THESIS FOR THE DEGREE OF DOCTOR OF PHILOSOPHY

On the temporal evolution of earth pressures in deep excavations in soft clay

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ABSTRACT

Urbanisation and sustainable development of cities drives the need to increasingly utilise underground space. Consequently, there is more demand for deeper and larger excavations in urban areas, pushing the limits of current engineering experience. The vast majority of the reported observations of earth pressures in deep excavations, however, are on lateral earth pressures only and cover the construction stage. Reports on the performance in the serviceability stage are scarce, especially for underground structures in soft clays. In particular, there is a lack of investigations on the evolution of earth pressures below permanent structures at the base of deep excavations. Additionally, quantifying the magnitude and evolution of earth pressures due to delayed heave restrained by structural elements, remains challenging. This thesis investigates the temporal evolution of earth pressures acting on underground structures in deep excavations in soft clay, by means of field monitoring and numerical analyses. The ultimate goal is to generalise the results and develop non-dimensional design charts that quantifies the magnitude and evolution of earth pressures beneath the base of deep excavations and underground structures. The research consists of three parts i) benchmarking of a soil model (Creep-SClay1S) against the observed response of two well-documented excavations, ii) field monitoring of the hydro-mechanical response of soil elements below the base of an excavation and underground structure, and iii) a parametric study, using the Finite Element Method, designed and evaluated using dimensionless parameter groups to generalise the results. The first part demonstrates that the Creep-SClay1S model can be used to compute the magnitude and evolution of earth pressures acting on underground structures in soft clays with sufficient accuracy. Subsequently, the field monitoring of the hydro-mechanical response with clustered instruments enabled unique observations of the evolution of effective stresses and the stress ratio $K{=}\sigma'_{\mathrm{h}}/\sigma'_{\mathrm{v}}$ at soil element level. The parametric study quantifies the impact of the normalised time between the end of excavation and the completion of the restraining structure at the base of the excavation on the emerging magnitude of the effective heave pressures for several scenarios. The results compare well to the field monitoring data and physical model tests. The work presented in this thesis reveals the mechanisms that control the development and evolution of earth pressures in deep excavations. The combination of field monitoring, dimensional analysis and the numerical modelling of the system response have been integrated into design charts. The results can readily be used as a tool for industry to assess the magnitude of effective heave pressure and complement detailed project-specific analyses.

Keywords: soft natural clays, deep excavations, earth pressures, heave, field monitoring, numerical modelling

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Johannes Tornborg
Göteborg, 2024

LIST OF PUBLICATIONS

This thesis consists of an extended summary and the following appended papers:

Paper A	Tornborg, J., Karlsson, M., Kullingsjö, A., & Karstunen, M. (2021). Modelling the construction and long-term response of Göta Tunnel. *Computers and Geotechnics*, *134*, 104027. https://doi.org/10.1016/j.compgeo.2021.104027
Paper B	Tornborg, J., Karlsson, M., & Karstunen, M. (2023). Permanent Sheet Pile Wall in Soft Sensitive Clay. *Journal of Geotechnical and Geoenvironmental Engineering*, *149*(6), 1–13. https://doi.org/10.1061/JGGEFK.GTENG-10955
Paper C	Tornborg, J., Karlsson, M., & Dijkstra, J. (2024a). Temporal effective stress response of soil elements below the base of an excavation in sensitive clay. *Canadian Geotechnical Journal*. https://doi.org/10.1139/cgj-2023-0355
Paper D	Tornborg, J., Karlsson, M., Dijkstra, J., & Karstunen, M. (2024b). On the development of effective heave pressure in deep excavations. *Manuscript.*

The author has carried the main responsibility for the work presented in the appended papers. For all papers this work included developing the methodology, formal analyses, visualisation and writing the original drafts. Co-authors have contributed with conceptualisation, supervision, reviewing and editing. For Papers B and C, the work also comprised the planning and management of the additional site investigations and conducting the subsequent laboratory tests. In Paper C the author's contribution also included the laboratory verification of sensors and the installation of instrument clusters in the field, in addition to the measurements of the vertical displacements (bellow-hose extensometer), sequential cutting of piezometer and bellow-hose pipes, excavating around the equipment, rerouting cables, etc. at the active construction site. The author was involved in formulating and writing the research proposals for the projects that financed the research.

OTHER RELATED PUBLICATIONS

- Tornborg, J., Karlsson, M., & Karstunen, M. (2019). Benchmarking of a contemporary soil model for simulation of deep excavations in soft clay. In H. Sigursteinsson, S. Erlingsson, & B. Bessason (Eds.), *17th European Conference on Soil Mechanics and Geotechnical Engineering* (pp. 721–728). ISSMGE. https://doi.org/10.32075/17ECSMGE-2019-0721
- Tornborg, J., Karlsson, M., Kullingsjö, A., & Karstunen, M. (2021). Experience from short- and long-term performance of deep excavations in soft sensitive clays. *IOP Conference Series: Earth and Environmental Science*, *710*(1), 012053. https://doi.org/10.1088/1755-1315/710/1/012053
- Tornborg, J., Dijkstra, J., & Karlsson, M. (2022). Monitoring the long-term hydro-mechanical response below an excavation bottom in sensitive clay. A. M. Ridley (Ed.), *11th International Symposium on Field Monitoring in Geomechanics* (pp. 1–7). ISSMGE

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NOTATION

Acronyms

CRS	Constant Rate of Strain
CSL	Critical State Line
DA	Dimensional Analysis
DG	Dimensionless Group
DSS	Direct Simple Shear test
EHP	Effective Heave Pressure
FEM	Finite Element Method
IL	Incremental Loading
NCS	Normal Compression Surface
OCR	Overconsolidation ratio
SPW	Sheet Pile Wall

Greek symbols

α_0	initial amount of anisotropy
γ,γ'	unit weight, submerged unit weight
$\dot{\varepsilon}_{\mathrm{std}}$	standard strain rate in laboratory test
ε_{a}	axial strain
η	stress ratio
κ	swelling index
κ^*	modified swelling index
λ	compression index
λ^*	modified compression index
λ_{i}	intrinsic compression index
λ_{i}^*	modified intrinsic compression index
μ^*	modified creep index
μ_{i}^*	modified intrinsic creep index
ν'	Poisson's ratio
σ'_{h}	horizontal effective stress
σ'_{v}	vertical effective stress
σ'_{h0}	initial horizontal effective stress
σ'_{v0}	initial vertical effective stress
σ'_{vc}	vertical preconsolidation pressure
$\sigma'_{\mathrm{vc,std}}$	vertical preconsolidation pressure determined at $\dot{\varepsilon}_{\mathrm{std}}$
ϕ'	friction angle
ϕ'_{CSL}	critical state friction angle
ϕ'_{peak}	peak friction angle
χ_0	initial amount of bonding
ω	rate of rotational hardening
ω_{d}	relative rate of rotational hardening due to deviatoric strain

Roman lower case letters

a	Rate of destructuration
b	Relative rate of destructuration due to deviatoric strain
c_{u}	undrained shear strength
c_{h}	horizontal coefficient of consolidation
c_{v}	vertical coefficient of consolidation
e_0	initial void ratio
k	permeability (hydraulic conductivity)
m	exponent
p'	mean effective stress
p'_0	initial mean effective stress
q	deviatoric stress
q_0	initial deviatoric stress
t	time
t_{p}	time at end of primary consolidation
u	pore pressure
u_{e}	excess pore pressure
z	depth

Roman capital letters

C_α	secondary compression index
C_{c}	compression index
D	drainage length
E_{oed}	oedometer stiffness
K	coefficient of earth pressure
K_0	coefficient of earth pressure at rest
K_0^{nc}	coefficient of earth pressure at rest for normally consolidated state
M	bending moment
M_{central}	central bending moment of slab
M_{wall}	bending moment of slab at location of wall
M_{c}	stress ratio at critical state in triaxial compression
M_{e}	stress ratio at critical state in triaxial extension
N	stability number
N_{cb}	critical stability number with respect to bottom heave
R	non-dimensional relative stiffness ratio
T	normalised time (time factor)
U	average degree of consolidation

Part I
Extended summary

1 Introduction

The research presented in this thesis investigates the temporal evolution of earth pressures in deep excavations in soft clay considering the response during the construction and the subsequent serviceability stage. The project was initiated by a lack of design guidelines and monitoring data elucidating the evolution of earth pressures below underground structures in soft clays.

1.1 Background

The percentage of the world's population that live in urban areas is expected to increase from 55% in 2018 to 68% in 2050 (UN 2019). The United Nations sustainable development goal 11 (UN 2023) specifically addresses the need for sustainable transport systems, adaptation to climate change and resilience to disasters. Utilisation of underground space may hence prove essential in order to include infrastructure such as, e.g., railway transportation and storm water retention systems in existing city landscapes (Broere 2016) as well as to facilitate energy storage solutions (e.g. Zamani et al. 2023).

The design of underground facilities requires accurate predictions of actions, such as earth pressures and displacements, during the construction as well as the serviceability stage. Accurate predictions are crucial in assuring appropriate levels of safety, to aid optimisation of constructions materials, minimise maintenance costs and environmental effects, and assist in the development of new solutions. Accurate predictions are also essential for construction using the Observational Method (Peck 1969a). Hence, the contribution from the geotechnical engineering profession is instrumental in supporting the transition to sustainable development (Lacasse et al. 2002). However, the response of an underground structure during the construction and serviceability stage is a complex soil-structure interaction problem (Potts and Zdravkovic 1999a; Muir Wood 2004; Kempfert and Gebreselassie 2006).

Due to the complexity of projects involving underground structures, geotechnical engineers have historically developed semi-empirical design methods for embedded retaining structures based on theoretical models and field observations. Examples include Bjerrum and Eide (1956) for base stability, Peck (1969b) for apparent earth pressure charts for loads in support systems, and Clough and O'Rourke (1990) for movements of retaining walls. Such methods are useful, but limited with respect to e.g. site conditions and the construction method. As urbanisation drives the need for underground space, the project settings may not be contained within the limitations of existing design methods or current practice. Furthermore, semi-empirical design methods in general have evolved from observations during the construction stage, while observations of the long-term (serviceability) performance of underground structures are scarce.

Numerical modelling may aid in exploring the behaviour of complex systems and develop new design tools (Randolph 2013). It also provides a means of simulating the hydro-mechanical soil response and soil-structure interaction during the construction and serviceability stages, and thus consider the entire service life of a structure. However, to do so accurately, the soil models adopted must incorporate relevant features of the soil behaviour. In the case of soft sensitive clays such features include anisotropy, destructuration and rate-dependency. In modelling the service life of underground structures, the latter feature allows to simulate and consider the effect of time-dependent processes such as e.g. creep and relaxation, in the soil adjacent to a structural element, in addition to generation and dissipation of excess pore pressures. In the context of numerical modelling of excavations and underground structures in soft soils, numerous researchers have made contributions to increase the knowledge of the field. A number of examples include studies of idealised deep excavation systems (e.g. Hashash and Whittle 1996; Karlsrud and Andresen 2005; Zdravkovic et al. 2005; Finno et al. 2007; Scharinger et al. 2009; Bertoldo and Callisto 2019; Schweiger and Tschuchnigg 2021) as well as case studies (e.g. Schweiger et al. 2009; Whittle et al. 2015; Dong et al. 2016; Rouainia et al. 2017). The results of numerical or physical modelling can be used to generalise system behaviour, and preferably be presented into appropriate non-dimensional parameter groups (e.g. Rowe 1952; Janbu 1954; Brown 1969; Mana and Clough 1981; Ukritchon et al. 2003; Mair 2008; Bryson and Zapata-Medina 2012). Generalised results have the potential to be applicable for a wide range of settings and still contain the complexity of the system that they aim to replicate.

The actions predicted by models need to be accurate, and ideally validated against observations of the system performance from the construction and serviceability stages. Such actions include the lateral earth pressures acting on temporary retaining structures during the construction stage (reported by e.g. Peck 1969b; Bjerrum et al. 1972; Clough and Reed 1984; Finno et al. 1989; Ng 1999; Kullingsjö 2007; Ng et al. 2012; Tan and Wang 2013) and on permanent structures during the serviceability stage (e.g. Carder and Darley 1998; Carder et al. 1999; Richards et al. 2007; Tan and Paikowsky 2008; O'Leary et al. 2016). In comparison, data on monitoring of earth pressures beneath the base of excavations and structures (e.g. Price and Wardle 1986; Katzenbach et al. 2000) are scarce, even more so in soft clays (Jendeby 1986). Simpson (2018) pointed out both the lack and the value of such observations. In contrast, monitoring of heave during the construction stage has been more frequently reported in the literature (e.g. Magnusson 1975; Liu et al. 2011; Simpson and Vardanega 2014; Whittle et al. 2015). Reports of long-term heave are, however, comparatively scarce for obvious reasons (i.e., construction within excavation). Yet, such reports provide a clear manifestation of the hydro-mechanical temporal response of soils with a low hydraulic conductivity to unloading (e.g. Burland and Karla 1986; Symons and Tedd 1989; Nash et al. 1996; Chan et al. 2018).

The effect of restraining the heave process, e.g. with a basement or tunnel slab, is studied less compared to the "free" heave (Chan and Madabhushi 2017; Tornborg 2017; Simpson 2018). The heave pressure, i.e., the action against an underground slab imposed by the soil due to restrained heave, due to the temporal hydro-mechanical response evolves with the consolidation process, and due to additional time-dependent processes such as creep and relaxation. Furthermore, the magnitude of the earth pressure acting on a soil-structure interface will depend on the relative stiffness of the soil and the structure. Additionally, the soil stiffness depends on the effective stresses that are evolving in time as function of the hydro-mechanical processes and additional time-dependent processes such as creep and relaxation. The type of restrictions provided by the structural elements further affect the response. Therefore, the design of underground structures, and the consideration of actions arising due to restrained heave, still remains challenging and requires further investigation.

This project was initiated by a lack of guidelines and monitoring data targeting heave pressure in soft clays. The hypothesis is that a heave pressure will form if a structural element is placed, such that it to some degree will restrain or "lock-in" the delayed heave caused by unloading. This hypothesis is based on observations and studies by Jendeby (1986), Price and Wardle (1986), Simpson (2018) and previous project-specific analyses in the design phase of Göta Tunnel in Gothenburg (Alén and Sällfors 2001).

An additional motivation for initiating this research project, was that the constitutive model Creep-SClay1S (Sivasithamparam et al. 2015; Gras et al. 2018) developed for simulating the behaviour of soft sensitive clays, had yet to be benchmarked for modelling of excavation problems. Furthermore, recent developments in InSAR remote observations provided the means to assess the long-term performance of some existing underground structures that had been extensively monitored and documented during the construction stage only. Here thus lay the basis for three research objectives, presented in the next section.

Since the project was initiated in 2017, physical model tests investigating heave pressure have been presented by Chan et al. (2022b). However, these model tests were limited to overconsolidated kaolin clay, a groundwater table located at the excavation base, and one construction time setting. This thesis seeks to generalise the development of earth pressures below underground structures for the case of natural soft clays, and takes a holistic view of the hydro-mechanical, time-dependent, response during the construction and serviceability stages.

1.2 Aim and objectives

The aim of this thesis is to investigate the temporal evolution of earth pressures acting on underground structures in deep excavations in soft clay by means of numerical analyses and field monitoring. The numerical analyses complement the field monitoring results by enabling to explore, generalise and quantify how factors such as e.g. construction time, geometry, ground profile and relative stiffness between soil and structure, affect the magnitude and evolution of earth pressures against underground structures.

The ultimate goal is to develop non-dimensional earth pressure charts that consider the hydro-mechanical soil response and soil-structure interaction during the construction and serviceability stages.

The main objectives are listed below:

(i) Identify well-documented and characterised case studies of deep excavations in soft clay and benchmark a constitutive soil model that accounts for rate-dependency (the Creep-SClay1S model) against the system response at field scale (**Papers A and B**).

(ii) Investigate the hydro-mechanical response of soil elements below the base of an underground structure in soft clay (**Paper C**). This involves identifying an appropriate site, performing site characterisation, designing and executing a field monitoring program including subsequent analyses of data.

(iii) Generalise the evolution of excavation-induced earth pressures below the base of underground structures in soft clay by performing a parametric study, using the Finite Element Method, and synthesise the results into charts containing appropriate non-dimensional parameter groups (**Paper D**).

The main motivation for each objective is:

(i) Assessing the applicability of the Creep-SClay1S model for project-specific analyses of deep excavations and underground structures and the generalisation study (objective iii);

(ii) Providing data missing in the literature on the hydro-mechanical response at excavation bases and below underground structures in situ, and;

(iii) Developing insights on the system behaviour, and the factors that influence the evolution of earth pressures due to restrained heave, with results presented in charts that can be readily implemented as tools for application in industry.

1.3 Limitations

The following limitations apply for this thesis.

(i) Considering rate-dependent constitutive soil models, this study is limited to the Creep-SClay1S model.

(ii) The effect of small-strain stiffness is not considered. However, the effect of varying initial soil stiffness and its evolution (with stress level) is studied in Paper D by dimensionless parameter groups relating to consolidation and relative soil-structure stiffness.

(iii) Installation effects of piles and other structural elements, or adjacent existing structures, are not included in the study of the generalised response (Paper D).

(iv) The clays studied and modelled in this thesis are assumed to be fully saturated. Furthermore, neither expansive clays nor thermal effects in the structure and/or the soil are considered.

1.4 Outline

This thesis consists of two parts: Part I - Extended summary and Part II - Appended papers.

The outline of Part I is presented below:

- Chapter 2 *Excavations and soft clay*. Literature review covering: a brief overview of the development of semi-empirical design tools, previous studies reporting on monitoring of earth pressures, key features of soft clay behaviour, and previous studies that have modelled excavations in soft clays by means of the Finite Element Method, including the soil models used.
- Chapter 3 *Methodology*. A description of the methodology of this research project.
- Chapter 4 *Summary of the appended papers*. A brief summary of each of the four appended papers including key results and conclusions.
- Chapter 5 *Conclusions and recommendations*.

2 Excavations and soft clay

2.1 Retaining structures and observations of earth pressures

The Introduction outlined excavations and underground structures as complex problems. This is illustrated in Figure 2.1, where a number of factors that may influence the response of excavations and the resulting actions on underground structures are summarised.

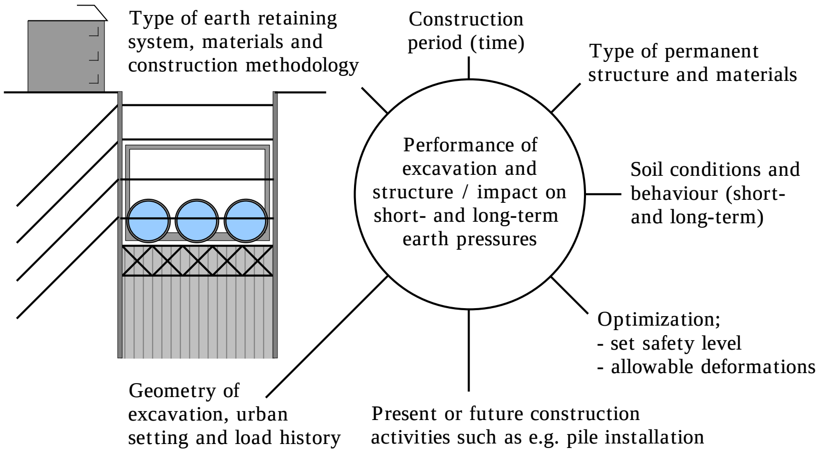

Figure 2.1: *Factors that may influence the response of excavations and actions on underground structures. Inspired by Gebreselassie (2003).*

To tackle the challenges associated with the design of excavations and underground structures, geotechnical engineers have historically developed semi-empirical design methods (here referred to as the use of empirical data, i.e. field or model observations, to modify and improve underlying theoretical models). The following paragraph provides a brief description on the development of some pioneering semi-empirical design methods relating to deep excavations.

Methods to approximate the limiting earth pressures acting on retaining structures were described in the eighteenth and nineteenth century by Coulomb and Rankine, respectively. However, as pointed out by Terzaghi (1936), the computation of the limiting earth pressures does not take into consideration the stress-strain behaviour of the soil. To cope with this limitation, Peck (1943) and Terzaghi and Peck (1967) synthesised field measurements of strut loads into apparent pressure diagrams for the estimation of maximum loads in bracing systems. Peck (1969b) and his student Flaate (1966) further refined the apparent pressure diagrams, based on observations e.g. from the construction of the Oslo subway. By considering the overall stability of the excavation by means of a stability number, $N{=}\gamma H/c_u$, a degree of mobilisation of the soil around the excavation was taken into account that today can be accounted for by means of e.g. finite element analyses. Contributions on the mechanisms that affect retaining wall design were also made by Rowe (1952), using physical model tests to generalise the influence of wall flexibility and sand arching on retaining wall bending moment. These examples of early semi-empirical design methods relate to the lateral earth pressure on retaining walls and support systems. Concerning basal stability in soft clays, Bjerrum and Eide (1956) concluded that the critical stability number (for which bottom heave failure will occur), N_{cb}[1], proposed by Terzaghi (1943) was non-conservative. On the basis of observations from 14 deep excavations, Bjerrum and Eide (1956) instead proposed N_{cb} values based on the work by Skempton (1951), by means of considering basal stability as a reversed bearing capacity problem. Practising engineers to this day still make use of these methods, although, in the spirit of how the methods originated, in slightly refined form (e.g. Karlsrud and Andresen 2008; Gaba et al. 2017; Fredriksson et al. 2018). Geotechnical design of retaining structures also involves estimation of displacements. Databases and methods for estimation of retaining wall deflections and ground movements are presented in e.g. Peck (1969b), Clough and O'Rourke (1990), Hsieh and Ou (1998), Long (2001), Moormann (2004), Wang et al. (2010), and Hung and Phienwej (2016).

The previously described semi-empirical design method of apparent pressure diagrams was developed mainly based on measurements of loads in internal bracing systems (Terzaghi et al. 1996), i.e., not direct monitoring of earth pressures. Studies that report monitoring of earth pressure on retaining structures in clay are presented in Table 2.1. A focus when writing Table 2.1 was to include reports that involve monitoring of earth pressures spanning both the construction and serviceability stages. Such case records, however, appear to be scarce compared to reports from the construction stage only, especially in soft clay settings. Therefore, some well documented cases specifically covering monitoring of horizontal earth pressure in soft clays, although on retaining structures and in the construction stage only, have been included in Table 2.1, given an immense value in well-documented case records. These offer insights into the expected response at a given site and can be used, in addition to laboratory tests, e.g. to calibrate or validate numerical models before conducting the analyses for a planned project.

From Table 2.1 it is clear that observations of the long-term evolution of earth pressures in soft clays are underrepresented compared to observations in stiff overconsolidated clays. It is also clear that observations of earth pressures below excavation bases and underground structures are scarce, and very scarce in soft clay settings. Specifically, the monitoring of vertical stresses below underground structures in clays historically have targeted the load sharing (total stresses) in raft-pile foundations (Jendeby 1986; Price and Wardle 1986; Katzenbach et al. 2000), i.e., not targeting the coupled hydro-mechanical response. Observations of earth pressures in excavations and below underground slabs, and their evolution as a result of restrained heave, were considered valuable by Simpson (2018), and it is only recently that such observations have been made in physical model tests (Chan et al. 2022b).

[1] subscript $_b$ added here as in Peck (1969b), since referring to a failure mechanism involving the soil below excavation level.

Table 2.1: Examples of reports on monitoring of earth pressures on underground structures in clay.

Reference	Notes
Flaate (1966)	Oslo soft clay. Horizontal stresses. Steel Sheet Pile Wall (SPW). Construction stage.
DiBiagio and Roti (1972)	Oslo soft clay. Horizontal stresses. Diaphragm wall (D-wall). Covering the construction stage plus 200 days after slab was cast.
Karlsrud and Myrvoll (1976)	Oslo soft clay. Horizontal stresses. Steel SPW. Construction stage.
Price and Wardle (1986)	London clay. Vertical stresses. Four years of monitoring. Pile-raft load sharing.
Carder and Symons (1989)	London clay. Horizontal stresses. Cantilever wall. Instrumented 12 years after construction.
Symons and Tedd (1989)	London clay. Horizontal stresses. Secant pile wall. Construction stage and four years after.
Carder and Darley (1998)	UK clays. Report covering 12 sites, various wall types. Horizontal stresses, including vertical stresses at one site (Aldershot).
Hansbo and Jendeby (1998)	Gothenburg soft clay. Vertical stresses. 14 years after construction. Pile-raft load sharing.
Richards and Powrie (1998)	Centrifuge tests. Prop loads and bending moment. 11 years (prototype) after excavation.
Ng (1998) and Ng (1999)	Cambridge stiff (Gault) clay. Horizontal stresses. D-wall. Construction stage including two years after slab cast. Interpreted stress paths.
Carder et al. (1999)	London clay. Horizontal stresses. Stabilising base slab. Instrumented 17 years after construction.
Katzenbach et al. (2000)	Frankfurt (overconsolidated) stiff clay. Vertical stresses. Covering several buildings. Pile-raft load sharing.
Liu et al. (2005)	Shanghai soft clay. Horizontal stresses. D-wall. Construction stage.
Kullingsjö (2007)	Gothenburg soft clay. Horizontal stresses. SPW. Construction stage.
Richards et al. (2007)	UK (Ashford) stiff clay. Horizontal stresses. Bored pile wall. Covering six years after construction.
Ng et al. (2012)	Shanghai medium-soft clay. Horizontal stresses. D-wall. Construction stage.
Tan and Wang (2013)	Shanghai soft clay. Horizontal stresses. D-wall. Construction stage.
Chan et al. (2022b)	Centrifuge tests. Vertical stresses. Studying heave pressure. Overconsolidated kaolin clay. Four years (prototype) after excavation.
Nejjar et al. (2023)	Paris, partly clay (overconsolidated). Horizontal stresses. D-wall. Construction stage.

2.2 Characteristic features and behaviour of soft clay

This section provides a brief introduction to the deposition of Swedish clays and a review of the lateral earth pressure at rest, followed by a review of characteristic features of soft sensitive clays. The section also includes a brief review of factors affecting the formation of excess pore pressure during unloading and the subsequent consolidation process. Lastly, the section reviews effective heave pressures due to restrained heave.

2.2.1 Soil deposition and lateral earth pressure at rest

The soft clays encountered in Sweden started to deposit as the glacial ice retreated during the last Ice Age, i.e. on the West coast of Sweden (Gothenburg region) approximately 13 000 years ago (Fredén et al. 1981) and on the East coast (Stockholm and Uppsala region) approximately 10 000 years ago (Lundin 1991; Fréden 2002). Examples of the major cities in Sweden that are located partly on soft clay deposits are e.g. Stockholm, Gothenburg, Uppsala and Norrköping. Contributions that characterise the features of these clay deposits include e.g. Larsson (1977), Larsson (1986), Westerberg (1999), Olsson (2013), and Wood (2016).

For a truly normally consolidated soil, the coefficient of lateral earth pressure at rest is commonly estimated using the formula by Jaky (1948):

$$K_0^{nc} = 1 - \sin\phi' \tag{2.1}$$

where K_0^{nc} is the coefficient of earth pressure at rest for normally consolidated soil and ϕ' is friction angle.

Comprehensive worldwide databases including K_0^{nc} for cohesive soils have been reported by e.g. Mayne and Kulhawy (1982) and Watabe et al. (2003). Watabe et al. (2003) refined the expression with respect to the friction angle as follows:

$$K_0^{nc} = 0.95 - \sin\phi'_{\mathrm{peak}} \tag{2.2a}$$

$$K_0^{nc} = 1.05 - \sin\phi'_{\mathrm{CSL}} \tag{2.2b}$$

where ϕ'_{peak} and ϕ'_{CSL} correspond to the peak and critical state friction angles, respectively, derived from triaxial compression tests.

In line with the results of these databases, laboratory tests on Swedish soft clays report K_0^{nc} in the range 0.50-0.55 (Sällfors 1975; Kullingsjö 2007; Olsson 2013). However, in natural clays, the process of ageing continuously affects the apparent preconsolidation pressure (Bjerrum 1967). The effect of ageing on the horizontal effective stress was posed as an open question by Schmertmann (1983), i.e., will the horizontal effective stress increase, remain constant or decrease during secondary compression of normally consolidated clays. Schmertmann (2012) concluded based on extensive evidence that it increases, although (similarly to the apparent vertical preconsolidation due to ageing), at a diminishing rate with time.

Upon one-dimensional unloading, several authors have investigated the ratio of horizontal to vertical effective stresses. As the horizontal effective stresses are locked-in to a greater extent then the vertical, K increases upon unloading (Brooker and Ireland 1965) and the following relationship was proposed by Mayne and Kulhawy (1982) based on a world-wide database:

$$K = K_0^{nc} OCR^m \tag{2.3}$$

with OCR representing the apparent overconsolidation ratio and m=sinϕ'.

The exponent m has been estimated by:

- Kulhawy and Mayne (1990) with m=sinϕ'_{tc} (subscript for triaxial compression) with ϕ'_{tc}=30° as a best fit
- Watabe et al. (2003) with m=sinϕ'_{CSL}-0.05, or alternatively, m=sinϕ'_{peak}+0.05
- Kullingsjö (2007) with m=0.6 for soft Gothenburg clay

If the OCR resulting from secondary compression is assumed to have the same effect on K as that due to unloading (approach discussed e.g. by Mesri and Castro 1987), it is possible to estimate the increase in K for natural aged clays using Equation 2.3. The increase in OCR due to secondary compression can be estimated with the following equation (Mesri and Hayat 1993):

$$OCR = (t/t_{\mathrm{p}})^{C_\alpha/C_{\mathrm{c}}} \tag{2.4}$$

where t is time, t_{p} is the time at end of primary consolidation, C_{c} is the compression index and C_α the secondary compression index. An estimate of t_{p} for clay layers in the field is challenging. Equation 2.3 in combination with Equation 2.4 can, however, be used to estimate the magnitude of the increase in K with time. In a similar study, Degago and Grimstad (2016) suggested a range for the exponent in Equation 2.4 to 0.02–0.07, this is also the range reported by Olsson (2013) for Gothenburg clay. Using C_α/C_{c}=0.07, K_0^{nc}=0.5 and m=0.6 result in K increasing with in average 0.055 over each log-cycle of time from t_{p}=1 year to t=10 000 years (t_{p}=100 years result in 0.050), the corresponding increase for C_α/C_{c}=0.02 is 0.015 and 0.014.

The increase in OCR and K_0 based on Equations. 2.3 and 2.4 only considers the effect of secondary compression, whereas other factors e.g. changes in bonding, may also affect the values (Mesri and Castro 1987). Figure 2.2 presents compiled data from field measurements of in situ K_0 in a natural aged soft clay deposit just north of Gothenburg (Bäckebol), Sweden, including a comparison to K_0 estimates based on Equation 2.3. Based on this figure, Equation 2.3 with K_0^{nc}=0.53, OCR based on preconsolidation pressure determined by means of CRS-tests (Sällfors 1975), and m=0.6, reproduce approximate values for K_0 measured in situ. The underprediction of Equation 2.3 compared to the measurement results above 4 m depth may arise from factors other than secondary compression contributing to an increase of K_0 in situ. As described in the next section, the in situ at rest stress state is of importance as it contributes to the induced anisotropy in the soil. Further references to K_0 in soft clays can be found in e.g. Watabe et al. (2003) and L'Heureux et al. (2017) and for stiff overconsolidated London clay e.g. Hight et al. (2003).

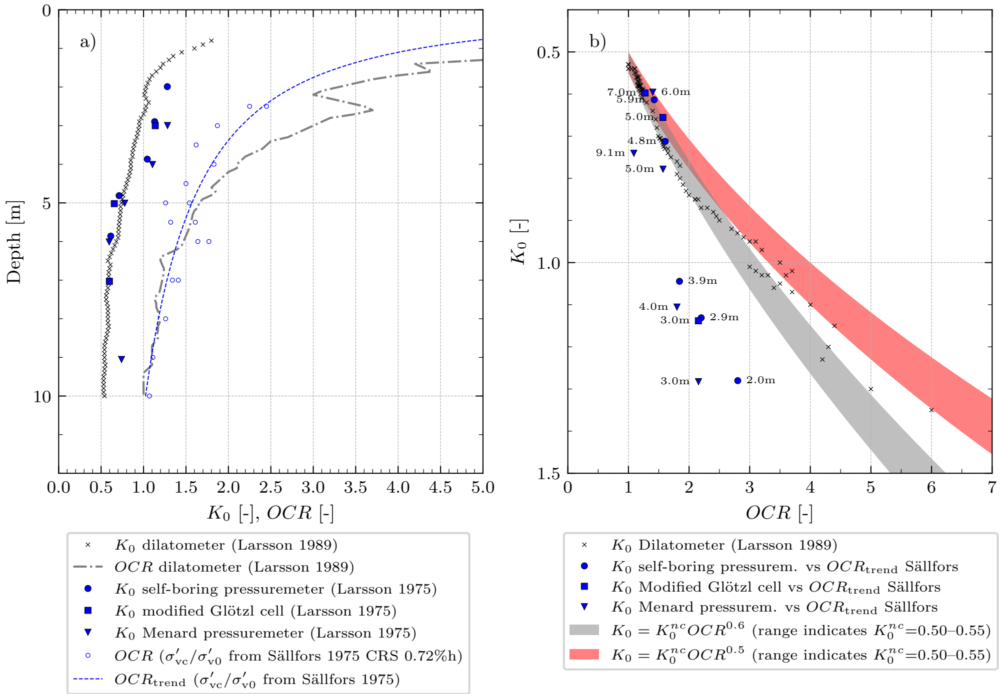

Figure 2.2: *a) K_0 and OCR in Swedish soft natural aged clay (data from Larsson 1975; Sällfors 1975; Larsson and Eskilsson 1989) compared to K_0 estimated using Equation 2.3.*

2.2.2 Anisotropy

A typical feature of natural soft clays is that they exhibit anisotropic shear strength (Ladd 1965; Lo and Morin 1972; Bjerrum 1973) and, more generally, anisotropy in terms of yielding (Wong and Mitchell 1975; Berre 1976; Larsson 1977; Tavenas and Leroueil 1977; Graham et al. 1983; Callisto and Calabresi 1998; Karstunen and Yin 2010; Olsson 2013). The origin of anisotropy in clays can be attributed to i) inherent anisotropy due to soil fabric (shape of particles and particle orientation), as well as macrostructure such as layering in varved clays, and ii) initial shear stress induced anisotropy (Ladd et al. 1977). Additionally, anisotropy may also evolve due to irrecoverable strains during, e.g., staged construction (Ladd 1991). The importance of considering anisotropic response in unloading/excavations as well as loading/embankments was demonstrated by e.g. Aas (1984) and Zdravkovic et al. (2002), respectively.

Papers A and B include laboratory tests exemplifying anisotropic shear strength of Swedish clays. Anisotropic undrained shear strength is often estimated using local empirical relations. The ones for Swedish clays (e.g. Larsson 1980; Larsson et al. 2007) are based on the concept presented by Ladd and Foott (1974). Additional anisotropic features of soils include e.g. critical state friction angles (Kulhawy and Mayne 1990), stiffness (Graham and Houlsby 1983; Grammatikopoulou et al. 2014; Amorosi et al. 2021), and permeability (Leroueil et al. 1990).

2.2.3 Structure

Soil structure is referred to as the combined effect of the particle arrangement and distribution (soil "fabric") and the interparticle forces (bonding) between particles (Lambe and Whitman 1969; Burland 1990). Leroueil and Vaughan (1990) expanded on the model by Bjerrum (1967) on ageing due to secondary compression (affecting particle arrangement) and included the ageing effects due to bonding. This is illustrated in Figure 2.3.

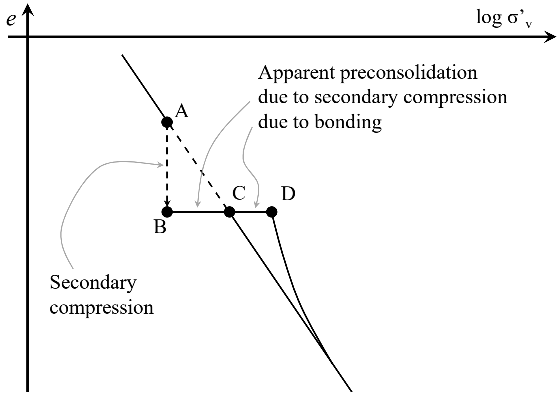

Figure 2.3: *Illustration of the effect of structure on apparent preconsolidation pressure. After Leroueil and Vaughan (1990).*

The term "intrinsic" was introduced by Burland (1990) to describe the properties of reconstituted clays. The intrinsic properties is typically characterised by an intrinsic compression line in oedometer tests and by an intrinsic yield surface in triaxial stress space (Gens and Nova 1993). Leroueil (2001) noted that compression, shearing or unloading can cause destructuration. The observed heave due to unloading may hence be comprised of i) primary heave, due to change in the effective stresses, and potentially, ii) secondary heave, characterised by a secondary swelling index (Mesri et al. 1978) due to destructuration arising in case the bonds are not sufficiently strong to withstand the reduced effective stresses. The occurrence of heave due to change in effective stress and destructuration, in relation to the viscous behaviour of clay was discussed by Vergote et al. (2022). An example of the relevance of considering destructuration in excavations that remain open was presented by Bertoldo and Callisto (2019).

2.2.4 Rate-dependency

The viscous behaviour of clay, resulting e.g. in deformations additional to those purely due to hydrodynamic processes, was acknowledged in the works by Buisman (1936), Šuklje (1957), and Bjerrum (1967). The impact of this characteristic feature on the apparent yield surface that emerge from laboratory tests conducted at various strain rates, was reported by e.g. Lo and Morin (1972), Tavenas and Leroueil (1977), and Graham et al. (1983).

Given the rate-dependency of the yield surface, the preconsolidation pressure and the emerging undrained shear strength will also be rate-dependent. A comprehensive collection of data summarising the influence of strain-rate on the preconsolidation of clays world-wide was presented by Watabe and Leroueil (2015).

The influence of the strain-rate on the undrained shear strength in triaxial testing was reported by e.g. Lo and Morin (1972), Bjerrum (1973), Tavenas and Leroueil (1977), Graham et al. (1983), Lunne et al. (1997), and Mayne (2006). In summary, these studies report a 10–20% change in peak strengths for a tenfold change in strain-rate. However, for laboratory testing procedures producing undrained or partially drained response, the observed rate-dependency may be affected by hydrodynamic processes (e.g. uncertainty relating to the pore pressure distribution within the sample) as pointed out by Muir Wood (2015). This was observed in interface testing of clays by Martinez and Stutz (2019) who found that the rate of shearing and the surface roughness affected the mobilised drainage conditions and hence the interface strength. Such findings highlight the interplay of rate-effects and consolidation, which is exemplified for an unloading situation in subsection 2.2.6.

At sufficiently low loading rates that allow for the dissipation of excess pore pressures, rate-dependency manifests as sustained deformation, so called secondary compression, without change in effective stress (i.e. "pure creep"). The concept of secondary compression and ageing was illustrated by Bjerrum (1967) for oedometric conditions. However, under conditions other than oedometric, such as e.g. those in a triaxial test, a soil under constant load may exhibit so called tertiary creep (accelerating creep) potentially leading to failure (e.g. Saito and Uezawa 1961).

The inverse phenomenon of creep is stress relaxation, which is characterised by decrease in stress over time at a fixed displacement boundary (e.g. Murayama and Shibata 1961; Lacerda and Houston 1973; Lacerda 1977). An example of an undrained triaxial compression test that includes relaxation stops is included in Section 3.3.

This review of rate-dependency highlights three motivations for accounting for rate-dependency in modelling excavations and underground structures in areas with soft clay, i.e., mapping strain-rate in laboratory tests to that expected in the field, accounting for viscoplastic shear strains possibly leading to failure due to tertiary creep, and accounting for background creep settlements and relaxation in simulations of the long-term response of underground structures.

2.2.5 Pore pressure change, consolidation and heave

As a soil is unloaded, a portion of the change in the total stress will result in an immediate change in pore pressure, Δu. This response governs the portion of the deformation that will be instant, and respectively, associated with the dissipation of excess pore pressures. Skempton (1954) introduced the concept of pore pressure coefficients, relating change in total stress to change in pore pressure under undrained triaxial conditions according to:

$$\Delta u = B\left[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)\right] \tag{2.5}$$

Parameters A and B are pore pressure coefficients, with B=1 representing fully saturated soil, and A depending on soil type, OCR and the imposed stress level (Bishop and Bjerrum 1960).

Larsson (1977) found A ranging from 0.75-0.81 in triaxial tests on Swedish soft clays. Analogues to pore pressure parameters are so called influence factors (Janbu et al. 1956; Christian and David Carrier III 1978) developed to estimate the portion of settlement that will occur instantaneously upon loading of foundations on clay. These factors depend on a number of variables such as, e.g., foundation geometry in relation to the depth of the clay layer and the embedment depth, as well as soil properties such as Poisson's ratio (Mayne and Poulos 1999). For excavations, the influence of the excavation geometry and retaining wall type on the excess pore pressures were studied by Callisto et al. (2014) and Bertoldo and Callisto (2016) using finite element analyses.

When it comes to excavations, the dissipation of the stabilising (negative) excess pore pressures with time affects the response during the construction stage, and possibly also the response in the serviceability stage. According to Terzaghi's theory of one-dimensional consolidation, the differential equation for the excess pore pressure dissipation is:

$$\frac{\partial u_e}{\partial t} = c_v \frac{\partial^2 u_e}{\partial z^2} \tag{2.6}$$

where u_e is excess pore pressure, t is time, z is depth and c_v is the vertical coefficient of consolidation:

$$c_v = \frac{k E_{oed}}{\gamma_w} \tag{2.7}$$

where k is permeability (hydraulic conductivity), E_{oed} is oedometer modulus and γ_w the unit weight of water. The consolidation process can be described by the average degree of consolidation U (which is 0 at loading and 1 after full consolidation) and the dimensionless time factor, T_v:

$$T_v = c_v t / D^2 \tag{2.8}$$

where t is time and D the length of the drainage path. A solution to Equation 2.6 provides an expression for U as a function of T (Muir Wood 2009):

$$U = 1 - \frac{8}{\pi^2} \sum_{m=0}^{\infty} \left\{ \frac{1}{(2m+1)^2} \exp\left[-\pi^2 (2m+1)^2 \frac{T_v}{4} \right] \right\} \tag{2.9}$$

Literature is abundant with reports covering the consolidation process due to loading and the settlements of clay layers. Reports on consolidation processes after unloading are more scarce, not surprisingly due to construction activities within excavations, but some well-documented examples of delayed heave processes include e.g. Price and Wardle (1986), Symons and Tedd (1989), Nash et al. (1996), and Chan et al. (2018). In Paper D, Equation 2.9 is illustrated in charts relating heave as well as effective heave pressure to the normalised time.

From this review, it is clear that the formation of excess pore pressures as well as the consolidation process depends on many factors (e.g. geometry, soil type and boundary conditions for the studied problem). However, the formation and dissipation of pore pressure with time is of great importance, since these processes affect the evolution of effective stress (affecting the strength and stiffness of the soil) and hence the response during, as well as after, the construction stage.

2.2.6 Idealised hydro-mechanical response

Tavenas and Leroueil (1981) exemplified that a high rate of excavation in low-permeable soils may initially show as an undrained response. Figure 2.4 illustrates in triaxial stress space how rate-dependency and consolidation may manifest during and after unloading. An imaginary effective stress path (ESP) during undrained unloading to various stress levels is illustrated by the red dashed line. Depending on the magnitude and direction of the imposed TSP, and the resulting effective stress response, the dissipation of the negative (stabilising) excess pore pressures or any additional pore pressure that can not fully dissipate, may move the ESP either towards a stable state (points 1–3) or towards the critical state line (point 4).

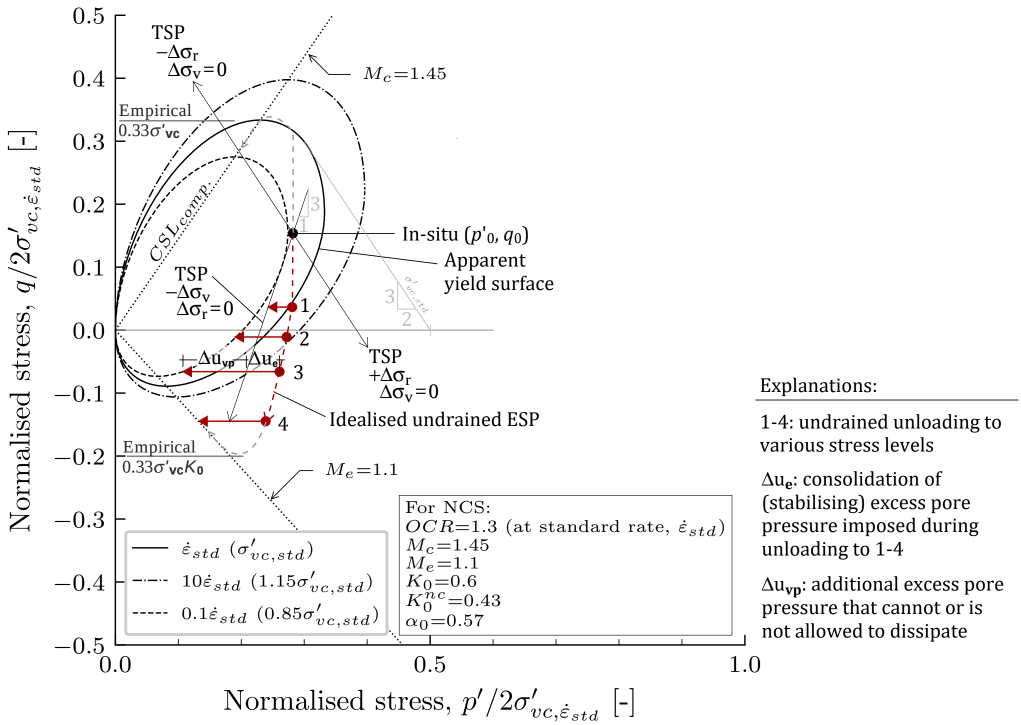

Figure 2.4: *Idealised effect of rate-dependency and dissipation of excess pore pressure after undrained unloading of a lightly overconsolidated soft clay. Inspired by Tavenas and Leroueil (1981) and triaxial tests by Larsson (1977).*

Although Figure 2.4 is a rough idealisation, it illustrates that not considering the rate-dependency of yield, in combination with not considering the dissipation of excess pore pressures (i.e. undrained analyses) is not conservative for excavation problems, as the apparent strength determined at laboratory strain rates as well as the effective stress state, are then assumed to be fixed.

2.2.7 Earth pressure formation due to restrained heave

The processes of consolidation and delayed heave after unloading was reviewed in subsection 2.2.5. If the heave process is restrained, e.g. by the construction of a basement or tunnel slab, an earth pressure will form against the restraining structural element. This pressure is referred to as effective heave pressure (EHP). The magnitude of this pressure will depend on the amount of dissipated heave and evolution of soil stiffness before the restraint is put in place, and the rigidity/stiffness of the restraint, i.e. the ability of the system to "lock-in" the remaining heave.

In monitoring of a deep excavation in London clay, Price and Wardle (1986) reported results from load cells in the raft foundation and in one pile. As the pile load reduced during a period of no construction, uplift pressure beneath the raft was observed in the nearby raft load cell. Burland and Karla (1986) were involved in the design of this structure, and they considered a scenario with EHP after casting the raft corresponding to 70% of the estimated long-term heave. Similarly, in a study of pile-raft foundations in a setting of Swedish soft clay, Jendeby (1986) observed an uplift pressure, attributed to restrained heave, in the early stages of the construction.

The design of Göta Tunnel in Gothenburg is the first case identified by the author were EHP was considered in the design of a major Swedish infrastructure project. The tender documents for Göta Tunnel specified an EHP-ratio corresponding to 80% of the in situ vertical effective stress at the foundation level. This was, however, considered to be an upper-bound and a detailed project-specific study was conducted by the Swedish Geotechnical Institute and Chalmers University of Technology (Alén and Sällfors 2001). The study revised the $0.8\sigma'_{v0}$ factor to $0.25\sigma'_{v0}$–$0.40\sigma'_{v0}$ (depending on the depth of the clay layer beneath the excavation bottom) based on project specific calculations, considering consolidation during an assumed construction time and some uplift of the structure. Unfortunately, no measurements were made in order to compare the design EHPs to the in situ earth pressures in the Göta Tunnel project.

Current design guidelines and practice

This section contains a brief review of design guidelines and practices for consideration of the formation of EHP. Figure 2.5, from the ICE Manual of Geotechnical Engineering Vol. II (Ingram 2012), illustrates the principle that heave pressure dissipates with vertical displacement. To dissipate heave pressure, Ingram (2012) discusses the option to allow for vertical displacement, e.g. by specifying a heave void. The effective stresses may then, however, reduce which results in a reduced capacity of the foundation including the piles. The principle in Figure 2.5 can be refined, as described by Simpson (2018), see Figure 2.6. Line A-A' is based on calculations in which different magnitudes of EHP are imposed in order to establish the relationship between heave and EHP. h_1-h_3 represents the instant heave during excavation, the heave before the slab is cast, and the long-term deflection of slab due to water pressure, respectively. The gradient of line B-C represents the long-term deflection (due to stiffness and restraints) of the slab due to EHP. Point C indicates the EHP. However, Simpson (2018) concluded that the current industry design practice, outlined in Figures 2.5 and 2.6, was flawed as the calculations of the soil and structural response are carried out separately, and rely on superposition, and thus recommended finite element analyses to predict EHP.

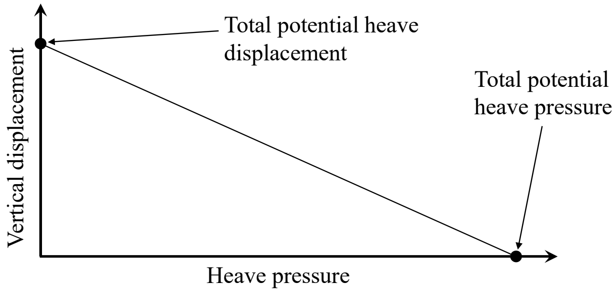

Figure 2.5: *Principle for dissipation of effective heave pressure with vertical displacement (adapted from Ingram 2012).*

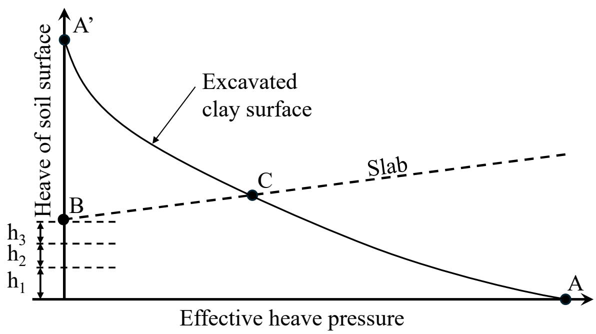

Figure 2.6: *Principle for dissipation of effective heave pressure with vertical displacement (adapted from Simpson 2018).*

In Sweden, no design guidelines or practice such as described by Simpson (2018) exist on how to consider EHP in design. However, in a pre-study to this thesis (Tornborg 2017), an upper-bound for initial estimates of EHP in a setting of Gothenburg clay was suggested taking EHP as $0.5\sigma'_{v0}$.

2.3 Modelling of excavations

As stated in the Introduction, the response of excavations and underground structures during the construction and serviceability stages is a complex soil-structure interaction problem and involves many factors. Due to the level of complexity, numerical modelling has been adopted by many researchers to study the site specific response of excavations, as well as to explore system behaviour by means of parametric studies. Table 2.2 provides a list of constitutive models that have been used to simulate fine grained soils in studies of deep excavations. The models in Table 2.2 were identified from a review of studies that have used the Finite Element Method to study deep excavations in fine grained soils, see Table 2.3. Table 2.3 includes the type of study (real case or idealised), type of retaining wall, studied stage (construction and/or serviceability) and the constitutive soil model(s) used.

Apparently, the majority of the studies listed in Table 2.3 have focused on observations and mechanisms during the construction stage. None of the listed soil models includes rate-dependency. This project, however, aims to investigate the temporal evolution of earth pressures in soft clay. Hence, rate-dependency and the possibility to account for background creep settlements is relevant. The rate-dependent Creep-SClay1S model (Sivasithamparam et al. 2015; Gras et al. 2018) is therefore used to simulate the response of soft sensitive clays in this work. This is a trade-off (compared to e.g. the HSS model) as the Creep-SClay1S model formulation used does not include the aspect of small-strain stiffness. A brief description of the model is provided in the next section.

Table 2.2: Constitutive models used to simulate fine grained soils in the studies of deep excavations listed in Table 2.3.

Model name/description	Reference	Abbreviation
Linear-elastic	-	LE
Mohr-Coulomb	-	MC
Mohr-Coulomb, with small strain stiffness	Potts and Zdravkovic (1999b) with small strain from Jardine et al. (1986)	MCss
Modified Cam Clay,	Roscoe and Burland (1968)	MCC
with small strain stiffness	Kovacevic et al. (2008)	MCCss
Hardening soil	Schanz et al. (1999)	HS
Hardening soil small-strain	Benz (2007)	HSS
Kinematic Hardening Model for Structured soils	Rouainia and Muir Wood (2000)	KHSM
Multilaminate Model for Soils, with small strain stiffness	Scharinger et al. (2009)	MMS MMSsss
MIT-E3	Whittle and Kavvadas (1994)	MIT-E3
MIT-S1	Pestana and Whittle (1999)	MIT-S1
ANISOFT	Andresen and Jostad (2002)	ANISOFT
e-ADP	Grimstad et al. (2006)	e-ADP
NGI-ADP	Grimstad et al. (2012)	NGI-ADP
Undrained Soft Clay	Hsieh et al. (2010)	USC

Table 2.3: Examples of finite element analyses of deep excavations in fine grained soils.

Reference	Type of study	Wall type[a)]	Stage[b)]	Constitutive model(s)[c)]	Modelling type[d)]
Wang and Whittle (2024)	Case study Singapore	DW	C	MC	UTSA
Chan et al. (2022a)	Idealised (centrifuge)	-	C, S	HSS	Consol.
Abdi and Ou (2022)	Case study Taipei & Singapore	DW	C	MC	UTSA
Schweiger and Tschuchnigg (2021)	Idealised	-	C	HSS, MC	UESA, Coupled
Li et al. (2022)	Case study Bangkok	DW	C	NGI-ADP	UTSA
Choosrithong and Schweiger (2020)	Idealised	DW	C	HSS	UESA
Do and Ou (2020)	Idealised	-	C	MC	UTSA
Bertoldo and Callisto (2019)	Idealised	-	C	KHMS	Coupled.
Simpson (2018)	Idealised	-	C, S	LE, MC	Consol.
Kovacevic et al. (2017)	Case study Dublin	Sec.P	C, S	MCCss	Coupled
Rouainia et al. (2017)	Case study Boston	DW, SP	C	MCC, KHM, KHSM	Coupled
Whittle et al. (2015)	Case study Boston	DW	C	MIT-E3	Coupled
Orazalin et al. (2015)	Case study Boston	DW	C	MIT-E3, MC	Coupled, UESA
Hong and Ng (2013)	Idealised (centrifuge)	-	C	MCC	Coupled
Bryson and Zapata-Medina (2012)	Idealised	-	C	HS	UESA
Lim et al. (2010)	Case study Taipei	DW	C	MC,USC,MCC,HS,HSS	UTSA,UESA
Scharinger et al. (2009)	Idealised	DW	C	MMS, MMSsss	UESA
Kullingsjö (2007)	Case study Gothenburg	SPW	C	MIT-S1, e-ADP, MC	UESA,UTSA
Finno et al. (2007)	Idealised	-	C	HS	UESA(?)
Zdravkovic et al. (2005)	Idealised	DW	C	MCss	UESA
Karlsrud and Andresen (2005)	Idealised	SPW	C	ANISOFT	UTSA

[a)] DW=diaphragm wall, Sec.P=secant piles, SP=soldier piles, SPW=sheet pile wall. [b)] C=Construction stage, S=Serviceability stage. [c)] See Table 2.2 for explanations. [d)] UESA=Undrained effective stress analysis (input of ϕ' and c'), UTSA=Undrained total stress analysis (input of c_u).

2.3.1 The Creep-SClay1S model

The Creep-SClay1S model is used in Papers A, B and D to simulate the behaviour of soft sensitive clays. In Paper D, the rate-independent SClay1S model (Karstunen et al. 2005) is also used. These models originate from the Modified Cam Clay model and have been developed by hierarchically introducing additional model features to the SClay1 model (Wheeler et al. 2003). The successive model development is outlined in Figure 2.7 including key references associated with each feature. A description of the Creep-SClay1S model is included in Appendix A of Paper A.

	Name	Feature added	Reference
1	SClay1	Anisotropy (inital and evolving)	Wheeler et al. (2003)
2	SClay1S	Destructuration	Karstunen et al. (2005)
3	Creep-SClay1	Rate-dependency (without destructuration)	Sivasithamparam et al. (2015)
4	Creep-SClay1S	Destructuration	Gras et al. (2018)

Figure 2.7: *Development of the Creep-SClay1S model.*

Additional model features in addition to those outlined in Figure 2.7 include temperature dependence (Li 2019) and cyclic accumulation (Zuada Coelho et al. 2021). Furthermore, Sivasithamparam and Castro (2016) included additional flexibility in the shape of the yield surface (the E-SClay1S model) to the rate-independent SClay1S model. A small-strain stiffness formulation was added to the E-SClay1S model by Sivasithamparam et al. (2021), in which the reduction of secant shear modulus is a function of the accumulated deviatoric strain. For prediction of cyclic strain accumulation at low loading amplitudes, Tahershamsi (2023) introduced the small-strain stiffness formulation of E-SClay1S in the cyclic accumulation model proposed by Zuada Coelho et al. (2021). The work presented in the papers appended to this thesis involves the SClay1S and Creep-SClay1S models. Exploiting the more recent model features are discussed in terms of recommendations for future studies (section 5.1).

3 Methodology

As the hydro-mechanical response of fine grained soils with low-permeability may evolve over a considerable time, the construction stage may only cover parts of the total change in effective stresses and deformations due to unloading, while remaining parts may occur during the serviceability stage, and hence affect the permanent structure. Previous reports on the modelling of actions on underground structures have, however (as was highlighted in the literature review), in general studied the deformations and actions on retaining structures during the construction stage only. When it comes to investigating the magnitude and evolution of heave induced earth pressure on underground structures, a holistic view of the response during the construction and serviceability stages is a necessity. Namely, the amount of action on the future structure, is affected by how much of that action that dissipates (as heave) during the construction stage. The hypothesis that was raised in the Introduction to this thesis (Sec. 1.1), is that a heave pressure will form if a structural restraint is placed such that it to some degree restrains the delayed heave caused by unloading.

In the current work, excavations and underground structures in soft clay are studied in order to reach the aim of this thesis, i.e to investigate the temporal evolution of earth pressures acting on underground structures in deep excavations in soft clay by means of numerical analyses and field monitoring. Excavations and construction of underground structures, however, are complex problems involving e.g. hydro-mechanical soil response, rate effects and soil-structure interaction. In the urban setting of soft normally consolidated or lightly overconsolidated clays, also the effect of background settlements needs to be considered. Therefore, a methodology is needed which can take a relevant level of complexity into account, but still, in the end, shed light on the mechanisms and factors that affect the actions and how these factors are related.

3.1 Outline and description of the methodology

The methodology adopted to reach the aim of this thesis consists of three parts, moving from site specific observations and modelling towards generalisation of the response. The methodology is described in the following and outlined in Figure 3.1, that also illustrate the order of the appended papers.

Part I consists of revisiting well-documented case-histories of excavations in soft clays. This relates to the first objective of this thesis, i.e. to benchmark a constitutive soil model that accounts for rate-dependency (Creep-SClay1S) against the soil–structure response at field scale. The cases that are revisited is an excavation for a part of Göta Tunnel in Gothenburg (Paper A), and a permanent sheet pile wall in Uppsala (Paper B), both in soft sensitive clays. These cases were selected as they were well-instrumented and documented, including project logbooks and photos, in the construction stage (and in Paper B also the serviceability stage), as well as well-characterised by in situ and laboratory tests.

Part I was undertaken since the Creep-SClay1S model had previously been benchmarked for unloading at element level (Wood 2016; Karstunen and Amavasai 2017) but not yet at boundary value level against well-documented deep excavations. Benchmarking against relevant field observations is, however, an important part to assess the applicability for industry usage of the

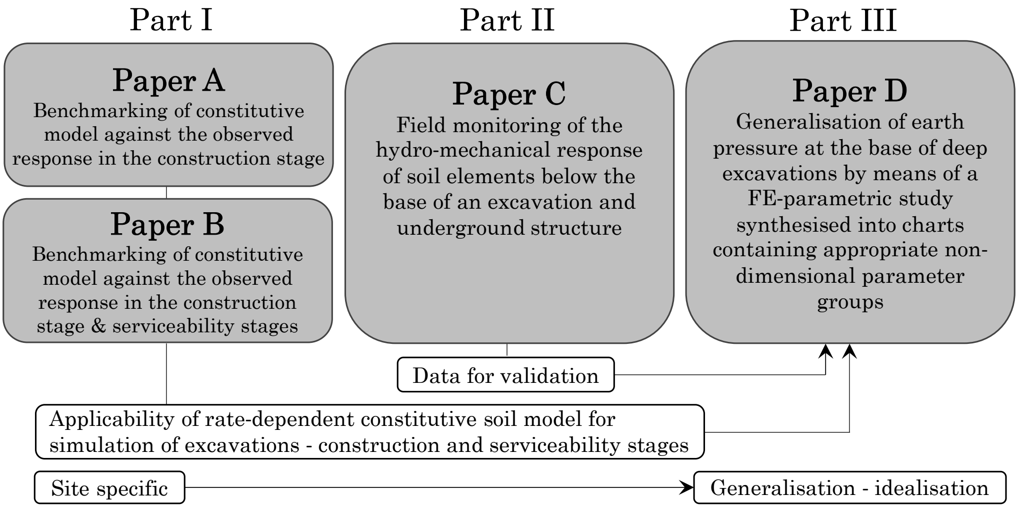

Figure 3.1: *Methodology presented as Parts I-III containing the appended papers.*

constitutive model as a tool for project specific analyses. Furthermore, for Part III of the work, i.e., to generalise the evolution of earth pressure due to restrained heave, an appropriate soil model is needed. An appropriate model meaning one that is i) capable of modelling the response during the construction and serviceability stages, and ii) has been benchmarked against relevant observations. Such a model must be effective stress based and include rate-dependency in settings with ongoing background creep settlements.

Part II comprises monitoring of the in situ hydro-mechanical response of soil elements below an excavation base and underground structure in soft sensitive clay. Such field observations were lacking in the monitoring programs in Papers A and B, as well as in the literature in general.

The methodology for monitoring in Part II consist of complementing a traditional control monitoring program, with additional instruments and clustered sensors. This relates to the second research objective, i.e., to explore the hydro-mechanical response below the base of an underground structure in soft clay. The combination of traditional control monitoring and additional clustered instruments provides an informed view of the temporal evolution of the system response, as well as the element level response, at field scale. This is important, as the interpretation of the response at soil element level below the excavation base and structure, is likely to be hindered if there is a lack of the simultaneous system level response (i.e., the bigger picture). This approach also relates to the active choice of concentrating the instruments and clusters that were installed for research purposes to one cross-section, i.e., it was considered better to have an informed observation in one section, ideally with sensor redundancy, than scattering isolated sensors in different parts of the excavation.

Additional site characterisation (sampling and laboratory testing) was carried out as it is essential for a complete description of the site, in the evaluation of the observed response, and to enable future studies, e.g. benchmarking of constitutive models including calibration of model parameters.

The additional sampling and laboratory testing also enabled project specific testing procedures, e.g. to perform oedometer tests with incremental unloading-reloading to stress levels that were to be expected during construction.

The monitoring is intended for observations of the temporal response during the construction stage, as well as part of the serviceability stage. This enables validation of numerical models that aim at simulating long-term response in soft clays. Part II does, however, not involve benchmarking of the Creep-SClay1S model against the gathered monitoring data. Rather, the monitoring data is revisited in Part III to partly validate the modelling approach adopted in the numerical parametric study, carried out to generalise the evolution of earth pressures. A Class A prediction of the system level response using the Creep-SClay1S model was, however, conducted within a Master's thesis (Harlén and Poplasen 2019) supervised by the author.

Part III is carried out to complement the findings of Part II by means of generalising the magnitude and evolution of effective heave pressure below underground structures in clay. A parametric study, using the Finite Element Method, is conducted and the results are synthesised into charts containing appropriate dimensionless groups (DGs). The complexity of the parametric study is increased sequentially concerning:

- Boundary conditions and geometry: starting with a *Building unit cell* and then a *Tunnel geometry* that includes a retaining wall and difference in elevation to the surrounding ground.
- Constitutive soil model for the clay: starting with the linear-elastic perfectly plastic Mohr-Coulomb model, followed by the SClay1S and Creep-SClay1S models.

This approach is adopted in order to distinguish the impact of different factors on the observed system behaviour. Part III covers the final objective of the thesis. The parametric study requires idealisation of the soil-structure interaction problem, and consideration of which parameters to include in the dimensional analysis. Once the boundary conditions and parameters are selected, dimensional analysis (DA) is carried out to form dimensionless groups (DGs). The parametric study is realised by means of creating scripts to generate the finite element model geometries and input of model parameters, as well as extract the results.

3.1.1 General notes on the methodology

Whereas Parts I and II contribute with the assessment of an appropriate model and relevant monitoring data for partial validation of Part III, respectively, those parts are inevitably site specific (e.g. in terms of excavation geometry, soil strata and construction time). Furthermore, the monitoring period is limited in relation to the service life of underground structures. The generalisation in Part III, including the numerical experiments synthesised into charts with appropriate DGs, has the potential to address these dearths and summarise the work (i.e., Parts I and II lead up to Part III) to benefit a wide-range of future projects. This is the benefit of the DA and DGs, i.e., presenting the results in a way that enables scaling and establishing the relationship between variables. The experimental (Chan et al. 2022b) and field monitoring data (Part II) that are used for validation are hence truly made the most out of.

The methodology relates to the aspects of the geotechnical triangle (Burland 2012), i.e., it involves establishing the ground profile (Parts I-III), observing behaviour (at laboratory and field scale, Parts I and II) and modelling, including validation of the response (Parts I and III). It should also be noted that to set-up and perform Part III and specifically the dimensional analysis, Parts I and II are instrumental, not only in benchmarking of a soil model and monitoring the hydro-mechanical response in situ, but also to provide "well-winnowed experience" (a part of the fourth aspect in the centre of the geotechnical triangle). Such experience aids the selection of what parameters to include in the DA (Sonin 2001; Palmer 2008; Longo 2021) and also to select the appropriate level of complexity in the parametric study and boundary conditions.

The DA provides an essential tool for the generalisation, i.e., it enables to:

(i) reduce the number of variables to a minimum set of dimensionless groups

(ii) identify relationships between variables (within a group, e.g. a variable to the power of n express the relative importance in relation to others variables within the group)

(iii) establish similitude, i.e., scaling laws how observations for a given setting can be extrapolated to another setting

(iv) synthesise experimental results into concise expressions or charts

(v) broaden the applicability of numerical experiments, as well as the field or model observations, that are used for validation

3.2 The trade-off between idealisation and realism

As was outlined in Figure 3.1, the methodology spans from site specific monitoring and analyses to a numerical parametric study, in order to arrive at generalised results and development of design charts. In such development, there is an inevitable conflict between realism and idealisation as described by Randolph (2013), see Figure 3.2. The additional aspect of soil model complexity has been added to Figure 3.2 by the author.

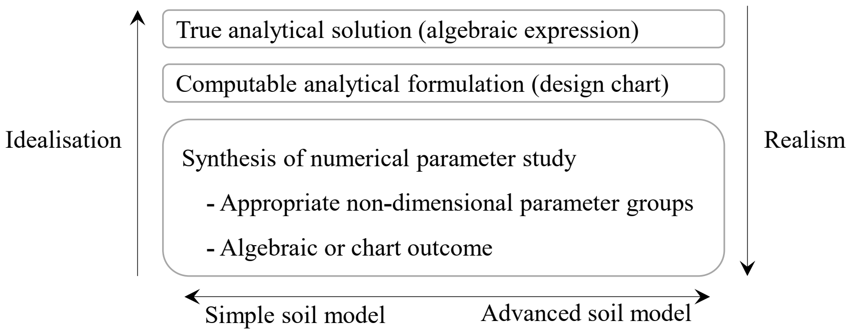

Figure 3.2: *Trade-off between idealisation and realism in development of tools. Adapted from Randolph (2013) with the author's addition of the aspect of soil model complexity.*

Generalisation by means of FE-analyses that include a soil model that has been benchmarked, and is able to model key features of soft clay response, helps in attaining useful tools that yet are not based on overly idealised soil behaviour. The trade-off here is that as assumptions of soil model features including model parameter values and boundary conditions are set, results can become increasingly accurate yet site-specific for a certain setting. It is the author's intention in Part III, that the developed charts should be general enough to enable insights on the mechanisms that control the studied system, but still contain a certain degree of complexity in order to represent the studied soil-structure system. In Part III, different geometries and soil models with varying complexity are used (starting with the linear-elastic perfectly plastic Mohr-Coulomb, to SClay1S and Creep-SClay1S), to aid different degrees of idealisation–realism. Idealisation does, however, not rule out tools to be valuable for first estimates and understanding of the mechanisms controlling the behaviour of a system. An example of this is the work by Janbu (1954), with results based on the simplification of constant undrained shear strength (i.e., no increase with depth). As long as the end-user is aware of such idealisations and the boundary condition that a tool contain, then the tool (e.g. a design chart) can readily complement detailed project-specific analyses.

3.3 Additional details on the studied soft sensitive clays

This section presents some of the laboratory tests that were conducted to characterise the soft sensitive clays at the sites presented in Papers B and C. These figures were not included in the appended papers (due to scope and limitations of space).

Figures 3.3–3.5 present the results from triaxial tests on samples from the instrumented site in Gothenburg, presented in Paper C. Table 3.1 provides an overview of these triaxial tests. The values for index parameters and the results from incremental loading oedometer tests on intact clay samples are included in Paper C. The results presented in Figures 3.3–3.5 are included here to provide a holistic picture of the response of a typical Swedish soft, lightly overconsolidated, sensitive clay in drained and undrained triaxial testing. The tests include standard undrained and drained triaxial compression and extension tests, as well as some non-standard test procedures. These non-standard procedures included (in separate tests); varying strain rate, relaxation stops, an unload-reload-unload sequence, drained triaxial tests in which the total vertical stress was kept constant during a constant decrease of the radial total stress. The latter non-standard tests (test id 5 and 9 in the figures) were conducted to simulate the situation on the retained side of a deep excavation. The drawback of this test type is that the strain-rate varies (the test is stress controlled). Hence, the response becomes partly drained as the strain rate increases rapidly when the sample approaches yield. The varying axial strain rate observed in these tests is indicated by the colorbar included in Figures 3.3–3.5.

The normalised results of the undrained tests, Figure 3.5, demonstrate the consistency and repeatability of the experimental results. The results (as well as results by Kullingsjö 2007 and Olsson 2013) have informed the selection of model parameter values used for the SClay1S and Creep-SClay1S models in Paper D.

Incremental loading tests were conducted on intact and remoulded clay samples from the site in Uppsala (Paper B) and Gothenburg (Paper C). Results from tests on intact samples are included in Papers B and C. Figures 3.6 and 3.7 present a comparison of the intrinsic compression lines and intrinsic modified creep indices, respectively, on remoulded clay samples from Uppsala and Gothenburg. Figure 3.6 also includes data from other reports. Due to the different depositional histories, the modified intrinsic compression indices, as well as creep indices for Gothenburg and Uppsala clays are noticeably different.

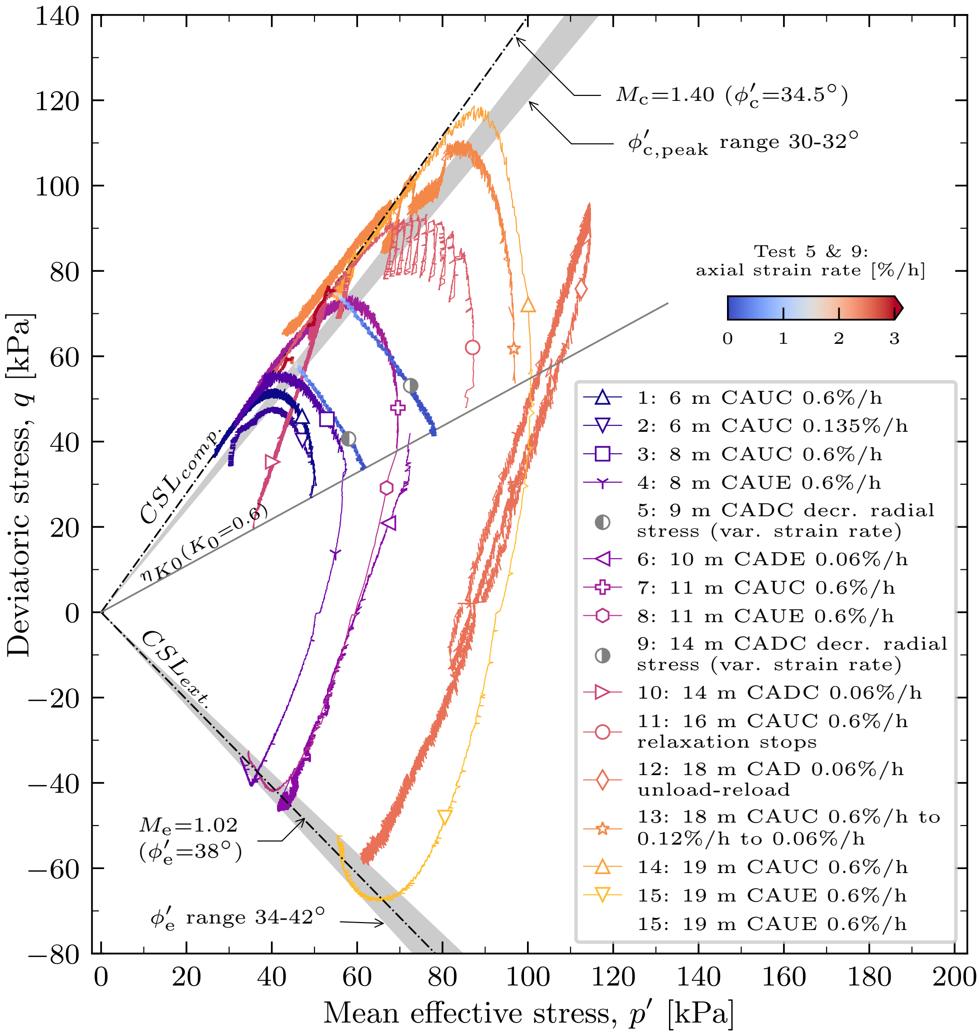

Figure 3.3: *Triaxial tests on samples from the site in Paper C (Gothenburg). Effective stress paths. Details are provided in Table 3.1.*

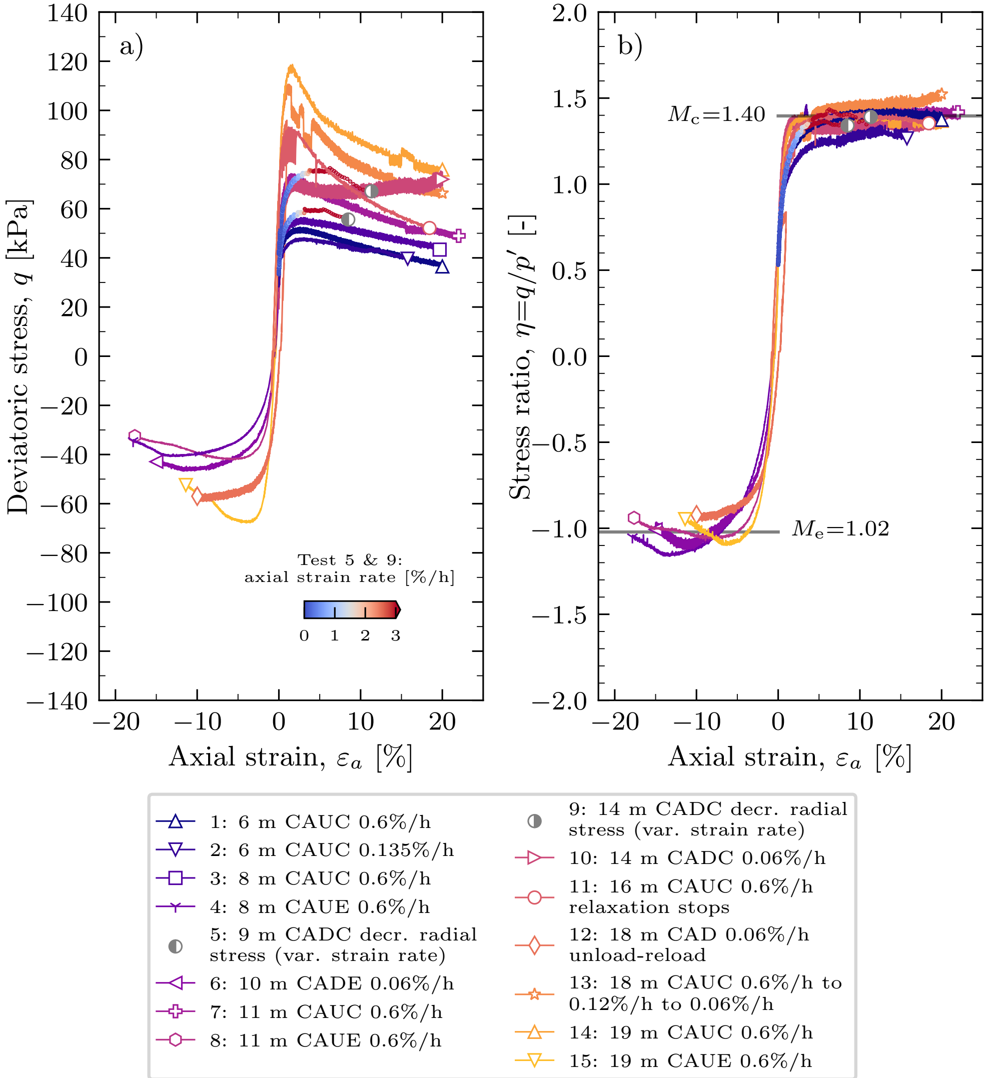

Figure 3.4: *Triaxial tests on samples from the site in Paper C (Gothenburg). Stress-strain response.*

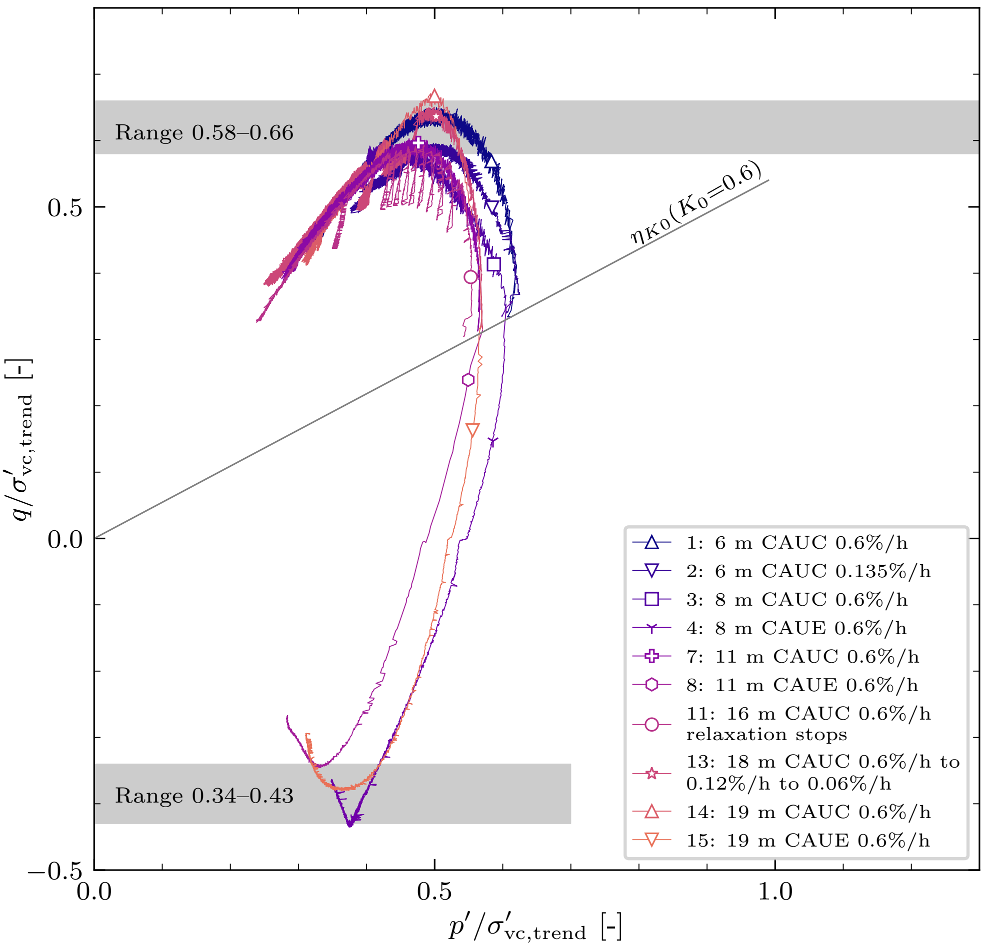

Figure 3.5: *Normalised effective stress paths for undrained triaxial tests on samples from the site in Paper C (Gothenburg).*

Table 3.1: Triaxial tests in Figures 3.3–3.5.

Id.	Depth [m b.g.l.]	Elevation [m]	Test	Note
1	6.1	-3.2	CAUC	
2	6.3	-3.4	CAUC	Strain rate 0.135 %/h
3	8.1	-5.2	CAUE	
4	8.3	-5.4	CAUC	
5	9.3	-6.4	CADC	Constant decrease of radial stress (-0.03 kPa/min).
6	10.1	-7.2	CADE	
7	11.1	-8.2	CAUE	
8	11.3	-8.4	CAUC	
9	14.1	-11.2	CADC	1)
10	14.3	-11.4	CADC	Constant decrease of radial stress (-0.03 kPa/min).
11	16.3	-13.4	CAUC	Standard strain rate with 1h relaxation stops.
12	18.1	-15.2	CAUC	Standard strain rate until peak, after peak varied 5 then 10 times slower than standard.
13	18.3	-15.4	CAD	Unload-reload then shear to failure in extension.
14	19.1	-16.2	CAUC	
15	19.3	-16.4	CAUE	

Note: standard strain rate CAU-tests 0.6%/h and CAD-tests 0.06 %/h unless stated otherwise.
1) After consolidation stage, test unloaded to higher OCR (along K_0=0.6 line) in order to "hit" the same point of the CSL as the CADC test performed on the sample from elevation -11.4 m.

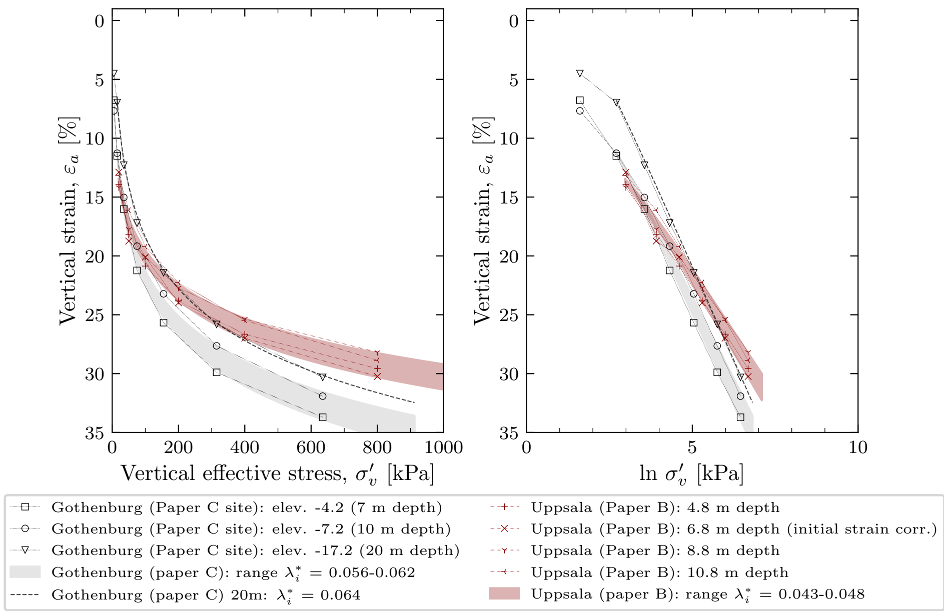

Figure 3.6: *Modified intrinsic compression index, λ_i^*, from incremental loading oedometer tests on remoulded Gothenburg and Uppsala clay.*

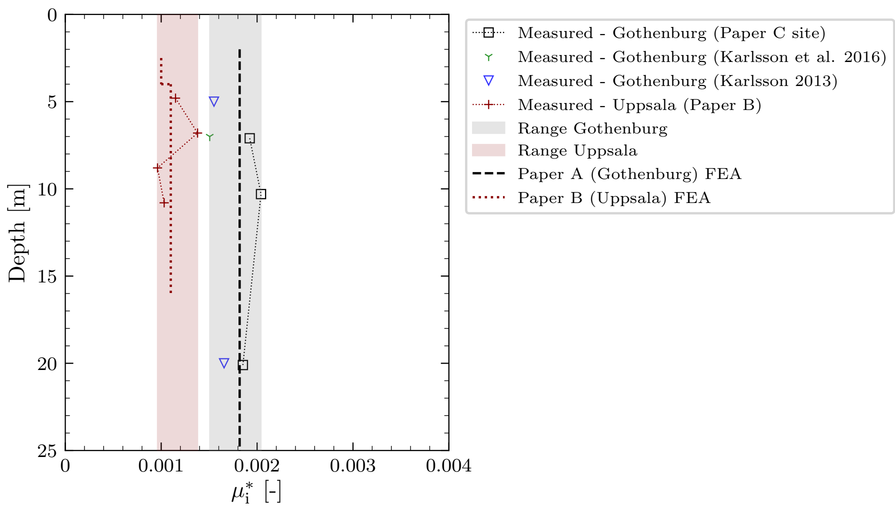

Figure 3.7: *Modified intrinsic creep index, μ_i^*, from incremental loading oedometer tests on remoulded Gothenburg and Uppsala clay.*

4 Summary of the appended papers

This chapter contains brief summaries including key findings of the appended papers.

Paper A: "Modelling the construction and long-term response of Göta Tunnel"

A section of Göta Tunnel, constructed 2000–2006, in Gothenburg, Sweden, is revisited in Paper A to benchmark the Creep-SClay1S model against the observed response of a well-documented excavation in soft sensitive clay.

Monitoring data for the particular section (1/430) was presented by Kullingsjö (2007) and comprised monitoring of displacements, pore pressures, horizontal earth pressures and strut forces during the construction stage. The excavation, including casting of a sealing slab, was carried out under water. To get input for the effective stress based finite element analyses used to benchmark the temporal response of the system with the Creep-SClay1S model, project logbooks and photos were reviewed. Evaluation of the parameter values for the Creep-SClay1S model were made based on calibration against the available oedometer and triaxial tests. As the field monitoring was discontinued at the end of the construction stage, the long-term response was assessed using recent InSAR data.

A brief summary of some key results and conclusions is provided in the following:

(i) Previous studies by Kullingsjö (2007) suggested that installation effects had an impact on the observed system response and should be accounted for. This was done since the tunnel foundation contained ten rows of relatively large ($0.4\times0.4\,\mathrm{m}^2$), driven concrete piles, including a procedure (extraction of soil using a hollow cylinder) to minimise the far-field displacements.

(ii) The agreement of computed and measured displacement reduced with increasing distance from the sheet pile wall (SPW). Not accounting for small strain stiffness in the modelling may be a reason for this disagreement. After dewatering, the continuing horizontal movements and settlements at distance behind the SPW were not properly captured, possibly due to undocumented construction activities in the studied or adjacent cross sections.

(iii) The observed temporal response of measured pore pressures (at different depths and distances behind the SPW), horizontal total stress behind the wall and the strut forces were captured well by the modelling.

(iv) Computed background settlements in the area, before construction and present day, were in line with those reported prior to construction and recent InSAR measurements.

(v) An advantage of the Creep-SClay1S model is that it enables the use of a single unified set of parameters for simulation of excavations and underground structures during the construction and serviceability stages.

Paper B: "Permanent Sheet Pile Wall in Soft Sensitive Clay"

Paper B comprises a benchmark of the Creep-SClay1S model against the observed response during the construction and serviceability stages of a permanent sheet pile wall in Uppsala, Sweden. Primary differences compared to Paper A are that the monitoring data spans the construction and serviceability stages, the geological setting on the Swedish East coast, as well as the anthropogenic loading history.

The studied sheet pile wall formed the temporary and permanent retaining structure in connection with a new double-track railway. Monitoring of displacements and tie back loads during the construction stage (2015–2017) and the subsequent serviceability stage were provided by the Swedish Transport Administration. Additional site characterisation (in situ and laboratory testing) was conducted to assess the current state of the soil and to enable deriving the values for the Creep-SClay1S model parameters. The anthropogenic loading history in Uppsala includes the effects of a groundwater drawdown, due to extraction of drinking water in the aquifer below the clay layer. Near the studied section this extraction was initiated between the late 1960s and early 1970s, and this loading history was included in the modelling. The paper discusses the challenges when taking into account the recent anthropogenic loading history to set up the initial, i.e. before construction, conditions and initial state. Similar to Paper A, project logbooks and photos were revisited in order to follow the construction process and get the input data for modelling of the temporal response.

A brief summary of some key results and conclusions is provided in the following:

(i) An objective of Paper A was to highlight the challenges in modelling the initial conditions and soil state in an area with anthropogenic loading history, in Uppsala caused by groundwater extraction and drawdown. Since the soil properties prior to the drawdown are unknown, an iterative approach was needed to model the state prior to the construction stage.

(ii) The computed rate of background settlements was underestimated both prior to and after the construction stage. One reason is that the parameter sets used in the Creep-SClay1S model produced a higher than expected overconsolidation ratio in the lower part of the clay layer when modelling the groundwater drawdown. After the construction stage, i.e. in the serviceability stage, cyclic loading from the railway (not accounted for in the modelling) could also have contributed to the discrepancy in the observed and computed settlement rates.

(iii) The computed horizontal displacements of the SPW underestimated the observed response, but the general trend was in good agreement with the monitoring data.

(iv) The measured anchor forces were seen to correlate with air temperature. Prior to the permanent insulation of the SPW, the winter of 2015–2016 resulted in anchor forces increasing by 30–40% before temporary measures (heating and insulation) were put in place. In the service life, although insulated, continued temperature-force correlation was observed. Except for the variation in force due to change in temperature, the computed force showed good agreement to measurements (computed overestimation varying by 5–15%, range due to the continued temperature-force correlation).

(v) As for Paper A, the Creep-SClay1S enabled to simulate the construction and serviceability stages using a single unified parameter set. In addition to Paper A, Paper B enabled benchmarking against monitoring data spanning both the construction and serviceability stages.

Paper C: "Temporal effective stress response of soil elements below the base of an excavation in sensitive clay"

Paper C presents details of field instrumentation conducted to monitor the hydro-mechanical response of soil elements below the base of an excavation and underground structure in soft sensitive clay. Such observations were identified to be lacking, both in the cases revisited in Papers A and B, as well as in the literature in general.

The instrumented site is located in central Gothenburg. The site was selected since the clay deposit was already well characterised, and the contractor had planned for a sound control monitoring program, which offered complementary data on the system level response.

The methodology for the instrumentation carried out by the author, i.e. additional to the control monitoring, is briefly summarised in this paragraph. A scope and layout of instrumentation was set to target the hydro-mechanical response of the soil elements below the base of the excavation and the permanent structure. A limitation that needed to be considered concerned the time to install sensors in an active construction site. The trade-off here resulted in a layout of three instrument clusters placed in one cross-section of the excavation. Verification of the calibration factors supplied by the manufacturer was performed in the laboratory at Chalmers for each sensor. Additional instruments to monitor the system level response were installed prior to the construction works (inclinometers and piezometers outside the excavation, and a bellow-hose extensometer and piezometers inside the excavation). Construction works (pile driving, SPW installation and excavation) required safe guarding (e.g. excavation by hand) of the instruments inside the excavation. Manual measurements of vertical displacements versus depth (bellow-hose extensometer) were conducted. As the bottom of the bellow-hose was "floating" in the clay layer, a reference elevation had to be established by surveying for each measurement occasion. Once the excavation reached the final depth, the three instrument clusters were installed for the monitoring of pore pressures, vertical and horizontal stresses below the excavation base, as well as an automatic extensometer for measuring the relative slab-clay displacement.

A summary of some key results and conclusions is provided in the following:

(i) The instrument clusters (each containing one piezometer and two earth pressure cells) enabled to monitor the hydro-mechanical response of soil elements below the base of an excavation and permanent structure at field scale including evaluation of the stress ratio, $K = \sigma'_{\mathrm{h}}/\sigma'_{\mathrm{v}}$.

(ii) The paper addresses the need to compensate for change in apparent earth pressure arising due to thermal volume change of the hydraulic earth pressure cell fluid. The inherent difficulty of such compensation (due to the unique boundary conditions/confinement in the field for each cell) was discussed. An approach was selected to address the uncertainty that arises, and the results are presented such as to provide transparency regarding this aspect.

(iii) The results provide data on the evolution of K in situ, and thus enable a unique comparison to prior, laboratory established, relationships. The general trend of the in situ stress path in the centremost instrument cluster, i.e. the cluster that most resembles oedometer conditions, approximately follows the relationship $K = K_0^{\mathrm{nc}} OCR^{\mathrm{m}}$ with K_0^{nc}=0.53 and m=0.6. Previous laboratory based relationships for K are thus corroborated, although approximately, by the field measurements.

(iv) At system level, the heave within the excavation was observed primarily down to elevation -12 m (i.e., 10 m below the excavation base) in the period from completion of excavation in the section until the slab was cast. At greater depth, no significant vertical displacement was observed during this period. This response demonstrates a conflicting nature in the evolution of vertical deformations after pile driving (delayed settlement) and excavation (delayed heave).

(v) After casting the working platform and slab, the effective stresses increased in the instrument clusters. However, when the backfill was placed between the final structure and SPWs, pumping within the excavation stopped and the first SPW was extracted, the pore pressures below the slab started to increase and regained to the level prior to construction. The regaining pore pressure caused the effective stresses in the instrument clusters to reduce considerably. Consequently, the structural load is primarily carried by water pressure and/or the piles.

As the monitoring data spans the construction and serviceability stages, it provides validation data for generalisation (Paper D) of the magnitude and evolution of effective stresses beneath underground structures in soft clay.

Paper D: "On the development of effective heave pressure in deep excavations"
Paper D comprises a parametric study, using the Finite Element Method, to generalise the development and evolution of effective heave pressure (EHP) beneath underground structures in deep excavations. This complements and extends the value of the field observations reported in Paper C, where the monitoring period was limited in time in relation to the service life of underground structures.

Identification of dimensionless groups underpinned the design and the evaluation of the results of the parametric study that was done using finite element analyses. The dimensionless groups were constructed from the variables that were considered to govern the soil-structure system behaviour. Two idealised system geometries were considered; a building unit cell and a tunnel geometry. Initially, the linear-elastic perfectly plastic Mohr-Coulomb (MC) model was used. This was followed by simulations using the Creep-SClay1S model, as well as a rate-independent version of the model (SClay1S). The system geometries and the soil models adopted provided the means to add complexity stepwise to the parametric study. Thus, mechanisms that affect the magnitude and evolution of EHP could be separated and quantified.

Some key results and conclusions are briefly summarised in the following:

(i) For infinite relative stiffness (between the soil and the structure), the simple building unit cell geometry resembles an oedometer. The dissipation of heave, hence the magnitude of EHP, can then be considered as a reversed process to that of consolidation settlement, and any EHP is uniform along the rigid slab. However, except for the case with high relative stiffness ($R > 0.1$ in this study), in general the distribution of EHP along the slab is non-uniform. This leads to ratios of EHP/$\sigma'_{v0} > 1.0$ at and near the idealised clamped supports of the slab.

(ii) The distribution of the bending moment M along the slab was normalised with the bending moment at the location of the fixed support (the wall) M_{wall}. The normalised bending moment (M/M_{wall}) along the slab was shown to depend on the relative stiffness of the slab and the soil. The bending moment M_{wall}, and hence M along the slab, due to EHP can be estimated as a function of normalised time T and relative stiffness R using the presented results.

(iii) The geometry of the excavation in relation to the depth of the clay layer is observed to affect the magnitude of EHP even for very small (undrained) T and infinite R. This revealed that the problem should be considered as an analogue to "settlement influence factors", i.e. the geometry affects the ratio of immediate to delayed heave. An EHP-influence factor was derived, to generalise the effect of the geometry of the excavation in relation to the clay layer depth.

(iv) The two-dimensional consolidation controls the development of the delayed heave, and hence the magnitude of EHP. A 2D time factor was introduced, which in combination with the derived influence factors is used to normalise the results.

(v) The effect of a reduced length of the embedded retaining wall (or, in practice, a reduced capacity of the retaining system) was investigated and seen to reduce the central EHP ratio by a maximum of 20%. The effect is only evident for small T (i.e., if there is any negative excess pore pressure to "cancel out" by the mechanisms caused by reduced stability).

(vi) A worked example on the use of the synthesised non-dimensional charts was provided. This example was the prototype excavation and underground structure presented by Chan et al. (2022b). Parameter values and dimensionless group values were calculated for this setting, and the results obtained from the non-dimensional charts compared well with the EHP ratio and bending moments observed in the prototype.

(vii) Finally, two factors specific to a setting of deep deposits of soft clay were investigated as follows:

Firstly, a simplified case representative to that of Central Gothenburg demonstrated the effect of background settlement on the evolution of EHP. For this specific setting, the maximum EHP occured 5–10 years (depending on excavation width) after activation of the restraining structural element. From that time onwards, the long-term EHP reduces due to "down-drag" with a notable reduction for excavation widths ≤50 m.

Secondly, the effect of a groundwater table located 1 m below ground level was investigated using the SClay1S model. The EHP ratio ($\sigma'_{\mathrm{v}}/\sigma'_{\mathrm{v0}}$) when an excavation remained dewatered for 10 years (extreme case) was simulated. When the groundwater thereafter was allowed to return below the slab, the effective stresses reduced. In another set of simulations, the groundwater was allowed to start regaining 90 days after construction of the slab. This caused the regain of water pressures under the slab to counteract the stress increase due to the evolving effective heave pressure. The long-term (120 year) value was, however, similar for these two set of simulations (groundwater return after 90 days or 10 years).

Taking values of T for a fictive tunnel construction in Gothenburg as an example, the results with a groundwater table located 1 m below ground level indicate a long-term EHP ratio (after the return of water pressures below the slab) of 0.1–0.2 (median typical T value, estimated to 0.1) and 0.3 maximal (5th percentile typical T value, estimated to 0.01). If, however, there would be no return of the water pressure beneath the slab in this example, the corresponding EHP ratios increase to 0.5 and 0.7, respectively.

(viii) The computed results for a scenario that resembled the site settings in Paper C showed good agreement to the observed evolution of pore and earth pressures below the excavation base and underground structure.

5 Conclusions and recommendations

The aim of this thesis was to investigate the temporal evolution of earth pressures acting on underground structures in deep excavations in soft clay by means of numerical analyses and field monitoring. The results of field monitoring at soil element level and site specific observations were complemented and generalised by means of numerical modelling. This enabled to quantify how factors such as normalised construction time (i.e., time, soil permeability, stiffness and layer depth), excavation geometry, relative stiffness, and background settlements, affect the evolution of heave pressure below underground structures. The final outcome is non-dimensional earth pressure charts that can readily be used in preliminary design stages, and as a complement to detailed project-specific analyses.

The methodology presented in this work consisted of three parts. The first part involved benchmarking of the Creep-SClay1S model against the observed response of two well-documented excavations and underground structures in soft clay (Papers A and B). The main conclusion from this part, is that the Creep-SClay1S model can be used to compute the temporal evolution of earth pressures acting on underground structures in soft clay, and the resulting displacements, with sufficient accuracy. As demonstrated by the two papers, a key advantage of the rate-dependent model is that it enables to simulate the construction and serviceability stages using a single unified parameter set. Furthermore, the effect of background creep settlements can be accounted for.

In the second part, the hydro-mechanical response of soil elements below the base of an excavation and underground structure in soft clay was monitored (Paper C). Clustered instruments enabled unique observations of the evolution of effective stresses and the stress ratio, $K{=}\sigma'_{\mathrm{h}}/\sigma'_{\mathrm{v}}$, at soil element level in situ. In contrast to observations of horizontal earth pressures on retaining structures, this type of study (into earth pressures below slabs or tunnels and covering the construction and serviceability stage) is rare, especially concerning soft clays. The results for the centremost cluster, i.e. cluster with most resemblance to oedometer conditions, indicated a general trend that approximately follow the laboratory based relationship $K{=}K_0^{\mathrm{nc}}OCR^{\mathrm{m}}$ with $K_0^{\mathrm{nc}}{=}0.53$ and $m{=}0.6$. Previous laboratory studies were thus corroborated, albeit approximately, by the field measurements. The effective stresses increased in the instrument clusters after casting the working platform and slab. However, as the pore pressure regained to its level prior to construction, the effective stresses in the instrument clusters reduced considerably. The total vertical stress exerted on the slab in the serviceability stage were hence primarily composed of water pressure and/or the pile loads.

A secondary benefit of Paper C is that it demonstrates how a traditional monitoring programme (with observations at system level), can be successfully complemented with instrument clusters. This enables to extract and leverage knowledge on the hydro-mechanical response of soil elements at field scale, as a complement to laboratory-based investigations. The results and documentation presented in Paper C enable future benchmarking of numerical models, including constitutive models, at element and boundary value level.

The final part of this thesis comprised of a parametric study (Paper D), conducted using the Finite Element Method, to complement the field monitoring results and to generalise the formation and evolution of heave pressure below underground structures in clay. This part make the combined effort and results of Papers A, B and C applicable to a wide range of settings and projects.

Dimensional analyses provided a means to design the parametric study and synthesise the results into charts containing appropriate non-dimensional parameter groups.

The results show that the magnitude and evolution of effective heave pressure below underground structures are affected by the normalised construction time, two-dimensional consolidation, relative stiffness, excavation geometry (in relation to clay layer depth), the stability of the excavation, background settlements, and (in scenarios involving dewatering during the construction stage) the return of water pressure below the slab.

The return of water pressures was shown to compensate the increase in effective stress due to effective heave pressure. The results of complementary site-specific simulations presented in Paper C indicates, taking a typical value of T for a fictive tunnel construction in Gothenburg as an example, a long-term EHP ratio of 0.5 if there is no return of water pressures below the slab and in the range 0.1–0.2 if the water pressure returns (these ratios correspond to the median of estimated typical T values). The main benefit of the dimensionless charts is, however, that they can replace such typical values or ranges. Rather, the dimensionless charts can readily be used to assess the EHP ratio by considering the parameter values for the specific project.

The effect of background settlements was demonstrated to reduce EHP in the long-term. The results show that whereas the maximum effective heave pressures are unaffected, the long-term EHP is reduced due to "downdrag". The reduction was shown to depend on the width of the excavation in relation to the depth of the clay layer.

The field monitoring data in Paper C, as well as data from physical model tests available in the literature, was used for partial validation and benchmarking of the results of the parametric study. The computed results showed good agreement with the observed temporal evolution of pore and earth pressure at field scale, as well as the magnitude of the central heave pressure and bending moments in the slab observed in the physical model tests.

The outcome of the parametric study, i.e. the synthesised charts in Paper D, reveals the mechanisms that control the development and evolution of effective heave pressure in deep excavations. The results can readily be used as a tool for industry to assess the magnitude of effective heave pressure to complement detailed project-specific analyses. This was the overall goal of this thesis. To conclude, this work increases the understanding for which factors substantially influence the development of the effective heave pressure. This can aid the design practice and assist in the development of design guidelines, reduce uncertainty in design, and ultimately lead to optimisation of construction material volumes for underground structures.

5.1 Recommendations

The recommendations, including proposed future studies, are listed in the following.

(i) The response during the construction stage affects the actions in the serviceability stage, and thus it is recommended to perform analyses that take this aspect into account. The earth pressures arising from a restrained heave process is a direct example of how the construction process (e.g. construction time) affects the actions in the serviceability stage.

(ii) As charts have been developed in this thesis for the assessment of effective heave pressure, it is strongly encouraged that industry practice includes i) independent project-specific analyses, if that may lead to optimised construction material volumes, and ii) field observations which aim to increase the knowledge base of actions on underground structures, e.g. in situ pore and earth pressures. Namely, relevant observations (at field scale or in laboratory experiments) are essential in order to benchmark and possibly adjust our conceptual models and design tools.

(iii) The use of design tools (e.g. analytical methods, empirical rules of thumb or synthesised charts, such as presented in Paper D) requires that the user/designer knows what underlying data, assumptions and boundary conditions constitute the basis of the tools, and hence understands their limitations.

(iv) Useful monitoring requires, among others, careful planning followed up at site and sensor redundancy. As such, industry including the asset owners must recognise the investment needed, and set out to monitor the system performance of underground structures, not just during the construction phase, but also in the long-term service life. Such monitoring can help also in extending the service life of structures and aid sustainable solutions in the future. In other words, investment in monitoring today may cost in monetary terms (although a small amount compared to the project costs), but fortunately the CO_2-impact of monitoring is negligible compared to that of the construction materials. Failure in long-ranging horizons and investments, however, may (both on project scale and the industry as a whole) result in big monetary costs and CO_2-impact (e.g. non optimised design with respect to set safety level, increased maintenance or unnecessary mitigation measures).

(v) The generalisation presented in Paper D can be extended to include the effects of pile installation (driven piles), adjacent existing structures and small-strain stiffness. Such studies would make valuable continuations to the adopted methodology by step-wise adding complexity to and refining the generalisation.

(vi) Recent formulations to extend the features of the Creep-SClay1S model involves temperature dependence, small-strain stiffness and cyclic accumulation (Li 2019; Sivasithamparam et al. 2021; Zuada Coelho et al. 2021; Tahershamsi 2023) tested at element level. The observed response documented in Papers A-C could be revisited to benchmark such extensions of the Creep-SClay1S model, either separately or in combination. Once validated, model extensions can be used to update and extend on the generalisation presented in Paper D.

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