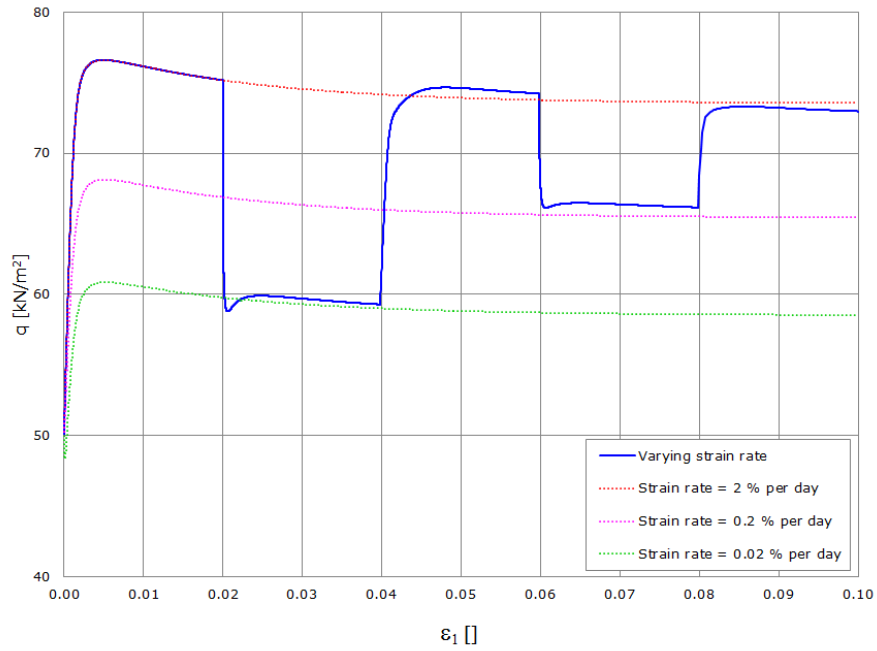

Figure 4.3 Results of undrained anisotropic triaxial compression tests at different strain rates in the $\varepsilon_1 - q$ plane.

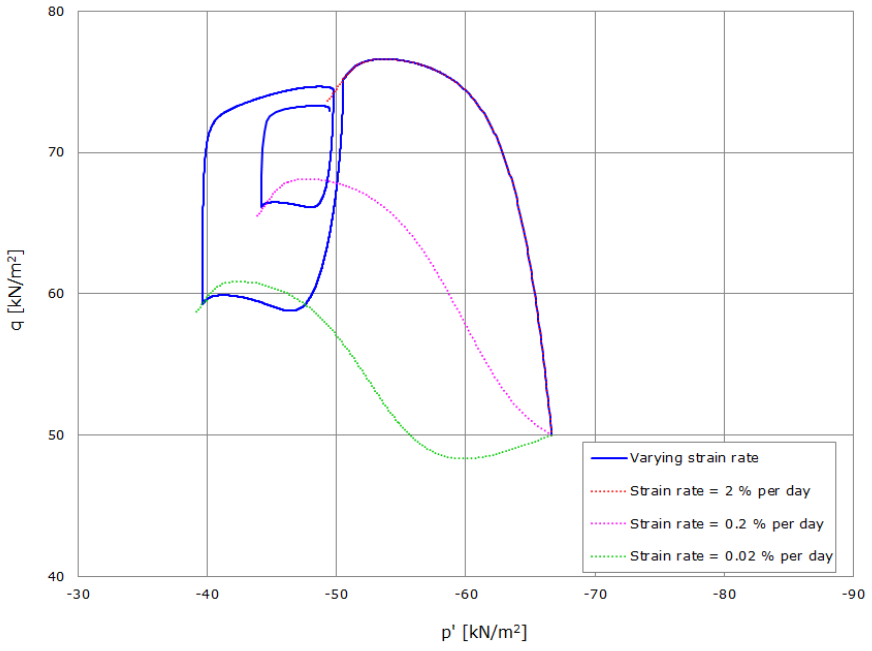

Figure 4.4 Results of undrained anisotropic triaxial compression tests at different strain rates in the $p' - q$ plane.

4.2 BONDING AND DESTRUCTURATION

The presence of bonding and the destructuration process is governed by the following three parameters:

- χ_0, representing the initial amount of bonding and defining the size of the intrinsic yield surface (i.e. p'_{mi} as explained in Eq. (2.16));
- ξ, absolute rate of destructuration;
- ξ_d, relative rate of destructuration.

As explained in Section 3.6, χ_0 can be determined based on the sensitivity index of the clay, while the destructuration parameters need to be calibrated based on the results of drained compression tests. Based on the input parameters described in Table 4.1 with some variations as in Table 4.2 a series of drained triaxial compression tests are performed using different values of ξ. Initially, the lower and upper bound for ξ are calculated according to Eq. (3.22) and Eq. (3.23), i.e. approximately 4.7 and 10.1, respectively.

Table 4.2 Modified Bothkenar clay parameters.

Parameter	Symbol	Value	Unit
Slope of critical state line in extension	M_e	1.1	-
Relative deviatoric rate of destructuration	ξ_d	0.2	-
Overconsolidation ratio	OCR	1.5	-
Initial anisotropy	α_0	0.5	-
Initial amount of bonding	χ_0	10.0	-
Coefficient of lateral stress in normal consolidation	K_0^{NC}	1.0	-

Three tests are performed by applying confining stresses at a stress ratio q/p' equal to α_0 (i.e. 0.5) such that the variation of the amount of bonding is mostly due to the variation of volumetric creep strains, since the deviatoric creep strains are approximately null. The load is applied in 1 day.

The results are shown in the $\varepsilon_v - p'$ plane, with ξ equal to 5, 7.5 and 10 (Figure 4.5). As expected, for larger values of ξ, the amount of volumetric strain at the same mean effective stress p' is larger.

The initial amount of bonding decreases during the loading phase and its variation is larger for larger ξ (Figure 4.6), since more volumetric strains are generated.

The value of the relative rate of destructuration,ξ_d, is usually assumed equal to 0.2 or 0.4. The effect of its variation is shown in Figure 4.7 for a case in which ξ is equal to 10. A larger value of ξ_d results in a more rapid destructuration process. In fact, at the end of the compression phase, the amount of bonding χ is close to 1 in all cases, mainly due to the absolute rate of destructuration ξ, but its value decreases even more for increasing ξ_d (Figure 4.8).

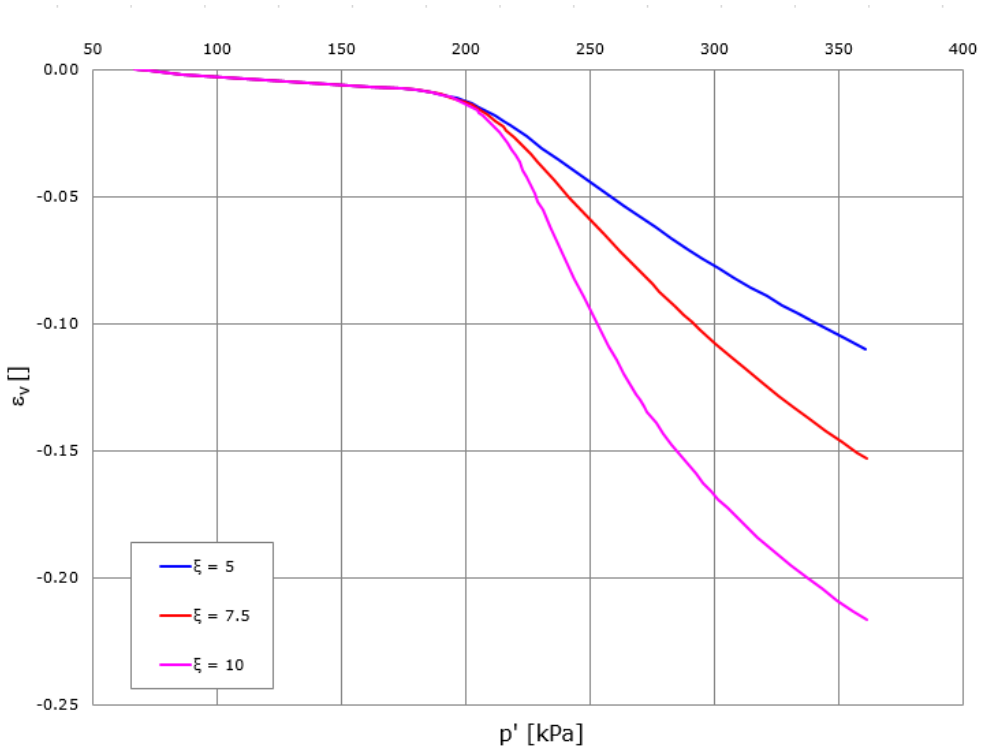

Figure 4.5 Results of drained triaxial compression tests at q/p' = 0.5, adopting different values for ξ.

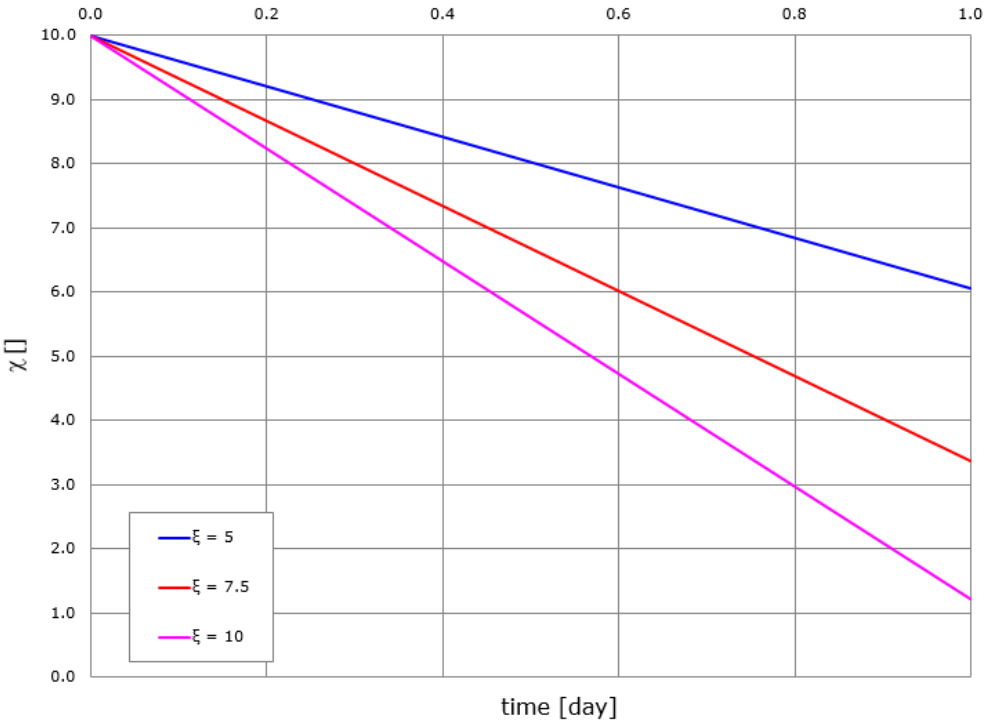

Figure 4.6 Variation of the initial amount of bonding at the end of the compression phase, due to ξ.

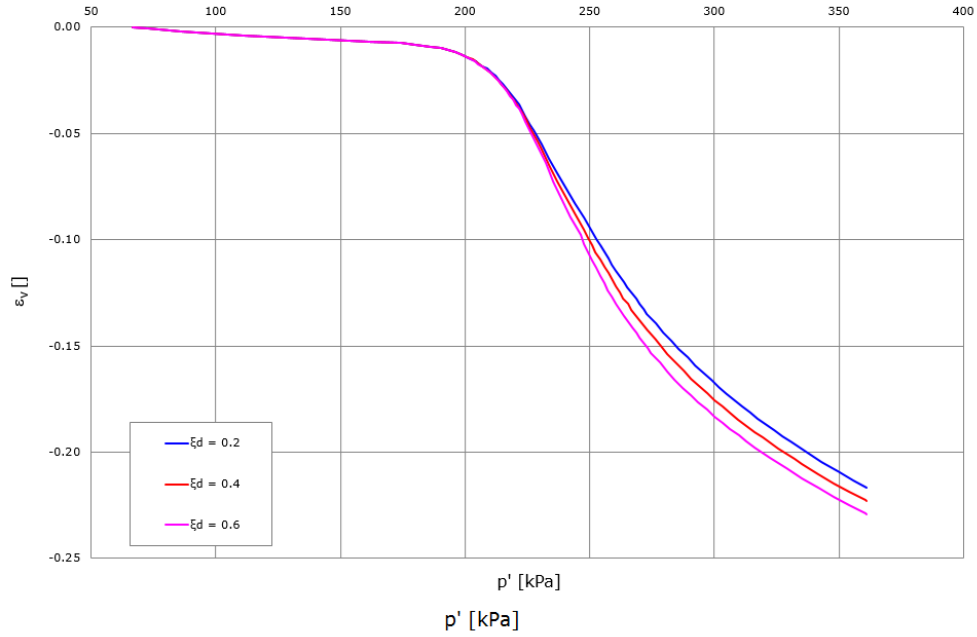

Figure 4.7 Results of drained triaxial compression tests at $q/p' = 0.5$, adopting different values for ξ_d.

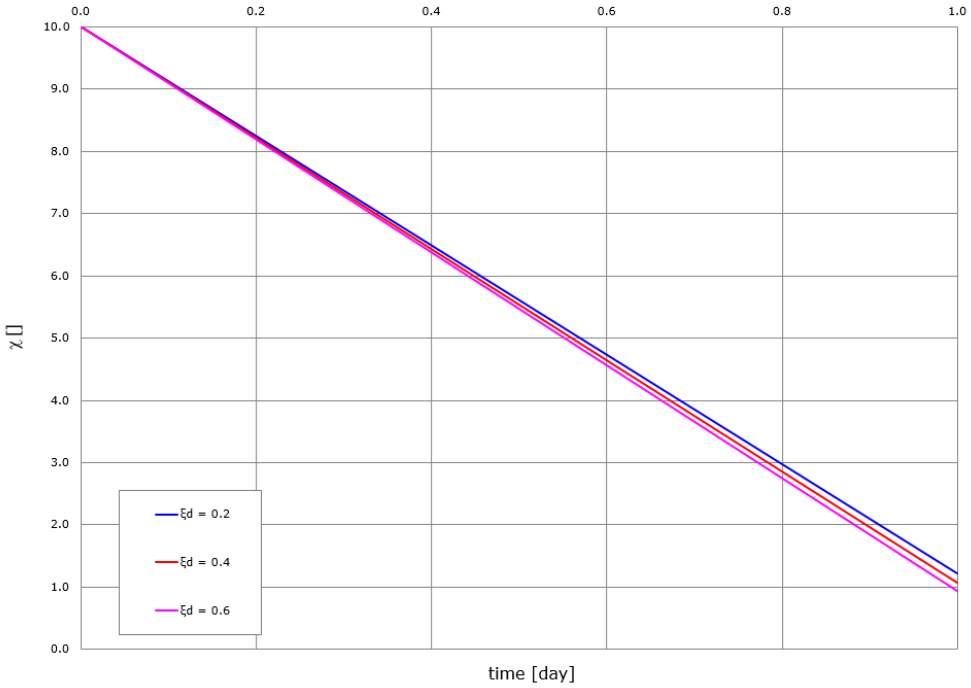

Figure 4.8 Variation of the initial amount of bonding at the end of the compression phase, due to ξ_d.

5 PRACTICAL APPLICATIONS

5.1 ONE-DIMENSIONAL CONSOLIDATION: COMPARISON WITH SOFT SOIL AND SOFT SOIL CREEP

To study the influence of creep and anisotropy, a one-dimensional consolidation analysis is performed, results of which are compared with soft soil and soft soil creep models. Creep-SCLAY1S has a hierarchical structure, and it is possible to switch off the evolution of anisotropy; hence a comparison has been made for the model with and without anisotropy. The model dimensions are 10 m x 10 m; the groundwater flow boundaries are closed on sides and bottom. The model is preloaded with 10 kPa, and additionally, a uniformly distributed load of 30 kPa is applied along the top surface. Consolidation analysis is performed in two phases. In the first phase, a load is applied in 100 days and in the subsequent phase the material is allowed to consolidate fully at a constant load for additional 900 days. The drainage type is Undrained A.

The model parameters for Creep-SCLAY1S and Soft soil are shown in Table 5.1 and Table 5.2 respectively. In this example, it is assumed that the intrinsic value of the modified compression index λ_i^* Creep-SCLAY1S is equal to that of modified compression index λ^* for soft soil. The saturated soil weight is kept as $18 kN/m^3$. To switch off the initial anisotropy and the evolution of anisotropy parameters α_0, ω and ω_d are kept as zero. In fact, when α is equal to zero, the yield curve will correspond to the Modified Cam-clay yield curve Eq. 2.15 and setting ω to zero will switch off the rotational hardening law Eq. 2.18. Parameters for the soft soil are same as that of soft soil creep without modified creep index.

Table 5.1: Soil parameters for Creep-SCLAY1S

Parameter	Symbol	Value	Unit
Modified swelling index	κ^*	0.02	-
Poisson'ratio	υ'	0.15	-
Modified intrinsic compression index	λ_i^*	0.1	-
Slope of critical state line in compression	M_c	0.98	-
Slope of critical state line in extension	M_e	0.74	-
Absolute effectiveness of rotational hardening	ω	27.48	-
Relative deviatoric effectiveness of rotational hardening	ω_d	0.49	-
Absolute rate of destructuration	ξ	0	-
Relative deviatoric rate of destructuration	ξ_d	0	-
Overconsolidation ratio	OCR	1	-
Pre-overburden pressure	POP	0	kN/m^2
Initial void ratio	e_0	1	-
Initial anisotropy	α_0	0.38	-
Initial amount of bonding	χ_0	0	-
Reference time	τ	1	day
Modified intrinsic creep index	μ_i^*	0.004	-
Coefficient of lateral stress in normal consolidation	${K_0}^{NC}$	0.72	-

Vertical permeability	k_y	0.04752	m/day

Table 5.2: Soil parameters for Soft Soil Creep

Parameter	Symbol	Value	Unit
Modified compression index	λ^*	0.1	-
Modified swelling index	κ^*	0.02	-
Modified creep index	μ^*	0.004	-
Cohesion	c_{ref}'	1	kN/m^2
Friction angle	φ'	25	°
Poisson'ratio	υ'	0.15	-
Coefficient of lateral stress in normal consolidation	K_0^{NC}	0.72	-
Initial void ratio	e_0	1	-
Overconsolidation ratio	OCR	1	-
Pre-overburden pressure	POP	0	kN/m^2
Vertical permeability	k_y	0.04752	m/day

The change of permeability for changing void ratio is not taken into account and c_k is kept at the default value. The displacements are reset to zero before the consolidation calculation phase. The maximum load fraction per step is kept at 0.01 in the numerical control parameters for the consolidation loading phase.

Figure 5.1 shows the evolution of vertical displacements for 1000 days. The influence of creep is visible when comparing the results of soft soil and creep based models. The soft soil model does not generate further settlement after consolidation, whereas creep models continue to settle with time. Soft soil creep, and Isotropic Creep-SCLAY1S are giving similar deformations, however, when anisotropic parameters are introduced to the Creep-SCLAY1S model it tends to provide minor deformations. This is due to model behaviour when ignoring the evolution of anisotropy; the model overestimates the settlements compared to field measurements since the model predicts lower soil stiffnesses (Sivasithamparam, Karstunen & Bonnier (2015)). It is also evident that the evolution of creep sets in at a very early stage in the analysis, and creep model predicts higher settlements compared to soft soil model.

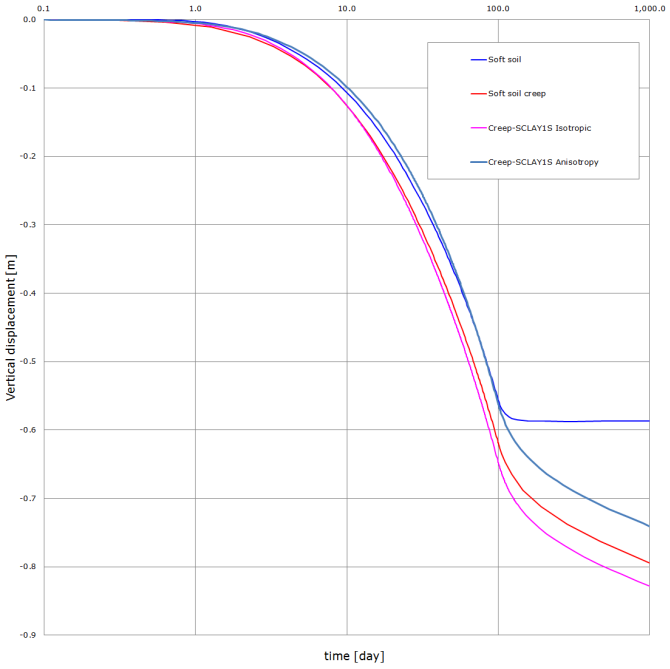

Figure 5.1 Evolution of vertical displacements

5.2 EMBANKMENT ON SOFT SOIL

This example illustrates an application of Creep-SCLAY1S model for embankment construction on sensitive clays. The embankment is constructed in two stages with consolidation phases in between. The whole analysis is conducted as fully coupled analysis. To take advantage of symmetry of the problem, only right half of the embankment is modelled. Standard boundary conditions are used for deformations, groundwater flow boundaries closed at the left-hand side, and other sides are kept as open.

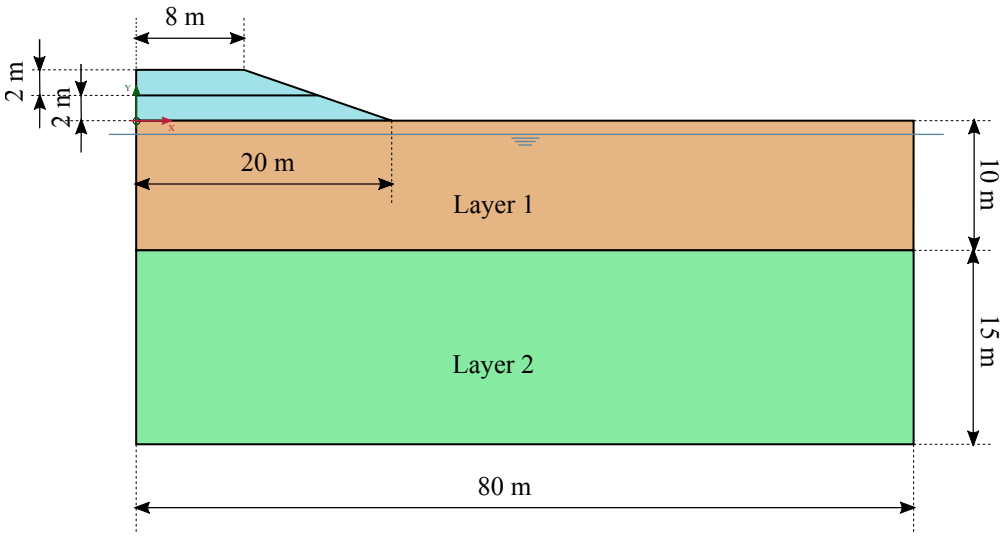

Figure 5.2 Model geometry

Figure 5.2 illustrates the model geometry and dimensions. The embankment is 4 m high and 20 m long and is situated over anisotropic clay deposits. The phreatic level is located 1 m below the ground level. The analysis is conducted using soft soil, soft soil creep and Creep-SCLAY1S model. The embankment is modelled using Mohr-Coulomb model, with the parameters adopted for the analysis listed in Table 5.3 and Table 5.4. Parameters for soft soil and soft soil creep (without modified creep index) are the same.

Table 5.3: Soil parameters for Creep-SCLAY1S model

Parameter	Symbol	Layer 1	Layer 2	Unit
Unsaturated unit weight	γ_{unsat}	15	15.5	kN/m^3
Saturated unit weight	γ_{sat}	18	15.5	kN/m^3
Modified swelling index	κ^*	0.02	0.0107	-
Poisson'ratio	υ'	0.15	0.35	-
Modified intrinsic compression index	λ_i^*	0.1	0.1786	-
Slope of critical state line in compression	M_c	0.98	1.6	-
Slope of critical state line in extension	M_e	0.74	1.04	-
Absolute effectiveness of rotational hardening	ω	27.48	25	-

Relative deviatoric effectiveness of rotational hardening	ω_d	0.49	1.02	-
Absolute rate of destructuration	ξ	7	4	-
Relative deviatoric rate of destructuration	ξ_d	0	0	-
Overconsolidation ratio	OCR	1.3	1.3	-
Pre-overburden pressure	POP	0	0	kN/m^2
Initial void ratio	e_0	1	1.8	-
Initial anisotropy	α_0	0.38	0.63	-
Initial amount of bonding	χ_0	5	10	-
Reference time	τ	1	1	day
Modified intrinsic creep index	μ_i^*	0.004	0.008	-
Coefficient of lateral stress in normal consolidation	K_0^{NC}	0.72	0.34	-
Horizontal & vertical permeability	k_x, k_y	0.04752	0.1	m/day

Table 5.4: Soil parameters for Soft Soil Creep and Mohr-Coulomb

Parameter	Symbol	Embankment	Layer 1	Layer 2	Unit
Material model		Mohr-Coulomb	Soft soil creep	Soft soil creep	
Unsaturated unit weight	γ_{unsat}	16	15	15.5	kN/m^3
Saturated unit weight	γ_{sat}	19	18	15.5	kN/m^3
Young'modulus	E'	5000	-	-	kN/m^3
Modified compression index	λ^*	-	0.1	0.107	-
Modified swelling index	κ^*	-	0.02	0.1786	-
Modified creep index	μ^*	-	0.004	0.008	-
Cohesion	c_{ref}'	1	1	1	kN/m^2
Friction angle	φ'	30	25	40	°
Poisson'ratio	υ'	0.3	0.15	0.35	-
Coefficient of lateral stress in normal consolidation	K_0^{NC}	0.5	0.72	0.34	-
Initial void ratio	e_0	0.5	1	1	-
Overconsolidation ratio	OCR	1	1.3	1.3	-
Pre-overburden pressure	POP	0	0	0	kN/m^2
Horizontal & vertical permeability	k_x, k_y	3.5	0.04752	0.1	m/day

Initial stresses are generated using *K0 loading* with soil clusters for the embankment inactive, the first half of the embankment is constructed in Phase_1, which consolidates for 6 months in the following phase. In third phase, the final stage of the embankment is constructed and is allowed to consolidate for 1000 days in the subsequent phase. Table 5.5 presents the staged construction details of the embankment.

The development of vertical stresses during the construction and subsequent consolidation process for the given soil models at a point approximately 3m (x=0, y=2.9) below the embankment is shown in Figure 5.3. Creep models show a considerable

Table 5.5 Staged construction details

Phase	Calculation type	Time (days)	Details
Initial Phase	K0 procedure	-	Embankment inactive
Phase_1	Fully coupled	100	Activate Layer 1
Phase_2	Fully coupled	180	-
Phase_3	Fully coupled	90	Activate Layer 2
Phase_4	Fully coupled	1000	-

displacement from the initial period compared to soft soil model which does not have any creep settlements. Initially, the model behaves stiffer than Soft soil creep due to bonding. Hence settlements are lower. However, at higher stresses debonding starts thus the model has significantly larger settlements than soft soil creep model.

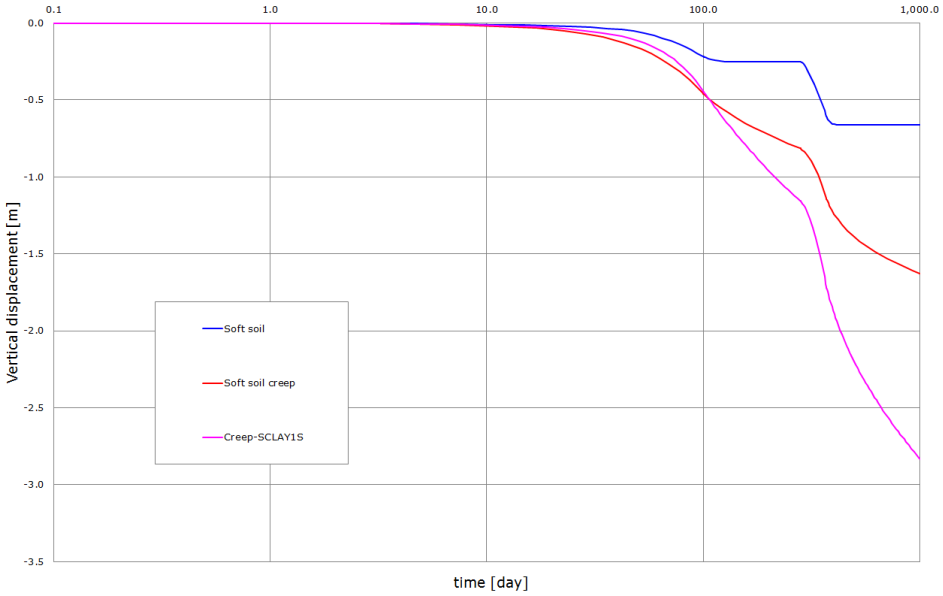

Figure 5.3 Evolution of vertical displacements

Figure 5.4 shows the development of excess pore pressures. Soft soil model is fully consolidated with no excess pore pressures after 1000 days. However, the consolidation process is delayed for Creep-SCLAY1S model and further settlements due to consolidation can still be expected.

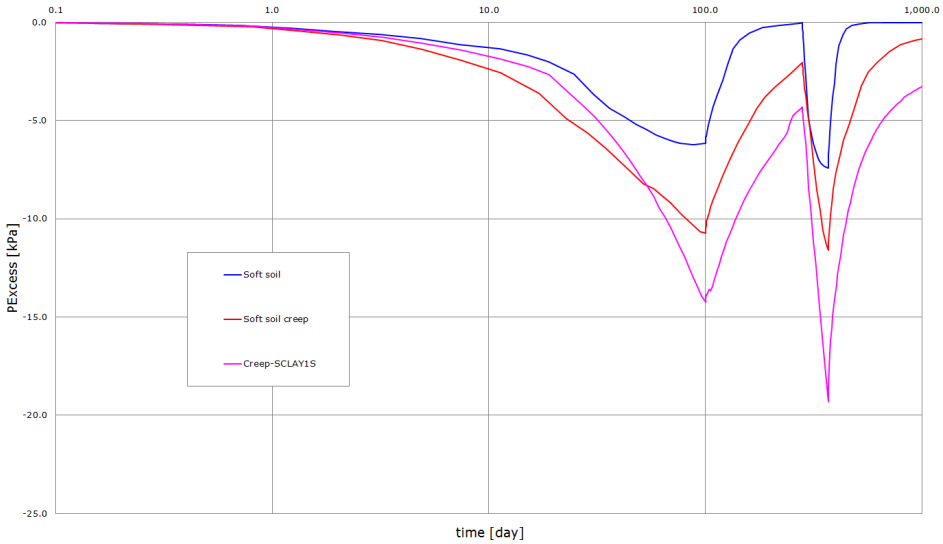

Figure 5.4 Pore pressure distribution

6 CONCLUSIONS

The formulation of Creep-SCLAY1S model is explained in the first part of this document. The model accounts for the creep, bonding and destructuration effects. There is no purely elastic domain in this model, and it allows for creep within the Normal Consolidation Surface (NCS). The model is governed by three hardening rules. First one determines the size of the normal consolidation surface based on the volumetric creep strains ϵ_v^c, the second one is a rotational hardening law which allows the evolution of anisotropy due to irreversible strain and the third one determines the degradation of bonding. Model is not able to simulate the non-linearity of small strain stiffness. Hence it is not appropriate for dynamic applications.

A detailed overview of soil parameters required and a summary of tests or formulas that can be used to determine them are presented. It is possible to obtain most of the parameters through commonly performed laboratory tests in engineering practice. Analytical solutions for anisotropy parameters and upper/lower bound values for absolute rate of bonding are proposed. These can give a good initial guess on the range of values to be used for anisotropy and bonding. However, calibration of results based on laboratory testing is needed for a more accurate prediction of those parameters. Results of a series of laboratory simulations that are conducted to highlight the main characteristics of the model are presented. The model uses the concept of the constant viscoplastic multiplier (Sivasithamparam, Karstunen & Bonnier (2015)), and can predict isotach behaviour which is experimentally evidenced in most undisturbed and remoulded soft clays and intact natural stiff clays. The impact of bonding and destructuration parameters on the model behaviour are well illustrated through laboratory simulations.

The performance of the model is compared with existing PLAXIS Soft soil, and soft soil creep models with a one-dimensional consolidation analysis. The hierarchical nature of the model allows to switch off the evolution of anisotropy, bonding and destructuration features. It is observed that the model behaviour is similar to soft soil creep model when anisotropy is switched off. However, activation of anisotropic model parameters leads to stiffer soil response and hence giving reduced settlements. An embankment construction problem on sensitive clays is simulated using the model. Structured clays exhibit a stiff behaviour as long as the level of bonding is high. Afterwards, the process of destructuration begins, giving relatively larger settlements. The results of the analysis match with this behaviour, Creep-SCLAY1S model predicts stiffer response compared to soft soil creep model initially. However, as soon as the process of destructuration begins the model starts producing considerably large settlements. The simulations conducted demonstrates better prediction capabilities of the model for sensitive anisotropic clays compared to conventional PLAXIS models for soft soils.

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