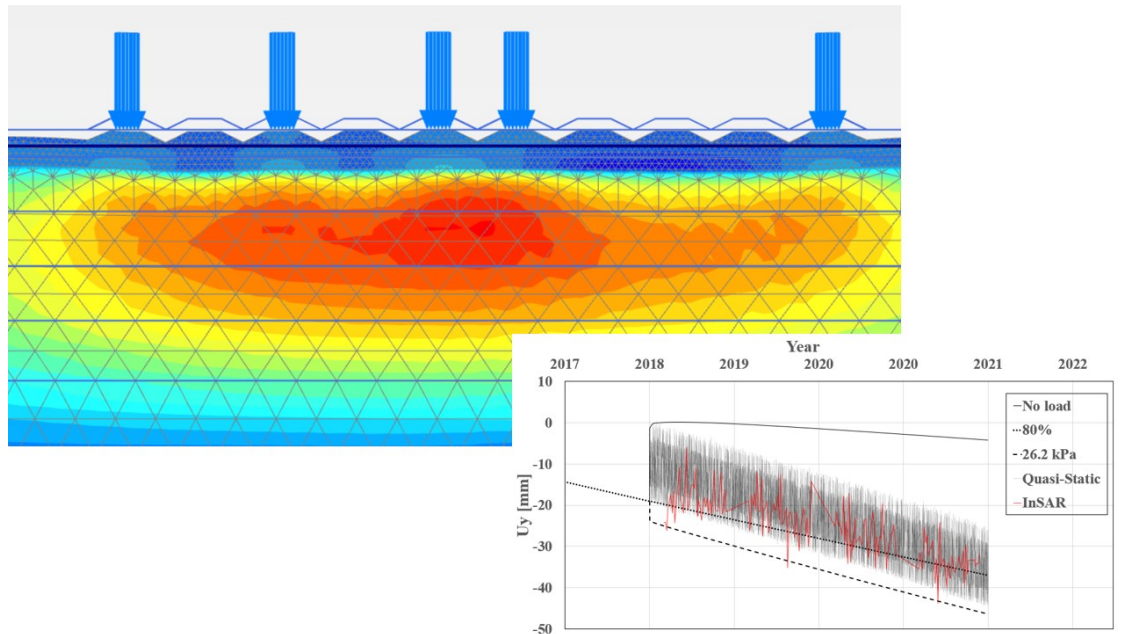




Quasi-static Loading Effects on a Marshalling Yard

Calibration of a Creep-SCLAY1S Model Using Satellite and In-Situ Measurements

Master's thesis in Infrastructure and Environmental Engineering

RICHARD BERG ARVID SANDBERG

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Gothenburg, Sweden 2026
www.chalmers.se

MASTER'S THESIS 2026

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Master's Thesis 2026

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Cover: A model for assessing quasi-static loading and associated settlement rates in comparison with InSAR measurements.

Typeset in L^AT_EX

Printed by Chalmers Reproservice

Gothenburg, Sweden 2026

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Abstract

Marshalling yards on soft clay deposits are subject to long-term settlements driven by consolidation and creep, posing challenges for track geometry maintenance and operational reliability. This thesis investigates time-dependent settlement behaviour at Sävenäs marshalling yard in Gothenburg, Sweden, through numerical modelling using the Creep-SCLAY1S constitutive model in PLAXIS 2D. The model accounts for anisotropy, bonding and rate-dependent deformation, making it well suited for the soft Gothenburg clay conditions at the site. Model parameters were calibrated using site-specific oedometer and triaxial test data, supplemented by theoretical relationships and literature values for similar Scandinavian clay deposits. The calibrated model was validated against two independent field datasets: bellow hose measurements tracking subsurface deformation at multiple depths, and surface settlement rates derived from Sentinel-1 InSAR observations. Good agreement was achieved at most monitoring locations, with simulated long-term settlement rates of approximately 4–5 mm/year consistent with satellite observations. The variability observed in InSAR data within the depot area is interpreted as reflecting cyclic vertical movements from operational loading rather than measurement noise. Operational loading was represented using a quasi-static approach, where train loads were activated and deactivated over time according to track occupancy patterns. The results show that excess pore pressure and settlements are most pronounced in the upper soil layers, with the influence of loading diminishing clearly with depth. Partial pore pressure dissipation between load cycles prevented progressive settlement acceleration, and the response approached a stable long-term condition consistent with partially drained behaviour. For preliminary assessments, a simplified constant load of approximately 50% of the full load was found to produce comparable results to the quasi-static scheme if only settlements in the clay is of interest. The study demonstrates that advanced constitutive modelling combined with remote sensing validation provides a reliable framework for assessing long-term settlement behaviour in railway infrastructure on soft clay.

Keywords: Soft clay, long-term settlements, Creep-SCLAY1S, quasi-static loading, bellow hose, marshalling yard, InSAR.

Acknowledgements

We would like to express our sincere gratitude to our supervisor, Ayman Abed at Chalmers University of Technology, for his continuous guidance, support and valuable insights throughout this project. His expertise and constructive feedback have been essential to the development and completion of this study.

We would also like to thank Khaled Alaydi at WSP for his engagement, practical input and support, which have contributed significantly to the quality and relevance of this work. Furthermore, we greatly appreciate the warm welcome at WSP and the inclusive environment, which made us feel part of the team and provided valuable industry insight throughout the project.

Richard Berg, Arvid Sandberg

Gothenburg, June 2026

Nomenclature

Below is the nomenclature of indices, sets, parameters, and variables that have been used throughout this thesis.

Greek symbols

α	Inclination of the Normal Compression Surface
α_0	Initial inclination of the Normal Compression Surface
β	Creep stress-dependency exponent
χ	Bonding variable
χ_0	Initial bonding
η	Stress ratio
γ	Unit weight
κ^*	Modified swelling index
λ^*	Modified compression index
λ_i^*	Modified intrinsic compression index
μ^*	Modified creep index
μ_i^*	Modified intrinsic creep index
ν'	Effective Poisson's ratio
σ	Total stress
σ'	Effective stress
σ'_h	Effective horizontal stress
τ	Reference time
θ	Lode angle
ψ	Dilatancy angle
ω	Absolute effectiveness of rotational hardening
ω_d	Relative effectiveness of rotational hardening
ξ	Absolute effectiveness destructuration

ξ_d	Relative effectiveness destructuration
ε_q	Deviatoric strain
ε_q^c	Creep deviatoric strain
ε_q^e	Elastic deviatoric strain
ε_v	Volumetric strain
ε_v^c	Creep volumetric strain
ε_v^e	Elastic volumetric strain

Roman symbols

a	Absolute effectiveness destructuration
b	Relative effectiveness destructuration
c_k	Coefficient of hydraulic conductivity change
e	Void ratio
E_{oed}^{ref}	Reference oedometer stiffness
K	Bulk modulus
K_0	Coefficient of earth pressure at rest
M	Critical state stress ratio
M_c	Critical state stress ratio in compression
M_e	Critical state stress ratio in extension
OCR	Overconsolidation ratio
p'	Mean effective stress
p'_c	Apparent preconsolidation pressure
p'_{eq}	Equivalent stress
p'_p	Size parameter of the Normal Compression Surface
POP	Pre-overburden pressure
q	Deviatoric stress

Acronyms

Below is the list of acronyms that have been used throughout this thesis listed in alphabetical order:

BLG	Bellow Hose
CRS	Constant Rate of Strain
CSL	Critical State Line
CSS	Current Stress Surface
FEM	Finite Element Method
ICS	Intrinsic Compression Surface
IL	Incremental Loading
InSAR	Interferometric Synthetic Aperture Radar
MC	Mohr–Coulomb
NCS	Normal Compression Surface
PLAXIS	Finite element software for geotechnical analysis
SGI	Swedish Geotechnical Institute
STVM	Största Tillåtna Vikt per Meter

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1

Introduction

The increasing demand for railway transportation places greater requirements on the capacity, reliability, and performance of railway infrastructure. In this context, marshalling yards have a critical role in ensuring efficient operation and maintenance of rolling stock. The construction and operation of such facilities impose demanding requirements on long-term performance, as yards are subjected to varying loads from trains while tolerances for track geometry remain strict [1]. Even relatively small differential settlements may lead to increased maintenance costs, operational disruptions and in the worst case, safety risks.

As marshalling yards are subjected to sustained and slowly varying loads, an accurate description of both stress-dependent stiffness and time-dependent deformation is essential. A key challenge is to predict the total settlement over the service life, including the contributions from primary consolidation and creep, as well as the risk of differential settlements affecting track performance. Although railway traffic introduces repeated loading, the yard is characterised by low train velocities and extended stationary periods, meaning that loading frequencies are low and inertial effects can be neglected [2]. The loading can therefore be treated as quasi-static, with operational variability represented using an operational loading approach based on time-dependent activation of static loads.

To capture the complex behaviour of soft clays typically encountered in Scandinavian conditions, advanced constitutive models are required [3]. These clays are characterised by high compressibility, low permeability and pronounced time-dependent deformation, leading to settlements that continue long after construction. Creep-SCLAY1S, the model adopted in this study, builds on the foundation in the S-CLAY1 model, [4]. It further accounts for anisotropy and rate dependency from Leoni et al. [5], and Grimstad et al. [6]. Moreover, it includes the effects of interparticle bonding and destructuration. This enables simulation of the gradual degradation of soil structure over time, consistent with established frameworks in [7], [8]. The applicability of the model has been demonstrated and validated against several natural and reconstituted clays. Karstunen et al. [9] and Sivasithamparam et al. [10], successfully validated the model against monitoring data in a test embankment in Murro, Finland. It has also been benchmarked against a project in Gothenburg by Tornborg et al. [11], making it a suitable choice for the present analysis.

This study highlights the importance of model choice in the design of railway infrastructure on soft soils. The Creep-SCLAY1S model is briefly described, followed by the definition of the embankment geometry, material parameters and initial conditions used in the numerical analyses. The results are compared with field observations obtained from Interferometric Synthetic Aperture Radar (InSAR) and bellow hose measurements. Finally, predicted settlements from quasi-static loading conditions in the analysed marshalling yard are evaluated against simplified loading methods.

1.1 Background

Sävenäs marshalling yard, constructed 1914 serves as an operational hub for regional passenger and freight traffic in western Sweden. It is one of eight active marshalling yards in Sweden. The yard consists of an entrance group (I-group) and a stationary R-group comprising 34 tracks arranged in sequence, as shown in Fig. 1.1. The layout includes variations in elevation, with a high point where trains are guided before entering the classification tracks, and a low point located within the R-group, forming a transition towards the adjacent depot area. Current capacity utilisation differs between the two groups, with approximately 28 percent utilisation in the I-group and 83 percent in the R-group.

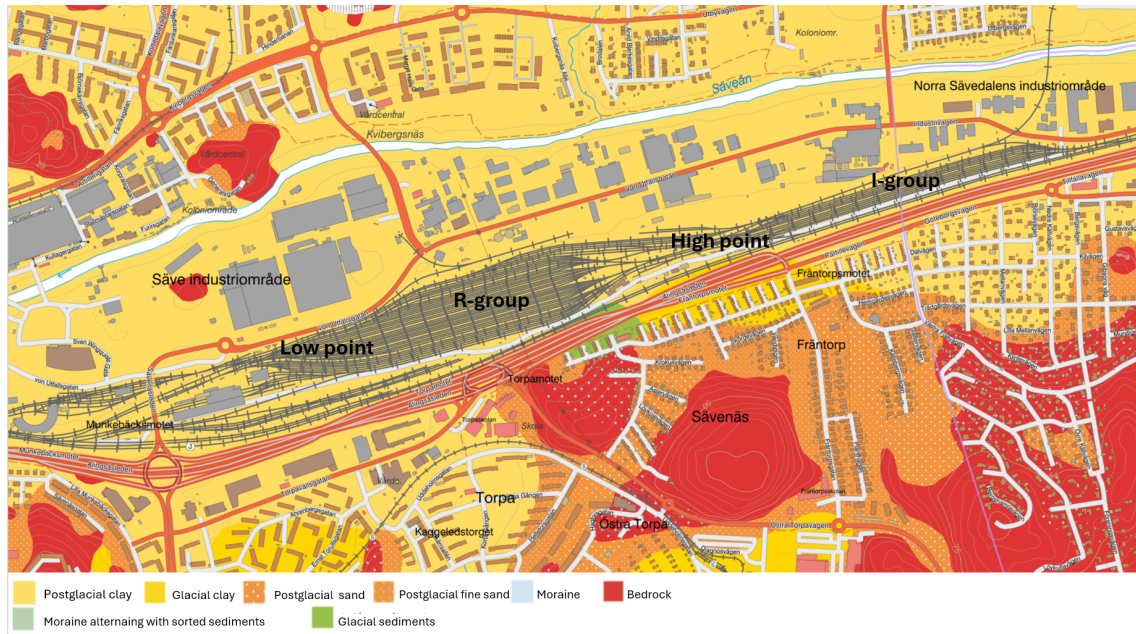


Figure 1.1: Map of Sävenäs marshalling yard with soil types. Credits: Lantmäteriet.se

The facility faces significant challenges related to both its current condition and future operational demands [12]. Substantial settlements have led to a deteriorated state, particularly in the low-lying areas where deformation is most pronounced. At

the same time, increasing traffic volumes towards 2040 are expected to place further demands on the yard, with the R-group approaching or exceeding its capacity.

The site is underlain by typical Gothenburg clay and has been subjected to repeated filling operations, both contributing to an increase in excess pore pressures and long-term settlements. Together with repeated loading over extended periods, this results in a complex ground response that is difficult to assess. Consequently, predicting how settlements will develop over time and affect track performance remains a key challenge.

Reconstruction of the facility is currently being carried out by the Swedish Transport Administration (Trafikverket) in collaboration with the consultancy company WSP. This study is conducted in connection with the project. Field measurements, including InSAR observations and bellow hose data, are available and provide a basis for assessing settlement behaviour.

1.2 Aim and Objectives

This thesis aims to investigate time-dependent settlement behaviour at the Sävenäs marshalling yard through numerical modelling, with particular focus on parameter selection and the representation of quasi-static loading.

The objectives of this thesis include:

- Calibration of Creep-SCLAY1S model parameters in PLAXIS 2D using site-specific geotechnical data.
- Comparison of model predictions with field measurements from InSAR and bellow hose observations.
- Evaluation of the influence from quasi-static loading on settlements compared to simplified loading conditions.

1.3 Societal, Ethical and Ecological Impacts

This study contributes to sustainable infrastructure by improving the understanding of settlements caused by quasi-static loading on soft clay. A more reliable prediction will reduce the risk of disruptions and improve the safety and reliability of the railway infrastructure. From a societal perspective, this will support a more efficient transportation system, which will contribute to society.

Historically, since the original construction in 1914, repeated embankment fillings have been necessary. From an ecological perspective, extending the service life reduces the need for future reconstruction and the associated consumption of construction materials and natural resources. Since the analysis in this project indicates a quasi-static loading behaviour that generates time-dependent deformation, the re-

sults can help prevent the need for additional improvement measures during the lifespan of the new marshalling yard. By improving the understanding and prediction of long-term settlements, the need for such repetitive measures can be minimised, supporting a more sustainable life-cycle approach to railway infrastructure.

1.4 Declaration of AI

Artificial Intelligence (AI) tools, primarily ChatGPT (OpenAI) and AI-assisted coding features in Visual Studio Code was used in this thesis. They were utilised for improving the structure and clarity of the text, language editing and assisting in the development and debugging Python scripts used in the thesis.

All AI-generated suggestions was reviewed and modified if necessary. The methodology, numerical modelling, interpretation of the results, and conclusions presented in this thesis were developed by the authors, who take full responsibility for the presented content and accuracy of the work.

1.5 Limitations

This thesis is subject to methodological and practical limitations that should be taken into consideration when interpreting the results.

- All numerical analyses were performed assuming plane strain conditions, which do not account for three-dimensional effects such as lateral variability in soil conditions and localised load distribution from individual tracks.
- The construction history of the yard is poorly documented, with limited information available regarding the timing, staging and magnitude of historical filling operations.
- Train loads are represented as static surface pressures based on given STVM values, with a single maximum load magnitude assigned to all active tracks regardless of train type.
- The quasi-static loading scheme was based on representative occupancy patterns obtained through discussions with Trafikverket personnel rather than actual recorded operational data. The assumed daily load cycle may therefore not fully reflect the true variability in train frequency.
- Triaxial test data within the analysed area were limited, with only two samples available from the broader site investigation. Since several key relationships in the Creep-SCLAY1S model are governed by shear behaviour, this restricts the reliability of the calibrated strength and anisotropy parameters.

Theory

2.1 Mechanical Behaviour of Natural Soft Clays

Natural soft clays exhibit a complex mechanical response governed by stress history, fabric anisotropy, bonding and time-dependent deformation. These characteristics arise from the geological origin of the material and its subsequent aging and loading history. Unlike reconstituted soils, natural clays possess a structured fabric that influences both stiffness and strength which evolves progressively during deformation.

Under increasing effective stress, soft clays undergo significant volumetric compression. In semi-logarithmic stress space, this behaviour is represented by a normal consolidation line and a flatter unloading-reloading line, seen in Fig. 2.1. The transition between these regimes defines the apparent preconsolidation pressure p'_c , which reflects both past stress history and structural bonding effects.

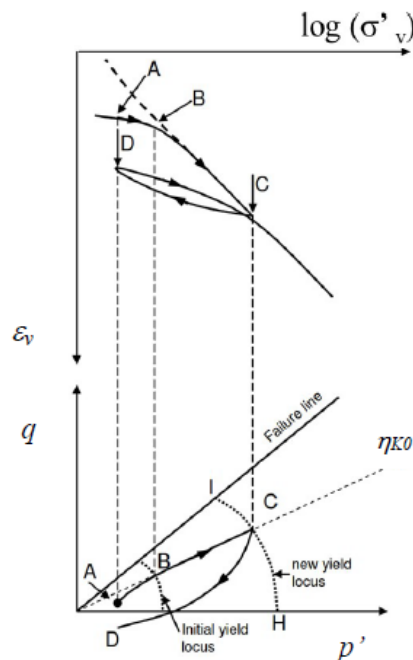


Figure 2.1: Typical compression behaviour of natural soft clay showing the normal consolidation line and apparent preconsolidation pressure [13].

In natural clays, the apparent overconsolidation may originate from ageing and bonding rather than mechanical unloading. As irreversible strains accumulate, de-structuration reduces structural strength and modifies compressibility. Shear and volumetric deformations are intrinsically coupled, and even minimal one-dimensional loading conditions involve deviatoric strain components.

A further defining feature of soft clays is time-dependent deformation. Strains continue to accumulate under constant effective stress conditions, indicating that creep is an intrinsic component of the material response. This behaviour is ideally considered when describing the long-term response of natural soft clays.

2.2 Creep-SCLAY1S Constitutive Model

Creep-SCLAY1S is an anisotropic elasto-viscoplastic constitutive model developed for natural soft clays. The model was developed at Chalmers University of Technology and has been implemented in PLAXIS. It accounts for time-dependent deformation, anisotropy and bonding, making it suitable for analysing long-term behaviour in soft soil deposits. The model is implemented in a full three-dimensional stress space formulation. However, for the sake of simplicity, only the triaxial representation is presented here. The complete tensor formulation is provided in Appendix A.1.

The model is formulated in terms of effective stress invariants. The mean effective stress p' and deviatoric stress q are defined as:

$$p' = \frac{1}{3}(\sigma'_1 + \sigma'_2 + \sigma'_3) \quad (2.1)$$

$$q = \sigma'_1 - \sigma'_3 \quad (2.2)$$

where σ' is the effective stress. These invariants describe the stress state in p' - q space and form the basis for the constitutive surfaces governing creep behaviour.

2.2.1 Strain Decomposition and Elastic Behaviour

Following classical elasto-viscoplastic theory, total strain increments are decomposed into elastic and creep components,

$$d\varepsilon_v = d\varepsilon_v^e + d\varepsilon_v^c \quad (2.3)$$

$$d\varepsilon_q = d\varepsilon_q^e + d\varepsilon_q^c \quad (2.4)$$

This decomposition reflects the fundamental assumption that the total deformation of the soil can be separated into a recoverable part, governed by elastic stiffness, and an irrecoverable time-dependent part associated with viscoplastic creep. While the elastic component dominates within the overconsolidated region, the creep component accumulates continuously and becomes increasingly significant as the stress state approaches and exceeds the preconsolidation pressure.

The elastic response is assumed isotropic and stress-dependent. The elastic volumetric strain increment is expressed as:

$$d\varepsilon_v^e = \frac{dp'}{K} \quad (2.5)$$

where the effective bulk modulus K , is defined by:

$$K = \frac{p'}{\kappa^*} \quad (2.6)$$

Here κ^* is the modified swelling index obtained from unloading–reloading behaviour. The shear modulus follows from isotropic elasticity using effective Poisson’s ratio ν' . The elastic formulation primarily governs recompression and unloading paths, whereas long-term settlements are dominated by creep strains.

It is important to note that unlike classical elastic-plastic models, Creep-SCLAY1S does not define a strictly elastic domain. Even within the overconsolidated region, creep strains may develop under sustained loading, consistent with experimental observations of time-dependent deformation in natural soft clays. The elastic formulation therefore primarily governs the immediate and recompression response, while long-term settlements are dominated by the creep component introduced in subsequent sections.

2.2.2 Surfaces and Stress State

The time-dependent behaviour of Creep-SCLAY1S is governed by a bounding surface referred to as the Normal Compression Surface (NCS). The NCS defines the stress level relative to which creep strain rate and direction are determined. Unlike classical plasticity models, where a yield surface separates elastic and plastic domains, the NCS in Creep-SCLAY1S controls the rate of viscoplastic deformation without defining a purely elastic region.

In triaxial stress space, the NCS is expressed as:

$$f_{NCS} = (q - \alpha p')^2 - (M(\theta)^2 - \alpha^2)[p'_p - p']p' = 0 \quad (2.7)$$

where α is a state variable representing the inclination of the surface and therefore

the degree of anisotropy, and p'_p controls the size of the surface. The stress ratio parameter $M(\theta)$ varies with Lode angle in order to represent different strengths in compression and extension and to ensure convexity of the surface in three-dimensional stress space.

The inclination parameter α enables representation of stress-induced anisotropy, while p'_p evolves as creep strains accumulate. The surface therefore expands and rotates during loading, reflecting changes in stress history and soil fabric.

The geometric interpretation of the model in p' - q space is illustrated in Fig. 2.2. The figure shows the NCS, the Current Stress Surface (CSS), the anisotropy parameter α , the equivalent stress p'_{eq} and the creep strain increment direction $\delta\varepsilon^c$. Note that the figure also includes the Intrinsic Compression Surface (ICS), which represents the fully destructured material response and will be introduced in Section 2.2.4.

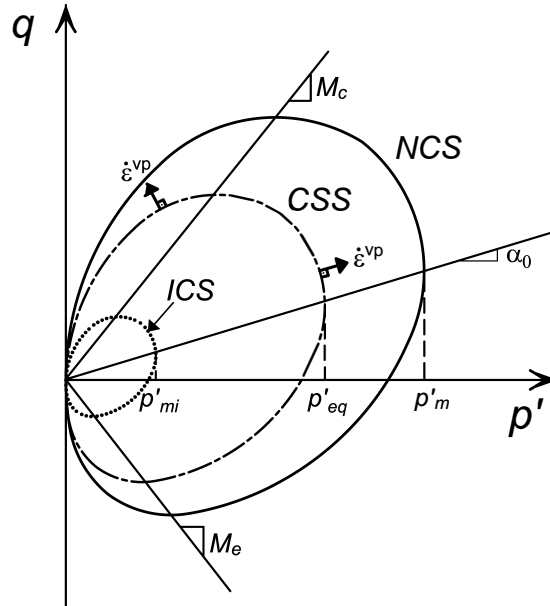


Figure 2.2: Geometric representation of the Creep-SCLAY1S model in p' - q . From [14]

The CSS represents the instantaneous stress state relative to the NCS. Its position determines the equivalent stress p'_{eq} , which governs the magnitude of the viscoplastic multiplier and therefore the creep strain rate. The normal to the NCS defines the direction of creep strain increments under the associated flow rule.

2.2.3 Creep and Viscoplastic Flow

Once the bounding surface framework is established, the rate of creep deformation is governed by a viscoplastic multiplier $d\Lambda$, defined empirically as:

$$d\Lambda = \frac{\mu_i^*}{\tau} \left(\frac{p'_{eq}}{p'_p} \right)^\beta \left(\frac{M_c^2 - \alpha_{K_0}^2}{M^2 - \eta_{K_0}^2} \right) \quad (2.8)$$

with:

$$\beta = \frac{\lambda_i^* - \kappa^*}{\mu_i^*} \quad (2.9)$$

Here μ_i^* is the intrinsic creep index, reflecting the rate of viscoplastic straining of the fully destructured material, and τ is a reference time that defines the time scale over which creep is measured, typically taken as one day in practice. The modified intrinsic compression index λ_i^* reflects the compressibility of the fully destructured material along its normal consolidation line in volumetric strain space. The exponent β controls the sensitivity of the creep rate to the stress ratio and depends on the ratio between the compressibility of the material and its creep index.

The stress ratio p'_{eq}/p'_p represents the relative position of the current stress state with respect to the NCS and is the primary driver of creep rate. When the current stress state lies close to the NCS, the ratio approaches unity and creep rates are high. Conversely, for stress states well within the overconsolidated region, the ratio is small and creep deformation proceeds slowly. This stress dependency gives rise to isotach behaviour, meaning that the current strain rate is uniquely determined by the current stress state, independent of the previous loading path. This is consistent with experimental observations in natural soft clays where different loading histories converge to the same creep rate at the same effective stress level.

An associated flow rule is adopted, meaning that creep strain increments are directed normal to the NCS in stress space:

$$d\epsilon^c = d\Lambda \frac{\partial f_{NCS}}{\partial \sigma'} \quad (2.10)$$

The magnitude of creep deformation is therefore jointly controlled by the distance of the current stress state from the NCS, through the multiplier $d\Lambda$, and by the geometry of the surface itself, through the flow direction.

2.2.4 Bonding and Surface Evolution

As described in Section 2.1, natural clays possess a structured fabric developed through geological history and post-depositional processes, resulting in additional strength beyond what would be expected for a reconstituted material at the same void ratio. To represent this behaviour within the Creep-SCLAY1S framework, the ICS is introduced. The ICS represents the fully destructured material response and is related to the NCS through the bonding variable χ :

$$p'_p = (1 + \chi)p'_i \quad (2.11)$$

When $\chi = 0$, the NCS coincides with the ICS and the clay behaves as reconstituted material. The ICS is not fixed, but evolves as irreversible volumetric deformation

accumulates. As the soil compresses under creep, the ICS expands to reflect the progressive densification of the material, expressed as:

$$dp'_i = \frac{p'_i d\varepsilon_v^c}{\lambda_i^* - \kappa^*} \quad (2.12)$$

Simultaneously, bonding degrades as creep strains accumulate:

$$d\chi = -a\chi (|d\varepsilon_v^c| + b|d\varepsilon_q^c|) \quad (2.13)$$

where a and b (also referred to as ξ and ξ_d) govern the rate of destructuration due to volumetric and deviatoric creep strains respectively. As bonding decreases, the NCS gradually approaches the ICS, reflecting the progressive loss of natural soil structure over time. This process is illustrated in Fig. 2.2, which shows how the stress path of an intact clay converges toward that of a reconstituted sample as destructuration progresses. Closely linked to this evolution is the development of anisotropy. The natural deposition and loading history of soft clays results in a fabric that is inherently directional, and this directionality is captured through the orientation of the NCS. As creep strains accumulate and the soil structure degrades, the orientation of the NCS evolves through a rotational hardening law:

$$d\alpha = \omega \left[\left(\frac{3\eta}{4} - \alpha \right) d\varepsilon_v^c + \omega_d \left(\frac{\eta}{3} - \alpha \right) |d\varepsilon_q^c| \right] \quad (2.14)$$

The parameters ω and ω_d control the rate and character of this rotation respectively, with ω governing the overall rate of rotation and ω_d defining the relative contribution of deviatoric strains [4]. Together, the destructuration and rotational hardening mechanisms ensure that both the size and orientation of the NCS adapt continuously to the evolving stress history and fabric of the soil, providing a more realistic representation of natural soft clay behaviour under long-term loading.

2.3 Model Parameters

The Creep-SCLAY1S model involves several material parameters that differ in their physical origin and method of determination. To maintain clarity, the parameters are organised according to the mechanisms they govern, and their determination is discussed in the following subsections. A complete summary is provided in Table 2.1.

2.3.1 Anisotropic Parameters

Anisotropy in the model is described through the initial state variable α_0 , representing the initial inclination of the NCS, and through the parameters ω and ω_d that

control its evolution during loading. For normally consolidated or lightly overconsolidated soils, α_0 can be estimated from the stress conditions at deposition [4]. If the clay deposit has primarily experienced one-dimensional consolidation, it is reasonable to represent the initial anisotropy as cross-anisotropy, with the stress ratio corresponding to the normally consolidated K_0 state. The stress ratio η_{K_0} is then estimated using Jaky's formula:

$$K_0 = 1 - \sin \varphi' \quad (2.15)$$

$$\eta_{K_0} = \frac{3M_c}{6 - M_c} \quad (2.16)$$

From this, the initial inclination α_{K_0} follows as:

$$\alpha_{K_0} = \eta_{K_0} - \frac{M_c^2 - \eta_{K_0}^2}{3} \quad (2.17)$$

Under plastic loading along the normally consolidated K_0 stress path, the variable α evolves toward an equilibrium value. The requirement that this equilibrium matches the expected initial anisotropy uniquely determines ω_d :

$$\omega_d = \frac{3(4M_c^2 - 4\eta_{K_0}^2 - 3\eta_{K_0})}{8(\eta_{K_0}^2 + 2\eta_{K_0} - M_c^2)} \quad (2.18)$$

An initial estimate of ω can be obtained by assuming that anisotropy is effectively eliminated when $\alpha/\alpha_0 = 10$, giving [5]:

$$\omega = \frac{1}{\lambda^*} \ln \left(\frac{10M^2 - 2\alpha\omega_d}{M^2 - 2\alpha\omega_d} \right) \quad (2.19)$$

This estimate depends only on λ^* the modified compression index, M and ω_d and should subsequently be defined through laboratory testing.

2.3.2 Destructuration Parameters

Bonding is represented by the initial bonding variable χ_0 , estimated from soil sensitivity:

$$\chi_0 = S_t - 1 \quad (2.20)$$

The sensitivity S_t relates the apparent preconsolidation stress of intact and reconstituted samples. As plastic strains accumulate, bonding degrades and the stress

path of the intact clay converges toward that of the reconstituted clay, as illustrated in Fig. 2.3.

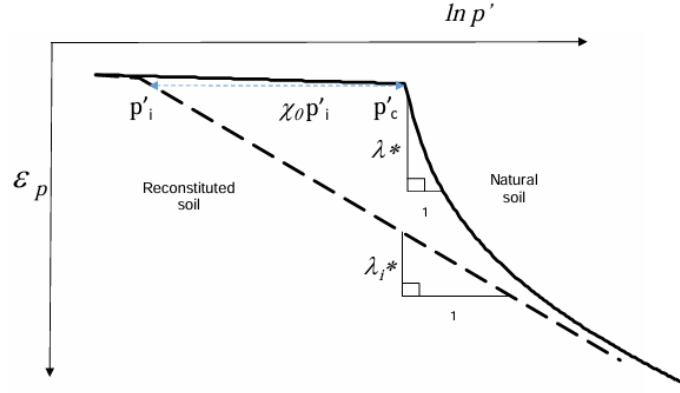


Figure 2.3: Relation between intact and reconstituted clay [15].

The determination of a and b is inherently uncertain and ideally requires dedicated destructuration tests. In the absence of such data, b typically ranges from 0.2 to 0.4, while a is bounded by theoretical upper and lower limits [7]:

$$a \leq \frac{1 + \chi_0}{\chi_0 (\lambda_i^* - \kappa^* + 1) \left[1 + 2b \frac{\alpha_0}{M_c^2} \right]} \quad (2.21)$$

$$a \geq \frac{\ln 2}{\left[\ln(2 + 2\chi_0) - \ln(1 + \frac{\chi_0}{2}) \right] (1 + b)(\lambda_i^* - \kappa^*)} \quad (2.22)$$

Table 2.1 summarises the model parameters together with their recommended methods of determination, and provides a reference for the parameter selection described in Chapter 3.

Table 2.1: Summary of parameters and their method of determination [16].

Parameter	Parameter determination
e_0	Oedometer test
POP	Oedometer / CAUD triaxial test
OCR	Oedometer / CAUD triaxial test
α	Eq. (2.17), 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)
ν'	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. (2.19)
ω_d	Eq. (2.18), depends on K_0^{NC}
a	To be optimized with curve fitting triaxial compression test, Eq. (2.21) and Eq. (2.22))
b	To be optimized with curve fitting oedometer test [7]
K_0^{NC}	Triaxial test

The determination and calibration of these parameters for the study are described in Chapter 3.

2.4 Monitoring Techniques in Sävenäs

This project utilised two main monitoring techniques, InSAR and bellow hoses. InSAR provides spatially extensive measurements of surface deformation along the satellite line of sight and bellows offer detailed observations of subsurface displacement. Together, these methods illustrate how loading influences deformation behaviour over time, both at the surface and at depth. In this section, a brief description of both techniques is provided.

2.4.1 InSAR

Spaceborne InSAR technology has been widely used in ground deformation and monitoring recent years. Satellites such as Sentinel-1, used in this study emits radar waves with different wavelengths hitting surfaces on earth. This scatters in all directions and the proportion of the energy hitting the satellite once again is called backscatter. Since the launch of Sentinel-1A in 2014 and Sentinel-1B in 2016 with the addition of Sentinel-1C in 2024 the mission has been frequently used in ground deformation monitoring including landslides, subsidence and tectonic motion. The data is especially valuable for the detection of very small ground displacements,

often just millimeters by comparing the phase differences between two or more radar images taken from the same position in space at different times.

The first applications of SAR images was used back in the 1980s with the use of differential InSAR, analyzing pairs of SAR images [17]. These measurements are commonly obtained from both ascending (northward) and descending (southward) satellite passes, allowing improved interpretation of deformation and wider capture area, see Fig. 2.4. However, there are limitations connected with various levels of coherence. Coherence is mainly affected by atmospheric and spatial variations over time [18]. Tropospheric delay from humidity and temperature changes slows the radar signal [19]. Coherence is further reduced by temporal decorrelation from surface changes vegetation, snow and spatial decorrelation due to differences in radar viewing geometry. A significant number of points are filtered out during processing because the analysis relies on coherent scatterers. The quality of these points is often assessed using statistical measures such as the coefficient of determination (R^2), where low values indicate large variability.

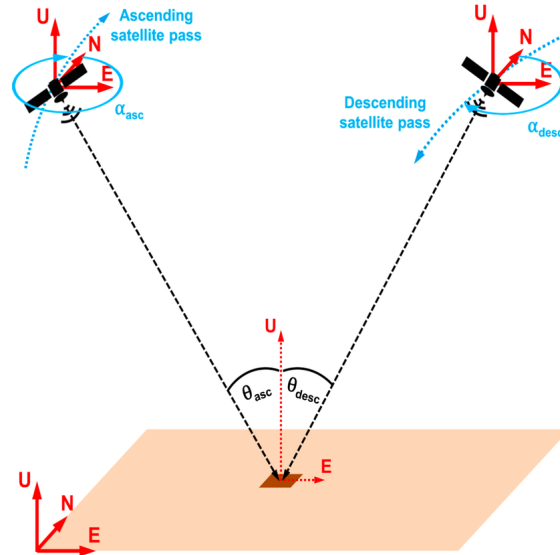


Figure 2.4: Schematic view of InSAR viewing geometry for line of sight (LOS) measurements on ascending and descending satellite passes [20].

2.4.2 Bellows

In addition to remote sensing methods such as InSAR, older in situ techniques are commonly used to measure vertical soil deformation with high resolution. The bellow hose settlement gauge, has been used to monitor soil movements since the development by Wager in 1972, [21]. The system consists of an axially flexible, spirally reinforced polyethylene tube equipped with metal rings at predetermined depths that act as measurement points. Installed in a borehole and grouted to the surrounding soil, the bellow-hose deforms together with the ground, allowing vertical displacements to be recorded at multiple depths [22]. Such instrument have been used in Sävenäs to quantify soil heave and settlement caused by loads from train

traffic with gradual deformation and compaction of the ground.

2.5 Repeated Loading in Soils

In general, soil behaviour under repeated loading may be classified as dynamic, cyclic, or quasi-static depending on the loading frequency and the significance of inertial effects [2]. For dynamic loading, such as seismic events or machine vibrations, frequencies are sufficiently high that inertial effects cannot be neglected. In this case, the full momentum equation must be solved, including the acceleration term,

$$\nabla \cdot \boldsymbol{\sigma} + \rho \mathbf{b} = \rho \ddot{\mathbf{u}} \quad (2.23)$$

where $\boldsymbol{\sigma}$ is the total stress tensor, ρ the density, $\mathbf{b}$ the body force vector and $\ddot{\mathbf{u}}$ the acceleration vector. Cyclic loading covers a broad range of frequencies and may or may not require the acceleration term, depending on the rate of loading. For cyclic loading at low frequencies, such as the intermittent train loading, inertial effects become negligible and the response at any instant can be treated as quasi-static. The acceleration term then vanishes and the governing equation reduces to the static equilibrium condition,

$$\nabla \cdot \boldsymbol{\sigma} + \rho \mathbf{b} = \mathbf{0} \quad (2.24)$$

which forms the basis for most geotechnical analyses of long-term behaviour and is the equilibrium condition solved in all consolidation and plastic analyses in PLAXIS, where inertial effects are excluded by formulation [16]. At Sävenäs, trains operate at low velocities and remain stationary for extended periods, meaning that loading frequencies are low and the quasi-static assumption is considered appropriate.

The choice of loading representation is also closely linked to drainage conditions in the soil. Most researchers assume that pore pressure cannot be effectively discharged from soft clay during cyclic loading due to its low permeability, and investigations into cyclic behaviour therefore often involve testing under undrained conditions. Higher loading frequencies have also been shown to generally lead to lower rates of strain accumulation under undrained conditions [23]. However, during prolonged cyclic loading, pore water is eventually able to discharge due to the extended duration of the loading. Sakai et al. [24] were among the first to characterise this behaviour as partially drained, noting that pore pressure in saturated clay is generated and discharged simultaneously. Subsequent studies have confirmed that drainage conditions play an important role in long-term cyclic response [25], [26], with partially drained conditions generally providing a more realistic representation of field behaviour where loads are applied intermittently over extended periods, as is the case at Sävenäs.

Creep-SCLAY1S is formulated for static loading conditions where creep and consol-

idation dominate and was not developed to explicitly represent strain accumulation caused by cyclic loading. The extended formulation Creep-SCLAY1Sc was developed to address this by introducing an additional viscoplastic term representing permanent deformations from repeated loading cycles [27]. However, the differences between Creep-SCLAY1S and Creep-SCLAY1Sc become significant primarily when loading frequencies approach truly cyclic conditions, where explicit accumulation of plastic strains governs the response. For the low-frequency loading at a marshalling yard, the additional cyclic term is expected to have a limited influence and the standard Creep-SCLAY1S formulation is therefore considered sufficient.

In this study, the operational loading is represented using a quasi-static approach, where train loads are activated and deactivated over time according to track occupancy patterns. Given that drainage conditions at Sävenäs are expected to be partially drained due to the low-frequency nature of the loading, this approach is considered a physically reasonable representation of the long-term deformation response.

3

Methods

This chapter describes the methodology used to develop and calibrate the numerical model used for settlement analysis at Sävenäs marshalling yard. The modelling was based on geotechnical investigations, monitoring data and operational information provided by WSP and Trafikverket, together with parameter relationships from previous literature.

The workflow consisted of site characterization, parameter determination, laboratory verification, field calibration against bellows and InSAR measurements and implementation of the final two-dimensional model used to evaluate quasi-static loading.

3.1 Site Conditions

The Sävenäs marshalling yard is located in an area with varying ground levels, with higher terrain surrounding the site. The bedrock surface generally slopes from south to north. Despite this variation, the yard itself is relatively level, with ground elevations typically ranging from approximately +10 m to +11 m, reaching about +13 m at the highest point and +9.5 m at the lowest point.

The upper soil layer consists of fill material of varying thickness, originating from historical construction and modification of the yard. The spatial distribution of fill thickness is shown in Fig. 3.1, with thicknesses typically ranging from approximately 0.2 m to 4 m across the site. The fill is underlain by soft clay, which governs the long-term settlement behaviour.

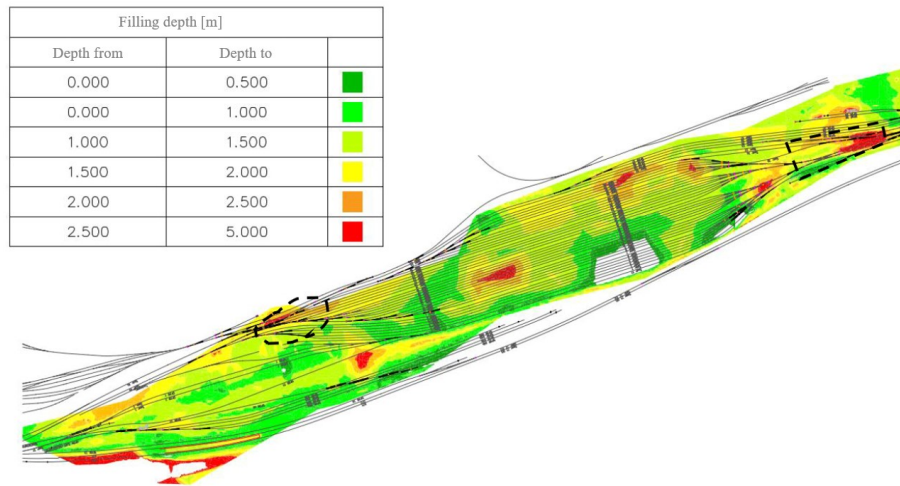


Figure 3.1: Interpretation of filling thickness. From Trafikverket, [28].

Beneath the fill, the thickness of the clay layer varies across the site, as illustrated in Fig. 3.2. The clay generally ranges from approximately 5 m to 70 m in thickness. In some locations, the clay rests directly on bedrock, while in others it overlies layers of friction soil or silt. A shallow dry crust, typically around 1 m thick, is present at the top of the clay layer. These variations contribute to non-uniform ground conditions and settlement patterns.

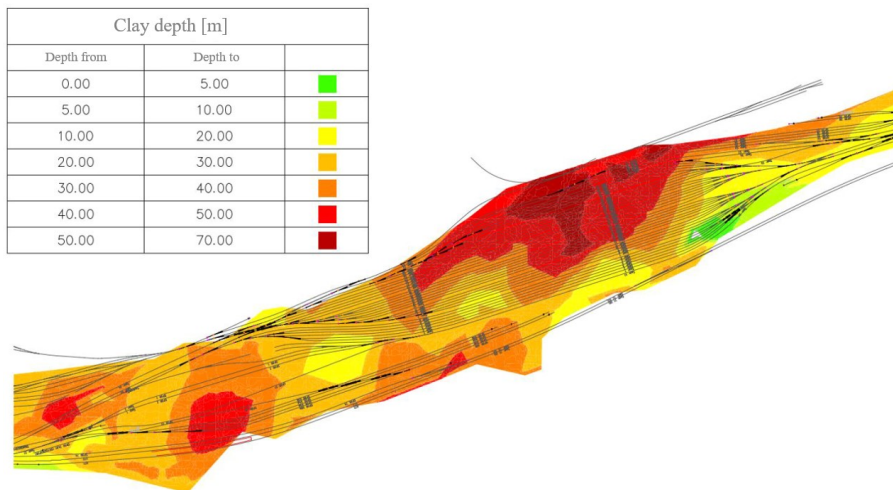


Figure 3.2: Interpretation of clay thickness. From Trafikverket, [28].

Groundwater conditions are characterised by a shallow groundwater level, typically located approximately 0.8–1.0 m below ground surface. Available pore pressure measurements indicate that the clay is close to hydrostatic conditions below this level, however some measurements have indicated artesian pressure.

Given the large extent of the yard, the analysis is limited to a representative area

within the R-group, see Fig. 1.1. The soil conditions are based on borehole investigations distributed across the site, as shown in Fig. 3.3. These boreholes provide the basis for defining both the soil stratigraphy and the material properties.

Samples obtained from the boreholes have been tested in the laboratory. Incremental loading (IL) oedometer tests were available from boreholes 25W046, 25W067, 25W070, 25W075, 25W085 and 25W097, while triaxial tests were available from two samples, 25W009 and 25W119, at depths of approximately 6 m and 10 m located outside the R-group.



Figure 3.3: Placement of boreholes in the R-group.

Typical properties of the clay deposit, derived from these investigations, are shown in Fig. 3.4. The clay exhibits high natural water contents, typically in the range of 60–100%, and moderate to high void ratios. The bulk density reflects the soft nature of the material, while permeability is low, indicating limited drainage capacity. The clay is generally moderately sensitive, with local occurrences of higher sensitivity.

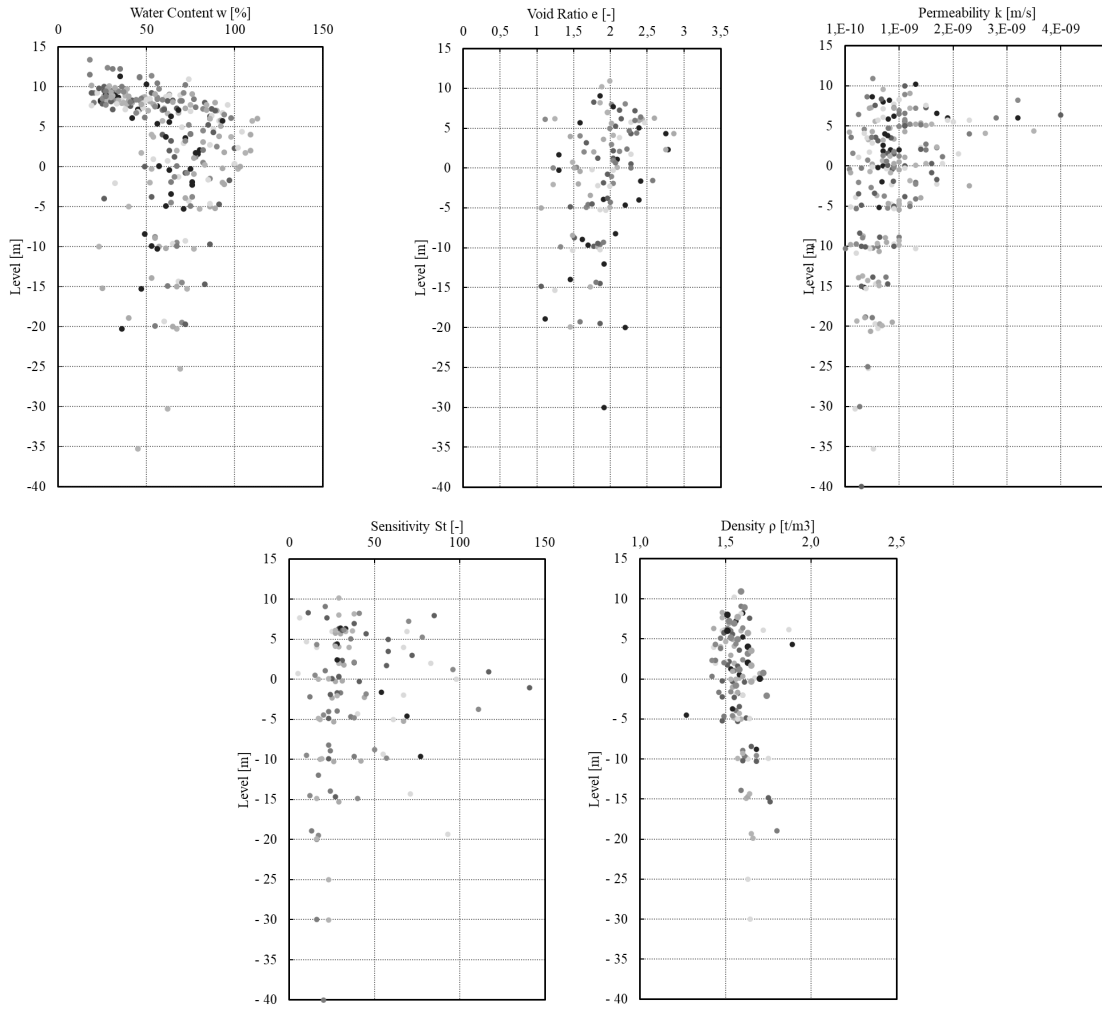


Figure 3.4: Typical characteristics of the soil deposit from Sävenäs.

3.2 Parameter Determination

Initial model parameters were established using available laboratory tests, in situ investigations and theoretical relationships presented in Chapter 2.3. Parameters that could be directly interpreted from laboratory testing were derived first, while parameters with limited supporting data were estimated using theoretical relationships and subsequently compared with parameter ranges reported in literature for similar Scandinavian clays. The resulting parameter set was used as initial input for the soil test simulations presented in Section 3.3 and was later refined through calibration against field measurements.

IL oedometer tests formed the primary basis for deriving the compressibility parameters used in the model. Results from borehole 25W097 were selected as a representative example and are presented in Fig. 3.5. The test data was plotted in $\varepsilon - \ln p'$ space, where λ_i^* , κ^* , σ'_c and $E_{\text{oed}}^{\text{ref}}$ were interpreted. Although the interpretation is performed in $\varepsilon - \ln p'$ space according to the Creep-SCLAY1S formulation, the interpreted isotropic preconsolidation pressure is back-calculated and presented

here as the vertical preconsolidation pressure, σ'_c . The same procedure was repeated for the remaining relevant boreholes within the study area, where the interpreted values were used as initial estimates for the corresponding soil layers and later evaluated through the soil matching procedure in PLAXIS. A constant $\nu' = 0,2$ and a $K_0 = 0,6$ was adopted for all clay layers.

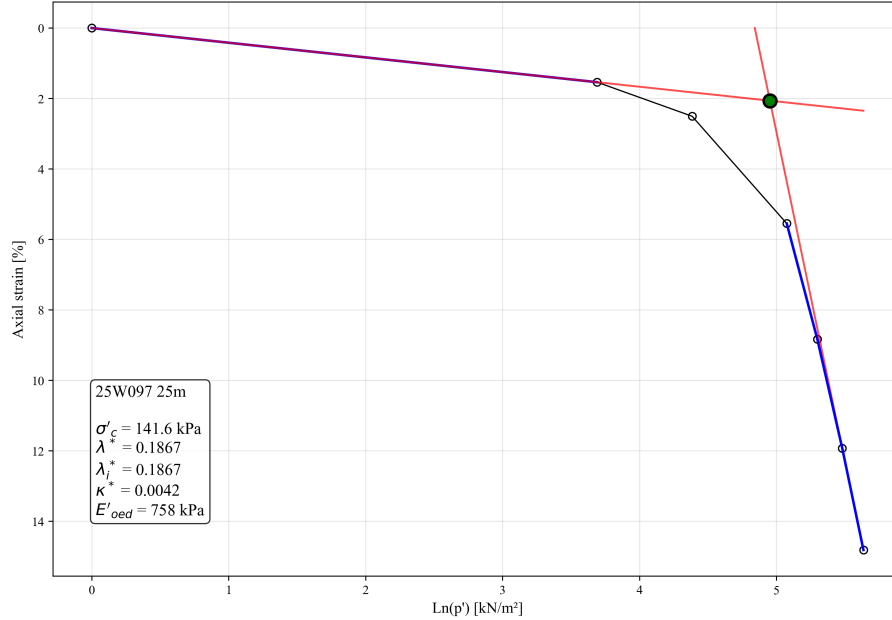


Figure 3.5: Interpretation of stiffness parameters for 25W097 at depth 25 meter.

The same tests were also used to derive parameters governing creep behaviour. Accumulated strain during each loading step was plotted against time on a logarithmic scale, shown in Fig. 3.6, and the modified creep index μ^* was interpreted from the slope of the final portion of the curve using a reference time τ of one day. The intrinsic creep index μ_i^* was then selected from the final loading stage of each test, corresponding to the highest applied stress level.

The derived μ_i^* values ranged between 0.003 and 0.01. Several tests produced relatively high values due to limited loading into the virgin compression range, and no reconstituted samples were available to verify the intrinsic creep response. Since the directly derived values were considered uncertain, they were compared with reported values from nearby clay deposits and previous studies on Gothenburg clay, especially Utby [15]. Based on this comparison, representative values within a more realistic range were selected as initial inputs for the subsequent soil test simulations. The full set of derived creep parameters is presented in Appendix B.2.

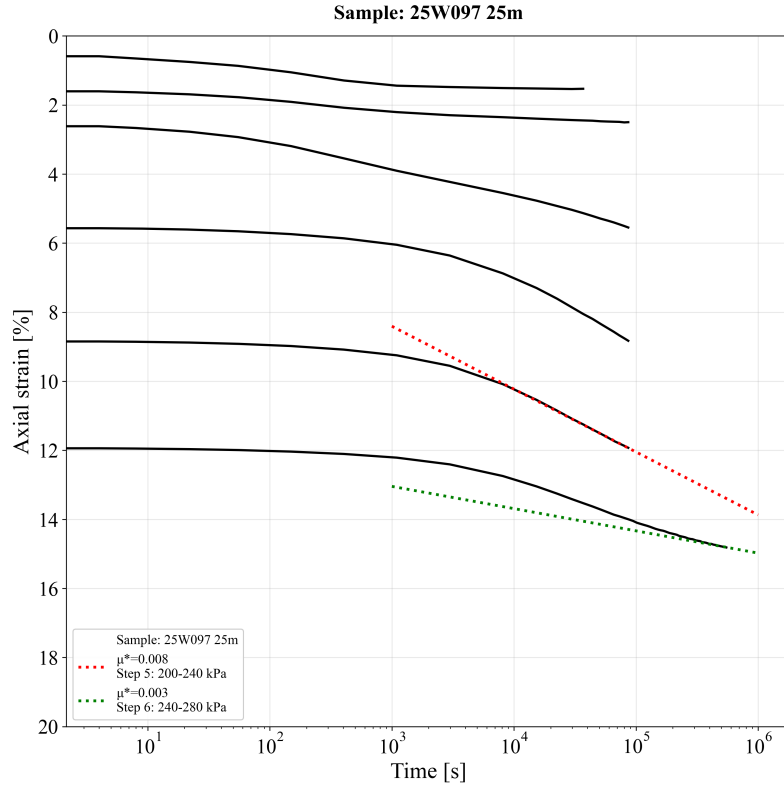


Figure 3.6: Determination of intrinsic modified creep index μ_i^* from oedometer test at 25W097, depth 25 meter.

Shear strength parameters were derived using consolidated undrained triaxial tests available from the site investigation. Due to no available triaxial tests within the R-group, results from boreholes in the I-group were also included. The critical state line in compression, M_c was established by plotting the mean effective stress against the deviatoric stress. The M_c value was defined as the slope of a straight line drawn from the origin to the corresponding laboratory data points, in accordance with [29]. The same procedure was applied to determine M_e by analyzing the extension triaxial test, see Fig. 3.7. The available compression and extension tests resulted in similar values, and both parameters were therefore assigned a value of 1.2.

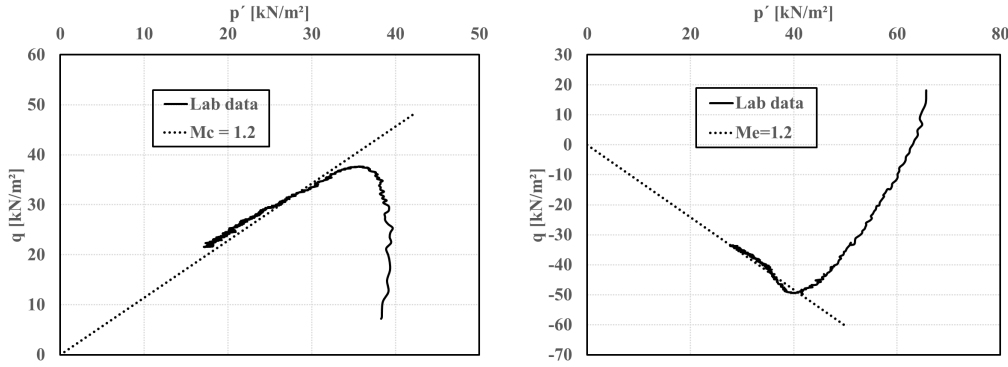


Figure 3.7: Estimation of shear strength parameters at 6.2 meter from 25W009 and 25W119 respectively.

Parameters governing anisotropy evolution could not be directly derived from the available laboratory testing and were therefore initially estimated using the relationships presented in Chapter 2. The parameter n_{K_0} was first estimated using Eq. (2.16), resulting in a value of 0.75. This value was then used to calculate α_{K_0} with Eq. (2.17), resulting in $\alpha_{K_0} = 0.46$. The rotational hardening parameters ω and ω_d was calculated by using Eq. (2.18) and (2.19), resulting in $\omega_d = 0.76$ and $\omega = 10.72$. Although these values were considered reasonable, they remained uncertain due to the limited availability of data describing anisotropic behaviour.

The destructuration parameters a and b were initially estimated using the relationships presented in Section 2.3.2, resulting in values of 4.76 and 0.3, respectively. The calculations were partly dependent on previously derived oedometer parameters, which introduced uncertainty as several oedometer tests were considered insufficiently loaded. The initial estimates were therefore compared with guideline ranges reported for Scandinavian clays and adjusted to final values of $a = 9$ and $b = 0.3$ [4].

Given the high variability in soil sensitivity across the area, particularly within the more sensitive clay layers, the theoretical framework presented in Section 2.3.2 was considered impractical to apply on a layer-specific basis. Consequently, a uniform value of $\chi_0 = 20$ was adopted for all modeled clay layers.

Hydraulic conductivity parameters were derived using available laboratory data. Borehole 25W097 was again used as a representative example, where an initial hydraulic conductivity of $k = 4.1 \times 10^{-5}$ m/day and a permeability change parameter of β_k were obtained from the Constant Rate of Strain (CRS) test, in accordance with [30]. The parameter c_k defines the logarithmic relationship between void ratio and hydraulic conductivity used in the PLAXIS formulation and controls the reduction in permeability as the soil compresses during consolidation [16]. The assigned value for this depth at this particular borehole was $c_k = 0.31$. The same procedure was repeated for all relevant boreholes within the study area.

$$c_k = \frac{e_0}{\beta_k} \quad (3.1)$$

The resulting parameter set from this stage was used as input for the soil tests presented in Section 3.3, where the constitutive behaviour was evaluated before calibration against field measurements was performed.

3.3 Soil Test

PLAXIS includes built in soil test simulations that are used to assist in parameter selection and verification. By replicating laboratory oedometer test and triaxial compression and extension tests within the software, the stress strain response of the selected constitutive model can be evaluated. This allowed for an initial check of whether the prior derived parameters produce realistic behaviour in terms of stiffness, yielding and volumetric response. The soil test simulations were used iteratively to adjust parameters within physically reasonable ranges until the model response is consistent with measured laboratory data. This approach ensured that the selected parameters are consistent and compatible with the chosen constitutive model prior calibration against field measurements.

Initially, for the oedoemetric soil match a cell pressure of 25 kPa is assigned for all samples and then increased corresponding to values from each test. For oedometer IL simulation, the general test function is used and load input is given in increments with same magnitude and time interval as performed in laboratory testing as demonstrated in Table 3.1.

Table 3.1: Simulation of IL test at 25W097, depth 25 meter.

Duration for each step (days)	Load increment (kPa)	Total stress (kPa)
0.4272	40	40
1.0	40	80
1.0	80	160
1.0	40	200
1.0	40	240
6.4	40	280

The load input of the sample and pre overburden pressure (POP) is adjusted since initial stress condition of sample extracted from site is not representing in situ stress condition due to disturbances from sampling. It is specific for each sample match. In Fig. 3.8, particular emphasis was placed on achieving consistent slopes to validate κ^* and λ_i^* . The complete results of matching oedometer tests can be seen in Appendix B.3.

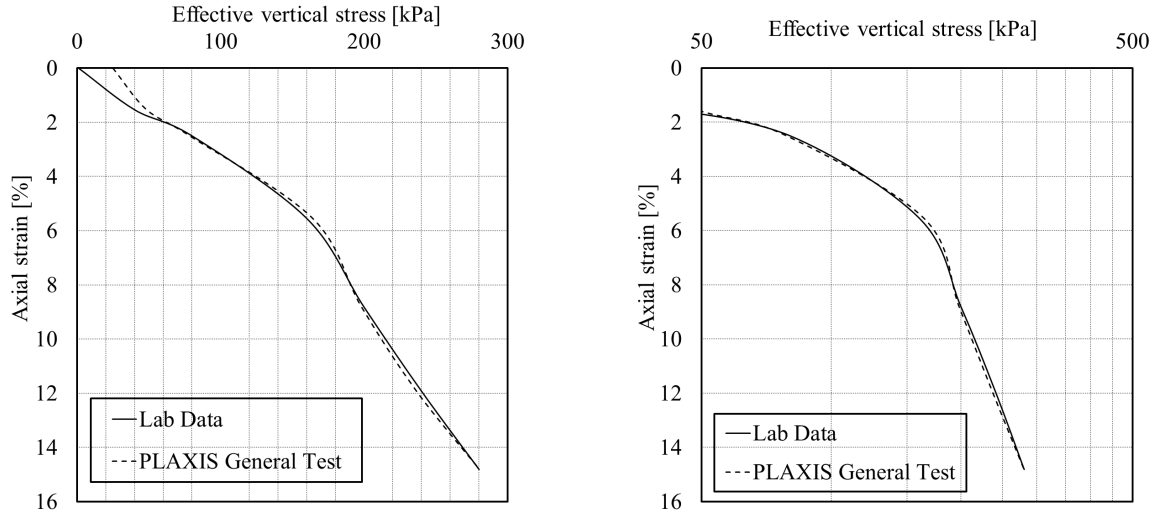


Figure 3.8: Illustration of the soil matching procedure in PLAXIS. From 25W097, 25 meter depth.

The triaxial simulations were performed to evaluate the shear strength in the soil. The input needed for simulating triaxial tests was the K_0 -value representing anisotropic behaviour and was set corresponding to each test. Further, the test was set to either compression or extension to evaluate M_c and M_e and an initial cell pressure was also set reproducing the specific laboratory test.

Once stiffness related parameters are established, the anisotropy related parameters were evaluated by matching the triaxial stress paths. The initial inclination of the yield surface and the rotational hardening parameters are adjusted so that the simulated stress strain curves align with the measured ones. As proposed by [31], undrained triaxial extension test data are available and the soil constant ω was further evaluated by means of curve fitting in PLAXIS. The initial estimates were thereafter compared with guideline ranges commonly reported for Scandinavian clays and adjusted to values considered more representative of the deposit, resulting in final adopted values of $n_{K_0} = 0.6$, $\alpha_{K_0} = 0.5$, $\omega_d = 0.8$ and $\omega = 30$.

Once satisfactory agreement was achieved at the element level, the parameters were adopted for subsequent construction of the finite element model.

3.4 Calibration against Bellows and InSAR

Following the laboratory validation presented in Section 3.3, field-scale calibration was performed to ensure that the selected parameter set reproduced the observed long-term settlement behaviour at Sävenäs. While the laboratory simulations validated that the selected parameters produced a reasonable constitutive response under controlled conditions, uncertainties still remained regarding long-term deformation at the site. Calibration was therefore performed using both bellows measurements and InSAR observations.

For each bellow hose location included in the study, the nearest borehole was identified and used to represent the local soil conditions. A total of ten bellows hoses were available within Sävenäs, as shown in Fig. 3.9. However, only bellows 1–6 were included in the calibration process, as these were located within the selected R-group study area. Hereafter, individual bellow hose locations are referred to as BLG followed by their respective number (e.g. BLG 1, BLG 6). The interpreted stratigraphy, fill thickness and material parameters derived in Section 3.2 were assigned to the corresponding bellows location. The full set of input data for the bellow hoses can be found in Appendix B.1.



Figure 3.9: Placement of bellow hose measurements. From Trafikverket, [28].

Simplified calibration models were then constructed in PLAXIS where each model consisted of a vertical soil column where the fill layer and underlying clay profile were represented according to the local borehole conditions. This simplified approach was used to reduce geometric uncertainty and isolate the time-dependent response of the soil profile during calibration.

Historical loading was simplified due to limited documentation regarding the timing and magnitude of additional filling operations after the initial development of the yard. Although multiple filling events likely occurred over time, the chronology of these activities was considered too uncertain to model explicitly. The entire estimated fill load was therefore applied in a single stage representing the initial construction period in 1914, followed by consolidation until 2014 when bellows measurements became available. The phase sequence used in PLAXIS during calibration

is presented in Table 3.2.

Table 3.2: Overview of analysed calculation phases based on model stages.

Phase	Calculation type	Date
Initial phase	Gravity Loading	Start
Plastic nil-step	Consolidation	–
Consolidation 1800 - 1914	Consolidation	Year 1800
Embankment fill	Consolidation	Year 1914
Consolidation to year 2014	Consolidation	Year 2014
Installation of BLG	Consolidation	2014-05-15
BLG measurement	Consolidation	2015-05-25
BLG measurement	Consolidation	2016-10-27
BLG measurement	Consolidation	2017-09-25
BLG measurement	Consolidation	2018-11-24
BLG measurement	Consolidation	2025-08-20
100-year consolidation	Consolidation	+100 years

Note: Phases defined by calendar dates represent monitoring or control points rather than predefined calculation durations.

Following each simulation, a vertical cross-section was defined through the centre of the calibration model in PLAXIS Output. The vertical displacement, u_y was extracted from all nodes intersecting this cross-section, creating a continuous settlement profile throughout the soil column. The simulated profile was then compared with the measured bellows data to evaluate whether the model reproduced the observed settlement distribution with depth.

Once a reasonable agreement between the simulated and measured bellows profiles had been achieved, broader settlement trends were evaluated using InSAR measurements. The measured settlement distribution across the railway yard can be seen in Fig. 3.10. The displacement-time relationship from the calibration models during the monitoring period was analysed to determine settlement rates in mm/year, which were then compared with nearby InSAR observations.

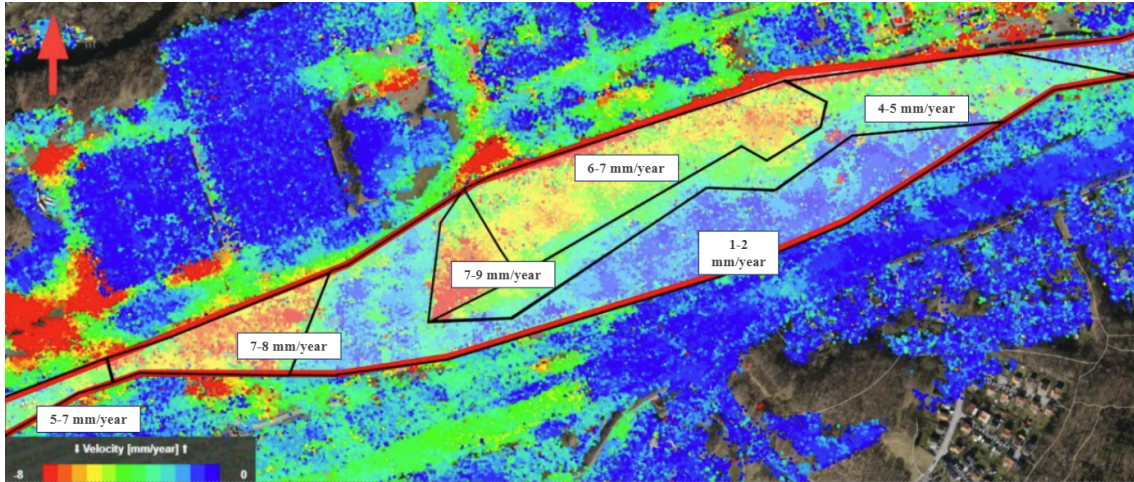


Figure 3.10: Speed of settlement at Sävenäs depot. Credits SkyGeo, [28].

An additional 100-year consolidation phase was included after 2025 to evaluate the long-term creep response after the calibration period. The purpose of this phase was to assess whether the predicted settlement rate gradually approached the lower background settlement rates observed from nearby InSAR points located outside the most heavily loaded parts of the yard. These locations were assumed to be less influenced by historical loading and were therefore used as an indication of the intrinsic long-term creep behaviour of the regional clay deposit.

A limited sensitivity analysis was also performed using one of the calibrated bellows models, see Table 3.3. Hydraulic conductivity, initial void ratio and the hydraulic conductivity change parameter were individually increased and decreased by 20 % while keeping all other parameters constant. The resulting changes in vertical displacement were evaluated to identify whether small variations in parameter selection significantly influenced long-term settlement predictions.

Table 3.3: Sensitivity of u_y for different parameter variations, hydraulic conductivity, void ratio and coefficient of hydraulic conductivity change. Based on data from bellow hose 4.

Description	Parameter	Sensitivity [%]
Hydraulic Conductivity 20%	k [m/day]	7,0%
Hydraulic Conductivity -20%	k [m/day]	- 7,0%
Void ratio 20%	e	2,3%
Void ratio -20%	e	- 2,3%
Hydraulic Conductivity Change 20%	c_k	2,3%
Hydraulic Conductivity Change -20%	c_k	- 4,7%

The calibrated parameter set obtained from this process was subsequently implemented in the final two-dimensional model presented in Section 3.5.

3.5 Final Numerical Model

Following parameter determination and calibration, a final numerical model was developed to analyse long-term settlement behaviour under operational loading conditions. The model incorporates the site stratigraphy, calibrated soil parameters and reconstructed loading history, and forms the basis for the analyses presented in Chapter 4.

3.5.1 Geometry and Input Data

No complete surveyed cross section was available for the analysed area. The geometry was therefore constructed as a representative section based on borehole data, interpreted stratigraphy and general site conditions within the R-group. The model is not intended to reproduce an exact field profile, but to capture the dominant features controlling settlement behaviour.

To evaluate settlement behaviour within different parts of the soil profile, vertical displacement data were extracted from selected nodes within the numerical model. Fig. 3.11 illustrates the interpreted stratigraphy together with the extraction nodes and cross sections used throughout the analyses. The blue nodes represent locations where vertical displacement-time relationships were extracted, while the orange dashed lines indicate the vertical cross sections used for evaluation of settlement development throughout the soil profile.

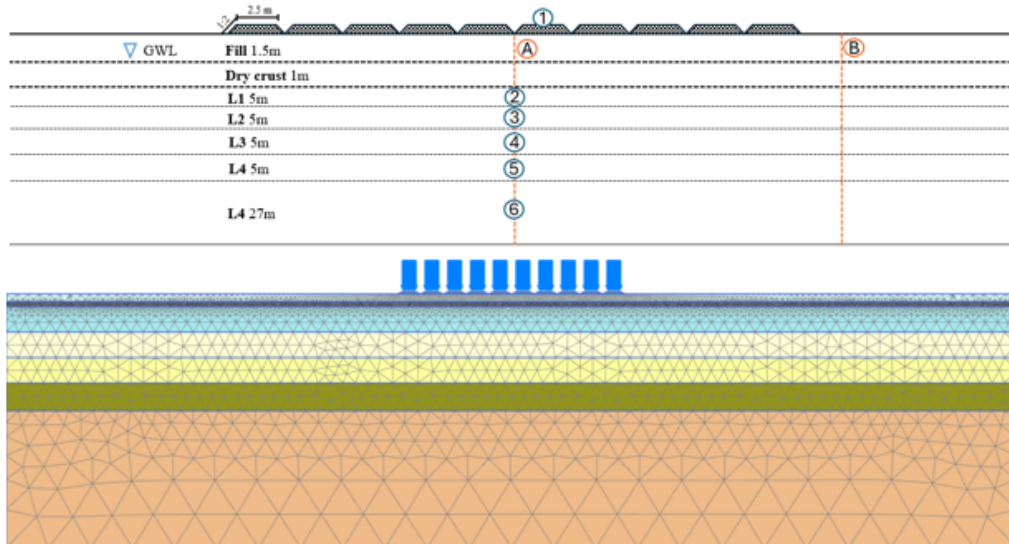


Figure 3.11: Illustration of the schematic cross section with the undeformed mesh within PLAXIS.

Table 3.4: Selected node locations

Node	Coordinates (x, y)	Description
1	(2.25, 10.5)	Top of embankment
2	(0.04, 4.42)	Clay layer 1
3	(0, 0)	Clay layer 2
4	(0, -5)	Clay layer 3
5	(0, -10.25)	Clay layer 4
6	(0, -17.05)	Clay layer 5

The interpreted soil profile consisted of fill, dry crust and five underlying clay layers denoted L1–L5. By comparing displacement development at different depths, settlements occurring within the upper structural layers could be separated from settlements developing within the underlying clay deposit.

The displacement-time relationships extracted from the numerical model were used to determine settlement rates in mm/year for comparison against nearby InSAR observations. Initial interpretation of settlement patterns within the yard was based on InSAR material provided through SkyGeo. However, since direct extraction of displacement time series from individual measurement points was not available within the provided SkyGeo material, time series used for comparison against the numerical analyses were instead extracted using the SGI InSAR service. Both platforms are based on the same satellite measurements from Sentinel-1. However the SGI service provided open access to displacement development at individual measurement points within the study area.

Since the primary focus of the study was long-term settlement behaviour within the underlying clay deposit, simplified constitutive descriptions were adopted for these layers with parameters in Table 3.5. The embankment and dry crust were therefore modelled using the Mohr-Coulomb drained for the embankment and fill, and undrained (A) for the dry crust.

Table 3.5: Adopted material parameters for the upper layers.

Parameter	Embankment	Fill	Dry crust
E' [MPa]	30	15	5
ν'	0.3	0.3	0.3
γ [kN/m ³]	22	18	18
k [m/day]	1×10^{-1}	1×10^{-2}	1×10^{-2}
K_0	0.7	0.7	0.7

The adopted constitutive parameters for the five clay layers L1–L5 are presented in Table 3.6. The parameters were derived through interpretation of laboratory testing, empirical correlations and subsequent calibration against field measurements.

Table 3.6: Parameter values for each clay layer L1–L5.

Type	Parameter	L1	L2	L3	L4	L5
Initial stress state	γ [kN/m ²]	15.5	16	16.5	17	17
	OCR	1.6	1.6	1.6	1.7	1.8
	K_0	0.6	0.6	0.6	0.6	0.6
	e_0	2.7	2.6	2.4	2	1.8
Hydraulic	k [m/day]	0.15E-3	0.12E-3	0.1E-3	0.05E-3	0.05E-3
	c_k	0.55	0.52	0.45	0.4	0.38
Standard	κ^*	0.015	0.0145	0.014	0.013	0.012
	ν'	0.2	0.2	0.2	0.2	0.2
	λ_i^*	0.08	0.085	0.09	0.095	0.1
	M_c	1.2	1.2	1.2	1.2	1.2
	M_e	1.2	1.2	1.2	1.2	1.2
Anisotropic	α_0	0.5	0.5	0.5	0.5	0.5
	ω_0	30	30	30	30	30
	ω_d	0.8	0.8	0.8	0.8	0.8
Destructuration	χ_0	20	20	20	20	20
	a	9	9	9	9	9
	b	0.4	0.4	0.4	0.4	0.4
Viscous	μ_i^*	0.002	0.002	0.002	0.002	0.002
	τ [days]	1	1	1	1	1

3.5.2 Calculation Phases and Quasi-static Loading

The calculation sequence for the final numerical model consisted of two main stages: reconstruction of the historical loading and simulation of quasi-static loading.

The historical development of the site was simulated to establish realistic stress conditions prior to the monitoring period. The model was initialised using gravity loading, followed by consolidation phases representing the natural state of the deposit. The embankment load was introduced in 1914 to represent the initial construction of the yard, after which consolidation was simulated until 2014. This approach reconstructs the present soil state based on long-term loading and consolidation.

Field measurements from bellow hoses are available from 2014 onwards and were used for calibration together with InSAR data, as described in Section 3.4. The quasi-static loading analysis is introduced from 2018 onwards, corresponding to the period where reliable InSAR-derived settlement trends are available for the selected area. The model is therefore first calibrated over the period 2014–2018, after which operational loading is applied.

Operational loading was implemented using a quasi-static loading approach to represent train occupancy within the yard. The loading scheme was defined based on

discussions with Trafikverket personnel, providing representative values for track utilisation and operational patterns. Loads were applied through time-dependent activation and deactivation of static loads in accordance with the specified occupancy levels.

A daily loading cycle was defined based on the obtained occupancy data:

- 08:00–16:00: 30% track occupancy
- 16:00–24:00: 50% track occupancy
- 00:00–08:00: 80% track occupancy

For each interval, the corresponding number of tracks was activated. Load locations were varied between cycles to represent operational variability and avoid unrealistic repetition at fixed positions. The loading sequence and associated calculation phases are summarised in Table 3.4 and were generated using a Python script executed within the PLAXIS environment. The full script can be seen in Appendix C.

In addition to the quasi-static loading case, reference simulations were performed using simplified loading conditions. This included no load, constant full loading and intermediate loading levels with 30 %, 50 % and 80 % of the full load as outlined in Table 3.7. The purpose was to evaluate the influence of load representation on settlement behaviour.

Table 3.7: Summary of PLAXIS calculation phases with simplified representation of intermediate time steps.

Phase	Calculation type	Date
Initial phase	Gravity loading	1814
Plastic nil-step	Consolidation	–
Consolidation to 1914	Consolidation	1914
Filling and operation start	Consolidation	1914
Consolidation to 2018	Consolidation	2018
Loading cases (GHOST / COMPARISON)	Consolidation	2022
Day 1 phases (3×8 h steps, varying track activation)	Consolidation	2018
$\vdots$	$\vdots$	$\vdots$
final steps	Consolidation	2022

3.5.3 Mesh Configuration

The finite element mesh was generated to provide an accurate representation of stress and pore pressure development while maintaining a reasonable computational cost. A model length of 200 m was adopted to minimise boundary effects, with lateral boundaries placed 100 m from the centre of the area of interest. The model depth was extended vertically until no noticeable influence from the bottom boundary was observed in the analysed domain. Insufficient depth was found to cause artificial pore pressure generation and upward displacement near the boundary.

During the mesh generation process, particular attention was given to ensuring an adequate resolution across all soil layers. As a minimum requirement, each layer was discretized using at least two elements in the vertical direction. This criterion was adopted to ensure that the mechanical response and pore pressure development within each layer could be captured with sufficient accuracy. This resulted in a relatively fine mesh in the upper layers, where stress and pore pressure variations are most pronounced, and a coarser mesh at greater depth while still satisfying the minimum discretization requirement.

The final mesh consists of 8581 6-noded elements and 17363 nodes. The adopted mesh is considered sufficiently refined to capture the stress and pore pressure response while maintaining computational efficiency.

4

Results

This chapter presents the results from the numerical analyses. First, the simulated laboratory tests are compared with oedometer and triaxial laboratory data to evaluate how well the selected parameter set reproduces observed soil behaviour. The calibrated model is then compared with field measurements from bellow hoses and InSAR observations. Finally, the influence of parameter variations, numerical limitations and quasi-static loading on long-term settlement behaviour is presented.

4.1 Simulation of Laboratory Results

The simulated laboratory tests were used to evaluate how well the selected parameter set reproduced the observed compression, creep and shear behaviour of the clay. The results from the oedometer and triaxial simulations are presented in the following sections.

4.1.1 Oedometer IL Test

The results of the IL simulations are presented in Fig. 4.1 and compared with the corresponding laboratory data. The general shape of the compression curves reflects the behaviour observed in the laboratory tests, as the parameters λ_i^* and κ^* were adjusted to reproduce the response.

For the oedometer samples, simulations with general test initiated with no set initial stress level, exhibited deviations within the normally consolidated region. However, when the simulations were initiated with an initial stress of 25 kPa and then increased to the first loading step, a more realistic behaviour was obtained, showing close agreement with the laboratory results and values on λ_i , κ^* with close resemblance to studied literature.

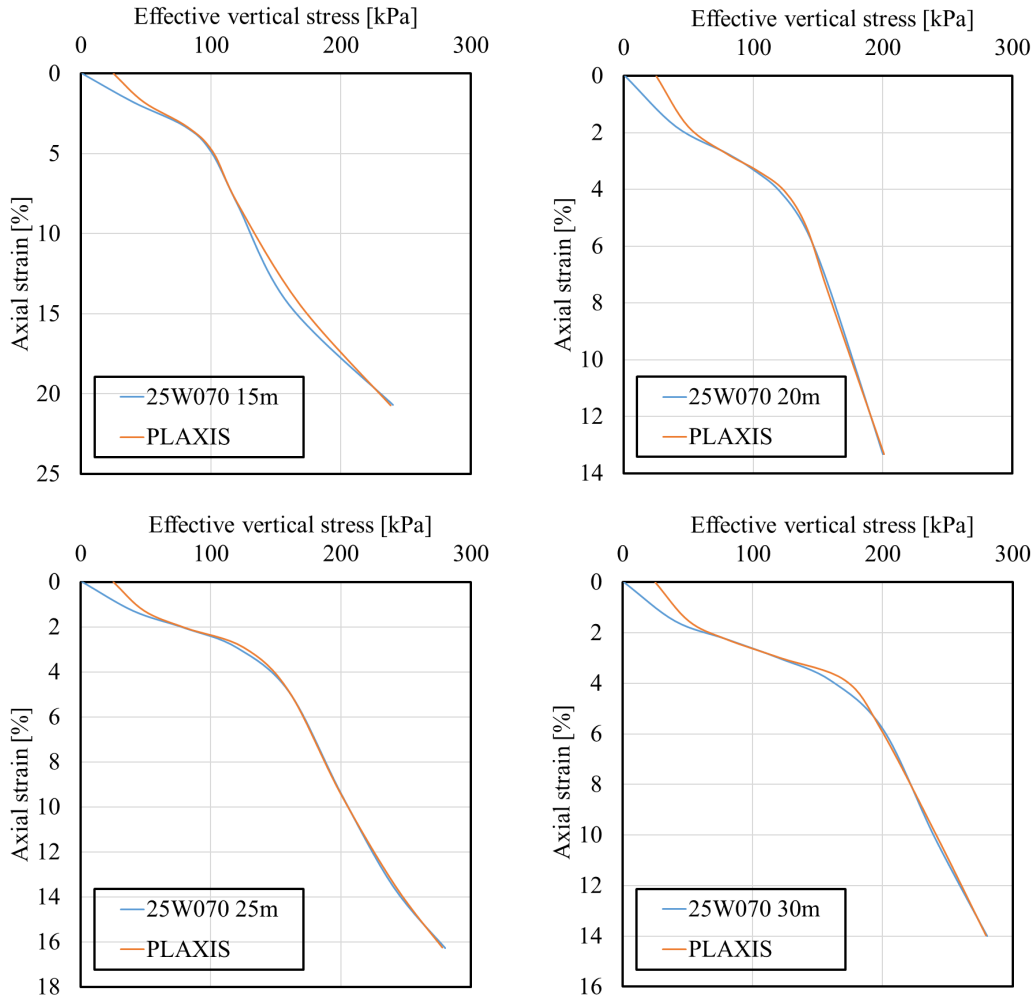


Figure 4.1: IL oedometer test from borehole 25W070, versus simulated general test in PLAXIS.

The index μ_i^* was initially derivated from the IL oedometer laboratory samples from the final points in the time versus axial strain curve. The inclination of this section is regarded as the intrinsic creep deformation. However, the values obtained initially were regarded as quite high due to insufficient time and loading. Hence, μ_i^* was further adjusted in this part. By applying a load step equal to each lab set and then letting it creep, the agreement between lab and simulations can be seen in Fig. 4.2. A decrease to 0,002 was used to match PLAXIS simulations to laboratory data which is approximately a change of 50 %. This value have similarities with two other projects, [11], [15] executed in Gothenburg area with values of 0.00142 and 0.00125 respectively. The inclination of the final points is reproduced with good accuracy and it concluded to be reasonable. Further validation on the creep is executed in chapter 4.2 by linking simulations to observed behaviour in bellow hoses and surface settlements from InSAR.

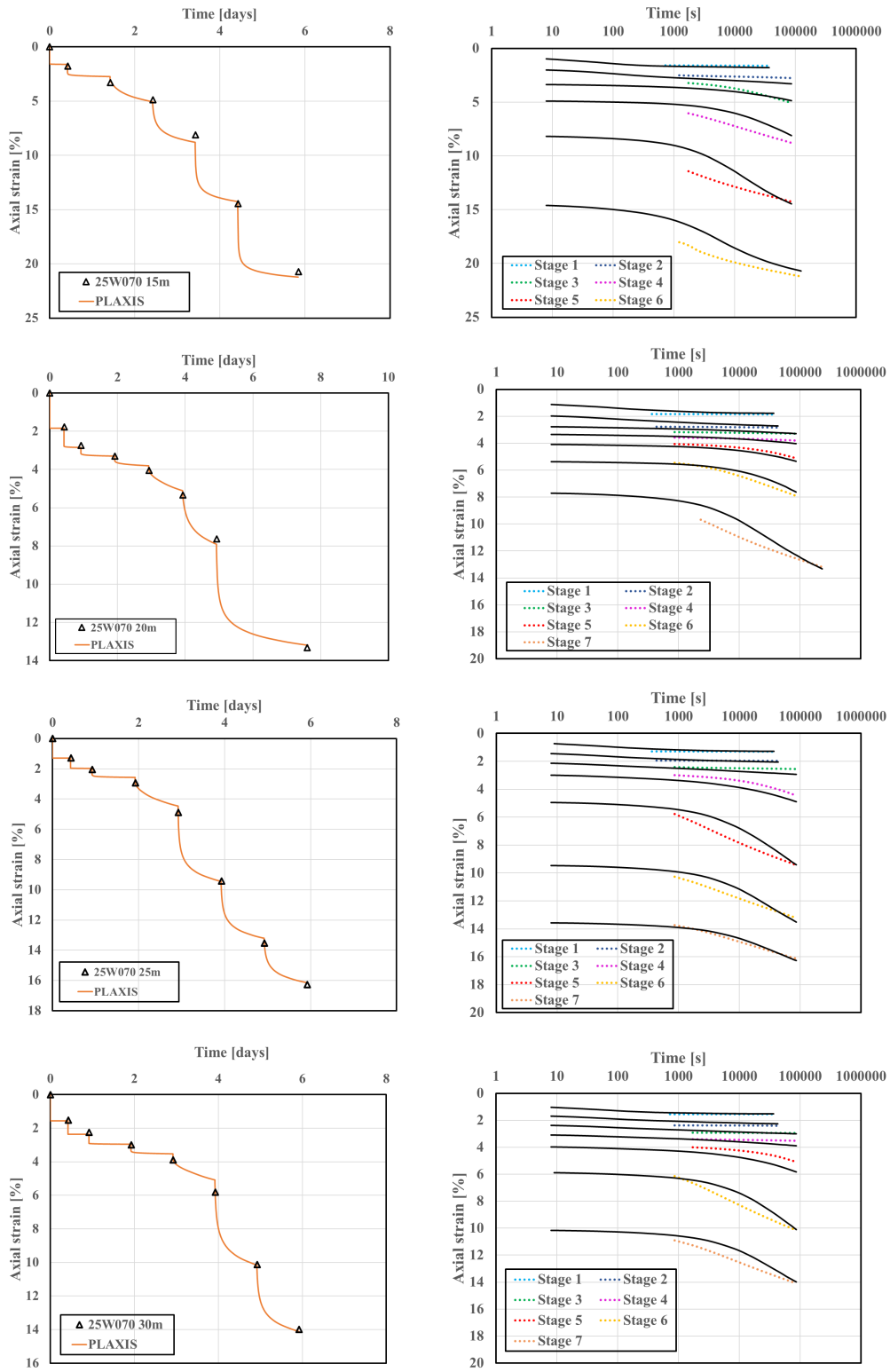


Figure 4.2: IL oedometer test from borehole 25W070, versus simulated general test in PLAXIS.

4.1.2 Triaxial Test

The triaxial test simulations show overall good agreement with the experimental results for both compression and extension tests at depths of 6.2 m and 10.2 m. The numerical model captures the general stress strain response as well as the peak and post peak behaviour with reasonable accuracy.

The results from the triaxial compression are presented in Fig. 4.3. The SoilTest reproduces the initial elastic response and the overall stress path in $p' - q$ space with good agreement to the laboratory data, indicating that the selected parameters provide a good representation in the shear behaviour.

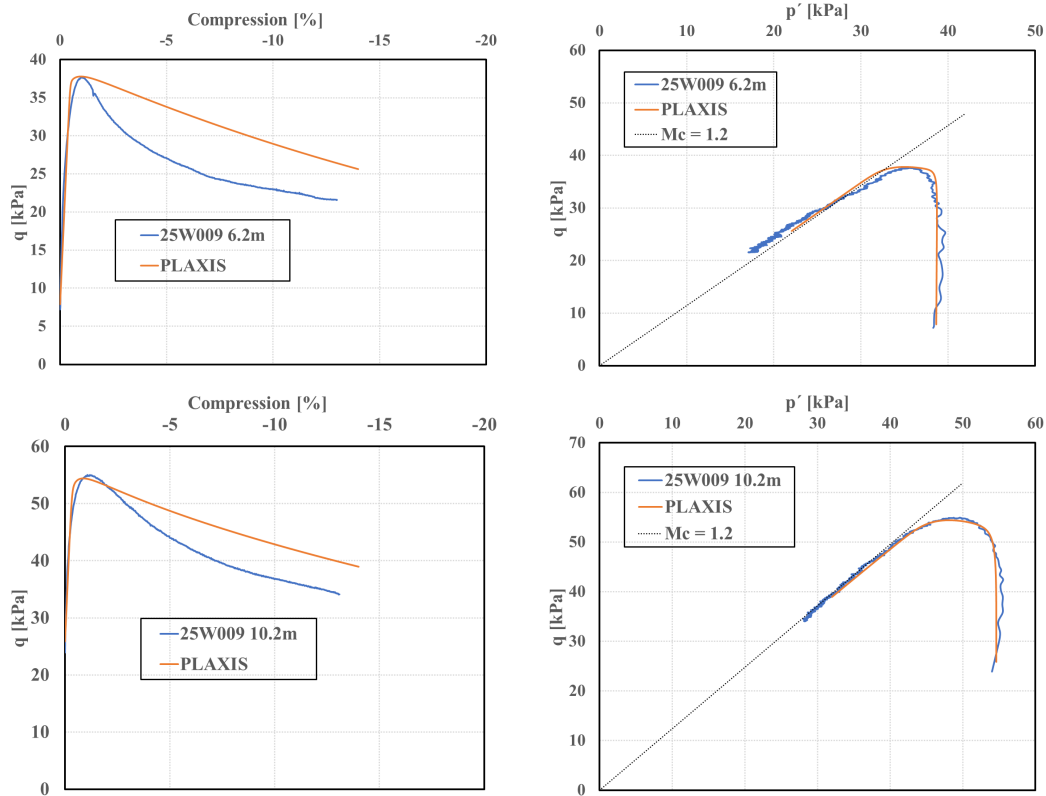


Figure 4.3: Undrained triaxial active test from borehole 25W009, versus simulated triaxial test in PLAXIS.

The corresponding results from the extension tests can be seen in Fig. 4.4. Here, similar agreement as in the compression is observed with the SoilTest model capturing the stress path and overall response under extension well.

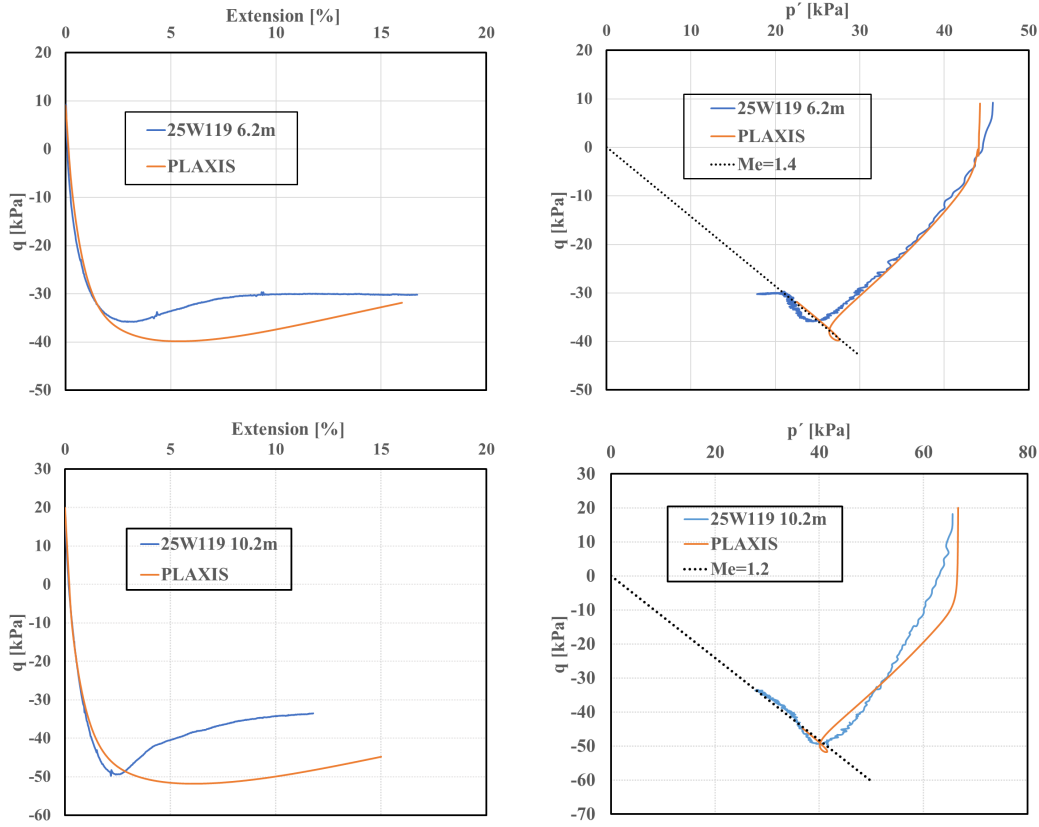


Figure 4.4: Undrained triaxial passive test from borehole 25W119, versus simulated triaxial test in PLAXIS.

A slight deviation can however be observed in the plastic region for both loading paths, where the numerical response exhibits a somewhat softer stiffness compared to the laboratory data. The plastic part does not plane out which may lead to larger deformations in the long run. This indicates that the constitutive model overestimates the strain of the material. Such discrepancies may be attributed to simplifications in the constitutive formulation or the inability of the model to fully capture the soil's non-linear stiffness degradation at large strains.

The slightly more pronounced deviation observed at 10.2 m may reflect depth dependent variations in soil structure or heterogeneity that are not fully represented in the parameter calibration.

4.2 Calibration against Bellows and InSAR

Fig. 4.5 presents the comparison between measured and simulated vertical displacements for the available bellow hose locations. The simulated settlement curves represent the long-term development of the deposit following the initial loading caused by filling in 1914, while the available bellow measurements begin around 2014 after a long period of prior consolidation and creep.

4. Results

For most monitoring locations, the model reproduced the overall long-term settlement trends well. The simulated deformation curves generally followed the measured settlement development over time, particularly for BLG 3, BLG 4 and BLG 5, where both settlement magnitude and long-term settlement rates showed good agreement.

Larger deviations were observed for BLG 1 and BLG 6. At these locations, the model captured the general long-term trend but was unable to reproduce certain shorter-term variations observed in the field measurements. These differences likely reflect local geological variability and uncertainties in the historical development of the site.

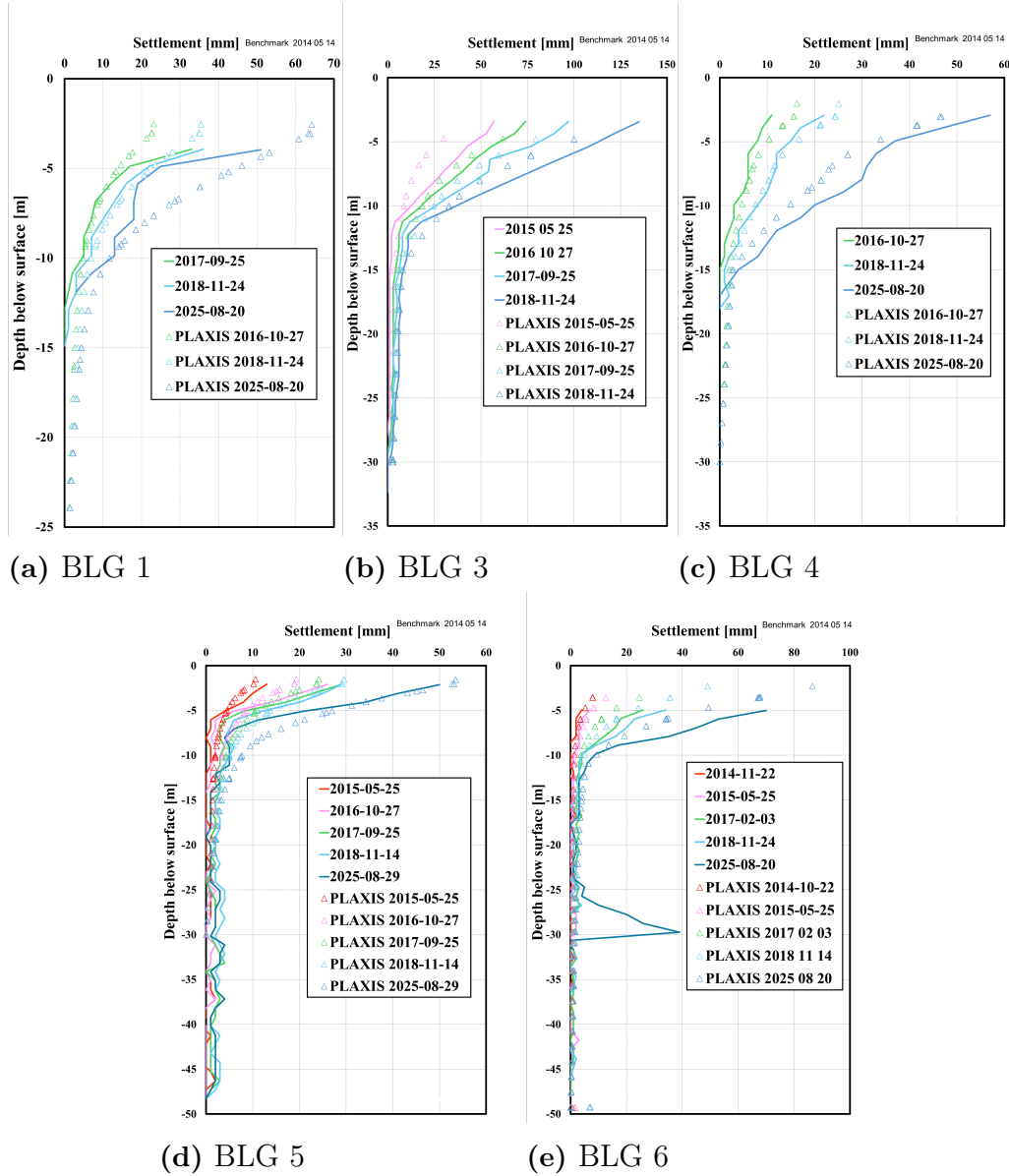


Figure 4.5: Bellow hose data. Comparison between measured and PLAXIS simulated vertical displacements.

The calibrated OCR distribution is presented in Fig. 4.6. Relatively high OCR values were required in several clay layers to reproduce the measured settlement behaviour. Lower OCR values resulted in excessive settlements during the early stages of the analysis, while higher OCR values delayed the onset of compression and produced better agreement with the observed settlement rates after 2014. The resulting OCR distribution suggests that the clay deposit could not be represented as purely normally consolidated at the beginning of the analysis.

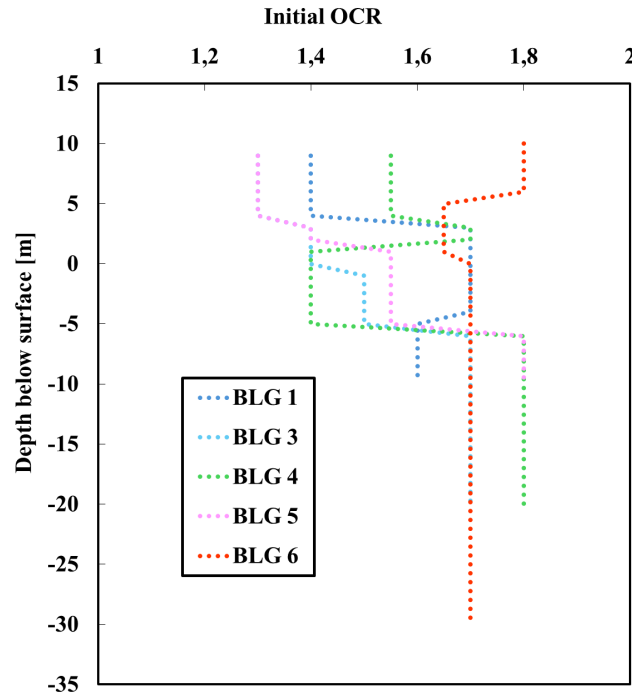


Figure 4.6: Initial OCR input for bellow hoses.

The InSAR measurements showed similar long-term settlement rates at the ground surface as those predicted by the numerical model. When comparing settlement rates between selected time intervals, the extracted PLAXIS settlements at the top of the bellow locations were generally consistent with the satellite observations. For example, BLG 5 showed a simulated settlement rate of approximately 5 mm/year, which corresponded well with the InSAR measurements in the same area. The additional 100 years beyond the reference period are intended to illustrate how the creep settlement rate develops over a long time span. The results show that the rate gradually decreases to approximately 1–2 mm/year, which is considered reasonable when compared with nearby InSAR points that are not affected by loading.

The agreement between bellow hose measurements, InSAR observations and numerical simulations indicates that the calibrated model provides a reasonable representation of the ongoing long-term settlement behaviour at Sävenäs. Uncertainties remain, primarily related to local stratigraphic variability and the historical loading conditions of the site.

4.3 Quasi-static Modelling Results

Fig. 4.7 illustrates the three stages used to represent the operational schedule of the railway depot. In the first stage, three loads are activated, followed by five loads in the second stage and eight loads in the third stage. The figures show how the excess pore pressure varies with the applied loads, highlighting a localised influence depending on which loads are active. A similar trend can be observed for the vertical settlements, where local variations occur in each phase, and the settlements do not extend deeper than approximately 15 meters. This sequence of three stages is repeated over a four-year period, with a randomised activation of tracks. The results show a consistent behaviour throughout the entire period, with a gradual decrease in excess pore pressure over time.

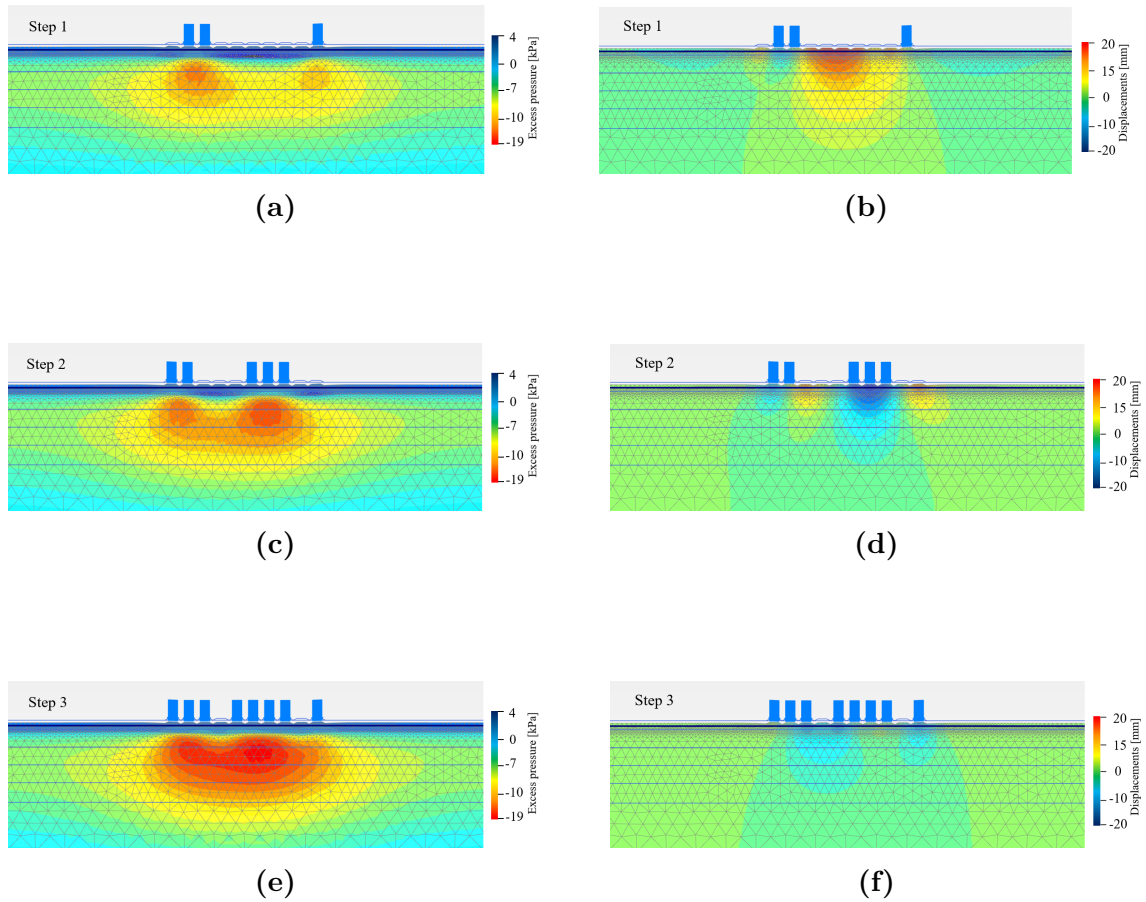


Figure 4.7: Excess pore pressure (on left) and phase displacements (on right) at different for three different load stages: (a) and (b) step 1, (c) and (d) step 2, (e) and (f) step 3.

Fig. 4.8 shows the settlements observed in 2018 and 2025, with 2014 as the reference year, as this is when the bellow hoses were installed. This comparison is performed at this stage to ensure that the model produces reasonable results with depth. The first section A-A is taken at the centre of the embankment, where the largest settlements are expected to occur. The second section, B-B is taken outside the area and

is more representative for comparison with the bellow-hoses, since these are not located directly within the track area.

Regarding the magnitude of settlements, the results appear to be realistic. The model results correspond well with BLG 1, 3, and 4, which are located in approximately the same area and at similar depths as represented in the model. In contrast, BLG 6 is positioned further away and at a significantly greater depth, resulting in larger settlements due to the greater thickness of the clay layer. BLG 3 is strongly influenced by additional factors, leading to increased settlements. Furthermore, reliable measurement data for 2025 are not available for this location, making it less suitable for comparison. Overall, the results show good agreement with real conditions. The rate of settlement also aligns well with InSAR data, with the point located highest in the bellows hoses showing a deformation rate of approximately 4–5 mm/year.

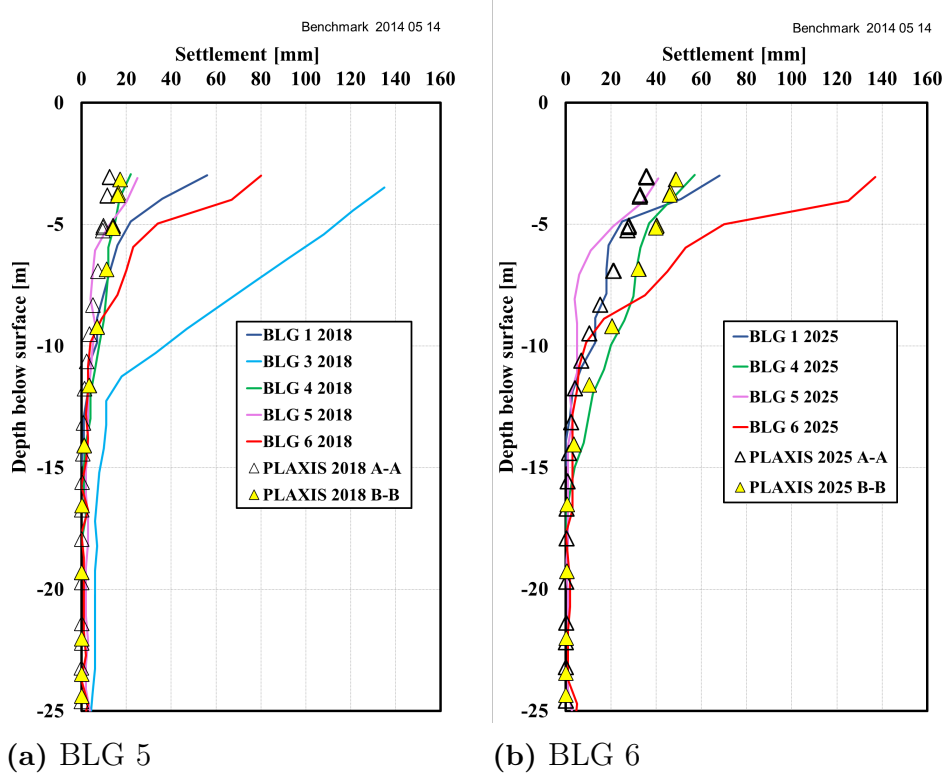


Figure 4.8: Bellow-hose data. Comparison between measured and PLAXIS simulated vertical displacements.

Fig. 4.9 presents the simulated settlement rate at the embankment surface, together with a representative InSAR point from SGI for comparison. Identifying a suitable InSAR point within the depot area proved relatively straightforward, as the observed surface velocities are generally consistent in the range of 4–5 mm/year, which aligns well with the rates predicted by the quasi-static model. The R^2 values within the railway depot area are notably lower than those observed in the surrounding regions, suggesting higher spatial variability in deformation behaviour.

Rather than indicating poor data quality, this variability is interpreted as reflecting the vertical movements induced by the operational loading, which is consistent with the behaviour predicted by the numerical model.

The measured velocity trend nonetheless appears slightly steeper than the simulated curve, despite the quasi-static model only reaching a maximum train occupancy of 80 %. This discrepancy is attributed to the use of a Mohr-Coulomb constitutive model for the embankment and fill material. Under repeated loading, plastic points develop within these layers, and since the Mohr-Coulomb model lacks a hardening mechanism, plastic strains accumulate progressively with each load cycle rather than stabilizing. This leads to a gradual overestimation of the settlement rate contribution from the embankment, which is a known limitation of the current modelling approach.

Despite this limitation, the overall comparison between the numerical model and InSAR observations shows a reasonable level of agreement and supports the validity of the modelling approach for the underlying clay layers, which remain the primary focus of this study. The results demonstrate that the quasi-static captures the general deformation behaviour at the surface with sufficient accuracy for the purposes of this investigation.

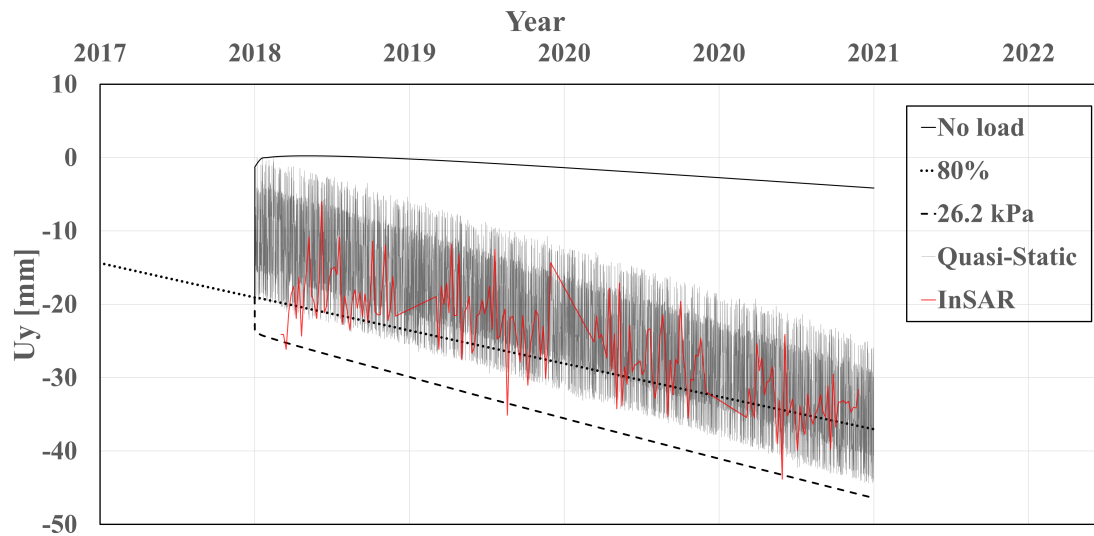


Figure 4.9: Quasi-static loading compared to InSAR measurements from SGI, point-ID: 838196368. Reference curves include no load, 80 % of full load and full load of 26.2 kPa.

In Fig. 4.10, the excess pore pressure and vertical displacement are presented for all clay layers. Focusing first on the excess pore pressure, it can be observed in L1 that the response from the quasi-static loading scenario remains at the 80 % reference level. However, as we move deeper into the profile, the relative magnitude of the quasi-static loading response compared to the other reference curves gradually decreases. In the deeper clay layers, the influence of the quasi-static loading

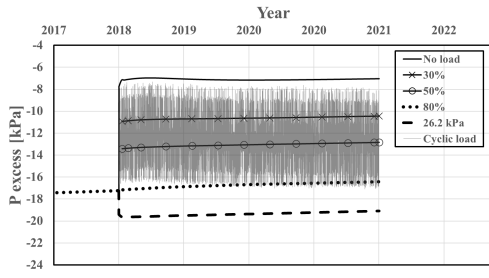
becomes significantly less pronounced. At the greatest depth, the clay layer can be considered relatively unaffected by the applied quasi-static loading. This suggests that the induced stresses dissipate with depth and that the deeper soil layers are only marginally influenced by surface loading conditions.

A similar pattern can be identified in the vertical displacement results. The largest displacement amplitudes occur in the uppermost clay layer, where the settlements exhibit a clear cyclic movement due to the loading conditions. However, the vertical displacement does not strictly follow the trends defined by the 80 % or 100 % load curves. Instead, it produces a slightly lower settlement rate. This deviation supports the earlier observations that there may be modelling uncertainties or simplifications affecting the results and that a considerable portion of the total deformation occurs within the embankment.

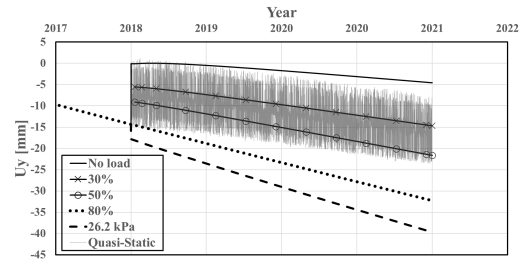
Despite this, it is important to note that significant deformations are still observed in L1. If the evaluation point had been positioned closer to the top of the clay layer instead of at its centre, the calculated settlement rate would likely have been considerably higher and more comparable to the values observed at the ground surface.

In the lower part of the profile, the magnitude of vertical displacement becomes much smaller relative to both the 100% and 80% reference curves. This further confirms that the deeper clay layers are only minimally affected by the quasi-static loading and that the deformation is primarily concentrated in the upper soil layers. This interpretation is consistent with earlier measurements obtained from the bellow hoses, which indicate that the majority of settlements occur in the shallow layers, with very limited deformation observed below 15 meters depth. The shape of the settlement versus depth curve produced by the numerical model closely resembles the trend observed in the field data, providing additional confidence in the model's ability to capture the overall deformation behaviour of the soil profile.

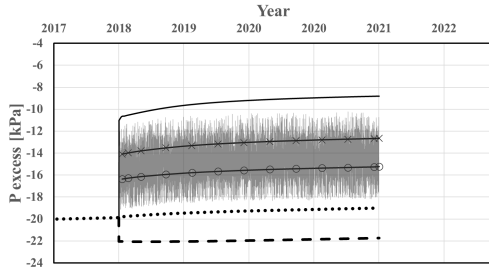
4. Results



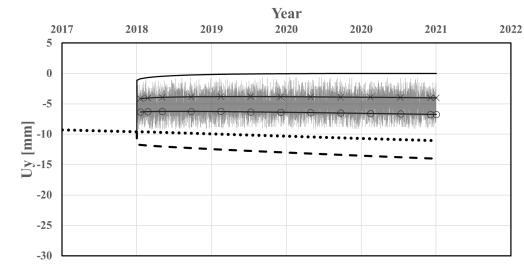
(a) L1



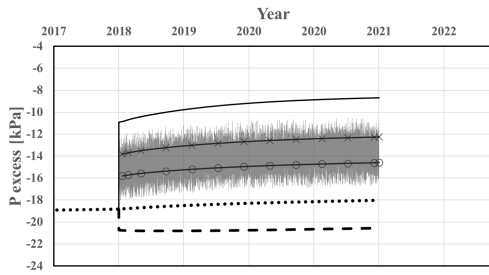
(b) L1



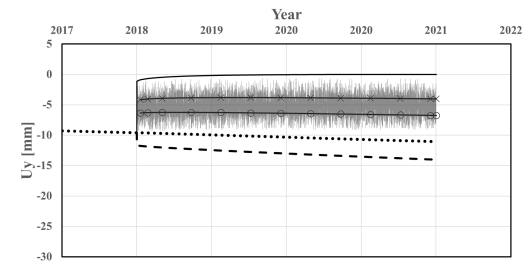
(c) L2



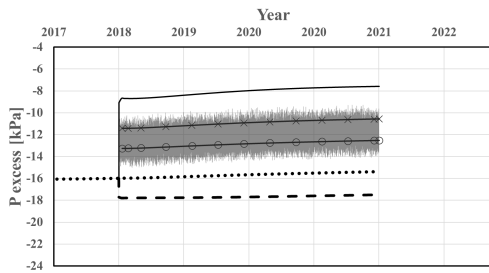
(d) L2



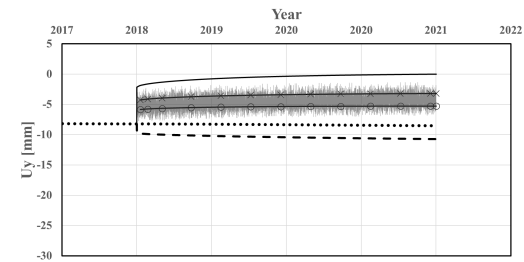
(e) L3



(f) L3



(g) L4



(h) L4

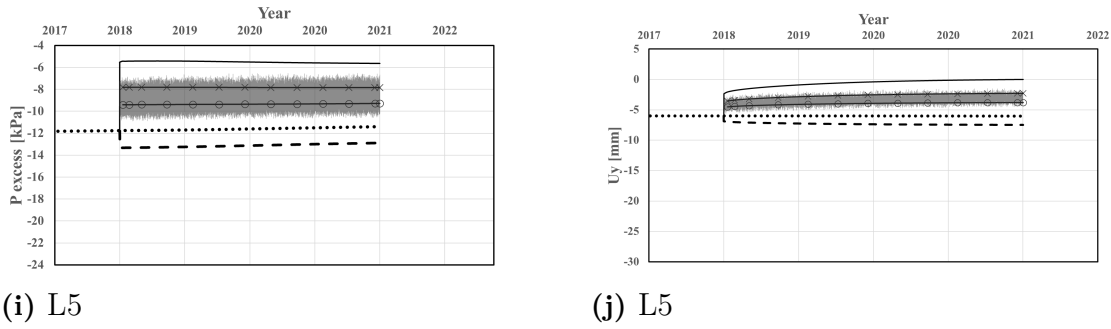


Figure 4.10: Comparison of excess pore pressure and displacement for L1-5 during the analysed time span.

4.4 Reset Displacements

During numerical modelling, an issue related to the "Reset displacements to zero" function in PLAXIS was found, which introduced non physical behaviour in the results. Fig. 4.11 illustrates the change in parameters when the reset displacements to zero option was activated between calculation phases. While the function is intended to isolate deformations caused after a specific phase, the results showed unrealistic settlement behaviour following the reset in phase 2014. Instead of only resetting accumulated displacements, the numerical response indicated that additional internal state variables were also affected. This resulted in abrupt settlement increases that were inconsistent with the expected physical behaviour of the deposit. The behaviour suggests that parameters such as OCR, permeability and void ratio evolution, were unintentionally influenced by the reset operation. Since these variables govern the ongoing creep and consolidation response, the resulting settlements became artificially distorted after resetting.

The reset displacement function was primarily used during the calibration of the bellows hose sections. As the parameters reset, focus was to ensure that the simulated soil conditions remained within ranges representative of present day field conditions post resetting 2014. Each parameter shown in the graphs was continuously evaluated to verify that the model response remained physically reasonable. Particular attention was given to ensuring that the parameter evolution did not become excessively steep, which would indicate unrealistic long term behaviour or unstable constitutive responses in future stages of the analysis. Nevertheless, the curves indicate that some behavioural change has occurred, as they become flatter after the reset displacements compared to their pre reset state. This suggests that, despite certain parameters being reset, aspects of the prior behaviour are still carried over from earlier stages. To avoid introducing these numerical problems, the final analyses were performed without displacement resets.

4. Results

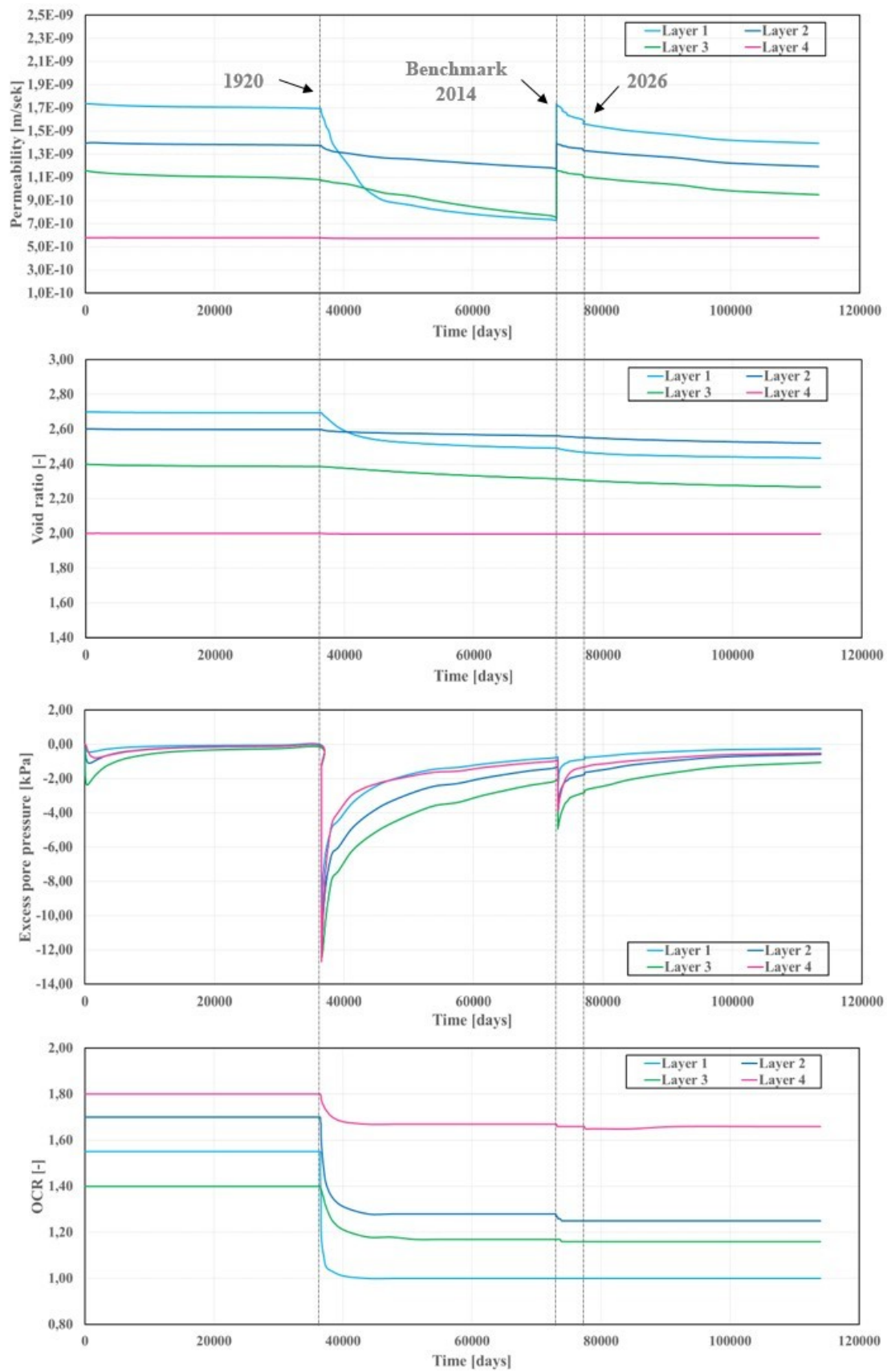


Figure 4.11: Consequence of resetting displacements in PLAXIS in permeability, void ratio, excess pore pressure and OCR.

5

Conclusion

This thesis investigated the long-term settlement behaviour of Sävenäs marshalling yard through advanced numerical modelling, with particular focus on the calibration of the Creep-SCLAY1S constitutive model and the representation of operational train loading using a quasi-static approach. The results demonstrate that the adopted modelling strategy provides a realistic representation of the observed settlement behaviour and that the combination of laboratory testing and field measurements constitutes a viable framework for calibrating advanced constitutive models for soft Gothenburg clay.

The calibrated model reproduced both laboratory behaviour and measured field settlements with good overall agreement. Comparison with bellow hose measurements and InSAR observations showed that the predicted settlement magnitudes and long-term settlement rates were generally consistent with the observed behaviour. Although local deviations were present, these are considered primarily attributable to natural spatial variability and uncertainties associated with the historical development of the site. The agreement obtained using two independent monitoring techniques increases confidence in the adopted parameter set and modelling methodology.

The implemented quasi-static loading approach successfully represented the intermittent loading conditions characteristic of a marshalling yard without the need for explicit cyclic modelling. Comparison with simplified reference cases demonstrated that operational loading cannot be adequately represented by assuming a single constant load level, as both loading duration and unloading periods influence the long-term settlement response. For the investigated conditions, a constant load corresponding to approximately 50% of the operational loading provided the closest simplified approximation. However, this value should not be regarded as generally applicable to all sites.

The study contributes to the practical application of the Creep-SCLAY1S model by extending its use beyond conventional embankment problems to a railway marshalling yard subjected to repeated operational loading. Furthermore, the script developed for quasi-static loading provides an alternative for analyses while retaining the ability to account for time-dependent loading patterns. The combined use of InSAR observations and bellow hose measurements also demonstrates the value of

integrating multiple field monitoring techniques for numerical model validation. In addition, a numerical issue related to the handling of internal state variables when using the Reset displacements option in PLAXIS was identified and documented, which is of practical relevance for future works within PLAXIS.

Several limitations should be considered when interpreting the results. All analyses were performed under plane strain conditions, neglecting three-dimensional variations in geometry and loading across the yard. In addition, the historical construction sequence is only partially documented, introducing uncertainty into the reconstructed stress history. The available IL tests were also not loaded sufficiently to uniquely determine all parameters required by the Creep-SCLAY1S model. Consequently, several anisotropy and destructuration parameters relied on theoretical relationships, recommended parameter ranges, and calibration against field observations rather than direct laboratory determination. Although the adopted parameter set reproduced the observed settlement behaviour well, its uniqueness cannot be guaranteed.

Future work should focus on reducing these uncertainties through improved site characterisation and model representation. The development of a numerical model based on a real cross section rather than an interpreted one would improve the representation of local geometry and stratigraphy. Additional laboratory testing specifically targeting anisotropy and destructuration would reduce uncertainty in constitutive parameters, while more detailed documentation of historical filling events would improve reconstruction of the initial stress state. Finally, the quasi-static loading procedure could be further refined by incorporating actual train schedules and load distributions, allowing differences between freight and passenger traffic to be represented more realistically.

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A

Constitutive Models

A.1 Generalized Creep-Sclay1S

The generalised 3D formulation of Creep-SCLay1S uses the following main definitions.

Deviatoric stress tensor:

$$\sigma'_d = \begin{bmatrix} \sigma'_x - p' \\ \sigma'_y - p' \\ \sigma'_z - p' \\ \sqrt{2}\sigma'_{xy} \\ \sqrt{2}\sigma'_{yz} \\ \sqrt{2}\sigma'_{zx} \end{bmatrix} \quad (\text{A.1})$$

Deviatoric strain tensor:

$$\epsilon_d = \begin{bmatrix} \frac{1}{3}(2d\epsilon_x - d\epsilon_y - d\epsilon_z) \\ \frac{1}{3}(-d\epsilon_x + 2d\epsilon_y - d\epsilon_z) \\ \frac{1}{3}(-d\epsilon_x - d\epsilon_y + 2d\epsilon_z) \\ \sqrt{2}\epsilon_{xy} \\ \sqrt{2}\epsilon_{yz} \\ \sqrt{2}\epsilon_{zx} \end{bmatrix} \quad (\text{A.2})$$

Deviatoric fabric tensor:

$$\alpha_d = \begin{bmatrix} \frac{1}{3}(2d\alpha_x - d\alpha_y - d\alpha_z) \\ \frac{1}{3}(-d\alpha_x + 2d\alpha_y - d\alpha_z) \\ \frac{1}{3}(-d\alpha_x - d\alpha_y + 2d\alpha_z) \\ \sqrt{2}\alpha_{xy} \\ \sqrt{2}\alpha_{yz} \\ \sqrt{2}\alpha_{zx} \end{bmatrix} = \begin{bmatrix} \alpha_x - 1 \\ \alpha_y - 1 \\ \alpha_z - 1 \\ \sqrt{2}\alpha_{xy} \\ \sqrt{2}\alpha_{yz} \\ \sqrt{2}\alpha_{zx} \end{bmatrix} \quad (\text{A.3})$$

In three-dimensional stress space the yield surface of the Creep-SCLAY1S model is a sheared ellipsoid, see Fig. A.1, given by

$$f_{surface} = \frac{3}{2}[\{\boldsymbol{\sigma}_d - p'\boldsymbol{\alpha}_d^2\}^T\{\boldsymbol{\sigma}_d - p'\boldsymbol{\alpha}_d^2\}] - [M(\theta)^2 - \frac{3}{2}\{\boldsymbol{\alpha}_d\}^T\boldsymbol{\alpha}_d](p'_m - p)p' = 0 \quad (\text{A.4})$$

where $\boldsymbol{\sigma}_d$ is the deviatoric stress tensor, p' is the mean effective stress, $\boldsymbol{\alpha}_d$ is the deviatoric fabric tensor characterising anisotropy in 3D, $M(\theta)$ is Lode angle dependant value of the critical state stress ratio, p'_m defines the size of the natural yield surface of the clay.

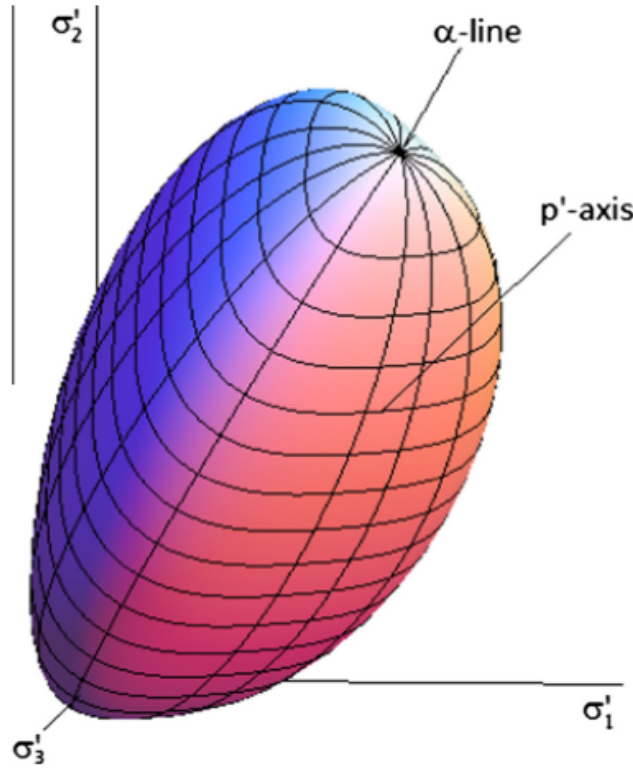


Figure A.1: Definition of Creep-SCLAY1S in general stress space. From [10].

The rotational hardening law describing the change of the orientation of the yield surface with irrecoverable straining can be expressed in general stress space as:

$$d\boldsymbol{\alpha}_d = \omega \left(\left[\frac{3\boldsymbol{\sigma}'_d}{4p'} - \boldsymbol{\alpha}_d \right] \langle d\varepsilon_v^e \rangle + \omega_d \left[\frac{\boldsymbol{\sigma}'_d}{3p'} - \boldsymbol{\alpha}_d \right] \langle |d\varepsilon_c^e| \rangle \right) \quad (\text{A.5})$$

The direction of creep strain rate is defined (associated flow rule) as

$$\dot{\varepsilon}_{ij}^c = \dot{\lambda} \frac{\partial p'_{eq}}{\partial \sigma'_{ij}} \quad (\text{A.6})$$

The total strain rate is sum of the elastic and creep part as

$$\dot{\epsilon}_{ij} = \dot{\epsilon}_{ij}^e + \dot{\epsilon}_{ij}^c \quad (\text{A.7})$$

The elastic part of the strains modelled using Hooke's law The elastic part of the strains modelled using Hooke's law

$$\sigma_{ij}^e = D_{ijhk} \dot{\epsilon}_{hk}^e \quad (\text{A.8})$$

where the elastic stiffness matrix D_{ijhk} is

$$D_{ijhk} = \frac{2G(1-\nu')}{1-2\nu'} \delta_{ij} \delta_{hk} + G(\delta_{ik} \delta_{jh} + \delta_{ih} \delta_{jk}) \quad (\text{A.9})$$

B

Numerical Modeling

B.1 Material Data

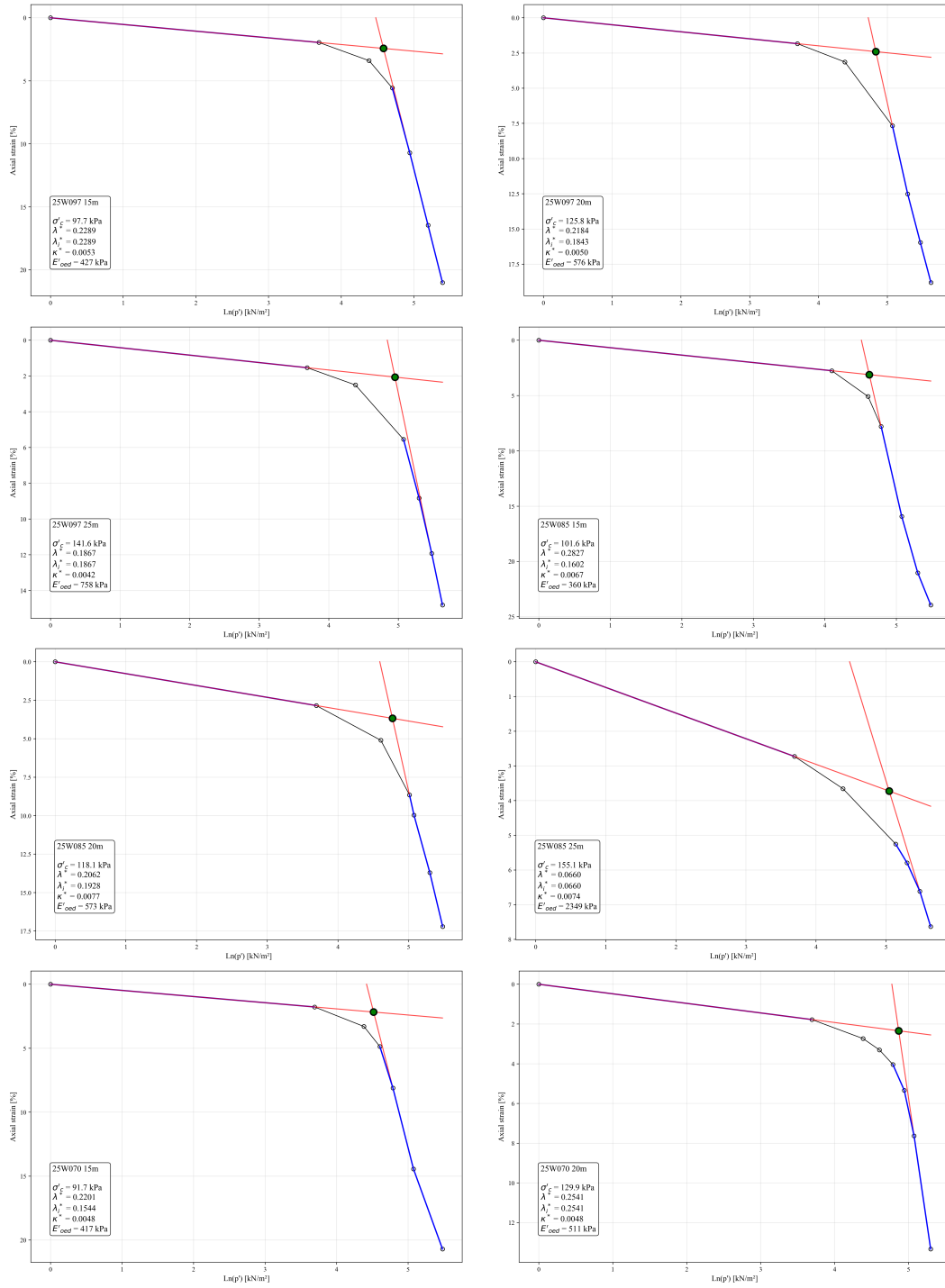
Table B.1: Material data for each bellow hose calibration model. C1 - C4 represents clay layer 1 to 4.

Parameter	1C1	1C2	1C3	1C4	3C1	3C2	3C3	3C4	4C1	4C2
BLG	1	1	1	1	3	3	3	3	4	4
Layer	C1	C2	C3	C4	C1	C2	C3	C4	C1	C2
γ	15,5	16	16	18	15,5	16	16	18	15,5	16
OCR	1,4	1,7	1,7	1,7	1,3	1,4	1,5	1,7	1,55	1,7
e_o	2,7	2,6	2,4	2,0	2,7	2,6	2,4	2,0	2,7	2,6
k	1,50E-04	1,20E-04	1,00E-04	5,00E-05	1,50E-04	1,50E-04	1,50E-04	5,00E-05	1,50E-04	1,20E-04
c_k	0,55	0,52	0,45	0,40	0,55	0,52	0,45	0,40	0,55	0,52
E^{ref}	500	600	700	1500	500	600	700	1500	500	600
$\kappa^{*}_{\varepsilon od}$	0,015	0,015	0,015	0,013	0,015	0,015	0,015	0,013	0,015	0,015
ν	0,2	0,2	0,2	0,2	0,2	0,2	0,2	0,2	0,2	0,2
λ_i	0,08	0,08	0,085	0,09	0,08	0,08	0,085	0,09	0,08	0,08
Mc	1,2	1,2	1,2	1,2	1,2	1,2	1,2	1,2	1,2	1,2
Me	1,2	1,2	1,2	1,2	1,2	1,2	1,2	1,2	1,2	1,2
ω	30	30	30	30	30	30	30	30	30	30
ω_d	0,8	0,8	0,8	0,8	0,8	0,8	0,8	0,8	0,8	0,8
ξ	9	9	9	9	9	9	9	9	9	9
ξ_d	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,4
α_0	0,5	0,5	0,5	0,5	0,5	0,5	0,5	0,5	0,5	0,5
χ_0	20	20	20	20	20	20	20	20	20	20
τ	1	1	1	1	1	1	1	1	1	1
μ_i	2,00E-03	2,00E-03	2,00E-03	2,00E-03	2,00E-03	2,00E-03	2,00E-03	2,00E-03	2,00E-03	2,00E-03
K_0	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6

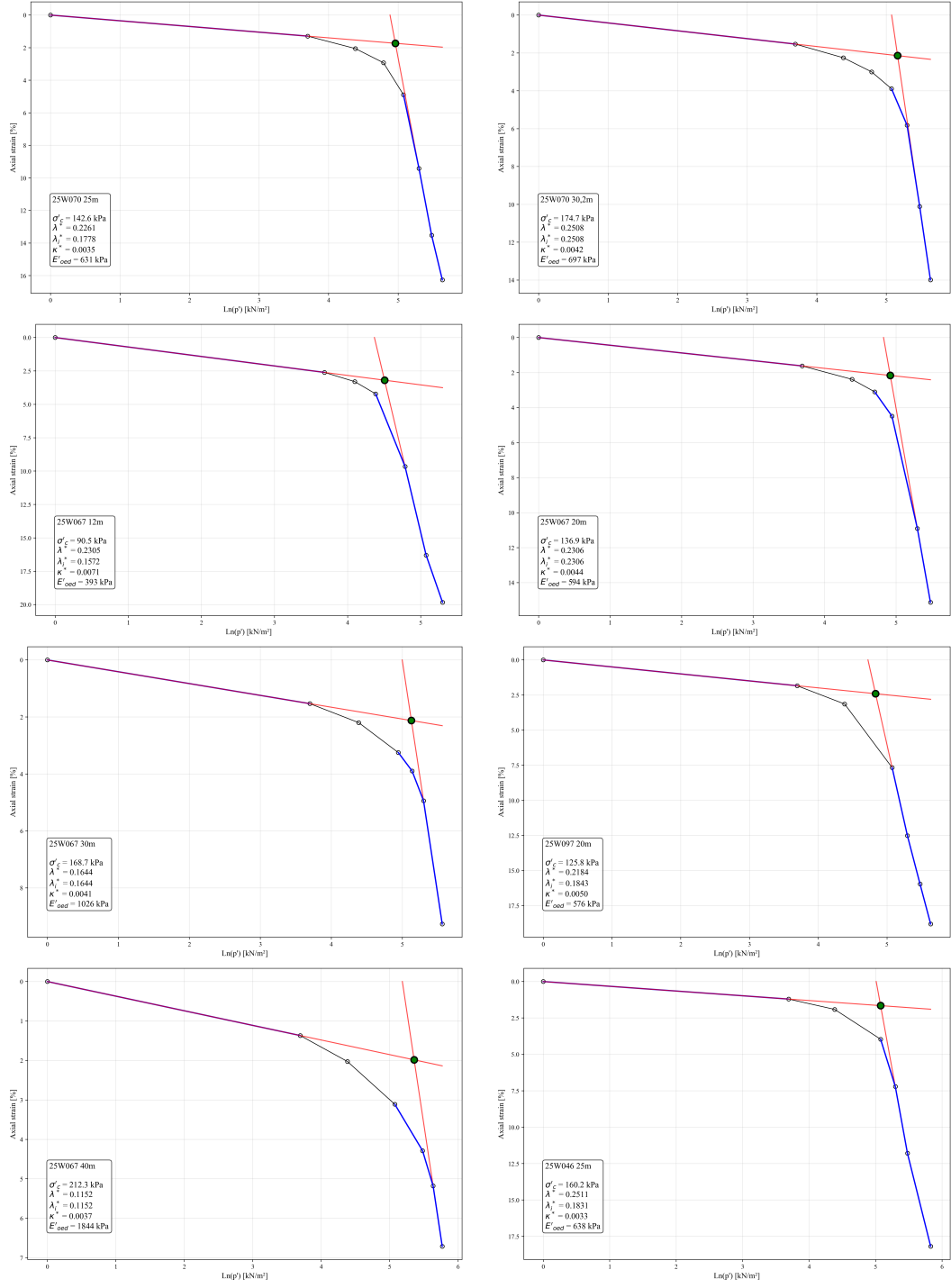
B. Numerical Modeling

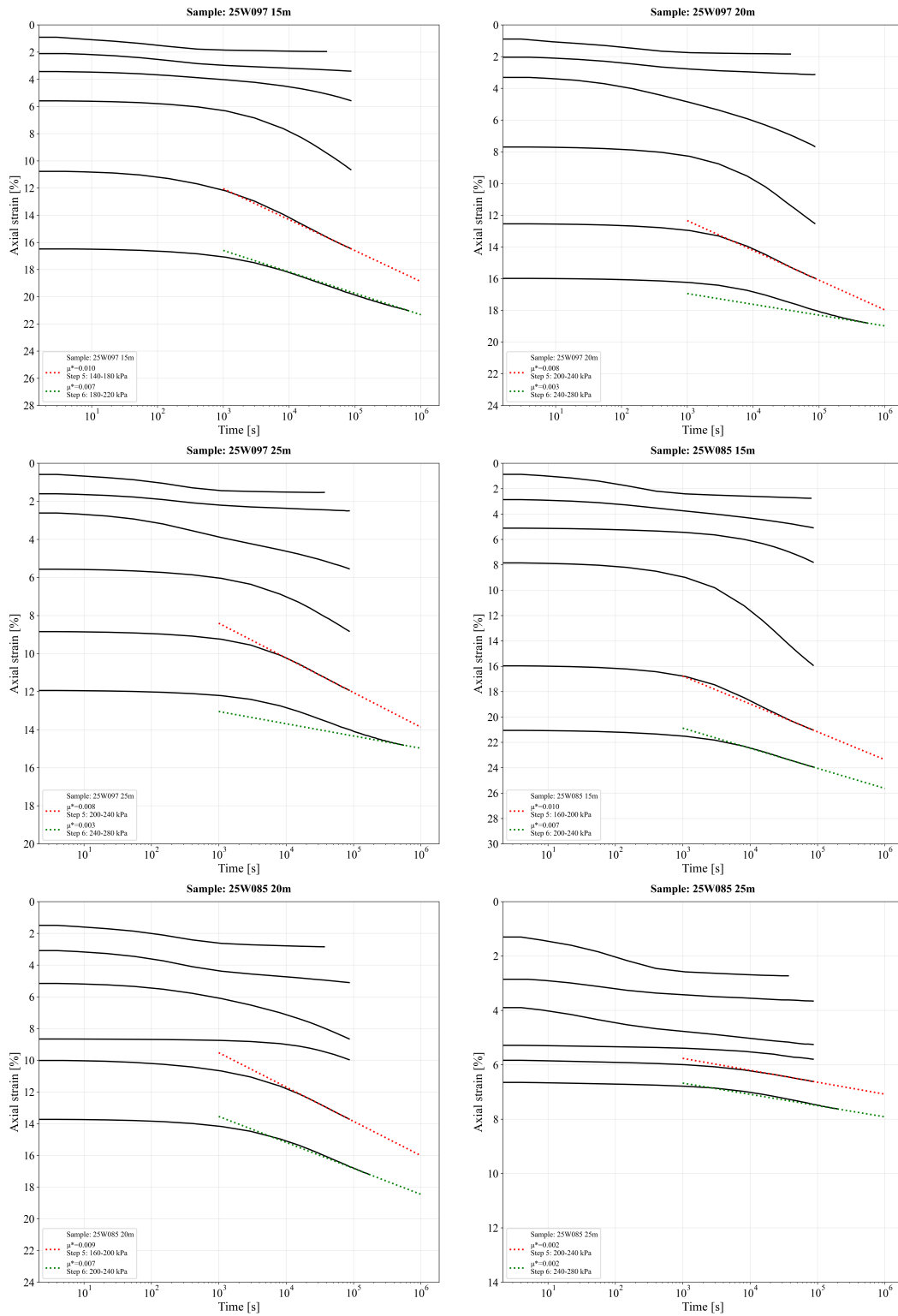
Parameter	4C3	4C4	5C1	5C2	5C3	5C4	6C1	6C2	6C3	6C4
BLG	4	4	5	5	5	5	6	6	6	6
Layer	C3	C4	C1	C2	C3	C4	C1	C2	C3	C4
γ	16	18	15,5	16	16	18	16	16,5	16,5	17
OCR	1,4	1,8	1,3	1,4	1,55	1,8	1,8	1,65	1,65	1,7
e_o	2,4	2,0	1,3	2,6	2,4	2,0	2,8	2,5	2,0	1,8
k	1,00E-04	5,00E-05	1,50E-04	1,20E-04	1,00E-04	5,00E-05	1,40E-04	1,20E-04	1,04E-04	5,00E-05
c_k	0,45	0,40	0,55	0,52	0,45	0,40	0,568	0,568	0,455	0,45
E_k^{ref}	700	1500	500	600	700	1500	500	600	700	1500
$\kappa_{\epsilon od}^*$	0,015	0,013	0,015	0,015	0,015	0,013	0,015	0,015	0,01	0,01
ν	0,2	0,2	0,2	0,2	0,2	0,2	0,2	0,2	0,2	0,2
λ_i	0,085	0,09	0,08	0,08	0,085	0,09	0,08	0,08	0,08	0,08
Mc	1,2	1,2	1,2	1,2	1,2	1,2	1,2	1,2	1,2	1,2
Me	1,2	1,2	1,2	1,2	1,2	1,2	1,2	1,2	1,2	1,2
ω	30	30	30	30	30	30	30	30	30	30
ω_d	0,8	0,8	0,8	0,8	0,8	0,8	0,8	0,8	0,8	0,8
ξ	9	9	9	9	9	9	9	9	9	9
ξ_d	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,4	0,4
α_0	0,5	0,5	0,5	0,5	0,5	0,5	0,5	0,5	0,5	0,5
χ_0	20	20	20	20	20	20	20	20	20	20
τ	1	1	1	1	1	1	1	1	1	1
μ_i	2,00E-03	2,00E-03	2,00E-03	2,00E-03	2,00E-03	2,00E-03	2,00E-03	2,00E-03	2,00E-03	2,00E-03
K_0	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6

B.2 Oedometer Tests

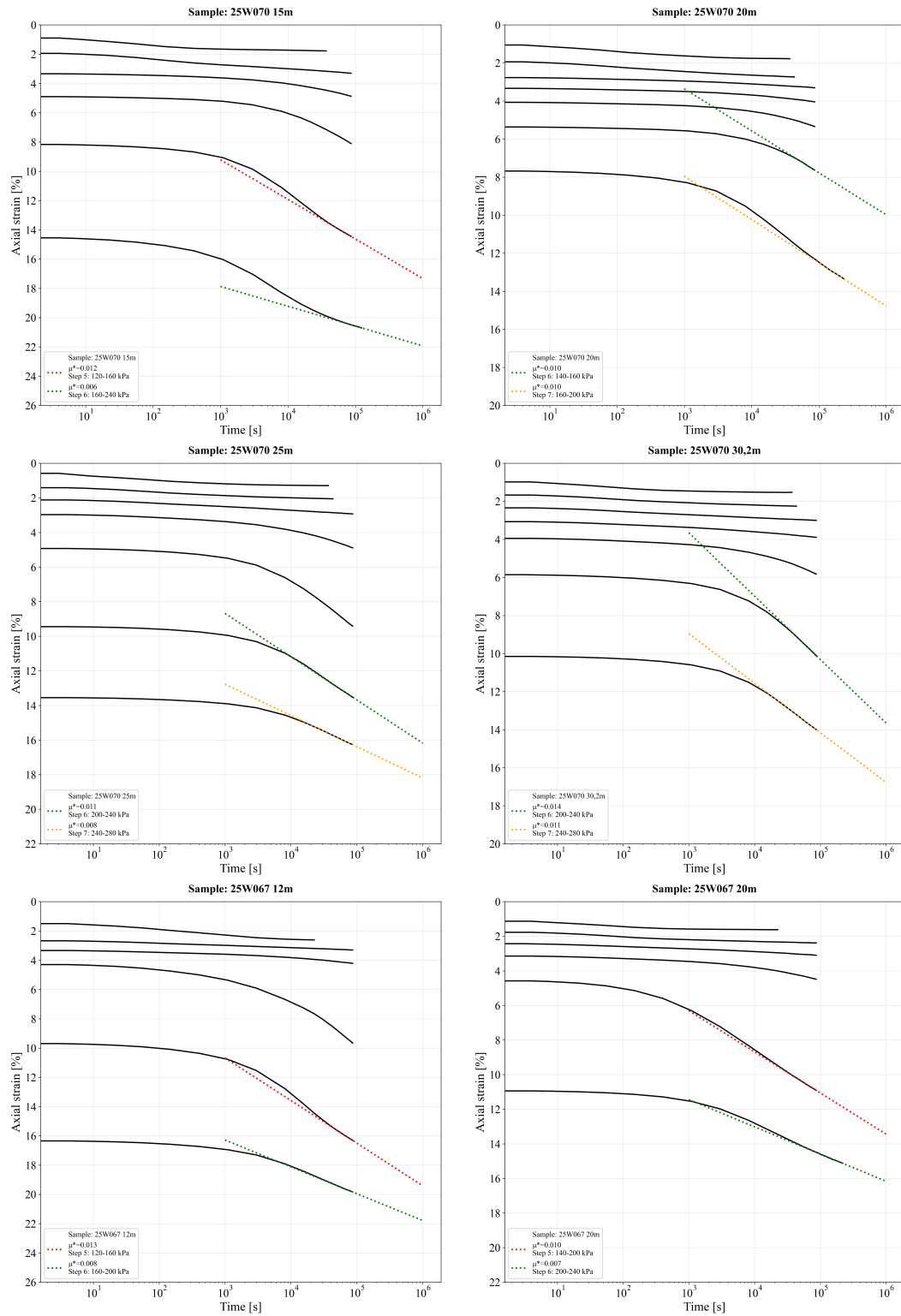


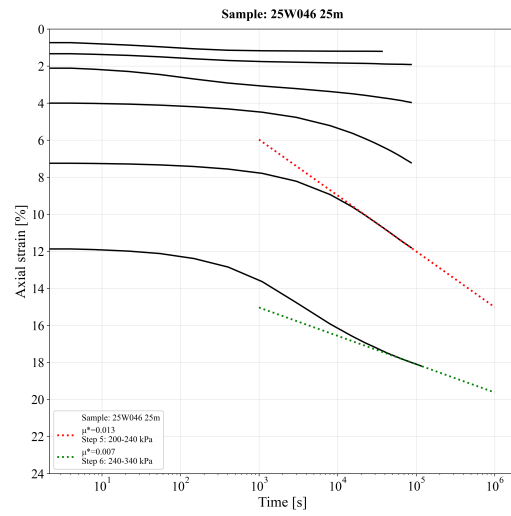
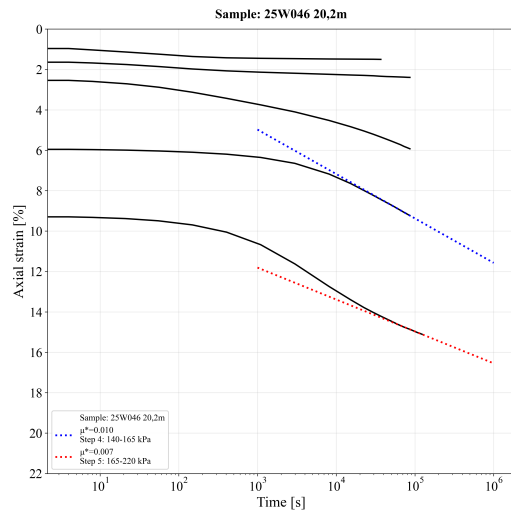
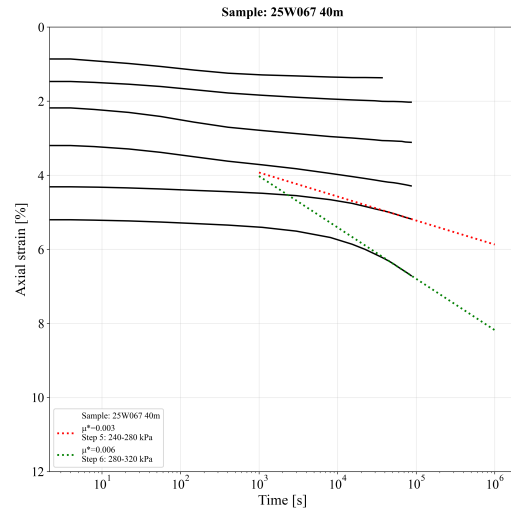
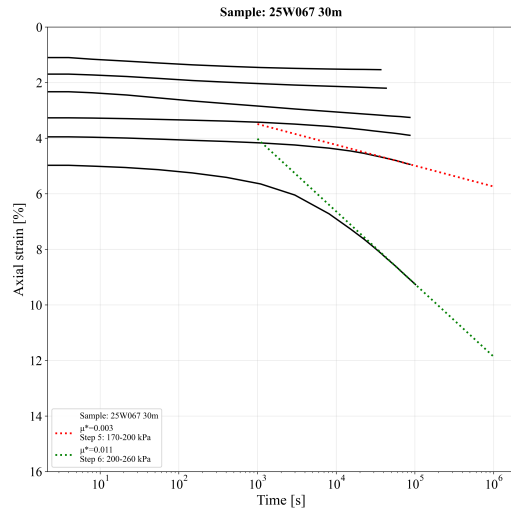
B. Numerical Modeling



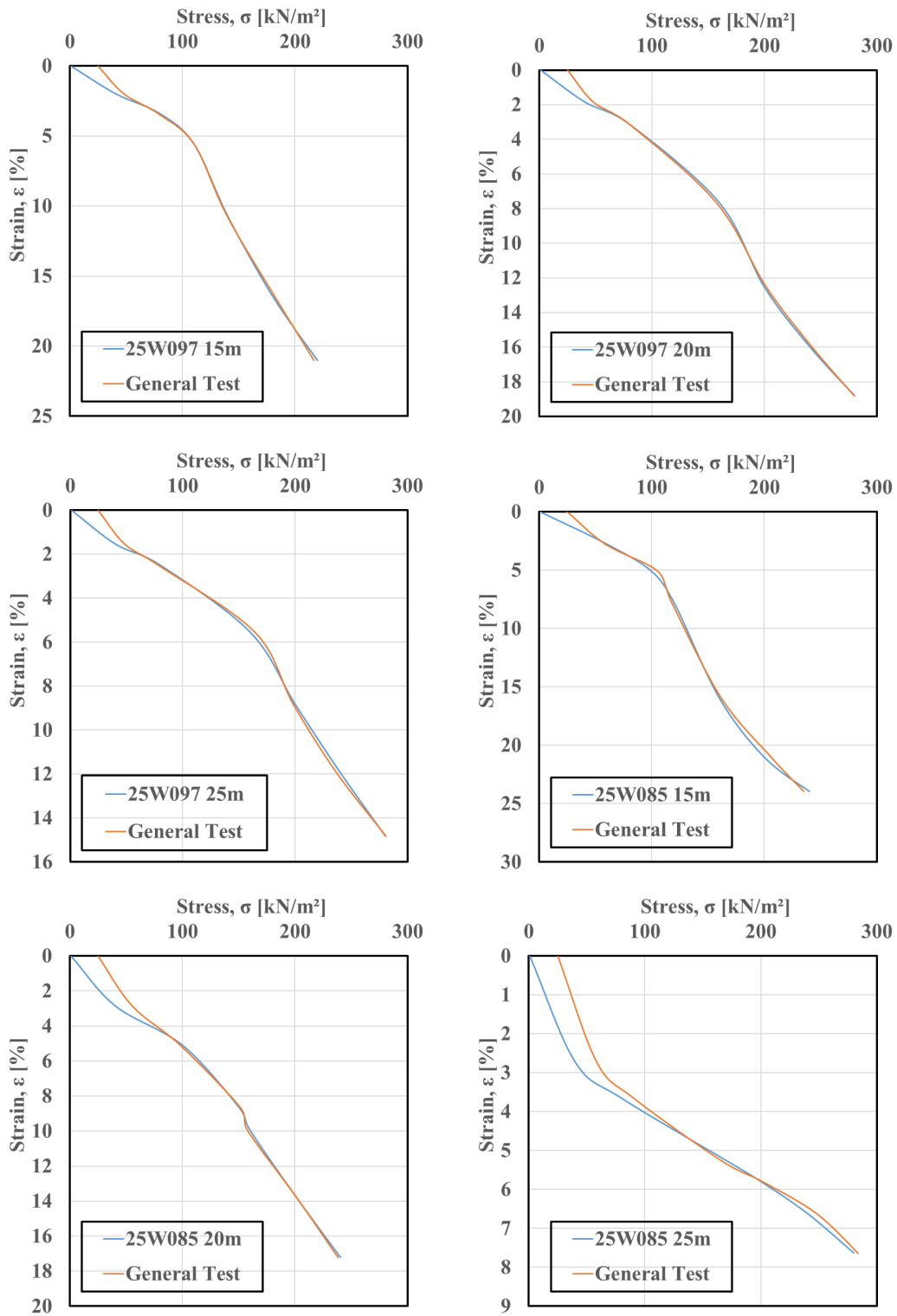


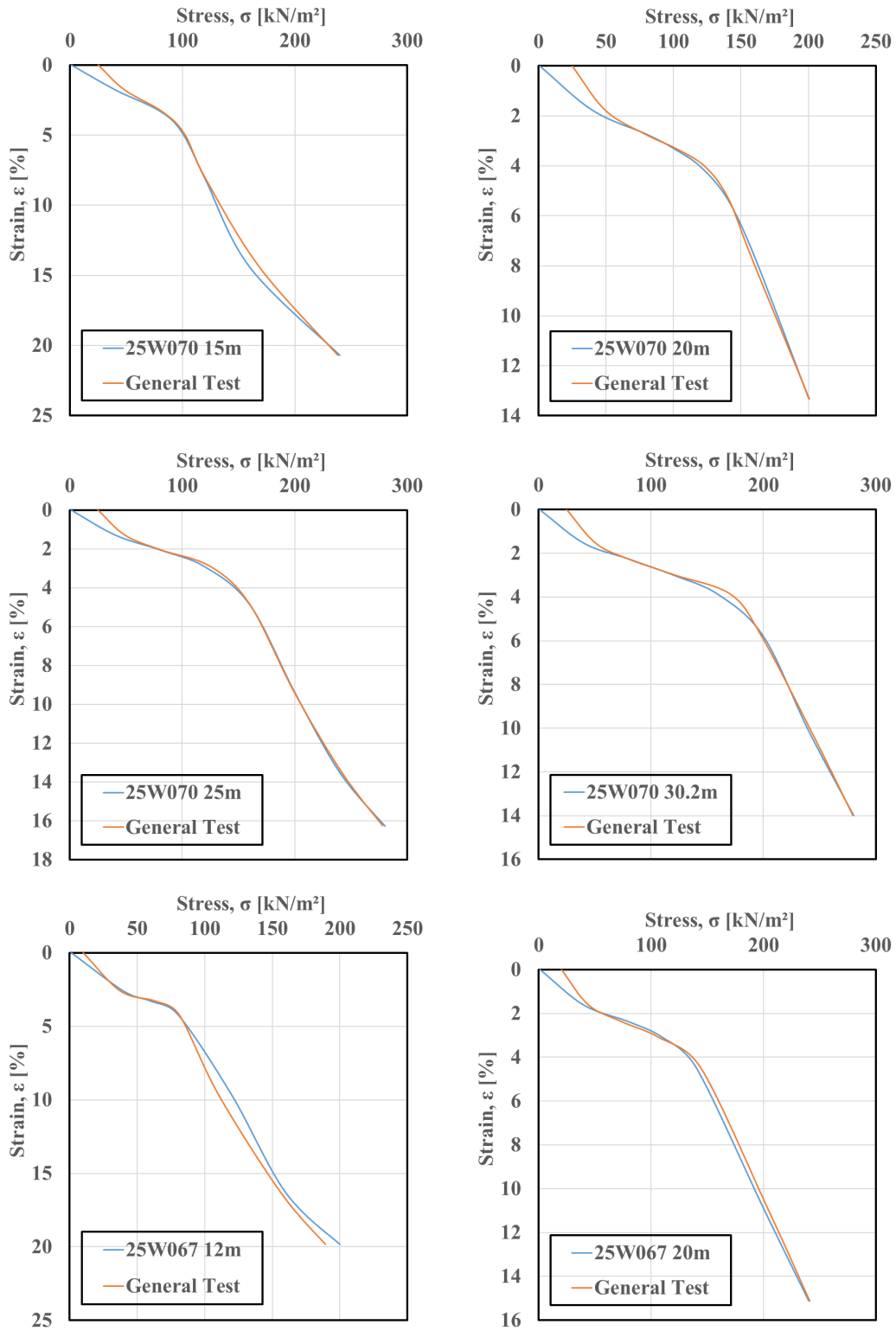
B. Numerical Modeling

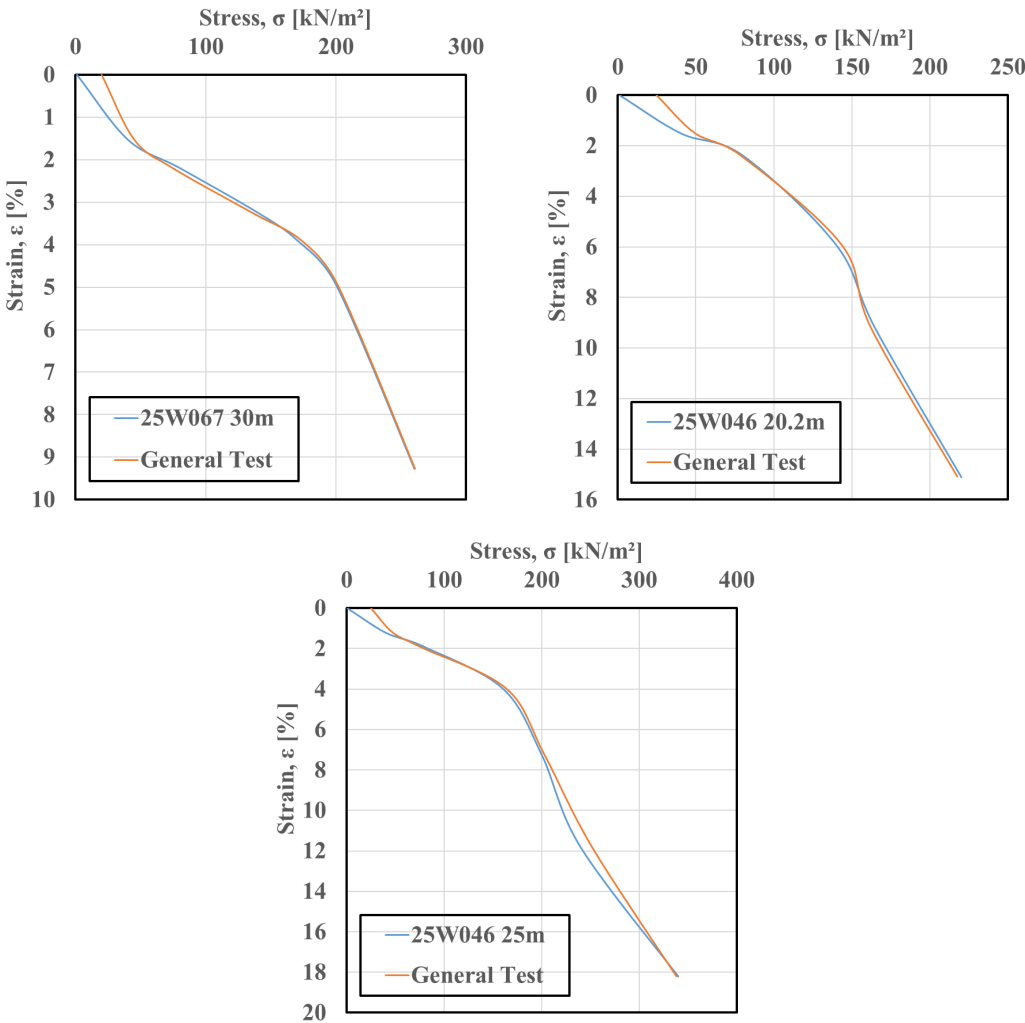




B.3 PLAXIS Soil Match







C

Python Scripts

Note: All sensitive values (passwords and private directories) have been replaced with placeholders.

C.1 Quasi-static Loading Script

```
1 from plxscripting.easy import *
2 import random
3 import json
4 import sys
5 from datetime import datetime
6
7 try:
8     # Load configuration
9     config_file = "<CONFIG_FILE_PATH>"
10
11     # Get number of tracks from command line
12     if len(sys.argv) < 2:
13         print("Usage: python cyclic_v2_remote_thesis.py <num_tracks>")
14         raise Exception("Number of tracks must be specified as command-line argument")
15
16     try:
17         NUM_TRACKS = int(sys.argv[1])
18     except ValueError:
19         raise Exception(f"Invalid track count: {sys.argv[1]}. Must be an integer.")
20
21     with open(config_file, 'r') as f:
22         config = json.load(f)
23
24     # PLAXIS connection settings
25     LOCALHOST_PORT = 'YOUR_PORT_NUMBER'
26     PASSWORD = 'YOUR_PASSWORD'
27
28     # Extract configuration
29     SCHEDULE_DURATION = config['simulation']['total_duration_days']
30     TRAIN_DURATION = config['simulation']['train_duration_hours'] / 24
31     DAILY_SCHEDULE = config['daily_schedule']
```

```
32     LOAD_VALUE = config['simulation']['load_kpa']
33
34     # Connect to PLAXIS
35     s_i, g_i = new_server('localhost', LOCALHOST_PORT, password=PASSWORD)
36
37     # Get phases and loads
38     phases_list = list(g_i.phases)
39     if not phases_list:
40         raise Exception("No phases found in model")
41
42     # Detect available loads dynamically
43     loads = []
44     load_idx = 1
45     while True:
46         try:
47             load = getattr(g_i, f"LineLoad_{load_idx}_1")
48             loads.append(load)
49             load_idx += 1
50         except:
51             break
52
53     if not loads:
54         raise Exception("No loads found in model")
55
56     # Find starting phase (ES2C or last phase)
57     embankment_consolidation_phase = None
58     for p in phases_list:
59         phase_name = str(p.Name)
60         if "ES2C" in phase_name or phase_name == "Phase_8":
61             embankment_consolidation_phase = p
62             break
63
64     if not embankment_consolidation_phase:
65         embankment_consolidation_phase = phases_list[-1]
66
67     last_phase = embankment_consolidation_phase
68     total_schedule_time = SCHEDULE_DURATION
69
70     # Create comparison phases
71
72     # No Load phase
73     phase_no_load = g_i.phase(embankment_consolidation_phase)
74     phase_no_load.Name = "COMPARISON_NoLoad"
75     phase_no_load.DeformCalcType = "Consolidation"
76     phase_no_load.TimeInterval = total_schedule_time
77     phase_no_load.PreviousPhase = embankment_consolidation_phase
78
79     for feature in phase_no_load.UserFeatures:
80         if feature.TypeName == "Deform":
81             feature.ForceFullyDrainedOnActivation = False
```

```
82         break
83
84     try:
85         phase_no_load.Deform.UseDefaultIterationParams = False
86         phase_no_load.Deform.MaxLoadFractionPerStep = 0.1
87     except:
88         pass
89
90     for load in loads:
91         load.deactivate(phase_no_load)
92
93     # Full Load phase
94     phase_full_load = g_i.phase(embankment_consolidation_phase)
95     phase_full_load.Name = "COMPARISON_FullLoad"
96     phase_full_load.DeformCalcType = "Consolidation"
97     phase_full_load.TimeInterval = total_schedule_time
98     phase_full_load.PreviousPhase = embankment_consolidation_phase
99
100    for feature in phase_full_load.UserFeatures:
101        if feature.TypeName == "Deform":
102            feature.ForceFullyDrainedOnActivation = False
103            break
104
105    try:
106        phase_full_load.Deform.UseDefaultIterationParams = False
107        phase_full_load.Deform.MaxLoadFractionPerStep = 0.1
108    except:
109        pass
110
111    for load in loads:
112        load.activate(phase_full_load)
113
114    try:
115        full_load_value = -LOAD_VALUE
116        for load in loads:
117            load.qy_start[phase_full_load] = full_load_value
118    except:
119        pass
120
121    # 1/3 Load phase
122    phase_third_load = g_i.phase(embankment_consolidation_phase)
123    phase_third_load.Name = "COMPARISON_ThirdLoad"
124    phase_third_load.DeformCalcType = "Consolidation"
125    phase_third_load.TimeInterval = total_schedule_time
126    phase_third_load.PreviousPhase = embankment_consolidation_phase
127
128    for feature in phase_third_load.UserFeatures:
129        if feature.TypeName == "Deform":
130            feature.ForceFullyDrainedOnActivation = False
131            break
```

```
132
133     try:
134         phase_third_load.Deform.UseDefaultIterationParams = False
135         phase_third_load.Deform.MaxLoadFractionPerStep = 0.1
136     except:
137         pass
138
139     for load in loads:
140         load.activate(phase_third_load)
141
142     try:
143         third_load_value = -LOAD_VALUE / 3
144         for load in loads:
145             load.qy_start[phase_third_load] = third_load_value
146     except:
147         pass
148
149     # Generate schedule based on occupancy rates
150
151     schedule = []
152     current_day = 0
153     event_count = 0
154
155     while current_day < SCHEDULE_DURATION:
156         for period in DAILY_SCHEDULE:
157             occupancy = period['occupancy_rate']
158             period_hours = period['hours_end'] - period['hours_start']
159             period_days = period_hours / 24
160
161             occupied_time_in_period = period_days * occupancy
162             trains_in_period = int(occupied_time_in_period / TRAIN_DURATION + 0.5)
163
164             if trains_in_period == 0:
165                 trains_in_period = 1 if occupancy > 0.15 else 0
166
167             period_start_day = current_day + (period['hours_start'] / 24)
168             gap_between_trains = period_days / max(trains_in_period, 1)
169
170             for train_idx in range(trains_in_period):
171                 train_start = period_start_day + (train_idx * gap_between_trains)
172
173                 if train_start + TRAIN_DURATION <= current_day + 1:
174                     # Calculate occupied tracks with smart rounding
175                     if occupancy < 0.4:
176                         occupied_tracks = int(NUM_TRACKS * occupancy) # Round DOWN for l
177                     else:
178                         occupied_tracks = int(NUM_TRACKS * occupancy + 0.9) # Round UP f
179                     occupied_tracks = max(1, occupied_tracks)
180
181                 schedule.append({
```

```

182         "type": "train",
183         "load_kpa": LOAD_VALUE,
184         "start": train_start,
185         "duration": TRAIN_DURATION,
186         "end": train_start + TRAIN_DURATION,
187         "occupancy": occupancy,
188         "occupied_tracks": occupied_tracks
189     })
190     event_count += 1
191
192     current_day += 1
193
194     # Create phases from schedule
195
196     phases_created = 0
197     for idx, event in enumerate(schedule, 1):
198         phase = g_i.phase(last_phase)
199         occupied_tracks = event['occupied_tracks']
200         phase.Name = f"Event_{idx}_{occupied_tracks}tracks"
201         phase.DeformCalcType = "Consolidation"
202         phase.TimeInterval = event['duration']
203
204         for feature in phase.UserFeatures:
205             if feature.TypeName == "Deform":
206                 feature.ForceFullyDrainedOnActivation = False
207                 break
208
209         # Randomly select which loads to activate
210         loads_to_activate = random.sample(range(len.loads)), occupied_tracks)
211         loads_to_activate.sort()
212
213         for i, load in enumerate.loads):
214             if i in loads_to_activate:
215                 load.activate(phase)
216             else:
217                 load.deactivate(phase)
218
219         last_phase = phase
220         phases_created += 1
221
222     except Exception:
223         import traceback
224         traceback.print_exc()
225

```

C.2 Phase Deletion Script

```

1 from plxscripting.easy import *
2

```

```
3  try:
4      LOCALHOST_PORT = 'YOUR_PORT_NUMBER'
5      PASSWORD = 'YOUR_PASSWORD'
6
7      s_i, g_i = new_server('localhost', LOCALHOST_PORT, password=PASSWORD)
8
9      # Get all phases
10     phases_list = list(g_i.phases)
11
12     # Find ES2C phase (the one to keep up to)
13     consolidation_phase = None
14     consolidation_index = -1
15
16     for i, phase in enumerate(phases_list):
17         phase_name = str(phase.Name)
18         if "ES2C" in phase_name or phase_name == "Phase_8":
19             consolidation_phase = phase
20             consolidation_index = i
21             break
22
23     if consolidation_index < 0:
24         consolidation_index = len(phases_list) - 1
25         consolidation_phase = phases_list[-1]
26
27     if consolidation_index >= 0:
28         phases_to_delete = phases_list[consolidation_index + 1:]
29     else:
30         phases_to_delete = []
31
32     deleted_count = 0
33     for phase in phases_to_delete:
34         try:
35             g_i.delete(phase)
36             deleted_count += 1
37         except Exception:
38             pass
39
40     except Exception:
41         import traceback
42         traceback.print_exc()
```

C.3 Configuration File

```
1  {
2      "simulation": {
3          "total_duration_days": 365,
4          "train_duration_hours": 8,
5          "load_kpa": 26.2
6      },
```



```
7     "daily_schedule": [  
8         {  
9             "name": "Morning",  
10            "hours_start": 0,  
11            "hours_end": 8,  
12            "occupancy_rate": 0.30  
13        },  
14        {  
15            "name": "Afternoon",  
16            "hours_start": 8,  
17            "hours_end": 16,  
18            "occupancy_rate": 0.50  
19        },  
20        {  
21            "name": "Evening",  
22            "hours_start": 16,  
23            "hours_end": 24,  
24            "occupancy_rate": 0.80  
25        }  
26    ]  
27 }
```

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