Università degli Studi di Napoli Federico II
Scuola Politecnica e delle Scienze di Base

Dipartimento di Strutture per l'Ingegneria e l'Architettura

Corso di Laurea Magistrale in
Ingegneria Strutturale e Geotecnica

Tesi di Laurea in
Tunnels and Underground Structures

# Uso di colonne miste calce-cemento per ridurre gli spostamenti indotti da uno scavo profondo - un caso di studio

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Anno Accademico 2023 - 2024

***December 17, 2024***

*No matter how your heart is grieving, if you keep on believing, the dreams that you wish will come true.*

- *Walt Disney Company*

# CONTENTS

# 1 Introduction

## 1.1 Context and importance of the study

The continuous development of urban areas often requires underground constructions close to existing structures and in challenging ground conditions. In Norway, deep excavations in soft marine clays are usually supported by sheet pile walls (SPW) and tieback anchors or internal struts, often in combination with ground improvement with lime-cement columns to safeguard the excavation pit and its surroundings until sufficient support is provided by the permanent structure. This MSc study focus on the soil-structure interaction between the support structure (cantilever SPW) and the adjacent building during the different phases in the construction process. This thesis derives from the observations and numerical analyses conducted during my internship at the Norwegian Geotechnical Institute (NGI).

## 1.2 Thesis objectives

During my internship at the Norwegian Geotechnical Institute (NGI), I observed the monitoring of a deep excavation in an urban centre and integrated these observations with numerical analyses. The monitoring was carried out during the various phases of the excavation, allowing for the collection of detailed and relevant data at each stage. The objective of my thesis study is to verify the possibility of reducing the lime content in the treated soil, thereby decreasing $CO_2$ emissions and reducing environmental impact, without increasing the risk of settlement in the adjacent buildings.

## 1.2 Structure of the thesis

The thesis is organized into six main chapters.

Chapter 1 provides a general introduction to the context and importance of the study, outlining the thesis objectives and its structure.

Chapter 2 presents a review of the existing literature, covering topics such as deep excavation, cantilever walls, the effects on adjacent buildings, and previous studies on the use of lime cement.

Chapter 3 describes the methodology used, including the measurement and analysis tools and techniques.

Chapter 4 is dedicated to the case study at Campus Ullevål, describing the project, investigations and characterization, the geotechnical model, displacement monitoring, numerical analysis, and the results and discussion.

Chapter 5 summarizes the main findings, practical implications, and suggestions for future research.

Finally, Chapter 6 contains the references.

# 2 Literature

## 2.1 Deep Excavations and Sheet Pile Walls

Deep excavations involve the removal of soil or rock to create significant underground spaces, such as basements, underground parking lots, tunnels, and other structures. These projects require careful planning and execution due to the complexities and risks associated with working below ground level.

Retaining structures ensure the stability of potentially unstable soil fronts and are divided into externally and internally stabilizing systems. Among the various types of external retaining structures, we find:

- Gravity walls
- Reinforced concrete cantilever walls
- Sheet piles, free or anchored
- Diaphragm walls or bored pile walls, free or anchored

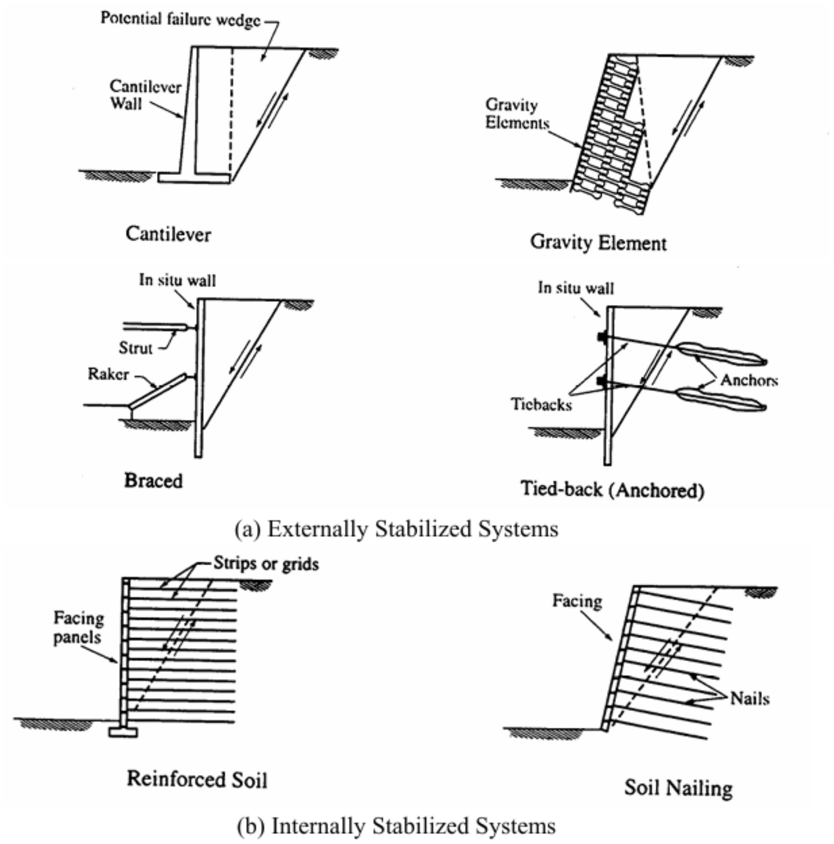



Fig2.1 - Variety of retaining walls (after O'Rourke and Jones, 1990)

In this section, we introduce Figure 2.2, which illustrate the sheet pile walls system.

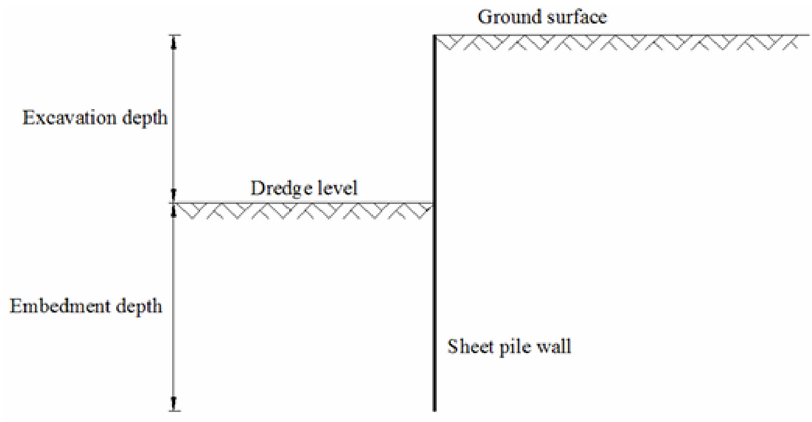



Fig2.2 – sheet pile walls system

Cantilever retaining walls, such as sheet piles, are flexible and often used in maritime construction or temporary excavation support. While commonly made from steel, they can also be manufactured from materials like wood and reinforced concrete, depending on the project's requirements. The stability of free-standing sheet piles relies on adequate embedding in the soil. Anchored sheet piles often utilize tie-back anchors or internal struts for additional support.

Before listing the specific phases of the project, it is important to understand the context and significance of each phase. This overview allows us to follow a structured approach and appreciate the importance of each phase in achieving the project’s goals for deep excavations and sheet pile walls.

*Phases of Project for Deep Excavations and Sheet Pile Walls*

1. Geotechnical Investigation:
   - Objective: Determine the mechanical characteristics of the soil, identify groundwater conditions, and define the geometry of stratigraphic contacts.
   - Activities: Perform in-situ and laboratory tests, analyze results to obtain geotechnical parameters necessary for the project.
2. Definition of Excavation Dimensions:
   - Objective: Establish the dimensions and depth of the excavation area required for the project.

- Activities: Create excavation plans and sections, determine excavation volumes and temporary storage areas for removed soil.

3. Survey and Verification of Conditions of Adjacent Structures:
    - Objective: Assess the condition and stability of nearby structures that may be affected by the excavation.
    - Activities: Inspect nearby buildings and infrastructure, record existing conditions, evaluate potential risks.
4. Definition of Permissible Movements for Adjacent Structures:
    - Objective: Establish acceptable levels of movement for structures near the excavation site.
    - Activities: Calculate predictable movements, compare them with acceptability criteria to avoid damage to adjacent structures.
5. Selection of Construction Schemes:
    - Objective: Choose the most suitable construction methods for the site conditions and project requirements.
    - Activities: Evaluate different construction techniques, select materials and equipment to be used, plan the sequence of excavation and construction operations.
6. Evaluation of Soil Movements and Groundwater Level Lowering:
    - Objective: Predict and monitor soil movements and changes in groundwater level during and after excavation.
    - Activities: Use geotechnical models to simulate site conditions, install monitoring instruments to measure actual movements.
7. Comparison of Calculated Soil Movements with Permissible Limits:
    - Objective: Verify that actual movements remain within established safety and stability limits.
    - Activities: Compare monitoring data with predictions, make project modifications if necessary to maintain safety.

8. Setup of Control and Monitoring Instrumentation During Construction:
    - Objective: Ensure continuous control of site conditions and stability of retaining structures throughout the construction phase.
    - Activities: Install and calibrate monitoring instruments, collect and analyze real-time data, make operational adjustments based on results.

*Factors Influencing Soil Pressure on Retaining Structures*

- Soil Conditions and Groundwater Regime: Properties of the soil, such as cohesion, internal friction angle, and presence of groundwater, significantly influence the pressure exerted on the retaining wall.
- Construction Methods: The sequence of excavation operations, the rate of progress, and support techniques influence pressure distribution on the retaining wall.
- Versatility of Construction Elements: The choice and effectiveness of construction elements, such as anchors, tiebacks, and struts, determine the wall's ability to resist soil pressures.
- Construction Timeframe: The duration of open and unsupported excavation sections affects soil movements and pressures exerted on retaining structures. Longer durations result in greater soil movements.

*Construction Techniques*

1. Installation of Sheet Piles: Sheet piles are driven into the ground using hydraulic or vibratory hammers. This process creates a tight interlocking system that provides a robust barrier against soil and water. The depth of installation depends on soil conditions and the required excavation depth.
2. Lime Cement Columns: In areas with loose and soft, compressible soils, lime cement columns can be used to stabilize the ground. This involves mixing lime and cement with the existing soil to improve its strength and reduce settlement. This method is particularly useful in soft soil conditions and helps ensure the stability of the excavation.
3. Excavation Process: Excavation is carried out in layers, starting with a shallow cut and progressing to deeper levels. Each layer of soil is carefully removed to maintain site

stability. Temporary structures, such as braces or struts, are often installed to support excavation walls during this process.

4. Concrete Platforms and Struts: To provide a stable working surface within the excavation, concrete platforms are constructed. Struts are installed to offer additional support to the excavation walls, ensuring they remain stable throughout the construction process. These platforms and struts also facilitate the movement of materials and equipment within the excavation site.

5. Removal of Soil and Debris: Once the excavation is secured with struts and other support systems, the remaining soil and debris are removed. This clears the way for the construction of the permanent structure. Efficient debris removal is crucial to maintaining a clean and safe working environment.

## 2.2 Effects of Deep Excavations on Adjacent Buildings

Deep excavations can have significant effects on adjacent buildings and structures. These effects need to be carefully considered and managed to prevent damage and ensure the safety of the surrounding environment. Here are some of the primary effects:

*1. Ground Settlement and Deformation*

Deep excavations can cause ground settlement and deformation, including both downward movement of the ground surface and lateral and vertical movements. These phenomena occur due to the removal of soil, which changes the stress distribution in the ground and reduces the support provided to adjacent buildings. Ground settlement and deformation can lead to cracks in the walls, floors, and foundations of nearby structures, as well as exert additional pressure on foundations, causing tilting or uneven settlement.

*2. Vibration*

Construction activities such as pile driving, drilling, and excavation can generate vibrations. These vibrations can travel through the ground and affect adjacent buildings. High levels of vibration can cause structural damage, including cracking of walls and dislodgement of building elements.

*3. Changes in Groundwater Levels*

Deep excavations can alter the local groundwater levels. Lowering the water table can lead to soil drying and consolidation, which can cause settlement. Conversely, if water accumulates in the excavation site, it can create hydrostatic pressure, affecting the stability of adjacent structures.

*4. Structural Damage*

The cumulative effects of ground settlement, vibration, and ground movement can lead to structural damage in adjacent buildings. This damage can manifest as cracks in walls, floors, and foundations, as well as misalignment of doors and windows.

*Mitigation Measures*

To mitigate these effects, several measures can be taken:

1. Monitoring: Implementing a comprehensive monitoring system to track ground movements, vibration levels, and groundwater changes. This allows for early detection of potential issues and timely intervention.
2. Stabilization Techniques: Using techniques such as underpinning, grouting, and soil nailing to stabilize the ground and adjacent structures.
3. Controlled Excavation: Adopting a controlled excavation process, including staged excavation and the use of temporary support systems to minimize ground movement.
4. Vibration Control: Employing construction methods that reduce vibration levels, such as using hydraulic hammers instead of traditional pile drivers.
5. Water Management: Implementing dewatering systems and barriers to control groundwater levels and prevent water ingress into the excavation site

One of the most interesting results that can be obtained from monitoring displacements is the deformed shape behind the excavation. This can be achieved through topographical measurements, extensometers, or settlement gauges. Horizontal ground movements can also be measured using inclinometers.

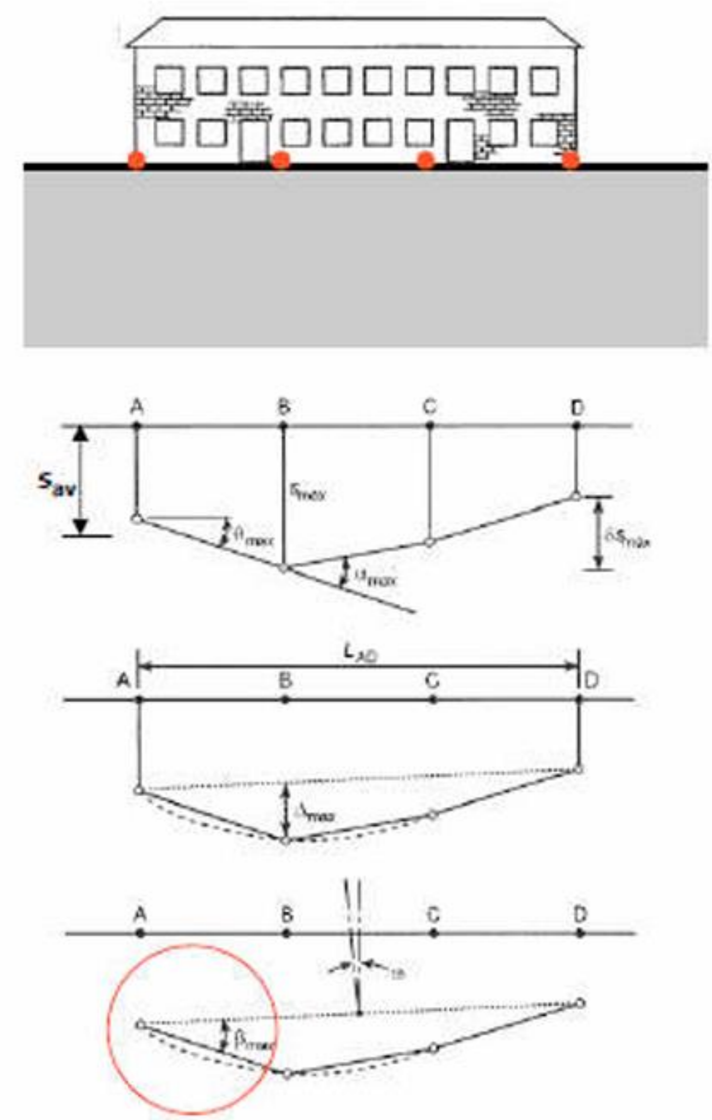



Fig2.3- angular distortion β

To define the permissible settlement values for surface structures, it is important to note that uniform settlement in itself is not capable of causing damage to the structure and, therefore, cannot be considered alone as an indicator of potential damage to a structure.

Skempton and MacDonald (1956), based on collected experimental evidence, suggested that the value of angular distortion β could be taken as an index to evaluate the potential damage to a structure. They observed that for $\beta < 1/300$, no significant cracking had occurred in load-bearing or infill masonry walls in any of the cases they collected; for damage to reinforced concrete load-bearing structures, however, β needed to reach the value of 1/150.

## 2.3 Previous studies and case studies

There have been several studies and case studies focusing on ground movements and the effects of excavation on adjacent structures. Here are a few notable ones:

a) Performance Monitoring Of Deep Supported Excavations In Soft Clay

The paper by Karlsrud et al. (1986) discusses the impact of instrumentation and performance monitoring on design practices for supported excavations in soft clay, citing examples from projects such as the Chicago subway and building foundations in Oslo conducted by NGI. They emphasize the importance of understanding when and why to instrument, selecting useful measurements, and efficiently organizing instrumentation projects for deep supported excavations.

b) Observed Performance Of A Deep Excavation In Clay

The researchers by Finno et al. (1989) measured surface and subsurface ground movements, pore water pressures, sheet-pile deformations, and strut loads, correlating these observations with construction activities. They found that over excavation led to larger-than-expected ground surface settlements, and the factor of safety against basal heave was critically low at certain construction stages.

c) Performance Of A Stiff Support System In Soft Clay

The paper by Finno et al.(2002) discusses the performance evaluation of an excavation support system during the renovation of a subway station in Chicago and its impact on an adjacent shallow-foundation supported building. They implemented a comprehensive monitoring program to assess field performance data and observed minor damage to the building as predicted in the design.

d) Observed Performance Of A Sheet pile-Supported Excavation In Chicago Clays

The study by Finno et al. (2018) investigates the behavior of a braced excavation in saturated clays in Chicago, focusing on factors such as ground movements, pore water pressures, sheet-pile deformations, and strut loads. To achieve this, the researchers established a test section adjacent to the excavation site and conducted extensive monitoring before, during, and after construction activities.

e) Assessment Of Building Damage Due To Excavation-Induced Displacements: The Gibv Method

The paper by Piciullo et al.(2021) describes a new methodology for the rapid assessment of potential building damage caused by excavation, combining ground impact with building vulnerability. This approach has been implemented in a GIS tool, demonstrating good accuracy in predicting damages and offering the ability to rapidly assess a large number of buildings for infrastructure projects in urban settings.

f) Calculation Of Building Settlement Induced By Tunneling Based On An Equivalent Beam Model

The paper by Zhu (2022) presents a study on the response of low- and medium-rise framed buildings with continuous foundations to tunnelling activities. It compares simplified beam models with advanced 3D finite element analyses, discussing the influence of infill panels and proposing analytical solutions and a meta-model for predicting structural responses and assessing risks.

*Observation*

To limit displacements, very rigid retaining structures are used, further stiffened by support systems; the construction time is reduced, and specialized labor is employed. In this perspective of minimizing the interference of the excavation with adjacent structures, construction aspects previously overlooked are becoming increasingly important. In particular, research is focusing on displacements induced by the construction of the retaining structure itself and by soil consolidation treatments, as well as on the study of three-dimensional effects such as the stiffening obtained near the corners of the excavations.

## 2.4 Lime Cement

*Introduction*

Lime cement piles are used to improve the bearing capacity and stability of soft and sensitive clay to depths of up to 25-30 meters. Lime and cement chemically react with the soil to form a soil material with greater shear strength and stiffness than the original soil. The "piles" are installed by a crawler rig with drilling equipment. The rig is equipped with a mixing tool, a rotating whisk that is drilled down to the desired depth for the bottom of the pile before the lime/cement mixing begins.

The lime-cement is blown into the soil through a nozzle using compressed air while the whisk rotates and is pulled up. This process creates columns of firmer mass in the soil where the lime/cement is mixed in.

Deep stabilization with lime piles has been used since the 1970s and is widely used in Sweden. Over time, cement and a mix of lime and cement have also been widely adopted as binders in deep stabilization. Larsson, S. (2021). The Nordic dry deep mixing method: Best practice and lessons learned. This reference should help readers locate the original source for more detailed information.

The Norwegian Geotechnical Society (NGF) has developed a "Guideline for ground reinforcement with lime cement piles." This guideline is primarily referenced, but some principles and special procedures are described in this chapter.

Dry Deep Mixing (DDM) is a widely accepted method for improving the strength and stiffness properties of soft soils. It involves mixing soil with binders like lime and cement using a rotating mixing tool.

The primary applications of lime cement piles include:

- Increasing bearing capacity under embankments and construction roads.
- Improving stability in cuts, excavations, and natural slopes.
- Reducing settlement under embankments and equalizing settlement under approach fills to bridges.
- Reinforcing foundations in trench lines.

The paper by Larsson reviews the development of Nordic DDM over the past 20 years, focusing on research into improving very soft glacial and post-glacial clays. It examines various theses, reports, and research articles on the topic.

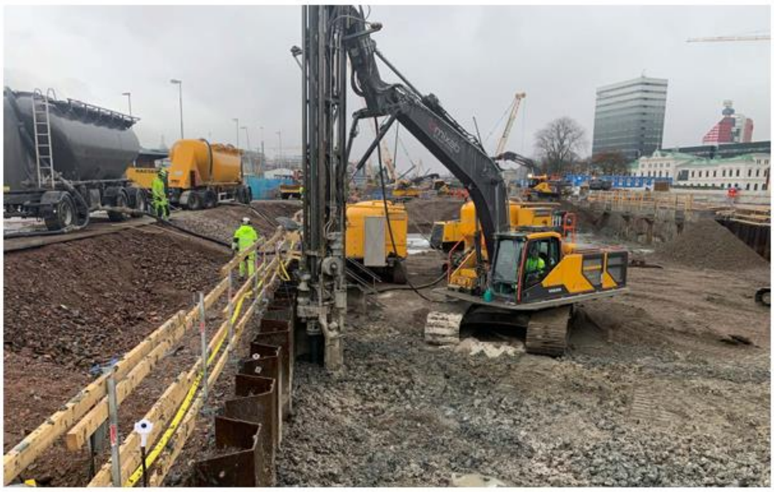


Fig2.5 - Example of machine used in the Nordic countries - Larsonn, S.(2021)

*Ground Improvement of Deep Excavations*

Dry deep mixing (DDM) offers various ground improvement systems that can be implemented depending on the application, such as singular columns, panels (or rows), grids, and blocks. These applications include reducing settlement, increasing stability and bearing capacity, protecting structures close to excavation sites, reducing active stresses on retaining walls, increasing passive load in excavations within retaining walls, and providing foundations for structures like houses, bridges, and wind turbines.

The piles can be installed as single piles, in single or double ribs, and in blocks. During the production of ribs, grids, and blocks, the piles are installed with an overlap of about 10-15 cm (approximately 20%). The piles must be installed consecutively so that the new pile has good contact with the previous pile and to avoid the adjacent piles hardening too much, which would make overlapping impossible. In accordance with norwegian guidelines, the following overlap is recommended depending on the pile diameter. (Handbook V221 – paragraph 1.7).

The methods more used to check the shear strength and homogeneity in lime cement piles are

- FKPS (Column penetration test). Using a cylindrical penetrometer with horizontal vanes, the test measures penetration resistance.
- FOPS (Reverse column penetration test). Involves pulling a pre-installed probe from the base to the surface of a lime-cement column to measure total pull-out force and assess structural integrity.

There have been several studies and case studies focusing on the interpretation of the strength and stiffness of lime-cement columns. A relevant study is "Trans-scale spatial variability of lime-cement mixed columns" by Wong et al. (2024), which examines the spatial variability of lime-cement columns. Another important study is "Strength and Verticality of Nordic Dry Deep Mixing Columns—A Case Study in Norway" by Hov et al. (2024), which presents a case study in Norway on the use of the Nordic dry deep mixing method.

*Sounding*

FOPS is a method for measuring drained strength in lime cement piles. FOPS is installed in conjunction with the establishment of the lime cement pile. The equipment consists of a wing attached to the end of a wire. The wing is mounted in front of the auger (see Figure 2.6) and lowered to about 0.5 meters below the bottom of the pile, while the wire runs through the pile all the way up to the surface. The pile is controlled by pulling the probe through the pile using the wire while simultaneously measuring the pull resistance. The FOPS should be pulled after an average minimum curing time of 3 days, as the pile will become too hard to pull through after this period. Some cases may require more days. The rig used must be capable of registering the pulling force.

Additionally, KPS sounding is used, where the probe is similar to the FOPS probe, but here the probe is pressed down into a pre-drilled hole in the pile. Registration occurs in a similar manner as with FOPS.

Further, pressure sounding (CPT and CPTU) is also used for pile control. Sounding can be done either vertically or at an angle in ribs for continuity control. Pressure sounding provides an indication of the homogeneity and shear strength in the piles. The pressure sounding curve is similar to FOPS and KPS curves and can be compared for evaluating the piles. The advantage of KPS and CPT/CPTU is that piles to be controlled can be chosen after installation.

Fig2.6 – FOPS FKPS CPT and KPS probe

# 3. Methodology

## 3.1 Measurement tools and techniques

### 3.1.1 Introduction

Geotechnical and structural instrumentation encompasses a range of essential equipment designed to understand the behaviour of soil and structures, as well as their interactions with each other and the surrounding environment. To address technical questions, it is crucial to compare expected values derived from models with actual values obtained from measurements. This process necessitates the development of a monitoring system.

Geotechnical and structural monitoring involves performing repeated measurements over time using appropriate instruments. Monitoring supports structural and geotechnical engineering throughout all phases of a project: from design to construction, and through to the management of structures.

Planning an effective monitoring system requires a systematic approach, which includes analysing all phases from objectives to implementation and interpretation of measurements.

Using instruments that provide complementary measurements is a good practice. This is particularly important when monitoring the effects on existing structures near excavations, such as in the case of retaining walls. In such conditions, it will be necessary to develop a predictive model of the vertical and horizontal displacements of the structures, as well as the deformations of the retaining wall itself.

*Planning a Monitoring System*

Objectives of Instrumentation is verify the quality of the performance of the structure at the end of the construction phase. Selection of Instrumentation, during the design phase, expected values must be determined, and based on these data, choose the instrumentation based on the following parameters:

- Operational Conditions: The conditions under which the instrumentation will operate can significantly influence the quality of the measurements and results.
- Reliability: The chosen instrumentation must be reliable and capable of providing accurate and consistent measurements over time.

*Measurement Parameters*

Understanding the behaviour of soil or a structure is not based on a single parameter but on multiple parameters. In complex situations, it is preferable to monitor several parameters to identify significant correlations.

Using instruments such as inclinometers near the retaining wall, along with topographical and geotechnical monitoring of adjacent structures, will allow verification of the correspondence between design predictions and the actual situation during the work.

### 3.1.2 Total Station

*Description*

A total station is an advanced measurement instrument used in surveying to precisely determine three-dimensional coordinates, including position (latitude and longitude), height, and distance. It combines an electronic theodolite (which measures horizontal and vertical angles) with an electronic distance meter (which measures the distance to the observed point). This instrument is essential for many applications in civil engineering, geotechnics, architecture, and construction.

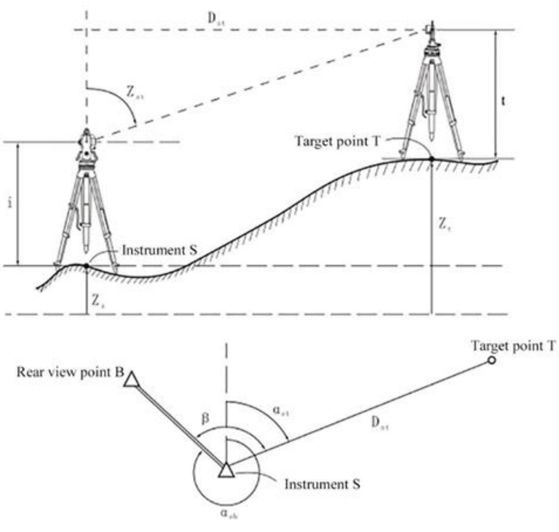



Fig3.1 – schematic of total station measurement technique

*Operation*

- Positioning: The total station is placed on a stable tripod at a known reference point.
- Angle Measurement: Using the theodolite, the operator measures the horizontal and vertical angles relative to a reference point.

- Distance Measurement: The electronic distance meter measures the distance between the total station and the point of interest using a laser or infrared beam.
- Coordinate Calculation: The angular and distance measurements are combined to calculate the exact coordinates of the point of interest. This data is automatically recorded and stored in the total station system.

*Measurement Techniques*

a) Alignment and Calibration: Before measurements begin, the total station is aligned and calibrated to ensure precision. This includes verifying the instrument height and calibrating the distance meter.
b) Angle Measurement: The operator aims at the point of interest using the optical sight and records the horizontal and vertical angles.
c) Distance Measurement: After aligning the sight, the distance meter measures the distance between the total station and the observed point.
d) Data Collection: The coordinates of the point are calculated and stored. This process is repeated for each point of interest, allowing the creation of a three-dimensional model of the study area.
e) Data Transfer: The collected data is downloaded to a computer or mobile device for further analysis and interpretation.

*Applications of a Total Station*

Total stations are extremely versatile and advanced instruments used in a wide range of applications in civil engineering, surveying, construction, and structural monitoring. Here are some of the main applications:

a) Topographical Surveys
    - Description: Used to create detailed maps of the terrain, including contours, boundaries, and topographic features.
    - Operation: The total station measures the angles and distances relative to reference points, providing precise three-dimensional coordinates.
b) Deformation Monitoring
    - Description: Monitoring the movements and deformations of structures such as buildings, bridges, dams, and retaining walls.
    - Operation: Total stations are used to detect minimal displacements over time, helping to prevent structural collapse and ensure safety.

c) Construction and Alignment
   - Description: Ensuring that structural elements are positioned accurately during construction.
   - Operation: Total stations are used to trace guidelines and reference points for the precise positioning of structures like columns, walls, and foundations.

d) Civil and Geotechnical Engineering
   - Description: Used for excavation, filling, and leveling projects.
   - Operation: Total stations provide precise data on elevation and position changes, helping to manage and control earth-moving operations.

e) Infrastructure Installation
   - Description: Essential for the installation of infrastructures such as roads, railways, and utility networks.
   - Operation: Total stations help trace routes and position infrastructure components with high precision, reducing errors and improving efficiency.

f) Structural Verification and Testing
   - Description: Used to verify and test existing structures, ensuring they meet design specifications.
   - Operation: Total stations accurately measure the dimensions and positions of structures, comparing actual data with design plans.

g) Soil Movement Monitoring
   - Description: Essential for monitoring soil movements in areas subject to landslides, subsidence, or other geological instabilities.
   - Operation: Total stations record ground displacements, providing critical data to assess risks and plan corrective measures.

### 3.1.3 Inclinometer

*Description*

An inclinometer is a geotechnical instrument used to measure the inclination or lateral movement of soil and structures. It is frequently employed to monitor the stability of slopes, dams, retaining walls, buildings, and other civil engineering structures. The inclinometer helps detect subtle shifts and progressive movements that might indicate potential failure or instability.

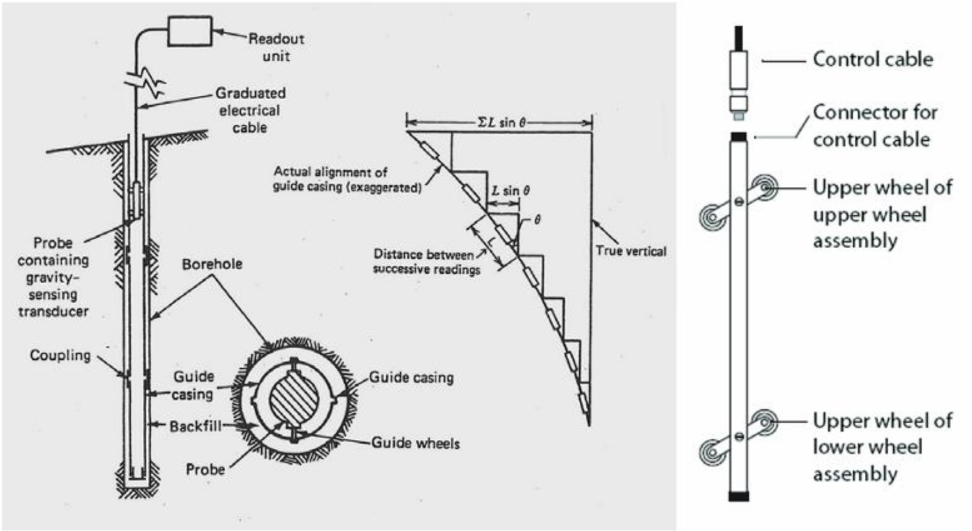



Fig3.2 – Schematic of inclinometer to monitor ground deformations (Mikkelsen)

*Operation*

The inclinometer typically consists of a tilt-sensitive probe that is inserted into a pre-installed inclinometer casing in the ground or a structure. The probe is lowered along the casing and measures the angle of inclination at various depths. This data is then used to calculate horizontal displacements and deformation of the monitored soil or structures.

*Measurement Techniques*

a) Installation of Inclinometer Casing: An inclinometer casing is inserted into a vertical or inclined borehole. The casing must be securely anchored and properly aligned to ensure precise measurements.

b) Initial Measurement: An initial reading is taken immediately after the installation of the casing to establish a baseline. This reading represents the "zero" state against which all subsequent readings will be compared.

c) Measurement Probes: The inclinometer probe, equipped with MEMS (Micro-Electro-Mechanical Systems) sensors or accelerometers, is lowered along the casing. The probe measures the angle of inclination at regular intervals (typically every 0.5 or 1 meter).

d) Periodic Monitoring: Measurements are repeated periodically (e.g., weekly, monthly) to monitor any changes in the inclination profile over time. This helps detect progressive movements and assess the stability of the soil or structure.

e) Data Analysis: The collected data is analyzed to determine horizontal and vertical displacements. This process includes converting the inclination angles into linear displacements and creating deformation profiles.

f) Interpretation of Results: The measurement results are interpreted to evaluate the movements of the monitored soil. Engineers use this data to make informed decisions regarding corrective actions or further studies.

*Applications of an Inclinometer*

a) Slope Monitoring

- Description: Inclinometers are used to monitor the stability of natural slopes and artificial embankments, detecting any movements that might indicate imminent landslides.
- Operation: Installed along the slope, inclinometers measure the inclinations and movements of the soil, providing data to assess stability and plan mitigation measures.

b) Retaining Wall Monitoring

- Description: Used to monitor the deformations of retaining walls during and after construction. This is essential to ensure that the wall remains stable and does not exhibit excessive movements.
- Operation: Inclinometers are inserted into casings along the retaining wall and detect inclinations that indicate lateral movements of the structure.

c) Dam and Levee Monitoring

- Description: Essential for monitoring the stability of dams and levees, especially in areas subject to variable water loads and extreme weather conditions.
- Operation: Inclinometers measure the inclinations and movements of the structure, helping to detect deformations that could compromise the integrity of the levee or dam.

d) Building and Structure Monitoring

- Description: Used to monitor the displacements and inclinations of buildings, especially in seismic risk areas or near deep excavation sites.

- Operation: Inclinometers are installed in the foundations or walls of buildings to detect movements that might indicate structural settlements.

e) Tunnel Movement Monitoring

- Description: Used to monitor soil movements around tunnels during and after construction.
- Operation: Installed along the perimeter of the tunnel, inclinometers measure soil deformations and displacements, helping to prevent collapses and damage to the structure.

f) Foundation Monitoring

- Description: Essential for monitoring the movements of foundations, especially in unstable soils or during the construction of tall buildings.
- Operation: Inclinometers are installed in the foundations to detect displacements and deformations, ensuring that the structure is built on stable bases.

### 3.1.4 Piezometer

*Description*

A piezometer is an instrument used to measure the water pressure in the pores of soil or other geotechnical structures. It is employed to monitor hydraulic pressure variations and to assess soil stability in situations such as slopes, dams, and foundations. It is essential for understanding the hydrogeological behaviour of soils, especially in geotechnical and civil engineering contexts.

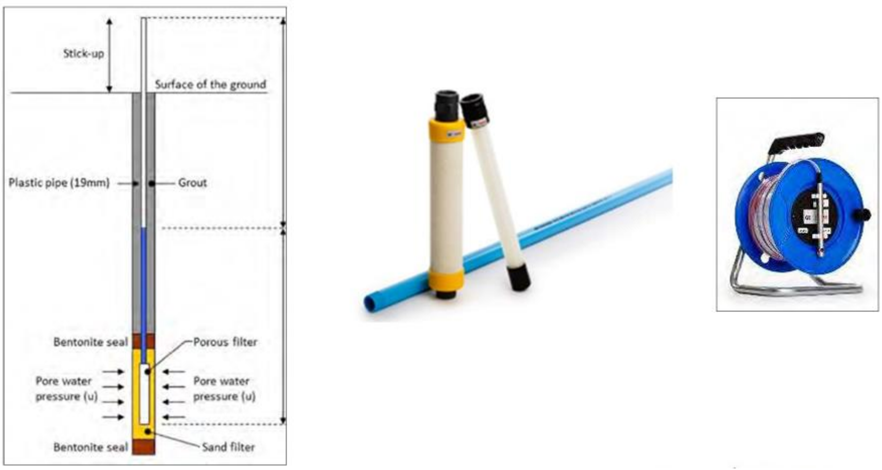



Fig3.3- Schematic of piezometer (Geosense)

*Operation*

- Installation: The piezometer is installed inside a borehole drilled into the ground. Its tip is positioned at the desired depth where pore pressure measurement is intended.
- Pressure Measurement: The piezometer measures the water pressure in the soil pores. This pressure can be correlated to the groundwater level (aquifer) and variations induced by loads, excavations, or other activities.
- Data Recording: Piezometers can be manual, where readings are taken periodically with specific manometers, or automatic, where data is continuously recorded and transmitted to a data acquisition system.

*Measurement Techniques*

1. Open Tube Piezometer: Consists of a tube with a filter at the base that allows water to enter. The water level inside the tube reflects the pore water pressure at the depth of the filter. Measurements are taken manually.
    - Advantages: Simple and low cost.
    - Disadvantages: Requires manual readings, can be influenced by turbulence and fluctuations.
2. Diaphragm Piezometer: Utilizes a diaphragm that responds to the water pressure in the pores. The deformation of the diaphragm is correlated to hydraulic pressure and can be measured precisely.
    - Advantages: Higher precision, possibility of automatic recording.
    - Disadvantages: More expensive and complex to install.
3. Vibrating Wire Piezometer: Based on a vibrating wire that changes frequency in response to pressure variations. This frequency variation is correlated to pore pressure.
    - Advantages: High precision and reliability, ideal for continuous and automatic measurements.
    - Disadvantages: High cost and requires specialized maintenance.

4. Pneumatic Piezometer: Uses a pneumatic system to measure water pressure. The air pressure needed to balance the water pressure in the soil is measured and correlated to pore pressure.
    - Advantages: Reliable and suitable for difficult conditions.
    - Disadvantages: Requires special equipment for measurements.
5. Geotech PVT: Direct push piezometers. These devices operate by inserting a laser-trimmed ceramic sensor into the ground through direct push technology. The sensor detects the pore water pressure and, due to its stability over time, provides accurate and reliable data.
    - Advantages: Offer highly stable and precise measurements over time, making them ideal for long-term projects due to advanced sensors and integrated data loggers.
    - Disadvantages: They are generally more expensive and require specific technical expertise for installation and maintenance compared to traditional piezometers.

*Applications*

1. Monitoring Dams and Levees
    - Description: Used to monitor water pressure inside dams and levees, preventing the risk of breaches or excessive infiltration.
    - Operation: Installed at various key points to detect abnormal pressures that might indicate structural issues.
2. Slope Stability Studies
    - Description: Essential for assessing the stability of slopes, especially in landslide-prone areas.
    - Operation: Measure pore pressure in the soil, providing data to model and predict soil movements.

3. Foundation Monitoring
   - Description: Used to measure pore pressure under the foundations of buildings and other structures, ensuring that soil conditions are stable and safe.
   - Operation: Provide data that helps prevent foundation settlements due to hydraulic pressure variations.
4. Hydraulic Barrier Control
   - Description: Employed to monitor the effectiveness of hydraulic barriers used to control groundwater contamination.
   - Operation: Measure water pressure upstream and downstream of the barrier to evaluate the system's effectiveness.
5. Deep Excavation Projects
   - Description: Used to monitor pore pressure around deep excavations, reducing the risk of soil settlements or collapses.
   - Operation: Installed at strategic points around the excavation area to detect pressure variations that might indicate instability.

## 3.2 Analysis tools and techniques

### 3.2.1 Introduction

Design and prediction are fundamental aspects of civil engineering projects, especially when it comes to deep supported excavations. These initial phases not only determine the feasibility and safety of the project but also influence the choice of construction methods and risk mitigation techniques. Designing a deep excavation involves detailed analysis of soil behavior under load and predicting its reaction during different construction phases. This prediction requires a deep understanding of the geotechnical properties of the soil, hydrogeological conditions, and the stresses it will be subjected to.

To achieve this, engineers use a combination of traditional methods and advanced tools. Modern analysis techniques, such as those offered by the Plaxis2D software, allow the creation of numerical models that simulate the behaviour of soil and structures under variable conditions. These advanced tools are essential for making accurate predictions and informed decisions throughout all stages of the project, from initial design to construction and long-term management of the works.

The adoption of analysis software like Plaxis2D offers numerous advantages, including the ability to model complex scenarios and evaluate the effectiveness of different engineering solutions.

This chapter will focus on the use of advanced analysis software like Plaxis2D for numerical modelling and simulation of structural and geotechnical behaviour.

### 3.2.2 Analysis Software – Plaxis2D

*Description.*

PLAXIS is an advanced finite element (FE) software developed by Bentley Systems, is used for the simulation and analysis of geotechnical problems. It is employed to model the behaviour of soils and structures under variable loads, considering the interaction between the soil and the structures.

*Operation*

The software uses advanced numerical models to predict the responses of soil and structures to various loading conditions. Input data can include soil characteristics, structural geometries, and loading conditions.

*Measurement Techniques*

- Model Calibration: Theoretical models are calibrated using experimental data collected during monitoring.
- Numerical Simulations: Conducting simulations that include analysis of deformations, stability, and dynamic response.

*Applications*

PLAXIS 2D is ideal for a range of applications, including:

- Excavations and Embankments: Analysis of stability and deformations during excavations and embankment constructions.
- Foundations: Design and analysis of shallow and deep foundations.
- Tunneling: Design and analysis of tunnels and galleries.

- Ground Improvement Projects: Analysis of ground improvement measures.

### 3.2.2.1 Hardening Soil Model

*Description*

The Isotropic Hardening Soil Model is designed to describe soil behavior under various loads. Unlike simple elastic models, this model considers material hardening, which means that soil strength increases with the application of additional loads. The model represents both compression hardening (when the soil is compressed) and shear hardening (when the soil is subjected to shear).

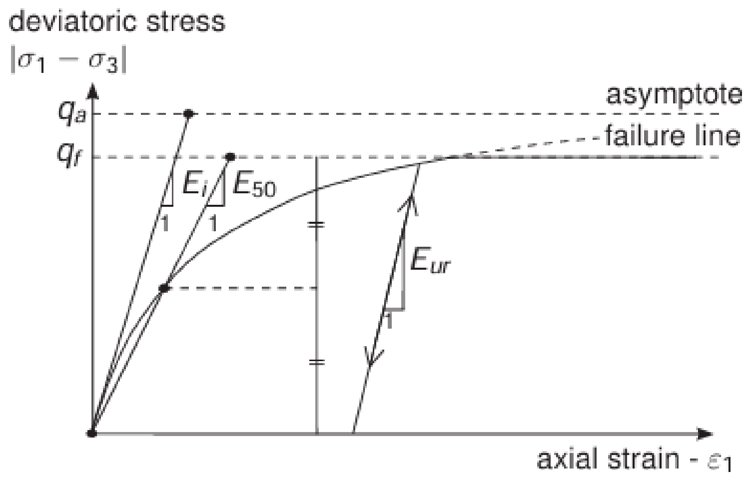



Fig3.4-Hyperbolic stress-strain relation (PLAXIS Material Models)

*Operation*

The functioning of the model is based on key concepts:

- Yield Surface: This surface represents the limit state beyond which soil begins to deform plastically. In the isotropic hardening model, the yield surface expands isotropically as plastic deformation increases, maintaining the same shape but increasing in size.
- Hardening Law: The model uses a hardening law that describes how soil strength increases with plastic deformation. This allows the model to predict soil behavior under progressive loads more accurately.
- Nonlinear Deformation Modulus: The model accounts for the nonlinear behavior of the soil, where the deformation modulus decreases as the level of applied stress increases.

*Soil Types*

The Hardening Soil Model (HS) is versatile and can be used to simulate the behaviour of various soil types. Here are the types of soil for which it is particularly suitable:

1. Clays: The Hardening Soil Model is ideal for soft and stiff clays. It takes into account the stress-dependent stiffness, making it perfect for assessing the behaviour of clays under variable loads.
2. Silts: Silts, which have characteristics intermediate between clays and sands, can be effectively modeled with the HS model.
3. Sands: The model is also suitable for sands, especially when it is important to consider the increase in stiffness with pressure.
4. Unsaturated Cohesive Soils: The model is useful for unsaturated cohesive soils, where it is necessary to consider isotropic hardening and the expansion of the yield surface.

In general, the Hardening Soil Model is used in situations where an accurate representation of soil stiffness and its variation with load and pressure is needed. This makes it suitable for a wide range of geotechnical applications, such as excavations, foundations, tunnel construction, and ground improvement projects.

*Key Parameters*

To use the isotropic hardening model in PLAXIS 2D, several key parameters must be defined:

- Stiffness Modulus $E_{50}$: Tangent elasticity modulus for drained shear conditions.
- Oedometer Stiffness Modulus $E_{oed}$: Elastic modulus during confined compression.
- Unloading/Reloading Stiffness Modulus $E_{ur}$: Elastic modulus during lateral relaxation.
- Hardening Coefficient (m): Parameter that describes the hardening of the material.

*Applications*

The isotropic hardening model is widely used in various geotechnical applications:

1. Foundation Design: Used to analyze and design shallow and deep foundations, considering the increase in soil strength with increased load.
2. Slope Stability: Essential for assessing the stability of natural and artificial slopes, especially in areas prone to landslides.

3. Excavations and Tunnels: Applied to analyze soil deformation during deep excavations and tunnel construction, predicting the behavior of the soil and support structures.

#### 3.2.2.2 NGI ADP Soil Model

*Description*

The NGI ADP Soil Model (Norwegian Geotechnical Institute - Advanced Deterministic Plasticity) is a constitutive model used in geotechnical engineering to simulate the behaviour of soils under various loading conditions. This model is particularly useful for analysing undrained soil conditions, such as those found in clays and silts, in both onshore and offshore applications.

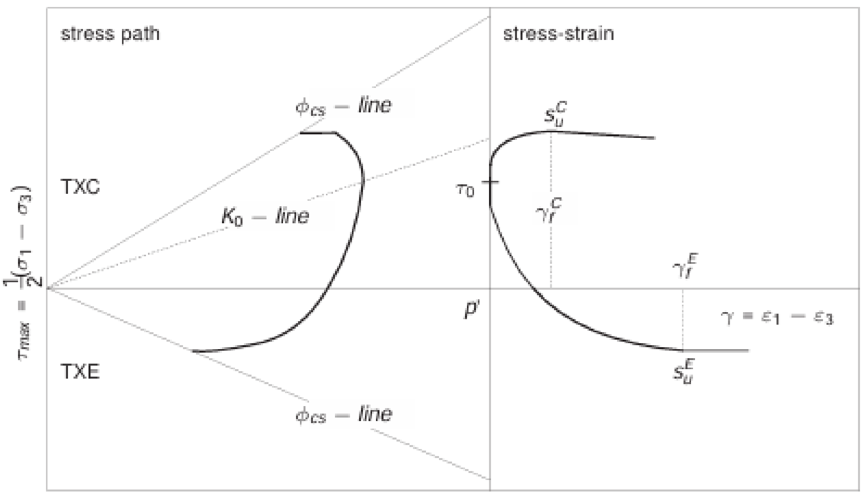



Fig3.5-Typical stress paths and stress strain curves for triaxal compression and triaxal extension

*Operation*

The NGI ADP model represents the plastic behaviour of soils, considering the stress history that influences soil strength and deformation. It uses critical state soil mechanics principles to accurately simulate soil behaviour under static and dynamic loads. The soil shear strength is defined by means of sus_u values for active, passive, and direct simple shear stress states.

*Soil Types*

The NGI ADP model is effective for:

1. Clays: This model is widely used for both soft and stiff clays. It takes into account the specific undrained shear strength and the stress history of these soils.

2. Silts: The model is also suitable for silts, which often exhibit properties intermediate between sands and clays. It is useful for both onshore and offshore projects.

3. Cohesive Soils: Cohesive soils with a high content of fine particles can be effectively modelled, as the NGI ADP model considers the anisotropic properties and the consolidation history of the soil.

*Key Parameters*

To use the NGI ADP model in software like PLAXIS, several key parameters must be defined:

- NGI ADP Model Parameters: These include specific parameters of the NGI ADP model, such as $S_U^P / S_U^A$ Ratio of (plane strain) passive shear strength over (plane strain) active shear strength, $S_U^D / S_U^A$ Ratio of direct simple shear strength over (plane strain) active shear strength and $S_{U,inc}^A$ Increase of shear strength with depth.
- Initial Conditions: The initial conditions of the model must be correctly set to reflect the initial state of the soil.

*Applications*

The NGI ADP model is used in many geotechnical applications, including:

- Slope Stability Analysis: Evaluating the stability of natural and artificial slopes, especially in landslide-prone areas. The model is calibrated for flat terrain, which provides a basis for calculating the stability of natural slopes.

- Tunnel Construction: Simulating soil behaviour during the construction of tunnels, predicting soil deformations and support structure behaviour.

- Foundation Design: Analysing and designing shallow and deep foundations, considering the increase in soil strength with increased load.

# 4 Case Study: Campus Ullevål

## 4.1 Project description

### 4.1.1 Introduction

As part of the construction project Campus Ullevål (CU), Norwegian Geotechnical Institute (NGI) have initiated the research project CURIOUS - Campus Ullevål: Research and Instrumentation Of Underground Structures to do research on problems related to geotechnical design of deep excavations in soft clay. The objective is to move the construction industry towards a more sustainable design of deep supported excavations in urban environments by implementing enabling technologies.

The research project has implemented a significant instrumentation program to monitor and assess the performance of the support structure and the surrounding assets during the excavation stages. The instrumentation also includes testing of advanced sensing techniques for comparison with more traditional monitoring.

Figure 4.1 shows the location of the case study site in Oslo, Norway. In the photos in Figure 4.1, we can see the old NGI building at the top, the excavation of the new building in the middle, and finally, the illustration below to the left belongs to Campus Ullevål AS. In Figure 4.2, we see a plan view of the building pit with sheet pile walls (SPW) and the location of the lime-cement stabilization. In Figure 4.3, we see a plan view of the monitoring (left) and a photo taken during the excavation (right).

The MSc study focuses on the soil-structure interaction between the support structure (cantilever SPW) and the adjacent building during the different phases in the construction process on the south part of the building pit.

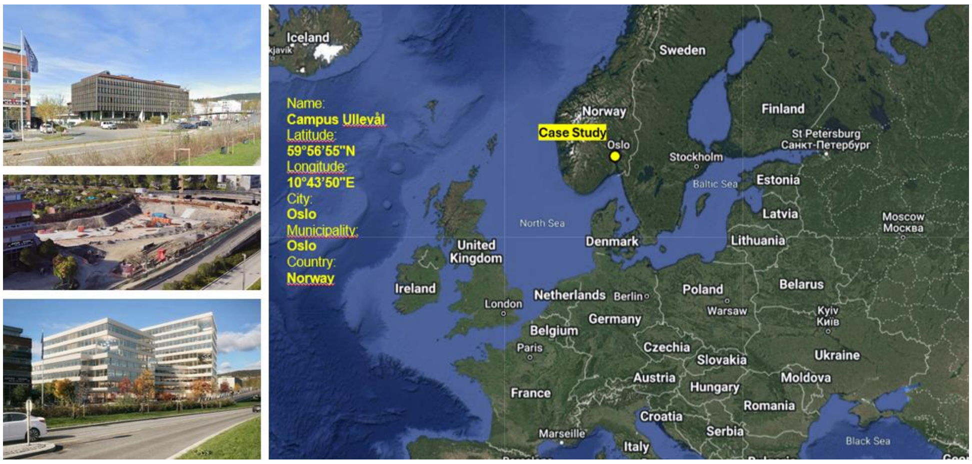



Fig4.1 – location of case study

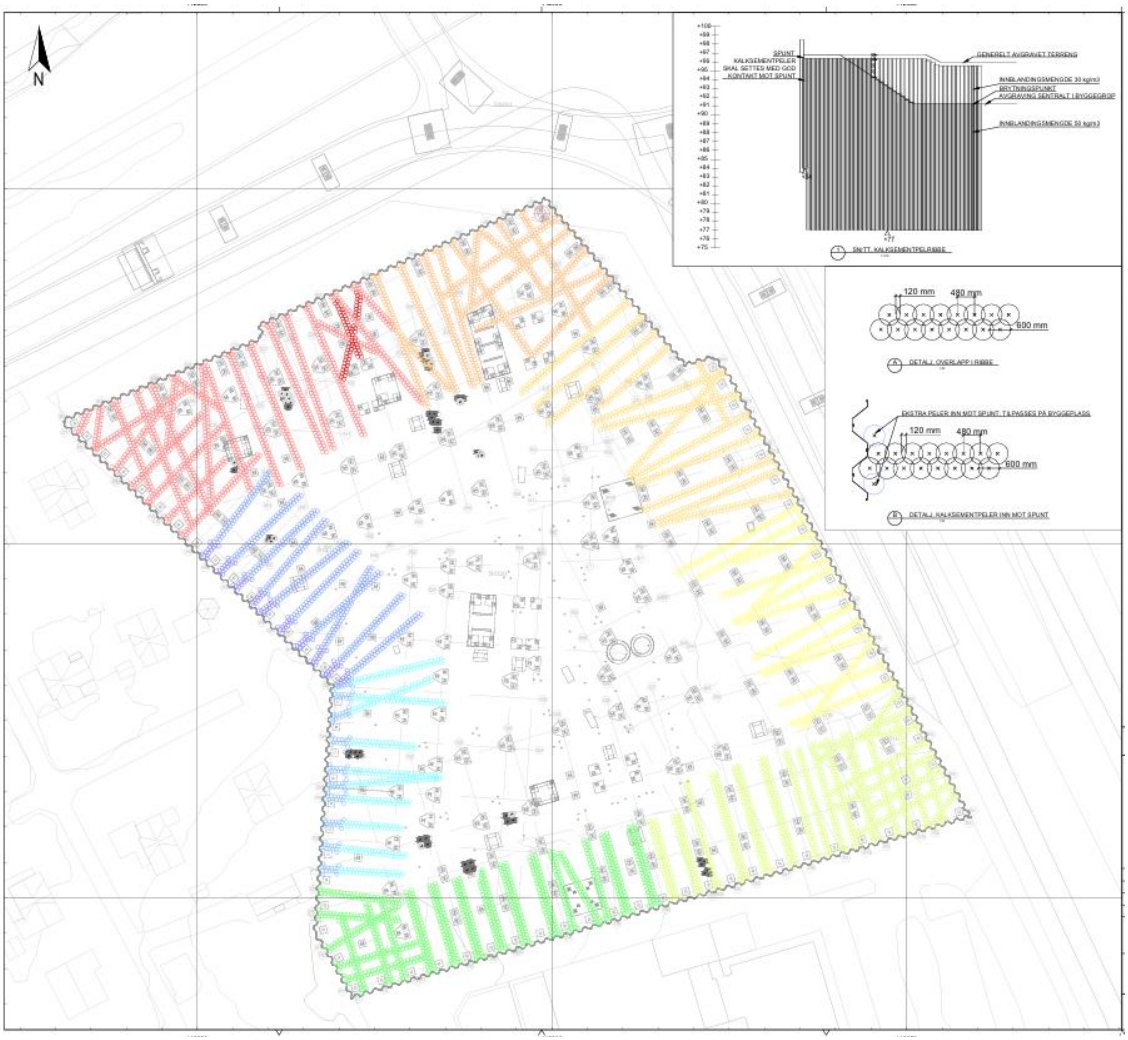


Fig4.2 - plan view of the building pit with the location of Sheet pile walls in black and in different colours the location of the lime-cement stabilization (NGI report)

Fig4.3 - plan monitoring focus on the south part of the building pit (left) picture building pit (right). In this picture they have started excavating the corner section of the berm and we can also see the building being studied in the front

### 4.1.2 Description of excavation works

The south part of the excavation for Campus Ullevål is close to an existing building which is partly founded on end bearing piles driven to bedrock and partly on shallow foundations.

Monitoring of the sheet pile wall (SPW) in terms of inclinometers and surveying points on the top of the SPW, as well as settlement measurements on the surrounding buildings have been carried out during the ground works.

The final excavation depth is situated at elevation +91.4 m, approximately 5 meters below the original terrain. The working steps for excavation can be summarized as follows:

a) **Sheet Pile Installation:** Sheet piles are driven down to elevation +83.5, with the top of the sheet pile placed 0.25-0.5 meters above the original ground level.
b) **Excavation:** Approximately 1-1.5 meters of excavation is carried out on the inside of the entire construction pit.
c) **Soil Cement Stabilization (LC-columns):** Loose soil inside the sheet piles is stabilized with soil cement in ribs against the sheet pile. Stabilization should extend down to elevation +77. The average coverage should be 30-40%, and within 12-20 meters from the sheet pile and into the construction pit, depending on the section/area. Good contact between KS and the sheet pile is assumed.
d) **Support on the North and East Sides:** Along the north and east sides, where the excavation depth is greatest and the calculated safety factor is lowest, support is provided outside the sheet pile. A strip approximately 0.5 meters wide is excavated about 5 meters outside the sheet pile pit.
e) **Complete Excavation of the Construction Pit:** The construction pit is excavated to elevation +91.4, except for a berm of earth embankment along the entire length of the sheet pile:
    1. The top of the earth embankment is approximately 1 meter below the original ground level and extends 4 meters horizontally from the sheet pile; at the shallowest excavation depths in the southwest, the embankment is limited to 3 meters at the top.
    2. Excavation slope is 1:1.5 down to elevation +91.4.
f) **Concrete Working Platform:** A concrete working platform is cast within the construction pit with sufficient thickness and quality to support applied loads.
g) **Placement of Walers:** Cushions are placed approximately 0.5 meters below ground level.

h) **Internal Struts:** Internal struts are placed with a maximum center-to-center spacing of 6 meters and braced towards the working platform. These struts will cut through the earth embankment, so local trenches are necessary to accommodate the struts. The working platform must be checked to have sufficient capacity to withstand the point load from the struts. Additionally, it must have sufficient area to resist sliding from the strut forces.
i) **Removal of Embankment:** After bracing is in place, the embankment can be removed, and the bottom plate can be cast right up against the sheet pile.
j) **Removal of Struts:** After sufficient curing time on the working platform, struts can be removed so that the sheet pile is cantelevered from the working plate (elevation +91.8).

## 4.2 Ground Investigations and Characterization

### 4.2.1 Introduction

It is planned that the existing building will be demolished, and a new building will be erected on the plot. In connection with the preliminary project phase, ground investigations have been carried out. This section summarizes the conducted ground investigations and the associated laboratory results.

### 4.2.2 Field Investigations

The field investigations were conducted from June to August 2021.The investigations were carried out using a GM85 GT and Geotech 605 tracked geotechnical drill rig. (NGI reports)

The drilling program was developed by NGI, and an overview of the location of the points is provided in the drilling plan in fig 4.4

Table 4.1 provides an overview of the drilling points and their coordinates, as well as the drilling methods.

The drilling points were measured in the NTM10 coordinate system and the NN2000 elevation system.

There are:

- 52 Total Soundings were performed to map the relative resistance of the soil, any layer boundaries, and the depth of the bedrock. Typically, three meters are drilled into the rock for a secure detection of it.

- CPTU soundings were carried out at 7 borehole points. The purpose of the CPTU soundings is a more accurate mapping of layer boundaries and as a basis for determining soil parameters, especially the undrained shear strength of clay.
- Pore pressure gauges were installed at two borehole points. Pore pressure gauges are used to measure the pore pressure in the soil. This is used to calculate in-situ stresses and estimate the groundwater level. Electric pore pressure gauges from the Geokon brand were used.
- 17 cylinder samples were taken at borehole point N15 and 14 cylinder samples at N24. A Ø72 mm sampler was used to achieve good quality samples. The samples were taken from 2 to 35 meters below ground level.

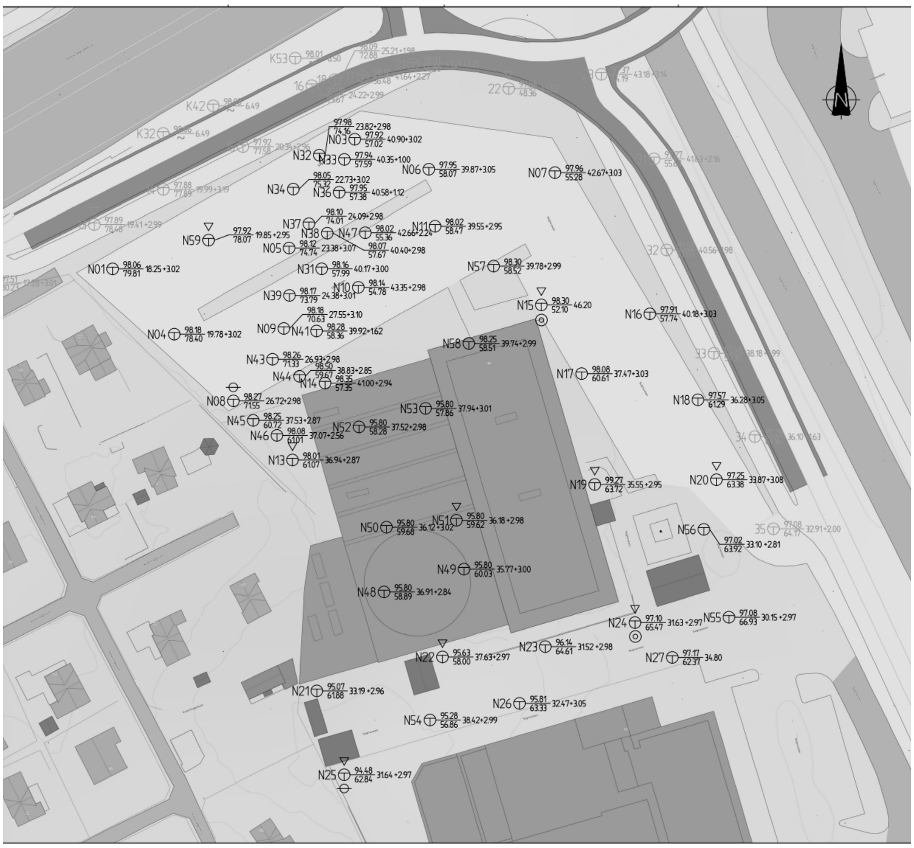



Fig.4.3-location program investigation

Table 4.1-points sounding (TOT = Total Sounding, CPT = Cone Penetration Test, PZ = Pore Pressure PR = Sampling)

| **Borehole Points** | **Metode** | | | |
|---|---|---|---|---|
| | TOT | CPT | PZ | PR |
| N01 | 1 | | | |
| N03 | 1 | | | |

| | | | | |
|---|---|---|---|---|
| N04 | 1 | | | |
| N05 | 1 | | | |
| N06 | 1 | | | |
| N07 | 1 | | | |
| N08 | 2 | 1 | 8,0 m<br>15,0 m | |
| N09 | 1 | | | |
| N10 | 1 | | | |
| N11 | 1 | | | |
| N13 | 1 | | | |
| N14 | 1 | | | |
| N15 | 1 | 1 | | 2,0-34,8 m |
| N16 | 1 | | | |
| N17 | 1 | | | |
| N18 | 1 | | | |
| N19 | 1 | 1 | | |
| N20 | 1 | | | |
| N21 | 1 | | | |
| N22 | 1 | | | |
| N23 | 1 | | | |
| N24 | 1 | 1 | | 3,0-30,8 m |
| N25 | 1 | 1 | 5,25 m<br>12,5 m<br>31,0 m | |
| N26 | 1 | | | |

| | | | | |
|---|---|---|---|---|
| N27 | 1 | | | |
| N31 | 1 | | | |
| N32 | 1 | | | |
| N33 | 1 | | | |
| N34 | 1 | | | |
| N36 | 1 | | | |
| N37 | 1 | | | |
| N38 | 1 | | | |
| N39 | 1 | | | |
| N41 | 1 | | | |
| N43 | 1 | | | |
| N44 | 1 | | | |
| N45 | 1 | | | |
| N46 | 1 | | | |
| N47 | 1 | | | |
| N48 | 1 | | | |
| N49 | 1 | | | |
| N50 | 1 | | | |
| N51 | 1 | 1 | | |
| N52 | 1 | | | |
| N53 | 1 | | | |
| N54 | 1 | | | |
| N55 | 1 | | | |
| N56 | 1 | | | |
| N57 | 1 | | | |
| N58 | 1 | | | |
| N59 | 1 | 1 | | |

The results of the CPTU soundings are shown as individual drillings in figure 4.5.

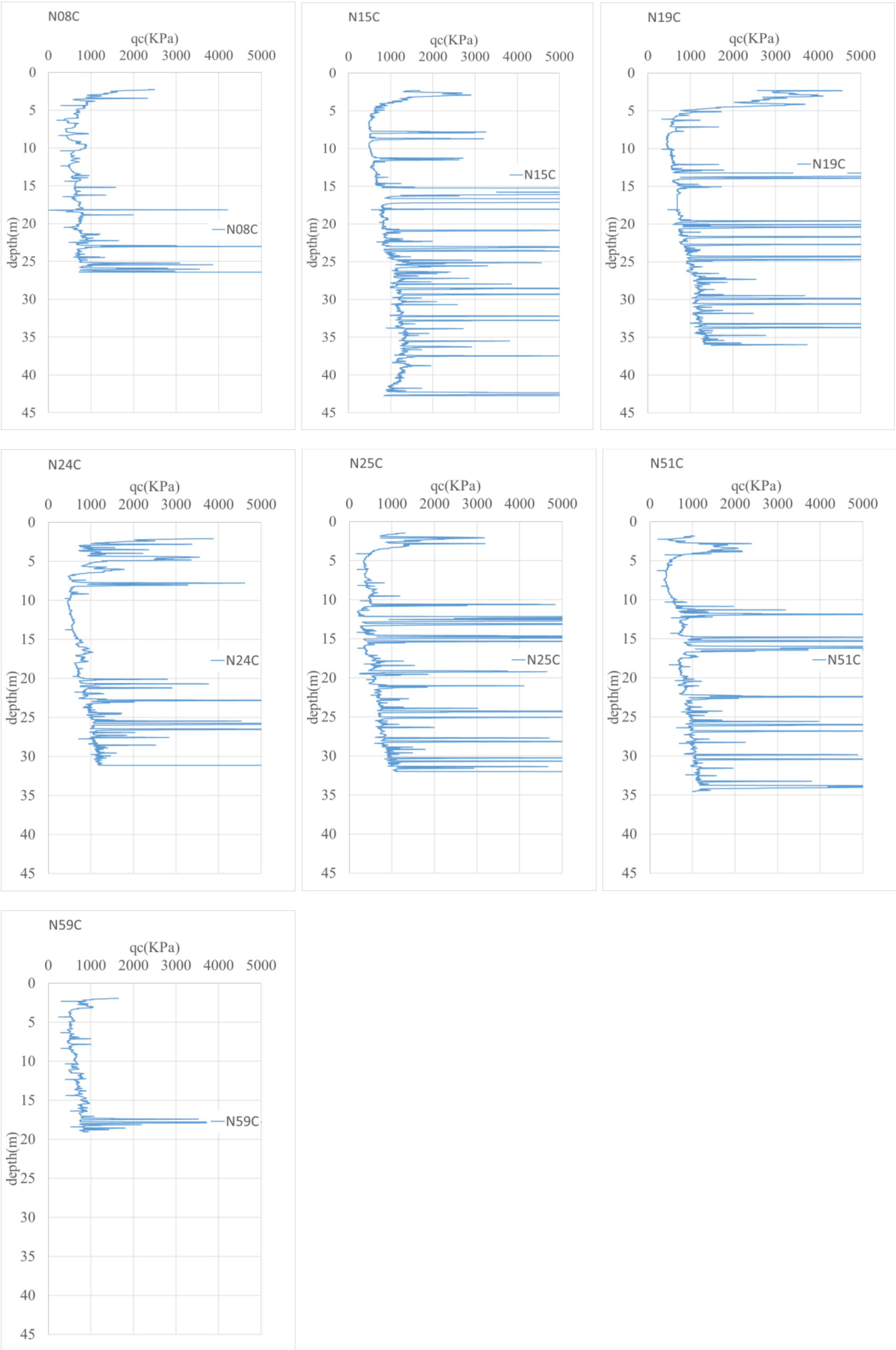



Fig4.5- Results of the CPTU soundings

### 4.2.3 Laboratory Investigations

All samples were analyzed in NGI's laboratory in Oslo. The laboratory program was prepared by NGI. There are:

- Routine Investigations**.** Standard routine investigations were carried out on all cylinder samples. This includes sample opening with visual material description, determination of natural water content (w), density (γ), and shear strength (cu) through both uniaxial compression tests and cone tests in undrained and remolded conditions.
- Triaxial Tests. Three triaxial tests were performed at both borehole points where there is a sample series. The tests were conducted with anisotropic consolidation and were sheared to failure in compression. The purpose of the triaxial tests is to determine the undrained shear strength and friction angle of the material, and they will be used as a basis for correlation to the interpretation of the CPTU soundings.
- Oedometer Tests. Two oedometer tests were performed at both borehole points where there is a sample series. The purpose of the oedometer tests is to determine the consolidation and deformation properties of the soil, as well as to estimate the preconsolidation stress (pc'). The tests conducted are of the CRS (Constant Rate of Strain) type.

The results of the triaxial tests are in table 4.2.

## 4.3 Geotechnical model

This section summarizes the interpretation of ground investigations and the determination of geotechnical design parameters for use in geotechnical detailed design.

The interpretation of ground conditions was conducted by NGI as part of the detailed design referred to the NGI reports.

### 4.3.1 Ground Conditions

The site is located at approximately elevation +98 in the north and +96 in the south. The thickness of the loose masses at the borehole locations on the site ranges from 35-45 m, except in the northwest corner where it decreases to around 20 m. The site was previously occupied by a 5-story office building in the southern half and surface parking in the northern half.

### 4.3.2 Experience based values

Parameters for masses that have not been investigated are taken from empirical values given in HB V220 - Geoteknikk i vegbygging.

- Fill Materials

Based on experience, the following strength parameters are used for fill materials:

Crushed stone, blasted rock, or similar: $a = 0$ kPa $\varphi'_k = 42°$

Fill materials (local, unknown quality): $a = 0$ kPa $\varphi'_k = 35°$

Fill materials above groundwater level: $\gamma = 19\ \text{kN/m}^3$

Fill materials below groundwater level: $\gamma = 22\ \text{kN/m}^3$

- Dry Crust Materials and Local Friction Materials

For dry crust, local friction materials, and local solid materials,

the following parameters are used: $a = 0$ kPa $\varphi' = 30°$.

density used: $\gamma = 19\ \text{kN/m}^3$

### 4.3.3 Interpretation of Ground Investigations

The soil consists of 1-2 m of fill material/dry crust over moderately stiff to stiff clay with occasional occurrences of silty clay, with relatively little variation across the area. From around 9 meters depth down to the bedrock, the clay is sensitive. Some cone penetration tests also indicate sensitive clay at depths shallower than 9 meters.

There is some local variation in the subsurface conditions. However, it is considered feasible to establish a single design profile for the entire site.

#### 4.3.3.1 Groundwater Level and Pore Pressure

Pore pressure measurements have been conducted in two boreholes (N08 and N25) on the site. The calculated groundwater level in the boreholes is approximately 2 meters deep for N08 (northwest on the site) and 1 meter deep for N25 (south on the site). Borehole N08 indicates a slightly under-hydrostatic condition, while borehole N25 indicates a slightly over-hydrostatic condition. A hydrostatic pore pressure distribution is assumed with a groundwater level between 1 and 2 meters below the ground surface, depending on the location on the site.

#### 4.3.3.2 Soil Classification and Index Parameters

Density. The density of the masses varies somewhat. There are some individual tests outside the trend that suggest there are a few layers with local variation. At approximately 25 meters depth, the clay masses appear to have a slightly lower density. The average density for local clay masses is set at $\gamma = 19.3\,\mathrm{kN/m^3}$.

Water Content. There is some variation in the measured water content, even for individual tests in the same sample tube. However, the measurements mostly fall within the range of 25 – 35%, which is quite typical for medium-stiff Norwegian clay.

Sensitivity. Both sample series show a marked increase in sensitivity at approximately 9 meters depth. From this depth and downwards, the clay is mostly quick (remolded strength lower than 0.5 kPa in cone tests); consequently, the measured sensitivity varies greatly from test to test, as small variations in remolded strength result in large differences in calculated sensitivity.

Plasticity. The measured plasticity index, with the exception of one sample, lies in the range of 7-10% at 0-20 meters depth, after which the variation increases.

#### 4.3.3.3 Over Consolidation Ratio and Undrained Shear Strength

The over consolidation ratio and the undrained shear strength is interpreted from pressure soundings and laboratory test. The pressure soundings are interpreted based on the formulas given in CPTU correlations for Norwegian clays: an update Paniagua et al. (2019). A compilation of interpretations of individual soundings, as well as the design profile, is presented in Fig.4.6

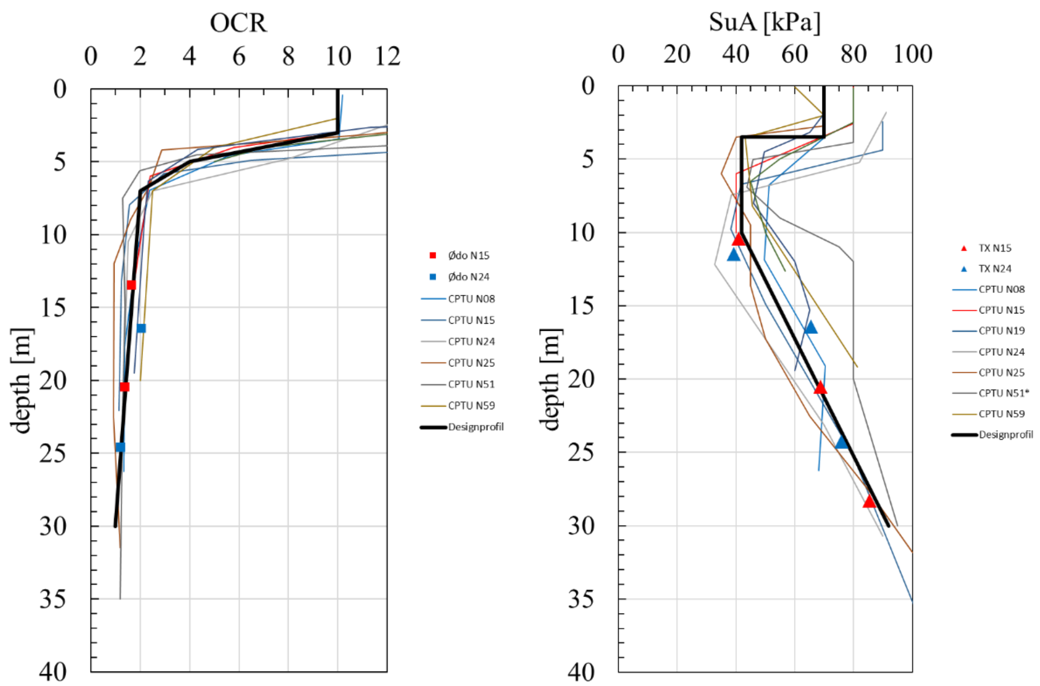



Fig4.6-Interpretation CPTU and laboratory text

#### 4.3.3.4 Clay Stiffness in the "Mohr-Coulomb" Soil Model

The clay stiffness in the Mohr-Coulomb soil model is interpreted from triaxial compression tests, see Appendix C for interpretation and Figure 14 for compilation. The following stiffness relationships are established:

$E_0$ $\quad 1400 \cdot s_u^A$

$E_{50}$ $\quad 750 \cdot s_u^A$

$E_{ur}$ $\quad 3 \cdot \mathrm{E}_{50} = 2250 \cdot s_u^A$

#### 4.3.3.5 Clay Stiffness in the NGI-ADP Soil Model

The NGI-ADP soil model is used to account for anisotropic undrained shear strength conditions in clays. This model helps in understanding and predicting the behaviour of clay under different stress conditions, which is crucial for geotechnical engineering projects. The undrained stiffness parameters in local clay masses are calibrated against triaxial compression tests, summarized in table 4.3.

| **Borehole** | **Depth [m]** | **$\tau_0$ [kPa]** | **$s_u^A$ [kPa]** | **$\tau_0 / s_u^A$ [~]** | **$G_{ur}/s_u^A$ [~]** | **$\gamma_f^C$ [%]** |
|---|---|---|---|---|---|---|
| N15 | 10 | 27 | 41 | 0,7 | 700 | 0,9 |
| N24 | 16 | 40 | 65 | 0,6 | 700 | 2,0 |
| N24 | 24 | 50 | 76 | 0,7 | 700 | 0,7 |
| N15 | 28 | 64 | 86 | 0,7 | 700 | 0,5 |

Table 4.3-parameters NGI-ADP model

**Campus Ullevål - Preliminary Project SUMMARY OF TRIAXIAL TESTS**

| Sample Identification | | | | | Index Properties | | | | | | Consolidation | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Hole nr. | Sample diameter | Cylinder Part | depth | soil type | $w_i$ | $w_l$ | $w_p$ | Ip | Leir Innh. | $\gamma_{tot}$ | Type of Test | $p'_{0v}$ | $\sigma'_{ac}$ | $\sigma'_{rc}$ | $K_0'$ | $\varepsilon_{vol}$ | $\varepsilon_{ac}$ | $w_c$ | B | $\Delta e/e_0$ | $\Delta e/e_0$ | Sample Quality |
| | *mm* | | *m* | | % | % | % | % | % | *kN/m* $^3$ | | *kPa* | *kPa* | *kPa* | | % | % | % | % | | | |
| N15 | 72 | 6-A-1 | 10,43 | Clay | 28,2 | | | 0,0 | | 19,70 | CAUA | 114 | 113,8 | 66,0 | 0,58 | 2,59 | 1,59 | 26,5 | 99,0 | 0,059 | 0,060 | Good to usable |
| N15 | 72 | 11-A-1 | 20,52 | Clay | 27,2 | | | 0,0 | | 19,50 | CAUA | 205 | 204,5 | 118,6 | 0,58 | 3,38 | 2,06 | 25,0 | 98,0 | 0,079 | 0,081 | Poor |
| N15 | 72 | 14-A-1 | 28,29 | Clay | 38,9 | | | 0,0 | | 18,50 | CAUA | 275 | 274,6 | 159,2 | 0,58 | 3,72 | 3,11 | 36,2 | 98,5 | 0,072 | 0,069 | Good to usable |
| N24 | 72 | 5-A-1 | 11,47 | Clay | 31,4 | | | 0,0 | | 19,30 | CAUA | 123 | 123,2 | 71,5 | 0,58 | 3,31 | 2,67 | 29,2 | 98,1 | 0,071 | 0,070 | Poor |
| N24 | 72 | 7-A-1 | 16,40 | Clay | 25,8 | | | 0,0 | | 19,60 | CAUA | 169 | 168,9 | 98,0 | 0,58 | 1,93 | 1,23 | 24,6 | 90,5 | 0,047 | 0,047 | Good to usable |
| N24 | 72 | 11-A-1 | 24,25 | Clay | 30,4 | | | 0,0 | | 19,60 | CAUA | 238 | 238,4 | 138,2 | 0,58 | 3,25 | 2,34 | 28,3 | 98,0 | 0,071 | 0,069 | Good to usable |

| | | | |
|---|---|---|---|
| $w_i$ | in-situ water content | $\varepsilon_{vol}$ | Volumetric strain during consolidation |
| $w_l$ | liquid limit | $\varepsilon_{ac}$ | Vertical strain during consolidation |
| $w_p$ | Plastic limit | B | Skempton's pore pressure factor |
| Ip | Plastic index | $\tau_f$ | Shear stress at failure |
| $p'_{0v}$ | In-situ vertical effective stress | $u_f$ | Pore pressure in the sample at failure |
| $\sigma'_{ac}$ | Vertical consolidation stress | $\varepsilon_f$ | Vertical strain failure |
| $\sigma'_{rc}$ | Horizontal consolidation stress | $\Delta e/e_0$ | $\Delta e = \varepsilon_{vol}(1+ei)$ and $ei = 2.75 * w_i$ |

Sample quality:
according to H211

| | |
|---|---|
| **Velding god til utmerket** | Very good to excellent |
| **God til brunkbar** | Good to usable |
| **Darlig** | Poor |
| **Veldig darlig** | Very poor |

| OCR | $\Delta e/e_0$[4] | | | |
|---|---|---|---|---|
| | **Veldig god til utmerket** | **God til brukbar** | **Dårlig** | **Veldig dårlig** |
| 1–2 | <0,04 | 0,04-0,07 | 0,07-0,14 | >0,14 |
| 2–4 | <0,03 | 0,03-0,05 | 0,05-0,10 | >0,10 |
| 4–6 | <0,02 | 0,02-0,035 | 0,035-0,07 | >0,07 |

Table 4.2-summary of triaxal tests

## 4.4 Displacement Monitoring

### 4.4.1 Introduction

The research project has implemented a significant instrumentation program to monitor and assess the performance of the support structure and the surrounding assets during the excavation stages. The measurement program presented in fig4.6 which includes the following instruments and points:

a) Web Camera with regularly updated overview images of the site

b) 6 pore pressure meters (Manual reading, NGI)

   a. 5 in the vicinity

   b. 1 within the construction site

c) 10 inclinometers (NGI-live)

   a. 5 inside the excavation pit

   b. 4 next to the back of the sheet pile wall

   c. 1 located 4 meters outside the sheet pile wall

d) 30 prisms for automatic movement measurement (NGI-live)

   a. 17 on top of the sheet pile wall

   b. 13 on buildings west of the excavation pit

e) 106 points for manual movement measurement (contractor)

   a. 39 reflectors on the sheet pile

   b. 9 on south of the excavation pit

   c. 16 on houses northeast of the excavation pit

   d. 6 asphalt spikes on the terrain outside

   e. 36 on the pedestrian bridge north and east of the excavation pit

f) 2 thermometers in the clay behind the sheet pile (not shown in the drawing).

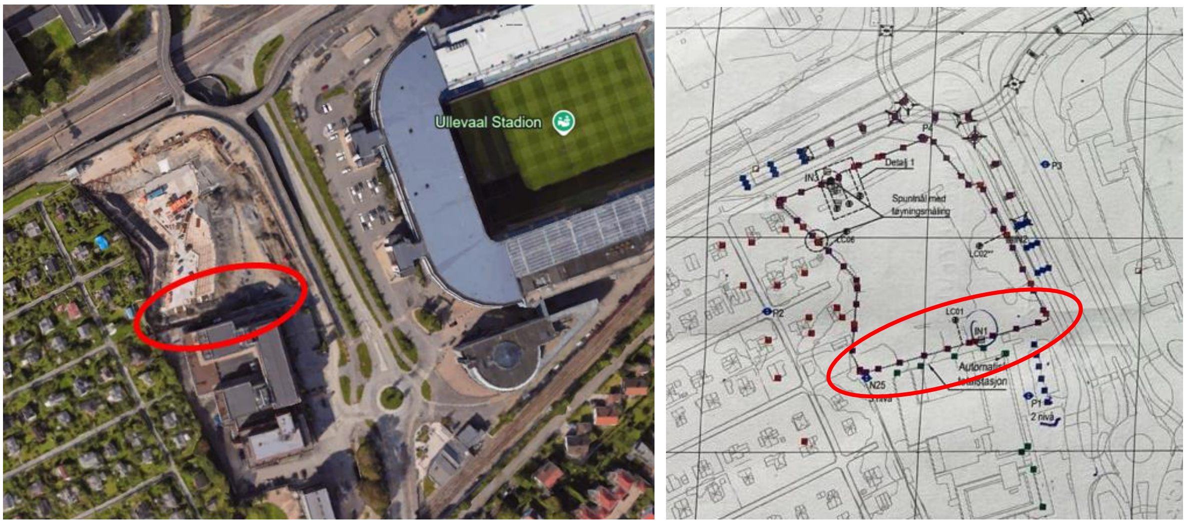



Fig4.7-focus of case study

The focus is on the soil-structure interaction between the support structure (cantilever SPW) and the adjacent building during the different phases in the construction process.

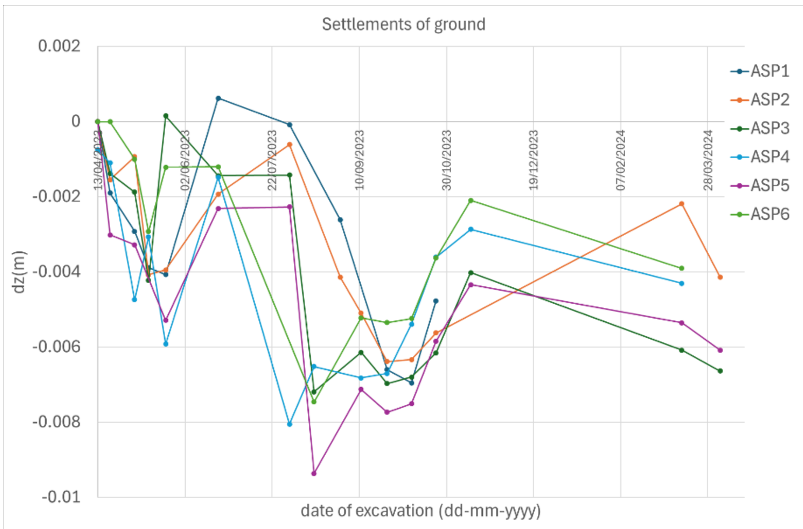
Monitoring of the sheet pile wall (SPW) in terms of inclinometers and surveying points on the top of the SPW, as well as settlement measurements on the surrounding buildings have been carried out during the ground works.

We want to obtain displacements orthogonal to the sheet pile wall, settlements of ground, settlements of building for compare with the numerical results.

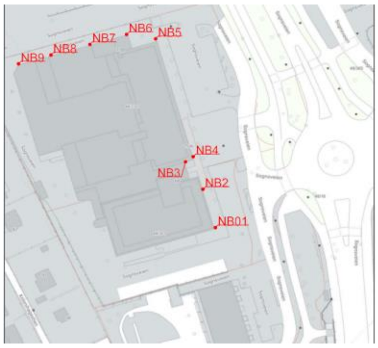
Important to consider the orientation of the sheet pile wall with respect to the coordinate system. Calculated displacements orthogonal (dn) to sheet pile wall.

### 4.4.2 Results of monitoring

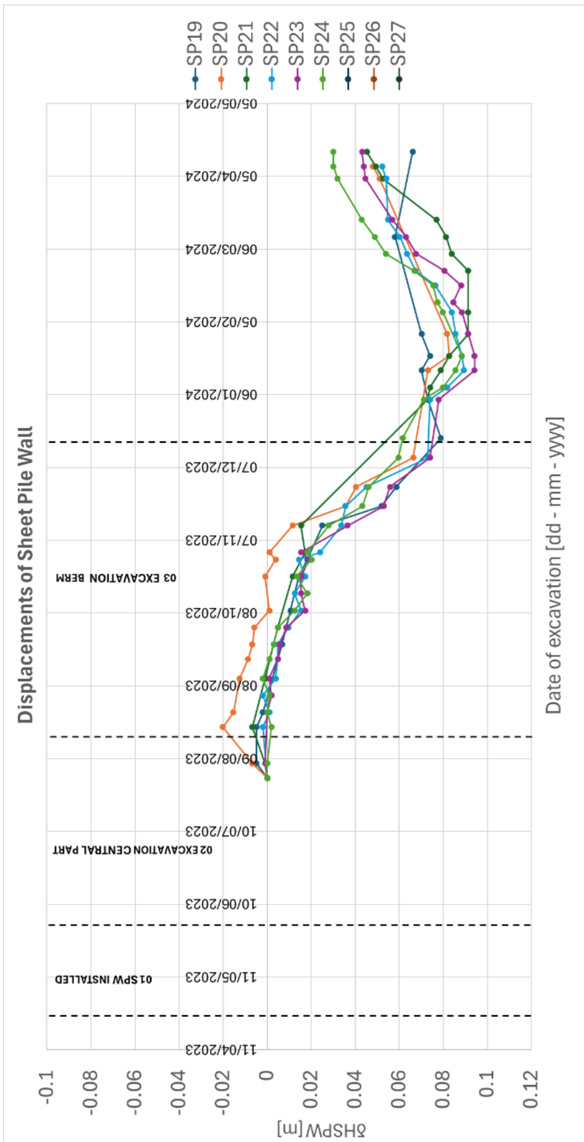
Results of ASP1-ASP6 adjacent building during the different phases of excavation.



Results of NB01-NB09 on building during the different phases of excavation.


Results of SP19-SP27 on sheet pile wall adjacent building during the different phases.



## 4.5 Back analysis

### 4.5.1 Introduction

From site and laboratory tests, the soil characterization was derived as follows: The top layer, composed of fill material and dry crust clay, shows a thickness of 1 to 2 meters. A medium stiff to stiff, low-sensitive clay of 6 to 7 meters thickness is situated below the top layer and overlays a soft to medium-stiff quick clay down to bedrock. The groundwater table was situated 1 to 1.5 meters below the original terrain during the investigations.

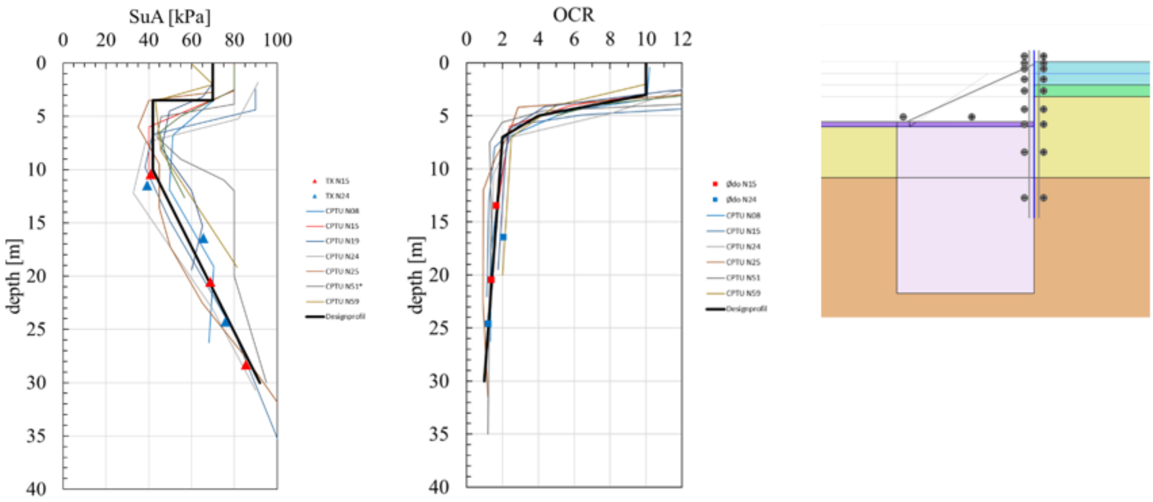



Fig4.8-Geotechnical model

The calculation phases in Plaxis 2D follow the steps outlined in the description of works.

### 4.5.2 Input parameters

Figure 4.9 and Tables 4.4 to 4.10 summarize the materials defined for the calculations performed in Plaxis. They are the same as the original detailed design. (NGI report).

The top layer/dry crust clay and the drained soil wedge are modelled as "hardening-soil" materials, while the underlying clay is modelled as an "NGI-ADP" material.

Parameters used for the different soil layers are summarized in Table 4.4 and Table 4.5 for "hardening-soil" materials and "NGI-ADP" materials, respectively.

The lime-cement stabilized clay is modelled as a "Mohr-Coulomb" material. It is modelled with an average strength and stiffness of the stabilized material.

Additionally, all interface materials are modelled as "Mohr-Coulomb" materials. Table 4.6 summarizes parameters for these materials.

The working platform is modelled as a linear elastic soil material with stiffness and unit weight equivalent to concrete. The parameters used are summarized in Table 4.7

The steel structures are modelled as elastoplastic 1D materials with parameters as given in Table 4.8 and Table 4.9

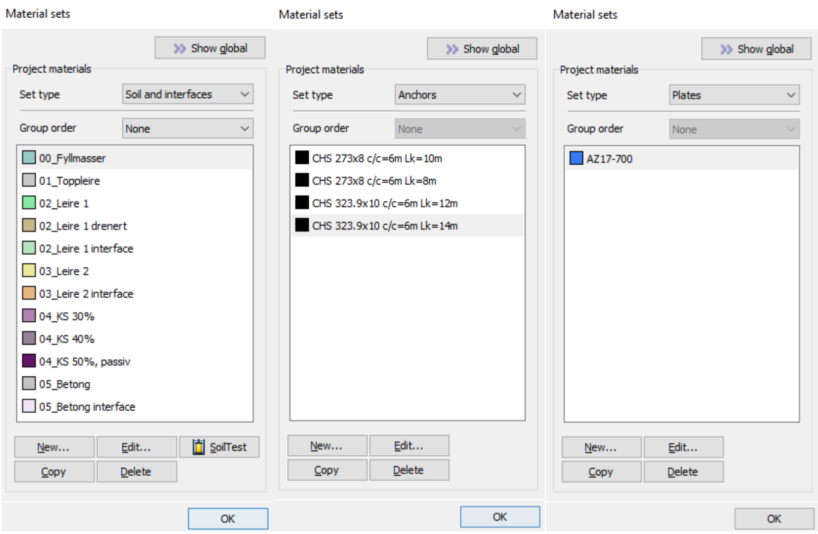



Fig 4.9 Overview of defined materials used in Plaxis calculations

Table 4.4 – “Hardening soil"-materials used in Plaxis-calculations

| **Symbol** | **Unit** | **00_Fill Materials** | **02_Clay 1 Drained** |
|---|---|---|---|
| Drainage | - | Drained | Drained |
| $\gamma_{unsat}$ | $kN/m^3$ | 19 | 19,3 |
| $\gamma_{sat}$ | $kN/m^3$ | 22 | 19,3 |
| $E_{50}^{ref}$ | $kN/m^2$ | $20 \cdot 10^3$ | $10 \cdot 10^3$ |
| $E_{oed}^{ref}$ | $kN/m^2$ | $20 \cdot 10^3$ | $10 \cdot 10^3$ |
| $E_{ur}^{ref}$ | $kN/m^2$ | $60 \cdot 10^3$ | $30 \cdot 10^3$ |
| $\nu_{ur}$ | - | 0,2 | 0,3 |
| $c'^{ref}$ | $kN/m^2$ | 5 | 0 |
| $\varphi'$ | ° | 35 | 27 |
| Tension-cutoff | - | Yes (0kPa) | Yes (0kPa) |
| $R_{inter}$ | - | 0,5 | - |

Table 4.5 - "NGI-ADP"-materials used in Plaxis-calculations

| **Symbol** | **Unit** | **01_Top Clay** | **02_Clay 1** | **03_Clay 2** |
|---|---|---|---|---|
| Drainage | - | Undrained C | Undrained C | Undrained C |
| $\gamma_{unsat}$ | $kN/m^3$ | 19,3 | 19,3 | 19,3 |
| $\gamma_{sat}$ | $kN/m^3$ | 19,3 | 19,3 | 19,3 |
| $G_{ur}/s_u^A$ | - | 700 | 700 | 700 |
| $\nu_u$ | - | 0,499 | 0,499 | 0,499 |

| | | | | |
|---|---|---|---|---|
| $s_u{}^A{}_{ref}$ | kN/m² | 70 | 42 | 42 |
| $s_u{}^A{}_{inc}$ | kN/m²/m | 0 | 0 | 2,5 |
| $y_{ref}$ | M | - | - | 87 |
| $s_u{}^{DSS}/s_u{}^A$ | - | 0,63 | 0,63 | 0,63 |
| $s_u{}^P/s_u{}^A$ | - | 0,35 | 0,35 | 0,35 |
| $\gamma_f{}^C$ | % | 0,90 | 0,90 | 0,90 |
| $\gamma_f{}^E$ | % | 2,40 | 2,40 | 2,40 |
| $\gamma_f{}^{DSS}$ | % | 1,40 | 1,40 | 1,40 |
| $\tau_0/s_u{}^A$ | - | 0,70 | 0,70 | 0,70 |

Table 4.6 - Lime cement-materials used in Plaxis-calculations "Mohr-Coulomb"-soil model

| **Symbol** | **Unit** | **04_LC 30%** | **04_LC 40%** |
|---|---|---|---|
| Drainage | | - | |
| $\gamma_{unsat}$ | kN/m³ | 19,5 | 19,5 |
| $E_{u,ref}$ | kN/m² | $26,91 \cdot 10^3$ | $33,78 \cdot 10^3$ |
| $\nu_u$ | - | 0,495 | 0,495 |
| $G_{ref}$ | kN/m² | $9 \cdot 10^3$ | $11,3 \cdot 10^3$ |
| $E_{oed}$ | kN/m² | $0,91 \cdot 10^6$ | $1,14 \cdot 10^6$ |
| $C_{ref}$ | kN/m² | - | - |
| Φ | ° | - | - |
| $s_u{}^A{}_{ref}$ | kN/m² | 72 | 90 |
| $s_u{}^A{}_{inc}$ | kN/m²/m | 0 | 0 |
| $y_{ref}$ | M | - | - |
| Tension-cutoff | - | No | No |
| $R_{inter}$ | - | 0,7 | 0,7 |

Table 4.7 - Other "Mohr-Coulomb" materials used in PLAXIS calculations.

| **Symbol** | **Unit** | **02_Clay 1 interface** | **02_Clay 2 interface** | **05_Plate interface** |
|---|---|---|---|---|
| Drainage | | - | | Undrained C |
| $\gamma_{unsat}$ | kN/m³ | - | - | - |
| $E_{u,ref}$ | kN/m² | - | - | - |
| $\nu_u$ | ~ | - | - | - |
| $G_{ref}$ | kN/m² | - | - | - |
| $E_{oed}$ | kN/m² | - | - | - |
| $C_{ref}$ | kN/m² | - | - | 100 |
| Φ | ° | - | - | 35 |
| $s_u{}^A{}_{ref}$ | kN/m² | 42 | 42 | - |
| $s_u{}^A{}_{inc}$ | kN/m²/m | 0 | 2,5 | - |
| $y_{ref}$ | M | - | 87 | |

| | | | | |
|---|---|---|---|---|
| Tensi-cutoff | - | Yes (0kPa) | Yes (0kPa) | Yes (100kPa) |

Table 4.8 - "Linear-elastic" materials used in PLAXIS calculations

| Symbol | Unit | 05_concrete |
|---|---|---|
| Drainage | - | Non-porous |
| $\gamma_{unsat}$ | $kN/m^3$ | 25 |
| $E_{ref}$ | $kN/m^2$ | $30 \cdot 10^6$ |
| $\nu_u$ | - | 0,2 |
| $G_{ref}$ | $kN/m^2$ | $12{,}5 \cdot 10^6$ |
| $E_{oed}$ | $kN/m^2$ | $33{,}33 \cdot 10^6$ |
| $R_{inter}$ | - | 0,5 |

Table 4.9 - Sheet Pile Materials Used in PLAXIS Calculations

| Symbol | Unit | AZ17-700 |
|---|---|---|
| W | kN/m/m | 1,02 |
| EA | kN/m | $2{,}79 \cdot 10^6$ |
| EI | $kN/m^2/m$ | $76{,}1 \cdot 10^3$ |
| ν (Nu) | - | 0 |
| $M_{pl}$ | kNm/m | 720 |
| $V_{pl}$ | kNm/m | 4220 |

Table 4.10 - Stiffness Materials Used in PLAXIS Calculations

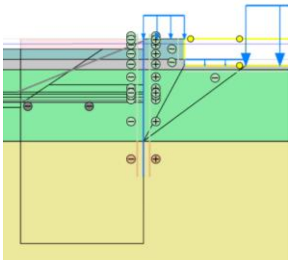
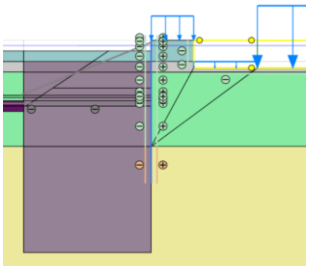
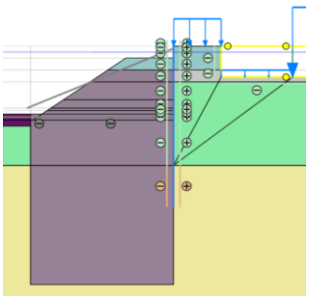
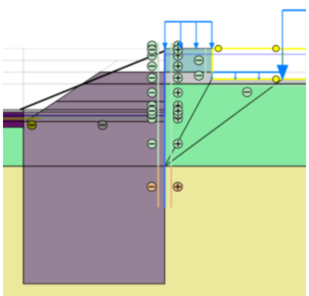
| Symbol | | CHS 323,9x10 c/c 6m | | CHS 273x8 c/c 6m | |
|---|---|---|---|---|---|
| | | $L_k$=14m | $L_k$=12m | $L_k$=10m | $L_k$=8m |
| $L_{spacing}$ | M | 6,0 | | 6,0 | |
| EA | kN | $2{,}07 \cdot 10^6$ | | $1{,}4 \cdot 10^6$ | |
| \|Fmax tens\| | kN | 3500 | | 2350 | |
| \|Fmax comp\| | kN | 1100 | 1400 | 990 | 1350 |

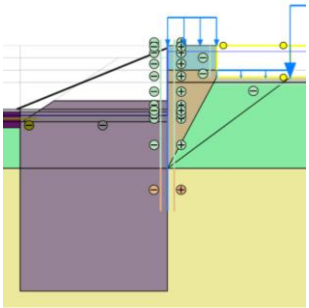
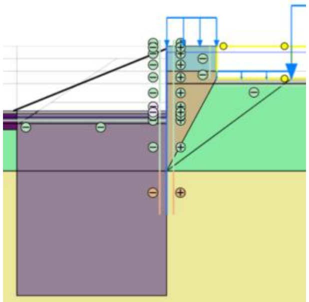
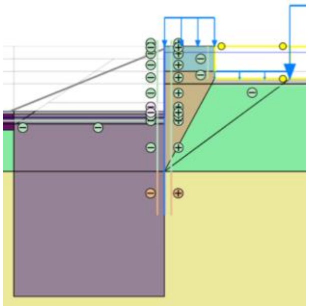
In Plaxis, the building is modeled using plate elements with cross-sectional properties equivalent to 15 cm thick concrete. Above-ground floors are considered by applying a line load of 10 kPa per floor, total 50 kPa. For the pile-founded section, a load of 5 kPa is applied for the ground plate. There is a private road between neighbouring building and the planned sheet pile wall.

A lime-cement stabilization with a coverage degree of 40% down to elevation +77 is planned. Double ribs will be placed at a distance of 12 meters from the sheet pile wall.

A summary of the calculation phases, along with a description of each phase and photo web camera, is provided in Table 4.11.

Table 4.11-descrption each excavation phase

| Phase | Description | Photo web camera |
|---|---|---|
| 01_SPW installed  | Installation of sheet piles and excavation of 1 meter inside the building pit. |  |
| 02_Lime cement  | Lime Cement stabilization ribs next to sheet piles. |  |
| 03_Excavation central part  | Excavation to elevation +91.3 at the center of the building pit, with a soil embankment left against the sheet piles. |  |
| 04_Bracing installed  | The working platform is cast in the building pit up to the embankment foot, and installation of internal struts to support SPW. |  |

| | | |
|---|---|---|
| 05_Excavation of berm  | The embankment is removed next to the sheet piles, including a portion of the embankment to model a favourable 3D effect for sectional excavation. |  |
| 06_Plate against SPW  | The working platform is finished casting against the sheet piles. |  |
| 07_Cantilever SPW  | Braces are removed, and the sheet piles protrude from the level of the working platform. Full size of drained area behind sheet piles. |  |

## 4.6 Result and discussion

### 4.6.1 Comparison between measured and calculated displacements of sheet pile walls – design parameters

Compare between Inclinometer data IN1, measurement SP21-22 and Plaxis design analysis 2D.

The measurements from middle Dec and during the winter is difficult to relate to since the frost effects are so big.

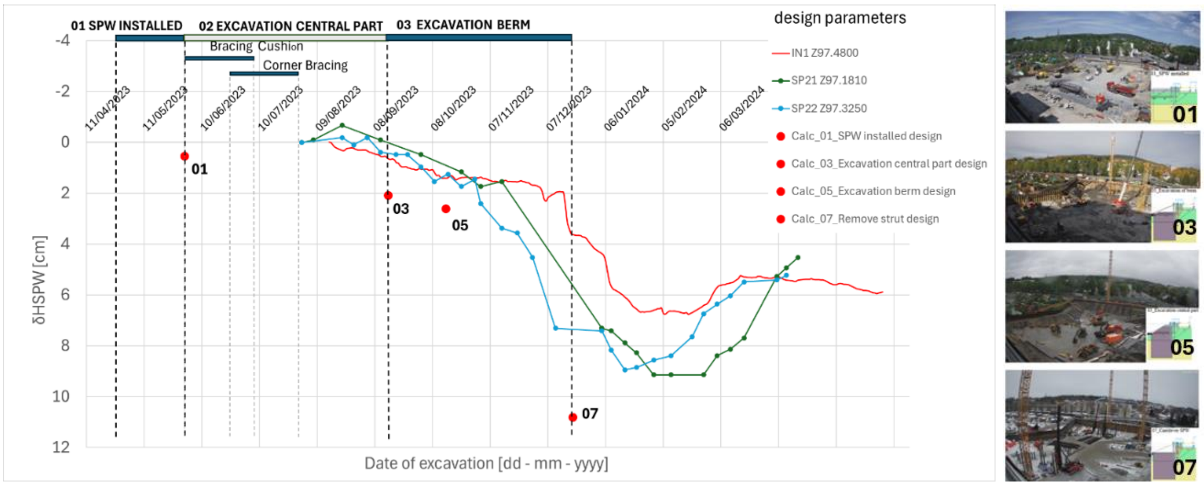
### 4.6.2 Comparison between measured and calculated displacement of sheet pile walls – parametric analysis

In this section explain that from the comparison between monitoring data and design results during the excavation phases, there is a difference. To understand why this difference exists, the first analysis involved eliminating the design load of a road, which was likely not accessible during the excavation phases; therefore, it was appropriate to assign a load of 0 instead of 19. The second step was to move the bedrock from 60 meters as per the design to 71.5 meters, which is the position of the inclinometer, as the displacement measurement from the data was approximately 0. Other aspects analyzed in subsequent analyses included increasing the Cu resistance of the treated soil (purple color) and increasing the stiffness of the treated soil Eu. The analyses are summarized in Table 4.12. The first analysis (n1) is based on the design parameters, while in the other analyses (n2 to n10), the design parameters are modified. The columns show the orthogonal displacement values of the sheet pile wall at the top (97.35m) for calculation phase 01, 03, 05 and 07 in Plaxis.

| | | | lime cement 40% | | δHSPW [cm] | | | |
|---|---|---|---|---|---|---|---|---|
| n | bottom cut-off (m) | qroad (KN/m/m) | Cu (KPa) | Eu (KPa) | 01SPW Installed | 03Excavation Central Pa | 05Excavation Berm | 07Cantilever SPW |
| 1 | 60.00 | 19.00 | 90.00 | 33780.00 | 0.55 | 2.08 | 2.60 | 10.80 |
| 2 | 71.50 | 0.00 | 90.00 | 33780.00 | 0.36 | 1.49 | 1.88 | 8.83 |
| 3 | 71.50 | 0.00 | 150.00 | 33780.00 | 0.36 | 1.32 | 1.88 | 8.83 |
| 4 | 71.50 | 0.00 | 150.00 | 56300.00 | 0.36 | 1.21 | 1.69 | 7.77 |
| 5 | 71.50 | 19.00 | 150.00 | 33780.00 | 0.52 | 1.31 | 1.91 | 9.80 |
| 6 | 71.50 | 19.00 | 150.00 | 56300.00 | 0.52 | 1.19 | 1.71 | 8.65 |
| 7 | 71.50 | 0.00 | 300.00 | 33780.00 | 0.35 | 1.31 | 1.86 | 8.77 |
| 8 | 71.50 | 0.00 | 300.00 | 112600.00 | 0.36 | 1.11 | 1.50 | 6.72 |
| 9 | 71.50 | 19.00 | 300.00 | 33780.00 | 0.52 | 1.29 | 1.91 | 9.98 |
| 10 | 71.50 | 19.00 | 300.00 | 112600.00 | 0.52 | 1.12 | 1.57 | 7.59 |

Table 4.12-parameters analyses plaxis, orthogonal displacement of the sheet pile wall at the top

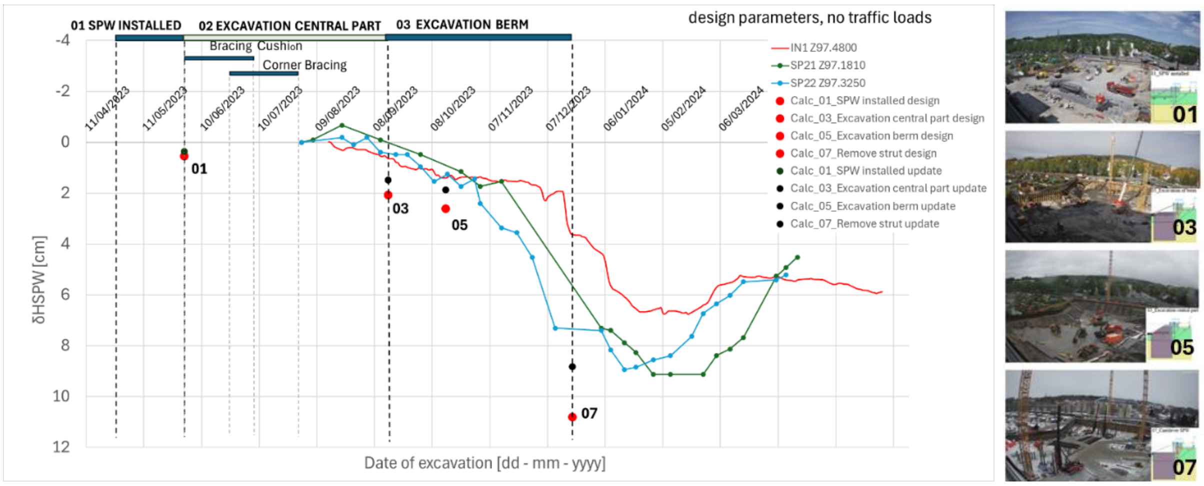
*Comparison between measured and calculated displacements of sheet pile walls – design parameters*



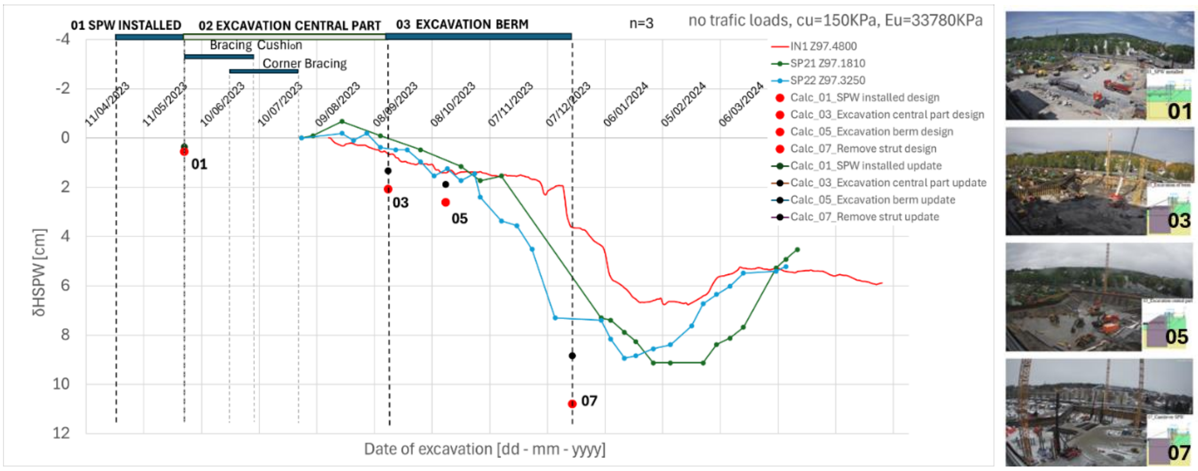

The first comparison was made with the results of design parameters (red dots in the figure) and the measured values SP21 SP22 at the top of the sheet pile wall (green and blue lines), with he measured values from inclinometer IN1 (solid red line) located about 20 cm behind the sheet pile wall and between points SP21 and SP22.

*Comparison between measured and calculated displacement of sheet pile walls – parametric analysis*

The black points are the calculated points after changing the design parameters in Plaxis, and they are compared with the design results (red points). Each analysis is compared with the measured values and the design values. The graph titles, from n3 to n10, represent the modified values, which are reported in Table 4.12.

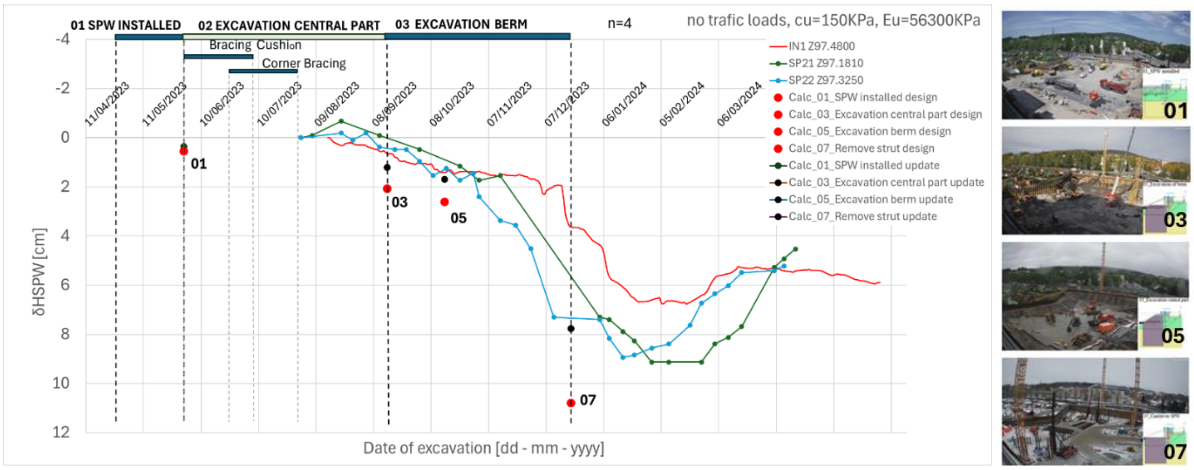


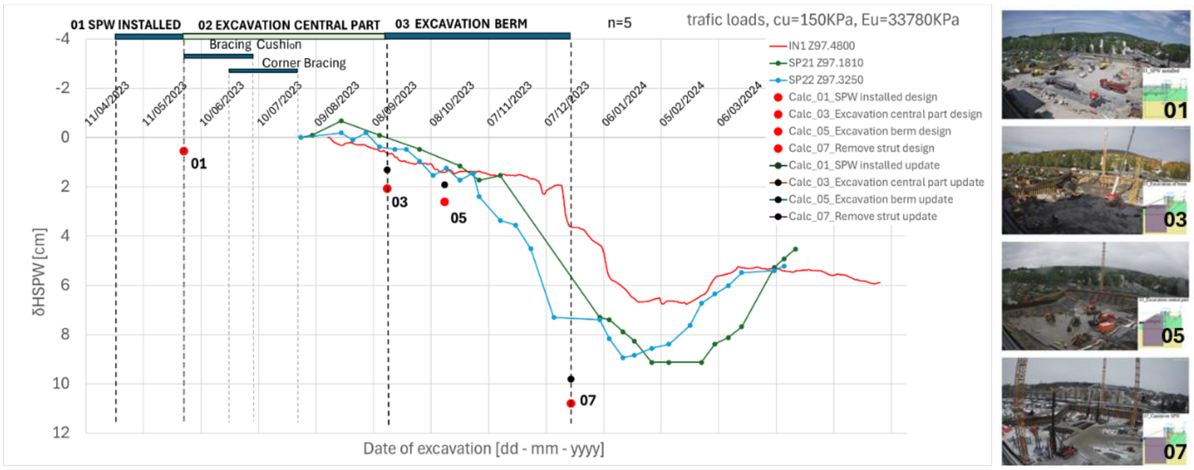

In this figure, we observe that by eliminating the road load and adjusting the bedrock level to 71.5m, the black points (calculated) closely align with the measured values.

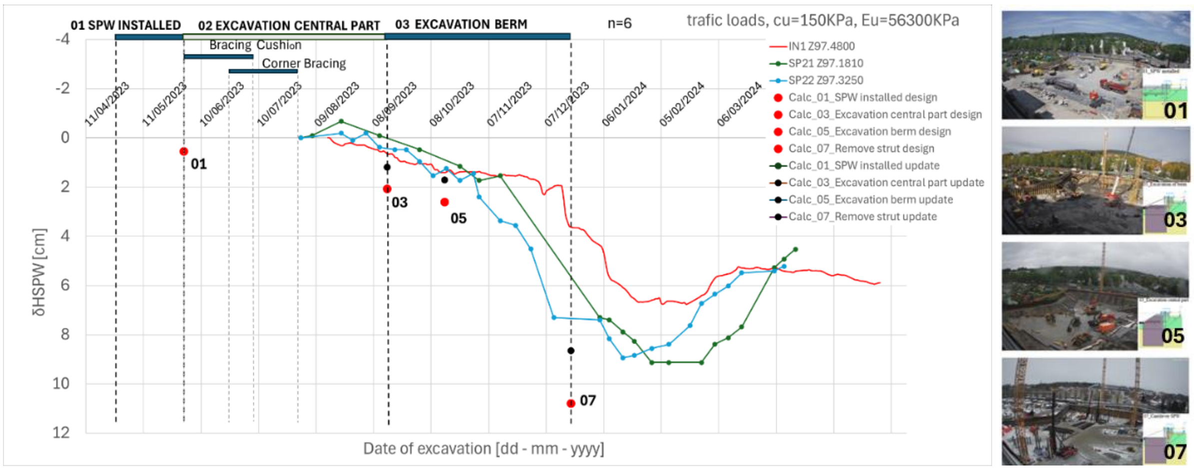



In this figure, the design resistance of the treated soil has been increased from 90 to 150, the stiffness remains a design parameter, and the road load is null. On the right side of the figure in Plaxis, we see photos of the excavation phases 01 Sheet pile walls installed, phase 03 Excavation central part , phase 05 Excavation of berm and phase 07 Cantilever Sheet pile walls.

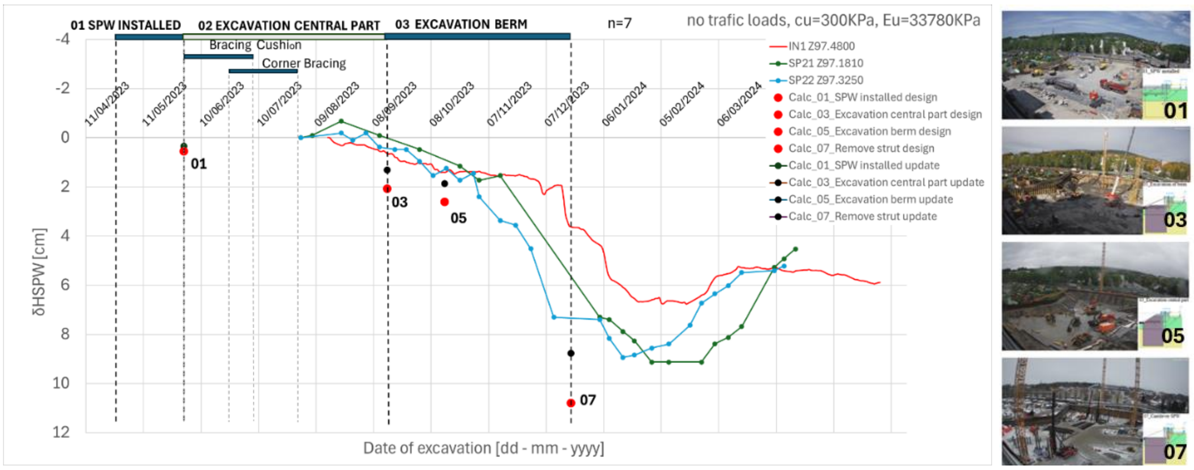
In the following figures, the resistance of the treated soil is increased from 90 to 150 to 300.

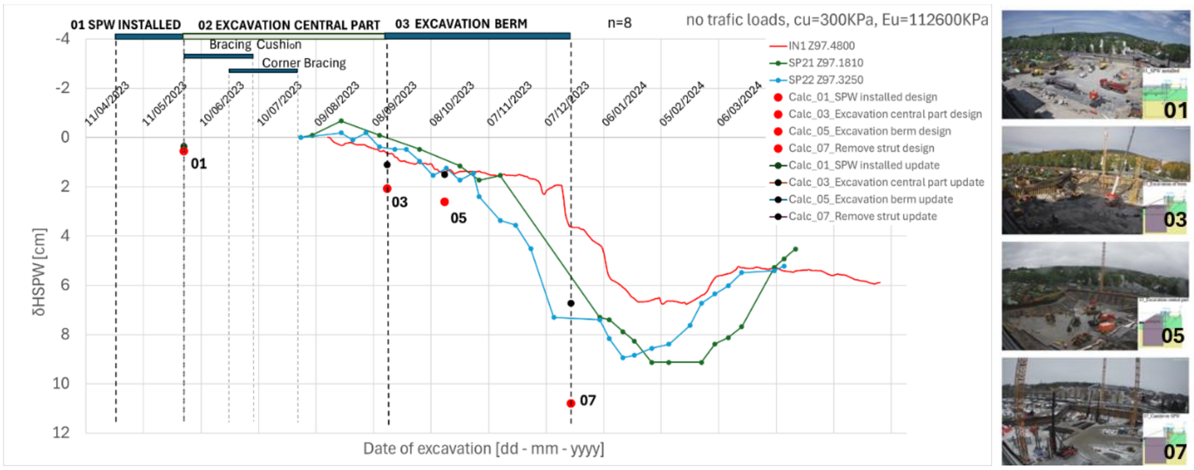


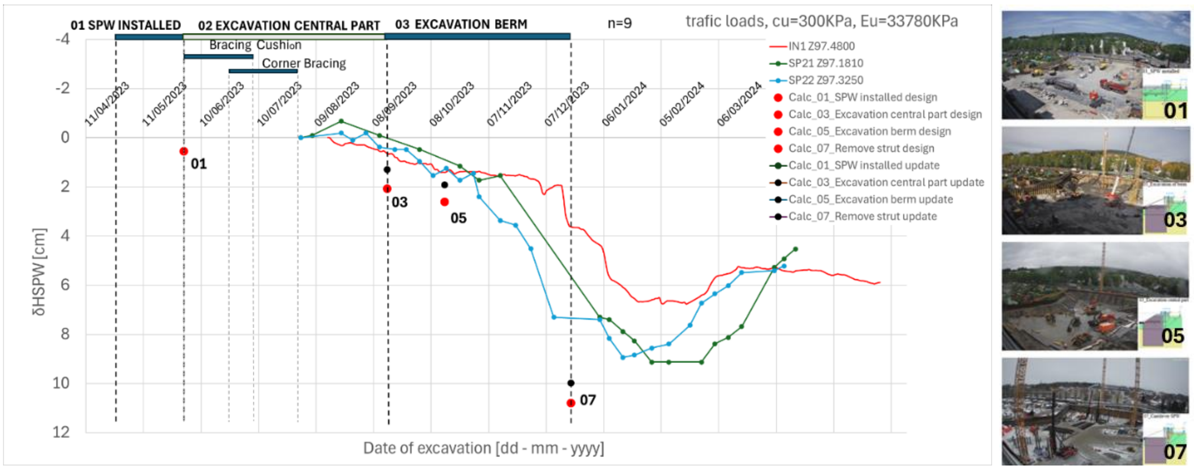




In analysis n4, we do not have the traffic load. We increased the resistance and stiffness of the soil, and we can observe how the black points have moved closer to the monitored values. In analyses 5 and 6, the road load is included, and we can observe how the black points diverge from the measured values but by increasing the stiffness, it moves closer to the measured values again (n6).



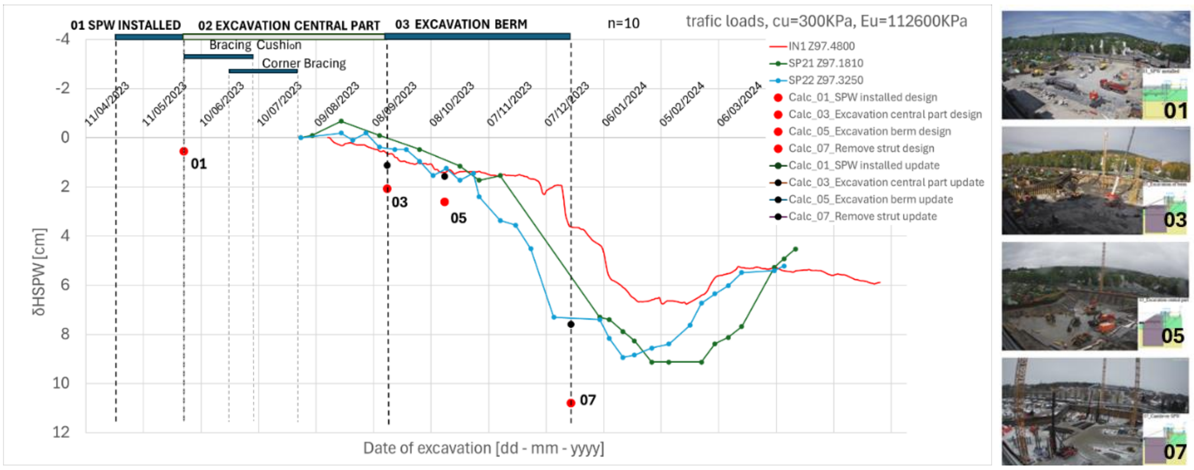




This back-analysis show that a significant higher strength and stiffness is required to come close to the measured displacements of the SPW (n8).




In analyses 7 and 8, the road load is null. The resistance of the treated soil has been increased, with the stiffness being at the design level in analysis 7 and further increased in analysis 8.

In analysis 9, the road load is added, stiffness of the treated soil is at the design parameter, and we see how the values diverge from the measured ones.




In analysis 10, the road load is added, stiffness of the treated soil is increased to nearly four times the design parameter, and we see how the calculated values move closer to the measured ones.

### 4.6.3 Comparison between measured and calculated displacement of Inclinometer IN1

The measured data from the inclinometer (IN1_red line) is compared with the calculated (Calc_black line) data from Plaxis during the excavation phases: 03 - central part excavation, 05 - excavation of the berm, and 07 - cantilever SPW. There are 10 analyses, and the parameters that are changed for the treated soil (purple) in the Plaxis model are reported in the table 4.13. In Figure 4.10, the position of the inclinometer IN1 in section is reported.

| | | | lime cement 40% | |
|---|---|---|---|---|
| n | bottom cut-off (m) | qroad(KN/m/m) | Cu (KPa) | Eu (KPa) |
| 1 | 60.00 | 19.00 | 90.00 | 33780.00 |
| 2 | 71.50 | 0.00 | 90.00 | 33780.00 |
| 3 | 71.50 | 0.00 | 150.00 | 33780.00 |
| 4 | 71.50 | 0.00 | 150.00 | 56300.00 |
| 5 | 71.50 | 19.00 | 150.00 | 33780.00 |
| 6 | 71.50 | 19.00 | 150.00 | 56300.00 |
| 7 | 71.50 | 0.00 | 300.00 | 33780.00 |
| 8 | 71.50 | 0.00 | 300.00 | 112600.00 |
| 9 | 71.50 | 19.00 | 300.00 | 33780.00 |
| 10 | 71.50 | 19.00 | 300.00 | 112600.00 |

Table 4.13-parameters analyses plaxis

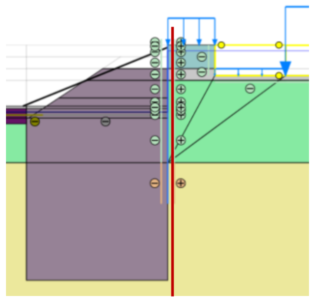
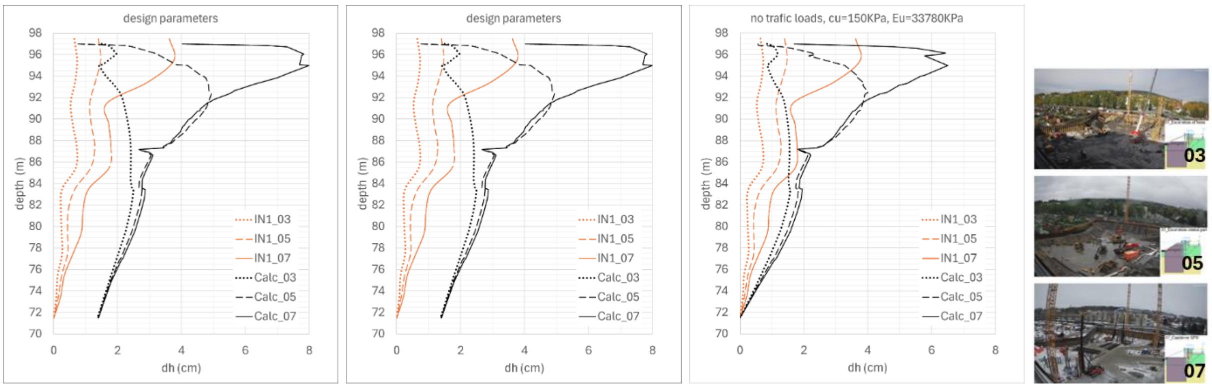
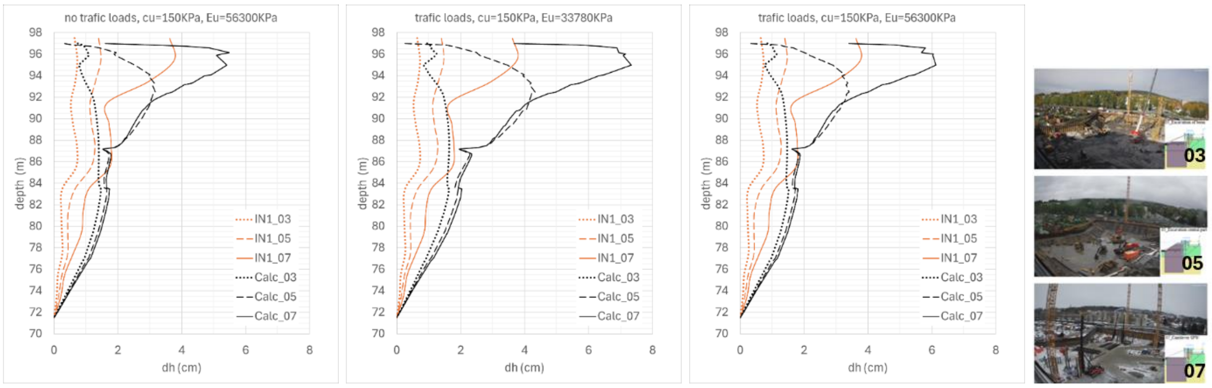
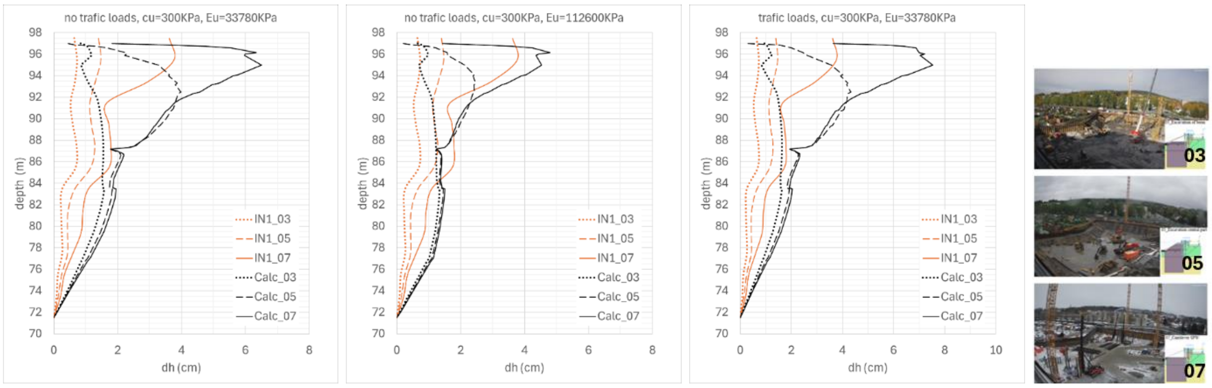


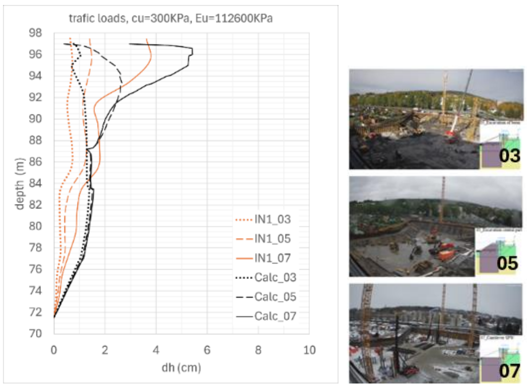
Fig 4.10-Inclinometer IN1 position (red line)





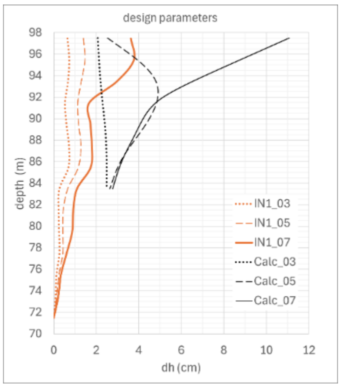



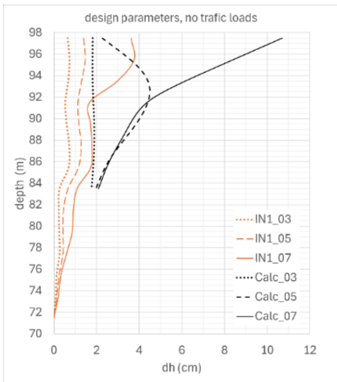

This back-analysis show that a significant higher strength and stiffness is required to come close to the measured displacements of inclinometer IN1 phase 07 (n8).

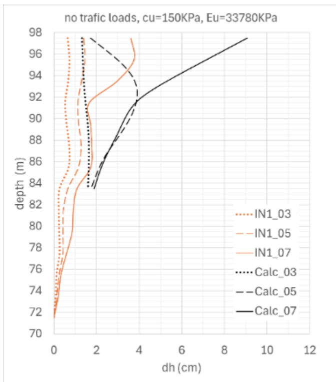



### 4.6.4 Comparison between measured and calculated displacement of Inclinometer IN1 and sheet pile walls

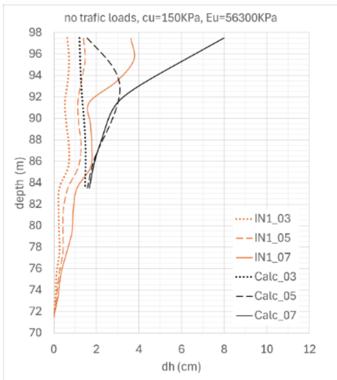
The measured values from the inclinometer (IN1_red line) are compared with the calculated values of the sheet pile walls (Calc_black line) from Plaxis analysis during the excavation phases: 03 - central part excavation, 05 - excavation of the berm, and 07 - cantilever SPW. There are 10 analyses, and the parameters that are changed for the treated soil (purple) in the Plaxis model are reported in the table 4.13

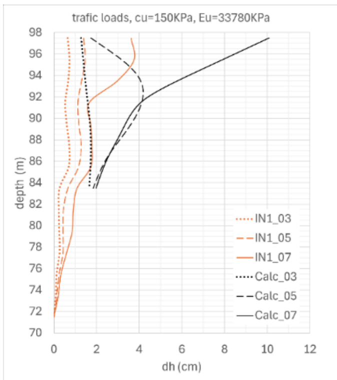


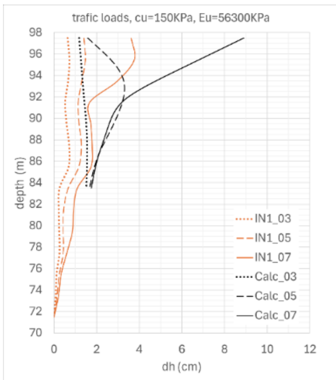



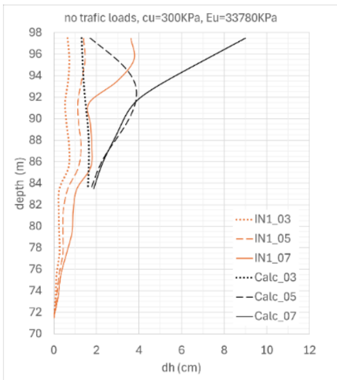


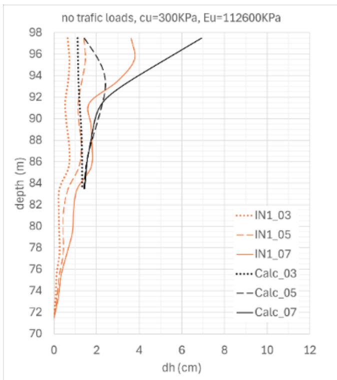


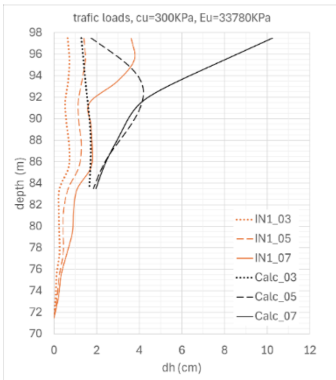





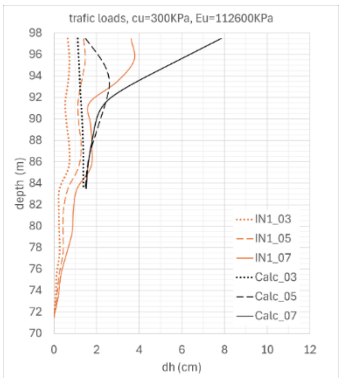


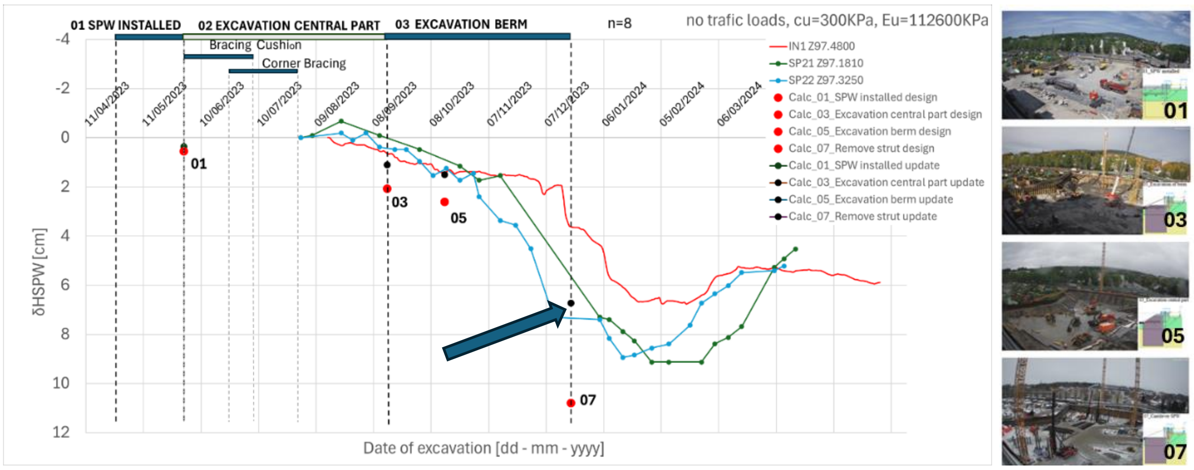





This back-analysis (n8) show that a significant higher strength and stiffness is required to come close to the measured displacements of inclinometer IN1 phase 07.




### 4.6.5 Discussion




The updated presentation and results seem to show that we are getting closer between the predicted and measured displacements. In the design analysis, an undrained shear strength (cu) of the treated soil equal to 90 kPa was used. This is the weighted average between the in-situ clay and the part that is improved, which has a design strength of 150 kPa. When modelling in 2D sections, NGI used this "smeared" strength in the soil clusters. The graphs (n=8) show that by increasing the undrained shear strength and stiffness of the treated soil and decreasing the road load, the displacements of the top of the sheet pile wall in the PLAXIS model are closer to the monitoring results.

Laboratory investigations on lime-cement treated soils samples from the site have not been conducted in this project. In recent years, lime has largely been replaced with CKD (Cement Kiln Dust), a type of waste from the cement industry. Results from previous research at NGI and experiences from other nearby projects show that clay can be stabilized with a relatively low amount of lime and cement, and that part of the lime can be replaced with a CKD byproduct. An increase in the amount of CKD is advantageous both for the project's economy and for reducing greenhouse gas emissions.

Moreover, research has shown that the inclusion of CKD not only improves soil stability and strength but also contributes to more sustainable management of industrial waste. The use of CKD reduces the need for virgin raw materials and decreases the overall environmental impact of the project. This approach also addresses growing environmental and regulatory concerns, promoting more eco-friendly and responsible construction practices.

The research project SUSI at NGI (Sustainable Soil Improvement) has shown that it is possible to reduce the amount of binder necessary to improve soil strength and deformation properties of sensitive clays. Results indicate that there is a lower limit to the amount of binder required. SUSI aims to define this lower limit and to quantify the environmental impact of this reduction in terms of CO2-emissions.

For a Ø600 mm LC-pile and the required rate of pulling the whip up the minimum amount of binder is 14 kg/m LC-pile.

The treated volume in plaxis is designed by the regulations in Statens Vegvesen handbook V221. Even though the LC-soil shows 400-600 kPa strength (FKPS and FOPS tests), the material is an average of the untreated soil volume and the stabilized soil volume.

Norwegian guideline apply limits and restrictions on the value that can be used. In this thesis work, parameter studies have been performed with increased average Cu from 90 kPa (design) up to 150 and 300 kPa, respectively and proportionally increase the stiffness.

Studies show that there is significant uncertainty in the calculation and the actual value to use. Uncertainties are mainly due to the actual application of the cement mixture along the entire treated pile and the application technique.

# 5 Conclusions and recommendation

The parameter study and comparison of the predicted performance of sheet pile walls (SPW) with monitoring data revealed significant benefits in soil treatment. Using higher strength and stiffness values in the treated soil, while ensuring compliance with safety regulations, future projects could achieve:

- *Reduction in Treatment Costs*
  - Less material usage thanks to higher strength.
  - Overall cost savings.
- *Increased Durability*
  - Soil more resistant to deterioration.
  - Reduced need for frequent maintenance.
- *Improved Structural Efficiency*
  - Support for higher loads with less deformation.
  - Enhanced structural safety.
- *Reduced Environmental Impact*
  - Use of sustainable materials that reduce $CO_2$ emissions.
  - Resource optimization, minimizing the use of virgin raw materials.
- *More Sustainable Treatment Practices*
  - Compliance with environmental regulations.
  - Use of recycled or waste materials.

These improvements not only ensure safety and regulatory compliance but also pave the way for more innovative, cost-effective, and eco-friendly soil treatment methodologies.

# 6 References


NGI (2022)
Campus Ullevål RIG - Forprosjekt
Datarapport - Grunnundersøkelser
Rapport 20210332-01-R

NGI (2022)
Campus Ullevål RIG - Detaljprosjekt
Prosjekteringsforutsetninger
Teknisk notat 20220608-01-TN

NGI (2022)
Campus Ullevål RIG - Detaljprosjekt
Tolkning av jordparametere
Teknisk notat 20220608-02-TN

NGI (2022)
Campus Ullevål RIG – Detaljprosjekt
Prosjektering av spunt og byggegrop
Notat 20220608-05-TN

NGI (2022)
Campus Ullevål RIG – Detaljprosjekt
Prosjektering av spunt og byggegrop
Notat 20220608-07-TN

NGI (2023)
Campus Ullevål RIG – Detaljprosjekt
Plan: Instrumentering for hovedbyggegrop
Tegning CU_G_13_XX_01

Norsk Geoteknisk Forening (2012)
Veiledning for grunnforsterkning med kalksementpeler

NTNU Geotechnical division (2017)
Geotechnics Field and Laboratory Investigations
Compendium for TBA4110

Regionale Verneombud (2022)
Veileder for grøftearbeid

Standard Norge (2020)
Eurokode 0: Grunnlag for prosjektering av konstruksjoner. NS-EN 1990:2002+A1:2005+NA:2020

Standard Norge (2020)
Eurokode 7: Geoteknisk prosjektering, Del 1: Allmenne regler. NS-EN 1997-1:2004+A1:2013+NA:2020

Karlsrud et al. (2005)
CPTU Correlations for Clays
International Conference on Soil Mechanics and Geotechnical Engineering, 16. Osaka 2005, Vol. 2, pp. 693-702

Paniaguaa et al. (2019)
CPTU correlations for Norwegian clays: an update
AIMS Geosciences, 5(2): 82–103.

SVV (2018) - Håndbok V220
Geoteknikk i vegbygging

SVV (2020) - Håndbok V221
Grunnforsterkning, fyllinger og skråninger

Grimstad, Gustav og Andresen, Lars (2010)
NGI-ADP: Anisotropic shear strength model for clay
Int. J. Numer. Anal. Meth. Geomech. 2012; 36:483–497

Wong, D.Y.-C., Sadasivan, V., Isaksson, J., Karlsson, A., & Dijkstra, J. (2024).
Trans-scale spatial variability of lime-cement mixed columns. Construction and Building Materials, 417, 135394

Larsson, S. (2021).
The Nordic dry deep mixing method: Best practices and lessons learned. Deep Mixing - An Online Conference, DFI Deep Foundation Institute. 30 p..

Hov, S., Berner, M. N., Rønning, S., & Bache, B. K. F. (2024).
Strength and Verticality of Nordic Dry Deep Mixing Columns—A Case Study in Norway.
5th International Conference on Geotechnics for Sustainable Infrastructure Development (pp. 1051-1066). Springer.

Horpibulsk, S., Rachan, R., Suddeepong, A., & Chinkulkinjwatt, A. (2011).
Strength Development in Cement Admixed Bangkok Clay: Laboratory and Field Investigations.
Soils and Foundations, 51(2), 239-251.

Hov, S., Berner, M. N., Rønning, S., & Bache, B. K. F. (2024).
Strength and Verticality of Nordic Dry Deep Mixing Columns—A Case Study in Norway.
5th International Conference on Geotechnics for Sustainable Infrastructure Development

Karlsrud, K (1986)
Performance Monitoring Of Deep Supported Excavations In Soft Clay.
4th International Geotechnical Seminar on Field Instrumentation and In-Situ Measurements (pp. 187-202)

Finno, R. et al. (1989)
Observed Performance of a Deep Excavation in Clay.
Journal of Geotechnical Engineering

Finno, R. J., Bryson, S., & Calvello, M. (2002)
Performance of a Stiff Support System in Soft Clay.
Journal of Geotechnical and Geoenvironmental Engineering

Finno, R. J., Kim, S., Lewis, J., & Van Winkle, N. (2018).
Observed Performance of a Sheet Pile-Supported Excavation in Chicago Clays.
Journal of Geotechnical and Geoenvironmental Engineering

Piciullo, L. et al. (2021)
Assessment of Building Damage Due to Excavation-Induced Displacements: The GIBV Method
Tunnelling and Underground Space Technology

Zhu, J., & Zhu, D. (2022).
Calculation of Building Settlement Induced by Tunneling Based on an Equivalent Beam Model.
Soil Mechanics and Foundation Engineering

Bentley (2022)
PLAXIS 2D Manual
Material Models