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Projekt A2014:14

On the fundamental relation between soil creep and cyclic pile-soil response

Final report from project

CHALMERS UNIVERSITY OF TECHNOLOGY

KTH

TRAFIKVERKET

STATENS GEOTEKNISKA INSTITUT

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Minna Karstunen & Jelke Dijkstra

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A2014-14 Final report for project: On the fundamental relation between soil creep and cyclic pile-soil response
Technical report
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Abstract

This report summarises the advances made at Chalmers on modelling and testing natural soft soils that are quasi-statically loaded with monotonic, and arbitrary stress paths. As such, the findings are directly applied in follow-up projects funded by Trafikverket, SBUF, FORMAS, Nordforsk and Vinnova. The main deliverables include a rate-dependent effective stress based constitutive model for natural soft soils, which in its generalised form is incorporated in a commercial finite element program, Plaxis, as a user-defined model that is available for other scientists and engineers. A Python based GUI toolbox for sensitivity analysis and multi-objective optimisation is a genetic tool that can assist both future model development, calibration of model parameters, as well as understanding which parameters are the most important to calibrate for a given application. New control software and hardware for cyclic triaxial testing of soft soils using a Bishop-Wesley cell, in combination with the new temperature-controlled GEO laboratory at Chalmers, enables for high quality cyclic testing of soft natural clays. A prototype setup for Constant Normal Stiffness testing of soft soils enables directly measuring the interface properties (soil/soil and soil/structure). Finally, first steps towards a fully-fletched bubble model for soft soils were made.

Keywords: Cyclic loading, natural clay, constitutive modelling, laboratory tests

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1 Introduction

1.1 Background

Physical railway infrastructure is often subjected to cyclic loads, in addition to the loads generated from the self-weight of the structure. In addition to variable environmental loads that are considered short-term loading, the repeated dynamic loads from operation of the railway system are of importance. In current design approach for shallow and deep foundations these loads are not explicitly considered. In the case of deep foundations on soft soils, the value for the α that factors the undrained shear strength on the pile–soil interface is chosen according to the expected duration of the load. However, the impact of repeated (dynamic) loads on the Ultimate Limit State, and especially on the Serviceability Limit State (SLS) is not yet well described. The SLS response of pile foundations in deep deposits of soft soils, such as the sensitive clays in the Gothenburg region, is mainly governed by the response of the soil in case shaft bearing (friction) piles are used. A quick inspection of the load distribution in shaft bearing pile systems indicates that the soil behaviour in layers located below the neutral plane controls the settlements of the pile (group) to a large degree (e.g. Fellenius, 2006). In addition to consolidation, the rate-dependence of the soil also drives the settlements of the foundation system. In engineering context the rate-dependence can be considered as creep that is further affected by the (cyclic) loading history. A combination of numerical and experimental developments are required to enable the modelling of the long-term behaviour of the foundations of physical railway infrastructure, *i*.e., the consolidation and creep settlements, additional settlements from monotonic and cyclic loading, the evolving stiffness and strength within and on the interface between structures and the surrounding clay.

1.2 Aim and objectives

The aim of the project was to study the interplay between creep and pile-soil interaction in sensitive soft soils. In order to do that, it is necessary to be able to model the long-term response of soft natural clays, accounting for both consolidation and creep. This is particularly challenging for sensitive clays. Furthermore, as the installation of piles often disturbs the soil, it is necessary to develop techniques for measuring the soil-pile interface properties, as a base for the dimensioning and design of deep foundations and embedded earth retaining structures. Finally, numerical tools that aid the calibration of the newly formulated and implemented models in this project will be developed.

The objectives of the project were the following:

- Element testing of high-quality samples of soft clay under long-term static loading as well as under undrained cyclic loads;
- Advanced constitutive and numerical modelling of creep and pile deformation phenomena in soft clays;
- Development of a constant normal stiffness (CNS) shear box. This allows for systematic study of pile-soil interface behaviour under appropriate boundary conditions;

1.3 Limitations and changes along the way

Very early in the project we realised that aiming for developing a totally new cyclic creep model that would be fully tested and validated within the time-frame of the project was overly ambitious. Based on the author's experience in teaching at courses on Basic and Advanced Computational Geomechanics (so-called Plaxis courses) to industry, many practitioners do not fully understand the effective stress-based soil mechanics, which is the essential underlying framework for constitutive modelling. Thus, the focus was to ensure that a simple underlying creep model is developed and implemented in such a way during the project that it can be made available, and used by practitioners and MSc students alike, as well as PhD students and researchers (see Sivasithamparam et al., 2015). The model was implemented as a user-defined soil model, and was made available to PLAXIS VIP users.

The new cyclic extension of the model, briefly introduced in this report, is yet unpublished, thus geared for internal research use. It enables understanding the problems associated with cyclic loading and assists in test planning and interpretation of experimental data. Much further experimental work and implementation would be needed to fully validate the model, and to further develop the model formulation, so that it could be implemented in such a way that full-scale boundary value level simulation could be run with reasonable computational time. In order to fully utilise the base-line creep model, additional research and development efforts were made with relation of consistent parameter determination and optimisation (see Gras et al., 2017; Gras et al., 2018).

On the experimental side, given two new set-ups were developed, the focus was to develop and build them, and to derive some preliminary data to demonstrate that they work, rather than have a huge set of experimental results as a results.

2 Cyclic behaviour of soft natural soils

The understanding, testing and modelling of the cyclic behaviour of soft natural soils still is not fully resolved. In engineering practice the most successful approach for the ULS design of foundations under cyclic environmental and operational loads, such as wind & wave loading and railway operation, appears to be a two step process (*e.*g., Andersen, 2009), *i.*e., (i) experimental determination of a site specific backbone design curve where the (undrained) strength and stiffness are modified as function of the number of load cycles and subsequently (ii) mapping of the updated strength and stiffness to the foundation engineering problem at hand. Alternatively, the reduction in undrained shear strength is assessed in-situ using cyclic CPT tests, though ideally full-flow penetrometers are used (Yafrate et al., 2009). Generally, strength reduction due to material softening from cyclic loading is partially compensated by the increased loading rate at which most cyclic loads occur. It therefore is paramount to discuss the modelling of rate-dependence in soft clays first, before discussing more salient details of cyclic loading.

Models or design methods that accurately capture the long-term SLS response of cyclically-loaded foundations are not well developed. The reason is that the majority of recent research on cyclic foundation response stems from offshore engineering where ULS still governs the design, and in that environment much larger displacements are allowed than onshore (Randolph and Gourvenec, 2017). Hence, in order to address the issues arising from cyclic loading of railway foundations on soft soils, there are large opportunities to expand state-of-the-art constitutive models for natural sensitive clays, most prominently Wheeler et al. (2003b), to incorporate rate-dependence, and to incorporate a more accurate description for cyclic hardening and softening mechanisms. Furthermore, experimental means need to be developed to capture the intricacies of natural clays under cyclic loading, as well as their behaviour on a soil-structure-interface.

2.1 Rate-dependent modelling of natural clays

For brevity here we focus on the SCLAY family as developed by the author as a logical extension of the Modified Cam Clay model (Roscoe, 1968) to capture essential features of natural clay including anisotropy Wheeler et al., 2003a and destructuration Karstunen et al., 2005b. The rate-dependent extension (*i.*e., Sivasithamparam et al., 2015; Gras et al., 2018) is further discussed in the next chapter.

2.2 Modelling the cyclic response of soft soils

A survey of the field for modelling of the cyclic response of soft soils is summarised in Appendix A. A breakdown of the report structure is shown in Figure 2.1. Depending on the number of loading cycles that need to be modelled, two different approaches are adopted. In case a few loading cycles (say < 1000) of arbitrary shape (strain, stress) needs to be captured with high-fidelity, the complete stress-strain curve as a function of time needs to be modelled. This approach is also called the implicit method A. Niemunis et al., 2005, *i.*e., the cyclic response implicitly follows from the model formulation. In the opposite case that is called the explicit method, commonly adopted for a large number of loading cycles, the rate of strain accumulation from cyclic loading is explicitly prescribed. In the given overview for few loading

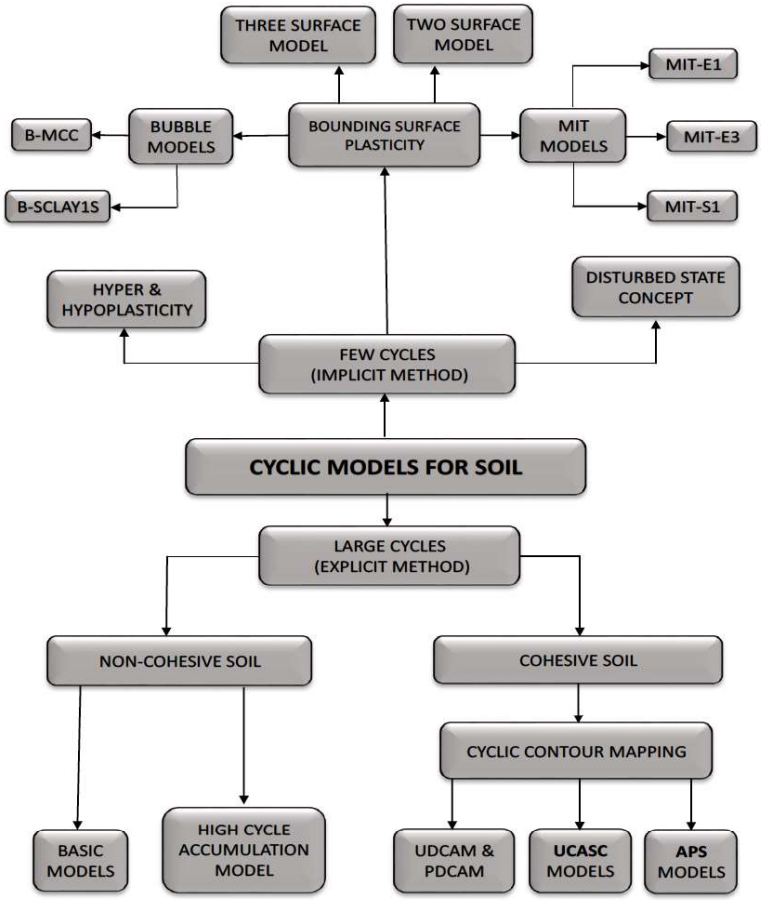


Figure 2.1: *Report structure of literature review on modelling cyclic behaviour in soft soils.*

cycles the focus has been on constitutive models that are based on bounding surface plasticity. In this approach the additional information required to capture the evolution of stiffness from load reversals within the yield envelope is incorporated with an additional loading surface that is centered around, and evolves, with the current stress.

Since writing this literature review an accumulation model for soft soils has been developed for clays by the authors and their co-workers.

2.3 Cyclic testing of soft soils

A survey of the field for modelling of the cyclic response of soft soils is summarised in Appendix B. A breakdown of the report structure is shown in Figure 2.2. The cyclic response of soft clays is often studied for undrained test conditions, as this is the condition that reflects most practical situations. The effective stress path leading to failure in undrained shearing is primarily governed by the generation of excess pore water pressures. These in turn depend on the mean (effective) stress level in the sample, as well as the amplitude and frequency of the cyclic load. Furthermore, different loading paths (*e.g.*, compression, extension, or simple shear) lead to different emerging strength. In uniaxial loading paths with lower loading frequencies (that lead to lower loading rates) failure is reached with a low number of loading cycles.

Similarly, a larger magnitude for the cyclic loading amplitude also drives failure more quickly. The opposite is true for small load amplitudes and high loading frequencies. The underlying mechanism for this behaviour seems to be related to the rate-dependence of the clay, and the behaviour of the porewater in the sample. The latter in turn depends on the stress (history) dependent stiffness. Both affect the rate of the gradual build up of excess porewater pressures. For small loading amplitudes and large loading frequencies the build up and unloading of the external cyclic load (total stress) is almost fully reversible, and mainly governed by the porewater pressures, without generating large magnitudes of irreversible strains (which are effective stress driven) and/or excess porewater pressures.

It should be noted that the cyclic response of clays is sensitive for more complex loading conditions where multiple components of stress are simultaneously cycled (in phase or out of phase) or the principal stress direction is rotated (controlled in a hollow cylinder or without control in a direct simple shear apparatus). These conditions are most relevant near the surface of a railway embankment or on the soil-structure-interface, and not necessarily in the clay deposit below.

Finally, some authors suggest to test clays in partially drained conditions, *i*.e., by keeping the back-pressure valve open during cyclic loading in triaxial conditions, so that a part of the porewater is allowed to drain from the sample. In these tests the stress state is highly non-uniform, precluding a single point interpretation based on externally measured strain and stress. As a results the findings of the results reported should be assessed with caution.

2.4 Model calibration of advanced constitutive models

Whilst more advanced models are shown to capture the soil behaviour with higher fidelity, the adoption of the models in industry still remains challenging. A big perceived obstacle for potential industry users is the model availability, the complexity and the difficulty of parameter derivation from laboratory data. The last concern perhaps is the largest obstacle, as most research focuses on the models themselves and their implementation at boundary value level, *e*.g., using user defined models in commercial Finite Element codes, rather than at consistent parameter derivation.

Unfortunately, the issue of consistent parameter evaluation from experimental data is rather limited. When done properly, the popularity of the constitutive model substantially increases (Herle and Gudehus, 1999). A general approach, of course, still has to be further calibrated to site specific conditions, but forms a base. A rather comprehensive study for soft soils is performed by Iizuka and Ohta (1987) for a viscoplastic model. This is one of the very few examples where also the site specific conditions are taken into account. Still, it requires user experience and empirical relations to soil classification data. Inverse modelling of in-situ test data is also attempted (*e*.g., for CPT (Obrzud et al., 2011) or the pressuremeter test (Zentar et al., 2001). Inverse modelling of the pressuremeter test appears to give most helpful information when combined with experimental results from a 1D laboratory compression test (Zentar et al., 2001). Finally, advanced inverse modelling during the construction process is also investigated. In that approach the model parameters are refined using feedback from the monitoring data. In this data assimilation (DA) approach good results are obtained when Particle Filters were employed (Shuku et al., 2012). For those DA approaches an initial prediction based on a constitutive model with appropriate model parameters still is required.

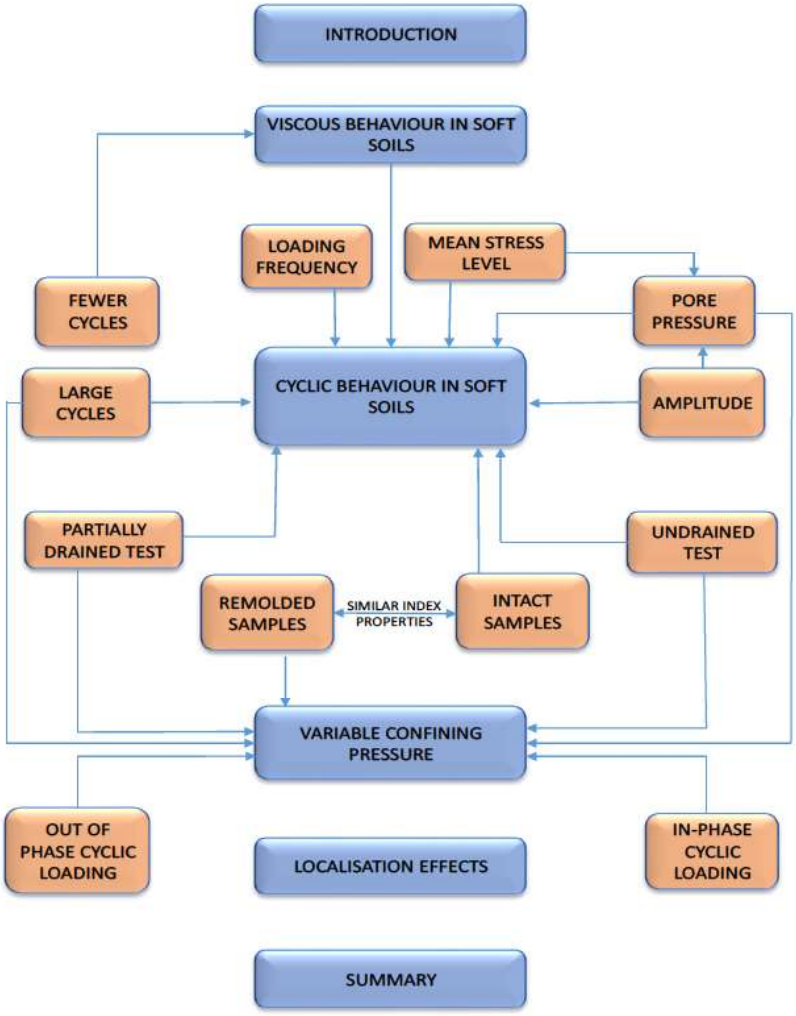


Figure 2.2: *Report structure of literature review on modelling cyclic behaviour in soft soils.*

3 Constitutive models for cyclic behaviour of soft soils

As discussed in the previous chapter, it is imperative to model the rate-dependent behaviour of soft soils first, as the cyclic response is a combination of the rate-dependent response that leads to an increase in the emerging (undrained) strength and stiffness, as well as the subsequent softening from repeated loading and unloading cycles.

3.1 Rate-dependent model for soft soil

The rate-dependent constitutive model for soft soils is an extension of the existing SCLAY1 model family that includes anisotropy and destructuration Karstunen et al., 2005b, SCLAY1S. The version without (Creep-SCLAY1) and with destructuration law (Creep-SCLAY1S) are

presented in respectively Sivasithamparam et al. (2015) and Gras et al. (2018). Here, we give a brief model description, originally elaborated in Amavasai et al. (2018):

The Creep-SCLAY1S model is an advanced constitutive model for soft soils that accounts for anisotropy, structural degradation and rate-dependency. The formulation is presented in the triaxial stress space, which is only suited to model cross-anisotropic samples in the oedometer and triaxial stress paths. For finite element implementation, the model has been generalised for the 3D stress space and can no longer be explained using invariants. This development and implementation was done in collaboration with Norwegian Geotechnical Institute (Dr Nallathamby Sivasithamparam) and Plaxis bv (Dr Paul Bonnier).

The Creep-SCLAY1S model is an extension of the Creep-SCLAY1 model proposed in Sivasithamparam et al., 2015. The formulation for anisotropy and degradation of structure are based on the SCLAY1S model developed by Karstunen et al., 2005b and the rate-dependent viscoplastic formulation is largely based on the Anisotropic Creep Model (ACM) developed by Leoni et al., 2008. The main model assumption is the absence of a purely elastic domain, similar to the ACM model. As a result viscoplastic (creep) deformation occurs at all stress states, as is observed in natural soils. The total strain rate is decomposed in an elastic and viscoplastic component as shown in Equation 3.1 and 3.2.

$$\dot{\epsilon_v} = \dot{\epsilon_v^e} + \dot{\epsilon_v^c} \tag{3.1}$$

$$\dot{\epsilon_q} = \dot{\epsilon_q^e} + \dot{\epsilon_q^c} \tag{3.2}$$

where the superscripts 'e' and 'c' refer to the elastic and viscoplastic components, respectively, and the subscripts 'v' and 'q' refer to the volumetric and deviatoric parts.

The model comprises of three reference surfaces defined in the triaxial stress space (Figure 3.1). The Normal Compression Surface (NCS) delimits small and large creep strain rates. The size of the NCS is defined by the isotropic pre-consolidation pressure p'_m as shown in Figure 3.1. The second surface is the Current Stress Surface (CSS) that represents the current state of effective stresses. p'_{eq} defines the size of CSS, as shown in Figure 3.1. An imaginary Intrinsic Compression Surface (ICS) proposed by Gens and Nova, 1993 is introduced to capture the effect of degradation of soil structure, as necessary for sensitive clays. The size of ICS is defined by p'_{mi} as denoted in Figure 3.1. The size of ICS is related to NCS with Equation 3.3, which introduces a state variable χ that represents the current amount of bonding in the natural soil. The initial value for χ is linked with the sensitivity of the clay.

$$p_m^{'} = p'_{mi}(1 + \chi) \tag{3.3}$$

All three surfaces have a similar shape and orientation, *i*.e., Equation 3.4.

$$p_s^{'} = p' + \frac{(q - \alpha p^{'})^2}{(M(\theta_\alpha)^2 - \alpha^2)\ p^{'}} \tag{3.4}$$

where p'_s can be equal to p'_m, p'_{eq} or p'_{mi} to define the sizes of NCS, CSS or ICS, respectively. p' and q represent the Cambridge invariants that correspond to the mean effective stress and deviatoric stress. α, which is a scalar in this simplified case, sets the orientation of the yield surface representing the change in anisotropy of the soil. $M(\theta_\alpha)$ is the stress ratio at critical state. To control the slope of the critical state in triaxial extension (M_e) and triaxial

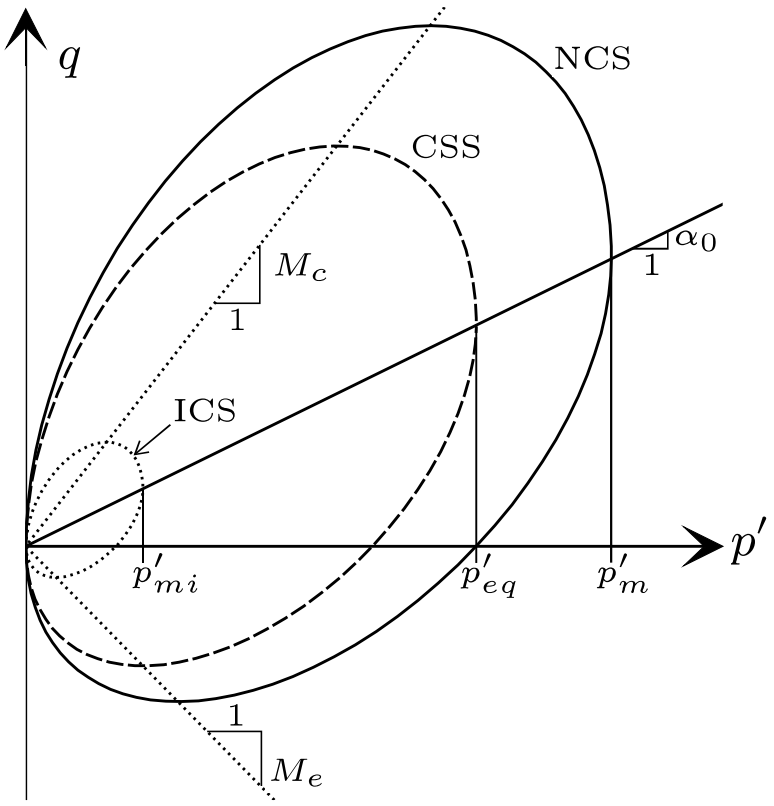


Figure 3.1: *Illustration of the constitutive surfaces used in the Creep-SCLAY1S model.*

compression (M_c) a modified Lode angle formulation is used (Sheng et al., 2000):

$$M(\theta_\alpha) = M_c\left(\frac{2m^4}{1+m^4+(1-m^4)sin3\theta_\alpha}\right)^{\frac{1}{4}} \tag{3.5}$$

where $sin3\theta_\alpha$ is defined as:

$$sin3\theta_\alpha = -\left[\frac{3\sqrt{3}}{2}\frac{(J_3)_\alpha}{(J_2)_\alpha^{\frac{3}{2}}}\right] \tag{3.6}$$

In Equation 3.5, m is defined as the ratio of M_e and M_c. Consequently, M takes the value of M_c in triaxial compression at θ_α = -30°, and at θ_α = 30°, M takes the value of M_e in triaxial extension. In Equation 3.6, $(J_2)_\alpha$ and $(J_3)_\alpha$ represent the second and third invariants of the modified deviatoric stress tensor, where the Lode angle is dependent on this stress state (see Sivasithamparam et al., 2015 for full details).

Following Wheeler et al., 2003a, an associated flow rule is used in the Creep-SCLAY1S model, which is reasonable for the soft soils modelled when combined with evolution of anisotropy. The viscoplastic strain rate in Creep-SCLAY1S is calculated as:

$$\dot{\epsilon_{ij}^c} = \dot{\Lambda}\frac{\partial p'_{eq}}{\partial \sigma'_{ij}} \tag{3.7}$$

$$\dot{\Lambda} = \frac{\mu_i^*}{\tau}\left(\frac{p'_{eq}}{(1+\chi)p'_{mi}}\right)^{\frac{\lambda_i^*-\kappa^*}{\mu_i^*}} \frac{M_c^2 - \alpha_{K_0^{NC}}^2}{M_c^2 - \eta_{K_0^{NC}}^2} \tag{3.8}$$

$\dot{\Lambda}$ in Equation 3.7 is the viscoplastic multiplier first postulated by Grimstad et al., 2010 to capture the rate dependent behaviour observed in soft soils. The viscoplastic multiplier in Equation 3.8 uses an intrinsic modified creep index parameter, μ_i^* that is derived directly from the laboratory data. μ_i^* is determined for the intrinsic material state where structure is completely erased either at large stress levels or in reconstituted samples. The reference time, τ, corresponds to the load step in an incremental loading 1D compression test for which the apparent pre-consolidation pressure is obtained. This model parameter enables to account for the rate dependency of the apparent pre-consolidation pressure that is used to define NCS. λ_i^* is the modified compression index, representing the stiffness of the soil after complete loss of structure at large strains. κ^* is the modified swelling index. The righthand term in Equation 3.8 ensures that under oedometric conditions, the resulting creep strain corresponds to the measured volumetric creep strain rate. Finally, $\alpha_{K_0^{NC}}$ sets the inclination of the ellipse in normally consolidated state.

Similarly to the SCLAY1S model Karstunen et al., 2005a, the Creep-SCLAY1S model accounts for three hardening processes. The isotropic hardening rule relates the change of ICS, by linking the intrinsic isotropic pre-consolidation pressure p'_{mi} and volumetric viscoplastic strain rate ($\dot{\epsilon_v^c}$) via:

$$\dot{p'_{mi}} = \frac{p'_{mi}}{\lambda_i^* - \kappa^*}\dot{\epsilon_v^c} \tag{3.9}$$

The second hardening rule links the change in the orientation of the reference surfaces as function of the viscoplastic strain rate (Wheeler et al., 2003b):

$$\dot{\alpha_d} = \omega\left(\left[\frac{3\eta}{4} - \alpha_d\right]\langle\dot{\epsilon_v^c}\rangle \; + \omega_d\left[\frac{\eta}{3} - \alpha_d\right]\dot{\epsilon_d^c}\right) \tag{3.10}$$

where η is the stress ratio ($=q/p'$) and ω & ω_d are model constants that control the absolute effectiveness of rotational hardening and the relative effectiveness of rotational hardening due to deviatoric viscoplastic strain rate, respectively. $\langle\rangle$ are Macauley brackets that describe the condition where $<\dot{\epsilon_v^c}>= 0$ if $\dot{\epsilon_v^c} < 0$ and $<\dot{\epsilon_v^c}>=\dot{\epsilon_v^c}$ if $\dot{\epsilon_v^c} \geq 0$.

The third hardening rule relates the destructuration of bonds with increasing viscoplastic strains. The bonding parameter χ evolves as a function of volumetric and deviatoric viscoplastic strain rate, and depends on ξ and ξ_d (Equation 3.11). ξ controls the absolute rate of structural degradation, whereas ξ_d controls the relative effectiveness of the degradation due to deviatoric viscoplastic strain rate.

$$\dot{\chi} = \xi\left([0-\chi]\left|\dot{\varepsilon_v^c}\right| + \xi_d\left[0-\chi\right]\dot{\varepsilon_d^c}\right) = -\xi\chi\left(\left|\dot{\varepsilon_v^c}\right| + \xi_d\dot{\varepsilon_d^c}\right) \tag{3.11}$$

Table 3.1 summarises the 14 model parameters for the Creep-SCLAY1S model. Although the initial void ratio is not a model parameter of Creep-SCLAY1S, it is convenient for tracking the change in void ratio, and also enables to account for void ratio dependency of the hydraulic conductivity in the subsequent analyses, if needed.

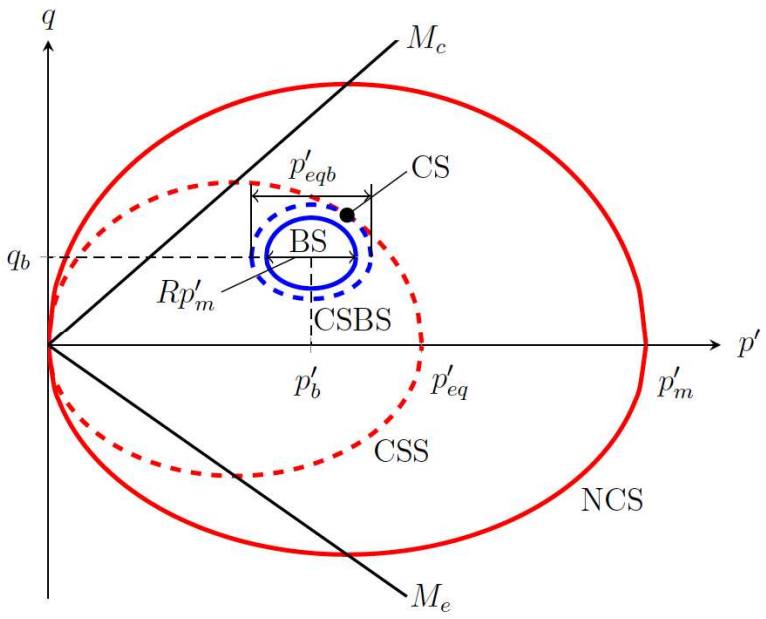
Table 3.1: Description of Creep-SCLAY1S model parameters

Type	**Parameter**	**Symbol**	**Experiment type**
Conventional	Modified swelling index	κ^*	IL oedometer or CRS test
parameters	Modified intrinsic compression index	λ_i^*	IL oedometer or CRS test
	Poisson's ratio	ν'	Triaxial test
	Slope of critical state line in compression	M_c	Triaxial compression test
	Slope of critical state in extension	M_e	Triaxial extension test
Anisotropic	Initial anisotropy	α_0	$\frac{(\eta_{K_0}^2+3\eta_{K_0}-M_c^2)}{3}$
parameters	Absolute effectiveness in rotational hardening	ω	Triaxial extension test
	Relative effectiveness in rotational hardening	ω_d	$\frac{3}{8}\frac{(4M_c^2-4\eta_{K_0}^2-3\eta_{K_0})}{(\eta_{K_0}^2-M_c^2+2\eta_{K_0})}$
Destructuration	Initial amount of bonding	χ_0	IL oedometer or CRS test
parameters	Absolute rate of destructuration	ξ_v	Back calculated from CRS and Triaxial tests
	Relative rate of destructuration	ξ_d	Back calculated from CRS and Triaxial tests
Viscous parameters	Intrinsic modified creep index	μ_i^*	1D Incremental loading IL oedometer (creep) test
	Reference time (days)	τ	Average time step employed in 1D Oedometer tests
Initial state parameters	Initial void ratio	e_0	Water content
	Pre-overburden pressure (kPa) (or OCR)	POP	IL oedometer or CRS test profile

3.2 Bubble model for soft soil

The bubble model is a logical extension of Creep-SCLAY1 to account for cyclic loading, as it adds a kinematic hardening bubble (akin to Al-Tabbaa and Muir Wood, 1989 bubble surface) within the bounding surface, represented by NCS. This bubble retains some memory of soil behaviour, therefore is more suitable to capture the cyclic response of soils, *i*.e., the Bubble Surface (BS) evolves during loading, unloading and reloading. Elastic behaviour is assumed when the stress state lies inside the Bubble Surface (BS) with respect to the recent history. Furthermore, both the bubble and bounding surface capture the rate-dependence of the soil. A short overview of model features is listed below:

- The model uses the kinematic hardening bubble surface, the bounding surface plasticity and critical state soil mechanics theory within a rate-dependent formulation.
- The degree of consolidation of a soil is represented by the Normal Consolidation Surface (NCS). It may expand or contract in stress space and the stress state can go outside of the Normal Consolidation Surface (NCS). The Normal Consolidation Surface (NCS) will be considered as bounding surface to the Bubble Surface (BS).
- The recent history of consolidation of a soil is represented by the Bubble Surface (BS) during loading, unloading and reloading. Elastic behaviour is assumed when the stress state lies inside the Bubble Surface (BS) with respect to the recent history.
- The Bubble Surface (BS) can translate, expand and contract with a loading path when the stress state lies outside the Bubble Surface (BS). It may tangentially contact with the Normal Consolidation Surface (NCS) at a point or even become identical with it, but it is not allowed to intersect it.
- Once the bubble touches the Normal Consolidation Surface (NCS), the Bubble Surface (BS) becomes identical to the Normal Consolidation Surface (NCS), and hence the parameter controlling the size of the bubble (R) becomes 1.
- The bubble translates in such a way that the stress state lies on the Bubble Surface (BS).
- The associated flow rule is used for the Current Stress Surface (CSS) to govern the viscoplastic flow;
- The total creep strain rate is a weighted sum of:
  - Creep strains rate corresponding to normally consolidated state or past history i.e., Normal Consolidation Surface (NCS) to Current Stress Surface (CSS) (see Figure 3.2)
  - Creep strains rate corresponding to over-consolidated state or recent history i.e., Bubble Surface (BS) to Current Stress Bubble Surface (CSBS) (see Figure 3.2)
- Creep strains from the Bubble Surface (BS) cannot be larger than the value at the Normal Consolidation Surface (NCS) at OCR=1 when stress lies inside the Normal Consolidation Surface (NCS).



NCS: Normal Consolidation Surface
CSS: Current Stress Surface
BS: Bubble Surface
CSBS: Current Stress Bubble Surface
CS: Current Stress

Figure 3.2: *The proposed model surfaces for the bubble model in triaxial stress space for isotropic case.*

3.3 Multi-objective optimisation for model calibration

With increasing model complexity, the number of model parameters increases. On average each model feature adds two additional model parameters, and possibly also state variable that needs to be tracked throughout the analyses. Hence, the models above for the rate dependency and cyclic behaviour of soils tend to have a relatively large number of parameters. Systematic calibration of those model parameters requires experimental data for a several loading paths. Furthermore, a consistent model parameter set should be obtained for the model at hand, so that all loading paths are modelled with sufficient accuracy without the need to change the values of the model parameters.

In this work algorithms for calibrating model parameters against objective functions has been developed. A three step approach, published in Gras et al., 2017; Gras et al., 2018, was followed:

- Determination of parameter bounds based on theoretical model limitations and physical bounds from experimental data Gras et al., 2018.
- Sensitivity study of model parameters: which parameters matter for specific loading paths. Not all model parameters matter for each loading path Gras et al., 2017.
- Optimisation of a consistent model parameter set against multiple object functions (experimental data) Gras et al., 2017.

The constitutive models were evaluated at single integration point level using a strain driver (Gudehus et al., 2008; Niemunis, 2008) after adding a graphical user interface using the Python PyQt library. The additional tools included a sensitivity analysis tool based on (*SAlib* 2016) to study the importance and sensitivity of parameters of a model for a given loading path, and an optimisation tool, based on a modified version (Fortin and Parizeau, 2013) of the popular NSGA-II genetic algorithm (Deb et al., 2002) implemented via the Deap library (Fortin et al., 2012), to optimise model parameters against experimental data, for a single loading path or multiples loading paths.

4 Cyclic testing of of soft soils

The following provides a summary of progress made in cyclic testing capabilities at Chalmers. The most important development has been the modification of conventional triaxial setup, used for static tests, and developing a LabVIEW based software to perform cyclic triaxial tests, with a focus on performing cyclic triaxial test of soft sensitive clay.

In addition to presenting some initial results, a new testing programme has been planned as part of an ongoing Formas project at Chalmers. The outcomes of the new testing programme will enable the further development of constitutive models for monotonic and cyclic loading of soft and sensitive clays.

4.1 Constant Normal Stiffness shearbox

A prototype device was developed for interface testing of soft clays under controlled normal stiffness (CNL) conditions, rather than constant normal load (CNL). The design facilitated means to consolidate a reconstituted sample –representing a remoulded clay near a displacement pile– at a target stress level separately from the loading device (Figure 4.1). Internally, a mechanical spring is already incorporated, to keep the sample stressed during transfer on the apparatus. Tall samples were foreseen, giving the option to shear the sample on two planes, *i*.e., on the interface (steel/concrete) and in the soil. These two subsequent measurements on a single sample would directly resolve α factor for direct shear.

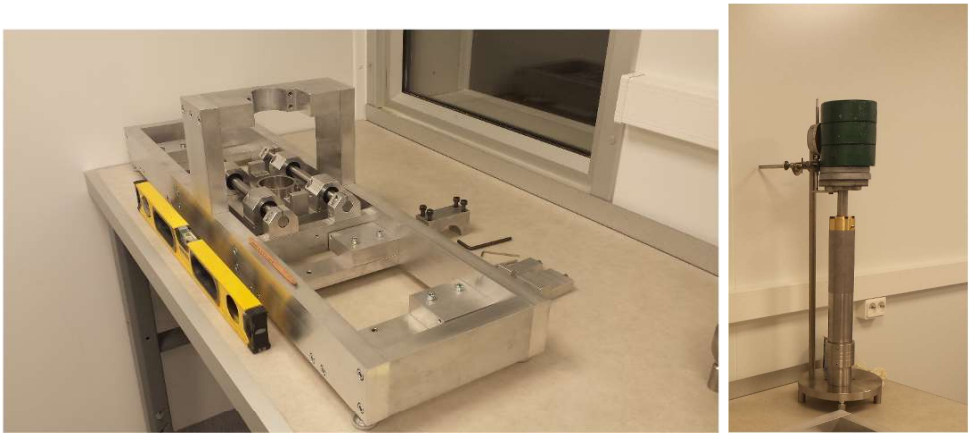


Figure 4.1: *Prototype setup of Constant Normal Stiffness shearbox with separate cell for sample preparation.*

4.2 Cyclic triaxial tests

4.2.1 Testing procedures and test programme

A conventional computer-controlled Bishop and Wesley (1975) type hydraulic stress path apparatus, equipped with submersible internal load cells, was modified to perform cyclic loading tests on 50 mm diameter, 100 mm mm high, specimens. External LVDT transducers

were used to measure the axial strains. Pore pressures were measured at the base of the specimen with sufficiently slow load application to allow full pore pressure equalisation during undrained monotonic shearing or cyclic loading stage. A suction cap connection system was employed to able cyclic loading and to centralise the applied load between the load cell and top platen. A computer-controlled high-precision neMESYS pressure syringe pump (Cetoni) system adopted to control cell, back and ram pressure and also to apply cyclic loading (Figure 4.2). In addition, a versatile software in LabVIEW was developed to control and monitor both monotonic and cyclic triaxial tests with modified triaxial testing setup.



Figure 4.2: *Prototype setup of the Bishop-Wesley cell with new Cetoni syringe pumps.*

A suite of seven carefully planned monotonic and cyclic undrained triaxial tests, listed in Tables 4.1 and 4.2 were performed on high-quality cylindrical undisturbed samples taken from the 8, 9- and 10 m depth, from the Utby test site in Gothenburg. The depth of the water table is 1 m below the ground surface at this site. The testing programme is intended to provide information that is most relevant to cyclic loading related to high-speed (HS) railway lines, such as those planned in Sweden. Cyclic loading was defined by two loading components, the average qmean, which was applied slowly before cyclic loading application and the sinusoidally varying cycles of amplitudes q_{cyc}. For each test, the given in-situ q_m value (as listed in Table 4.2) was applied through undrained stress-controlled loading. The completed experiments show very clearly and consistently that the conditions under which load cycling is imposed (from in-situ stress conditions) impacts on the behaviour of intact specimens.

Test	Depth (m)	e_0	K_0	q_{mean} (kPa)	p'(kPa)	q_f (kPa)	S_u (kPa)
M-1	8	1.34	0.6	25	39	59	≈ 30
M-2	9	1.32	0.6	25	44	60	30
M-3	10	1.47	0.6	31	41	62	≈ 30

Table 4.1: Summary of undrained monotonic triaxial test conditions for Utby clay.

Test	Depth (m)	f (Hz)	q_{mean} (kPa)	$\frac{q_{mean}}{2S_u}$	q_{cyc} (kPa)	$\frac{q_{cyc}}{2S_u}$	q_{max} (kPa)	$\frac{q_{max}}{2S_u}$
C-1	8	0.01	24	0.4	20	0.33	44	0.73
C-2	8	2.e-3	24	0.4	20	0.33	44	0.73
C-3	9	0.01	24	0.4	20	0.33	44	0.73
C-4	10	0.01	24	0.4	20	0.33	44	0.73

Table 4.2: Summary of cyclic triaxial test conditions for Utby clay.

4.2.2 Results monotonic tests

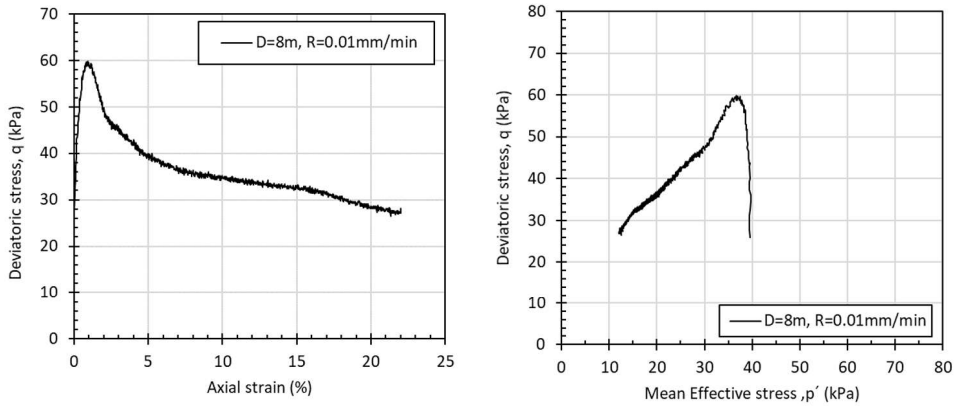


Figure 4.3: *M-1: CAUC triaxial test on a sample from* 8 m *depth.*

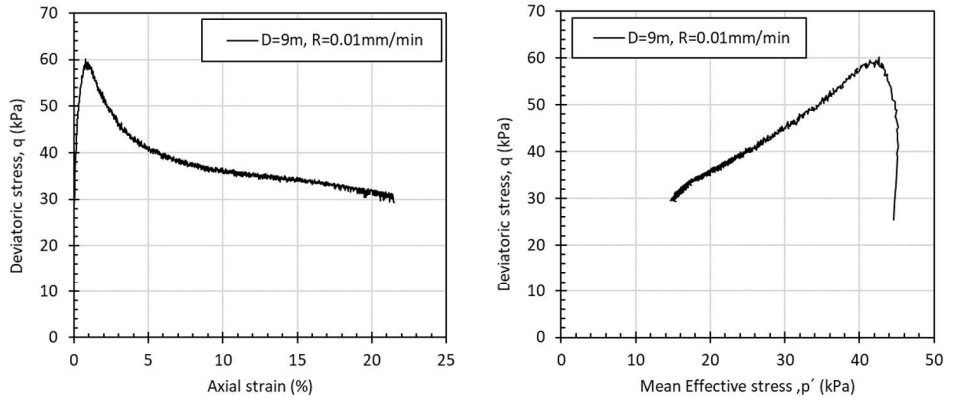


Figure 4.4: *M-2: CAUC triaxial test on a sample from* 9 m *depth.*

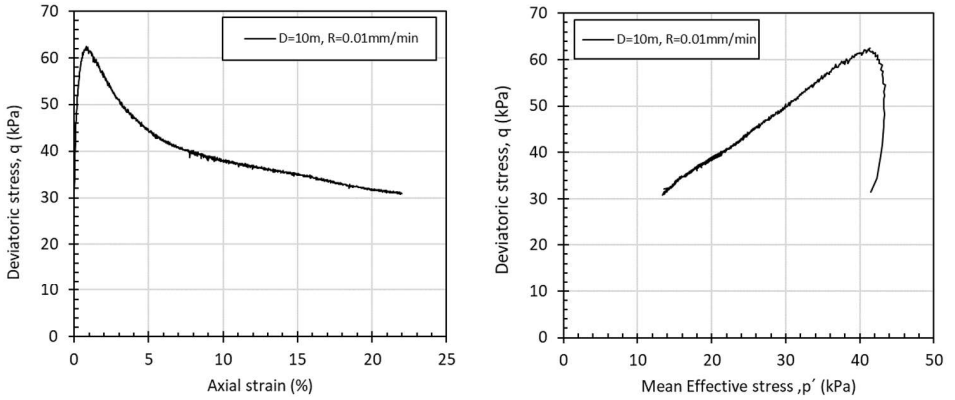


Figure 4.5: *M-3: CAUC triaxial test on a sample from* 10 m *depth.*

4.2.3 Results cyclic tests

Test C-1: depth $= 8\,\mathrm{m}$ **&** $f = 0.01\,\mathrm{Hz}$

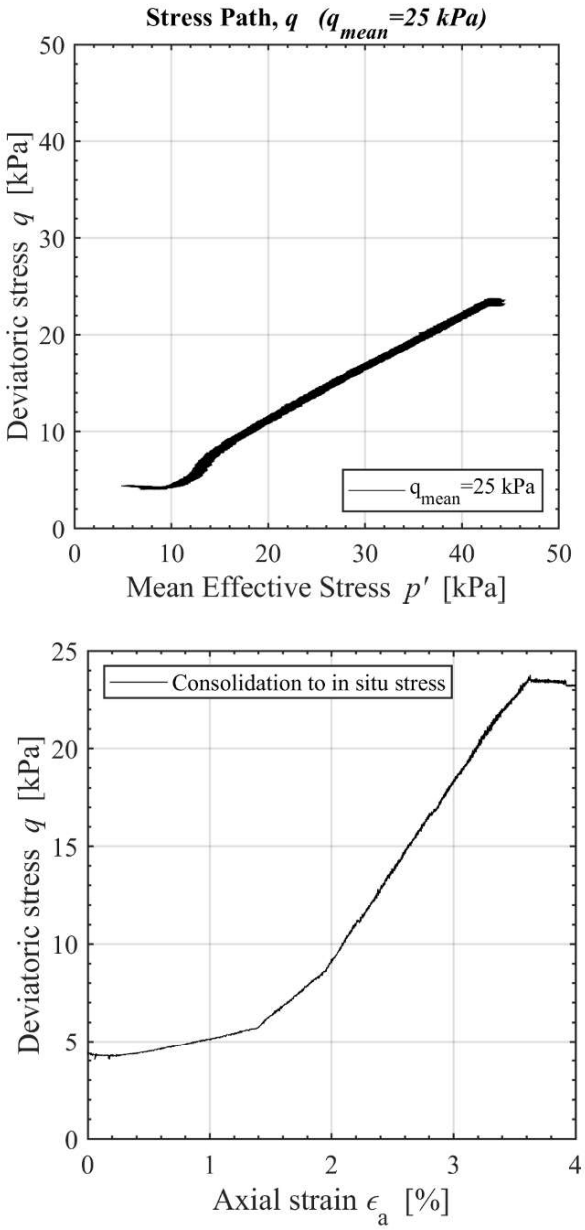


Figure 4.6: *Stress path during anisotropic consolidation stage (left) and deviatoric strain versus axial strain (right).*

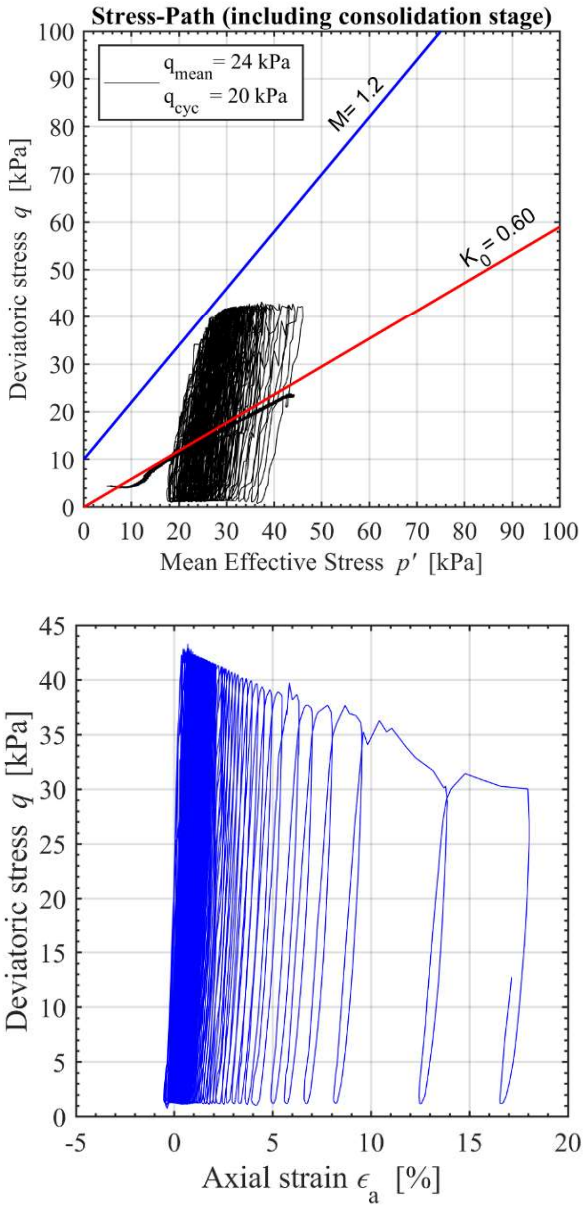


Figure 4.7: *Cyclic stage: stress path (left) and axial strain versus deviatoric stress (right).*

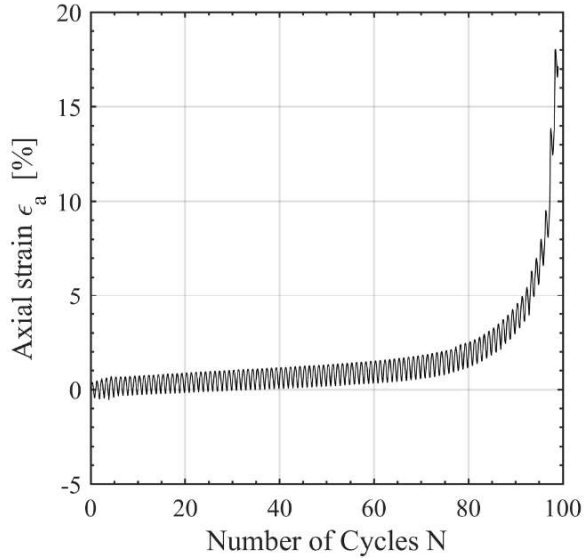


Figure 4.8: *Evolution of axial strain as function of the number of loading cycles.*

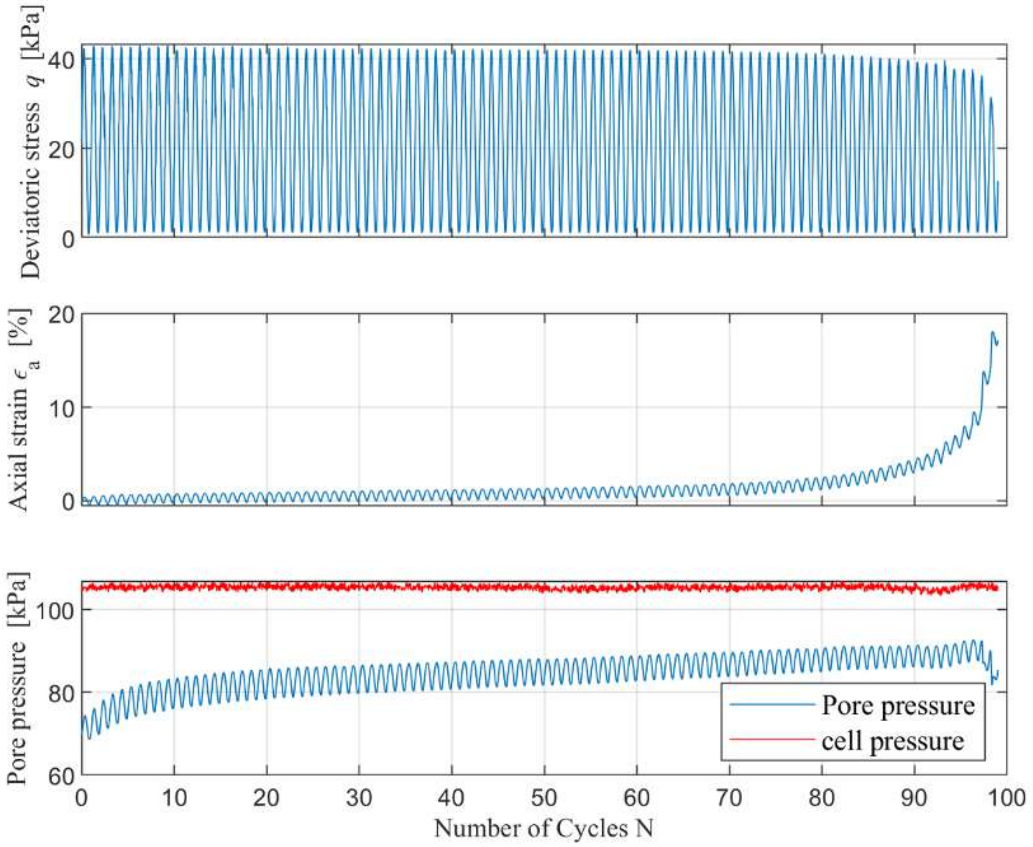


Figure 4.9: *Top to bottom: Evolution of deviatoric stress q, axial strain ϵ_a and pore water pressures with number of loading cycles N.*

Test C-2: depth $= 8\,\mathrm{m}$ **&** $f = 0.002\,\mathrm{Hz}$

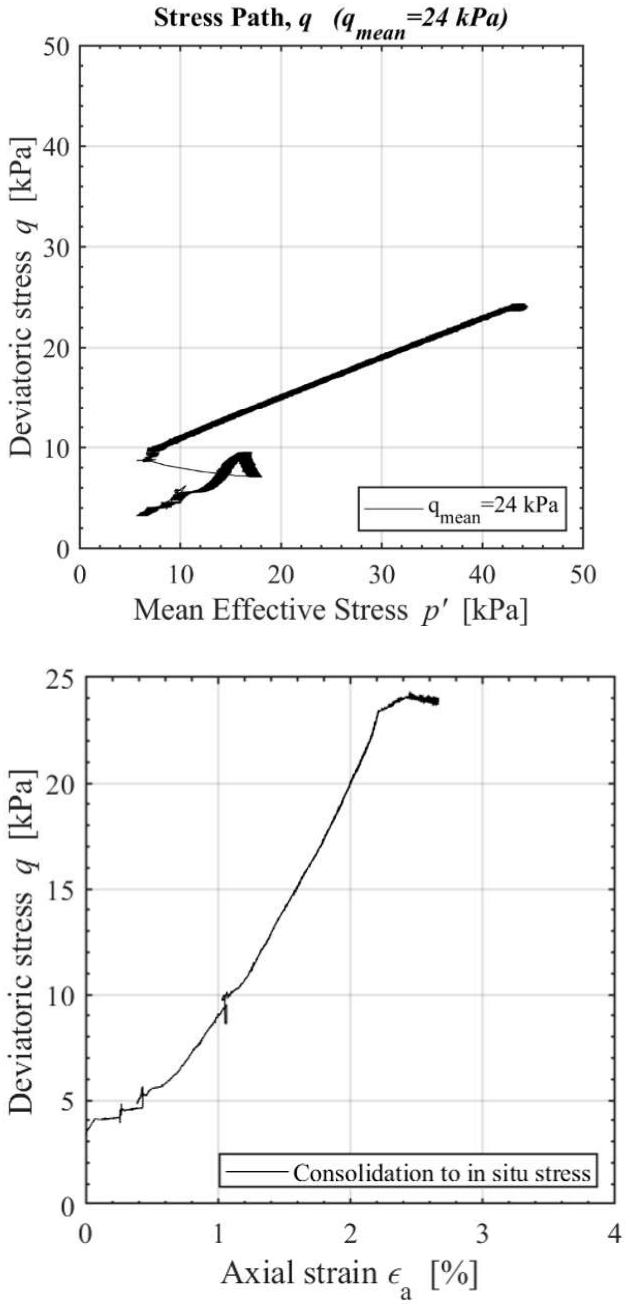


Figure 4.10: *Stress path during anisotropic consolidation stage (left) and deviatoric strain versus axial strain (right).*

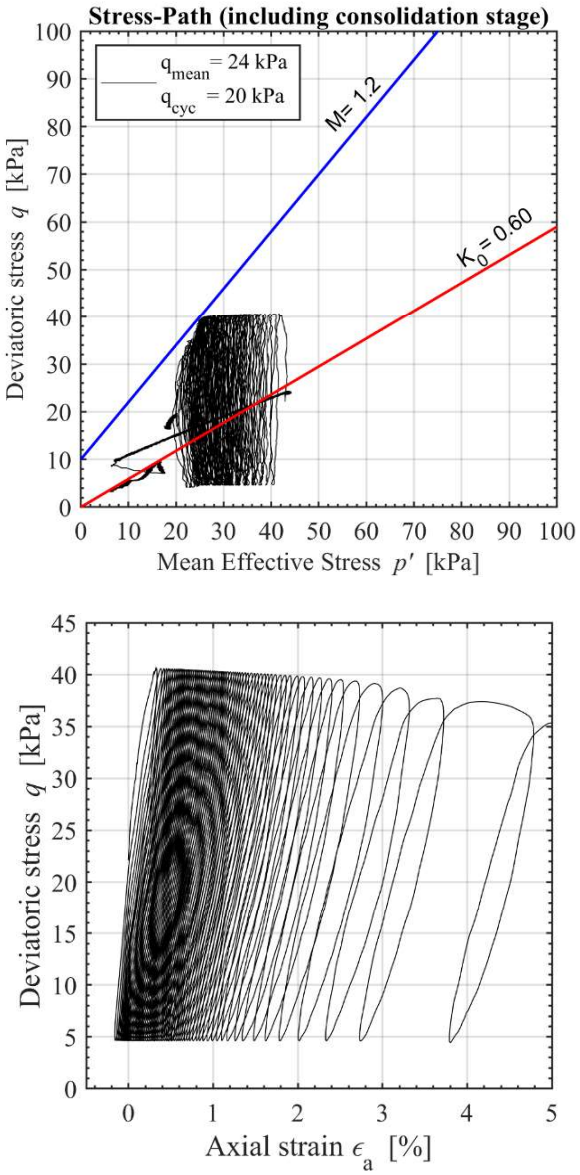


Figure 4.11: *Cyclic stage: stress path (left) and axial strain versus deviatoric stress (right).*

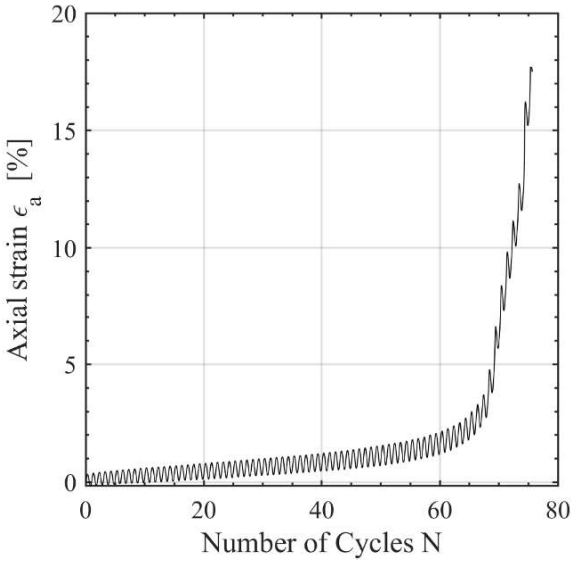


Figure 4.12: *Evolution of axial strain as function of the number of loading cycles.*

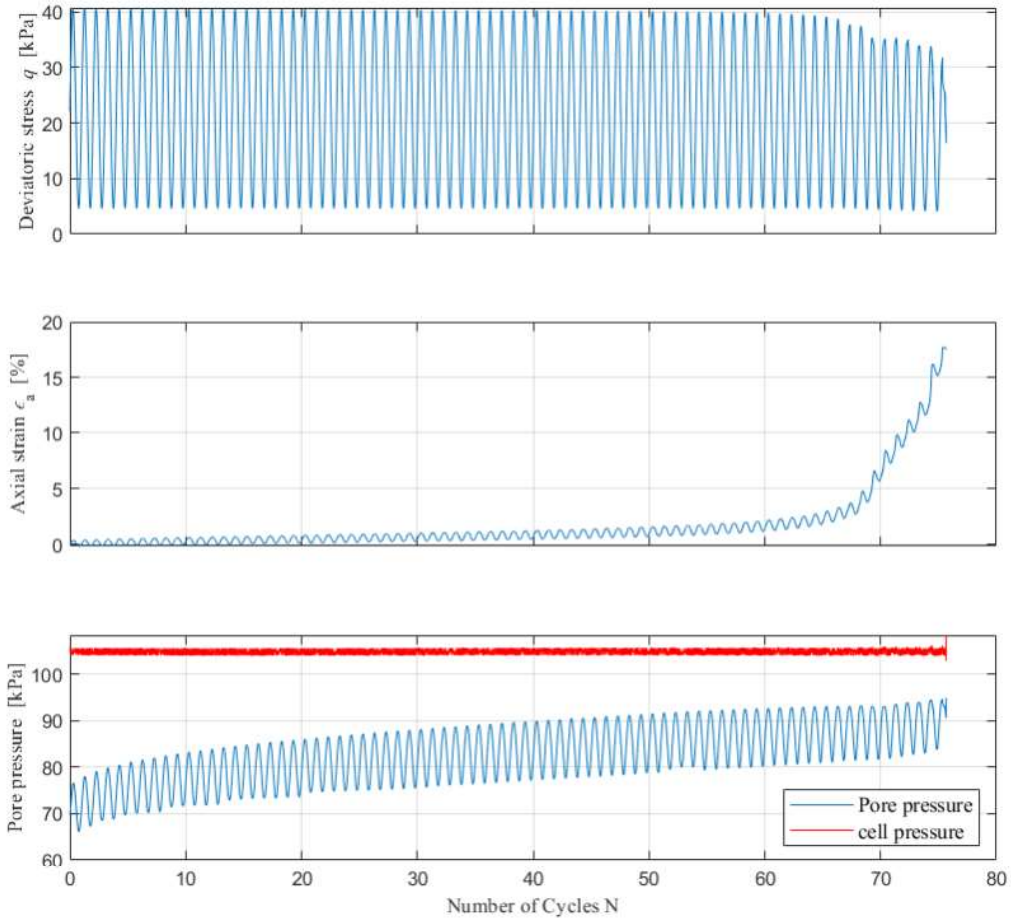


Figure 4.13: *Top to bottom: Evolution of deviatoric stress q, axial strain ϵ_a and pore water pressures with number of loading cycles N.*

Test C-3: depth $= 9\,\mathrm{m}$ **&** $f = 0.01\,\mathrm{Hz}$

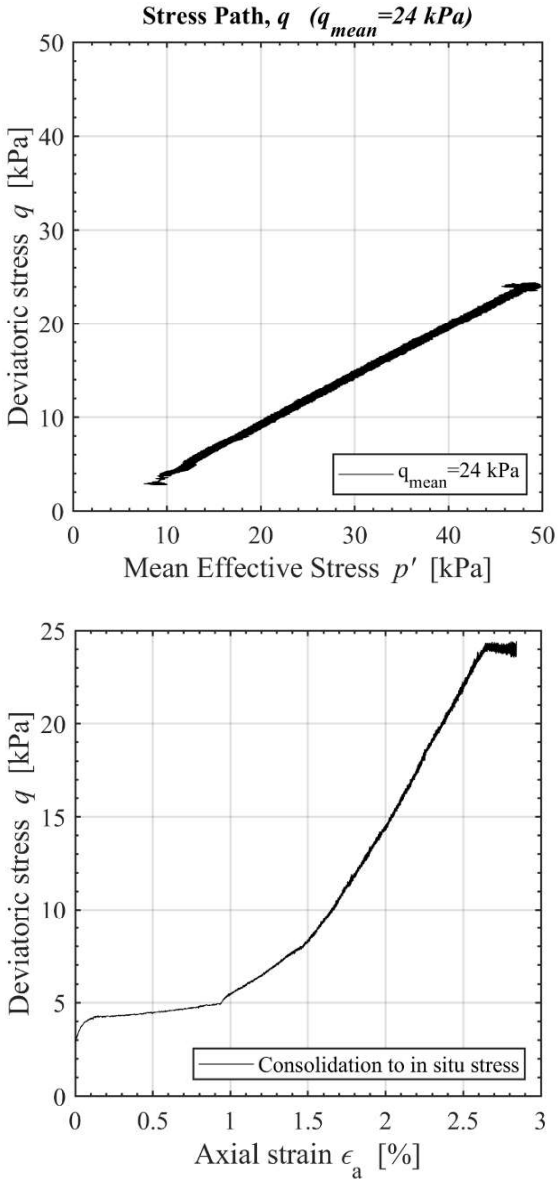


Figure 4.14: *Stress path during anisotropic consolidation stage (left) and deviatoric strain versus axial strain (right).*

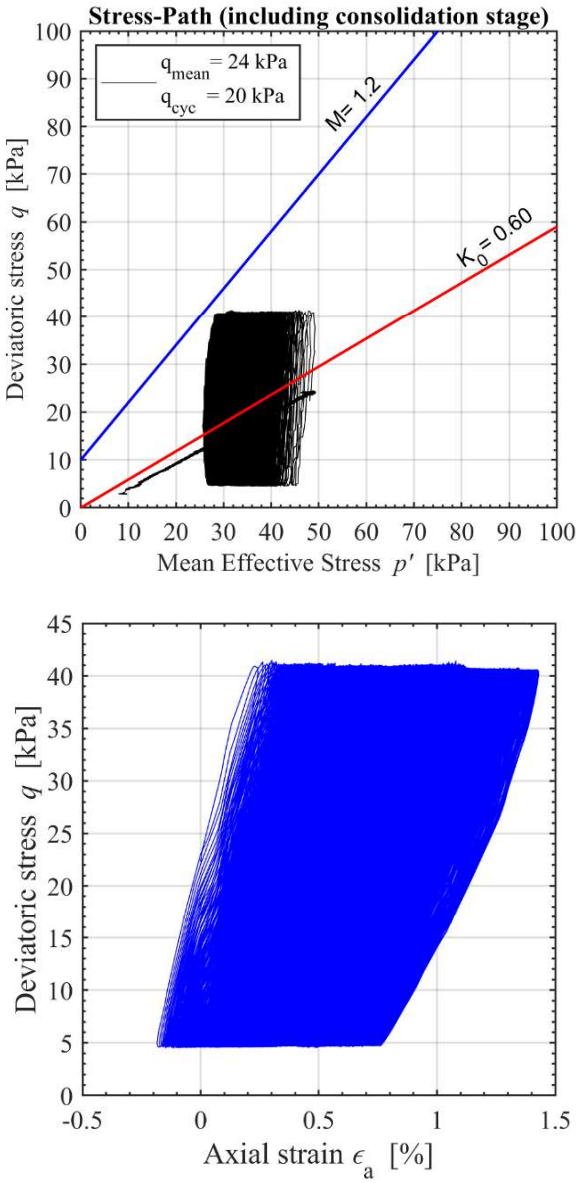


Figure 4.15: *Cyclic stage: stress path (left) and axial strain versus deviatoric stress (right).*

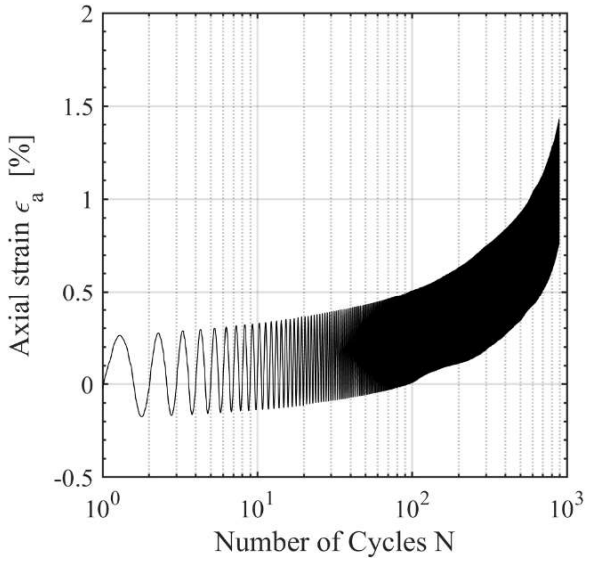


Figure 4.16: *Evolution of axial strain as function of the number of loading cycles.*

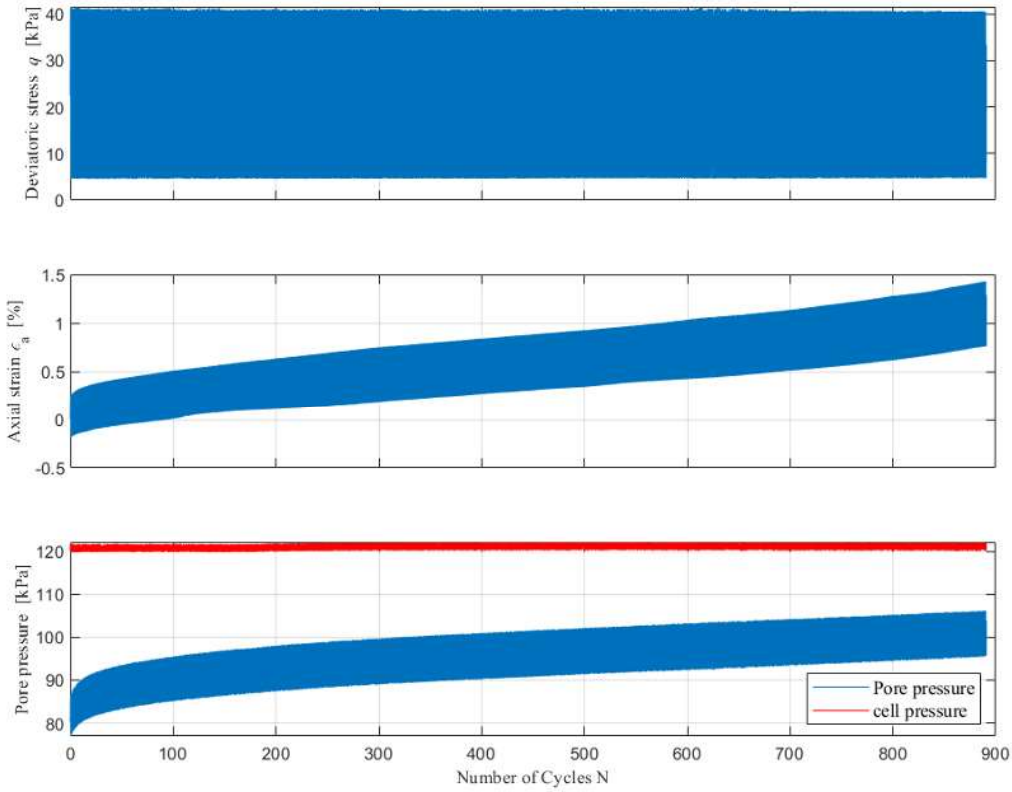


Figure 4.17: *Top to bottom: Evolution of deviatoric stress q, axial strain ϵ_a and pore water pressures with number of loading cycles N.*

Test C-4: depth $= 10\,\mathrm{m}$ **&** $f = 0.01\,\mathrm{Hz}$

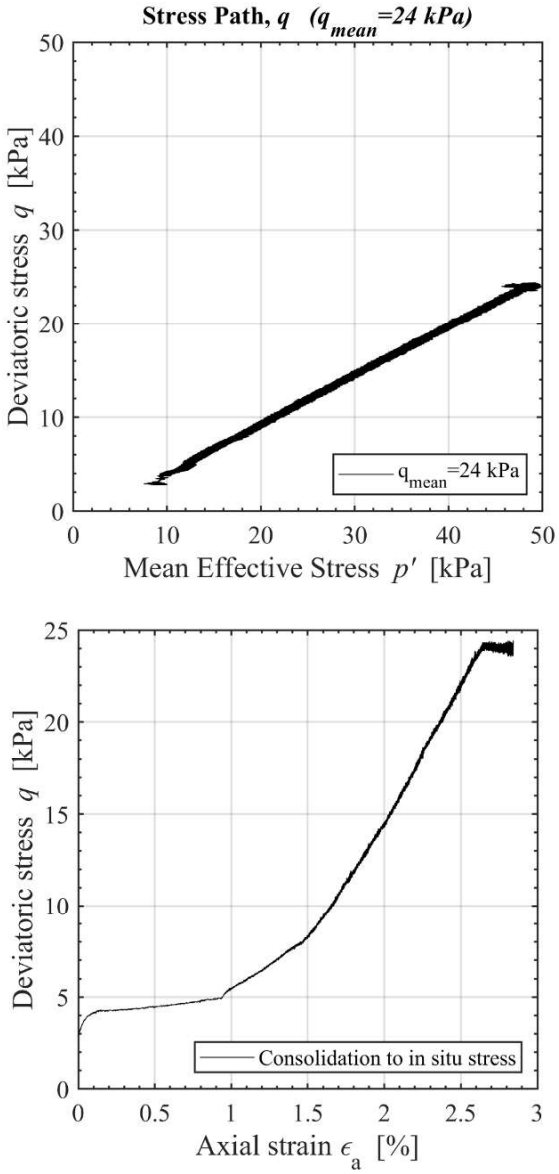


Figure 4.18: *Stress path during anisotropic consolidation stage (left) and deviatoric strain versus axial strain (right).*

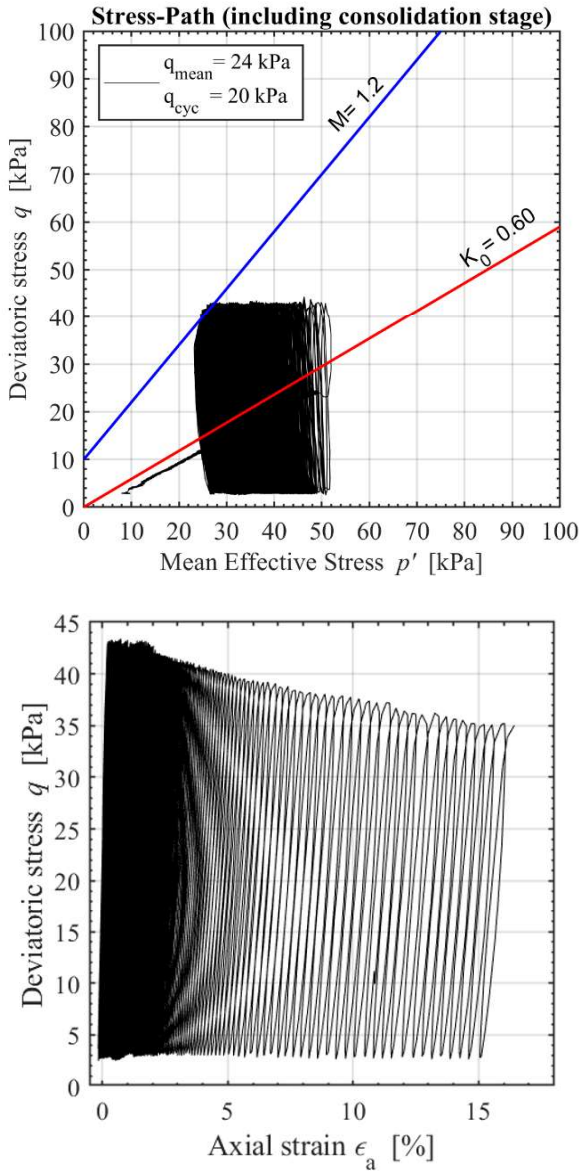


Figure 4.19: *Cyclic stage: stress path (left) and axial strain versus deviatoric stress (right).*

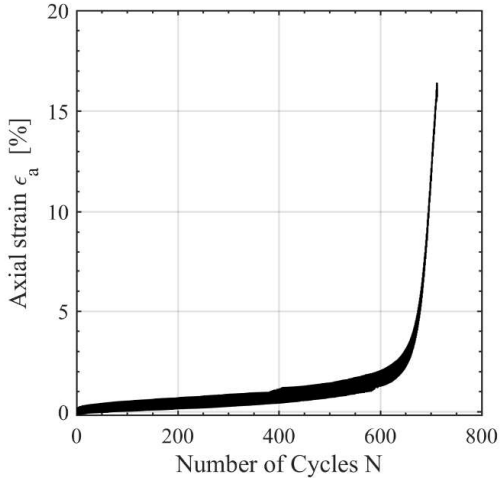


Figure 4.20: *Evolution of axial strain as function of the number of loading cycles.*

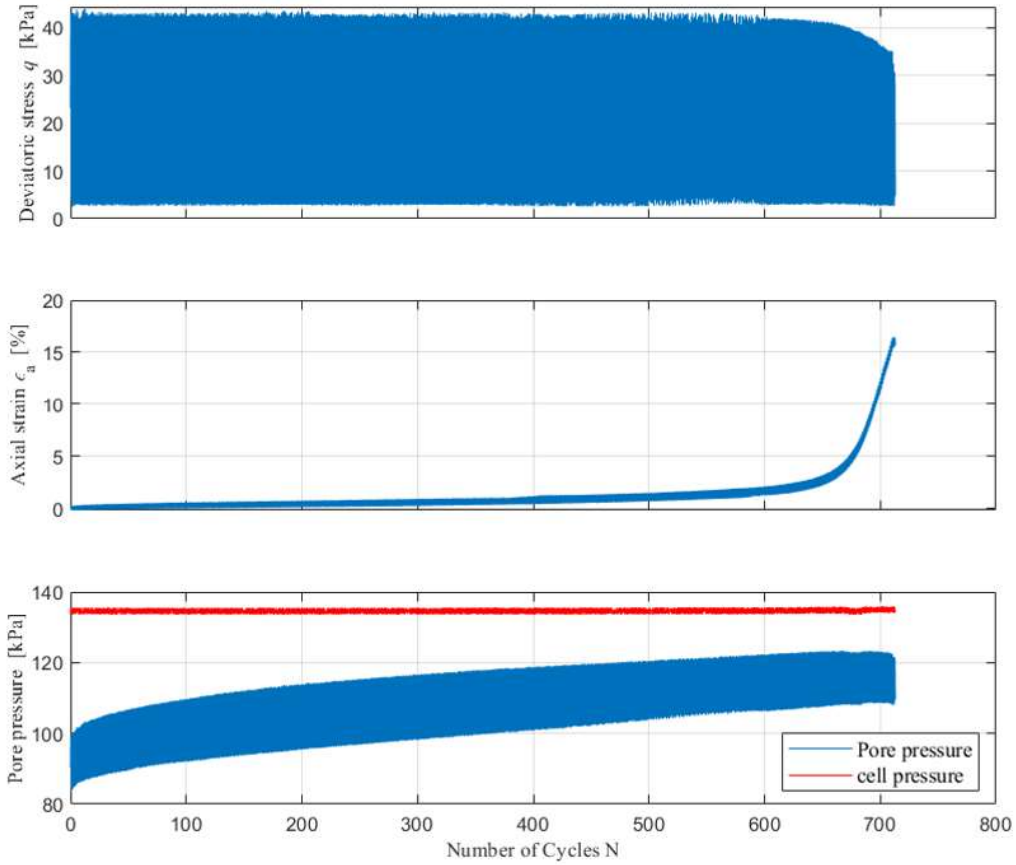


Figure 4.21: *Top to bottom: Evolution of deviatoric stress q, axial strain ϵ_a and pore water pressures with number of loading cycles N.*

4.2.4 Discussion of results

The limited results presented underline the capability of testing soft sensitive clays in triaxial conditions for up to 1000 loading cycles. During the consolidation stage in test C-2 the absolute displacements were reset. With the exception of one data point this has no impact on the subsequent strain calculations, however, the raw data is kept as is.

In all tests the mean deviatoric stress q_{mean} and cyclic devatior stress q_{cyc} is kept constant with regard to the undrained shear strength S_u. The only deviation is the change in loading frequency in test C-3. Hence, the expectation is that tests C-1, C-2, and C-4 would lead to a similar number of loading cycles until failure, in case differences between samples are negligible. This, clearly, is not the case. In test C-1 failure is reached at a low number of loading cycles, perhaps due to relatively large disturbance during consolidation to the anisotropic in-situ effective stress level. Furthermore, test C-4 did not reach a clear defined failure after a similar amount of load cycles as applied in test C-3. The largest difference between the two tests are in the generation of excess porewater pressures in the first few loading cycles, perhaps affected by the consolidation stage prior.

Given the tests were intended to first and foremost test the equipment, the sample depth and quality were not as rigorously controlled. It becomes apparent that small differences between samples can lead to large differences in the number of loading cycles until failure. Furthermore, it is hypothesized that as opposed to normal test procedures using monotonic loading paths, more strict termination criteria for the end of consolidation are required before commencing the cyclic shearing stage in (undrained) cyclic triaxial tests.

5 Conclusions, recommendations & impact

5.1 Conclusions

This report summarises the advances made at Chalmers on modelling and testing natural soft soils that are quasi-statically loaded with monotonic, and arbitrary stress paths. As such, the findings are directly applied in follow-up projects funded by Trafikverket, SBUF, FORMAS, Nordforsk and Vinnova. The main deliverables are summarised below:

- The development of a rate-dependent effective stress based constitutive model for natural soft soils: Creep-SCLAY1S (Sivasithamparam et al., 2015; Gras et al., 2018). The model in its generalised form is incorporated in a commercial finite element program, Plaxis, as a user-defined model that is available for other scientists and engineers.
- A Python based GUI toolbox for sensitivity analysis and multi-objective optimisation of advanced constitutive models in geotechnics (Gras et al., 2017; Gras et al., 2018). This is a genetic tool that can assist both future model development, calibration of model parameters as well as understanding which parameters are the most important to calibrate for a given application.
- New control software and hardware for cyclic triaxial testing of soft soils using a Bishop-Wesley cell and standard sized samples of 50 mm diameter and 100 mm height. This in combination with the new temperature-controlled GEO laboratory at Chalmers, enables for high quality cyclic testing of soft natural clays.
- A prototype setup for Constant Normal Stiffness testing of soft soils. This enables directly measuring the interface properties (soil/soil and soil/structure) as needed for advanced FE analyses.
- First steps towards a fully-fletched bubble model for soft soils. Further model development, supported by systematic testing, is however needed to have a model that can account for the small strain stiffness and cyclic degradation of soft sensitive clays. A robust FE implementation needs to be developed to ensure that the resulting model can be used in boundary-value level simulations without excessive computational time.

5.2 Recommendations

The recommendations are split into scientific recommendations and lessons learned for engineering practice.

5.2.1 Scientific recommendations

- Although the developments for an all-compassing model that captures a vast range of arbitrary loading paths with high fidelity are progressing reasonably well, more systematic testing, based on drained stress probing is required, to make informed decisions on the model formulation. Adequate scaling of the test conditions (test miniaturisation) should be considered to achieve this within a reasonable amount of time.

- For the large number of cyclic loads associated to operational loads of physical railway infrastructure, cyclic accumulation models should be further developed, starting from the Creep-SCLAY1S model. The Creep-SCLAY1Sc model under development at the Geotechnics Research Group at Chalmers is a good example of this.

- In addition to advanced optimisation strategies for model calibration against laboratory data, methods for data driven model updating using field monitoring data should be developed.

5.2.2 Engineering practice

- Future laboratory experiments designed to capture the cyclic response of soft soils should focus on a larger number of loading cycles with small loading amplitude, whilst capturing the (small) deformations and damping properties. For the purposes of the planned high speed rail, such testing should be done both for natural clays, as well as stabilised materials that are representative for lime cement columns, given the latter is often used in vibration mitigation as well as for ground improvement.

- Global Sensitivity Analyses and multi-objective optimisation strategies should be complemented with updating strategies based on monitoring data from operational infrastructure. This would support the rigorous application of Observational Method, as well as enable proactive maintenance and asset management.

5.3 Impact

The results of the project have had both direct and indirect impact. Creep-SCLAY1S model has been used as a basis for three doctoral theses at Chalmers: Wood (2016), Yannie (2016) and Li (2019), who all now work in industry in Sweden as geotechnical specialists. Their knowledge on the models have been applied in a practical context as part of the West Link project, where Creep-SCLAY1S model has been used in time-dependent analyses of deep excavations to support design innovations. The model is publicly available in Plaxis (as USDM soil model), which is the most common software for the Finite Element Analysis of geotechnical structures.

Most of the current PhD projects at Chalmers, many of them funded by Trafikverket/BIG, involve the application of the models developed in this project on various geotechnical applications, from embankments and slopes to deep excavations, pile installation as well as modelling high speed trains. When appropriately used, the models enable understanding and elaborating many complex phenomena encountered in geotechnical engineering practice. The Creep-SCLAY1S model is also used in education at Chalmers (year 5 MSc course ACE150 Soil Modelling and Numerical Analyses, has been running since 2019), and furthermore, the model has been used in tens of MSc projects at Chalmers.

Internationally, the models have been used both for research and consultancy projects in NUI Galway, Ireland, Norwegian Geotechnical Institute, Norway, and Deltares, the Netherlands. In the latter, the applicability of the model to simulate the response of peat has been explored, as well as the model is the basis for an to overlay for the effects of fibres that is under development. Furthermore, in the Netherlands, Creep-SCLAY1S is one of the recommended models for the advanced analysis of flood defences, see POVM (2020).

At Chalmers, the funding for this project and its results have enabled substantial growth in the number of both academic and research staff, from three permanent faculty staff in 2014 to five in 2020. In parallel, the project has contributed to high quality research output, with some already highly cited scientific papers, which has enabled Geotechnics Research Group at Chalmers to have success in sustained highly competitive research funding. The increase in research activity and staff numbers has also enabled doubling of the amount of credits for education in geotechnics, both at Bachelor and Master's level. As such, the funding has contributed towards increasing volume and (international) status of geotechnics education and research at Chalmers.

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BIG – Branschsamverkan i grunden
Forskningsprogram för effektiv och säker grundläggning av vägar och järnvägar

BIG

BIG – Branschsamverkan i grunden - är ett forskningsprogram för effektiv och säker grundläggning av transportsystemets infrastruktur. Programmet etablerades under senhösten 2013, och påbörjade sin verksamhet den 1 januari, 2014.

Målsättningen är att sänka kostnader för byggande och underhåll av transportsystemets infrastruktur genom ett långsiktigt och systematiskt utvecklingsarbete inom geoteknikområdet.

I BIG samverkar Trafikverket, Chalmers tekniska högskola, Luleå tekniska universitet, Kungliga tekniska högskolan och Statens geotekniska institut.

Resultat från BIG projekt publiceras i vetenskapliga artiklar och parternas egna rapportserier. Alternativt så publiceras rapporterna i denna gemensamma BIG rapportserie.

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