

TEMPERATURE EFFECTS IN DEEP CONSTRUCTION PITS: E05 PROJECT IN GOTHENBURG

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Aos meus Pais

Defects do no harm when there is the will and power to correct them.

Machado de Assis

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ABSTRACT

This thesis has as its main objective to evaluate the effect that temperature changes due to climate have on temporary support systems, specifically on strut elements. To that effect, the case chosen to be studied was the E05 project in Gothenburg. This project is a deep excavation in exceedingly difficult soil conditions, consisting of extensive layers of very soft clay, followed by sandy/gravelly *moraine* and lastly strong and capable bedrock. The soft clay was improved by the use of Lime-cement stabilising columns.

The construction occurs over the span of several years in a location where the temperature ranges greatly from one season of the year to another. Because of that the design loads caused by the fluctuations are of importance. These loads are based on recommendations in CIRIA C760 for the temperature restraint coefficient, which is dependent on the type (flexibility) of the wall and the ground conditions. This coefficient is the core of this work.

The author opted to focus on two tasks, the study three scenarios related to distinct types of retaining walls to determine the range of the restraint coefficient in Gothenburg soil, and the use of Plaxis 2D models to try and simulate the load fluctuations due to temperature.

The results obtained from monitoring were within the expected range. It was concluded that the stiffness of the bedrock and wall significantly influence the restraint of the struts. Overall, the results support the safety of the CIRIA recommendations for the restraint coefficient. However, it was found that there is a substantial variation in restraint in different situations, such as different strut levels, which is not considered when using a singular coefficient for all struts. The results of this master thesis can assist with establishing a differentiated basis for temperature restraint coefficients on future projects with similar conditions.

Comparing the results from the Plaxis modeling with the monitoring data showed that they were acceptable, although there was a slight underestimation.

KEYWORDS: Temperature fluctuations, finite element method, soil improvement, restraint coefficient, Gothenburg clay, temporary retaining structures, struts, Plaxis 2D.

RESUMO

Esta tese tem como principal objetivo avaliar o efeito das mudanças de temperatura devido ao clima nos sistemas de suporte temporário, especificamente em escoras. Para isto, o caso de obra a ser estudado foi o Projeto E05 em Gotemburgo. Este projeto consiste em uma escavação profunda em condições de solo muito difíceis, com extensas camadas de argila muito mole, seguida por uma *moraine* com areia/cascalho e, por fim, rocha matriz resistente. A argila mole foi melhorada pelo uso de colunas estabilizadoras de cal-cimento.

A construção ocorre ao longo de vários anos num local onde as temperaturas variam consideravelmente de uma estação do ano para outra. Como consequência, as cargas de projeto causadas pelas variações de temperatura são de importância. Essas cargas são baseadas nas recomendações do CIRIA C760 para o coeficiente de restrição de temperatura, que é dependente do tipo (flexibilidade) da parede e das condições do solo. Análise desse coeficiente é o cerne deste trabalho.

Optou-se por se concentrar o estudo em duas tarefas: estudar três cenários relacionados com diferentes tipos de paredes de contenção para determinar o intervalo do coeficiente de restrição no solo de Gotemburgo; E usar modelos Plaxis 2D para tentar simular as variações dos esforços em função da alteração de temperatura.

Verificou-se que os resultados obtidos da monitorização da obra estavam de acordo com o comportamento previsto. Concluiu-se que a rigidez da rocha matriz e da parede influenciam significativamente o comportamento das escoras. No geral, os resultados confirmam as recomendações do CIRIA para o coeficiente de restrição. No entanto, constatou-se que há uma variação substancial no coeficiente de restrição em diferentes situações de obra, assim como para diferentes níveis de escoras, o que não foi considerado em projeto ao usar-se um único coeficiente para todos os elementos de suporte.

A comparação dos resultados do modelo Plaxis com os dados de monitorização mostrou que o valor do coeficiente de restrição era aceitável, embora tenha havido uma ligeira subestimação. Os resultados desta tese de mestrado pretendem ajudar a estabelecer uma base de referência para os coeficientes de restrição de temperatura para serem utilizados em projetos com condições semelhantes.

PALAVRAS-CHAVE: Flutuações de temperatura, método dos elementos finitos, melhoramento de solo, coeficiente de restrição, argila de Gotemburgo, estruturas de suporte temporárias, escoras, Plaxis 2D.

ABSTRAKT

Denne afhandling har som sit primære mål at evaluere effekten af temperaturændringer på midlertidige understøttelsessystemer, specifikt på strutelementer. Til dette formål blev E05-projektet i Gothenburg valgt som case til undersøgelse. Dette projekt er en dyb udgravning i ekstremt vanskelige jordforhold og består af omfattende lag af meget blød ler, efterfulgt af sandet/grusagtig moræne og til sidst stærkt og robust grundfjeld. Den bløde ler blev forbedret ved brug af lime-cement stabiliseringskollonner.

Byggeriet strækker sig over flere år på et sted, hvor temperaturen varierer meget fra den ene årstid til den anden. På grund af dette er designbelastningerne forårsaget af disse udsving af afgørende betydning. Disse belastninger er baseret på anbefalinger i CIRIA C760 for restriktionskoefficienten ved temperatur, som afhænger af typen (fleksibiliteten) af muren og jordforholdene. Denne koefficient udgør kernen i dette arbejde.

Forfatteren valgte at fokusere på to opgaver: undersøgelse af tre scenarier vedrørende forskellige typer af støttemure for at bestemme spændvidden af restriktionskoefficienten i Göteborgs jord samt anvendelse af Plaxis 2D-modeller til at forsøge at simulere belastningsudsvingene som følge af temperatur.

Resultaterne opnået fra overvågning var inden for det forventede område. Det blev konkluderet, at stivheden af undergrunden og væggen signifikant påvirker restriktionen af tiranterne. Generelt understøtter resultaterne sikkerheden i CIRIA-anbefalingerne for restriktionskoefficienten. Dog blev det fundet, at der er en betydelig variation i restriktionen i forskellige situationer, såsom forskellige tirantniveauer, hvilket ikke tages i betragtning, når der anvendes en enkelt koefficient for alle tiranter. Resultaterne af denne kandidatafhandling kan hjælpe med at etablere en differentieret basis for temperaturrestriktionskoefficienter på fremtidige projekter med lignende betingelser.

Sammenligning af resultaterne fra Plaxis-modelleringen med overvågningsdataene viste, at de var acceptable, selvom der var en let underestimering.

NØGLEORD: Temperatursvingninger, finite element-metode, jordforbedring, restriktionskoefficient, Göteborgs ler, midlertidige støttestrukturer, struts, Plaxis 2D

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SYMBOLS, ACRONYMS AND ABBREVIATIONS

A – area (m^2)

c – Specific heat capacity ($\text{J/kg/}^\circ\text{C}$)

c/c spacing – Center to center spacing

c' – Cohesion (kPa)

c_u – Undrained shear strength (kPa)

D – disturbance factor

E – Elastic modulus (KPa)

EA – Axial stiffness

EI – Bending stiffness

G_{ur}/S_u^A – Ratio unloading/reloading shear modulus over (plane strain) active shear strength

h_{ref} – Reference depth, Plaxis Thermal parameter (m)

I – Inertia (m^4)

K – earth pressure coefficient

K_0 – At-rest earth pressure coefficient

$KV1$ – Korsvagen 1

$KV2$ – Korsvagen 2

L – Length (m)

LC – Lime-cement

M – Constrained Modulus (MPa)

N_{temp} – Load change due to temperature effect (kN)

S_u^A – Increase of shear strength with depth ($\text{kN/m}^2/\text{m}$)

S_u^P/S_u^A – Ratio of (plane strain) passive shear strength over (plane strain) active shear strength

S_u^A – Reference (plane strain) active shear strength (kN/m^2)

S_u^{DSS} / S_u^A – Ratio of direct simple shear strength over (plane strain) active shear strength

$s_u^{C,TX}/S_u^A$ – Ratio triaxial compressive shear strength over (plane strain) active shear strength

τ_0/S_u^A – Initial mobilization

u – Strut horizontal deformation (m)

V_s – Shear wave velocity (m/s)

w_n – Water content (%)

γ_t^C – Shear strain at failure in triaxial compression (%)

γ_t^E – Shear strain at failure in triaxial extension (%)

γ_t^{DSS} – Shear strain at failure in direct simple shear (%)

y_{ref} – Reference depth, Plaxis parameter (m)

α – Thermal coefficient of expansion for prop material (1/°C)

β – Restraint coefficient (%)

γ – Unit weight (kN/m³)

ΔT – Change in temperature (°C)

ΔL – Change in strut length (m)

λ – Thermal conductivity (W/m/K)

ρ – Density (t/m³)

ν – Poisson ratio

σ_{ci} – uniaxial compressive strength of the intact rock (MPa)

σ_ψ – Absolute value of confining pressure σ'_3 at which $\psi = 0^\circ$ (MPa)

σ_c – Rock mass uni-axial compressive strength (MPa)

σ_t – Tension cut-off and tensile strength (MPa)

ϕ – Friction angle (°)

ψ – Dilatancy angle (°)

DTH hammer – Down the hole hammer

FEM – Finite element method

GSI – Geological Strength Index

MC – Mohr-Coulomb

RMR – Rock mass rating

THM – Thermo-hydro-mechanical

R_{inter} – Reduction of strength at the interface

RQD – Rock Quality Designation

SaSiCL – Sand, silt and clay

SiCL – Silt and clay

SiSa – Silt and sand

SSI – Soil-Structure Interaction

T_{ref} – Reference temperature, Plaxis thermal parameter (°C)

1

INTRODUCTION

1.1. SCOPE

Retaining walls in deep excavations typically require temporary support systems, commonly consisting of props placed at varying levels depending on the excavation's size. These props, often tubular sections, though not excessively costly themselves, entail significant time and risk during installation and removal. Consequently, a deeper understanding of the fundamental behaviour of propping systems used in large excavations, including their responses to factors such as temperature, is crucial for developing more cost-effective designs. This understanding not only facilitates safer and more efficient operations for contractors but also mitigates potential risks associated with inadequate comprehension of temporary works in large excavations.

Temperature emerges as one of the most critical factors affecting prop actions, capable of generating forces on the strut that significantly impact both design considerations and post-installation performance. The importance of temperature cannot be underestimated, as evidenced by numerous studies. For example, a 2000 investigation into the propping system at the Canada Water Station analysed load data from various props (Powrie and Batten 2000a), revealing that thermal effects contributed to up to 37% of the measured load.

Recognizing the relevance of thermal effects, the Construction Industry Research, and Information Association (CIRIA) provides recommendations to how to account for these effects in its design guide, CIRIA C760: Guidance on Embedded Retaining Wall Design (Gaba et al., 2017). These recommendations, informed by numerous prior studies on deep excavations utilizing temporary propping systems, suggest that conventional design methods for braced excavations often overestimate prop loads generated by temperature when compared to values observed during subsequent construction phases. This tendency may result in unnecessary over-designing of support systems.

1.2. OBJECTIVE

The overall objective is to through sensitivity studies using FEM to improve the model for temperature induced loads in the supports of multi-level strutted retaining systems for deep excavations allowing for a more optimized and therefore more sustainable design of construction pits.

Saying this, the following objectives and questions were established for this work:

- According to monitoring data, what is the range of the restraint coefficient, β , which represents the percentage of the effect of the temperature to be considered in the behaviour of the struts, in typical Gothenburg ground conditions.
- What factors may have the greatest influence on the restraint coefficient of a determined strut.
- Calibrate constitutive models for relevant soil types by back-analysis to measurement data.
- Can Plaxis simulations (with best estimate soil parameters) reliably predict the temperature restraint coefficient, i.e., match the monitoring data.
- Based on the above, is it possible to propose a more accurate temperature restraint coefficient for these types of walls in these ground conditions to allow for designers and contractors to optimize future projects?

1.3. THESIS STRUCTURE

The document is structured into seven chapters.

The first chapter provides a brief introduction to the thesis topic, outlining the main objectives.

Chapter two offers a concise explanation of temporary support systems, focusing on the design basis of struts. It also discusses the specific subject of this thesis, which is the impact of temperature changes on the loads experienced by support struts.

The third chapter presents the case study, from which measurements were collected and will be compared with the results of the Finite Element Method (FEM) model. It also includes a description of the geotechnical conditions of the project.

Moving forward, the fourth chapter analyses the monitoring data gathered from the construction site.

In the fifth chapter, there is an introduction to the Finite Element Method and the relevant information on what actions were taken to accurately model the problem on PLAXIS 2D , by Bentley. Additionally, it presents the achieved results.

Chapter six compares and analyses the measured results, recommendations from CIRIA, and the findings of the PLAXIS analysis.

Lastly, the seventh chapter concludes the document by summarizing the obtained results and providing recommendations for future research.

2

TEMPORARY SUPPORT SYSTEMS

2.1. PROPPING SYSTEMS

Propping systems are essential components in construction, providing temporary support to structures during various phases of building projects. These systems come in a variety of forms and are used for different purposes, ranging from supporting formwork during concrete pours to stabilizing excavation walls.

In bottom-up construction sequences, temporary props are extensively employed. These props typically consist of steel sections, such as a tubular, box, and universal columns. However, there's a growing trend towards utilizing high-capacity modular reusable systems as a more sustainable alternative. These modular systems often incorporate hydraulic actuation, allowing for on-site adjustment and pre-stressing. In top-down construction, permanent propping is primarily provided by concrete floor slabs. Nevertheless, steel props may also be utilized, especially in complex top-down sequences or where additional support is required. Floor slabs are generally robust enough to support temporary loads during construction, but careful consideration is necessary to establish a robust load path, particularly when the slab changes in level. Figures 2.1 and 2.2 are examples of two different retaining systems.



Figure 2.1 – View of the Almedal excavation site

While the cost of temporary propping systems is usually small compared to other construction expenses, the impact of poorly planned and executed temporary props can be significant. Delays and disruptions caused by inadequate propping can lead to increased costs and project setbacks. Therefore, it's crucial to incorporate temporary propping activities explicitly into the design, planning, and programming of construction works.

Efficient design of propping systems is important, but significant cost savings can only be achieved if construction programs are effectively managed. Economy can be achieved through measures such as reducing the number of props or prop levels, utilizing optimization methods. Temporary props are typically robustly designed due to the risks associated with their use. While failures are rare, they can occur due to poor detailing, misjudgment of ground conditions, or accidental damage. Proper design and consideration of these factors are crucial to prevent potential failures and ensure the safety and stability of the structure.

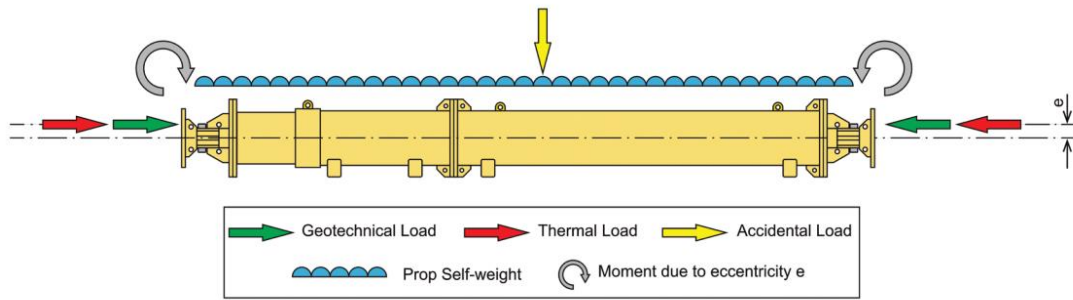


Figure 2.2 – View of the Korsvägen station excavation site (Kasper T., 2023)

2.2. DESIGN BASIS

The design of temporary prop systems is a critical aspect of construction projects, ensuring structural stability and safety during various phases of building. Key considerations in this process include load calculation, analysis methods, site conditions, safety factors, comparison with experience, and adjustment for optimization.

Load calculation is the foundation of prop system design, involving the determination of loads from formwork, concrete, equipment, and other factors. Analysis methods, such as limit equilibrium or Soil-Structure Interaction (SSI) methods, impact calculated prop loads. Adjustments may be necessary to account for underestimation, with increases of 15% for multiple props and 30% for single props commonly applied. Figure 2.3 exemplifies the loads to be used in the design of struts.



Notes

- 1 The characteristic prop self-weight is taken as its total weight applied across its entire length as a uniformly distributed load (UDL) and is considered as a permanent (G_k) action.
- 2 The characteristic geotechnical loading is considered as a permanent ($G_{k,GEO}$) action.
- 3 The characteristic thermal loading (see Box 8.1) is derived from a temperature increase, and a coefficient of wall restraint, and is treated as a variable ($Q_{k,temp}$) action.
- 4 A characteristic accidental point load (typically 10 kN to 50 kN from Williams and Waite, 1993) should be positioned to act at the worst case location. When considering buckling, this is usually acting vertically at mid-span. Accidental loading is considered as a variable ($Q_{k,accidental}$) action.
- 5 Unless moment-free bearing details are employed, an eccentricity of load application (e) should be considered. The magnitude of this eccentric load can be taken as equal to 10 per cent of the height of the bearing plate in contact with the wall (or waling beam), unless twin waling beams are used.

Figure 2.3 – Schematic diagram of actions to be considered for prop design (Gaba et al., 2017)

Site-specific factors, like soil type and ground conditions, must be carefully evaluated to ensure the prop system can effectively handle the environment. Safety factors are then applied to provide a margin of safety against uncertainties and unexpected loads.

Comparison with experience and field measurements helps validate calculated prop loads, with discrepancies prompting thorough investigation and adjustment as needed. Ultimately, the goal is to optimize the design for safety and efficiency, ensuring that temporary prop systems can reliably support construction activities without compromising structural integrity.

The specific details of the design of the struts analysed in this thesis will be further explained in section 3.3.4.

2.3. TEMPERATURE EFFECTS

During construction, variations in air temperature will occur, increasing and decreasing the prop installation temperature causing the strut to expand and contract. The relationship of this change in length can be portrayed by:

$$\Delta L = \alpha \cdot \Delta T \cdot L \quad (2.1.)$$

Where

ΔL = change in prop length

α = thermal coefficient of expansion for prop material

L = prop length

ΔT = change in prop temperature from the installation temperature

The range of ΔT may be influenced by different factors such depth of the excavation (higher level props will have a bigger fluctuation than lower-level props), presence of a roof slab (may cause a thermal dampening effect of the box), colour of strut, etc.

When the prop is restricted from expanding freely it may cause an additional load to appear in the prop. This load can account for a significant part of the struts design load and for a uniform cross section steel prop it can be given by the following:

$$N_{temp} = \alpha \cdot \Delta T \cdot E \cdot A \cdot \left(\frac{\beta}{100} \right) \quad (2.2.)$$

Where

E = Young's modulus of the prop material

A = cross-section area of the prop

β = restraint coefficient of the prop (%)

The value of β should be selected carefully and appropriately by the designer to suit the circumstance of each project.

The restraint coefficient is no more than a simplified manner to try and quantify how much the soil-retaining wall system allows the strut to expand when subjected to heat. In the context of equation 2.2, it represents the percentage of the temperature effect to be considered in the strut's behaviour. A system with complete freedom to expand has a restraint coefficient of 0%, indicating no increase in load. Conversely, a system that cannot expand at all is 100% restrained, indicating maximum load increase. The extent of restraint in a system is influenced by factors such as the stiffness of the surrounding soil and the connections between the prop, wall, and soil mass. In practical situations, such as those discussed in this thesis, the restraint coefficient is likely to fall somewhere between these two extremes.

A β of 70% indicates a very rigid wall-soil system, which allows very little deformation of the strut and results in a significant increase in load due to temperature (70% of the maximum value). In contrast, a lower β value, such as 20% or 30%, indicates a more flexible wall, which allows more freedom for the strut to expand and contract, resulting in lower temperature-induced loads.

2.4. CIRIA RECOMMENDATIONS FOR RESTRAINT COEFFICIENT

The restraint coefficient utilized for additional loads in props induced by temperature fluctuations is a critical aspect of structural design, significantly impacting the safety and cost-effectiveness of construction projects. The recommendations provided by CIRIA in their C760 guideline, as of 2017, offer insights into determining this coefficient based on wall flexibility and ground conditions.

According to CIRIA, for stiff walls situated in stiff ground conditions, a restraint coefficient of 50% is suggested, while for flexible walls in similar ground conditions, a coefficient of 30% is recommended. These recommendations, derived from field case studies, aim to provide a balance between safety and economy in design. Previous versions of CIRIA guidelines, such as CIRIA C580 (Gaba et al., 2003), were even more conservative, advocating 70% on stiff walls and 50% on flexible walls.

The assessment of various field cases, predominantly situated in the UK., has been instrumental in formulating these recommendations. However, it's essential to acknowledge that these guidelines may not encompass the full range of construction scenarios, potentially leading to overly cautious designs. Factors such as depth and other site-specific conditions could also influence the restraint coefficient, as highlighted by Powrie and Batten (2000b).

Studies by Richards et al. (1999), Batten and Powrie (2000), Ivanova (2012), and Chambers et al. (2016) shed further light on the complexities involved in determining this coefficient. Temperature variations have been observed to exert significant effects on prop loads. The restraint coefficient provided by the wall, installation temperature, and load in the prop are among the primary factors influencing the impact of temperature on prop loads.

Powrie and Batten's research highlights that temperature-induced loads can account for up to 40% of the total load in some cases. Factors such as the stiffness of adjacent ground and support system geometry play pivotal roles in determining the actual restraint coefficient.

“The stiffness of the propping system was back calculated and compared with the design value. It was found that the system stiffness averaged 35%, compared to the 70% recommended in the design guide, Ciria C580. The combination of these factors resulted in a maximum measured thermal load of approximately 1000 kN, compared with a design thermal load of 3202 kN.”, (Chambers et al, 2016).

In conclusion, while CIRIA recommendations offer valuable guidance for determining restraint coefficients, it might be wise to consider site-specific conditions and recalibrate design values based on empirical data. This approach may ensure structural integrity while optimizing project costs.

3

CASE STUDY

3.1. PROJECT INTRODUCTION

The West Link Project in Gothenburg, West Sweden, aims to modernize the city's transportation network to accommodate its growing population and economy. Gothenburg, the second-largest city in Sweden, is strategically located on the country's western coast, serving as a vital hub for trade, commerce, and industry, as showed in Figure 3.1. With a bustling port and thriving business sectors, Gothenburg faces significant traffic congestion and transportation challenges, especially in its city center. To address these issues and support sustainable urban development, the West Link Project was initiated as a comprehensive solution to improve public transit and connectivity within the city.

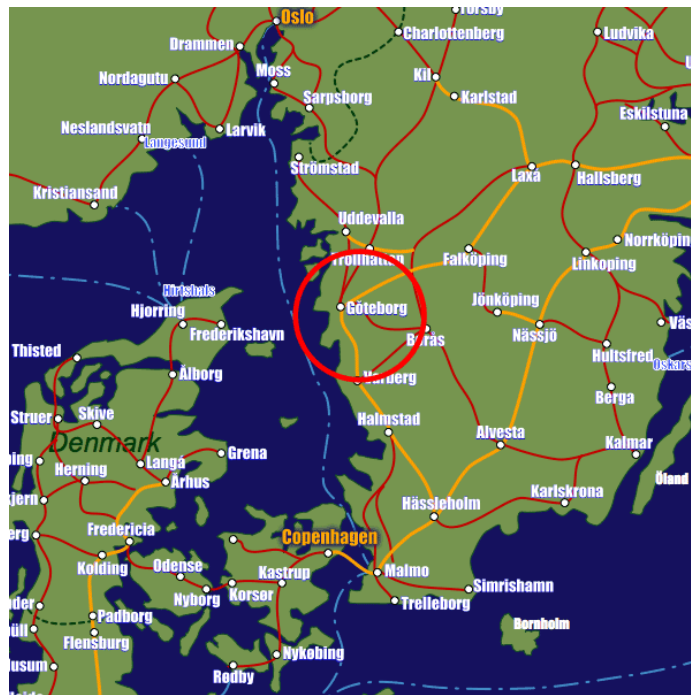


Figure 3.1 – Overview of the city of Gothenburg

At its core is the construction of a new railway tunnel, the West Link, which will run underground beneath the city centre, connecting Gothenburg's central station with new underground stations to the west and south. The project aims to increase the capacity of Gothenburg's public transit system by

expanding the rail network and creating new underground stations. Figure 3.2 illustrates the new lines and stations to be constructed.



Figure 3.2 – Map overview of West Link Project (Trafikverket - West Link Project)

This project is an eight-kilometer-long double track railway, including a six-kilometer railway tunnel, underneath Gothenburg. Three new underground stations will be created in the city; at Gothenburg Central Station the existing above-ground terminus will be changed to an underground transit station, and the stations at Haga and Korsvågen will be built. Figure 3.3 shows a satellite image of where the new Korsvågen station will be constructed.



Figure 3.3 – Overview of the Korsvågen construction site (Trafikverket - West Link Project)

3.2. GEOLOGICAL CONDITIONS

The geological conditions on the sites where the excavations are located are a good representation of a typical Gothenburg soil, which is a superficial fill layer, followed by soft, sensitive Gothenburg clay, followed by sandy/gravelly moraine and finally strong granite bedrock, as schemed in Figure 3.4. The excavations can reach up to 21 m in some locations. The stratification of the soil layers can differ extremely in separate locations, having some places where the soft clay layers are 5 m thick and others where it can extend for over 100 m. In typical situations the thickness of the layers can be the following: the fill material can go from 0 to 4 m; the soft clay layer goes from 5 to 20 m and the moraine can go from 2 to 10 m.

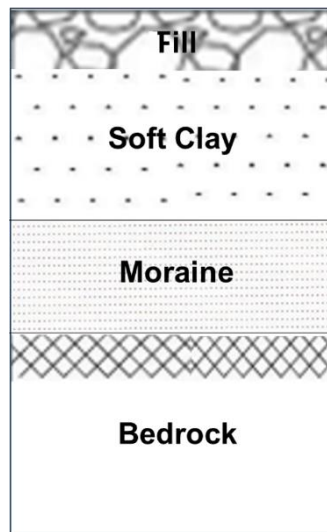


Figure 3.4 – Scheme of typical Gothenburg soil profile

3.2.1. GOTHENBURG CLAY

The Gothenburg soft clay is one of the geological challenges of this project. This material, when untreated, is extremely poor for excavations and construction given its low stiffness and propensity for large deformations, as illustrated in Figure 3.5. In many situations this type of clay can present thixotropy behaviour and can potentially liquify. Many definitions of thixotropy exist, but the following seems to be quite relevant to this case “the property of a material that enables it to stiffen in a relatively short time on standing, but upon agitation or manipulation to change to a very soft consistency or to a fluid of high viscosity, the process being completely reversible” (ASTM (D653-14), 2014).

For example, at Korsvägen central station the clay can be found to have properties such as: water content, $w_n = 75\%$, an undrained shear strength, $c_u = 25$ to 33 kPa, a sensitivity ranging from 20 to 150 and $M_0 = 9$ to 19 MPa and $M_L = 0.75$ to 1.3 MPa for the untreated clay. These types of clay are many times also referred as “Quick clays” which are “defined as a clay with a sensitivity of 50 or more and a fully remoulded shear strength of less than 0.4 kPa” (Rankka, K. et al. 2004).

With that it is necessary to improve the soil and for that it was utilised a technic known as lime-cement stabilisation. With this process it can be achieved an economic design of the retaining walls, and it is possible to minimize wall deflections and associated settlements.



Figure 3.5 – Example of sensitive, soft clay

Lime-cement stabilisation of the clay is always necessary when it comes to this soil conditions to ensure a safe excavation and transport of the clay. This process is extremely well-known, and it has been proven many times to be quite effective, improving considerably both the strength and stiffness of the soil. In Scandinavian countries, such as Sweden, where similar ground conditions are found in many different places this technique has been perfected and it can be done quite successfully. Figure 3.6 shows an excavation made in soil after improvement with lime-cement.



Figure 3.6 – Lime-cement stabilisation

The method of soil stabilization known as lime-cement piles involves the *in situ* mixing of quicklime or cement with soft soil, as illustrated by Figure 3.7. This soft soil can be composed of materials such as clay, silt, or organic soils. During the process, a rotating mixing tool is used to penetrate the ground and effectively reshape the soil. Once the desired depth is reached, the appropriate combination of lime, lime-cement, or cement is introduced and mixed with the stabilizing soil as the tool continues to rotate and ascend to the surface.

Since its introduction in the 1970s (Broms & Boman 1975), the utilization of lime-cement columns has steadily grown. This technique is commonly applied in areas where soft marine deposits are prevalent,

such as port and harbour regions. The primary objectives of this method are to enhance load-bearing capacity and minimize ground settlements. Moreover, its application has expanded to include embankments, roadways, and foundations located in soft ground conditions.

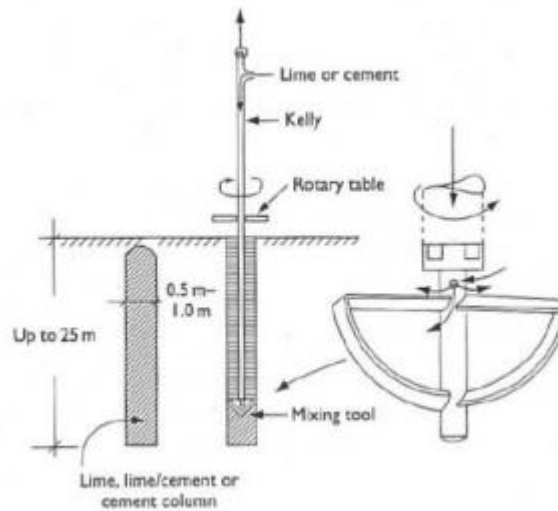


Figure 3.7 – Execution lime-cement columns or cement piles (Moseley & Kirsch, 2004)

The improvement of soil is primarily achieved through chemical reactions that take place between lime, cement, and soil particles. Quicklime reacts with fine clay particles, causing them to clump together in a process known as flocculation. Cement, on the other hand, reacts with soil particles and helps to bind them, resulting in the aggregation of granular materials.

For soils with high organic content, lime-cement mixes are often recommended when other mixtures are unable to achieve the desired shear strength. The main objective of employing LC (Lime-Cement) piles in soil improvement is to enhance the bearing resistance and shear strength of the soil.

In this specific project of the Lime cement block stabilization was used throughout the whole clay volume in the construction pit and 1m outside of the construction as block stabilization with 800mm columns with c/c spacing 750mm (triangular column pattern, with 50mm overlap), as illustrated in Figure 3.8. With the objective of: Increasing passive resistance, reducing wall deflections, and optimizing retaining wall system and ease excavation and muck handling / transport.

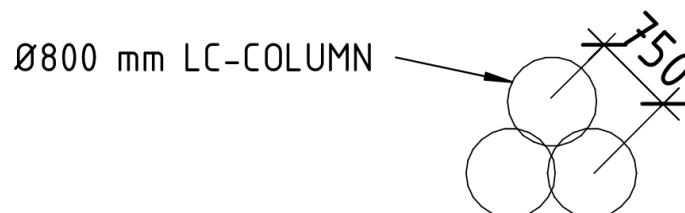


Figure 3.8 – Lime-cement columns distribution

3.2.2. MORaine

Moraine soil is a type of soil formed by the deposition of material carried and deposited by glaciers. Glaciers are large bodies of ice that move slowly over land and as they advance and retreat, they deposit this material in ridges, mounds, and irregular formations, creating moraines.

Characteristics of moraine soil can vary widely depending on factors such as the composition of the material carried by the glacier, the distance it travelled, and the processes of deposition. However, some general characteristics of moraine soil include mixed particle sizes, variable texture, erratic distribution, poorly sorted and variable drainage.

In the case study the moraine soil exhibits significant heterogeneity, varying from fine silty sand to gravel, and occasionally containing boulders up to 3 meters in diameter. When encountering larger boulders, the primary challenge lies in distinguishing them from the bedrock and effectively penetrating them with drilling equipment. Despite these occasional obstacles, overall penetration of the moraine was proven relatively straightforward using specialized foundation methods such as secant pile walls and RD pile walls.

3.2.3. ROCK

This geological layer is the most challenging and it makes the solutions utilised in the construction quite unique. The rock mass is mostly gneiss-metagranites and some diabase dikes which are intersecting the rocks. The rocks exhibit a wide range of strengths and varying degrees of weathering and fractures, adding complexity to the execution of retaining structures.

Several features of the rock mass contribute to these challenges:

- The presence of alternating and often steep rock surfaces, which were difficult to accurately define during pre-investigations and posed challenges during the installation of retaining walls.
- Fractures and fault zones within the rock mass exhibit low strength and high-water conductivity, complicating construction efforts.
- The upper bedrock consists of highly weathered rock, presenting challenges for rock curtain grouting and the installation of toe anchors.

With the aim of surpassing this challenges a 3D model of the rock mass was made using data from a retrieved from soil-rock probing's, also known as Jb-soundings, which were conducted at approximately 4-meter intervals. Rock cores were also extracted to assess the quality of the rock.

Soil-rock probing, referred as Jb-sounding, is the most common probing method used in Sweden. Classified as a dynamic method mainly used to assess the subsurface conditions and determine the transition between soil and rock layers, Figure 3.9 shows the procedure of how this method is executed. The method consists of hydraulically drilling in the unconsolidated sediment by using a rotary hammer and measuring the penetration resistance. Three types of Jb-soundings exist: Jb-1, Jb-2 and Jb-3, each of them measuring and recording different parameters:

- Jb-1 – Depth and Penetration resistance
- Jb-2 – Depth, Penetration resistance, Rate of penetration, Force input, Rate of revolutions, Hammer pressure and Rotational pressure (pressure in turning engine).
- Jb-3 – Depth, Penetration resistance, Rate of penetration, Force input, Rate of revolutions, Hammer pressure, Rotational pressure (pressure in turning engine), Flushing media pressure and Flushing media flow.

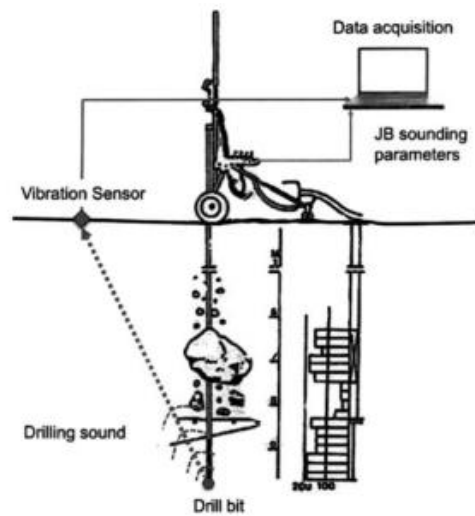


Figure 3.9 – Acoustic measurements during Jb-probing (Kalm, H. 2019)

This method can provide an estimate of the existing rock surface. This initial estimate while reasonable for an initial assessment also provides some considerable inaccuracies. To that end, the rock level after final excavation was remeasured and sensitivity studies were made from the design models to verify the safety given the actual rock levels. Figure 3.10 illustrates the rock level found after excavation.



Figure 3.10 – Retaining wall/Rock surface interface, north wall Korsvägen 2 (Kasper T. 2023)

Investigations of the rock mass quality are essential as the forces from the retaining walls will be transmitted into the rock mass and the area is highly sensitive to water loss. Rock core drilling was therefore performed to enable characterization and classification of the rock mass.

Rock mass quality was calculated per meter, using a classification named Rock Mass Rating or RMR, the results represent averages for each meter. The RMR classification was created by Bieniawski (1989) is based on the principle of assigning weights to the six parameters considered to contribute more significantly to the behaviour of the rock mass. The sum of the weights assigned to each of these parameters is an index, which corresponds to one of five quality classes of rock masses, considered by the author. The parameters are the following:

- Intact Compressive Strength of Rock
- RQD (Rock Quality Designation)
- Spacing of Discontinuities
- Condition of Discontinuities
- Groundwater conditions
- Orientation of Discontinuities

The final result may be a value ranging from 0 to 100. Based on the value the rock classified as defined in Figure 3.11. The rock quality of from the different sites studied in this case usually ranged from good to fair.

RMR _{base} -value	Class	
81-100	Very good	Green
61-80	Good	Light Green
41-60	Fair	Yellow
21-40	Poor	Orange
0-20	Very poor	Red

Figure 3.11 – RMR classification (Bieniawski, 1989)

3.3. STRUCTURAL ELEMENTS

The retaining system utilized in the different excavations sections were chosen to better full fill the necessities and requirements of the ground conditions at each location excavated. To the that effect different types of walls were used when there use suited the situation, namely were installed Secant piles, RD piles, and Sheet pile walls. These walls are sometimes called embedded walls due to the fact that they have a significant part of their height beneath the excavation base. The design of these structures will depend on its complexity. For simpler solutions such as cantilever or single-propped walls the design can be performed by simple limit equilibrium methods (Matos Fernandes M. 2020). For more complex structures, such as multi-propped or multi-anchored, the design has to be made with the support of numerical models.

3.3.1. SECANT PILES

Secant pile walls are a type of retaining wall system constructed by drilling concrete piles into the ground. The term "secant" refers to the fact that the piles are constructed in such a way that they intersect or overlap with one another. This overlapping arrangement creates a continuous barrier that can withstand lateral earth pressure and prevent soil or water from infiltrating the excavated area.

Secant pile walls are typically composed of two types of piles: primary piles and secondary piles. Primary piles are installed first, followed by secondary piles, which are constructed in between the

primary piles, as illustrated in Figure 3.12. The primary piles may be comprised of unreinforced low-strength concrete, unreinforced or reinforced structural concrete. The secondary piles always comprise reinforced structural concrete.

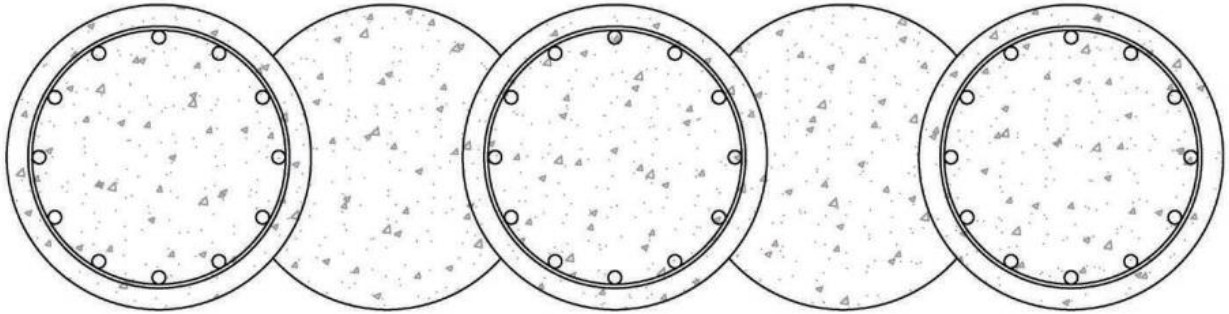


Figure 3.12 – Secant piles sections (Gaba et al., 2017)

Installing the secant pile walls in the existing soil conditions presented, i.e., clay, moraine, and rock, required particular solutions. Figure 3.13 shows an example of the secant piles at the Korsvägen excavation site.



Figure 3.13 – Example of secant piles, Korsvägen 1

3.3.2. RD PILES

RD drilled piles are special piles, installed by drilling in difficult ground and environmental conditions. They are steel pipe piles with different sizes ranging from the RD220 up to RD1200 which are spirally or longitudinally welded. RD piles are interlocked to each other's by welded connectors, creating a continuous pipe pile retaining wall against horizontal and vertical loads.

The RD pile wall is always installed by percussion drilling methods, using a DTH hammer. The drilling method is concentric. Figure 3.14 illustrates this installation process.

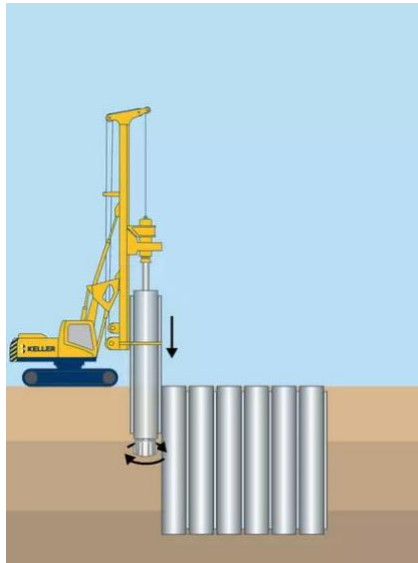


Figure 3. 14 – Drilling of RD interlocked piles

Adjacent piles are always interlocked using a pair of a narrow and a wide connector. The connector type specially developed for this type of pile. The connector and the injection channel integrated in it, shown in Figure 3.15, ensure the water tightness of the bottom of the pile wall and its rigid contact with rock without separate injection pipes.

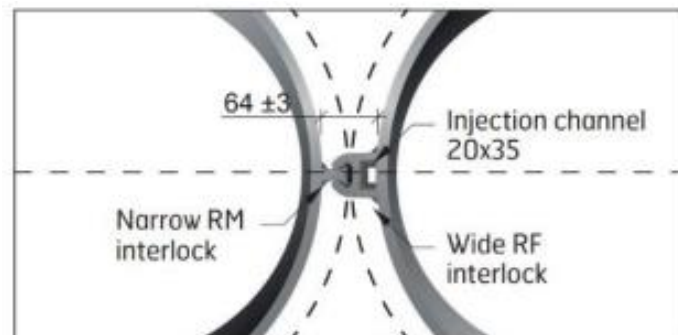


Figure 3.15 – RM/RF interlocking (SSAB)

The SSAB RM/RF connectors can be used with pile sizes RD220 to RD1200.

Considering that RD piles were specifically engineered for challenging ground conditions, it's evident that installing them in existing clay, moraine, and rock offers the potential to create the highest quality retaining structure. However, this construction method entails relatively high material costs and necessitates special measures to ensure successful implementation. RD pile wall is specially designed for challenging conditions, for instance, for faster implementation of retaining walls and other foundation structures. Figure 3.16 shows an example of the RD piles at the Korsvågen excavation site.



Figure 3.16 – Example of RD piles, Korsvägen 2

3.3.3. SHEET PILES

A sheet pile wall is a versatile and efficient type of retaining wall. It consists of a series of interlocking steel or concrete sheets that are driven or vibrated into the ground vertically to create a continuous barrier. These sheets are typically designed with a corrugated or flat profile, allowing them to effectively withstand lateral pressures from soil or water.

Sheet pile walls are commonly used in situations where conventional retaining walls may not be feasible due to space constraints, soil conditions, or water presence. They are frequently employed in waterfront developments, foundation construction, bridge abutments, flood control projects, and excavation support systems. The interlocking nature of the sheets ensures structural stability and creates a watertight barrier when used in water-retaining applications.

One of the key advantages of sheet pile walls is their flexibility and adaptability to various site conditions. They can be installed in tight spaces and can be driven to considerable depths to provide adequate support against lateral soil pressure.

There are numerous variations of sheet pile walls, distinguished by their section shapes and materials. In the past, sheet piles crafted from wood, aluminium, and even vinyl were utilized. Today the most commonly used sheet piles are made of steel and the predominant sections utilised are the U and Z sections, as showed in Figure 3.17, although several other types of sheet pile sections exist.

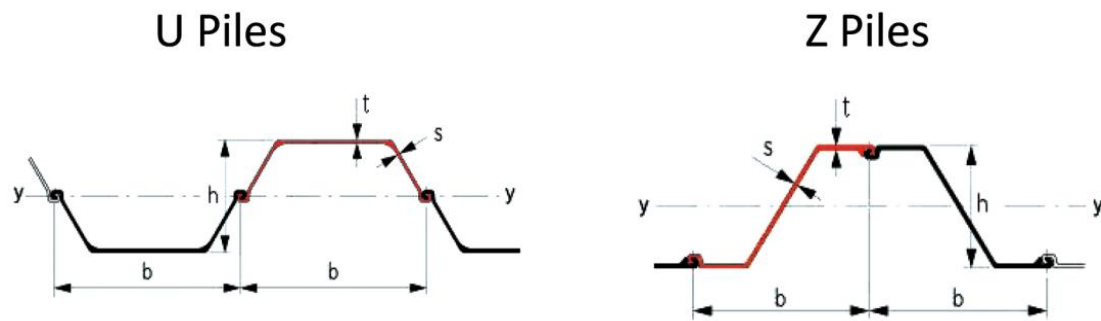


Figure 3.17 – Secant pile sections (Gaba et al., 2017)

The key factor that distinguishes these sections is their interlocking mechanisms, which are responsible for connecting individual piles to create the complete sheet pile wall. Typically, these interlocks are mechanically crimped together to ensure a secure connection between piles. In some cases, welding may also be employed for additional reinforcement.

In U sections, the interlocks align with the neutral axis, where maximum shear stresses occur. On the other hand, in Z sections, the interlocks are positioned in areas with minimal shear stresses. As a result, U profiles exhibit reduced efficiency and section properties compared to Z profiles. Currently, Z profiles are preferred, and their lengths can be tailored for specific locations and requirements.

The installation process of sheet pile walls involves driving or vibrating the sheets into the ground using specialized equipment such as pile drivers or hydraulic presses, as illustrated by Figure 3.18. This ensures that the sheet piles are securely embedded in the soil. Figure 3.19 shows an example of the RD piles at the Almedal excavation site.

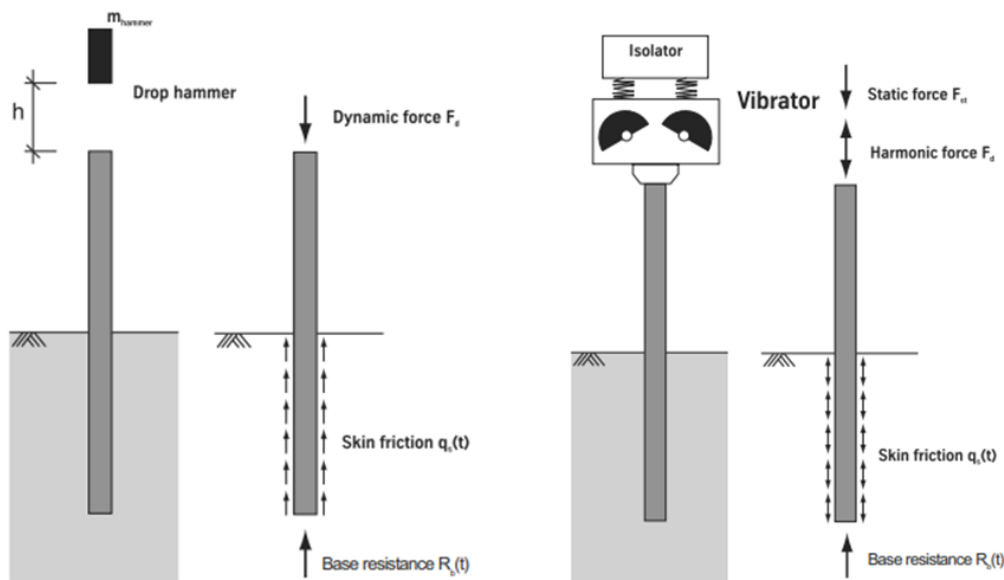


Figure 3.18 – Pile driving methods, impact (left) and vibration (right) (Thyssen Krupp, 2010)



Figure 3.19 – Example of sheet pile wall, Almedal

3.3.4. STRUTS

As was introduced in section 2.2 the design of the steel props will follow the guides of the CIRIA guide C760 along with the Eurocodes SS-EN-1990 and SS-EN-1995. The loads applied to the systems in the design are as presented in Figure 3.20.

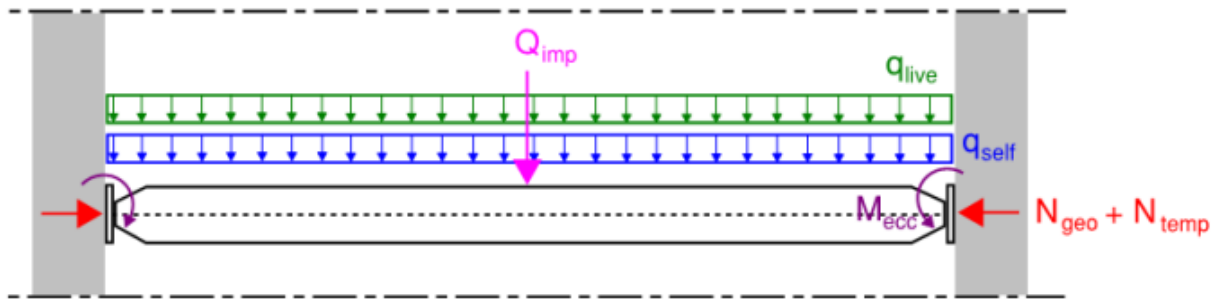


Figure 3.20 – Loads applied for strut design

N_{geo} characterises the geotechnical load and it includes soil pressure, water pressure, pre-stressing, and surcharge.

N_{temp} is the load due to temperature variations, given by:

$$N_{temp} = \alpha \cdot \Delta T \cdot E \cdot A \cdot \left(\frac{\beta}{100} \right) \quad (3.1.)$$

Where

- E = Young's modulus of the prop material
- A = cross-section area of the prop

- α = thermal coefficient of expansion for prop material, i.e. 1.2×10^{-5} per °C for steel
- ΔT - Temperature difference from the installation temperature. Calculated based on SS-EN 1991-1-5 section 6.1.3.3 (3).

$$\Delta T_{e,exp} = T_{e,max} - T_0 \quad (3.2.)$$

According to Annex A.1 of the Eurocode SS-EN-1991-1-5, the initial temperature (T_0) can be taken as 10 °C.

As per Trafikverket's regulations (TRVFS 2011:12), Appendix 2, Table 2a, the maximum shade air temperature in the Gothenburg area is $T_{max} = 35$ °C.

Additionally, Figure 6.1 in SS-EN 1991-1-5 indicates that the maximum uniform bridge temperature component is $T_{e,exp} = 51$ °C (Figure 3.21). In this case, Type 1 should be utilized since the struts are made of steel material.

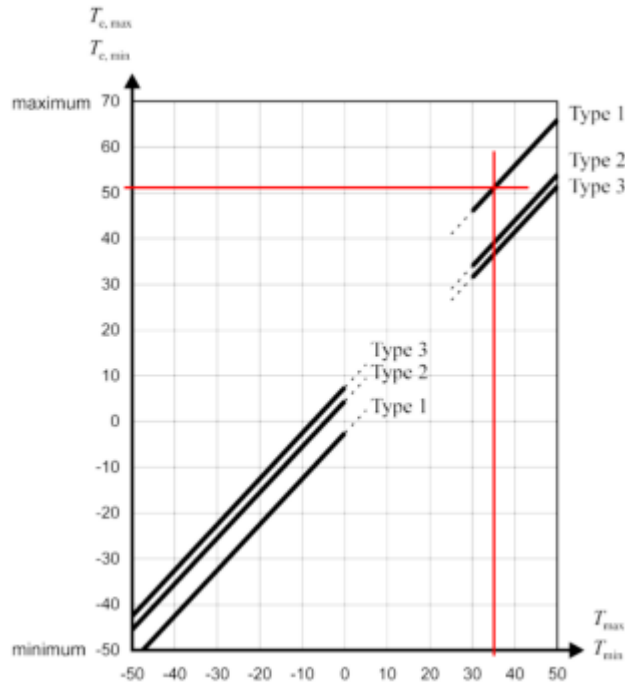


Figure 3.21 – Correlation between minimum/maximum shade air temperature and minimum/maximum uniform bridge temperature component (EN 1991-1-5 2003)

With this the characteristic maximum expansion temperature component becomes:

$$\Delta T_{e,exp} = 51 - 10 = 41^\circ\text{C} \quad (3.3.)$$

- β is the restraint coefficient for the strut. For the conditions at Korsvägen I, Korsvägen II and Almedal β is assumed to be 30 %. As per the CIRIA C760 recommendations.

Consequently, the additional axial load on the strut is determined as follows:

$$N_{temp} = \alpha \cdot \Delta T \cdot E \cdot A \cdot \beta = 1.2 \cdot 10^{-5} \cdot 41 \cdot 210 \cdot 10^6 \cdot A \cdot 0,3 = 31 \cdot A \text{ (kN)} \quad (3.4.)$$

According to Trafikverket's project specifications failure of the strut should not be considered in the design, instead it is necessary to design the struts for an accidental impact represented by a point load, $Q_{imp,ALS} = 100kN$, applied at the center of the strut span. In ULS conditions, the struts will be designed for an impact load of $Q_{imp,ULS} = 25kN$.

Additionally, apart from the self-weight of the strut, q_{self} , a vertical live load of $q_{live} = 1kN/m$ is applied to account for external loads resulting from site operations.

The combinations utilised to verify the struts, according to Trafikverket's regulations (Trafikverket 2014), were the following:

- ULS: $1.5 \cdot N_{geo,k} + 1.5 \cdot 0.6 \cdot (N_{temp} + q_{live})$ (3.7.)

- ULS: $0.85 \cdot 1.5 \cdot N_{geo,k} + 1.5 \cdot (N_{temp} + q_{live})$ (3.6.)

- ALS: $N_{geo,k} + Q_{imp,ALS} + 0.5 \cdot (N_{temp} + q_{live})$ (3.8.)

The self-weight, q_{self} , the vertical live load, q_{live} , and the bending moment from the eccentricity are applied in all load combinations.



Figure 3.22 – Example of struts used on Korsvägen

The struts in each of the constructions sites and for each level differ from each other having different sections and different design loads. Table 3.1 presents the struts properties considered in this study.

In Korsvägen 1:

- Level 1 – Type 3
- Level 2 – Type 1 or 1a
- Level 3 – Type 1
- Level 4 – Type 2

In Korsvägen 2:

- Level 1 – Type 1 or 1a
- Level 2 – Type 2 or 2a
- Level 3 – Type 3 or 3a

Table 3.1 – Strut Properties

Korsvägen 1			
Strut Type	D (mm)	t (mm)	A (mm ²)
1	1219	25	93777
1a	1219	28	104766
2	1219	22,5	84576
3	914	22,5	63016
Korsvägen 2			
Strut Type	D (mm)	t (mm)	A (mm ²)
1	1420	25	109563
1a	1420	27,7	121161
2	1420	23	100943
2a	1420	23	100943
3	914	20	56172
3a	914	22,2	62197

Using equation (3.4) combined with the sections in the Table 3.1. it is possible to calculate the temperature increase to be considered in the design. For example: for strut rows 1 to 7 in Korsvägen 2 the types used 1, 2 and 3. With that $N_{temp,level1} = 1741$ kN; $N_{temp,level2} = 3155$ kN; $N_{temp,level3} = 3401$ kN.

3.5. MONITORING

In order to ensure the safety and stability of existing structures during construction, monitoring is essential to track any ground movements and settlements that may occur. To achieve this, monitoring instruments are strategically placed in selected locations. These instruments are used to monitor the expected settlement and response of temporary structures, as well as the surrounding ground and existing structures such as buildings, highways, railways, and bridges. The purpose of this monitoring is to prevent any harmful or unexpected deformations.

The monitoring of retaining structures includes various measurements and instruments:

- Inclinometers are used on RD, secant, and sheet pile walls to measure any tilt or displacement.
- 3D-Deformation targets, such as prisms, are installed on walls to track any deformations in three dimensions.
- Surface settlements are measured vertically to monitor any ground sinkage.
- Surface displacements are measured horizontally to monitor any lateral movements.
- Strain gauges are installed on struts to measure the strain and load on these structural elements.

In RD-piles, the inclinometer pipes are fixed by concreting them into the pile after installation, in previously identified pipe, Figure 3.23 (left). In sheet piles, the inclinometer pipes are welded to the sheet pile profile before installation, Figure 3.23 (right). In secant piles, inclinometer pipes are also attached to the reinforcement cages of specified secondary piles before casting.



Figure 3.23 – Example of inclinometers, sheet pile (left) and RD pile (right)

Deformation targets are installed during the excavation of the construction pit, specifically at each waling beam and the top of the pile.

Strain gauges are provided for the specified struts by attaching them to the steel pipe most often on both ends, and temperature measurements are taken in conjunction with the loads.



Figure 3.24 – Example of strain gauge

An example (Korsvägen 1) of locations of monitoring points on the retaining walls and ground surface can be found in Figure 3.24, while the locations of monitoring points on the struts are shown in Figure 3.25.

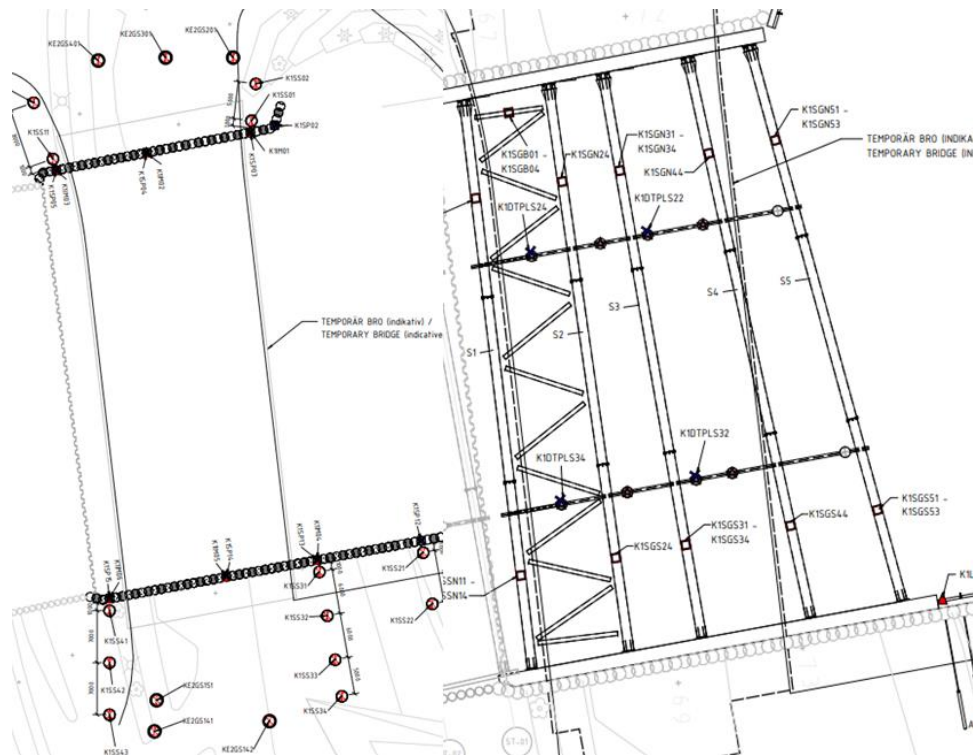


Figure 3.25 – View of monitoring points

To determine if any monitored values indicate potential issues, specific trigger levels for monitoring are defined for each section. These trigger levels consist of two values: the Alarm value and the Limit value. The Alarm value represents the threshold at which measured values are still within an acceptable range of deflection, settlement, or load. The Limit value represents the maximum values that should not be exceeded to prevent any damage to the structures.

Overall, this monitoring process ensures that any ground movements, settlements, or deformations are detected and addressed promptly to maintain the safety and integrity of the construction site and surrounding structures.

4

ANALYSES OF THE MONITORING DATA

4.1. INTRODUCTION

As mentioned earlier, the analysis serves three purposes. Firstly, it aims to determine the range of the restraint coefficient in typical Gothenburg ground conditions based on monitoring data. Secondly, it aims to identify the factors that have the greatest influence on the restraint of the struts. Lastly, it aims to evaluate whether Plaxis simulations can produce results comparable to the monitored data. In this chapter the first two objectives will be studied.

To achieve these goals, the following sections of the construction, shown in Figures 4.1 to 4.5, were examined:

- In the Korsvägen 1 construction pit, the first row of struts was studied.
- In the Korsvägen 2 construction pit, the second, fourth, sixth, eighth, and tenth rows of struts were studied. In addition, the deflections of the piles marked in Figure 4.1 and were analyzed throughout all phases of construction to ensure that the calculated wall deflection in the Plaxis model closely matched the measured data.
- In the Almedal construction pit the struts analyzed were named ST-6, ST-13, and ST-30

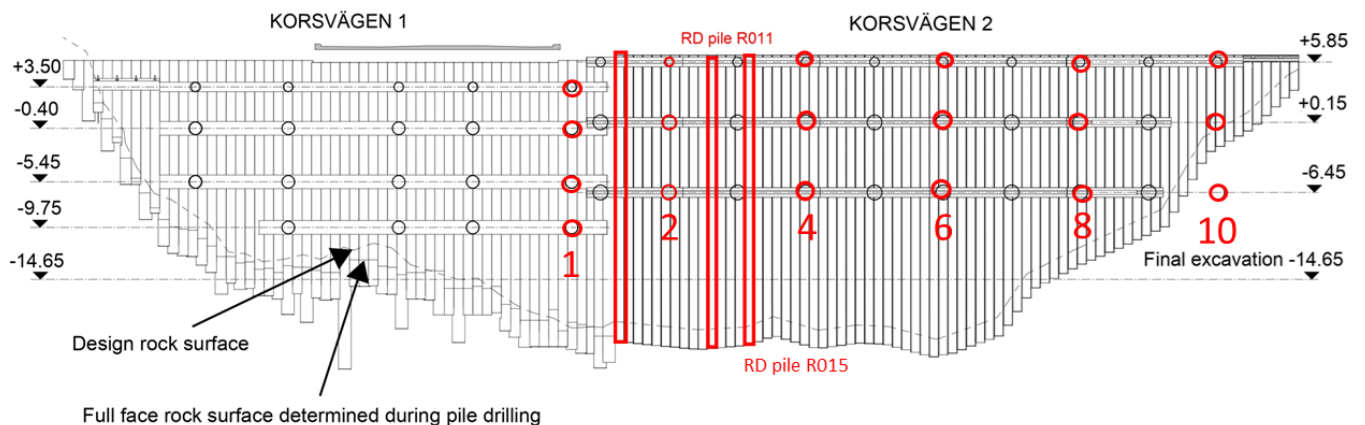


Figure 4.1 – Elevation view of Korsvägen 1 and 2 south wall (West Link Project)

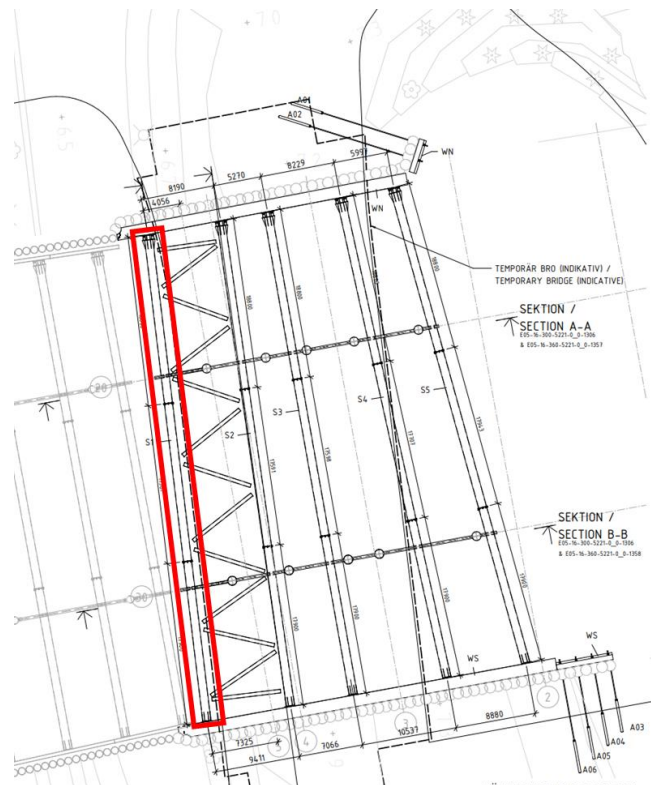


Figure 4.2 – Plan view of Korsvägen 1 (West Link Project)

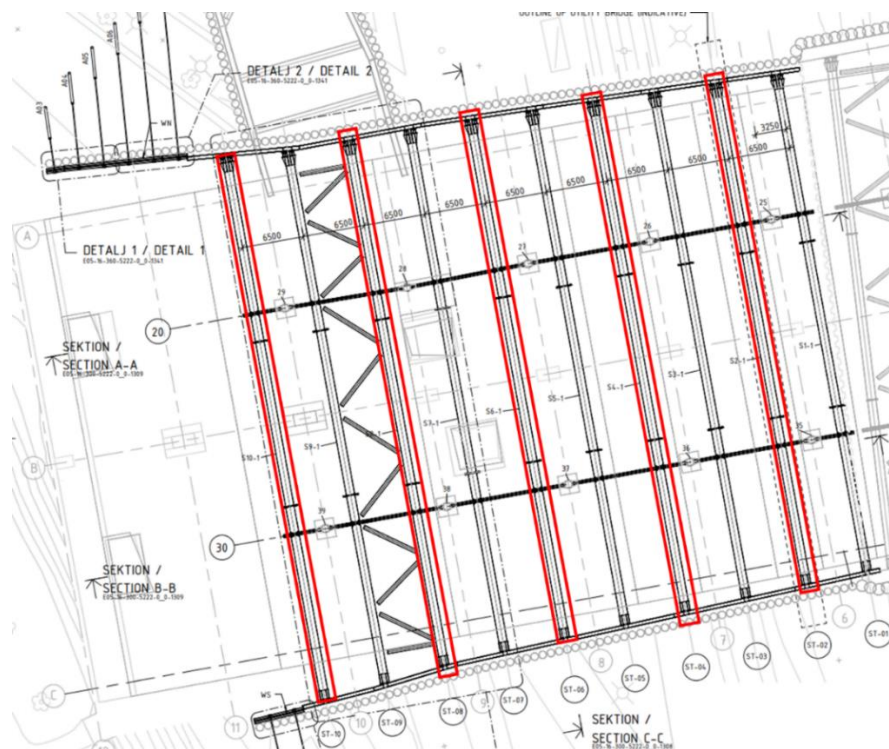


Figure 4.3 – Plan view Korsvägen 2 (West Link Project)

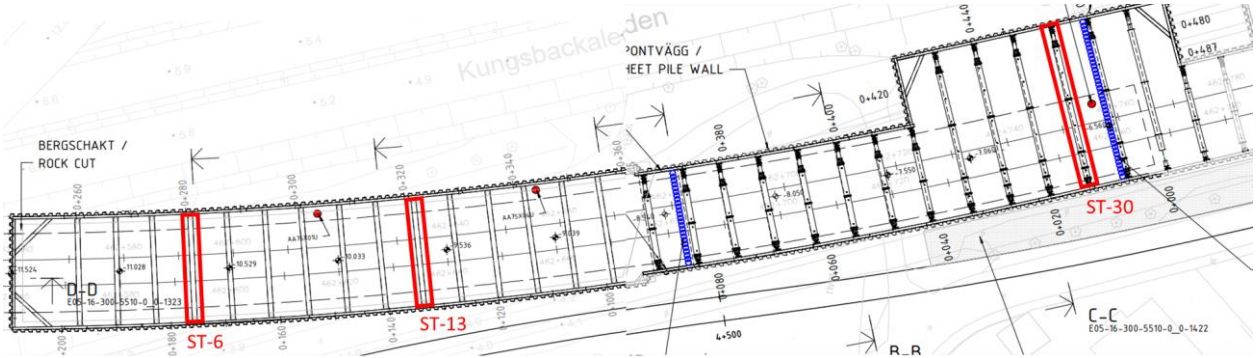


Figure 4.4 – Plan view Almedal (West Link Project)

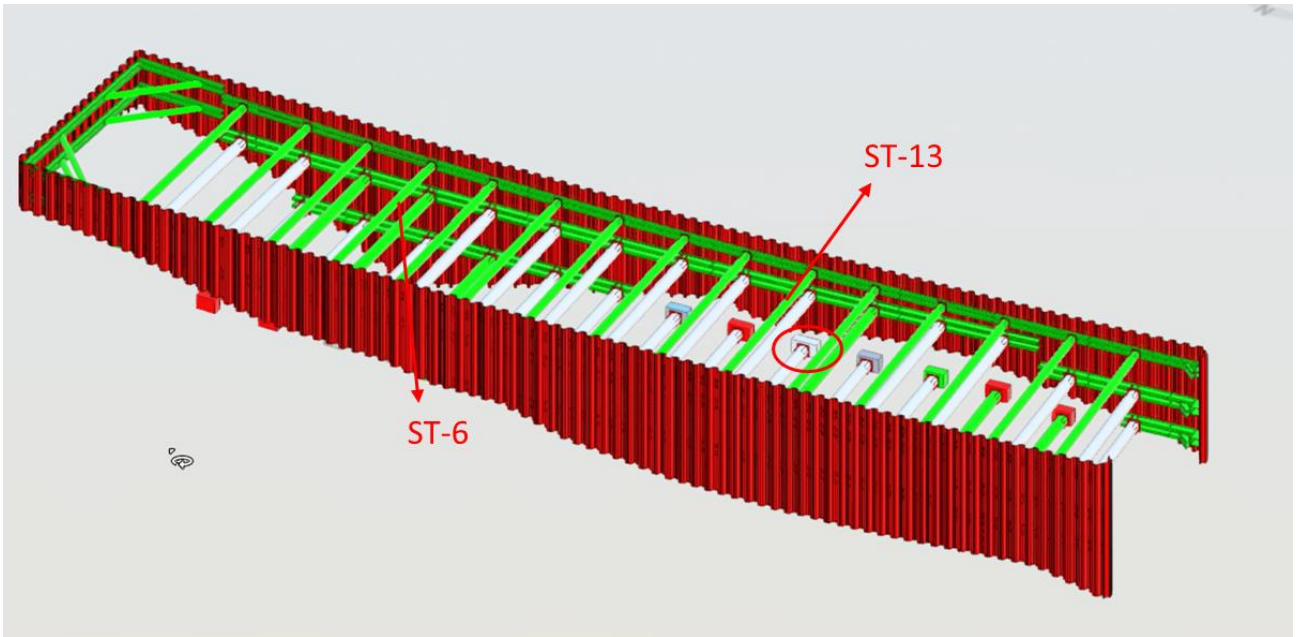


Figure 4.5 – 3D view Almedal (West Link Project)

4.2. APPROACH

Given that for each strut marked in the Figures above it was provided the hourly data for both the strut load and temperature since the installation until after the excavation had reached its final level.

The approach taken to analyze the monitoring data was simple and practical:

1. A period of at least 2/3 months after the installation of all strut levels and the excavation to the final dept was chosen.
2. Within this time interval it was calculated for each strut studied the daily difference between the maximum and minimum values of both the temperature variation, ΔT and the load, ΔN_{temp} .
3. The restraint coefficient for the daily maximum fluctuations was calculated using equation (4.1).
4. Lastly it was also calculated the restraint coefficient considering not daily values but the difference between maximum and minimum values for the whole period.

$$\beta = \frac{\alpha \cdot \Delta T \cdot E \cdot A}{\Delta N_{temp}} \quad (4.1.)$$

With these values it was possible to estimate the average restraint coefficient for each strut. Figure 4.6 shows an example group of struts monitored over time.

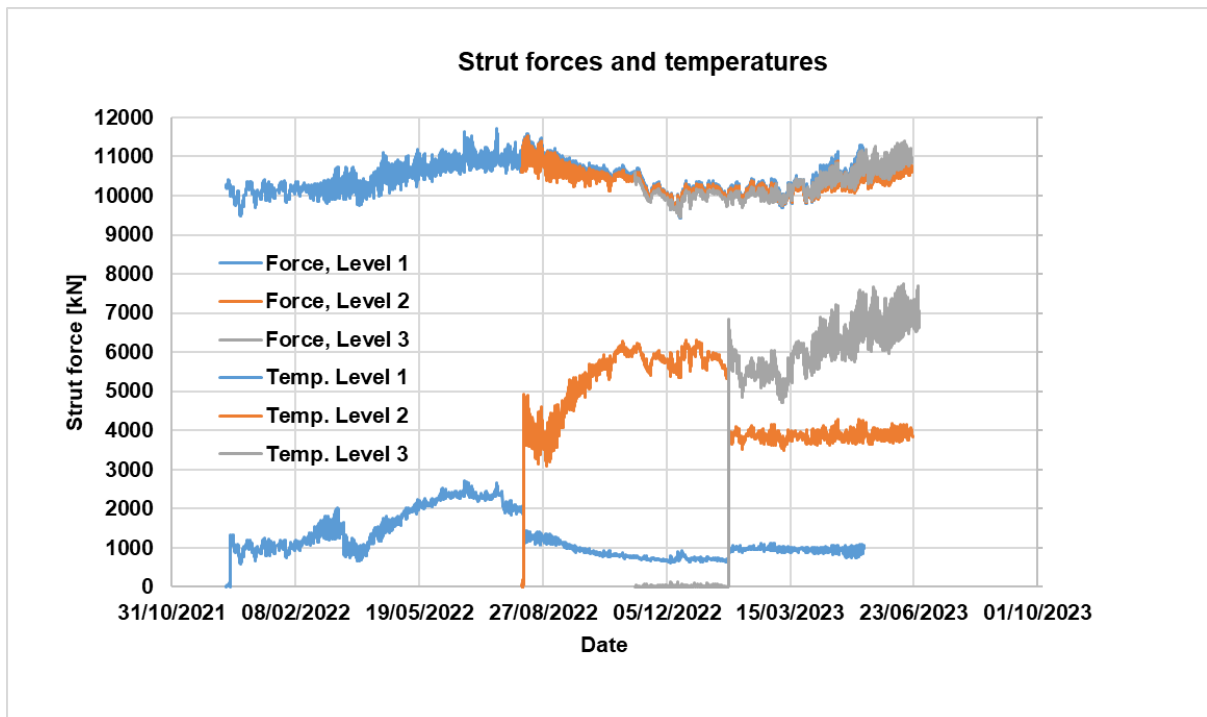


Figure 4.6 – Measured strut Forces

Table 4.1 – Example of hourly strut loads and temperatures measurements

Date	Load	Temperature
2023-05-08 00:00:00	6226	10,2
2023-05-08 01:00:00	6202	9,4
2023-05-08 02:00:00	6154	8,8
2023-05-08 03:00:00	6110	8,1
2023-05-08 04:00:00	6063	7,5
2023-05-08 05:00:00	6025	6,9
2023-05-08 06:00:00	6014	6,5
2023-05-08 07:00:00	6020	6,4
2023-05-08 08:00:00	6091	6,8
2023-05-08 09:00:00	6269	8,1
2023-05-08 10:00:00	6341	9,4
2023-05-08 11:00:00	6450	10,3
2023-05-08 12:00:00	6925	11,8
2023-05-08 13:00:00	6890	12,5
2023-05-08 14:00:00	7162	13,5
2023-05-08 15:00:00	7310	14,6
2023-05-08 16:00:00	7229	15,3
2023-05-08 17:00:00	7231	19,1
2023-05-08 18:00:00	7004	18,8
2023-05-08 19:00:00	6812	17,7
2023-05-08 20:00:00	6704	16,4
2023-05-08 21:00:00	6617	15,2
2023-05-08 22:00:00	6530	13,9
2023-05-08 23:00:00	6431	12,7

$$\beta = \frac{0,000012 \cdot (19,10 - 6,43) \cdot 210000000 \cdot 0,10972}{(7231 - 6020)} \cdot 100\% = 37,02\% \quad (4.2.)$$

Table 4.1 and equation (4.2) are an example of the calculation of the restraint coefficient on a specific day for strut level 3 row two on the Korsvägen 2 construction site.

4.3. RESULTS

4.3.1. AVERAGE RESTRAINT COEFFICIENT

The presented results in the following diagrams represent averages derived from a comprehensive range of values. Detailed charts and tables in the appendix provide a comprehensive overview, encompassing all the daily values calculated for each individual strut. These additional resources may offer a more comprehensive understanding and allow for detailed analysis beyond the average values presented in the main diagrams.

Figures 4.7, 4.8, and 4.9 present the results for Korsvägen 2, KV2, displaying the average restraint coefficient values for the three levels of struts in all strut rows. The heights for strut levels 1, 2, and 3 are 5.85, 0.10, and -6.45, respectively.

The results indicate that the top level (level 1), located in front of the fill layer, has a restraint coefficient ranging from 14% to 26%. This coefficient increases gradually from row 2 to row 10, suggesting that the proximity to the rock surface influences the restraint even at the higher-level struts, although slightly.

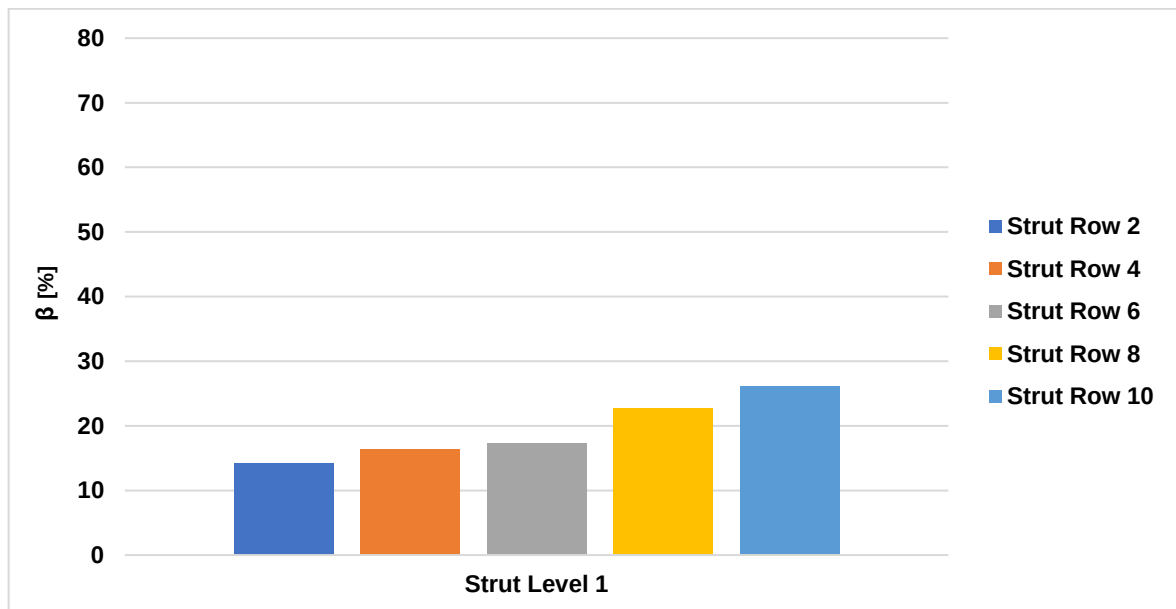


Figure 4.7 – Average restraint coefficient KV2 strut level 1

For the second level, located in front of a clay layer, the restraint coefficient ranges from 19% to 30% for strut rows 2 to 8. In strut row 10, which is fixed almost to the rock on the south wall, the coefficient exceeds 70%. This high value suggests that the proximity to the rock significantly influences the restraint of the strut (further evaluation on this on 4.4.1 and 4.4.2).

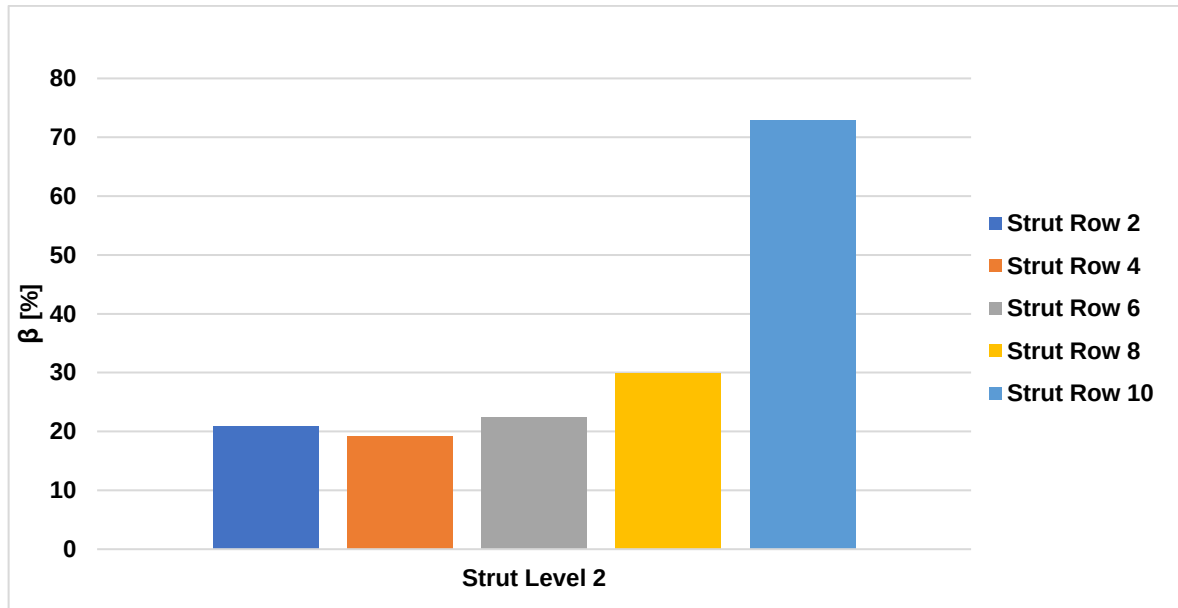


Figure 4.8 – Average restraint coefficient KV2 strut level 2

The final level, adjacent to a clay layer, shows a restraint coefficient of around 40% for strut rows 2, 6, and 8. For row 10, where the strut is fixed on the rock surface on the south wall, the restraint is considerably higher, exceeding 70%, as observed for level 2. Row 4 exhibits a restraint coefficient of approximately 53%, which is unclear given the various possibilities that could affect the stiffness of the wall, soil, and walling beam system at that location.

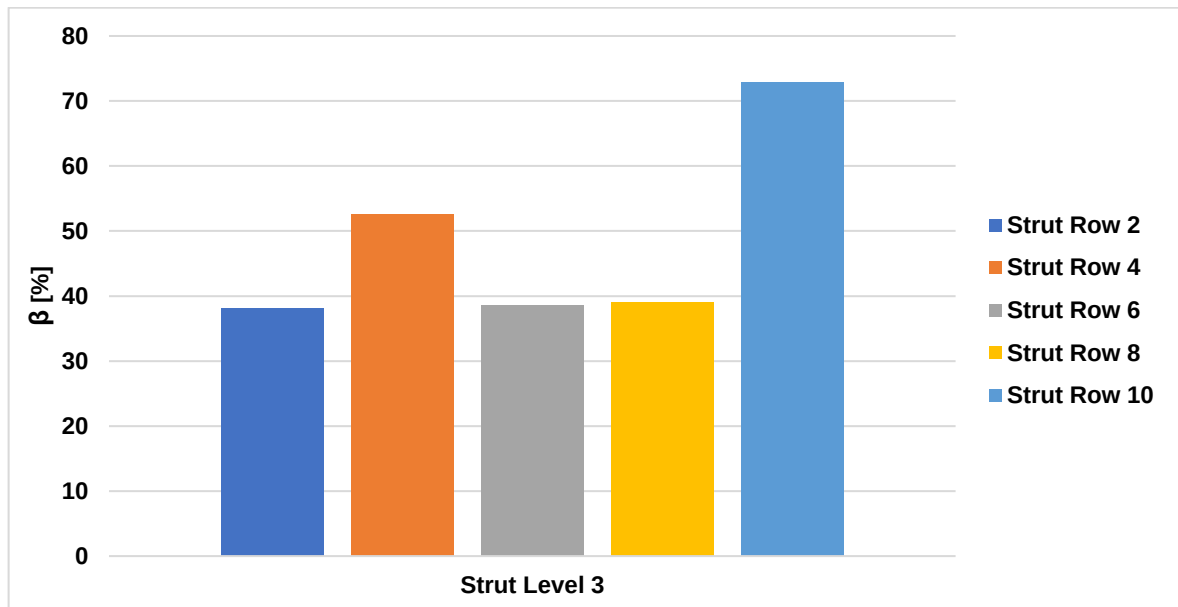


Figure 4.9 – Average restraint coefficient KV2 strut level 3

For Korsvågen 1, KV1, the strut heights are 3.5, -0.4, and -5.4, respectively. For levels 1,2 and 3 the average restraint coefficients are 25, 33 and 35% respectively, as illustrated in Figure 4.10. When comparing these values with similar conditions (comparable rock levels) in strut rows 2, 4, and 6 at Korsvågen 2, there is a good agreement.

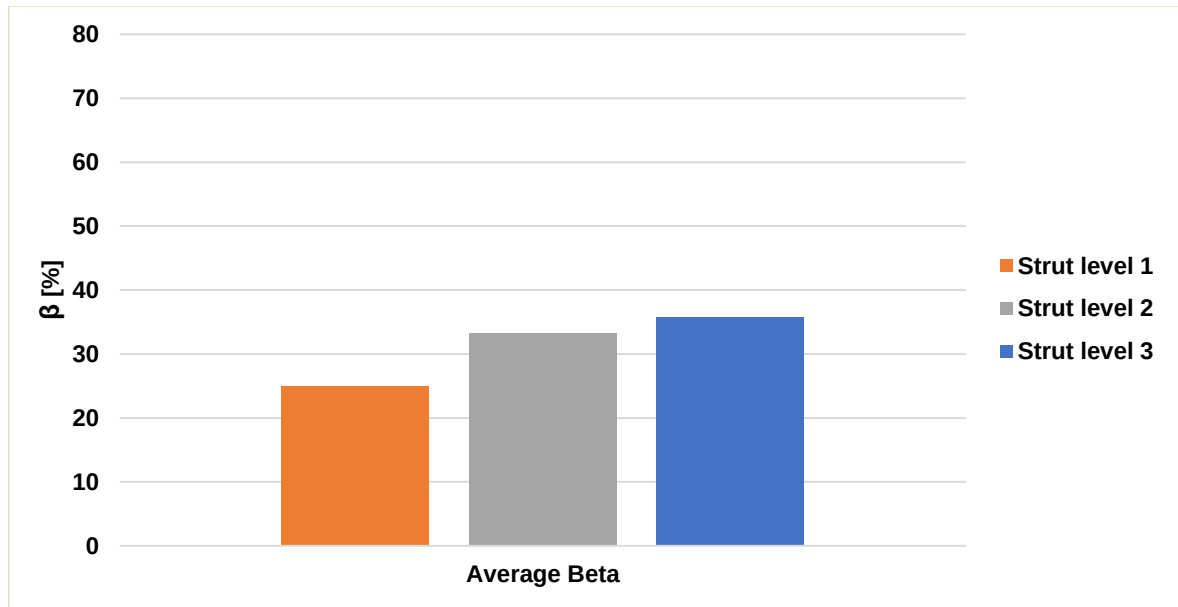


Figure 4.10 – Average restraint coefficient KV1 strut row 1

In Almedal, where the strut heights for levels 1, 2, and 3 are 3.3, -0.4, and -5.5, respectively, the average restraint coefficient, shown in Figure 4.11, is considerably lower for all strut levels not fixed on the rock surface, ranging from approximately 14% to 17%.

The only exception is ST-13 level 3, where one side is fixed on the rock, but even for this strut, the coefficient is much lower, just under 30%, compared to similar conditions in Korsvågen 2. These observations suggest that the stiffness of the wall and the restraint of the rotation of the pile toe due to the embedment of the wall may play a substantial role in the restraint of the strut, with sheet pile walls having lower bending stiffness than RD and Secant pile walls (further analysis in section 4.4.3).

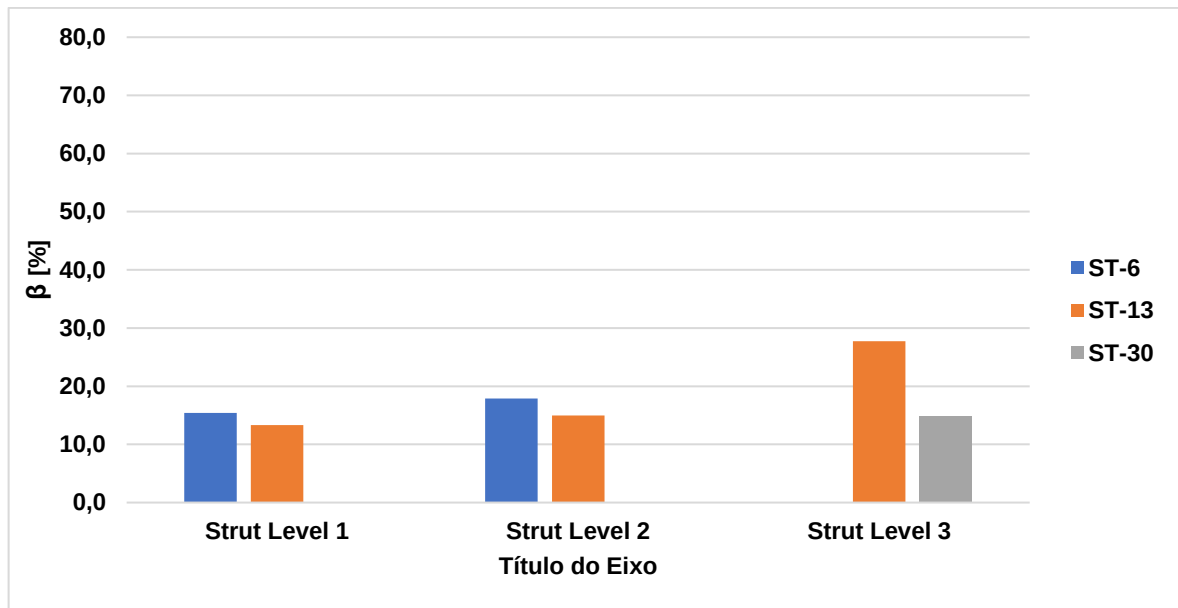


Figure 4.11 – Average restraint coefficient Almedal, ST-6, ST-13, and ST-30

As previously mentioned, the results provided in this section represent average values derived from a larger data set. Figures 4.12 and 4.13 illustrate two examples that not only display the average values, but also showcase the values considering the maximum variation in temperature and load over the entire period, as well as the maximum restraint coefficient resulting from the daily values.

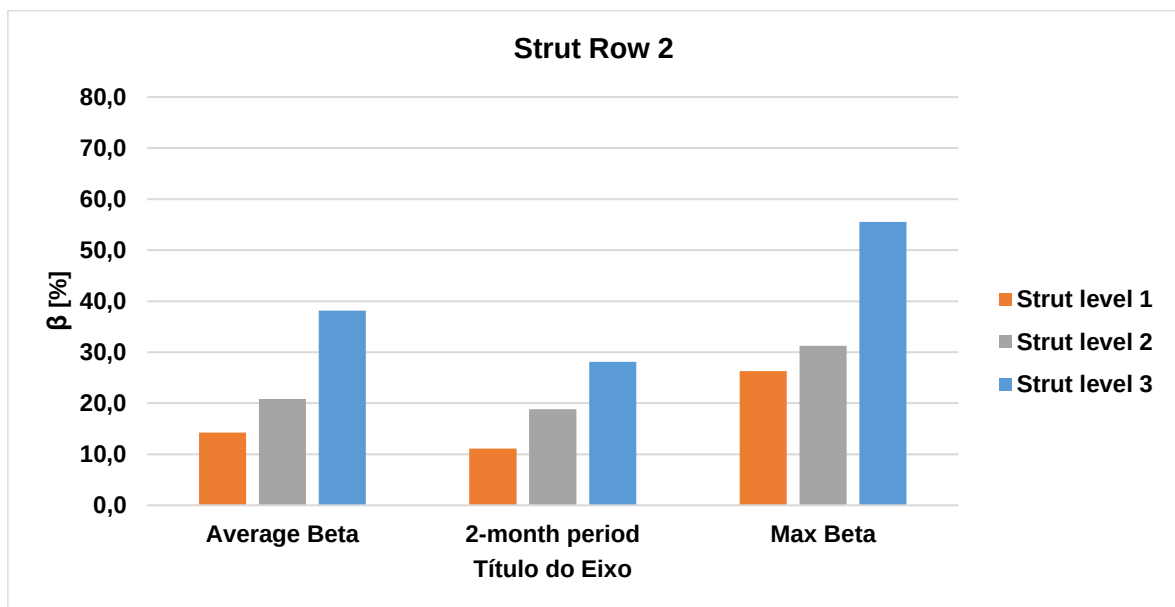


Figure 4.12 – Restraint coefficient Korsvägen 2 strut row 2

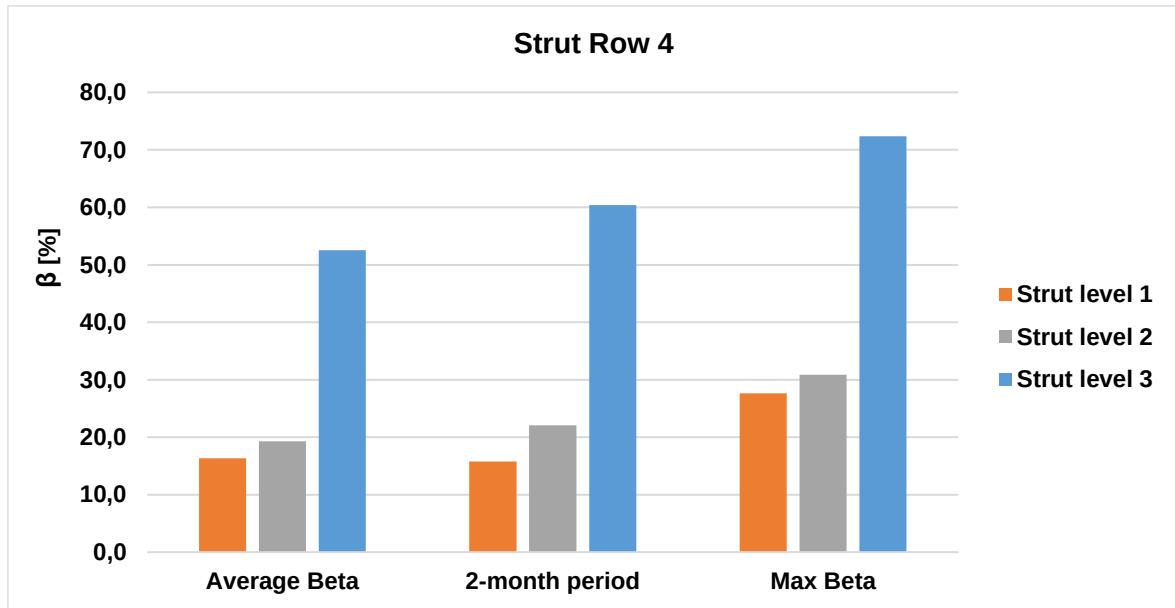


Figure 4. 13 – Restraint coefficient Korsvågen 2 strut row 4

These examples serve two purposes. Firstly, they demonstrate that the average values closely align with the values for the entire period, indicating their reliability as representative measures. Secondly, they highlight that despite the average values providing a comprehensive overview of the situation, the maximum restraint coefficient resulting from daily evaluations can occasionally be significantly higher. Therefore, it may be prudent to consider factors that increase the coefficient to ensure accuracy and account for potential fluctuations.

4.3.2. KORSVÄGEN 2 – STRUT ROW 10

To ensure that the strut was properly monitored during the hole construction, on both the Korsvågen sites, the forces and temperatures were taken from both ends of the struts, probably due to their incredibly high lengths. As these measurements are being taken from the same strut the expectation would be that loads would be exactly the same, this not what was observed in a few cases.

The following two diagrams in Figures 4.14 and 4.15 show for strut row 10 at Korsvågen 2 the restraint coefficient for the daily temperature and load variations and the force fluctuations for the entire period of construction. The south wall is where this strut is fixed at the rock, as illustrated in Figure 4.17, and in the north wall it is at a distance of almost 6 m.

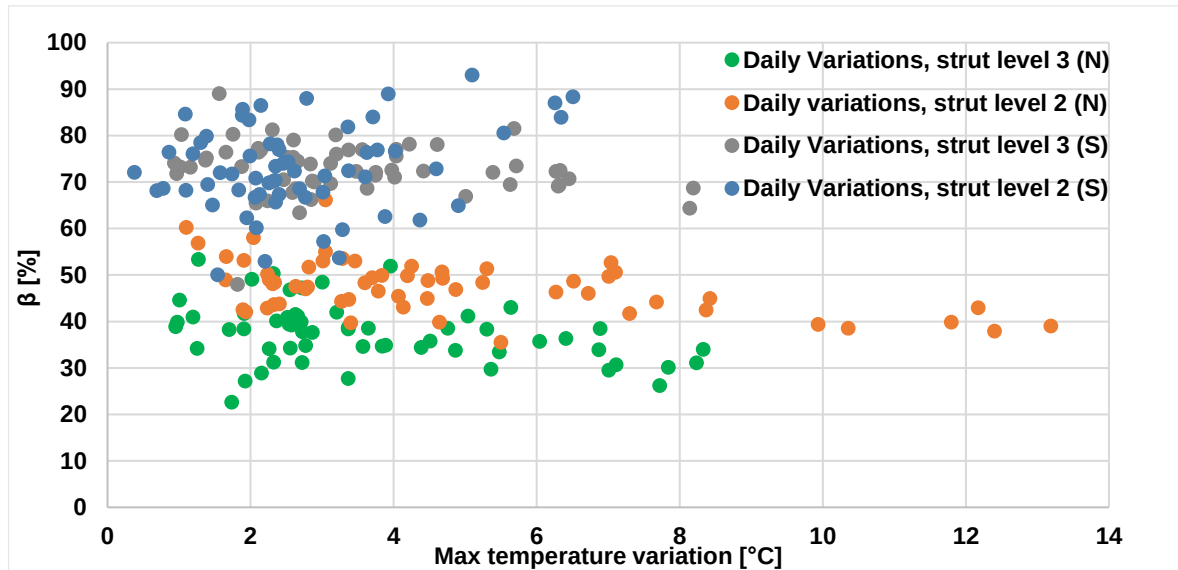


Figure 4.14 – Restraint coefficient KV2 strut row 10 daily values

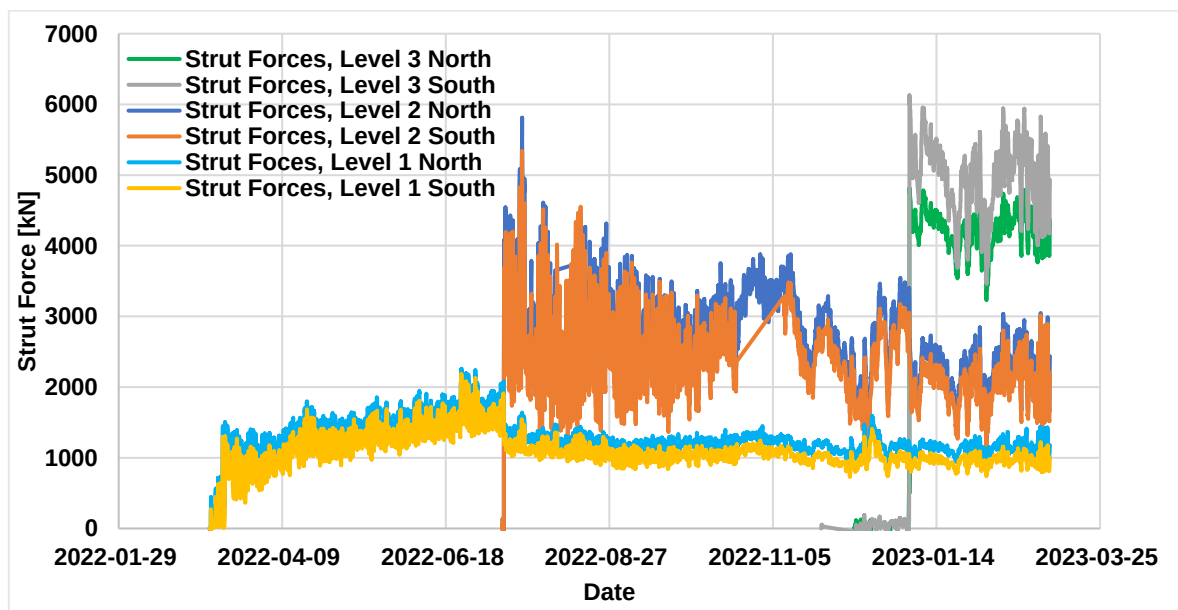


Figure 4.15 – Measured strut forces KV2 strut row 10

The following observations can be made from the Figures 4.14 and 4.15:

- The strut experiences much higher restraint at the south wall, with restraint coefficient values ranging from 60% to 90%, while the range at the north wall is about 30% to 60%.
- The fluctuations in the forces of the strut exhibit good agreement when comparing the two sides.
- For levels 1 and 2, the overall forces are slightly higher on the north side, but overall, they show good agreement. The most noteworthy observation is at the third level, where the south wall experiences values that are up to 1000 kN higher than the north wall. The maximum force for the north wall is around 5000 kN, whereas for the south wall it reaches around 6000 kN.
- There are no significant differences in the measured temperatures between the two sides. As seen in Figure 4.16.

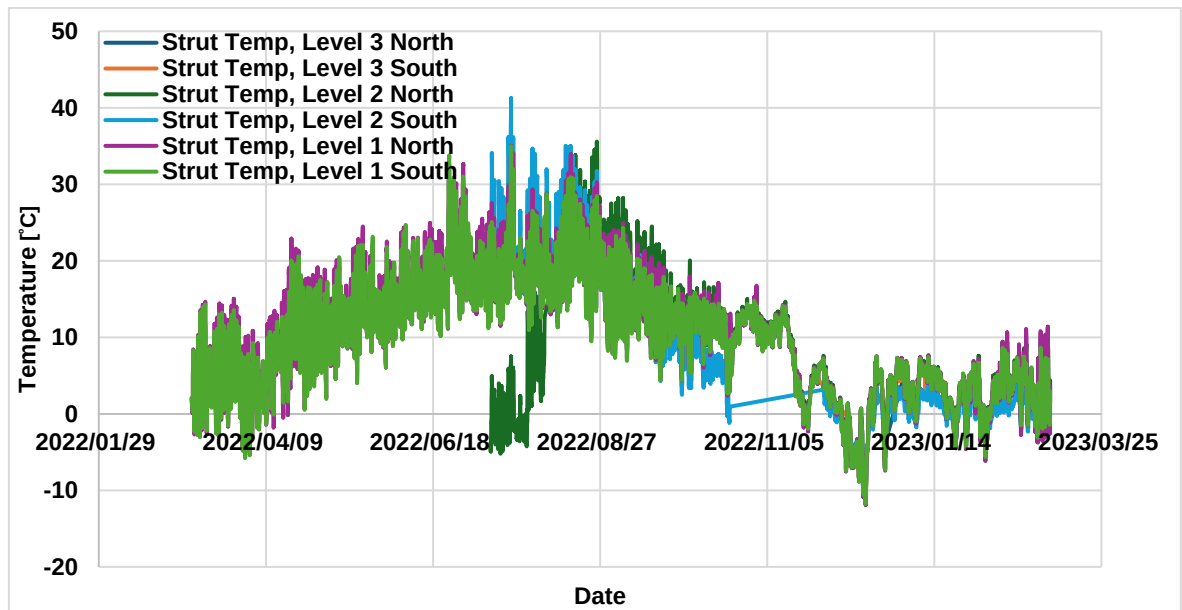


Figure 4.16 – Measured strut temperatures KV2 strut row 10



Figure 4.17 – Korsvägen 2 strut row 10 south wall

4.4 CORRELATION ANALYSIS

In order to better understand what most may influence the restraint coefficient; a few factors were investigated comparing similar situations of rock level and strut level in the three different construction sites in a way to narrow down what factor has the biggest impact on restraint coefficient.

The options studied were:

- Vertical distance between the strut and rock surface: Korsvågen 2 – Strut Rows 2, 4, 6, 8 and 10
- Closest distance between the strut and rock surface: Korsvågen 2 – Strut Rows 2, 4, 6, 8 and 10
- Average stiffness of the soil layers behind each strut level: Korsvågen 2 – Strut Rows 2, 4, 6, 8 and 10
- Wall bending stiffness: Korsvågen 1 – Strut Row 1; Korsvågen 2 – Strut Row 2; Almedal – ST-13
- Pile embedment: Korsvågen 1 – Strut Row 1; Korsvågen 2 – Strut Row 2

4.4.1. DISTANCE TO ROCK

Both diagrams for vertical, Figure 4.18, or closest, Figure 4.19, distance show that there is a clear correlation between the restraint coefficient and the distance to the rock surface. No great distinction can be made between both Figures. But the conclusion is fairly clear that the restraint coefficient has an inverse relationship with the distance from the rock surface, i.e., the further from the rock is the lower the coefficient will be, increasing as it gets closer.

The relationship does not appear to be perfectly linear, as there is some variability in the data points, but the overall trend suggests a semi-linear behaviour.

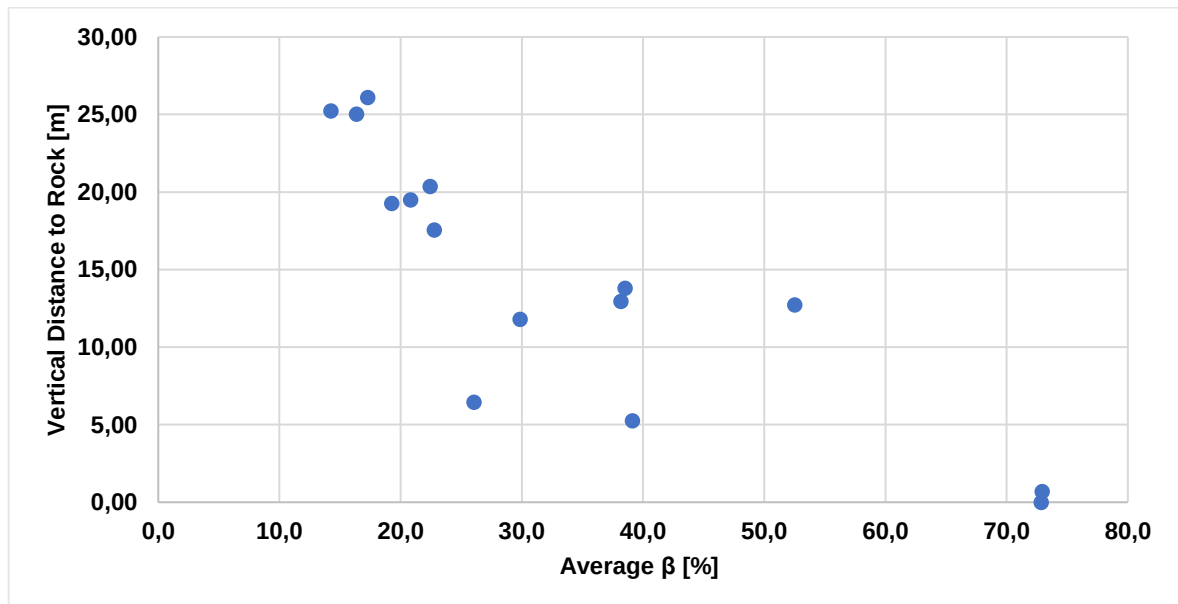


Figure 4.18 – Correlation between vertical distance to the rock and average restraint coefficient

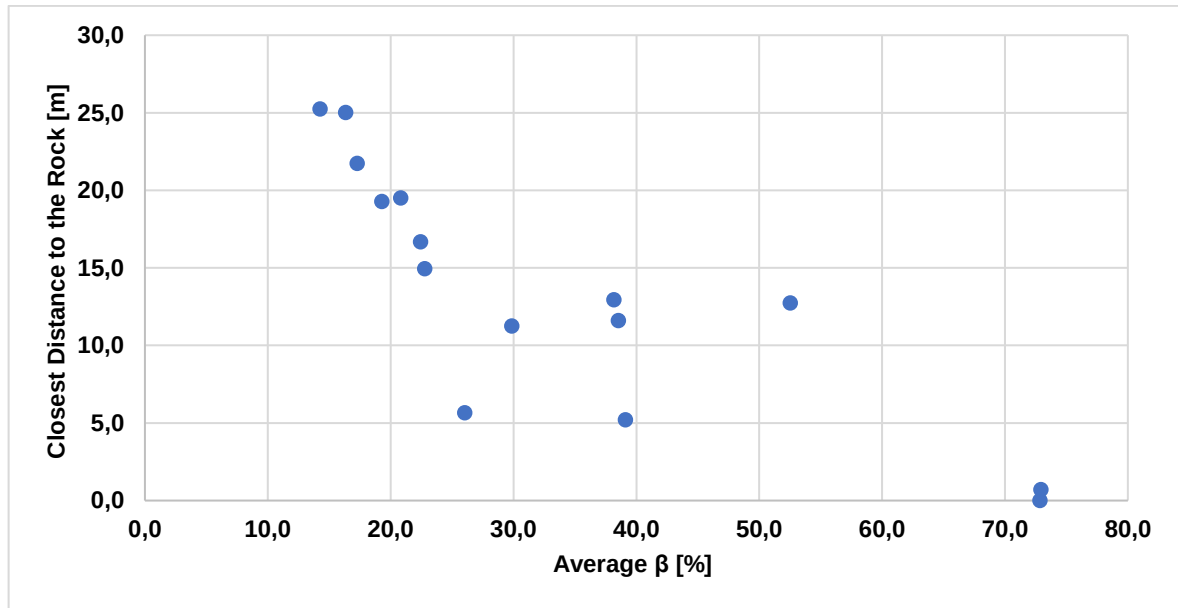


Figure 4.19 – Correlation between closest distance to the rock and average restraint coefficient

4.4.2. SOIL STIFFNESS BEHIND THE WALL

This correlation was made not with the average stiffness of the soil layers used for the design of the retaining walls and struts, but with the best estimate stiffness calculated during the calibration of the Plaxis model as explained on 5.3.1. This stiffness parameters better represent the actual soil conditions. To calculate the average soil stiffness, a simplification was made by considering the effective area that would influence each strut, this was done by taking into account the layers 3 meters above and below the strut. This correlation is presented on Figure 4.20.

Upon observation, the data points do not display a clear or strong correlation between the two variables. Specifically, within the stiffness range of 25 MPa to 45 MPa, there is no discernible pattern that indicates a significant impact on the restraint coefficient, as the values of Average restraint coefficient do not show a consistent trend with changes in Average Stiffness. There is no substantial evidence to suggest that soil stiffness has a considerable effect on the restraint coefficient for the retaining walls and struts. Further investigation may be required to explore this relationship in more detail, potentially considering a wider range of stiffness values or additional soil parameters.

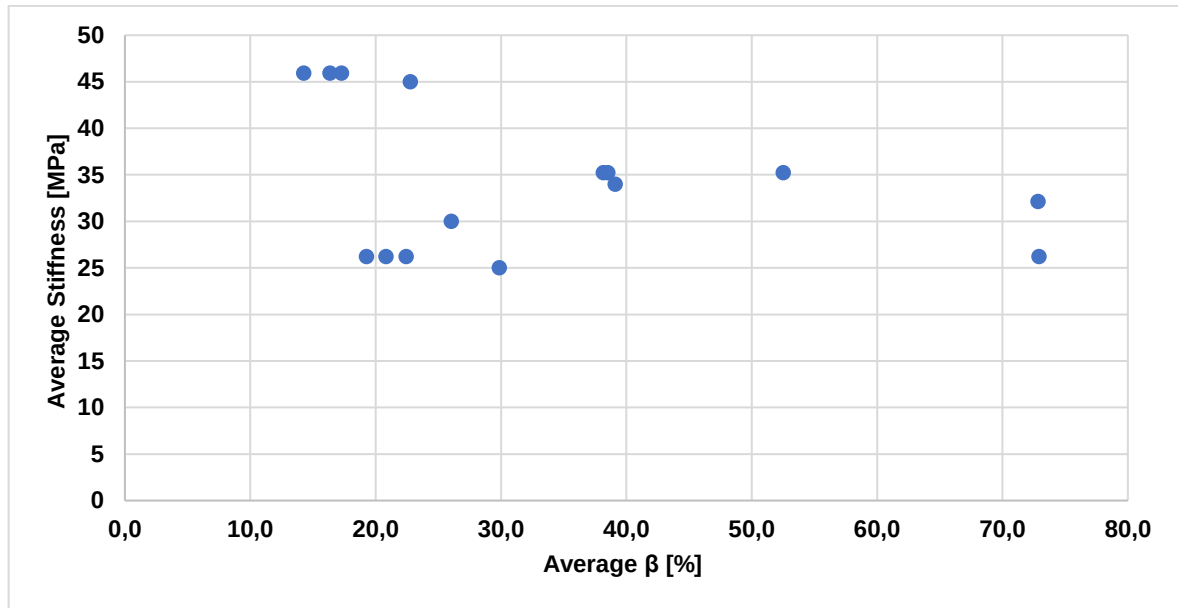


Figure 4.20 – Correlation between average soil stiffness and average restraint coefficient.

4.4.3. WALL STIFFNESS

Strut level 1 (orange squares), Strut level 2 (gray triangles), and Strut level 3 (blue circles). The diagram, in Figure 4.21, aims to show the relationship between the bending stiffness of different wall types and the restraint coefficient (Average Restraint coefficient).

The top values, which are the highest on the y-axis, correspond to the Secant pile wall with a bending stiffness of 1922857 kNm²/m. The middle values represent the RD pile wall with a bending stiffness of 938437 kNm²/m, and the lower values correspond to the sheet pile wall with a bending stiffness of 220500 kNm²/m.

There is no strong or clear overall correlation between bending stiffness and the restraint coefficient across all strut levels. However, when looking at each strut level separately, there is a suggestion of a small tendency for the restraint coefficient to increase with the bending stiffness of the wall. This tendency is not pronounced and does not form a clear linear pattern, but it does suggest that there might be a relationship between these two parameters.

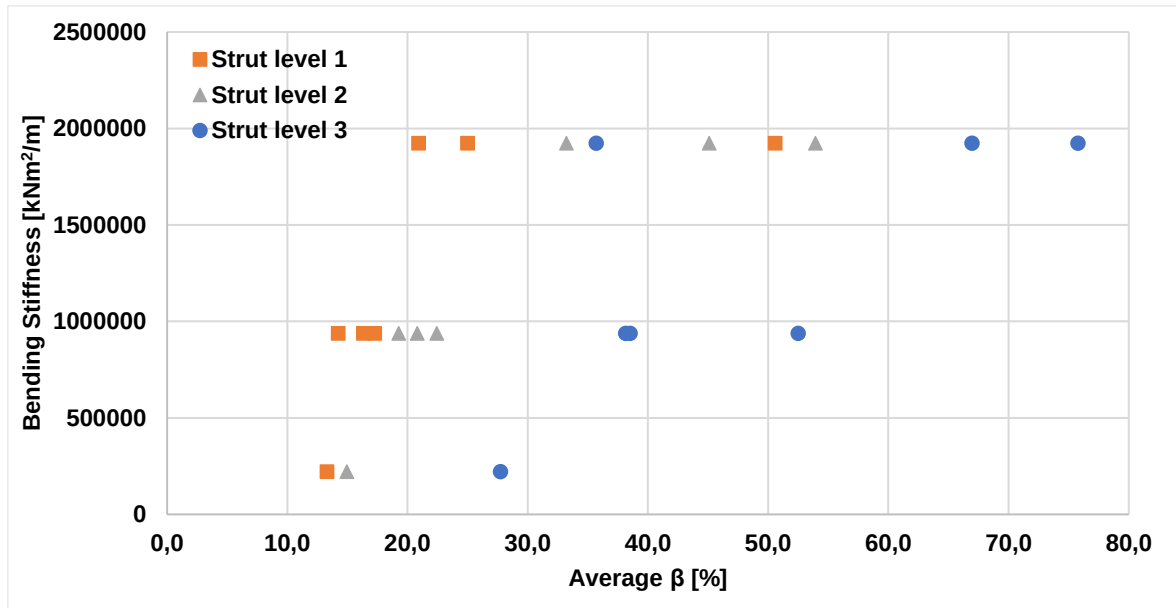


Figure 4.21 – Correlation between wall bending stiffness and average restraint coefficient.

The last correlation made is regarding the pile embedment in the rock, Figure 4.22. In Korsvägen 2 all RD piles were embedded 1,65m into the rock. In Korsvägen 1 the embedment for the secant piles varied from pile to pile, the embedment was taken as the average of two piles to each side of the strut. The embedment ratio is the pile embedment in relation to the pile diameter. The diameters are 0,813 and 1,200 m for the RD and secant piles respectively.

The embedment ratio shows a very narrow range of values. To this range it does not appear to be any discernible correlation to be taken from this diagram.

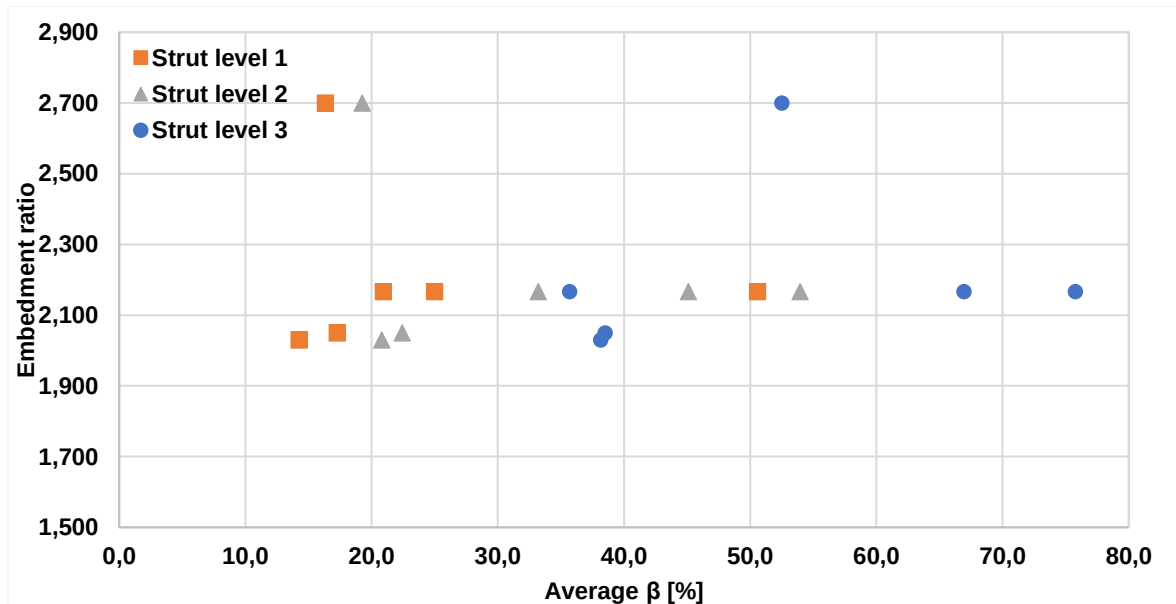


Figure 4.22 – Correlation between the wall embedment ratio and average restraint coefficient

The correlations suggest that among the various factors studied, the distance between the strut and the rock exhibits the most significant influence on the coefficient. Nevertheless, due to the intricacy of the problem, it becomes exceedingly difficult to pinpoint with complete certainty the potential influence of other factors and to what extent they contribute to the coefficient.

It is important to bear in mind that even though efforts were made to identify scenarios with similar characteristics for comparison purposes, several distinctions exist between each situation, especially when comparing different types of retaining walls. These distinctions can have an effect on the correlations.

5

FINITE ELEMENT MODELLING ON PLAXIS 2D

5.1. NUMERICAL MODELLING

5.1.1. FINITE ELEMENT METHOD

In geotechnical engineering, the Finite Element Method (FEM) is a computational technique used to analyse and predict the behaviour of soil and rock structures under various loading conditions. It is particularly valuable for studying complex soil-structure interactions and geotechnical problems that involve non-linear behaviour or irregular geometries.

In FEM-based geotechnical analysis, the soil and rock mass are modelled as continua, and the governing equations of soil mechanics, such as those describing stress and strain distributions, are discretized over a mesh of finite elements. Each element represents a small portion of the soil or rock mass, and these elements are interconnected at nodes. The behaviour of the soil or rock within each element is typically described using constitutive models that capture the material's mechanical properties, such as elasticity, plasticity, and strength.

FEM allows geotechnical engineers to simulate a wide range of geotechnical processes, including slope stability analysis, foundation design, tunnelling, embankment construction, retaining wall design, and groundwater flow analysis. By performing numerical simulations with FEM, engineers can assess the performance and safety of geotechnical structures, predict deformations and settlements, evaluate the effectiveness of different construction techniques, and optimize design parameters.

While finite element analysis yields result close to reality, it requires careful consideration of boundary and geometry modelling, an efficient and high-quality mesh, as well as appropriate selection of constitutive models and soil parameters. Incorrect choices in any of these aspects can affect the accuracy and reliability of the analysis.

5.1.2. PLAXIS 2D

The type of analysis conducted in each situation depends on the complexity of the problem at hand. In the case addressed in this thesis, a 2D analysis was performed. However, in scenarios where plain strain or axisymmetric, illustrated in Figure 5.1, conditions do not accurately reflect the actual conditions and boundaries of the problem, a 3D analysis may be necessary.

To conduct the analyses, the software utilized was PLAXIS 2D. This is a highly regarded software amongst geotechnical engineers and allows for the creation of detailed numerical models of soil structures. It encompasses a wide range of applications, such as foundations, excavations, embankments, retaining walls, tunnels, and slopes.

The software provides the capability to simulate various loading conditions, including static loads, seismic events, and changes in pore water pressure. Additionally, PLAXIS 2D offers advanced analysis functionalities, enabling the consideration of nonlinear soil behavior, consolidation, time-dependent effects, and staged construction sequences.

Engineers can examine key factors like soil deformation, stress distribution, stability, and settlement, thereby gaining valuable insights into the performance of their designs. Furthermore, the software supports parametric studies and sensitivity analyses, enabling engineers to explore different design options and assess the influence of various parameters on the behavior of soil structures. Overall, PLAXIS 2D empowers engineers with the tools they need to make informed decisions and optimize their geotechnical designs.

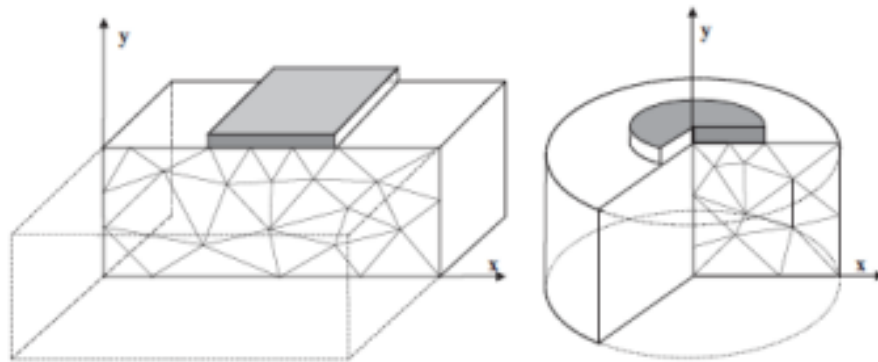


Figure 5.1 – Example of a plain strain (left) and axisymmetrical problem (right) (Plaxis 2023a)

5.2. ORIGINAL DESIGN MODEL

In the Korsvägen 2 site, the retaining wall design was approached in a sectional manner, considering the rock level at each specific location. Piles R011 and R015 were selected for comparison between measured values and Plaxis calculations. These piles were at a section where the design was denominated as DS-B4, which meant that the rock level was between -18.9 and -19.65, with 3 strut level. For the model the level was taken -19.65 and with an embedded length of 1.65m the pile toe was at -21.3. Final excavation level set above the bedrock. Figure 5.2 shows the designation of each design section.

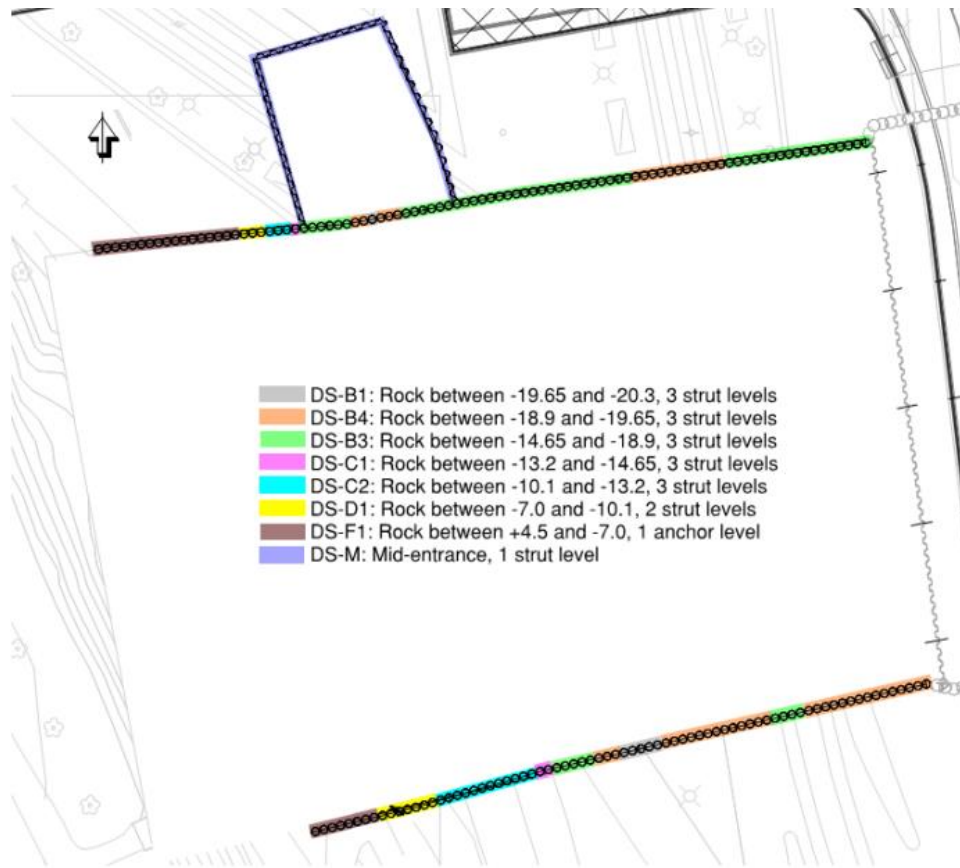


Figure 5.2 – Design sections Korsvagen 2

For the Plaxis 2D model of this section only the south wall was included and modelled. The wall was supported by strut levels modelled as fixed-end anchors, each with a length of 30m. The groundwater level was set at +7.0, and the boundaries of the model were defined to $x_{\min} = -30$; $x_{\max} = 70$; $y_{\min} = -50$ and $y_{\max} = 70$, as schemed on Figure 5.3. Moreover, a linear surcharge load of 20 kPa was applied over the full width behind the wall.

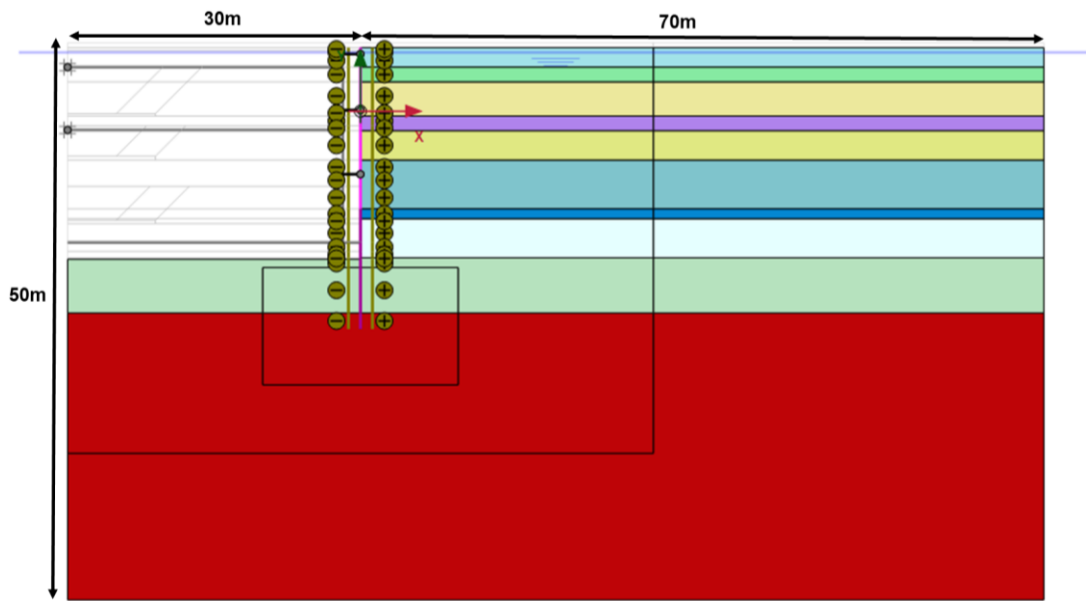


Figure 5.3 – Plaxis model soil layers and boundary definition

Two different material models were used to represent the soil layers in the model. The NGI-ADP material model was used for untreated clays on the active side, while the Mohr-Coulomb model was used for all other layers. The NGI-ADP model may be used for analyses involving undrained loading of clays, more information about any of this material models can be found on the Plaxis Material Manual (Plaxis 2023b)

Table 5.1 – Properties of the untreated clays on the active side modelled with NGI-ADP material model

	Unit	Clay 1	Clay 2	Silty Clay 1	Silty Clay 2	Silty Clay 3
Level top/bottom	[m]	+4.5/+3.0	+3.0/-0.5	-2.0/-5.0	-5.0/-10.0	-10.0/-11.0
γ	[kN/m ³]	15.8	15.8	19.2	19.2	19.2
G_{ur}/s_u^A	[-]	600	600	600	600	600
γ_r^C	[%]	1.5	1.5	1.5	1.5	1.5
γ_r^E	[m]	3.0	3.0	3.0	3.0	3.0
γ_r^{DSS}	[m]	2.0	2.0	2.0	2.0	2.0
$s_u^{A_{ref}}$	[kPa]	25.0	25.0	31.0	36.0	45.0
$s_u^{C,TX}/s_u^A$	[-]	0.99	0.99	0.99	0.99	0.99
y_{ref}	[m]	+6.0	+3.0	-2.0	-5.0	-10.0
$s_u^{A_{inc}}$	[kPa/m]	0.0	1.2	1.8	1.8	1.8
s_u^P/s_u^A	[-]	0.60	0.61	0.60	0.57	0.56
σ_0/s_u^A	[-]	0.7	0.7	0.7	0.7	0.7
s_u^{DSS}/s_u^A	[-]	0.76	0.87	0.84	0.76	0.70
ν	[-]	0.495	0.495	0.495	0.495	0.495
K_0	[-]	1.05	0.89	0.70	0.75	0.75

Table 5.1 provides information on the soils modelled using the material model from the NGI's database Norwegian Geotechnical Institute (NGI-ADP), these parameters were estimated based on the and ground investigations. Table 5.2 has the parameters soils using the Mohr-Coulomb. The values of E (Young's modulus), c_u (undrained shear strength) for the undrained materials and c' (effective cohesion) for the drained materials were assumed to be constant throughout the depth.

The rock was modelled with its top level at -19.65 m, considering it as a non-porous material. The rock parameters were determined based on $RMR_{bas} = 40-60$. The Hoek-Brown material model was used for the rock, with the following parameters: γ (unit weight) = 26.0 kN/m³, E (Young's modulus) = 13500 MPa, ν (Poisson's ratio) = 0.2, σ_{ci} (uniaxial compressive strength of the intact rock) = 145 MPa, m_i (Rock mass parameter) = 13.0, GSI (Geological Strength Index) = 45, D (disturbance factor) = 0.1, m_b (Rock mass parameter) = 1.644, ψ_{max} (dilatancy angle) = 0°, σ_ψ (absolute value of confining pressure σ'_3 at which $\psi = 0^\circ$) = 0 MPa, s (Rock mass parameter) = 0.0018, a (Rock mass parameter) = 0.508, σ_c (Rock mass uni-axial compressive strength) = -5.84 MPa.

Table 5.2 – Properties of the soils modelled with Mohr-Coulomb material model

	Unit	LC stabilised clay	Fill	Layered Sand/Silt/Clay	Bottom stratified sand layer, upper	Bottom stratified sand layer, lower
Level top/bottom	[m]	+4.5/-11.0	+7.0/+4.5	-0.5/-2.0	-11.0/-15.0	-15.0/-19.7
Drainage type		Undrained (B)	Drained	Drained	Drained	Drained
γ	[kN/m ³]	19.0	21.0	18.5	21.0	21.0
E	[MPa]	40	30	15	13.5	19.5
ν	[-]	0.25	0.3	0.31	0.31	0.31
c_u	[kPa]	100	0.0	0.0	0.0	0.0
ϕ	[°]	0	35	33	33	33
Ψ	[°]	0	0	0	0	0
K_0	[-]	1.0	0.426	0.455	0.485	0.455

The RD pile wall used in the project consists of $\varnothing 813 \times 20.0$ mm piles made of S355 grade steel. The bending stiffness, EI , of the plate elements representing the RD pile wall is 925 MNm²/m. The design takes into account a 0.3 mm loss of wall thickness due to corrosion over the 10-year design life.

Interface elements with an interface strength factor, R_{inter} , of 0.67 are employed on both the active and passive sides of the wall. Design calculations are performed both with the wall toe modelled at the rock surface level without toe rotational restraint, and with pile embedment modelling using RMR_{bas} 40-60, which includes toe rotational restraint.

The struts in the wall are modelled as fixed-end anchors. The axial stiffness (EA) of the struts is set to 11800, 21380, and 23040 MN per strut for strut levels 1, 2, and 3, respectively, with a spacing of 6.5 m. These values are determined based on the strut cross-sections $\varnothing 914 \times 20.0$ mm, $\varnothing 1321 \times 25.0$ mm, and $\varnothing 1422 \times 25.0$ mm for strut levels 1, 2, and 3, respectively. All struts are made of S355 steel with an elastic modulus (E) of 210 GPa. The steel waling beams 3xHEB-400, 3xHEB-900, and 3xHEB-1000 for strut levels 1, 2, and 3, respectively, are disregarded in the stiffness of the fixed-end anchors. These waling beams are also made of S355 steel.

The construction sequence is shown in Table 5.3. For the purpose of his thesis, it is only of interest the model from the initial phase until final excavation (phase 11).

Table 5.3 – Plaxis analysis modelling sequence

Phase	Description
1	Initial phase
2	Excavation to +6.5 and surface load
3	Lime-Cement stabilization of clays on passive side
4	RD-pile wall installation
5	Lowering the ground water level to elevation -0.25 m on passive side and excavation to level +4.5 m, with a 10 m wide trench to -0.25 m, on passive side
6	Install pre-stressed strut at elevation +5.85 m with a pre-stress force of 1500kN
7	Lowering the ground water level to elevation -4.60 m on passive side and excavation to level -1.50 m, with a 10 m wide trench to -4.6 m, on passive side
8	Install pre-stressed strut at elevation +0.10 m with a pre-stress force of 4500kN
9	Lowering the ground water level to elevation -11.15 m on passive side and excavation to level -7.7 m, with a 10 m wide trench to -11.15 m, on passive side
10	Install pre-stressed strut at elevation -6.45 m with a pre-stress force of 6500kN
11	Lowering the ground water level to elevation -15.15 m on passive side. Excavate to final depth of -15.15 m
12	Lay out gravel fill for installation of bottom slab up to elevation -14.35 m
13	Installation of bottom slab with CL at -13.45 m
14	Casting of tunnel to -7.7 m, and backfill between permanent structure and retaining wall
15	Removal of strut installed at -6.45 m
16	Casting of tunnel to -1.5 m and internal middle slab with CL at -1.925 m, and backfill between permanent structure and retaining wall
17	Removal of strut installed at +0.10 m
18	Casting of tunnel to +4.5 m and backfill between permanent structure and retaining wall
19	Removal of strut at +5.85 m
20	Backfilling to +7.0 on both sides of the wall

The main reason for the choice of this design section is that when compared to the monitored pile the overall shape of the of the deflection at each stage of the construction process already shows quite a good agreement. Figures 5.4 and 5.5 display, respectively, the calculated and measured deflection and the development of the measured deflections at the same level from before the excavation necessary for strut 2 to be installed until after final excavation. Figure 5.6 has the measured strut forces in relation to the Plaxis obtained values (straight lines).

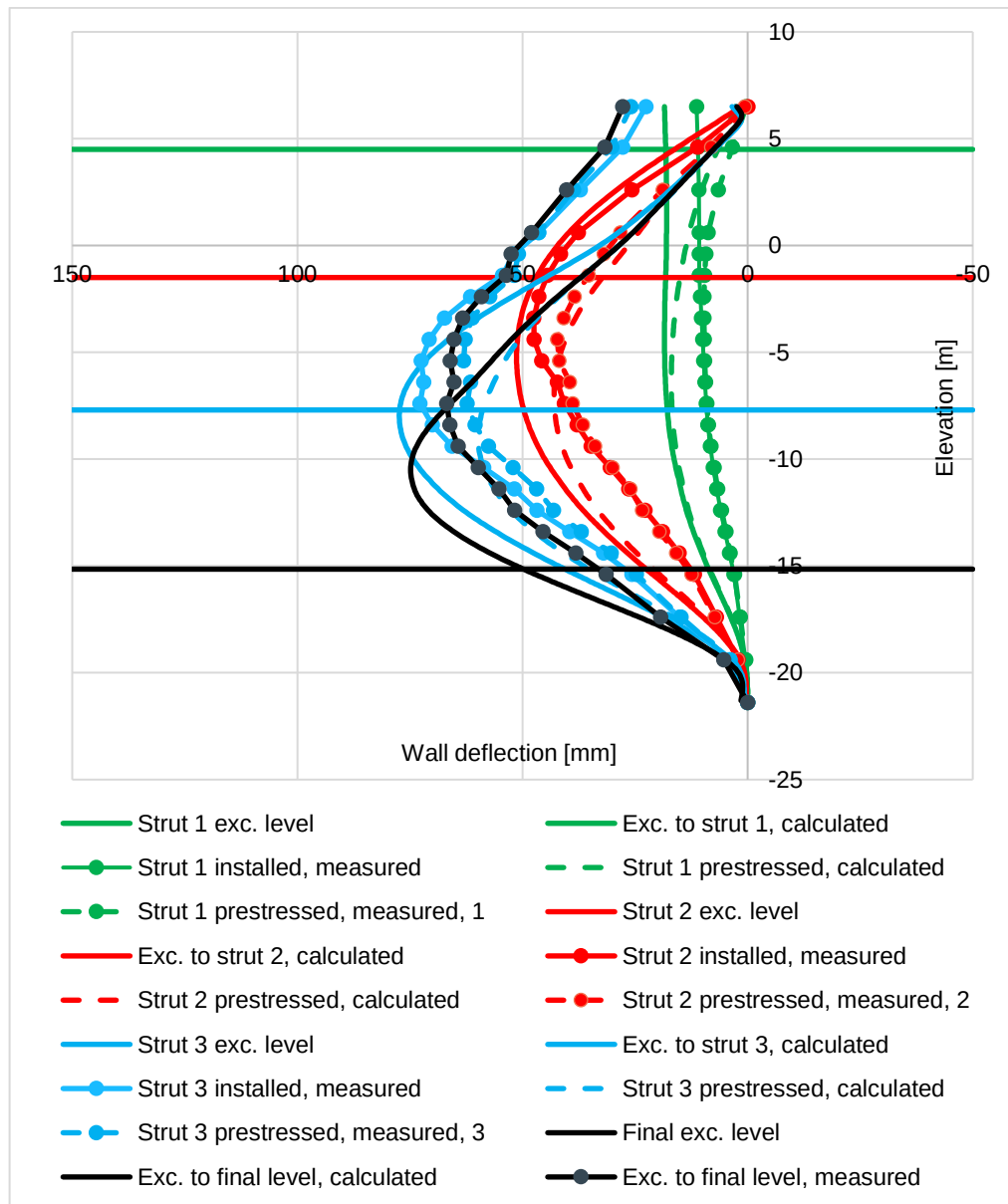


Figure 5.4 – Comparison of calculated (original design model) and measured wall deflections

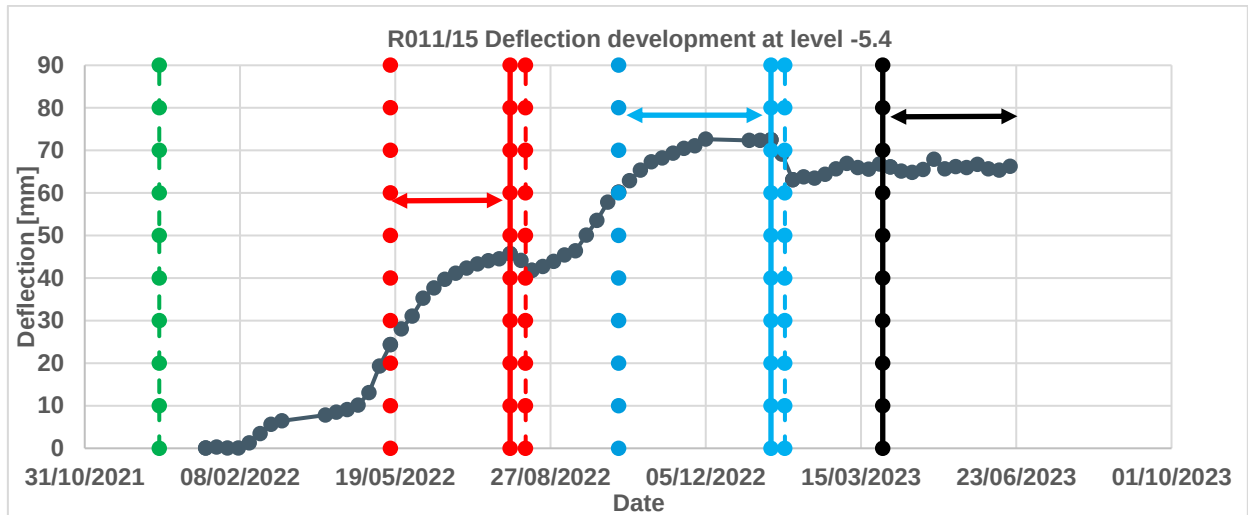


Figure 5.5 – Time development of measured deflection at level -5.4. Construction stages shown by vertical lines (line colours and line types according to Figure 5.4). Periods without excavation works are marked with arrows.

A few observations can be made from the three Figures which may be relevant to the following back analysis:

- All measured backward deflections are smaller than calculated, which implies a conservative soil modelling.
- Following the excavation to a certain level, there was a period of 2 to 3 months before the installation of strut levels 2 and 3 (indicated by arrows in Figure 5.5). During this period, there was a notable increase in wall deflections due to the consolidation of the LC stabilized clay. This may indicate a transition from undrained to drained conditions for the LC stabilized clay.
- Measured strut forces are smaller much than predicted. Further comments on 5.3.7.

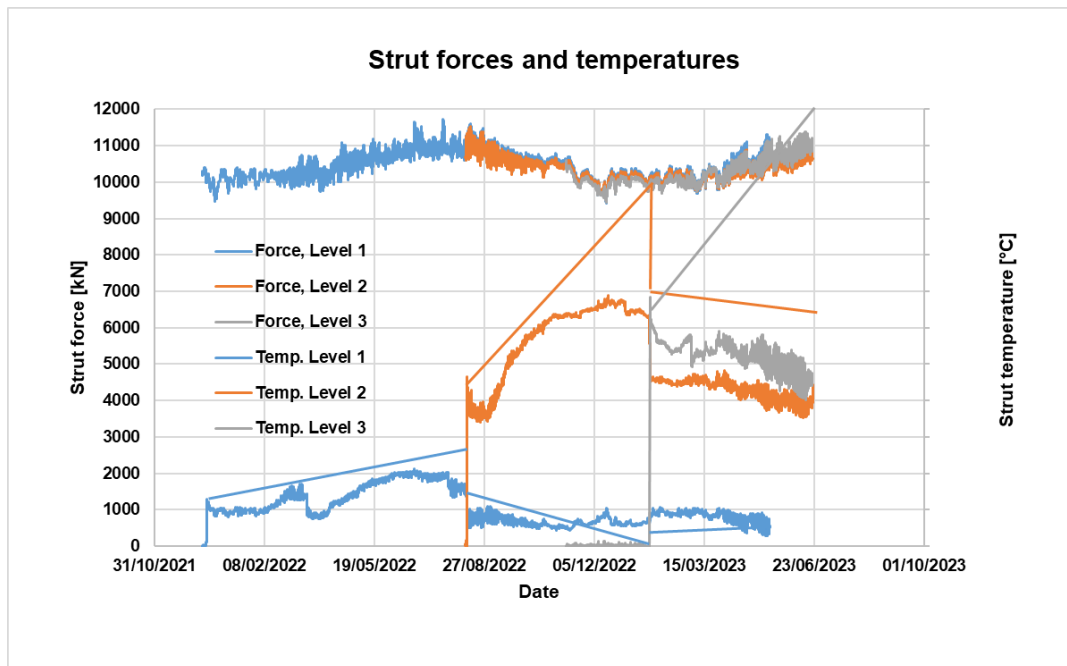


Figure 5.6 – Comparison of measured strut forces and temperatures and Plaxis (original model) calculation

5.3. BACK ANALYSIS

In order to obtain more accurate results for the strut forces, adjustments needed to be made to the existing model used for the design of this section. These adjustments aimed to make the software results less conservative and closer to reality. The adjustments will also aim to reach a best estimate of the soil parameters stiffness, i.e., a more correct representation of actual soil conditions.

As said before the measurements chosen upon which the model results will be compared to are average values. For the piles is the average of R011 and R015, for the struts is strut rows 2 and 4, represented in Figure 5.7.

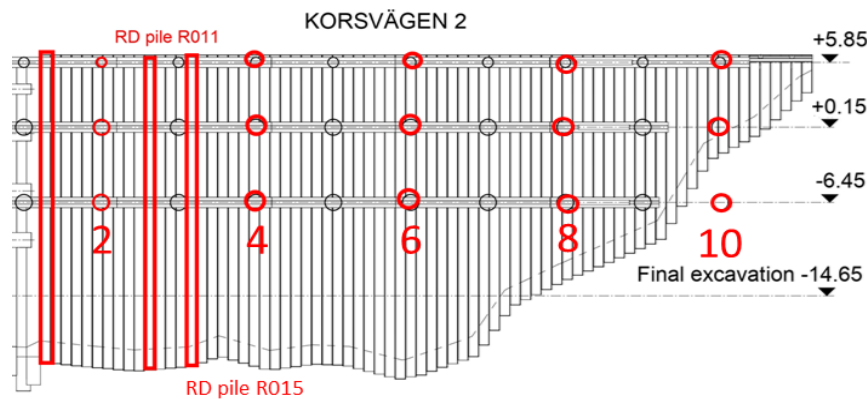


Figure 5.7 – Korsvägen 2 south wall view with piles used for comparison marked

5.3.1. SOIL STIFFNESS CALIBRATION

To calibrate the soil stiffness, the first step in the process involved adjusting the stiffness of the soil layers to align the measured backward deflection, caused by the pre-stressing of the strut, with the calculated value. This calibration process followed an iterative approach, gradually increasing the stiffness of the soil layers positioned behind each strut level. The objective was to achieve a ratio between the measured and calculated values that would approach 1, indicating a close match.

The adjusted layers and their respective parameters for each strut level are detailed in Table 5.4.

Table 5.4 – Adjusted soil stiffness parameters

	Soil Layer	Design Value (E or G_{ur}/S_u^A)	E [MPa]	G_{ur}/S_u^A	Increase Factor
Strut Level 1	Fill	20	60	-	3,0
	Clay1	600	-	1800	3,0
Strut Level 2	Clay2	600	-	1800	3,0
	SaSiCL	15	33	-	2,2
	SiCL1	600	-	1800	3,0
Strut Level 3	SiCL2	600	-	2000	3,2
	SiCL3	600	-	2000	3,2
	SiSa-Upper	14	33	-	2,4
	SiSa-Upper	20	39	-	2,0

The prestress values for struts 1, 2, and 3 were 1500 kN, 4500 kN, and 6500 kN respectively. It was observed that the measured backwards deflection caused by the prestress differed from the calculated values in the original design model. The backwards deflection for the first strut level was measured to be 11.32 mm, whereas the calculated value was 14.54 mm. Similarly, for the second level, the measured deflection was 9.57 mm, while the calculated value was 15.68 mm. Lastly, the measured deflection for the third level was 18.87 mm, whereas the calculated value was 10.60 mm.

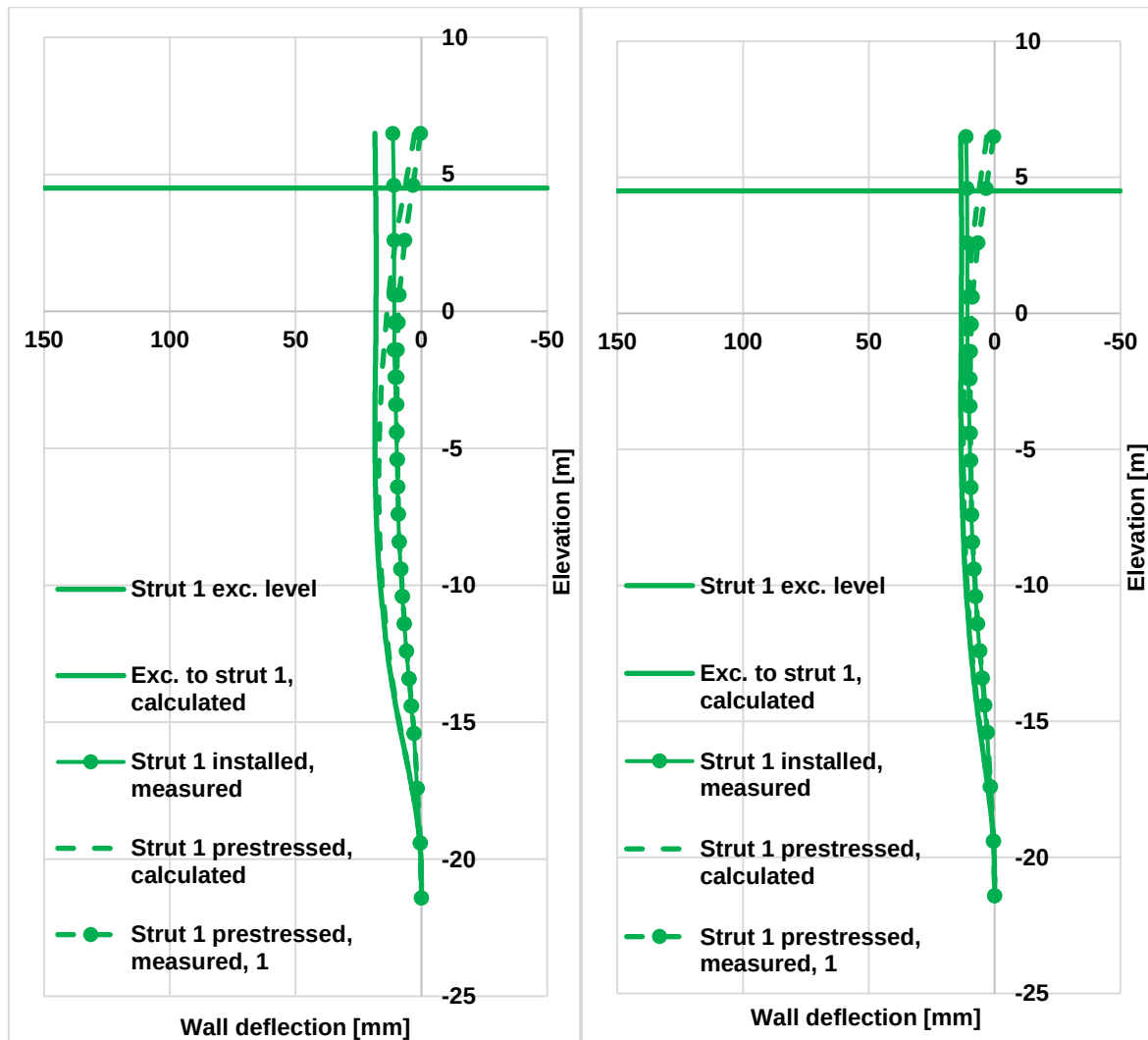


Figure 5.8 – Comparison of calculated and measured wall deflections after the prestressing of strut level 1, original design model (left) and after soil stiffness calibration (right)

After calibrating the soil stiffness, there was a significant improvement in the backwards deflection compared to the initial model, Figures 5.8 and 5.9 exemplify this improvement for the excavation and installation of strut levels 1 and 3. The measured deflection values were recorded as 11.76 mm, 10.29 mm, and 10.25 mm for strut levels 1, 2, and 3 respectively. This indicated a better agreement between the measured and calculated values, Table 5.5 shows the scale of the improvement.

Table 5.5 – Comparison of backwards deflection ratios

Backwards deflection Ratio (Calculated/measured)		
	Original Model	Soil stiffness calibration
Strut Level 1	1,64	1,04
Strut Level 2	1,68	0,94
Strut Level 3	1,81	1,03

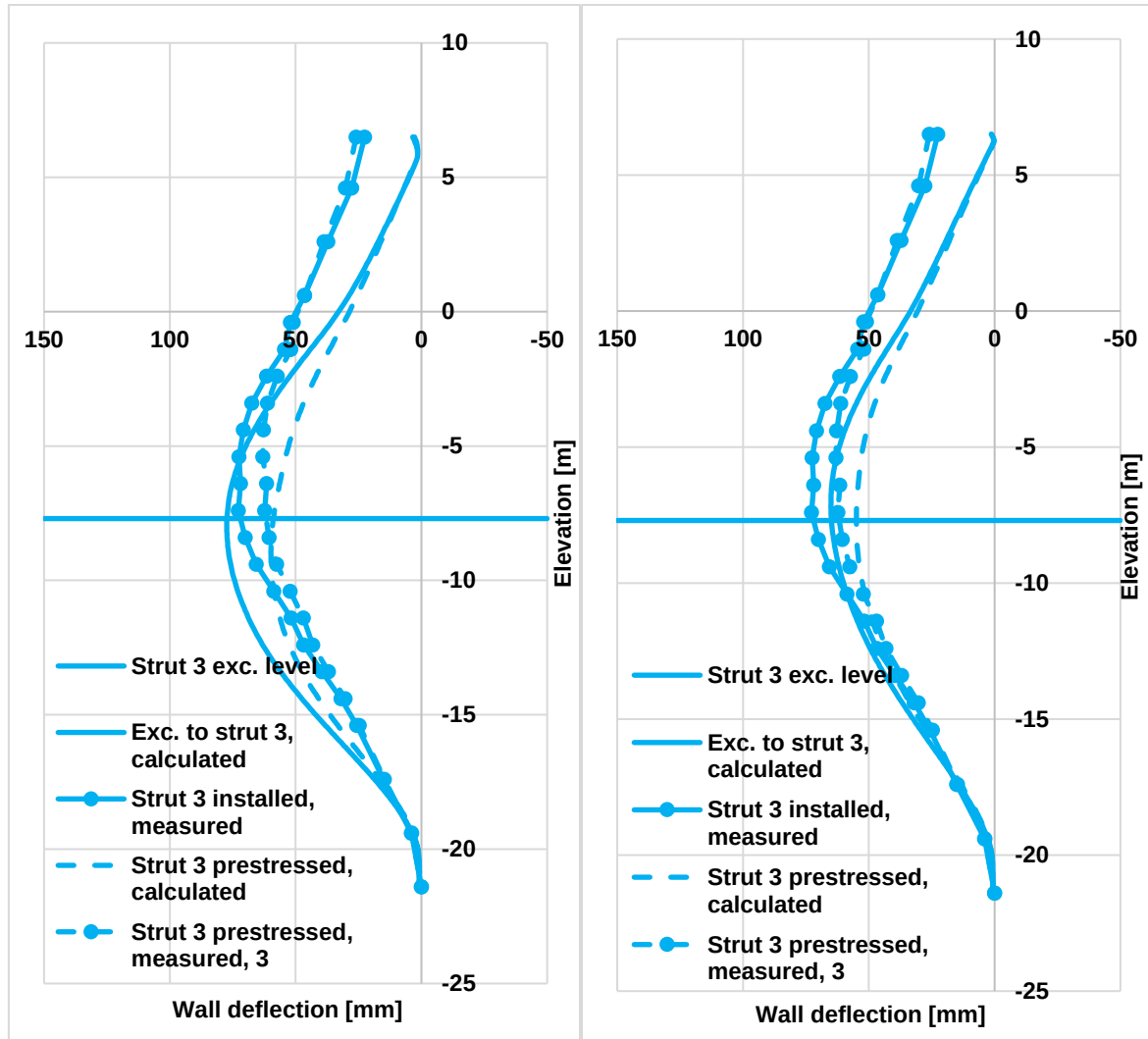


Figure 5.9 – Comparison of calculated and measured wall deflections after the prestressing of strut level 3, original design model (left) and after soil stiffness calibration (right)

Although the backwards deflection showed improved agreement, there were still some differences in the overall wall displacement. The measured maximum deflection was considerably larger, and the toe rotation demonstrated a less restrained behaviour in the PLAXIS calculations.

5.3.2. LIME-CEMENT STABILISED CLAY

In the second phase of the back analysis, three modifications were implemented. Firstly, the groundwater level on the active side of the excavation was reduced by 1 meter, going from 7.0 m to 6.0 m. Secondly, the 20 KPa surcharge on the surface was eliminated. These adjustments were made based on observations that indicated these changes would better reflect the actual conditions at the site.

Following these modifications, the parameters of the Lime-cement (LC) stabilized clay were altered. The specific changes are presented in Table 5.6. Initially, the LC clay was shifted from undrained to drained conditions, with an undrained shear strength value (c_u) of 0 kPa, change made due to consolidation process previously mentioned. Additionally, the stiffness of the clay was adjusted to decrease with depth. This was done under the assumption that the effectiveness of the lime-cement stabilization does not extend throughout the entire depth of the clay. At the top of the lime-cement clay, the Young's modulus was set to 40 MPa, while at the bottom it was reduced to 3 MPa. The decision to choose 3 MPa at the bottom was based on the understanding that the lime-cement's effectiveness virtually none existing at that depth, hence the value represents the minimum stiffness of the untreated clay.

Table 5.6 – Lime-cement stabilised clay properties

	Unit	LC stabilised clay (original)	LC stabilised clay (back analysis)
Level top/bottom	[m]	+4.5/-11.0	+4.5/-11.0
Drainage type	[-]	Undrained (B)	Drained
γ	[kN/m ³]	19.0	19.0
E	[MPa]	40	40
E_{inc}	[-]	0	-2.1
γ	[kPa]	6.5	6.5
ϕ	[°]	0	32
c_u	[kPa]	100	0

The choice of the bottom stiffness was corroborated by tests conducted in the untreated clay which showed that a value of 3 MPa was the lower bound value for that soil. These tests were seismic dilatometer tests. Refer to the Figure 5.10.

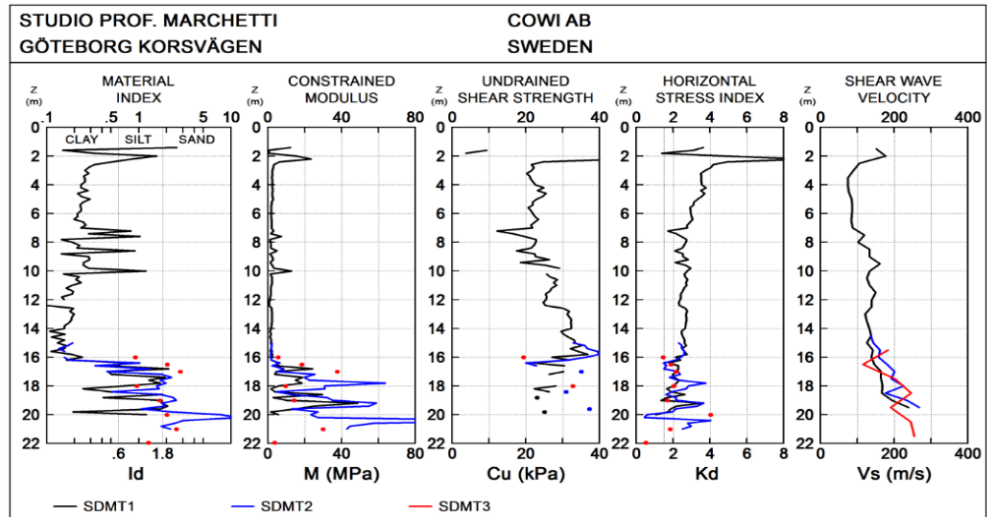


Figure 5.10 – Seismic dilatometer test results

Figure 5.11 emphasizes the importance of considering the decrease in stiffness of the lime-cement stabilized clay with depth when assessing the maximum deflections of the wall. However, it is important to note that this choice is a hypothesis regarding the behaviour of lime-cement and further analysis is required to draw a definitive conclusion.

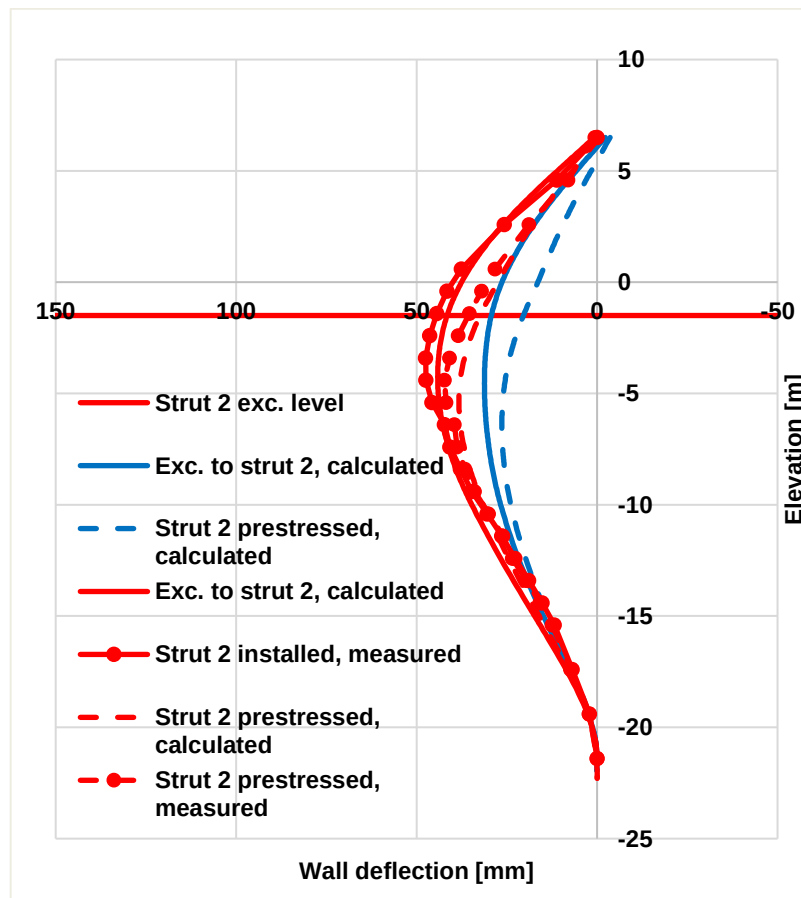


Figure 5.11 – Comparison of calculated and measured wall deflections after the prestressing of strut level 2, decreasing stiffness (red) and constant stiffness (blue)

5.3.3. ROCK CALIBRATION

Next step in the analysis involved the calibration of the rock parameters. As previously mentioned, these values were initially based on the conservative estimate derived from RMR_{bas} 40-60. However, it became apparent that relying solely on this estimate might lead to an underestimation of the rock quality. Therefore, to ensure a more precise and reliable assessment, the RMR_{bas} 60-80 range was considered, which is deemed to provide a more accurate reflection of the true rock quality. An example of one of these analyses is shown in Figure 5.12.

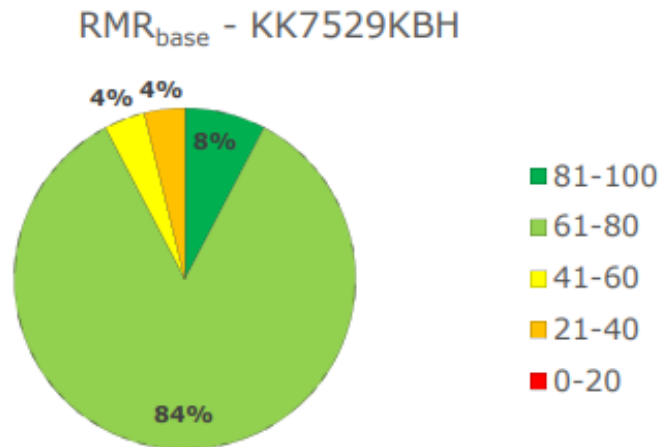


Figure 5.12 – Example RMR classification of a sample taken from the site.

The new properties for used for the rock are listed on Table 5.7 in contrast to the previously used parameters.

Table 5.7 – Rock properties, Hoek-Brown material model

	Unit	RMR _{bas} 40–60	RMR _{bas} 60–80
Level top/bottom	[m]	+4.5/-11.0	+4.5/-11.0
Drainage type	[-]	Undrained (B)	Drained
γ	[kN/m ³]	26	26
E	[GPa]	13,5	40
σ_{ci}	[MPa]	145	145
GSI	[-]	45	65
m_i	[-]	13	13
D	[-]	0.1	0.1
m_b	[-]	1.644	3.487
Ψ_{max}	[°]	0	0
σ_{ψ}	[MPa]	0	0
s	[-]	0.0018	0.0179
a	[-]	0.508	0.502
σ_c	[MPa]	-5.84	-19.24

Studies conducted using rock core samples have provided further support for this revised assessment. An example of one of these analyses is shown in Figure 5.12, which revealed an average RMR_{base} value of 67, indicating that the rock mass is classified as good. The lowest calculated RMR_{base} value obtained was 39, indicating poor rock mass conditions. Conversely, the highest RMR_{base} value recorded was 92, indicating very good rock mass quality.

5.3.4. WALL STIFFNESS INCREASE

In order to increase the capacity of the piles at specific locations most of the RD-piles were implanted with plates welded to the RD-profiles, where needed. Also, in several locations HEB profiles and a jet-grout layer were added inside the bottom part of the RD-piles. Figure 5.13 illustrates how the section of the pile looked after this implementation.

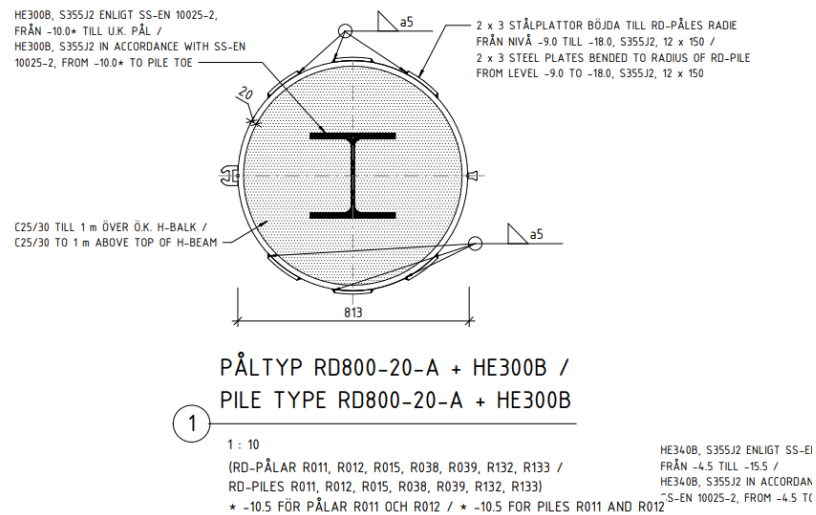


Figure 5.13 – Section of pile RD800-20-A with welded plates, HE300B beam and concrete filling

The values of the area, A , and moment of inertia, I , of the RD-piles with the welded plates were determined in AutoCAD, Figure 5.14 and Table 5.8.

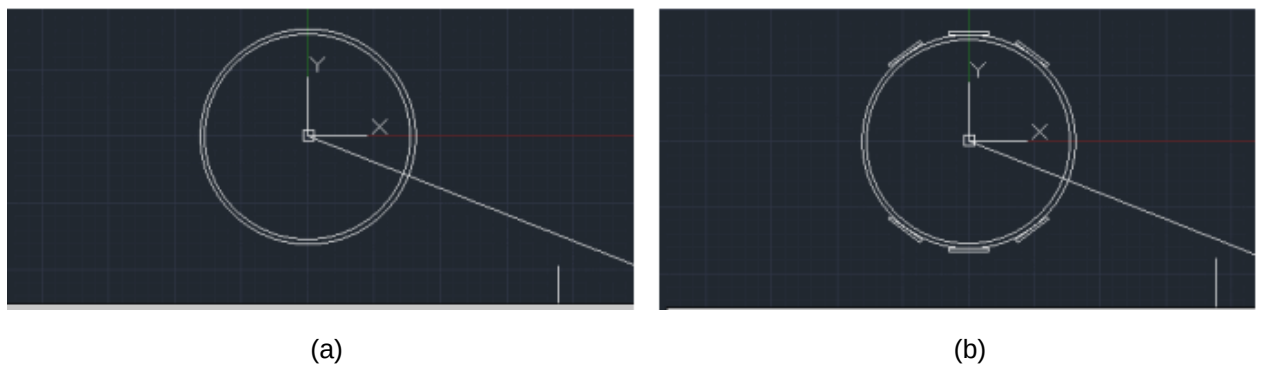


Figure 5.14 – AutoCAD drawing of RD pile with welded plates: a) without welded plates; b) with welded plates

Table 5.8 – RD pile properties

	A [mm ²]	I [mm ⁴]
Without Plates	49825.6595	3.919x10 ⁹
With Plates	60625.6595	5.342x10 ⁹
Ratio	1.22	1.36

As the measured values were an average of two piles which did not have the exact same length of the HEB profile and jet-grouting, for simplification it was considered that these were implanted from -10 to the pie toe. The profile used was HE300B with steel S355J2 and the concrete used was C25/30, with $E = 210\text{GPa}$ and $E = 33\text{GPa}$, respectively. Table 5.9 presents properties for each of these improvements.

Table 5.9 – Concrete filling and HE300B beam properties

	A [mm ²]	I [mm ⁴]
Concrete	469298	1.752x10 ⁹
HE300B	14910	2.517x10 ⁸
	EA [kN/m]	EI [kNm ² /m]
Concrete	15486839	578364
HE300B	3131100	52857

The final value of the axial stiffness, EA, and bending stiffness, EI, where calculated by simply adding up the values for each element, as exemplified in equation (5.1).

$$EA = 924.7 \cdot 10^3 \cdot 1.22 + 15486839.9 + 3131100 = 32897940 \text{ kN/m} \quad (5.1.)$$

$$EI = 11.74 \cdot 10^6 \cdot 1.36 + 578364.748 + 52857 = 1891222 \text{ kNm}^2/\text{m} \quad (5.2.)$$

5.3.5. ROCK LEVEL

Lastly it was proven necessary to alter the rock level and consequently pie toe level. The rock surface made using the Jb-soudings proved in certain section to be significantly inaccurate. Seeing that the design sections were divided by the rock level, for example design section DS-B4 was used as base for all piles which the rock level was between -18.9 and -19.65. The piles that were identified to be outside the design boundaries are presented in Figure 5.15. Both piles used for this analysis were outside this range.

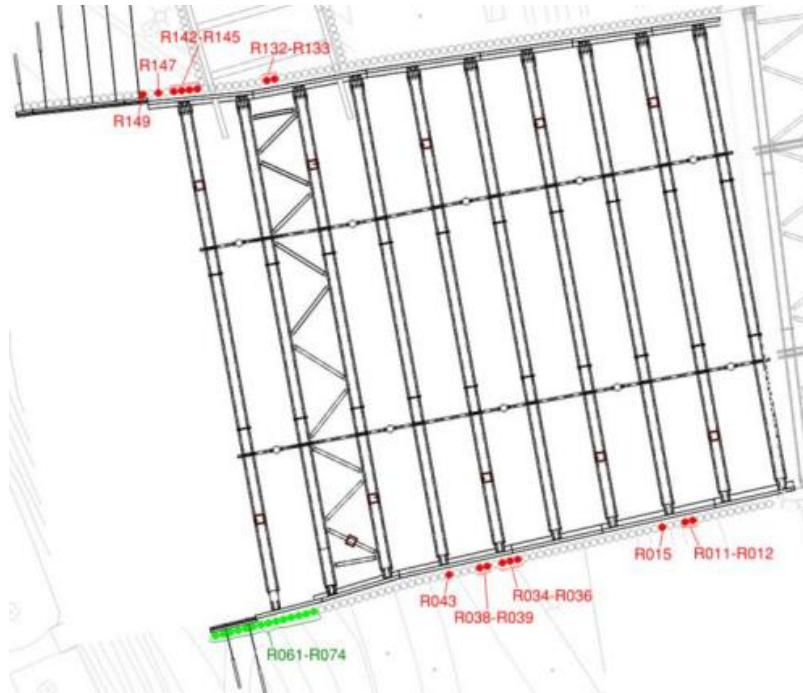


Figure 5.15 – Plan view of Korsvagen 2 with piles found to have different rock levels after excavation marked

For R011 the rock actual rock level was -20.86 and the pile toe was at -22.51. For R015 the rock actual rock level was -20.42 and the pile toe was at -22.07. Both piles are embedded 1.65 m. For the model the average of both piles was used, i.e., rock level at -20.64 and the pile toe at -22.29.

5.3.6. CORRECTION OF MEASURED VALUES

After the installation of strut level 3, there was observed movement of the top of the wall towards the inside of the excavation at the south wall and towards the outside at the north wall. This movement indicated that both walls shifted together, although the displacement at the south wall was slightly larger. The exact cause of this movement is uncertain, a possible hypothesis is it occurred due to the significant temperature fluctuations and from the large size of the excavation (60 m struts). Figure 5.18a illustrates this movement.

To try an address this unexpected movement, the following steps were taken:

1. The temperature effect was eliminated from the measured strut forces by utilizing equation (2.2) and the average value of restraint coefficient from strut rows 2 and 4. This allowed for the isolation of the temperature impact on the strut forces, Figure 5.20.
2. The deflection of the wall based on the strut forces was calculated. This involved determining the expected deflection of the wall considering only the applied loads on the struts. The calculation was performed by:

$$u_{force} = \frac{N_{without\ temperature\ effect} \cdot L/2}{EA} \quad (5.3.)$$

$$u_{temperature} = \alpha \cdot \Delta L \cdot L \cdot (1 - \beta) \quad (5.4.)$$

$$u_{total} = u_{force} + u_{temperature} \quad (5.5.)$$

These calculations were used to create two graphs: the first graph illustrated the measured wall deflection at the strut level relative to the wall deflection at the end of strut installation over time. The second graph depicted the expected deflection of the wall (u_{total}) based on the measured strut force data also over time.

Upon analysing these graphs (Figures 5.16 and 5.17), a noticeable discrepancy between the measured values and the calculated values was observed. Specifically, for the top of the wall (strut level 1), and to a lesser extent for strut level 2, the discrepancy between the measured and calculated values increased as time progressed, as anticipated.

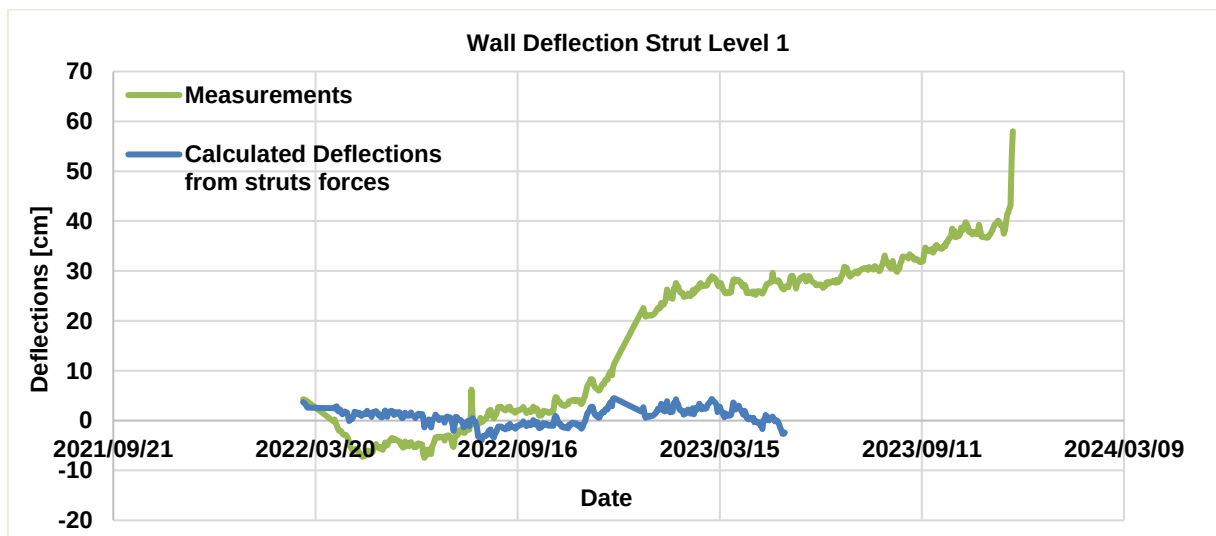


Figure 5.16 – Time development of measured deflection at strut 1 level

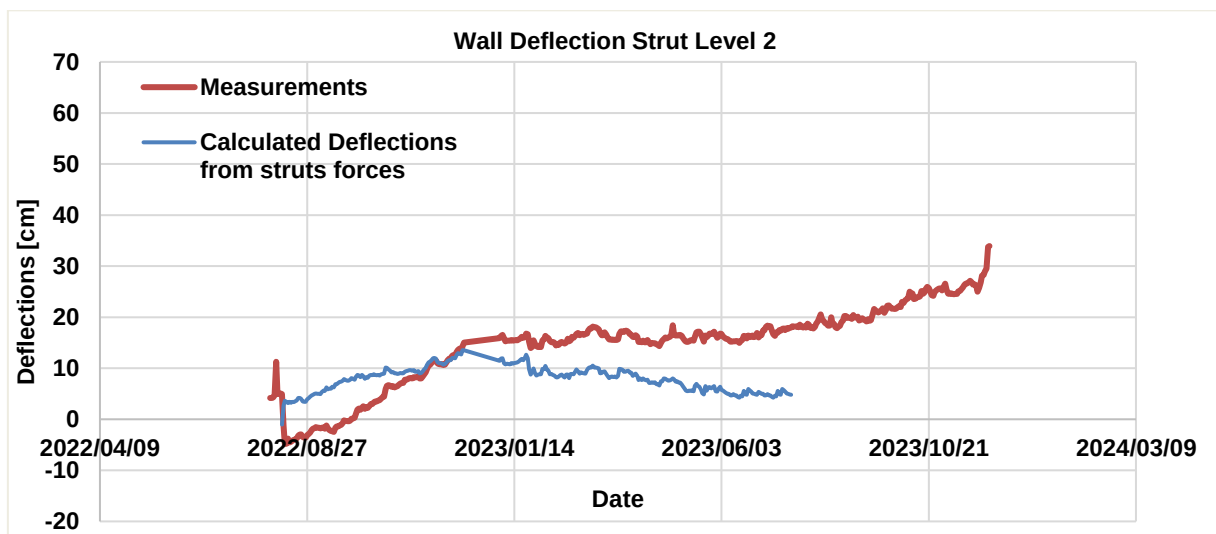


Figure 5.17 – Time development of measured deflection at strut 2 level

As this discrepancy became apparent towards the end of 2022/beginning of 2023, a corrective measure was implemented. The difference between the two plotted lines was deducted from the average values of the measurements of R011 and R015 to compensate for the overall movement of the wall. Figure 5.18 shows the measurements before and after the correction and as it can be observed the deflection at the top passed to close to zero, as expected, and the maximum values did not alter.

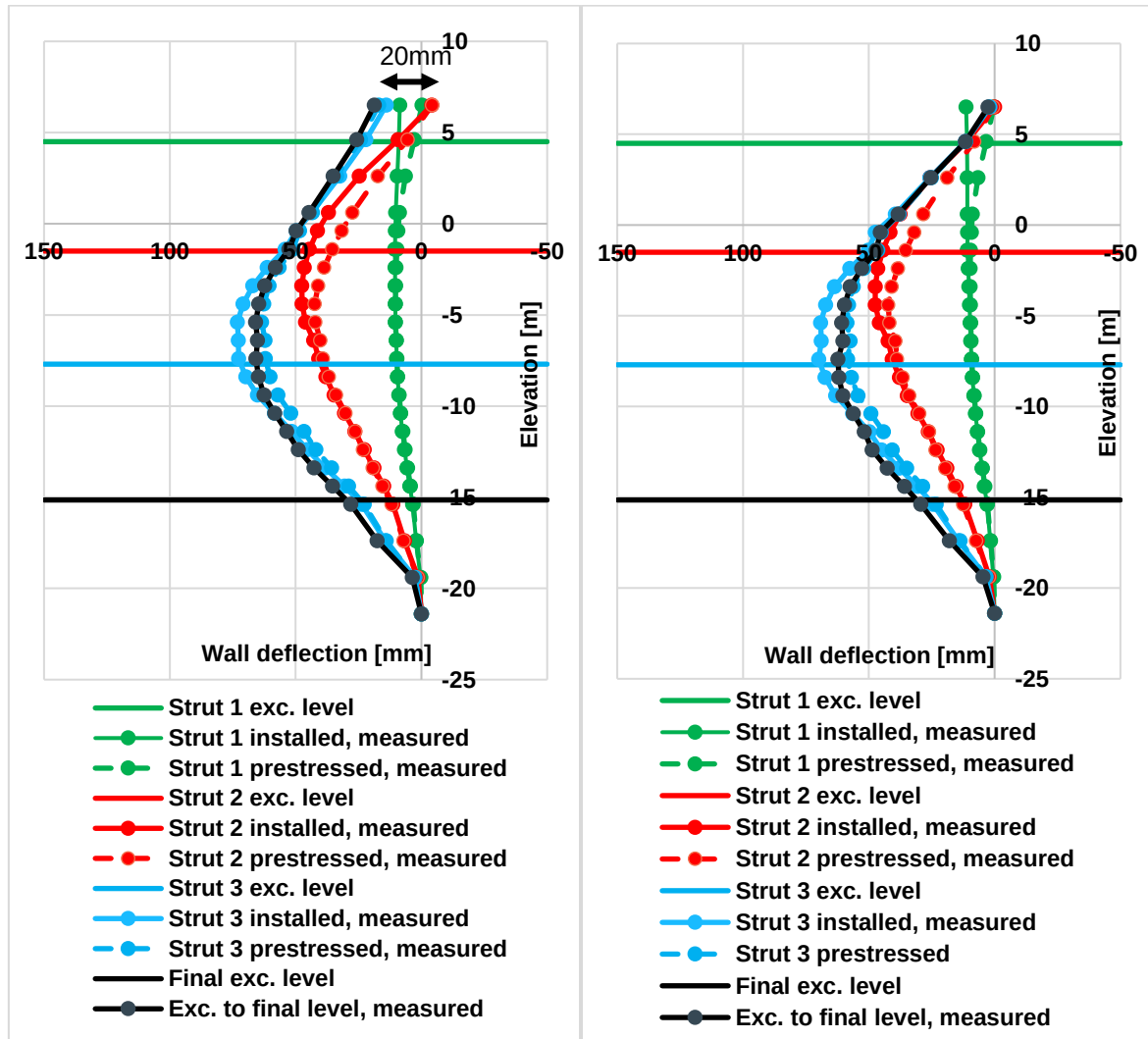


Figure 5.18 – Comparison of measured wall deflections (average values from struts R011/15), before correction (left) and after correction (right)

5.3.7. FINAL RESULTS OF THE CALIBRATION

The final deflections are presented in Figure 5.19 and the main observation to be take is that a reasonable and improved agreement has been found.

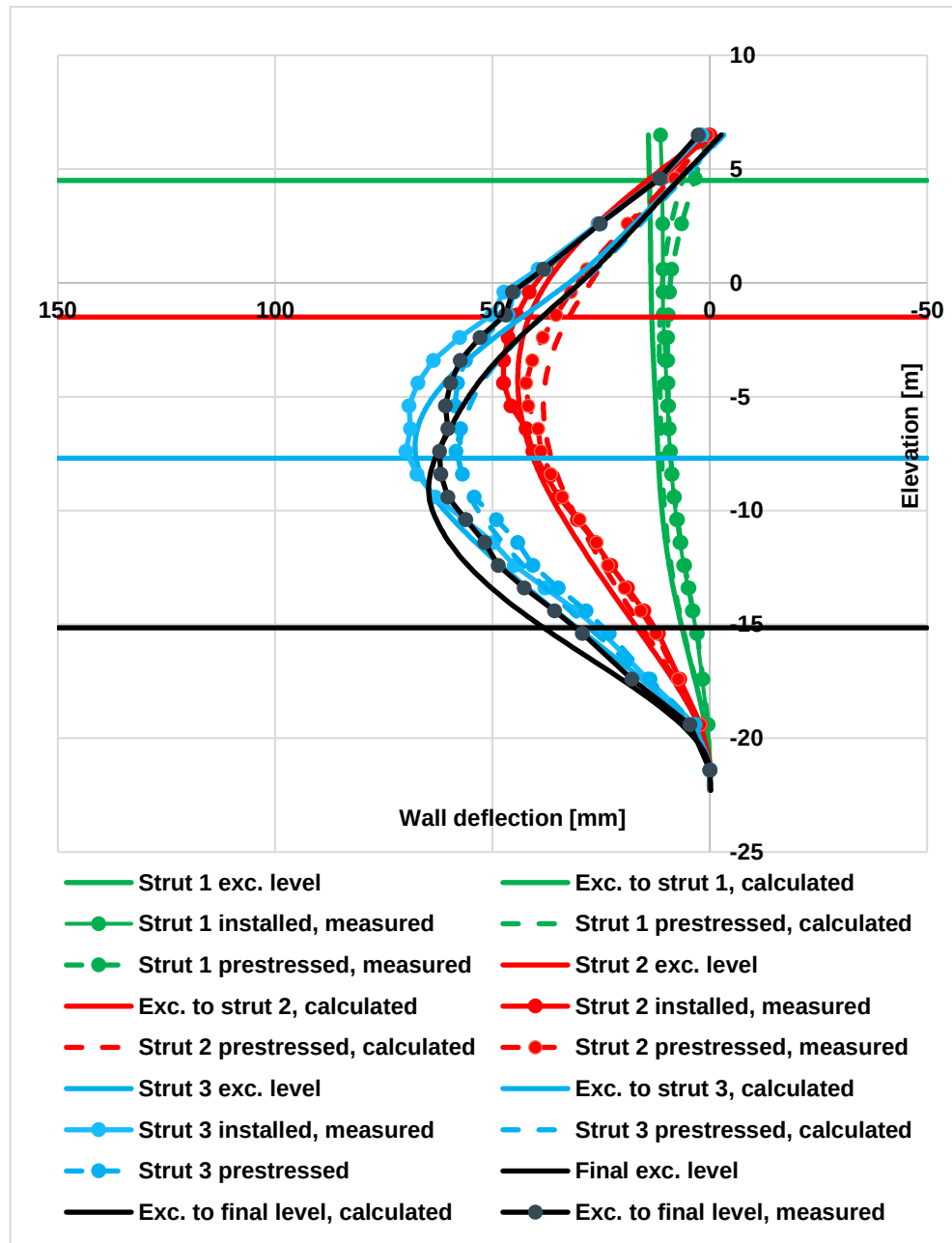


Figure 5.19 – Final comparison of calculated and measured wall deflections after back analysis

In Figure 5.20, the measured strut forces and temperatures during the entire construction period are compared to the calculated strut forces obtained from the Plaxis model. The solid lines represent the original design, while the dashed lines represent the results after the back analysis. Here are the key observations:

- For strut level 1, both models show good agreement with the calculations until after the prestressing of strut level 2.
- For strut level 2, the measured maximum strut forces during the construction stages are noticeably smaller than the calculated forces from the original model. However, after the calibration, the forces exhibit good agreement before the installation of strut level 3. After the installation of strut level 3 there is better agreement, although they still remain significantly larger than the measurements.
- When it comes to strut level 3, the strut force experiences a considerable decrease after the back analysis, dropping from nearly 12000kN to slightly under 8500kN. However, the measured maximum strut forces during the construction stages are still smaller than the calculated forces for both the original and back analysis models.
- At final excavation level, the sum of the measured strut forces is in the order of 11300 kN. The corresponding calculated value for the original design is 18000 kN. The corresponding calculated value for the calibrated model is 14000 kN. Despite the improvement at the end the sum of all calculated forces is still much higher than the measured.

It's important to note that all these values, both from measurements and calculations, are considered without considering the influence of temperature fluctuations.

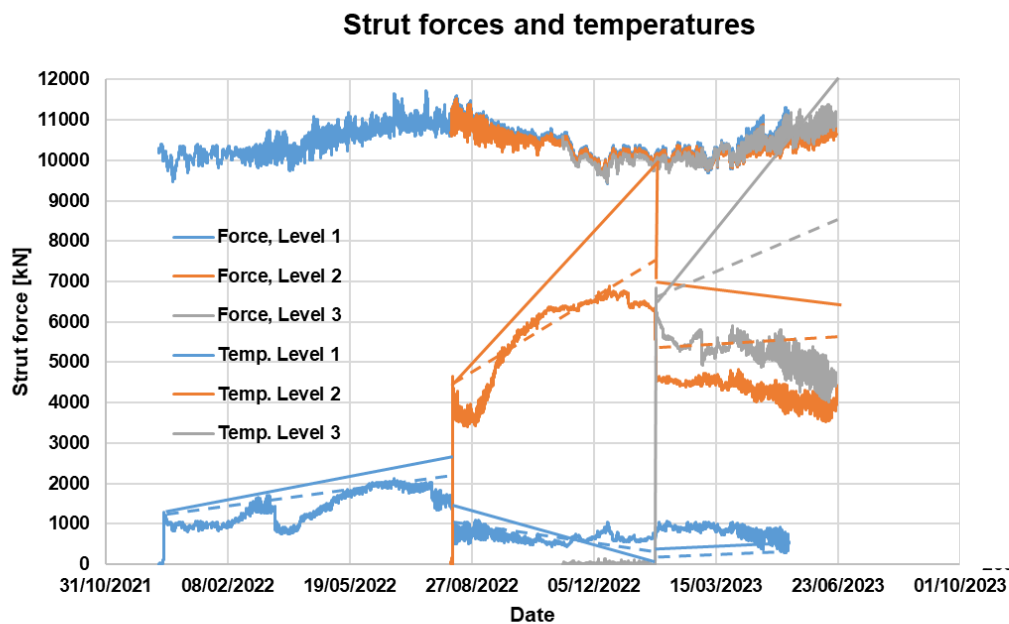


Figure 5.20 – Comparison of measured strut forces and temperatures and Plaxis calculations, original design model (solid lines) and after back analysis (dashed lines)

The observations made by the continuous monitoring of this part of the structure and the results and conclusions taken from this back analysis can both also be found in a presentation given at the Påldag 2024 conference (Kasper T. et al 2024).

5.4. TEMPERATURE MODELLING

In PLAXIS 2D, thermal effects can be considered for both soil and structures when significant temperature changes occur. Thermal calculations are used to determine the temperature distribution within the ground. These calculations are often coupled with groundwater flow calculations due to the influence of groundwater on heat transport. PLAXIS 2D allows for fully coupled transient thermo-hydro-mechanical (THM) calculations, which consider temperature changes, stress, deformation, and groundwater flow simultaneously.

To perform thermal calculations, specific conditions, parameters, and calculation types must be defined. Thermal boundary conditions, illustrated in Figure 5.21, are crucial and are used to specify the temperature distribution across the model's geometry. Several types of boundary conditions are available, such as temperature imposition on a line or cluster, prescribed flux at the boundary (inflow or outflow), and other options.

In addition, PLAXIS 2D allows for the definition of climate conditions to simulate convective effects resulting from changes in air temperature.

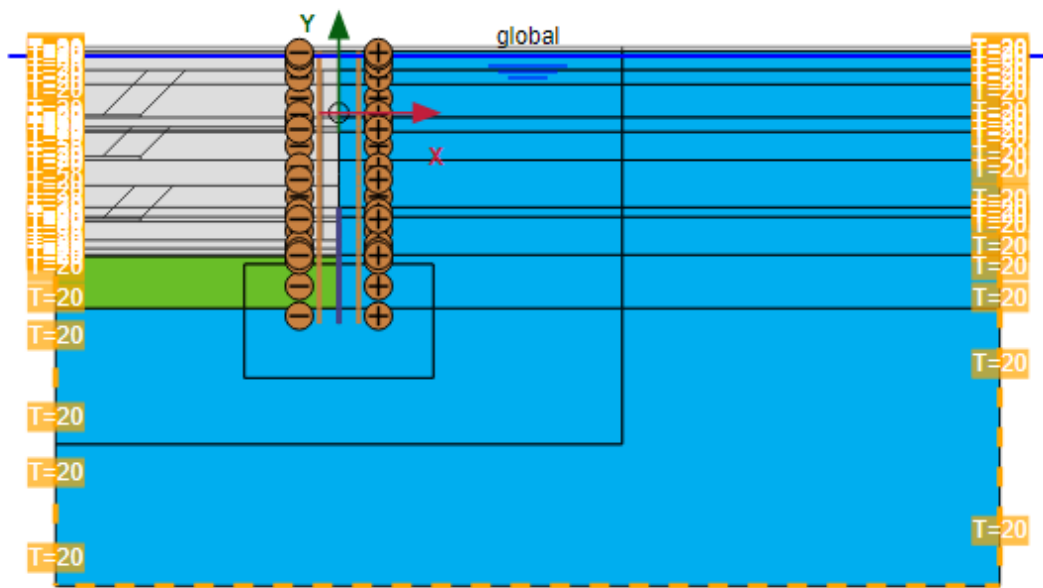


Figure 5. 21 – Plaxis model thermal boundary conditions

5.4.1. VALIDATION OF THE USE OF THERMAL CONDITIONS

In order to try and validate the use of the thermal conditions in cases such as the ones presented in this thesis, a simpler model was made on Plaxis 2D to be compared with hand calculations. The model, shown in Figure 5.22, was of an 8-meter deep, 20-meter-wide excavation with 2 strut levels and only 2 soil layers.

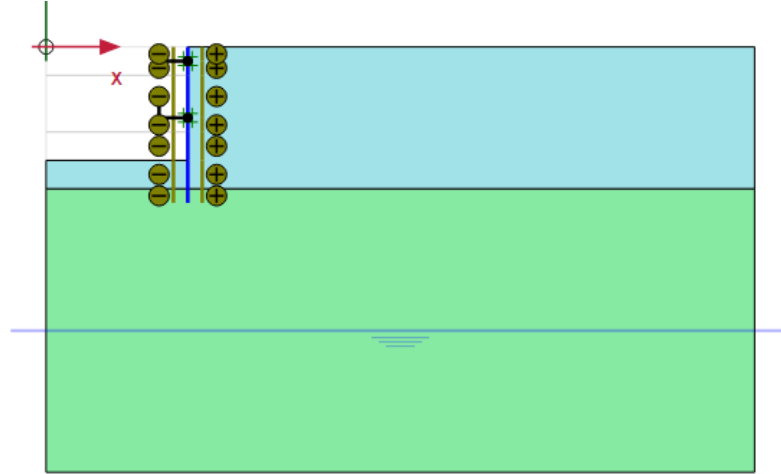


Figure 5.22 – Simple model used for thermal validation

Both struts were modelled as fixed-end-anchors and given an axial stiffness, EA of 11800 MN and the thermal parameters presented on Table 6.1. The excavation in the model was done in stages with a temperature in the model of 20°C. After final excavation a final phase was added where the temperature was changed to 40°C and the struts were fully restrained.

With a $\Delta T = 20^\circ\text{C}$ the expected result was the following ($\beta = 100\%$):

$$N_{temp} = 0,000012 \cdot 20 \cdot 11800000 \cdot 1 = 2823 \text{ kN} \quad (5.3.)$$

The results taken from the model before and after the heating phase were the following:

$$\text{For strut level 1: } N_{temp} = 3721,62 - 899,62 = 2823 \text{ kN} \quad (5.4.)$$

$$\text{For strut level 2: } N_{temp} = 3514,67 - 682,67 = 2823 \text{ kN} \quad (5.5.)$$

$$(N_{\text{After Heating}} - N_{\text{Before Heating}} = N_{\text{due to temperature}})$$

These results validate that at least for fully restrained situations Plaxis can give precise agreement with hand calculations for the force of fully restrained for fixed end anchors. A chapter of a manual published by Bentley in 2015 also has the validation of the use of heat flow in two-dimensional conditions for soil elements and structural elements such as node-to-node, beam, and fixed-end-anchors (Plaxis 2015).

5.4.2. INPUT INFORMATION

As only the thermal analysis of the struts is meaningful for this study the only thermal properties attributed where to the struts, parameters given can be seen on Table 5.10. This way all other structures and soil layers will have no direct reaction due to the attribution of a temperature to the model.

Table 5.10 – Struts thermal properties

Strut Properties	
Specific heat capacity [J/kg/°C]	460
Thermal conductivity [W/m/°C]	45
Density [t/m ³]	7,8
Thermal expansion [1/°C]	0,000012

As said before Plaxis offers a wide range of conditions and calculation types in order to better represent each situation in question. The following points explain the different conditions used in this case and if relevant how they were defined, the definitions for each of the following were taken from the Plaxis 2D Reference Manual (Plaxis 2023a):

1. Thermal Calculation Type:
 - Earth gradient: The initial temperature is calculated based on assigned values of the ThermalFlow condition.
 - Use temperatures from previous phase: The temperature distribution remains the same as at the end of the previous phase.
 - Steady state thermal flow: Calculates temperature using a steady-state thermal flow finite element calculation. It relies on Thermal flow boundary conditions and appropriate thermal model parameters.
2. Model Conditions:
 - Thermal flow boundary conditions: Behavior set as "temperature," distribution as "uniform," Tref (reference temperature) as "20°C " and time dependency as "constant."
 - ThermalFlow: Tref set as "20°C " and h_{ref} (reference depth) set as the ground surface ("7.0m").
 - Climate: Tref varies from phase to phase, surface transfer set as "1 kW/m²/C," and time dependency as "constant."
3. Calculation Type:
 - Fully coupled flow-deformation: Time-dependent analysis of deformation and pore water pressures. When combined with temperature calculation, it results in a fully coupled Thermo-Hydro-Mechanical (THM) analysis.

The implementation of these points in the model is explained in more detail in section 5.4.3.

5.4.3. MODELLING SEQUENCE

To make a thermal analysis on Plaxis 2D, the temperature must be considered in all phases of construction, Figures 5.23 and 5.24 illustrate, respectively, the temperature in the model during the construction phases and after the temperature increase. With that in mind the following changes to the staged construction were made:

- In the initial phase: Thermal calculation type is set as “earth gradient”, ThermalFlow in Model conditions is activated, Tref is set as “20°C” and h_{ref} is set as the ground surface.
- In all subsequent phases until the final excavation the only thing changed is the Thermal Calculation Type which is set to “Use temperature from previous phase”, this means that the temperature that will be considered in phases of construction is the same as in the initial phase, “20°C” (ThermalFlow is turned off). As the soil and wall have no thermal parameters attributed to them, the temperature has no effect on them. The struts have thermal parameters, but as the temperature remains constant no change is noticeable in these phases when compared to the model before the introduction of thermal analysis. Figure 6.1 shows the temperature distribution through the soil during these phases.

After the final excavation two new phases are created:

- Firstly, the Thermal calculation type is set as “Steady state thermal flow”; Thermal boundary conditions are activated; Climate condition is activated with air temperature at 20°C.
- In the final phase the Calculation Type is set as “Fully Coupled deformation” and Thermal calculation type as “Use temperature from previous phase”; A time interval of 10 days is attributed; Thermal boundary conditions remain activated; In the climate condition the air temperature is set to 35°C.

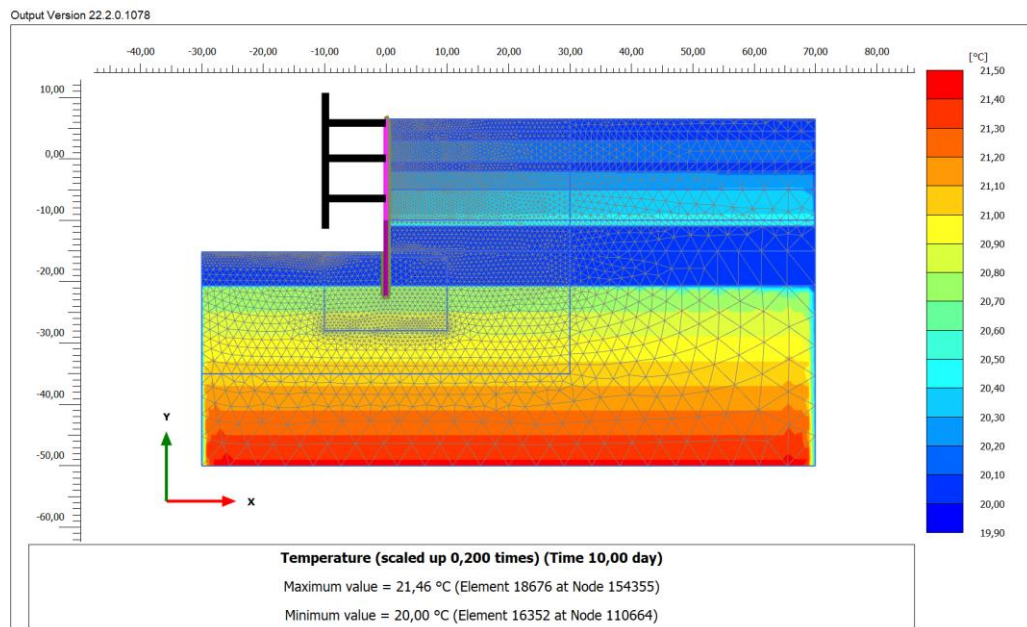


Figure 5.23 – Model temperature until final excavation

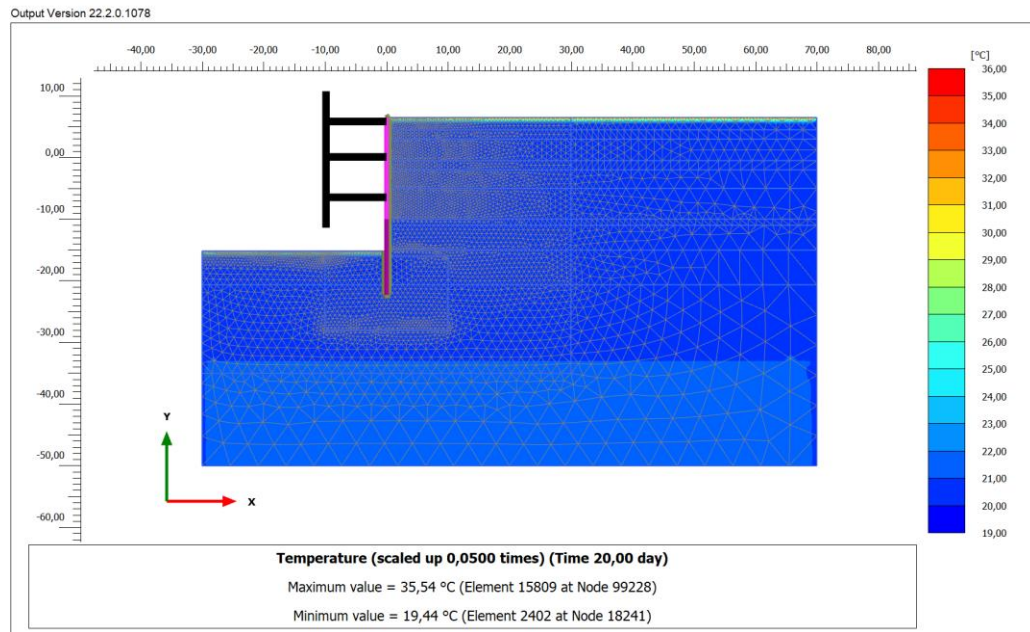


Figure 5.24 – Model after imposed temperature increase

5.4.4. RESULTS

The results presented in Table 5.11 were obtained from the Plaxis model, considering a temperature increase of the struts from 20 to 35 degrees Celsius, resulting in a delta T of 15 degrees Celsius. Two scenarios were analysed:

1. Fully restrained model: In this situation, all the struts are assumed to be fully restrained after the final excavation, meaning that they are not allowed to expand. This corresponds to a restraint coefficient of 100%, and it is expected that the load increase due to temperature will be at its maximum. The results for strut levels 1 and 2 demonstrate almost perfect agreement with the expected restraint coefficient value of 100%. However, for strut level 3, a slight discrepancy is observed, indicating that the actual load increase was 10% higher than initially anticipated.
2. Unrestrained model: In this case, no additional restraints are applied to the struts, apart from the inherent stiffness of the soil and wall system. It is important to note that the Plaxis model was calibrated to closely approximate actual field conditions. Therefore, for this scenario, it is expected that the restraint coefficient should align closely with the average value derived from the measured values for each respective strut level. The results exhibit an increase in the load values in lower depths. This trend is in line with the anticipated behaviour.

Table 5.11 – Plaxis results for strut forces

Strut Level	Y [m]	ΔN [kN]	β [%]
Fully Restrained			
1	5,85	-2123,16	99,99
2	0,10	-3847,50	100,94
3	-6,45	-4635,14	110,74
No Restraint			
1	5,85	-261,10	12,30
2	0,10	-612,21	16,05
3	-6,45	-1210,44	29,19

Table 5.12 exhibits the results of the horizontal deformation of the strut calculate by the model. As expected, the results decrease from the highest to the lowest strut level, i.e., the retaining system gets stiffer with depth, so the strut is more restrained and is not allowed to expand as much.

Table 5.12 – Plaxis results for struts horizontal expansion

Strut Level	Y [m]	ΔL [m]
1	5,85	0,00474
2	0,10	0,00454
3	-6,45	0,00399

6

COMPARISON OF RESULTS

6.1. MEASURED RESULTS VS CALCULATED RESULTS

To calculate the results presented in this chapter, the equations previously introduced in Chapter 2 were utilized with the following values for α , ΔT , E , and L as in equation (6.1.) The sections area A remained the same as in Chapter 5.

$$N_{temp} = \alpha \cdot \Delta T \cdot E \cdot A \cdot \beta = 1.2 \cdot 10^{-5} \cdot 15 \cdot 210 \cdot 10^6 \cdot A \cdot \beta = A \cdot \beta \cdot 37.8 \text{ kN} \quad (6.1.)$$

$$\Delta L = \alpha \cdot \Delta T \cdot L \cdot (1 - \beta) = 1.2 \cdot 10^{-5} \cdot 15 \cdot 30 \cdot (1 - \beta) = 0.0054 \cdot (1 - \beta) \quad (6.2.)$$

Table 6.1 and Figure 6.1 show the comparison between the measure and calculated results for the strut forces. The values referred to as "measured" and "CIRIA" were determined with equation (6.1.). The computed values were obtained from the Plaxis model, which provided the strut force values before and after the temperature increase. ΔN was subsequently calculated, followed by the restraint coefficient.

Table 6.1 – Comparison of load increase and restraint coefficient results

Strut Level	$\beta_{\text{measured}} [\%]$	$\Delta N_{\text{measured}} [\text{kN}]$	$B_{\text{CIRIA}} [\%]$	$\Delta N_{\text{CIRIA}} [\text{kN}]$	$B_{\text{comp}} [\%]$	$\Delta N_{\text{comp}} [\text{kN}]$
1	14,3	302,6	30	637,0	12,30	261,1
2	20,8	795,5	30	1144,7	16,05	612,2
3	38,2	1582,7	30	1244,2	29,19	1210,4

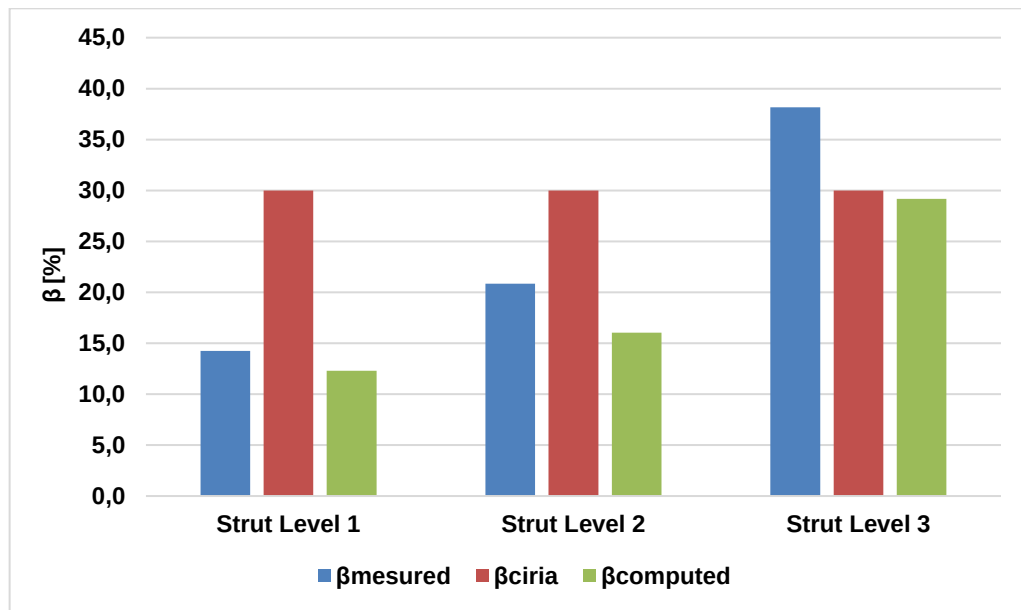


Figure 6.1 – Comparison of restraint coefficients

Table 6.2 and Figure 6.2 show the comparison between the measure and calculated results for the strut expansion. The "measured" and "CIRIA" values were calculated using equation (6.2.). The computed ΔL was directly extracted from the Plaxis model (horizontal expansion, u_x , of the strut after heating), and then the computed restraint coefficient was retroactively determined using equation (6.2.)

Table 6.2 – Comparison of strut deformation and restraint coefficient results

Strut Level	β_{measured} [%]	$\Delta L_{\text{measured}}$ [m]	B_{CIRIA} [%]	ΔL_{CIRIA} [m]	β_{computed} [%]	$\Delta L_{\text{computed}}$ [m]
1	14,3	0,00463	30	0,00378	12,30	0,00474
2	20,8	0,00427	30	0,00378	15,91	0,00454
3	38,2	0,00334	30	0,00378	26,11	0,00399

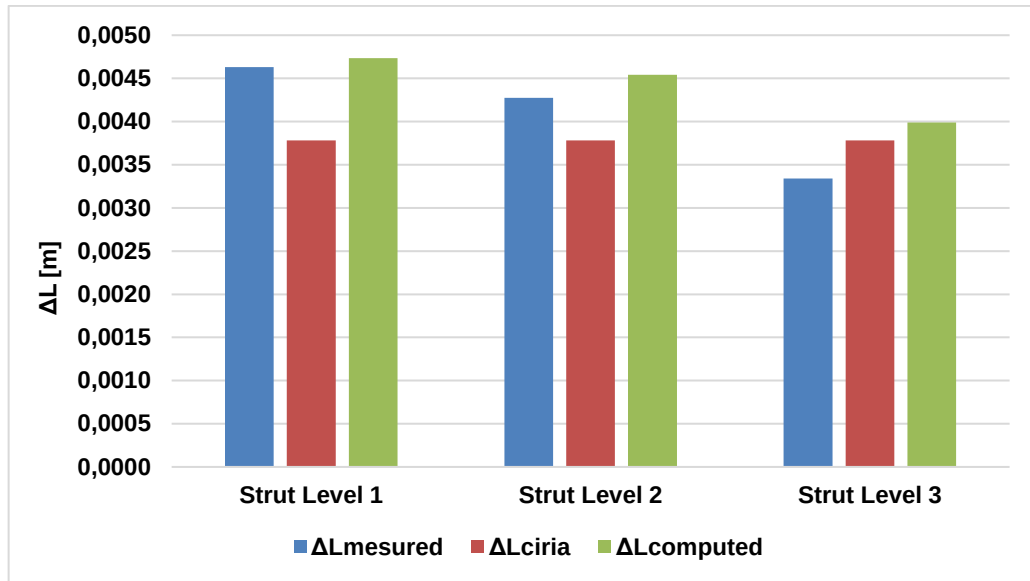


Figure 6.2 – Comparison of strut expansion

Several observations can be made based on the analysis of the results:

- Overall, the restraint coefficient shows good agreement between the measured and computed values. However, when comparing the values taken from the Plaxis model for all levels, it is evident that they consistently underestimate the force increase. Notably, the difference between the measured and computed values is more pronounced for the lower-level strut. For example, it is found that the difference between the measured and computed values is 2% for level 1, 4% for level 2, and 10% for level 3.
- In terms of the chosen value of 30% for the project, based on CIRIA recommendations, it is important to note that it is higher than the measured values of 14% and 20% for strut levels 1 and 2, respectively. This suggests that the load increase considered during the design phase was lower than the actual loads experienced in practice. In other words, the design approach followed a conservative approach, which can be beneficial from a safety standpoint. However, for strut level 3, the calculated restraint coefficient resulting from the measured value was found to be 8% higher. This indicates that the design underestimated the loads caused by temperature fluctuations, and therefore may have been non-conservative in this particular case.
- The restraint coefficients calculated from the ΔN and ΔL values derived from the Plaxis results demonstrate excellent agreement with each other. The deviation between the calculated restraint coefficients is minimal, with only a small discrepancy of 3% observed at the final strut level.
- Lastly, when comparing the deformations between the measured and computed results, there is generally excellent agreement. However, it is worth noting that the computed results tend to exhibit slightly higher values compared to the measured values. The largest difference observed between the measured and computed deformations is 0.65 mm.

6.2. MEASURED RESULTS VS CIRIA RECOMMENDATIONS

The following diagrams, in Figures 6.3 to 6.5, have the results for the average restraint coefficient for the three strut levels at all construction sites in relation to the value recommended by CIRIA.

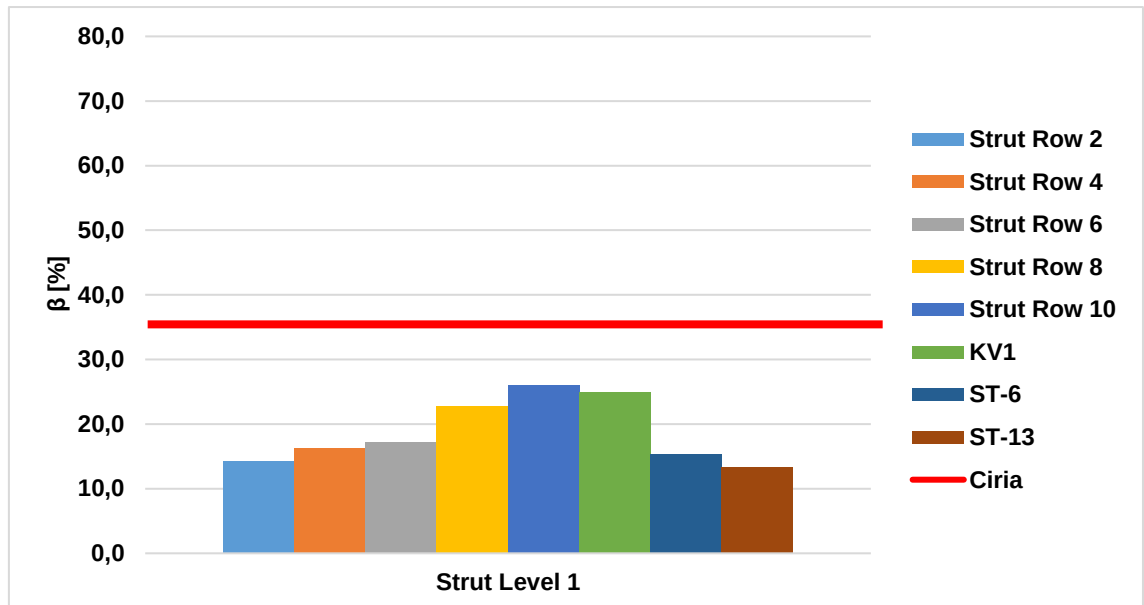


Figure 6.3 – Comparison average restraint coefficient and CIRIA recommendation, strut level 1

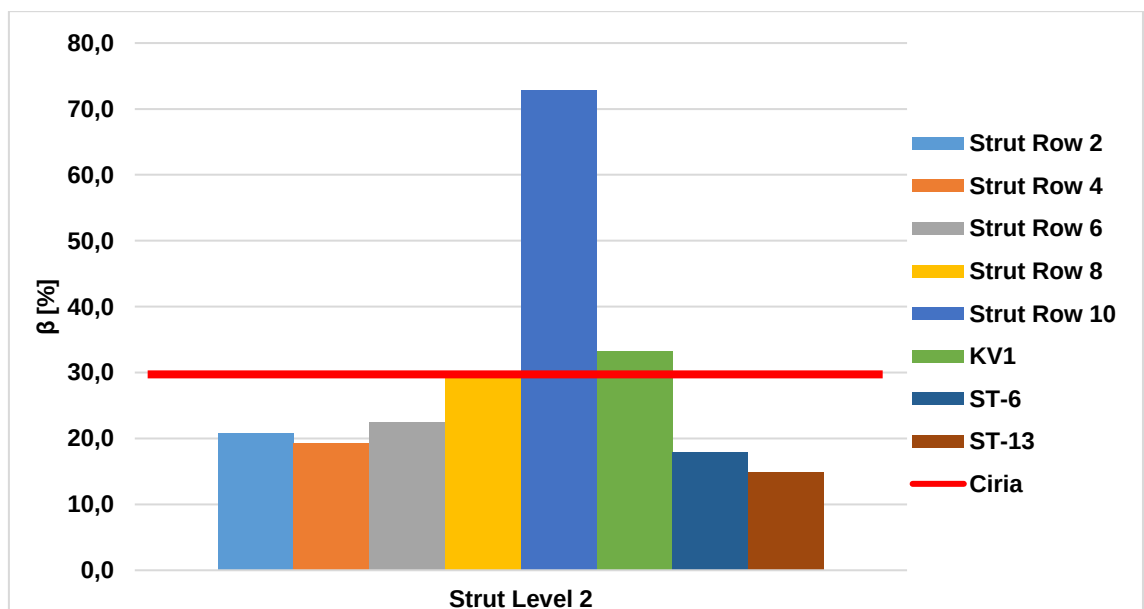


Figure 6.4 – Comparison average restraint coefficient and CIRIA recommendation, strut level 2

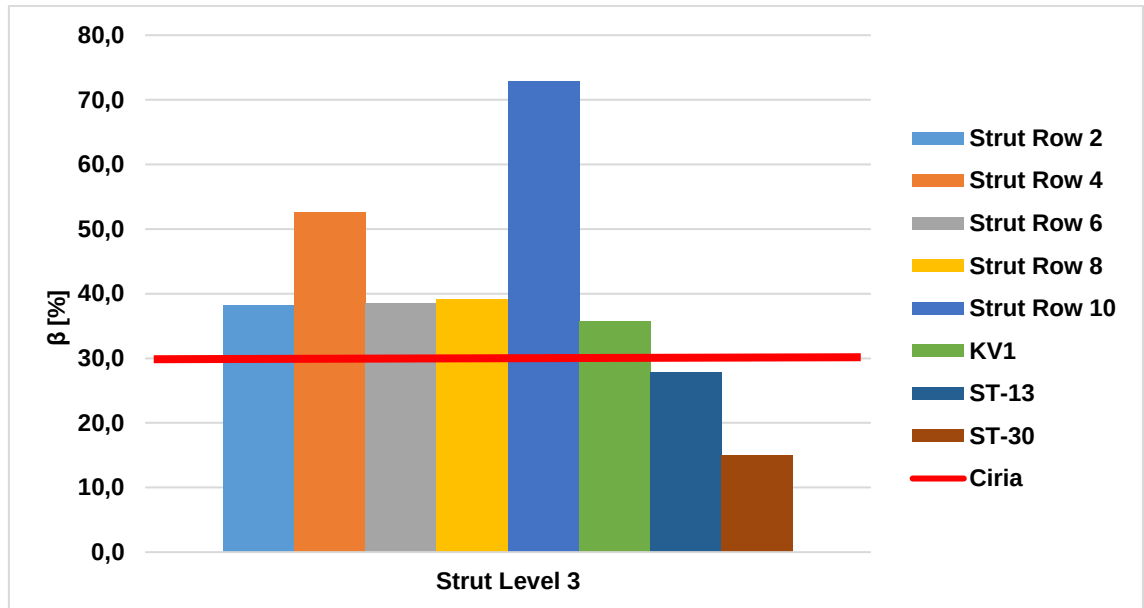


Figure 6.5 – Comparison average restraint coefficient and CIRIA recommendation, strut level 3

These diagrams provide further support for the information discussed in section 6.1. The following observations can be made:

- For strut levels 1 and 2, the results once again confirm that the recommendation of 30% for the restraint coefficient is generally reasonable and not overly conservative for the majority of the struts. However, it is important to note that there is one particular case where a significantly higher value is observed, which is with strut row 10 on Korsvägen 2. As previously mentioned, this higher value is attributed to the proximity of this specific strut to the rock surface. This localized condition necessitates a higher restraint coefficient for this particular strut.
- In the case of strut level 3, it can be observed that in most cases, a restraint coefficient of 30% does not appear to be sufficiently cautious. When the strut is not extremely close or fixed to the rock, an appropriate value for the restraint coefficient would range between 45% to 50%. This higher value is necessary to adequately account for the increase in loads caused by temperature fluctuations in these situations.
- The lower value observed for ST-13 and ST-30, as explained in section 4.4.4., can be attributed to the lower stiffness of the sheet pile walls and the lack of rotational restraint at pile toe. These factors, which are unique to these struts, result in a deviation from the general trend observed for the other struts.

Overall, these findings provide additional evidence to support the proposed recommendations for the restraint coefficient, taking into account the specific conditions and constraints of each individual strut level. By considering these factors and tailoring the restraint coefficient accordingly, more accurate and reliable designs can be achieved.

On Table 6.3 it can be seen the value of the loads due to temperature used for the design of strut row 1 to 7 on the Korsvägen 2 construction site in relation to the maximum load increase removed from the average data of strut rows 2 and 4 using equation (2.2.) and considering the average restraint coefficient calculate for this same two struts.

Table 6.3 – Comparison load increase due to temperature used for the design and measured.

Strut Level	$\Delta N_{\text{temp, design}}$ [kN]	$\Delta N_{\text{temp, measured}}$ [kN]	Increase Factor
1	1741	753	2,31
2	3155	802	3,93
3	3401	3190	1,07

As can be expected from the results presented before the design values for struts 1 and 2 overestimate considerably the load increase. While level 3 shows that the restraint coefficient used was a good representation of the conditions encounter on site.

7

CONCLUSION

7.1. CONCLUSION

In the introduction of this thesis the initial objectives and questions, upon which the work done was based, were established. This conclusion will try to illustrate if these objectives were met and questions were answered, and if so to what degree.

Several of the conclusions take only indicate possible conclusions. So, to that effect possible future works that could be developed to increase the understanding of this problems will also be suggested in this chapter.

- According to monitoring data, what is the range of the restraint coefficient, β in typical Gothenburg ground conditions?

The answer to this question is not straightforward. The findings of this dissertation suggest that the restraint coefficient can be influenced by multiple factors, not just soil conditions, as explained in the following point. However, some general conclusions can still be drawn.

1. For rigid walls (Secant and RD piles): Struts located between 15 and 25 meters from the rock showed a range of 15 to 25%. Struts between 5 and 15 meters from the rock exhibited a range of 25 to 40%. Struts positioned within 0 to 5 meters from the rock recorded values higher than 70%.
2. For flexible walls (Sheet piles): The average β never exceeded 30%. For struts fixed on both sides of the sheet pile wall, all values ranged from 10 to 20%. For struts with one side fixed on the rock surface, the average coefficient was higher than 25%.

- What factors may have the greatest influence on the restraint coefficient of a determined strut?

In the previous point, it was highlighted that several factors can influence the restraint coefficient, with some factors having a more significant impact than others. Among these factors, the distance between the strut and the rock emerged as particularly influential. It was observed that the closer the strut is to the rock, whether vertically or otherwise, the higher the restraint coefficient. This relationship follows a semi-linear trend, indicating that proximity to the rock increases the level of restraint.

Another factor that was investigated is the bending stiffness of the wall. The study found a slight tendency for the restraint coefficient to increase with higher bending stiffness. This implies that walls with greater stiffness are likely to exhibit greater levels of restraint.

The average soil stiffness, on the other hand, did not yield any discernible pattern or substantial impact on the restraint coefficient within the stiffness range of 25 MPa to 45 MPa. This suggests that variations in soil stiffness within this range may not significantly affect the restraint coefficient.

Lastly, the study examined the pile embedment ratio. Although no direct correlation was established between the embedded length of the pile and the restraint coefficient, it can be hypothesized that pile embedment does have some influence over the restraint coefficient. This assumption is supported by the fact that when comparing the restraint coefficient of the rigid and the flexible walls it was noticeable that the flexible walls have much lower values. This disparity may suggest that the rotational restraint of the pile toe does play some role in influencing the overall restraint coefficient, and with that, so does the embedment.

- Calibrate constitutive models for relevant soil types by back-analysis to measurement data.

This objective served as an intermediate step in the overall process and was crucial for obtaining an accurate depiction of the real conditions. As mentioned earlier, certain assumptions were made in order to achieve the best possible estimation of the soil and structure stiffness. For instance, the representation of Lime-cement stiffness as decreasing over depth was based on hypotheses. Consequently, the model may not at times perfectly capture all the intricate details of the case.

- Can Plaxis simulations (with best estimate soil parameters) reliably predict the temperature restraint coefficient, i.e., match the monitoring data.

The calculated restraint coefficient using Plaxis in the selected case shows results that are generally in the same order of magnitude as the values resulting from measurements taken on site, although with an overall underestimation of approximately 20%. Since only one comparison scenario was studied, it is not possible to definitively determine the exact cause of this underestimation.

- Based on the above, is it possible to propose a more accurate temperature restraint coefficient for these types of walls in these ground conditions to allow for designers and contractors to optimize future projects?

If anything, what can be concluded in this regard is that optimization can be achieved by analyzing the specific conditions of a set of struts in a more detailed manner.

Overall, considering the results obtained, the values recommended by CIRIA do seem to have some merit, bearing in mind that they are supposed to encompass a every singular situation, situations this that might be in very different conditions.

According to CIRIA, for stiff walls situated in stiff ground conditions, a restraint coefficient of 50% is suggested, while for flexible walls in similar ground conditions, a coefficient of 30% is recommended. It its observed that the recommendation is made for stiff ground conditions only, the conditions encountered in this work are of a very soft ground. With that said, with the results it is possible to conclude that for a majority of scenarios a value of 30% for a rigid wall in soft ground conditions does seem reasonable. But for a situation with a flexible wall in soft ground conditions a lower value might not be a bad idea, perhaps 20%.

7.2. FUTURE DEVELOPMENTS

The subject of this thesis pertains to a specific and practical area of study. While some research has been conducted in this field, there is still a need for further investigation to enhance our understanding of the behavior of struts and soil in different situations, considering the inherent complexity of each unique case. To complement the work carried out in this study, the following proposals are suggested:

- **Exploring the Influence of Soil Stiffness:** Initially, it was expected that soil stiffness would have a greater impact on restraint than the stiffness of the retaining system. However, this was not observed in the scenarios studied, which involved rigid RD piles. It may prove beneficial to investigate more flexible walls, such as sheet piles in the Almedal area, to determine if soil stiffness holds greater relevance in such cases.
- It would also be quite relevant for other scenarios to be modelled on Plaxis, the back analysis for the secant pile and sheet pile walls, might be important to confirm that the best estimate parameters used, and all other changes made to the model actually improve the model to a better representation of reality. Subsequently the temperature analysis can be conducted to address if the underestimation of the coefficient was perhaps caused by an underestimation of the stiffness of the retaining system or if Plaxis will always underestimate the temperature restraint coefficient.

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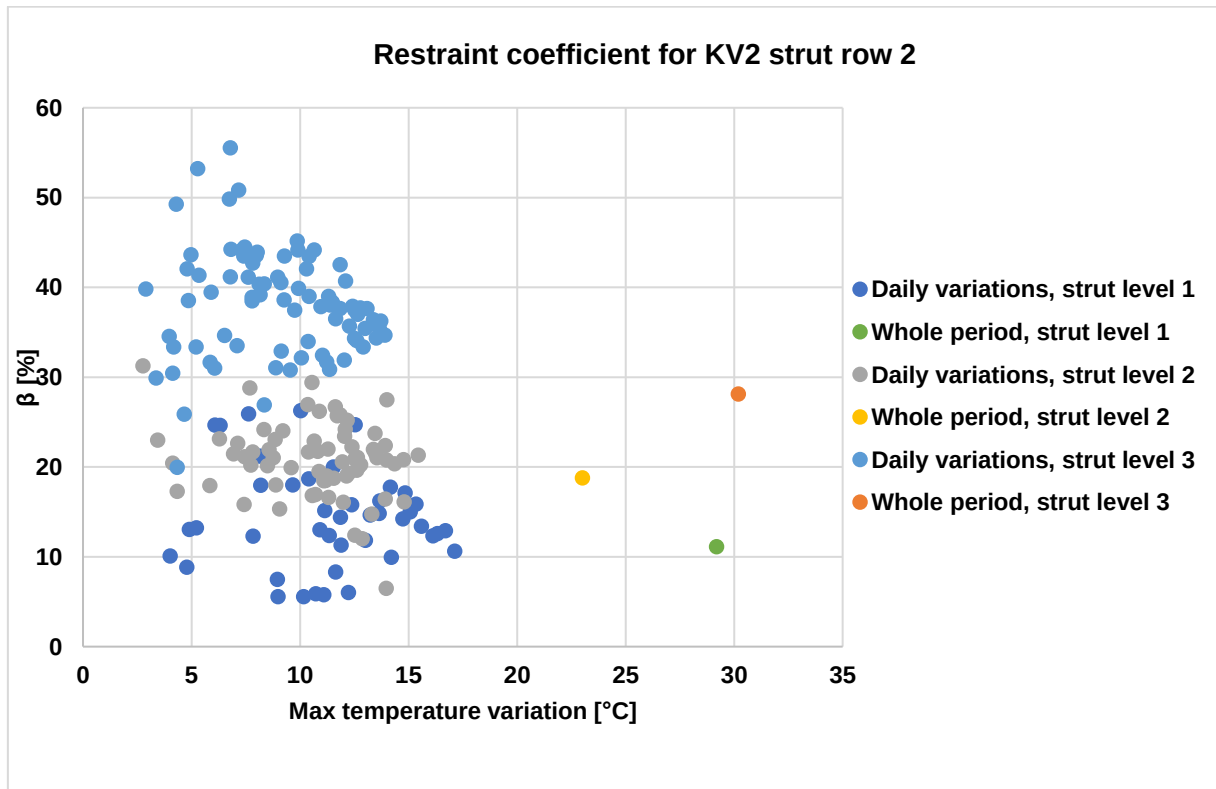
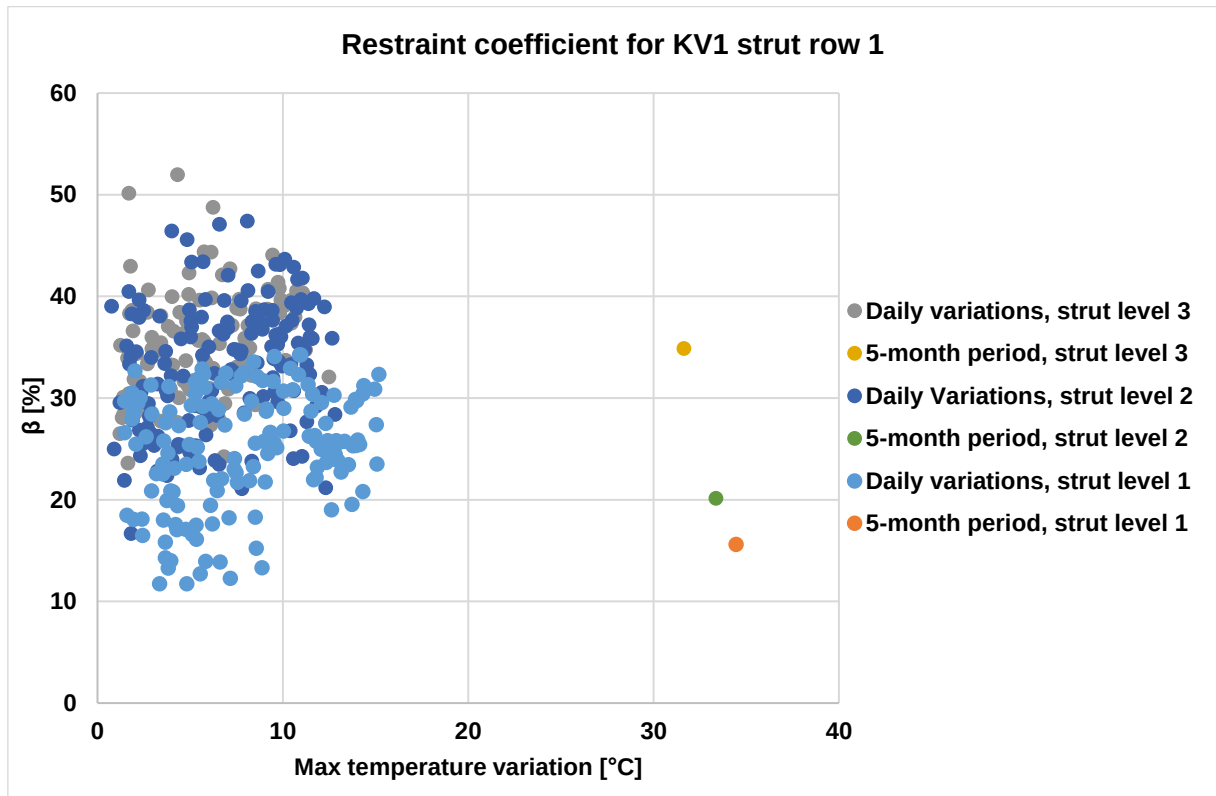
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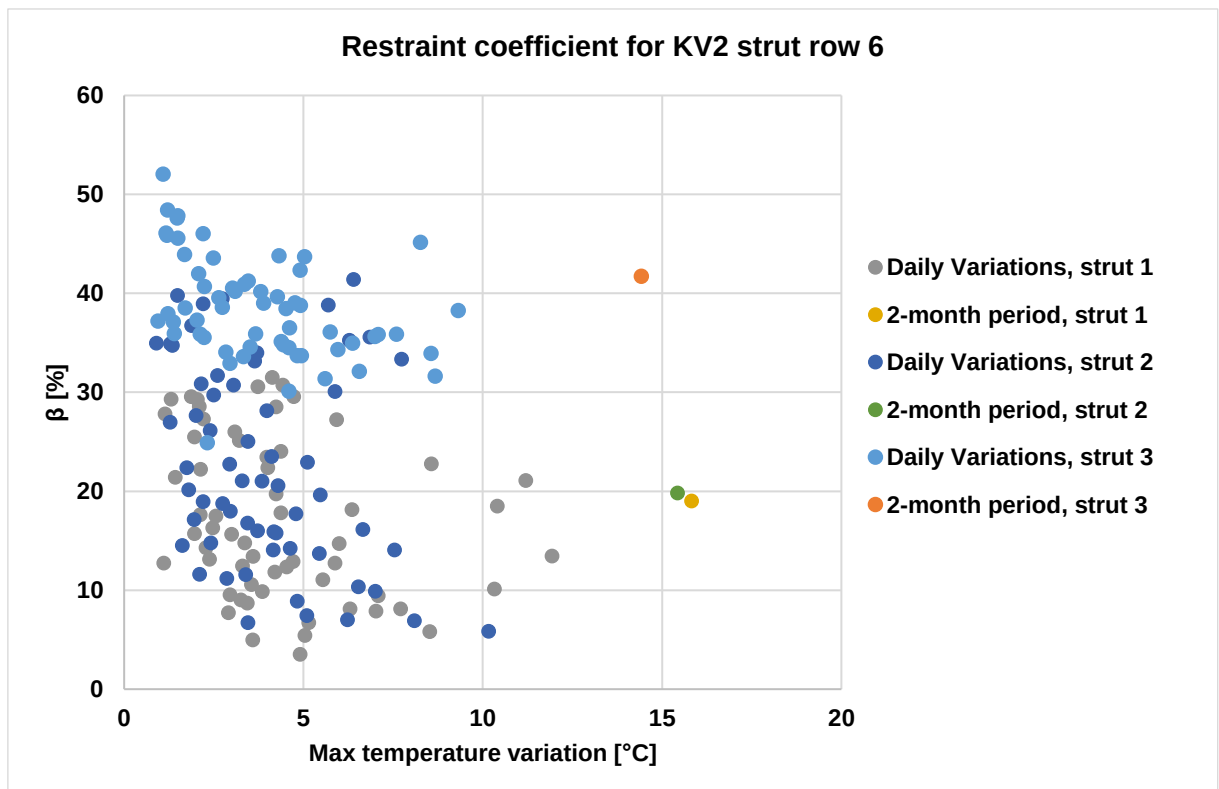
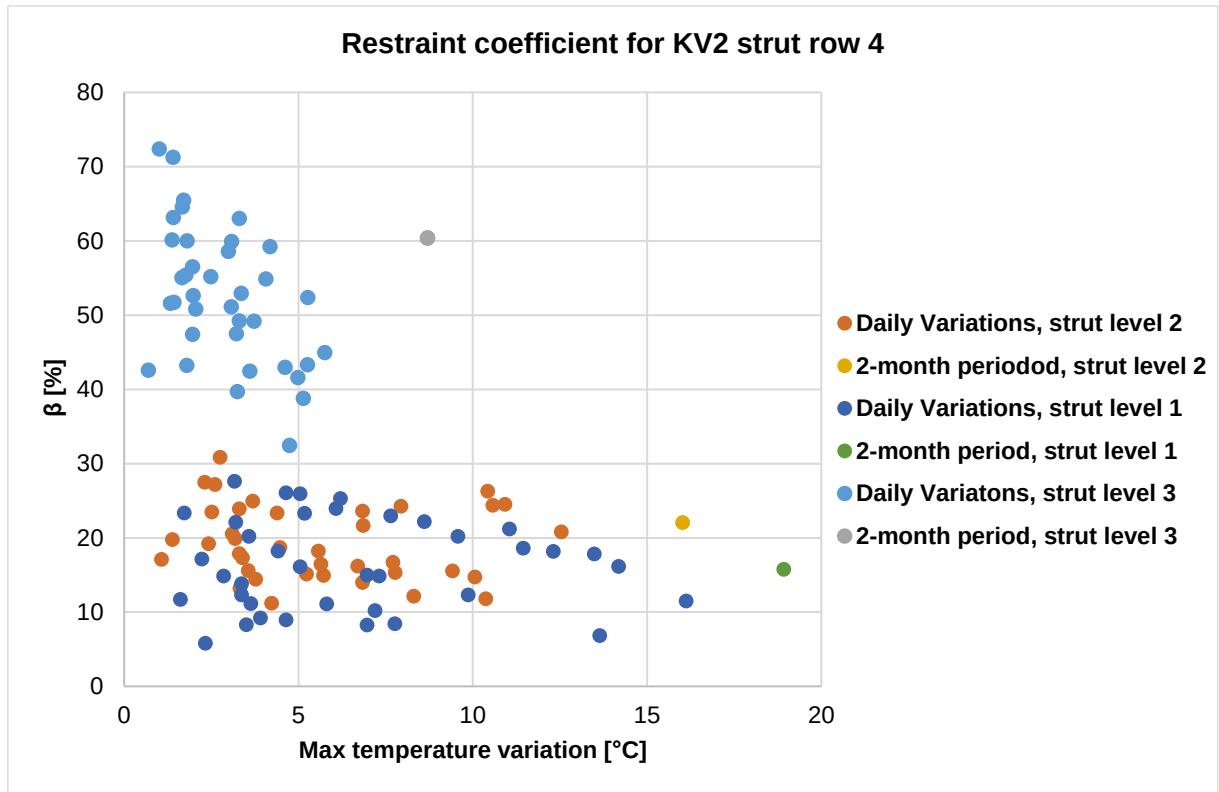
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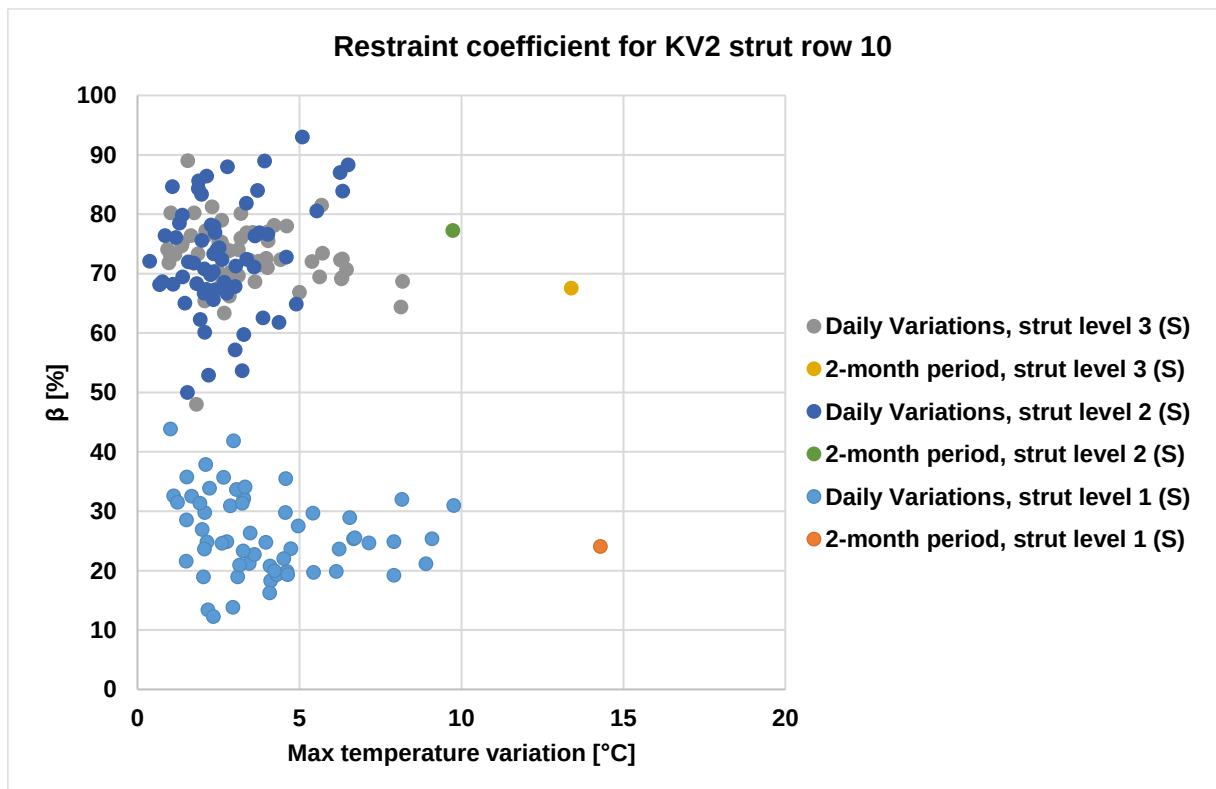
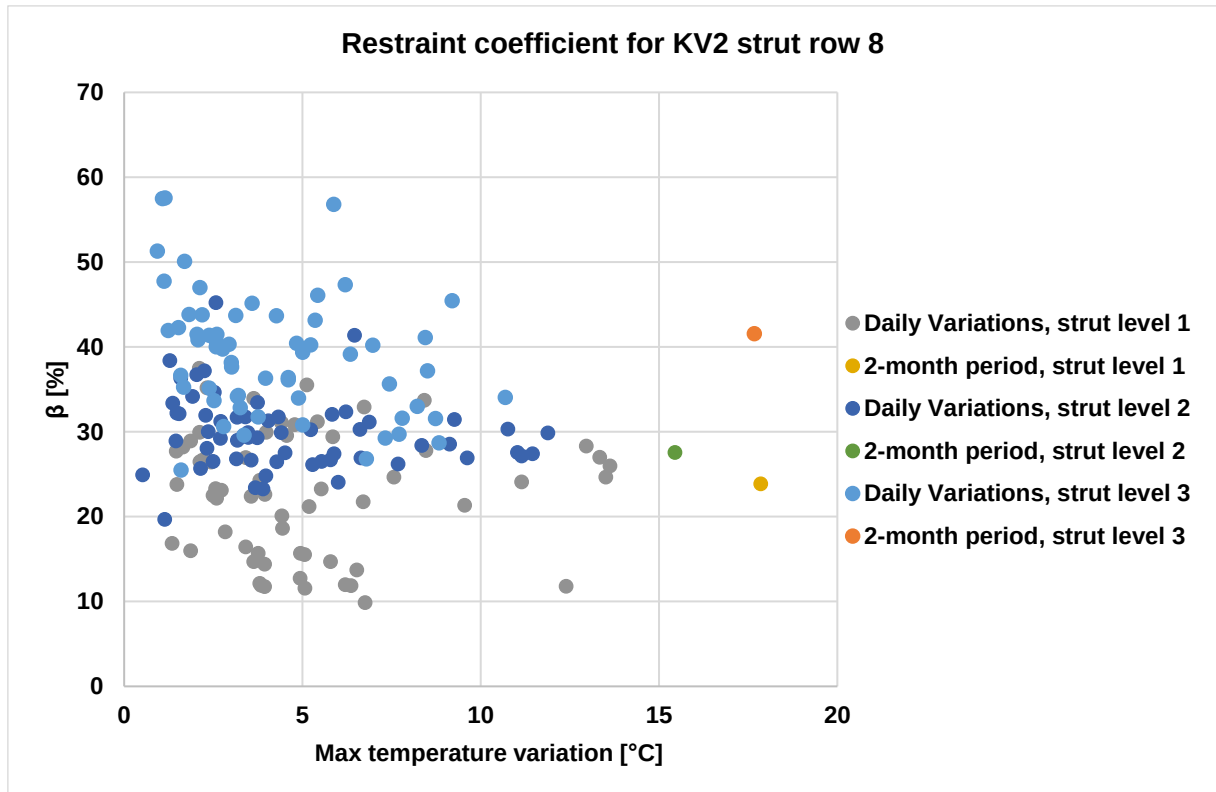
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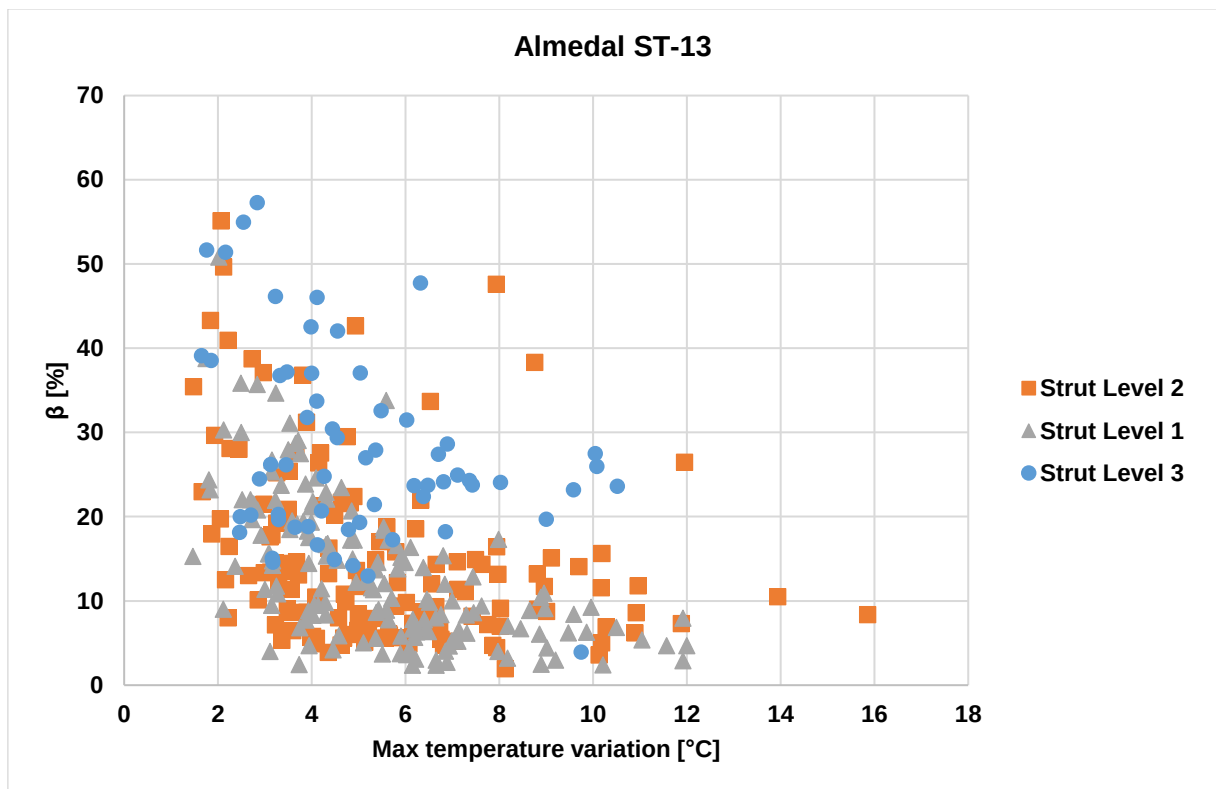
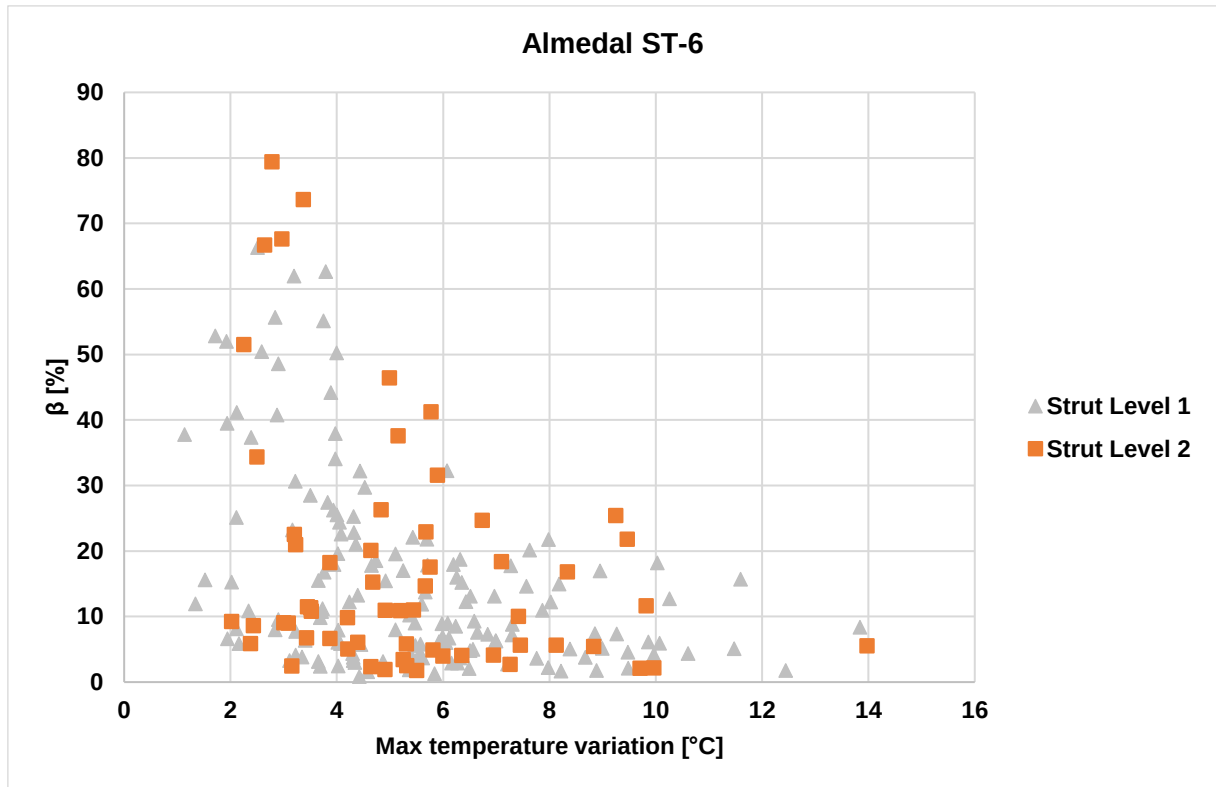
APPENDIX

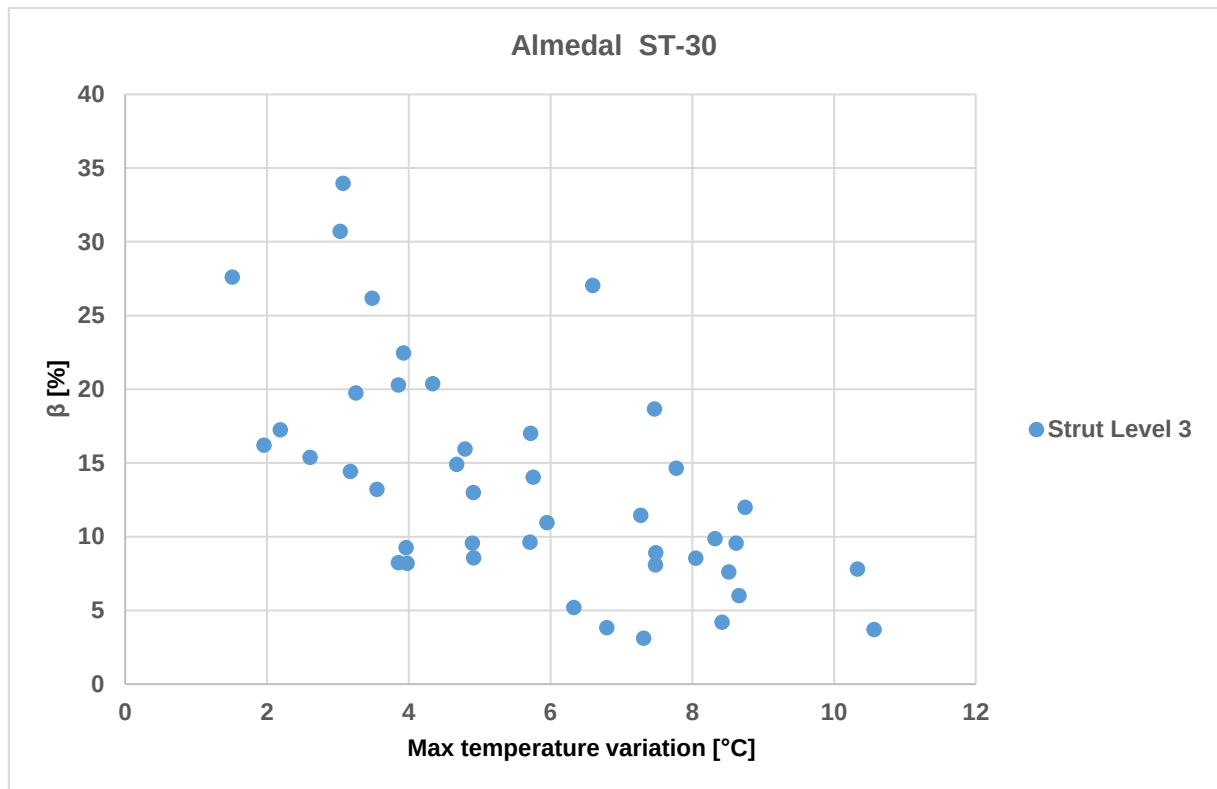
A1. DAILY RESTRAINT COEFFICIENTS











KV2					
	Strut Level	Average β	2-month period	Max β	Min β
Strut Row 2	1	14,3	11,1	26,3	5,6
	2	20,8	18,8	31,3	6,5
	3	38,2	28,1	55,5	20,0
Strut Row 4	1	16,3	15,7	27,6	5,8
	2	19,3	22,1	30,8	11,2
	3	52,5	60,4	72,3	32,4
Strut Row 6	1	17,3	19,0	31,5	3,5
	2	22,4	19,8	41,4	5,9
	3	38,5	41,7	52,1	24,9
Strut Row 8	1	22,8	23,9	37,5	9,9
	2	29,8	27,6	45,2	19,7
	3	39,1	41,6	57,5	25,5
Strut Row 10	1	26,0	24,1	43,9	12,3
	2	72,9	77,3	93,0	50,0
	3	72,8	67,5	89,0	48,0
KV1					
Strut Row 1	1	25,00	15,62	34,24	11,72
	2	33,22	20,13	47,43	16,68
	3	35,69	34,88	51,98	23,61
Almedal					
ST-6	1	15,4	26,9	66,3	0,8
	2	17,9	43,8	67,6	1,9
	3	-	-	-	-
ST-13	1	13,3	15,0	50,8	2,3
	2	15,0	21,2	55,1	1,9
	3	27,8	20,5	57,3	3,9
ST-30	1	-	-	-	-
	2	-	-	-	-
	3	14,9	30,5	64,6	3,1

A2. MEASURED LOADS AND TEMPERATURES

