Detailed investigation of deep mixing in quick clay during construction

Master's thesis in the Master's Programme Infrastructure and Environmental Engineering

FANNY BACKLUND
ALVA SELLGREN

DEPARTMENT OF ARCHITECTURE AND CIVIL ENGINEERING
DEVISION OF GEOLOGY AND GEOTECHNICS

CHALMERS UNIVERSITY OF TECHNOLOGY
Master's thesis ACEX30
Gothenburg, Sweden 2026

MASTER'S THESIS ACEX30

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Supervisor: Daniel Tamlen, AWER Sverige AB
Examiner: Mats Karlsson, Chalmers

Examensarbete ACEX30
Institutionen för Arkitektur och Samhällsbyggnadsteknik
Chalmers Tekniska Högskola, 2026

Department of Architecture and Civil Engineering
Division of Geology and Geotechnics
Chalmers University of Technology
SE-412 96 Göteborg
Sweden
Telephone +46 31 772 1000

Department of Architecture and Civil Engineering
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Detailed investigation of lime-cement columns in quick clay during construction
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ALVA SELLGREN

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ABSTRACT

Numerous landslides associated with deep mixing installation in quick clay has been reported. Deep mixing is a ground improvement method commonly used to stabilise roads and railway embankments. However, during the deep mixing installation process, the method temporarily reduces the stability of the area before the soil improvement is achieved. This temporary reduction in stability is mainly caused by excess pore pressure in the permeable soil layers and by remoulding of the clay. Due to the high sensitivity of quick clay, remoulding can result in a significant loss of shear strength. The excess pore pressure is generated by the air pressure used to inject the binder during installation.

This thesis aims to investigate a safe framework to be able to perform deep mixing with a reduced risk of failure during construction. Numerical analyses and literature studies were conducted to investigate the effect of deep mixing during the construction phase and see how the trigger factors influence the stability of the slope, and identify which trigger factor that is the most critical.

The results indicate the importance of accurately characterising the permeability of the permeable soil layer, usually laying underneath the clay. The permeability is correlated to the dissipation rate of excess pore pressure, which could be considered when planning the installation procedure and the construction sequence. The analysis also indicated that the distance to the injection point within the friction layer had a limited influence on the pore pressure recovery time. It should be noted that the PLAXIS 2D analysis was performed with simplified geometry of the friction layer.

Furthermore, the results demonstrate a linear relationship between the coverage ratio of the lime-cement columns and the factor of safety. The coverage ratio therefore represents a design parameter regarding both the temporary reduction in stability caused by remoulding during construction and the long-term increase in stability achieved after completion.

Key words:
Geotechnics, Quick clay, Deep mixing, Landslide, Excess pore pressure, Remoulding, PLAXIS 2D, slope stability.

Detaljerad undersökning av djupstabilisering i kvicklera under byggfasen
Examensarbete inom masterprogrammet Infrastruktur och Miljö

FANNY BACKLUND

ALVA SELLGREN

Institutionen för arkitektur och samhällsbyggnadsteknik
Avdelningen för Geologi och Geoteknik
Chalmers tekniska högskola

SAMMANFATTNING

Ett flertal skred har rapporterats i samband med installation av djupstabilisering i kvicklera. Djupstabilisering är en markförstärkningsmetod som ofta används för att stabilisera väg- och järnvägsbankar. Under installationsskedet kan metoden dock medföra en tillfällig försämring av stabiliteten innan den avsedda förstärkningen har uppnåtts. Denna reduktion av stabiliteten orsakas främst av porövertryck i permeabla jordlager samt vid omrörning av kvickleran. På grund av kvicklerans höga sensitivitet kan omrörning leda till en betydande minskning av jordens skjuvhållfasthet. Porövertrycket uppkommer till följd av det lufttryck som används vid injektering av bindemedlet under installationen.

Syftet med detta examensarbete är att undersöka ett säkert ramverk för att möjliggöra djupstabilisering med minskad risk för brott under byggskedet. Numeriska analyser och litteraturstudier har genomförts för att studera effekterna av djupstabilisering under installationsfasen samt för att identifiera de faktorer som har störst betydelse för släntstabiliteten.

Resultaten visar att en noggrann karakterisering av permeabiliteten i de genomsläppliga jordlagren är av stor betydelse, de permeabla lagren är oftast lokaliserat under leran. Permeabiliteten styr dissipationstakten för porövertrycket och bör därför beaktas vid planering av installationsmönster och byggsekvens. Analyserna påvisar även att avståndet till injektionspunkten inom friktionslagret har begränsad påverkan på tiden för återhämtning av porttrycket. Det bör dock noteras att analyserna baseras på antaganden en förenklad geometri av friktionslagret.

Resultaten visar dessutom ett linjärt samband mellan täckningsgraden av kalkcementpelarna och säkerhetsfaktor. Täckningsgraden utgör därmed en dimensioneringsparameter gällande både den tillfälliga stabilitetsminskningen som orsakas av omrörning under byggskedet och den långsiktiga stabilitetsökning som uppnås efter färdigställandet.

Nyckelord: Geoteknik, Kvicklera, Djupstabilisering, Skred, Porövertryck, Omrörning, PLAXIS 2D, Släntstabilitet.

Contents

Acknowledgment

The thesis has been carried out by Awer Geoteknik and Chalmers University of Technology. Awer have provided the input data for the reference project, project framework and supervision. An extra thanks to Daniel Tamlen our supervisor at Awer Geoteknik, along with the colleges at Awer for interesting discussion, encouragement and guidance. We would also like to thank Nibben Peterzéns for an insightful interview about his personal practical experience with deep mixing. Additional thanks to Göran Sällfors for sharing his expertise and interest. We would also like to thank our examiner and supervisor Mats Karlsson at Chalmers for invaluable guidance.

Gothenburg, June 2026
Fanny Backlund & Alva Sellgren

Notations

Roman letters

c	Cohesion
c'	Effective cohesion
$C_{u,red}$	Reduced undrained shear strength
c_{ur}	Remoulded shear strength
E_{50}	Secant modulus
E_{oed}	Oedometer modulus
E_{ref}	Young's modulus
E_{ur}	Unloading / reloading modulus
G	Shear modulus
k	Permeability
K_0	Lateral earth pressure at rest
$K_{0,nc}$	Lateral earth pressure ratio in the NC-region
OCR	Over-consolidation Ratio
S_u	Undrained shear strength
$S_{u,DSS}/S_u$	Undrained shear strength for direct, active and passive zone
$S_{u,inc}$	Inclination in undrained shear strength by depth
$s_{u,ref}$	Reference undrained shear strength
St	Sensitivity
w_L	Liquid limit

Greek letters

Δu	Excess pore pressure
γ_f^c/γ_f^ϕ	Shear strength reduction factors
γ_{sat}	Saturated unit weight
γ_{unsat}	Unsaturated unit weight
n	Coverage ratio / Porosity
ν	Poisson's ratio
ϕ'/ϕ	(Effective) internal friction angle
σ'	Effective stress
σ_v, σ_h	Vertical and horizontal stress
τ	Shear stress
u_s	Air pressure

Acronyms

CPT	Cone penetration test
NGI	Norwegian Geotechnical Institute
PBL	Swedish planning and building act
RPM	Rotation per minute
SF	Safety factor
SGI	Swedish Geotechnical Institute
SGU	Geological survey of Sweden

1 Introduction

1.1 Background

Landslides are defined as the downslope movement of earth material under the influence of gravity [1]. They may result in widespread destruction that affects both individuals, infrastructure, and facilities vital to society [2]. According to Clague and Stead [1], the risk of landslides is expected to increase in this century due to an increase in global population, settlements, and the development of landslide-prone regions that were previously sparsely populated. Climate change further contributes to this increased risk, as rising temperatures accelerate glacier melting and shifting precipitation patterns, potentially leading to more frequent and intense triggering events.

In response to these challenges, various ground improvement methods are used to improve slope stability, one of which is deep mixing stabilisation. Deep mixing is an in situ mixing method in which different binders are mixed with the soil, one of the binder used is lime-cement. This method increases soil strength and permeability to mitigate the risk of slope failure [3]. However, the installation of lime-cement columns involves risks, as the injection of binder under air pressure can cause soil disturbance remoulding and increased pore water pressures, leading to temporarily reduced soil stability. These effects are particularly of concern in soil profiles containing quick clay, as its metastable particle structure collapses when disturbed, causing the shear strength to decrease to values close to zero and in certain terms can resemble the viscosity of water [4].

Numerous landslides have occurred in quick clay, causing extensive damage and severe consequences. There are also documented cases where landslides have been triggered during the installation of lime-cement columns. The combination of these two poses a significant risk, as disturbances during installation can initiate slope failures, with particularly severe consequences in deposits containing quick clay. Therefore, it is important to carefully control and monitor the installation process of lime-cement columns, especially in sensitive clay.

This thesis investigates, through analysis, calculation, and modelling, the installation process of lime-cement columns to identify critical factors and opportunities for improvement.

1.2 Aim

The aim is to find a safe framework to implement in deep mixing stabilisation in quick clay areas to reduce the risk of landslide during construction.

- How does soil remoulding caused by deep mixing installation affect the short-term stability of slopes with quick clay?
- How does the excess pore pressure generated by the air pressure used in injection influence the factor of safety? How does permeability affect the

recovery of excess pore pressure?

- What is the most critical factor for deep mixing in a slope during construction?
- Is it possible to identify a general critical threshold value for excess pore pressure?

1.3 Limitations

- The investigation is conducted during the construction phase, this is when the temporary reduction in stability occurs.
- Only one reference project will be used in the investigation.
- The analysis will be performed in a simplified FEM-model using PLAXIS 2D.
- The analysis will not include an analysis for mass displacement.
- The pore pressure is only investigated in the friction layer.

2
Theoretical introduction

2.1 Quick clay

Quick clays are a type of clay in which the particle skeleton collapses when disturbed, causing the shear strength to decrease to values close to zero. These disturbances may include initial landslides or mechanical remoulding. To identify quick clay, undisturbed soil samples are extracted and tested in a laboratory using fall cone test to determine the undrained shear strength and sensitivity [2]. A clay is classified as quick clay when the sensitivity exceeds 50 and the remoulded undrained shear strength c_{ur}, is less than 0.4 kPa [5].

Quick clay is found in areas characterised by marine deposits. During the last glaciation, the melting glaciers transported fine sediments through meltwater and the smallest particles were carried furthest and deposited on the seabed as marine clay [2]. These plate-shaped particles form an open 'house-of-cards' structure, with saline pore water filling the voids. The salt acts as an electrochemical bond, providing an attraction force between the edges and the surfaces of the clay particles. This force stabilises the skeleton despite its high water content. Over time, exposure to freshwater such as precipitation or groundwater causes the salt ions to leach out, weakening the soil structure and leading to the formation quick clay. When the sensitive structure is overloaded or disturbed, the particle skeleton collapses, causing the particles to transition to a liquid suspension as they float within their pore water [6].

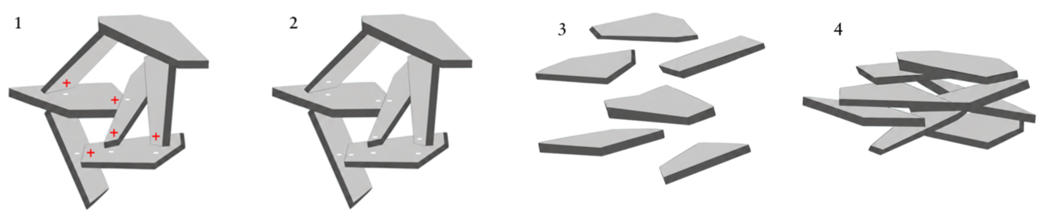

Figure 2.1: *Transformation of marine clay with salt-stabilised "house-of-cards" structure (1) to quick clay via salt leaching (2) leading to structural collapse when loading or disturbed (3) then resetting to a denser, stable grain structure after the collapse (4)*

2.1.1 Formation of quick clay

For quick clay to form, the salt in the pore water must be washed away through leaching. Several factors determine the likelihood and speed of this process.

Geological context

Quick clays are almost exclusively found in areas below the highest post-glacial shoreline, where sediments were originally deposited in salt water [7]. The presence of coarse-grained materials such as till, sand, or gravel beneath the clay has an impact, as these layers allow fresh water to flush through and leach the salt. Since leaching is a slow process, quick clay is more commonly found in thinner clay deposits or along the edges of deep valleys, where groundwater circulation is more efficient.

Hydrogeological factors

The movement of the groundwater is an important factor in the formation of quick clay. Artesian conditions, when the groundwater pressure acts in an upward direction, can accelerate salt leaching. The size of the catchment area and annual precipitation also affect the process as they determine the volume of water available to infiltrate the soil. Additionally, if the underlying bedrock has peaks, the groundwater outflow becomes more concentrated, leading to more intensive local leaching [7].

Chemical composition

The chemical properties of groundwater strongly affect clay sensitivity. The groundwater associated with the limestone bedrock is typically rich in calcium and magnesium, which contribute to stabilising the clay structure and preventing quick clay formation [7]. In contrast, soft water promotes high sensitivity. Although quick clay develops mainly from marine clays, it may also form in brackish or freshwater environments if it is in contact with organic soils, particularly peat. Organic materials change the ionic composition and act as dispersing agents, increasing the sensitivity of the clay.

2.2 Deep mixing

Deep mixing is a ground improvement method in which binders, typically lime and cement, are mechanically mixed into the soil. Lime-cement columns were introduced in Sweden in the min-1970's [8]. While lime had previously been utilised as binder for surface soil stabilisation, in the 1970's injection was introduced to a larger depth. The columns were assumed to interact with the soil and create greater strength in the soil as a composite material [3].

This process initiates chemical reactions with the water in the soil and improves the the permeability and strength of the soil [3]. The mixture creates a column that can be installed in different patterns and dimensions to adapt the construction to different challenges. The purpose of the columns is to distribute load to deeper parts of the strata with stiffer soil and thereby reduce deformations. Lime-cement columns are used mainly to stabilise road and rail pavements but can also be used for smaller bridges and other applications [8].

2.2.1 Mixing process

Deep mixing can be performed using the wet or dry method [8]. The dry method uses compressed air to inject a dry binder with lime-cement in situ. The binder mixes with the soil and reacts with the natural water in the soil. This method is most effective in soils with a high water content. In the wet method, the binders are mixed with water before being injected into the soil, which is more suitable for soils with a low water content. In some cases, additional reinforcement has been applied during the wet method to further increase stability. In Sweden, the dry method is the most commonly used.

Despite good mixing, the strength of a column can vary throughout its cross-section [8]. One probable reason for this is the spatial variation in soil characteristics. To reduce the risk of weak zones, design guidelines have been implemented for column dimensions and load limits. Mechanical mixing for lime-cement columns is done using rotating mixing tools equipped with paddles or helical blades.

The mixing process is essential for lime-cement columns to ensure an even distribution of binders in the soil [3]. The mixing process is illustrated in Figure 2.2 starting with a penetration of the mixing tool down to the required depth. The penetration causes remoulding and disaggregation of the soil, which changes the conditions. The second phase of the mixing process is to distribute the binder. This is done at the same time as the mixing tool is withdrawn and rotated, to create an even distribution of the binder throughout the column. The mixing provides optimal conditions for the chemical reactions to take place consistently, resulting in a more homogeneous column to avoid large agglomerates and accumulations of binder. The last step involves molecular diffusion; when lime reacts with water, calcium hydroxide is produced, which diffuses in the dissolved state and thus improves the degree of homogenisation. While cement reacts with water, it hardens and fills the pores in the soil particles and creates a complete column.

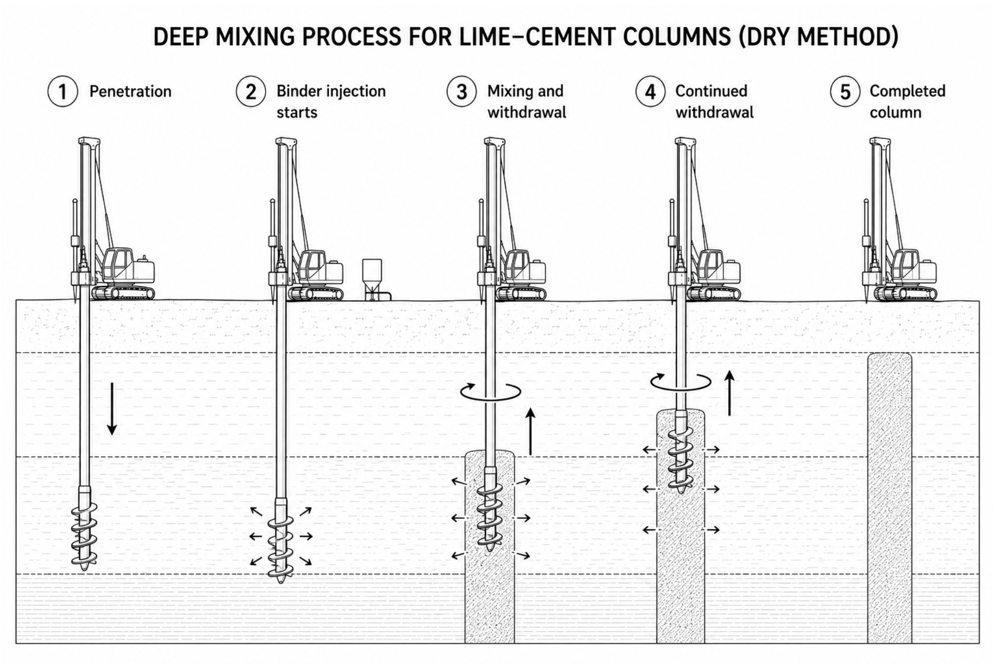

Figure 2.2: *Illustration of the dry deep mixing process for deep mixing columns (generated with AI).*

2.2.2 Execution

Ground improvement with lime-cement columns requires preparation of the ground surface, including the removal of vegetation and organic material [8]. The surface soil might need to be stripped, which means that the bearing soil, the dry crust, is removed and the machines work on relatively loose ground. Binder injection stops a bit before the surface, making the column tops looser and they might need to be excavated or replaced with compacted friction soil. An alternative to this is to use scraping or levelling combined with geotextiles. This method makes the geotextile bed a part of the upcoming construction and the machine works through it, which allows the columns to go all the way up to the working bed. There are also seasonal constraints, stabilisation should be avoided at the coldest months of the year, as low temperatures delay the hardening of the column tops which can lead to reduced bearing capacity.

2.2.3 Specifications and Layout

The design and dimensions of lime-cement columns are based on results from geotechnical investigations and laboratory tests of binder-stabilised soil. The columns built in Sweden usually have a diameter of 500, 600 or 800 mm , with 600 mm being the most common. The depth of the column can be up to 25 metres depending on the constraints of the drilling rig and the project requirements [8].

The installation pattern for the columns is adapted for the different challenges. When the goal is to reduce settlements and shorten consolidation time, the columns are installed in a square- or triangle pattern with a certain diameter and spacing. The spacing between the columns underneath the pavements needs to be sufficiently distanced from each other so that the arching effect has an effect. To increase the stability even more, geotextile can be used as tensile reinforcement between the pavement and the columns. To resist horizontal loads, columns are typically constructed in patterns such as slabs, blocks, or grid structures.

2.2.4 Considerations

When injecting the binder, the air pressure contributes to soil displacement. Moreover, the actual remoulding of clay on the outside of the mixing tool can increase both the total earth pressure and the pore water pressure, leading to reduced soil stability, especially in soil profiles containing quick clay [4].

Pore pressure

The pore water pressure is critical when it comes to landslides, which makes this critical to monitor during the installation of columns [8]. Therefore, it is important to closely monitor the pore pressure build-up by using piezometers, fixed measuring points, and potentially inclinometers. Monitoring is especially important if the safety factor prior to reinforcement is less than 1.5 [8]. The injection must be stopped if pore pressure build up is detected to allow the pressure to return to equilibrium.

One way to minimise pressure build-up is to perform the installation in stages, e.g. every second or third column section or even sparse spacing, so that the pore pressure has time to equalise and the soil gains strength between installation phases [8]. If pore pressure build-up occurs, it could take days or even months for it to recover to the initial pressure [4].

In Juvik et al. [9], the results are presented from a project including ground improvement with lime-cement columns in quick and sensitive clay. In this project, the increase in pore pressure was a concern since the area already had poor slope stability prior to the ground reinforcement and the soil profile consisted of quick clay. To prevent an increase in the pore pressure, which would further decrease the stability, measures were carried out by conducting installation on every fifth panel. Furthermore, piezometers were installed to monitor the pore pressure with established critical values. If these values were to be exceeded, the construction work would stop immediately.

Equipment

The quality and homogeneity of the column depend highly on the mixing tool, their movements and speed. Injection blades have a significant impact on the shear strength of the final column. A tool with more blades is generally more effective than one with fewer blades [4].

The rotational speed (RPM) typically ranges between 100-200 rotations/minute, and the speed affects the quality of the column. High speeds improve mixing efficiency, but there are risks for slinging effects or cavity formation, especially on columns with smaller diameters. Conversely, lower speeds may result in insufficient mixing. The withdrawal speed of the mixing tool should be around 10-25 mm/rotation, depending on the surrounding soil. With a lower withdrawal rate, mixing becomes more efficient, leading to more homogeneous columns [8]. Rotation and withdrawal speed affect the time it takes to install one column, making it an important factor in terms of project time and price [4].

2.3 Landslides

The fundamental mechanism governing slope failure is when the driving shear strength exceeds the resting shear strength of the soil or rock mass. The triggers for this to occur are processes that either reduce the available shear strength or increase the acting stress or reduce stress in supporting parts of the shear plane, often by elevating the pore water pressures and soil remoulding. These processes may be initiated by natural factors, such as erosion and increased precipitation, or by human activity, e.g., excavation, loading, and other forms of ground disturbance [1]. The outcome and geometry of a landslide are directly related to the soil parameters, the thickness of each soil layer, and the topography of the area [6].

2.3.1 Classification of different types of landslides

Landslides can be classified into different categories based on their failure mechanisms, and the different categories can be combined and find in the same landslide [1]. The most common failure mechanisms in quick clay are retrogressive landslides [6].

An initial landslide can be triggered by localised overloads [6]. This initial failure can then propagate and develop into an extensive landslide over a larger area, especially in quick clay. The failure mechanism for a progressive landslide starts with an initial landslide and progresses backward as the soil beneath vanishes. This mechanism continues until stable ground is reached or no more sensitive clay remains.

Progressive landslides can be further categorised based on their mode of propagation. Bernander [10] explains the three types as downhill progressive, uphill progressive and laterally progressive. A downhill progressive landslides start at the upper part of the slope and propagate downward. These often lead to large areas of flat ground being displaced below the foot of the slope. Uphill progressive slides start in the lower part of the slope and propagate upward, often resulting in a characteristic saw-tooth pattern on the ground surface. Laterally progressive is when the instability spreads sideways along the elevation contours of the slope.

Rotational landslides can occur in all types of clay [6]. A characteristic feature of a rotational landslide is that a coherent mass of the slope moves as a single block while simultaneously undergoing rotational movement. The movement forms a circular movement. The top of the slope moves downward while the footing of the slope experiences heave, similar to a seesaw.

2.3.2 Landslides in quick clay

Due to the unique structure of marine and quick clay, as well as their response to loading, is the reason why the failure differs significantly from the failure in bedrock or sand. The occurrence of quick clay per se does not cause landslides, but the sensitivity and quickness cause the failure to spread rapidly in large areas due to the characteristics of clay [2]. In quick clay, the failure is often progressive, which means that the slip surface grows gradually. When a specific point in the clay deforms, the peak strength decreases to residual strength due to its plate-like particle structure, which makes the particles reorient themselves parallel to the slip surface when the salt in the quick clay is leached out weakening the electrochemical bonds between the particles. Once one part of the surface weakens, the load is transferred to the adjacent point, which then also fails. This creates a chain reaction in which the failure propagates through the clay layer, which is not always visible on the surface [1].

2.4 Previous failures

While numerous landslides have occurred in recent years, two specific cases are of particular interest to this project, the landslide in Lökeberg, Sweden in 2019, and Nesvatnet, Norway in 2025. In both cases, the landslides were triggered during the installation of lime-cement columns in soil profile containing sensitive quick clay. These cases highlight the significant risks associated with deep mixing in slopes with sensitive clay identified in the soil profile.

2.4.1 Nesvatnet, Norway

In August 2025, a significant landslide occurred at Nesvatnet, located north of Trondheim in Norway. The event took place during an active construction phase in which ground improvement was being implemented via lime-cement columns. According to an investigation by Bane NOR [11], the landslide was triggered by the installation process with two primary failure mechanisms:

- Pore pressure increases in a permeable layer beneath the quick clay due to the air pressure generated when the binding material is injected.
- Soil disturbances in the column itself and the surrounding soil due to remoulding of the clay creating a weakened zone.

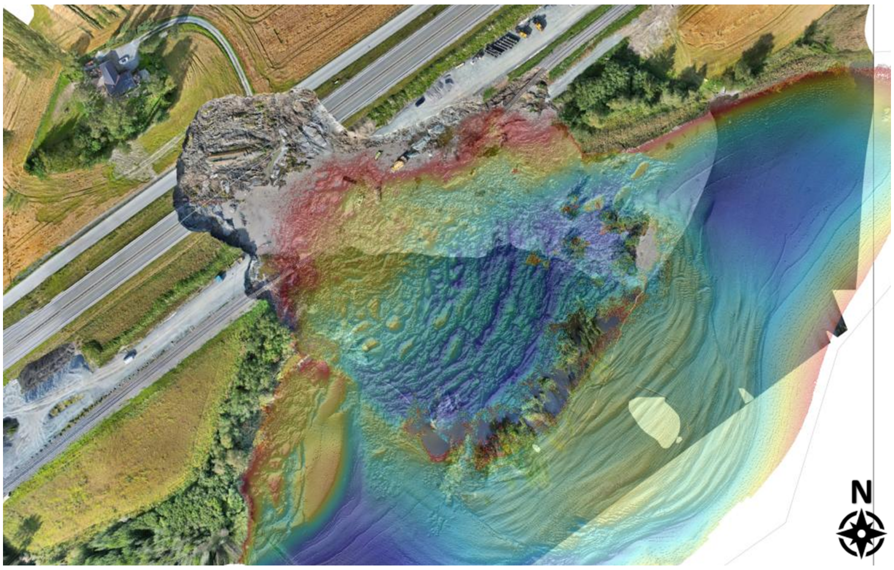

Figure 2.3: *Drone photography of the landslide scar. [11]*

The geotechnical profile in the area is characterised by a dry crust of 1-3 meters over a clay deposit with a thickness of 2 to 18 meters. Quick clay has been confirmed in some boreholes in various depths, typically 2 to 4 meters below the surface, with some occurrences immediately beneath the dry crust. In addition, there are intercalated permeable layers within the clay profile. After the clay deposit there is a thin layer of moraine above the bedrock.

Landslide sequence

In between the installation of each column, the pore pressure is checked and approved before the installation of the next column can begin. During the day of the landslide, everything went according to set procedure and they were about to start installing the next column when an alarm in the mast of the drill rig was triggered, the mast was set out of plumb. The landslide was described by a witness in the area, who explained that the landslide started with a big block around the newly installed columns was set loose, triggering the initial failure.

Pore pressure monitoring and stability analysis

The investigation further highlighted the importance of pore pressure measurements with piezometers. During a test field, a pore pressure increase limit was set; installation would immediately stop if the limit of 20 kPa was passed, but the piezometer would react when the pore pressure was 15 to 18 kPa. The test field showed a pore pressure increase of 30 - 40 kPa which recovered back to normal after one to two days. The report also highlights the correlation between the distance of the piezometer and the measured pore pressure increase. During the field test, a piezometer located approximately 15 meter from the deep mixing installation, showed no noticeable increase in pore pressure compared to a piezometer located 2 meter from the installation. Highlighting that the piezometer tip was located in the clay layer, not in the permeable layer where the pore pressure builds up.

The investigation done by Bane NOR includes a stability calculation where the factor of safety based on total stress resulted in 1.26. The load from the drill rig is set to 25 kPa, which is less than the actual load but is representative for a 2D-calculation, excluding the three dimensional effects. The calculations resulted in a critical excess pore pressure of $\Delta u = 51.5$ kPa. The source of this excess pore pressure in the permeable layer is considered to be generated by the air pressure used to distribute the binder during the injection. The air pressure injected in Nesvatnet was approximately $u_s = 700$ kPa during installation, which indicates that the actual excess pore pressure may have been significantly higher than the critical value that theoretically would trigger slope failure.

Key findings

A key conclusion from the event was that both the distance between the piezometer and the installed column and the specific depth of the sensor tip are vital. The piezometer tip should be positioned within the permeable layers, where the pressure accumulation is most critical [11]. Even though the piezometer tip would be located in the permeable layer, there is a risk that the pore pressure increase would occur too rapidly to take action.

2.4.2 Lökeberg, Sweden

On 13 November, 2019, a landslide occurred in Lökeberg in Sweden [12]. New constructions were planned on the estate and the municipality's building permit process required a geotechnical investigation to assess slope stability before a permit could be granted. The geotechnical investigation showed that the stability of the area was insufficient. Despite this, lime-cement columns were used for ground improvement. The soil profile was characterised by fill material with a varying thickness consisting of humus, gravel, sand, and clay overlaying a clay layer with a thickness of 9 to 13 meters with the upper three meters characterised as a dry crust clay. After the landslide, quick clay was identified in some sections of the area. Boulders and large stones are present both at the ground surface and within the layers. The clay layer is underlain by a layer of friction soil, which was assessed as sensitive to erosion and prone to liquefaction when in a saturated state.

Figure 2.4: *Photograph over the area of the landslide [12].*

2.5 Stability assessment and practical experience

2.5.1 PLAXIS 2D

PLAXIS 2D is a finite element programme developed to analyse deformation, stability, and groundwater flow in two dimensions for geotechnical engineering [13]. The software provides geotechnical engineers with a tool to simulate initial ground conditions and evaluate different ground improvement or stabilisation measures for a given project. The mechanical behaviour of soils can be modelled using different material models.

2.5.1.1 NGI-ADP material model

NGI-ADP is a material model for PLAXIS where the anisotropic conditions in the soil is taken into consideration [14]. The models shear strength is defined by means of S_u values for active, passive and direct shear strength state. Allowing the model to take the anisotropic conditions into account, reflecting the different soil behaviours in compression shear or extension, as seen in Figure 2.5. The material model NGI-ADP is most suitable for short-term calculation of undrained soil during construction load, excavation and slope stability.

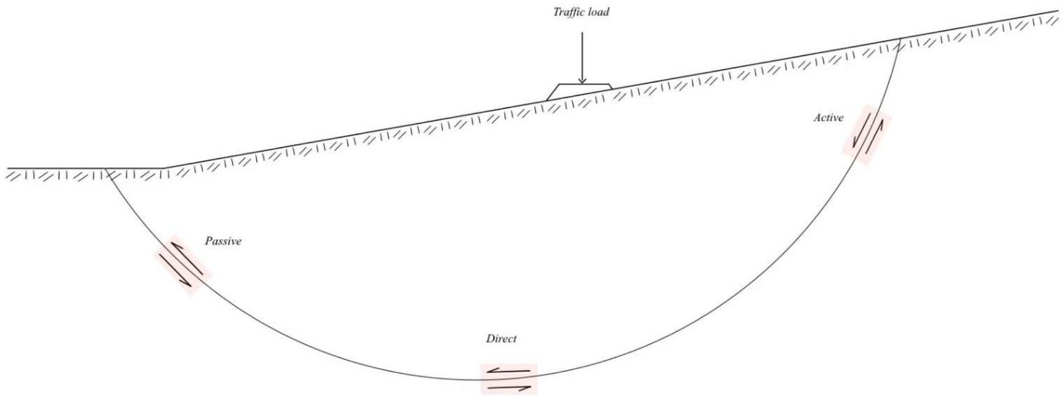

Figure 2.5: *Anisotropic conditions in a slope.*

2.5.2 Slope stability

The purpose of a slope stability investigation is to determine whether the safety of the slope is adequate for the planned exploitation [15]. The analyses are performed for both existing and planned construction, to ensure sufficient safety. The Swedish planning and building act (PBL) have requirements regarding the slope safety in order to reduce the risk of landslide and slope failure.

Stability of a slope depends on the equilibrium between driving and resisting force within the soil mass. Over time, the slope stability may change due to factors such as climate change, geometric changes caused by erosion, increased pore pressure, or soil disturbance resulting from human activity such as deep mixing.

Stabilisation measures

If the safety requirements for a slope are not fulfilled, stabilisation measures must be implemented. Commonly used measures in slope stabilisation include stress decrease by excavation, slope flattering, support embankment and culverting of watercourse or erosion protection and also lime-cement stabilisation [15]. The purpose of these or other measures is to maintain or increase the equilibrium of the forces acting within the slope.

To account for a temporary reduction in safety caused by deep mixing or other soil disturbances, a percentage improvement can be performed. The percentage improvement reduces the probability of slope failure and is determined with consideration of the coefficient of variation, which reflects the level of uncertainty in the input data based on the extent and quality of the geotechnical investigations [16].

Practical experience

In an interview with Nibben Peterzéns [17], technical manager at Dmixab, who has extensive experience in the installation of lime-cement columns and explained his practical experience of the installation process. According to Peterzéns the shear strength of the soil generally recovers to its initial condition within 12 hours after disturbance. In quick clay, the recovery is faster and typically occurs within 3-4 hours.

Peterzéns further explained that the most critical factor during the installation process is the increase of pressure within the soil. The pressure is governed by the injection of the binder material and contributes to excess pore pressure. To reduce this pressure, a square-shaped drill rod can be used to create air pockets during rotation, allowing excess pressure to dissipate upwards. Another possible solution could be to reduce the flushing pressure when reaching friction layers in the soil, thereby limiting the development of excess pressure. However, Peterzéns explained that adjusting the pressure during installation is time-consuming in relation to the time it takes to install a column and requires large compressors to manage the pressure reduction.

3
Reference project

To analyse the effects of deep mixing installation in a quick clay area, the reference project "Etapp 3 - Aröd" was used. In this project, lime-cement columns were considered as a possible ground improvement method but were ultimately rejected due to the risk of triggering a landslide.

Aröd is a coastal area in the northern part of the Kungälv Municipality. The area contains valuable natural and cultural environments, with certain parts designated as areas of national interest. In the comprehensive municipal plan, Aröd is identified as a redevelopment area with potential for urban densification. However, this requires the implementation of municipal water and wastewater systems, as mandated by the County Administrative Board.

The area in Aröd, where the new water and wastewater infrastructure is planned, is geotechnically sensitive. The stability of the slope is currently marginal and quick clay has been confirmed at several investigation points. One proposed stabilisation method was the use of lime-cement columns, as they require minimal space. However, this method was considered too uncertain due to concerns that the installation process itself could trigger a landslide, given the sensitive quick clay and the existing low stability of the slope. Currently, there are limited technical guidelines regarding the installation of lime-cement columns in such sensitive conditions.

The project area includes the entire Aröd coast, with a critical section identified in the northern part, marked in Figure 3.1. The map on the left displays the surface characteristics mapped by the Geological Survey of Sweden (SGU), showing that the area consists of postglacial sand and sandy till. The map on the right illustrates the estimated soil depth, which ranges from zero to ten meters and increases progressively towards the water.

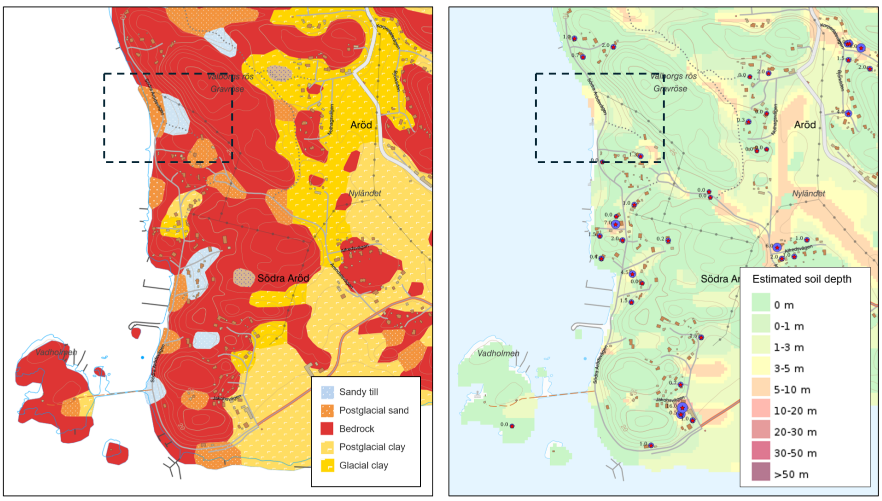

Figure 3.1: *Overview of surface characteristics and soil depth in the Aröd area.*

Previous soil sampling and geotechnical investigations had been conducted in the area and the resulting data were used for this study. The boreholes located along the critical slope are shown in Figure 3.2. The critical area is characterised by a slope descending toward the water, as illustrated in the elevation profile, the plan view and the sectional view in Figure 3.3. The data used to evaluate shear strength and density are compiled in Appendix B. The section view exclusively presents the relevant CPT evaluations and laboratory testing results from these earlier investigations.

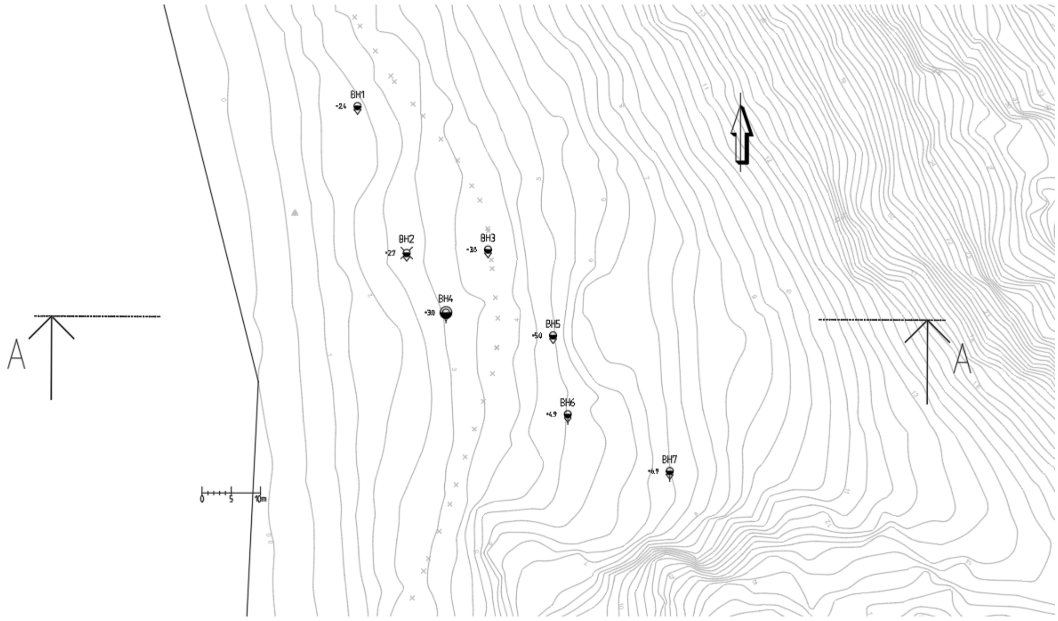

Figure 3.2: *Overview of one of the critical investigation site.*

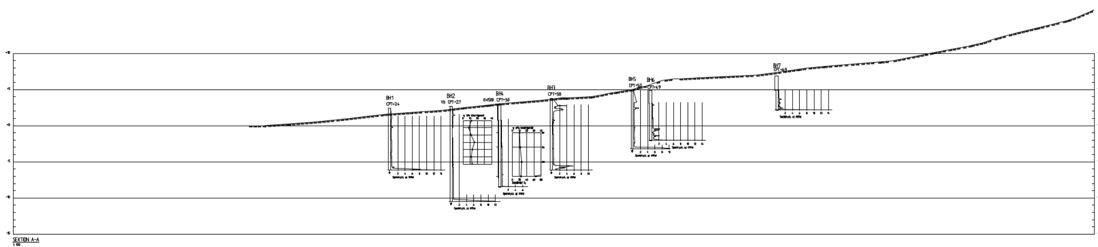

Figure 3.3: *Elevation of soil profile of the critical slope*

The geometry of the investigated slope has been simplified to facilitate numerical modelling. It consists of a thin layer of sand overlaying a four meter thick layer of clay. Beneath the clay layer another layer of clay is defined with increased shear strength with depth. Below the clay deposits, a thin friction layer of sandy till is included on top of the bedrock shown in Figure 3.4.

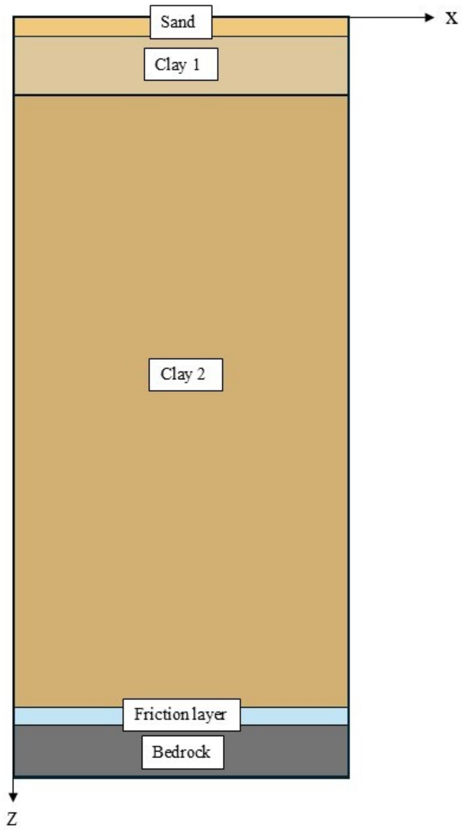

Figure 3.4: *Overview of the simplified soil profile.*

Laboratory results from one of the boreholes in the study area show that the clay meets the criteria for quick clay, with a sensitivity values greater than 50 and remoulded shear strength below 0.4 kPa. The values for borehole 4 is presented in Table 3.1.

Table 3.1: *Sensitivity S_t and remoulded shear strength c_{ur} at different depths for BH4.*

Depth [m]	**Sensitivity** S_t [-]	c_{ur} [kPa]
4	157	<0.06
6	176	<0.06
8	212	<0.06
10	65	0.19

Although site-specific data were used for slope geometry and soil parameters, significant simplifications have been made to both, which should be noted when interpreting the results.

4 Numerical analysis

A numerical analysis were made using the Finite Element method (FEM) PLAXIS 2D to establish a representative geometry and set of parameters. To simulate the installation of lime-cement columns and evaluate its effect, the finite element software PLAXIS 2D were used to analyse soil behaviour.

4.1 Soil Parameters

The soil parameters are presented in Table 4.1. For the clay, these values are derived from combination of in-situ cone penetration tests and laboratory analyses based on undisturbed samples, including fall cone tests and direct shear tests. The values for the pavement, sand and frictional soil are based on characteristic values from guidelines of Swedish Transport Administration [18], as well as the Young's Modulus for the clay that are calculated through Equation 4.1.

$$E'_{\mathrm{ref}} = 250 \cdot S_u \tag{4.1}$$

Table 4.1: *Summary of input data relevant for soil layers.*

Soil layer	Material parameter	Value	Unit
Pavement*	Unit weight, γ_{unsat}	21	kN/m^3
	Sat. Unit weight, γ_{sat}	21	kN/m^3
	Friction angle, ϕ'	45	$^\circ$
	Young's modulus, E'_{ref}	50	MPa
Sand*	Unit weight, γ_{unsat}	20	kN/m^3
	Sat. Unit weight, γ_{sat}	18	kN/m^3
	Friction angle, ϕ'	33	$^\circ$
	Young's modulus, E'_{ref}	5	MPa
Clay 1	Unit weight, γ_{unsat}	16	kN/m^3
	Sat. Unit weight, γ_{sat}	16	kN/m^3
	Shear strength, $s_{u,ref}$	12	kN/m^2
	Young's modulus, E'_{ref}	3	MPa
Clay 2	Unit weight, γ_{unsat}	15	kN/m^3
	Sat. Unit weight, γ_{sat}	15	kN/m^3
	Shear strength, $s_{u,ref}$	12.5	kN/m^2
	Shear strength w. depth, $s_{u,inc}$	1.25	kN/m^2/m
	Young's modulus, E'_{ref}	3.125	MPa
Frictional soil*	Unit weight, γ_{unsat}	19	kN/m^3
	Sat. Unit weight, γ_{sat}	21	kN/m^3
	Friction angle, ϕ'	35	$^\circ$
	Young's modulus, E'_{ref}	30	MPa

* *Parameters are characteristic values based on Swedish Transport Administration [18]*

4.2 Initial Conditions

For the model to represent the actual in-situ stress state, a specific initialisation procedure was used. The initial phase was performed using the linear-elastic perfectly-plastic material model Mohr-Coulomb with gravity loading as the calculation type. This approach was chosen to establish the current horizontal stresses and the initial stress state prior to further analysis. When using gravity loading in combination with Mohr-Coulomb, the coefficient of laterial earth pressure at rest, K_0, depends strongly on Poisson's ratio, v [13]. Therefore, K_0 was calculated using Equations 4.2 and 4.3 and Poisson's ratio by Equation 4.4, as specified by the Swedish Transport Administration [18]. The input parameters for the material model Mohr Coulomb is presented in Table 4.2.

$$K_{0,nc} = 0.31 + 0.71 \cdot (w_L - 0.2) \tag{4.2}$$

$$K_0 = K_{0,nc} \cdot OCR^{0.55} \tag{4.3}$$

$$v = \frac{K_0}{1 + K_0} \tag{4.4}$$

Table 4.2: *Summary of parameters for earth pressure coefficients when using material model Mohr-Coulomb.*

Soil layer	w_L[%]	$K_{0,nc}$	OCR	K_0	ν
Clay 1	0.70	0.67	1.0	0.67	0.40
Clay 2	0.85	0.77	1.0	0.77	0.44

4.3 NGI-ADP

To account for the anisotropy behaviour of the clay, the material model NGI-ADP was used, defining different strength parameters for active, direct simple shear, and passive stress states. The input parameters were calculated using Equations 4.5, 4.6, and 4.7 and the relation $s_{u,DSS}/s_u^A$ to calculate $s_{u,ref}^A$. The anisotropy factors $s_{u,DSS}/s_u^A$ and s_u^P/s_u^A is set to 0.63 and 0.35, respectively, as the report Thakur et al. [19] recommends for clays with low plasticity index. The input parameters used in PLAXIS for the clay layers are shown in Table 4.3. The geometry of the used model is presented in Figure 4.1

$$E'_{ref} = E_{50} \tag{4.5}$$

$$E_{ur} = 3 \cdot E_{50} \tag{4.6}$$

$$G = \frac{E}{2(1+v)} \tag{4.7}$$

Table 4.3: *Summary of calculated shear strength parameters and elastic properties for clay layer input in the NGI-ADP material model.*

Soil layer	$s_{u,DSS}/s_u^A$	$s_{u,ref}^A$	s_u^P/s_u^A	$s_{u,inc}^A$	ν_{ur}	G_{ur}/s_u^A
Clay 1	0.63[19]	19.05	0.35[19]	-	0.495	158
Clay 2	0.63[19]	19.84	0.35[19]	2	0.495	158

Note: $s_{u,ref}$ and $s_{u,inc}$ are given in kN/m^2 and kN/m^2/m respectively.

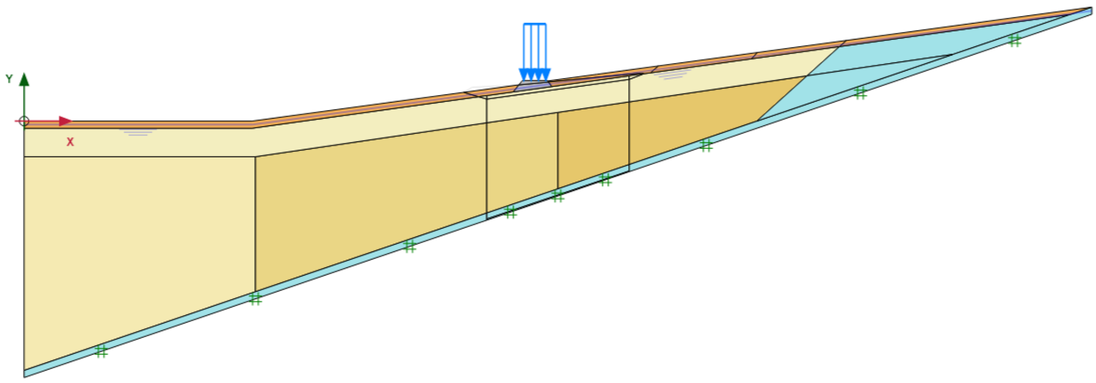

Figure 4.1: *PLAXIS 2D model before any measures.*

The failure plastic strains γ_f^C, γ_f^E and γ_f^{DSS} represent the shear strain at failure for triaxial compression, triaxial extension and direct simple shear, respectively. These were set to the values in Table 4.4 after consulting with supervisor.

Table 4.4: *Summary of shear strain at failure for the clay layers.*

Soil layer	γ_f^C [%]	γ_f^E [%]	γ_f^{DSS} [%]
Clay 1	2.0	4.0	2.45
Clay 2	2.0	4.0	2.45

4.4 Percentage improvement

To account for the temporary reduction in stability caused by the deep mixing, a percentage improvement in the slope was applied using excavation and berm [15]. The improvement was calibrated to achieve a sufficiently high factor of safety prior to installation, ensuring that the slope maintained the required stability even during the installation phase.

According to the report SGI [15], the required safety factor depends on the level of uncertainty associated with the input data. In the reference project used in the analysis, all field investigation and laboratory testing methods were used, with the exception of triaxial testing. Therefore the input data are considered highly reliable, which justifies increasing the safety factor from 1.24 to 1.32, an increase of 7%. The model presented in Figure 4.2 shows the excavation and berm needed to achieve the percentage improvement. A line load of 25 kPa was applied on the driving side of the slope to represent the drilling rig. The load corresponds to the estimated ground pressure beneath the rig and is induced to account for three-dimensional effects that cannot be directly represented in the two-dimensional model.

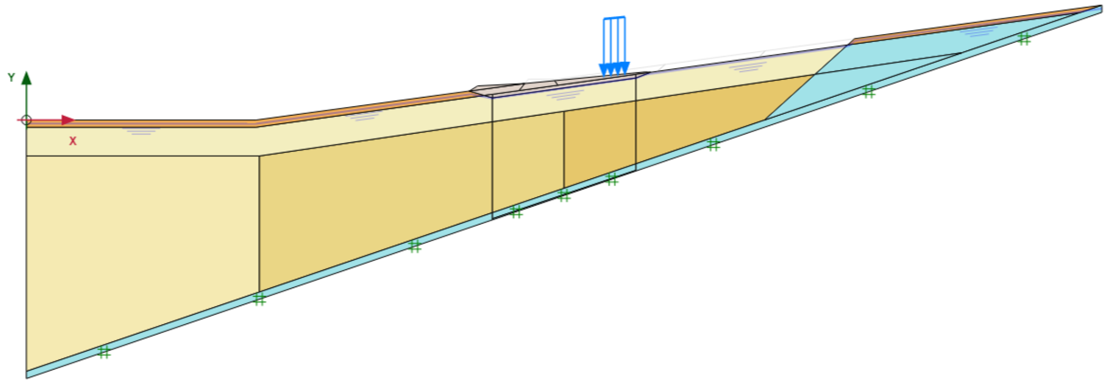

Figure 4.2: *PLAXIS 2D model after percentage improvement.*

4.5 Installation of lime-cement columns

When the initial conditions had been established, the installation order of the lime-cement columns was simulated. Firstly, the effects of soil disturbance caused by the injection of binders and remoulding in the soil, which leads to a reduced strength in the soil. For quick clay, the remoulding shear strength was conservatively assumed to be close to 0 kPa. To represent the disturbed zone surrounding the installed columns, a zone of reduced strength was introduced. The size of the reduced zone were formed based on the calculations done for Aröd, presented in the geotechnical report with recommendations on measures for the slope [20] and is presented in 4.3. The measure recommended had a demand of a column coverage ratio (η) of 25% in the stabilised zone. The equivalent shear strength of the zone was reduced by 25%, reflecting the proportion of remoulding soil within the installation area, and calculated using Equation 4.8, where the whole treated zone simulate that all the columns is installed at once.

$$C_{u,\text{red}} = \left(1 - \frac{\eta}{100}\right) \cdot C_{u,\text{clay}} \tag{4.8}$$

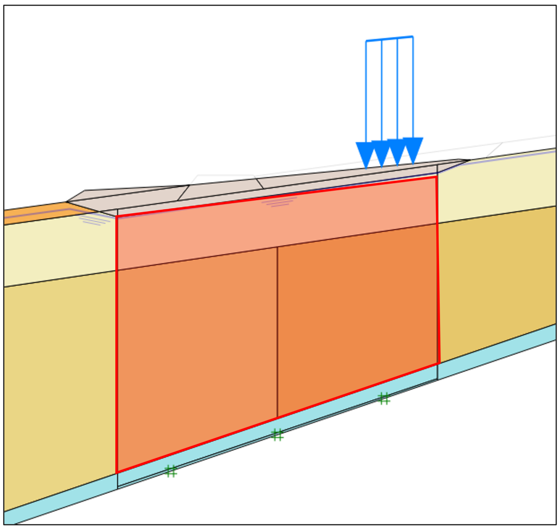

Figure 4.3: *The disturbed zone from deep mixing.*

In order to further analyse the effect of strength reduction caused from the disturbance, the analysis was performed with multiple different coverage ratios to find the critical limit.

4.6 Increase of pore pressure

Once the factor of safety had been improved to 1.32, the effects of the deep mixing installation was analysed. To simulate the pressure generated during injection, an increase in pore pressure was applied in the underlying friction layer. The affected area was assumed to be within the deep mixing zone presented in Figure 4.4. The analysis was performed for a range of excess pore pressure values in order to identify the critical pore pressure level for the investigated slope.

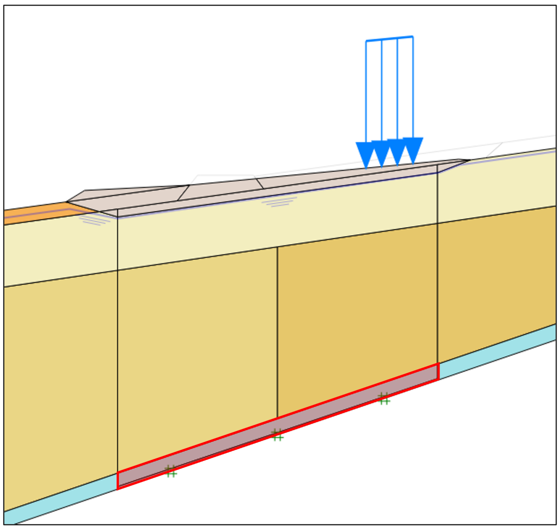

Figure 4.4: *The zone where the pore pressure increase is applied.*

To analyse the dissipation time of pore pressure generated during the installation of a single column, a numerical simulation of the injection process was performed using a new PLAXIS 2D model shown in Figure 4.5. The analysis was conducted using a *Fully coupled flow-deformation analysis*, where an excess pore pressure was applied for a period of five minutes to represent the injection phase. The subsequent dissipation of the excess pore pressure was then monitored to determine the time required for the pore pressure to recover back to the initial conditions. To focus specifically on the zone containing the lime-cement column, the geometry was simplified into a 30-meter-wide horizontal profile instead of a 150-meter-wide inclined slope. All soil parameters and layers were kept intact. With this model, a groundwater flow boundary was introduced in the centre to represent the pressure from the binder injection. Finally, to evaluate the influence of the friction layers permeability on the recovery time, the analysis was repeated on a range of different permeability values.

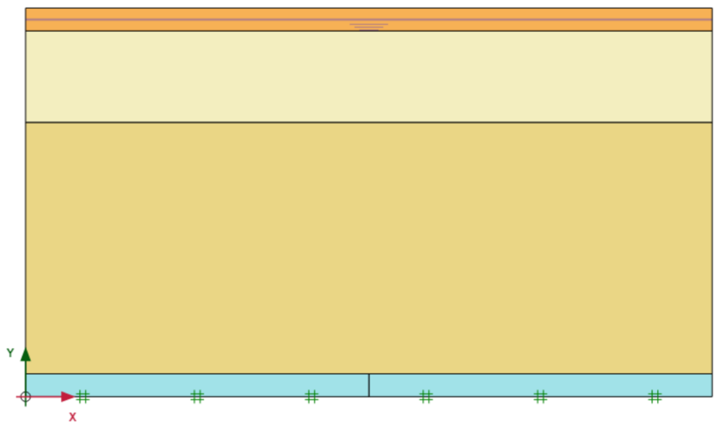

Figure 4.5: *Simplified PLAXIS 2D model to investigate dissipation time for a single column.*

5 Results

In the following section the results from the numerical analyses will be presented. The results will present the relationship between excess pore pressure, reduction in strength caused by the soil disturbance, and slope stability.

5.1 Reduced strength

Results from the analysis of the correlation between coverage ratio of deep mixing and safety factor is presented in Figure 5.1. The graph includes the target limit, presenting the factor of safety prior the implementation of percentage improvement. The results indicates that the critical coverage ratio is 22% for the investigated slope. The failure limit, SF=1 is reached when the coverage ratio is approximately 55%. The coverage ratio represents a scenario in which all lime-cement columns are installed at once, resulting in disturbance of the entire treated zone. The graph also indicates a linear relationship between the reduction of safety and increase of coverage ratio.

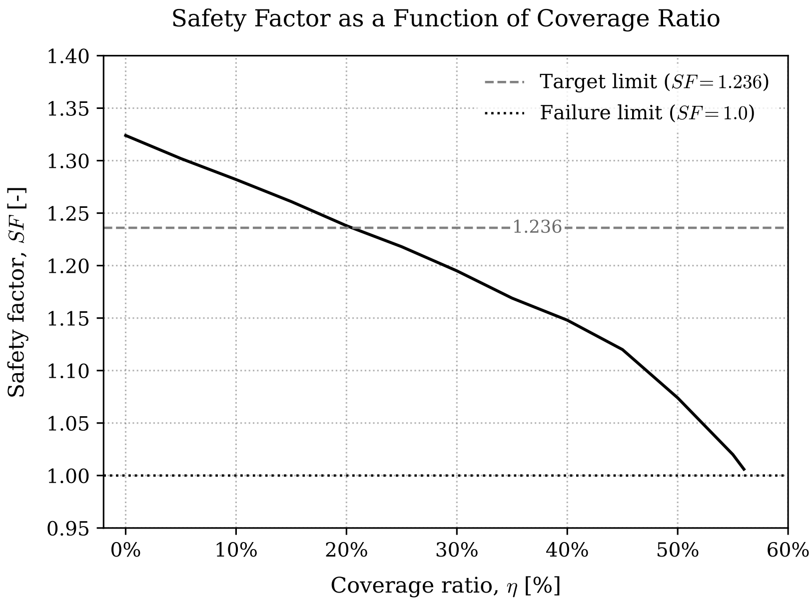

Figure 5.1: *Change of safety factor with an increase in the coverage ratio with market target and failure limits.*

The percentage reduction of the safety factor in relation to the coverage ratio is illustrated in Figure 5.2. The results correlate the percentage reduction in shear strength with the percentage reduction in safety factor, meaning a coverage ratio of 15% representing a reduction of 15% of the shear strength in the deep-mixing zone. The target limit safety factor of 1.236 results in a 7% reduction in the safety factor, which is around 22% coverage ratio.

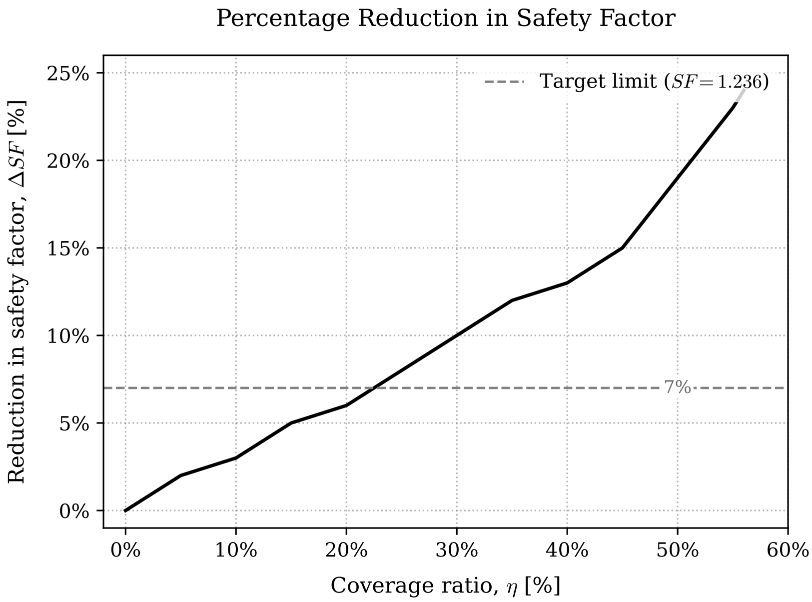

Figure 5.2: *Percentage relation between the safety factor and coverage ratio.*

5.2 Excess pore water pressure

The correlation between excess pore pressure and the factor of safety is presented in Figure 5.3. The target limit represents the factor of safety prior the implementation of percentage improvement. Based on the analysis, a critical excess pore pressure of 50 kPa was identified assuming a uniform excess pore pressure distribution throughout the entire assumed influence area. The results further indicates that the excess pore pressure up to 30 kPa do not have a significant impact on the safety factor for the investigated slope. The analysis shows that the failure limit is reached at approximately 75 kPa, when SF=1.

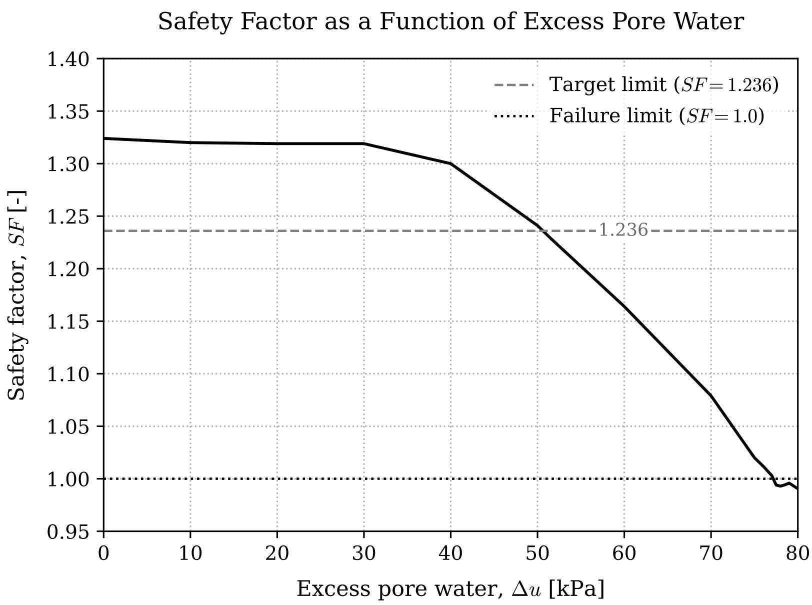

Figure 5.3: *Change of safety factor with an increase in pore pressure, with marked target and failure limit.*

The dissipation of excess pore pressure over time is shown in Figure 5.4. The graph presents results from a monitoring point located 10 m from the injection point, with a constant injection pressure of 50 kPa applied in all cases. The results show a clear influence of soil permeability on both the build-up and dissipation of excess pore pressure. Higher permeabilities results in a more rapid increase in excess pore pressure immediately after the blowout, followed by a faster dissipation. For example, the case with k=1 m/day has a sharp increase in excess pore pressure, but also dissipates more rapidly.

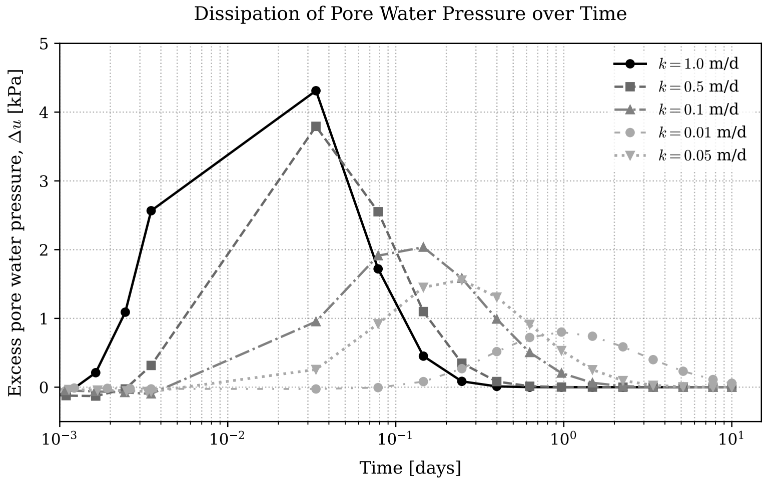

Figure 5.4: *The dissipation time for different permeabilities for a monitoring point 10 m from the injection point.*

With a permeability of k=1 m/day, the excess pore pressure dissipated and returned to the initial pore pressure after approximately 6 hours, regardless the level of excess pore pressure, presented in Figure 5.5. The graph illustrates the development of excess pore pressure generated by the injection at a monitoring point with a distance of 0, 5 and 10 meters from the injection point. The analysis indicates that the distance from the injection point does not have a significant effect on the recovery time for the pore pressure.

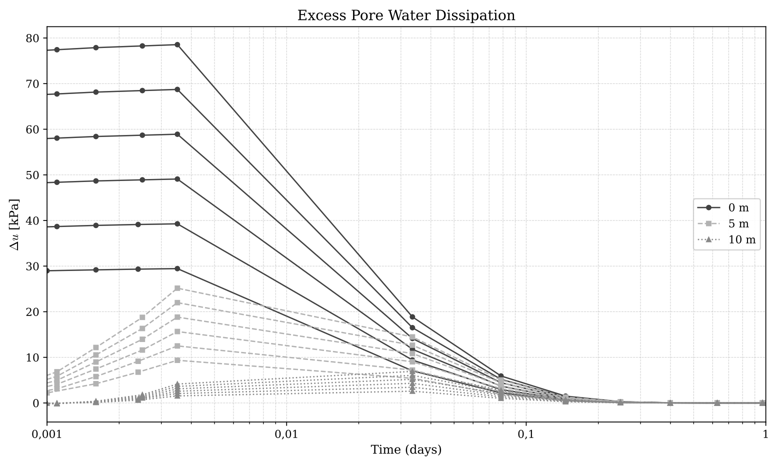

Figure 5.5: *The Dissipation of excess pore pressure over time with different pressure for 0, 5 and 10 m from the injection point.*

Since the analysis only looked at the installation of a single column, whereas several columns are typically installed during a working day, a simplified calculation was performed to investigate how excess pore pressure may accumulate over time during the installation of multiple columns. In the calculation, a new column was assumed to be installed every 15 minutes, and the pressure build up was assumed to last for 5 minutes, consistent with the previous analyses. The results are presented in Figure 5.6 and show the development of the excess pore pressure at a point located 10 m from the injection point. It should be noted that the calculation is based on the pressure response generated by a single column, with the same pressure history applied repeatedly. The presented calculation should be regarded as a simplified estimate of the potential accumulation of excess pore pressure during the installation process.

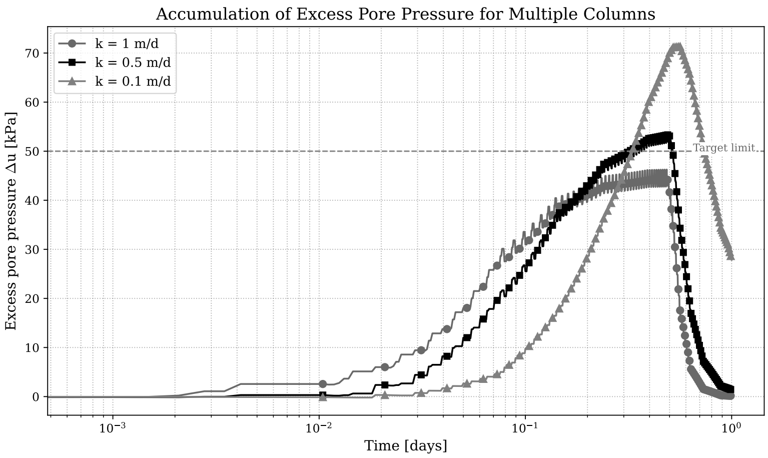

Figure 5.6: *The accumulation of excess pore pressure when installing multiple columns every 15th minute.*

6 Discussion

The discussion is based on literature studies and numerical calculations.

6.1 Disturbed zone

The analysis was performed in PLAXIS 2D and is based on a two-dimensional representation of the problem. As a result, the disturbed zone extends infinitely into the slope, which does not accurately represent the actual field conditions. The actual deep mixed area is 20 x 20 m in plan view. In a two-dimensional calculation, the three-dimensional effects that may influence slope stability are neglected. To account for these spatial effects and obtain a more realistic representation of the installation process, a three-dimensional analysis would need to be performed. The analysis also assume that all the lime cement columns is installed at the same time, with no hardening which is unrealistic.

The result indicated that the coverage ratio has a linear relationship with the safety factor. The coverage ratio directly influences the reduced temporary stability and also the reached improvement in slope stability, it represents a key design parameter to ensure that the target factor of safety is satisfied.

6.2 Pore pressure

The numerical analysis showed that the slope safety factor became critical when the excess pore pressures exceeded 50 kPa. This is similar to the findings from the investigation of the Nesvatnet landslide, where the critical excess pore pressure was estimated to be 51.5 kPa. For this analysis, the safety factor reached 1 when the excess pore pressure exceeded approximately 70 kPa. This corresponds well with the effective vertical stress at 10 m depth, which was 82 kPa. As excess pore pressure approaches the effective stress, the effective stress is reduced towards zero, resulting in a significant loss of shear strength and slope stability, causing failure.

However, in this analysis, several assumptions were made. The entire friction layer beneath the deep-mixing zone was assumed to experience the same excess pore pressure simultaneously, which is unlikely to represent the actual spatial distribution of the pore pressure in the field. Furthermore, it was assumed that the increase in pore pressure occurred only within the frictional soil layer, while the overlaying clay layer was considered impermeable and therefore unaffected by the pressure increase.

The analysis showed that although the magnitude of excess pore pressures strongly influences the factor of safety and the likelihood of slope failure, it has little influence on the dissipation time. Instead, the dissipation process was found to be highly dependent on soil permeability, with substantial differences in dissipation time between the investigated permeability values.

It should also be noted that this analysis only considered the pore pressure generated by the installation of a single deep-mixing column. In practice, multiple columns are installed within a short period of time, which can lead to an accumulation of excess pore pressure if sufficient dissipation does not occur between installations. Which was calculated by hand, and showed the correlation with permeability, installation sequence, recovery time and also the peak of the excess pore pressure. Consequently, the development of pore pressure is governed not only by permeability and time, but also by the installation sequence and the available drainage paths that control how and in which directions the water can dissipate.

6.3 Combination and comparison

The combination of soil disturbance in quick clay and an increase in pore pressure within the underlaying friction layer has the potential to trigger slope failure. In the numerical analysis, the shear strength within the disturbed zone was reduced in proportion to the planned coverage ratio of lime-cement columns. This represents a conservative simplification, as the hardening and strength gain provided by the installed columns were not considered during the analysis.

The analysis indicated a critical coverage ratio of approximately 55% at which the slope fails. Such scenario is considered unrealistic in practice, since the strength gain from the installed columns would begin to develop during construction. In comparison, the critical excess pore pressure leading to failure was estimated to be approximately 75 kPa. This level of excess pore pressure appears to be more realistic, considering that air pressures in range 500 - 700 kPa is are injected during the installation process.

These findings indicates that the excess pore pressure represents a more critical factor for triggering a landslide.

7 Conclusion

The aim of this thesis was to find a safe framework to implement in deep mixing stabilisation in quick clay areas to reduce the risk of landslide during construction. The safe framework included finding critical guidelines and identifying the most critical factors of deep mixing in a slope. Through a literature study and numerical analysis, the impact of soil disturbance and excess pore pressure was investigated.

The disturbed zone from remoulding contributes to a reduction in slope stability. However, the excess pore pressure appears to have a greater impact on slope stability. This finding is consistent with the conclusion drawn from the investigation of the landslide in Nesvatnet.

The results highlights the importance of monitoring and investigating the permeability and controlling the excess pore pressure in the friction layers. The permeability has a significant influence on the dissipation rate and therefore also affects the build-up excess pore pressure during the construction phase. Since the excess pore pressure is essential for the slope stability and an accurate assessment of both the permeability and geometry of permeable layers is crucial for deep mixing in sensitive clay. Furthermore the analyses indicated a critical value for the excess pore pressure, when the factor of safety became critical. Although the exact threshold is site specific, it demonstrates the importance of defining a project-specific critical pore pressure. This finding emphasises the importance of monitoring the pore pressure during construction and taking the dissipation time into consideration. The dissipation rate should be taken into account during the sequence plan of installation.

Key recommendations

Based on the findings, the following recommendations are proposed for a safe framework for deep mixing in quick clay:

- Identify the geometry and extent of the friction layer in the soil profile. Characterise the permeability, it controls how fast the pore pressure recovers.
- Determine a site-specific critical threshold value at which all installation activities are suspended, and operations remain stopped until the pore pressure has fully recovered.
- Make sure that the piezometers is installed in the permeable layer and monitor pore pressure continuously during construction.

7.1 Further studies

Several key elements has been noticed during this study which may be of value for further studies.

- Perform three-dimensional numerical analyses to investigate the combined effect of excess pore pressure and soil disturbance. Taking the effect of three-dimensional effects into consideration.
- Carry out additional field measurement using piezometers and pressure monitoring systems to further improve the understanding of pore pressure generation and actual pressure injected during installation.
- Investigate the influence of installing multiple columns and effect of accumulation of pore pressure within a limited time period using three-dimensional conditions.
- In a further analysis mass displacement should be performed. Evaluating the influence on pore pressure development, soil behaviour and slope stability.
- Validate the proposed framework using additional case studies with varying soil condition and slope geometry.

8
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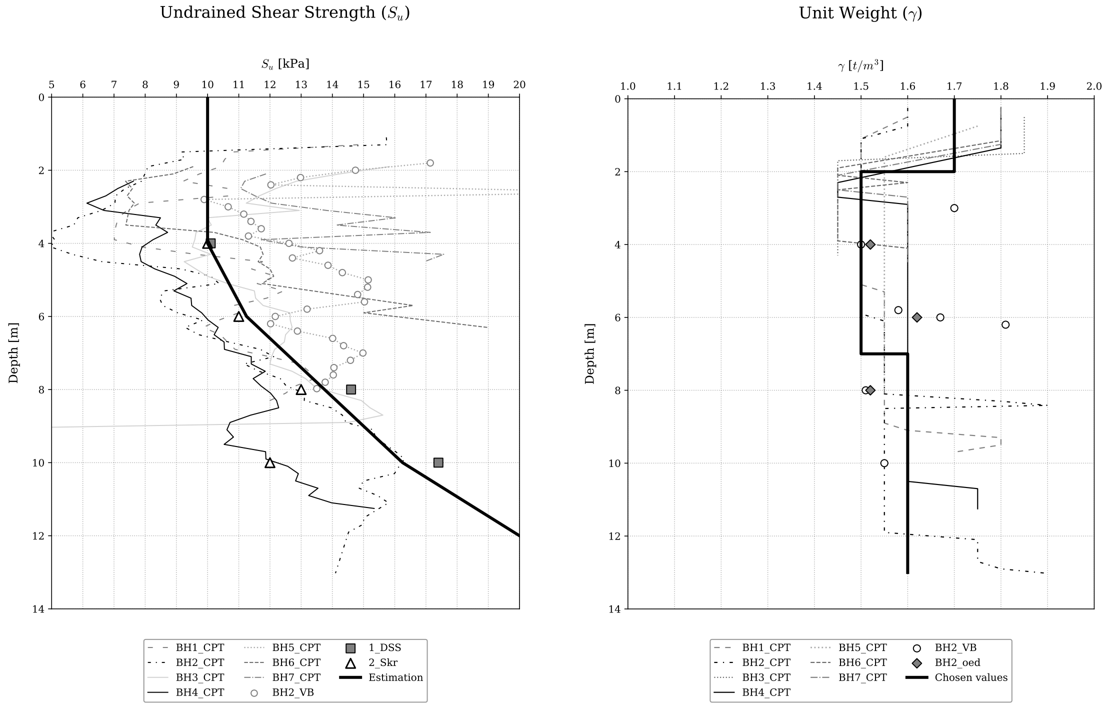
Appendix A



(a) *Undrained shear strength (S_u) profile.* **(b)** *Unit weight (γ) profile.*

Figure 8.1: *Compilation of geotechnical parameters from CPT sounding and laboratory testing, showing interpreted design profiles.*

Appendix B

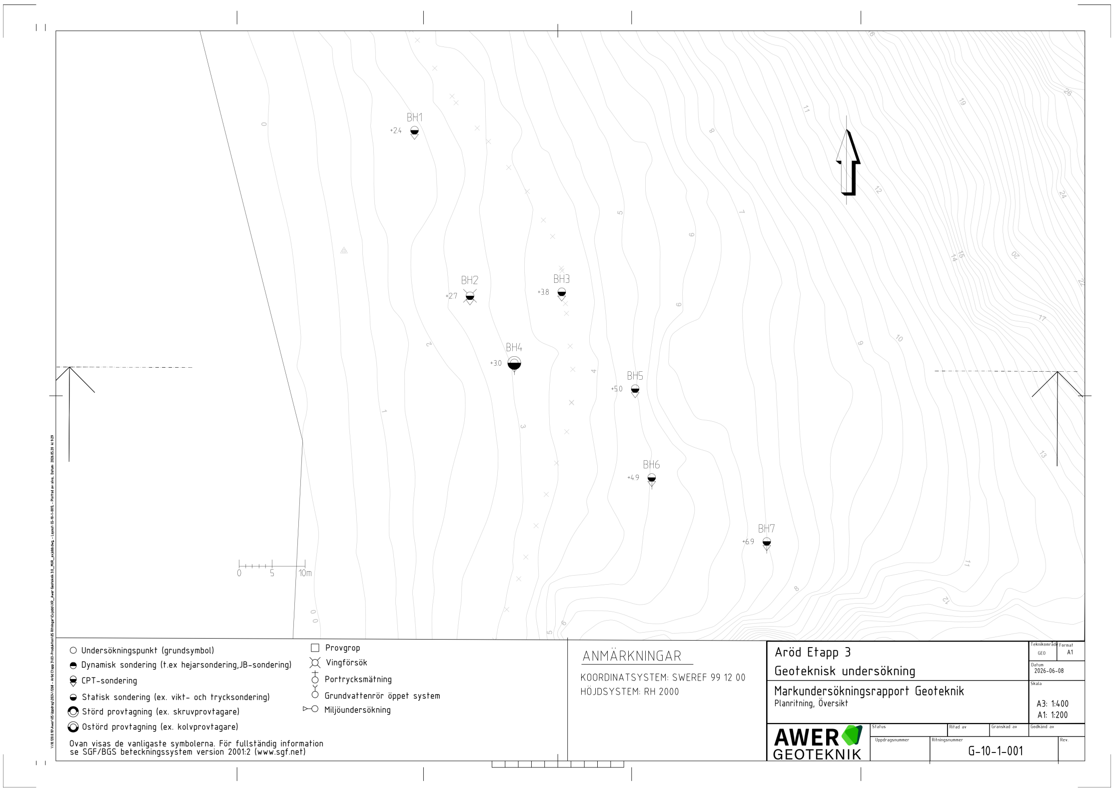


Figure 8.2: *Overview of investigation site*

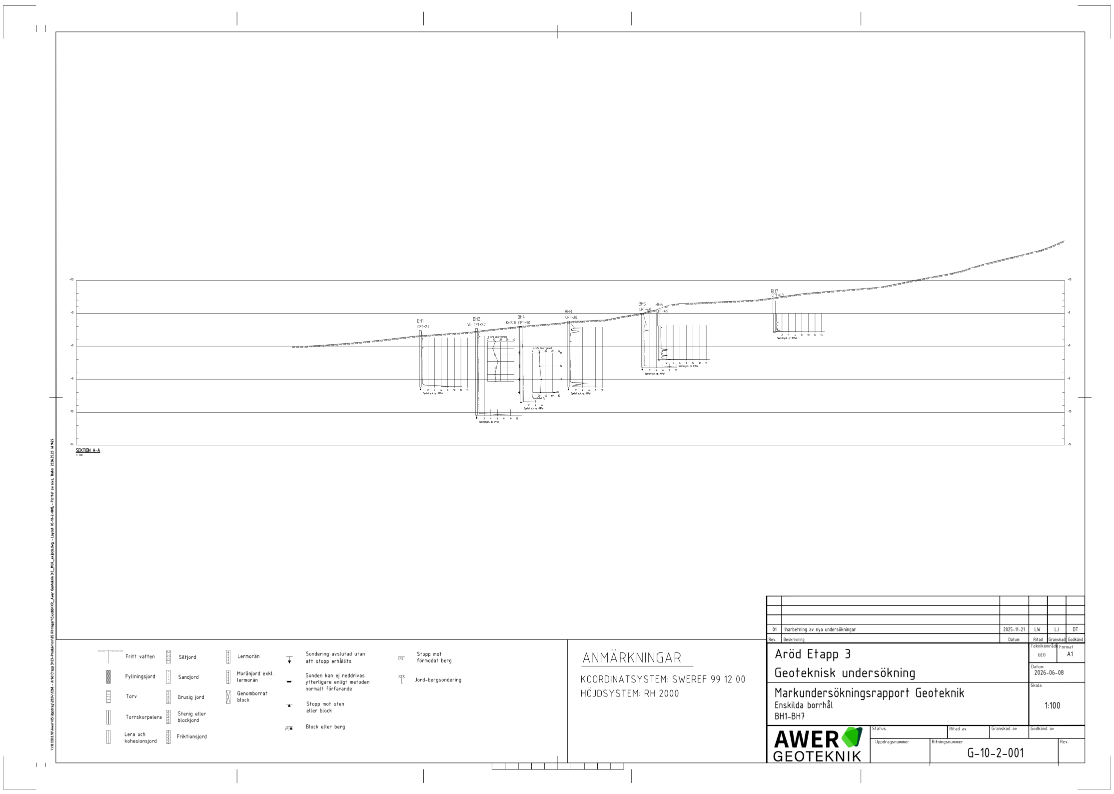


Figure 8.3: *Critical section including relevant CPT evaluation and laboratory testing*

DEPARTMENT OF ARCHITECTURE AND
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