



Advantages and disadvantages of deep soil mixing versus stone columns in soft soil

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CHALMERS UNIVERSITY OF TECHNOLOGY
Gothenburg, Sweden 2024

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Cover: An illustration of a unit cell model in Plaxis

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Abstract

Urbanization has led to a scarcity of suitable land for new infrastructure, resulting in constructions being built on unsuitable soft soil. To counter this, deep foundations and ground improvement techniques are used to enhance the stiffness and strength of the soil and mitigate risks. There is also a growing emphasis on sustainable construction to reduce the carbon footprint. The aim of this thesis was to investigate the differences between deep soil mixing and stone columns to understand the advantages and disadvantages with the methods. To accomplish this aim both a literature review and numerical analyses were conducted.

The literature review focused on giving relevant background to the methods. The review covers an introduction to ground improvement followed by more detailed chapters about deep soil mixing and stone columns, including general information, different applications and the installation process as well as some specific details related to each method. The numerical analyses were conducted in Plaxis 2D using a unit cell approach. The analyses were carried out for end-bearing columns with parameters from two distinct locations, Östra berg and Väg 160. The aim of the numerical analyses was to evaluate the differences between the methods, as well as the impact of different aspects including ground conditions, embankment height and cc-spacing.

The results of the numerical analyses showed that deep soil mixing is slightly more effective for reducing final settlements. However, considering other aspects such as rate of consolidation, stone columns are more favourable. The high permeability in the stone columns accelerates the settlements leading to increased settlements during construction and decreased post-construction settlements. In addition to the numerical analyses described above a sensitivity analysis was conducted. The sensitivity analysis showed that the parameters that the model is most sensitive to are permeability and preconsolidation pressure, thus OCR or POP. The results from the literature study showed that the main advantage with stone columns are the smaller environmental impact compared to deep soil mixing, which is also strengthened with the simplified carbon calculation made as part of this project. The main advantage with deep soil mixing is the extensive knowledge about the method and its successful application in many projects in Sweden.

Keywords: Deep soil mixing, Stone columns, Soft soil creep, Numerical analysis, Ground improvement, Embankment, Unit cell

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Nomenclature

Abbreviations

CRS	Constant Rate of Strain
DSM	Deep Soil Mixing
DSM	Deep soil mixing
HS	Hardening Soil
LC	Lime/Cement
MC	Mohr-Coulomb
NC	Normal Consolidated
OC	Overconsolidated
OCR	Over Consolidation Ratio
POP	Pre Overburden Pressure
SC	Stone Columns
SS	Soft Soil
SSC	Soft Soil Creep

Greek letters

$\delta\varepsilon_p^c$	Incremental volumetric creep strain
η	Improvement factor
γ	Unit weight
κ^*	Modified swelling index
λ^*	Modified compression index
μ^*	Modified creep index
ν	Poisson's ratio
ν_{ur}	Poisson's ratio for unloading-reloading
ψ	Dilantancy angle
σ	Normal stress
σ'	Effective stress
σ'_0	In-situ effective stress
σ'_c	Preconsolidation pressure
σ'_v	Vertical effective stress
σ_1	Major principle stresses
σ_2	Intermediate principle stresses
σ_3	Minor principle stresses
τ	Shear stress

τ_c	Time dependency of the preconsolidation stress
ε	Total logarithmic strain
φ	Friction angle

Roman letters

E'_i	Drained stiffness
E'_{50}	Secant modulus
E'_{oed}	Tangential oedometer modulus
E'_{ur}	Elastic unloading and reloading modulus
f_c	The yield surface of the Soft Soil model
K^*	Post-installation coefficient of lateral earth pressure
K_0^{nc}	Coefficient of lateral earth pressure at rest for normally consolidated state
M^*	The slope of the critical state line
p'_0	The size of the yield surface?
c	Cohesion
D	Diameter
E	Young's modulus
k	Permeability
L	Length
p'	Mean effective stress
q	Deviator stress
t'	Effective creep time

1

Introduction

The increasing urbanizing development to meet the demands of the growing population has created a scarcity of adequate land for the construction of new infrastructure. Consequently, this has led to constructions being built on unsuitable soft soils. These soft soils generally possess unfavorable properties such as low shear strength and susceptibility to large settlement and lateral movements.

To handle the disadvantages of constructing on soft soils, different types of deep foundations and ground improvement techniques have been developed. These techniques aim to improve the bearing capacity of the soft soil and mitigate the risks that comes with construction on these types of soil. In addition to this, there is a growing emphasis on adopting sustainable constructions and techniques to lower the carbon footprint.

To stabilize the ground with stone columns is a widely utilized method around the world, however it is very rarely used in Sweden. Two possible reasons why, could be because of the very soft soils in large parts of Sweden which are often considered to sensitive or having too low shear strength to be able to be stabilized with stone columns. The other possible reason is because there is a lack of knowledge to propose stone columns as an option for ground improvement and very limited reference projects.

It is of great interest for Ramboll AB to seek for more sustainable alternatives for ground improvement, since sustainability is a central part of Ramboll's business strategy. Similar to the situation in Sweden, Ramboll lacks knowledge and understanding of how stone columns perform as a ground improvement method and how the surrounding soil behaves. This is the basis for the aim and objectives of this thesis which will be further described in Section 1.1. Included in this chapter are also the method of this thesis as well as a list of limitations.

1.1 Aim and objectives

The aim of this thesis is to through literature study and numerical analyses investigate the differences between dry and wet soil mixing and stone columns as ground improvement methods. The aim is further to conduct an extensive review of the methods to give a proper background which will results in a section with advantages and disadvantages summarized in a tabular format to account for the first of the the listed objectives below. This will then be followed up with a numerical analysis

to account for the remaining objectives.

Following list are the set objectives for this thesis:

1. Through literature review account for the main advantages and disadvantages of the ground improvement methods wet and dry deep mixing as well as stone columns.
2. Evaluate the necessary parameters from the selected sites to be able to set up a Plaxis model as true to the reality as possible.
3. Investigate how different parameters affect the performance of the improvement method including embankment height, centre to centre (cc) spacing and ground condition, by carrying out a numerical analysis in Plaxis 2D.
4. Evaluate and compare the deep mixing method and stone columns through numerical analyses.
5. Conduct a sensitivity analysis to examine how sensitive the models are to key parameters.

1.2 Method

Initially, a literature study was performed to study the ground improvement methods deep mixing and stone columns. For this, scientific papers and books were reviewed in order to provide relevant information about the methods in terms of areas of application, installation process and other technical details. This resulted in four tables accounting for advantages and limitations for the ground improvement methods, based on the aspects economy, environment, installation and other assessments.

To be able to build the numerical model, soil parameters were evaluated. These were evaluated from two different sites, based on already conducted site investigations and laboratory tests. The parameters that were not able to be evaluated were assumed based on different papers and recommendation from experts on the area. Finally the evaluated parameters were back-analysed with the soil test tool in Plaxis 2D, to investigate how compatible the selected parameters are compared to the laboratory tests from the site.

After achieving a satisfactory set of parameters for the soils in question, a unit cell model was set up in Plaxis 2D, to conduct a numerical analysis. The numerical analysis was performed for different embankment heights and a variety of cc-spacing, to figure out the impact of those aspects. A comparison was also made between the two sites to see how the different ground conditions affected the results. In addition to these calculations, a sensitivity analysis was performed on the key parameters as well as the used mesh, to see how sensitive the models were.

At last, the results from the numerical analysis were reviewed and discussed, based on the aim and objectives of this thesis, in order to present reasonable conclusions.

1.3 Limitations

- The numerical analysis will only be conducted in 2D, no 3D analysis will be made.
- The installation effects will only be considered in the literature review, so the effects will not be modelled in Plaxis 2D.
- The numerical analysis will only be performed as a unit cell model stabilizing the ground beneath an embankment.
- The calculations will be done using end-bearing columns, thus floating columns will not be analysed.
- There will not be any additional site investigations made.
- Only singular columns will be considered, installed in a squared pattern.

2

Literature study

2.1 Ground improvement

When encountering a site with poor ground conditions there are different alternatives to assess the problem. Three quick actions, based on a more avoiding strategy, are to either change site for the planned construction, redesign the building or structure to better suit the current conditions, or to replace the problematic material with a more suitable material (K. Kirsch & Bell, 2013). Due to technical or economical reasons these solutions might not be possible to conduct, which would mean that some type of ground improvement would be necessary.

The basic theory of ground improvement is to modify the properties of the current material in the ground, to improve the soil characteristics (K. Kirsch & Bell, 2013). This is done to achieve a specific result, such as increasing the shear strength or density of the soil to improve stability or bearing capacity. This includes reducing the compressibility of the soil to reduce settlement. According to K. Kirsch and Bell (2013) there could be problems connected to ground water which can be solved by changing the permeability. Increased permeability improves deep drainage, leading to more effective preloading or surcharge techniques. Improved drainage could also increase the rate of consolidation.

2.1.1 Categories of ground improvement

According to Nicholason (2015b) there are four different fundamental categories in terms of ground improvement and processes to apply to achieve better geotechnical properties. The categories are divided as follows:

- Mechanical modification
- Hydraulic modification
- Physical and chemical modification
- Modification by inclusion, confinement, and reinforcement

The different categories will be described in the following paragraphs. Further is one example from each category presented in Figure 2.1

Mechanical modification can be described as controlled densification (Nicholason, 2015a). Methods using compaction falls within this category and also techniques where a material is added to the ground with the purpose of strengthening the soil, without using any structural members.

Hydraulic modification is about controlling or altering the hydraulic characteristics in different ways (Nicholason, 2015a). Some common applications is to lower the water table either by dewatering wells or drainage, increase or decrease the permeability, or creating hydraulic barriers.

Physical and chemical modification refers to stabilizing the soil through processes of physical mixing or material injection (Nicholason, 2015a). Methods using thermal treatment are also included in this category. The aim with physical and chemical modification is that the soil will be improved to a "new" material with notably improved characteristics.

The last category, modification by inclusion, confinement, and reinforcement, involves the use of structural members (Nicholason, 2015a). Some examples of these types of reinforcements are soil anchors, geosynthetics or gabions where a granular material is confined.

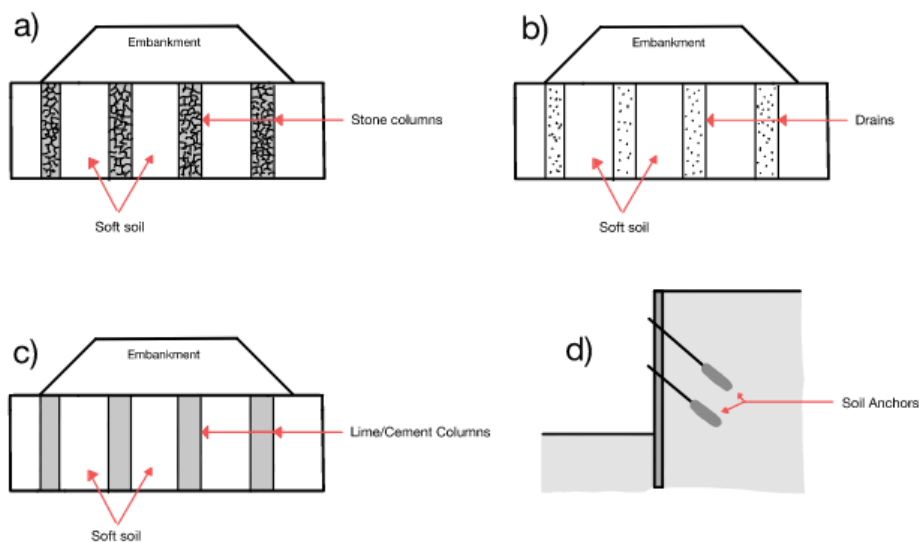


Figure 2.1: Categories of ground improvement. a) Mechanical modification using stone columns. b) Hydraulic modification using drains. c) Chemical modification using lime/cement columns. c) Modification by reinforcement using soil anchors.

Another way to categorize the ground improvement methods is to divide them based on in which phase of the construction the improvement will be made (Nicholason, 2015b). There are three different phases, before, during and after construction. Pre-construction improvements are often a part of the planning and design phase

of the project, hence, this is often the most cost-effective alternative. When the improvement is implemented during construction it is called a "part-of-construction improvement". Something that distinguishes these types of improvement is that they can become a permanent component of a project, for example different reinforcements to stabilize a slope. When an improvement is made after construction, also known as "post-construction improvement", it is often a corrective process. This is usually the last alternative to solve a problem that occurs after the completion of the project. Because of this, it can be very costly.

In the following Sections, 2.2 and 2.3, two ground improvement methods have been reviewed in more detail, namely deep soil mixing (DSM) and stone columns (SC).

2.2 Deep soil mixing

Deep soil mixing is an in-situ ground improvement method to improve soft clay and organic soils without compaction (Kitazume & Terashi, 2013). Fundamentally, different binders are mixed with the soil, and there are mainly two types of binders, dry or wet (Topolnicki, 2012). This leads to a chemical reaction with the soil and/or the water in the soil, depending on whether the dry or wet method is implemented. This results in columns in the soil that improve the strength and stability.

The DSM method originates from a method called mixed-in-place piling, which was developed by Intrusion-Prepakt Inc. during the 1950s (Topolnicki, 2012). This method was picked up by the Nordic countries as well as Japan where the techniques were further developed into what today is known as the two main methods of deep soil mixing, the Japanese method and the Nordic method. Today, DSM has spread and is used as a ground improvement method worldwide. Since the origin of the method, contractors have proposed their proprietary techniques which has led to a variety of different deep mixing processes.

2.2.1 Application

The DSM method, both dry and wet, has a large number of applications. The dry method is typically used for soil stabilization and settlement reduction (Muttuvel et al., 2021). The wet method is used to improve bearing capacity for structural support, shear strength improvement and for settlement control under embankments. Furthermore, the dry and wet methods are well suited for increasing bearing capacity, protection of close-to-excavation site structure, reducing active stresses and increase of passive load on retaining walls, as foundation for houses, bridges and wind turbines, to reduce liquefaction mitigation and vibrations and as remediation for contaminated sites (S. Larsson, 2021).

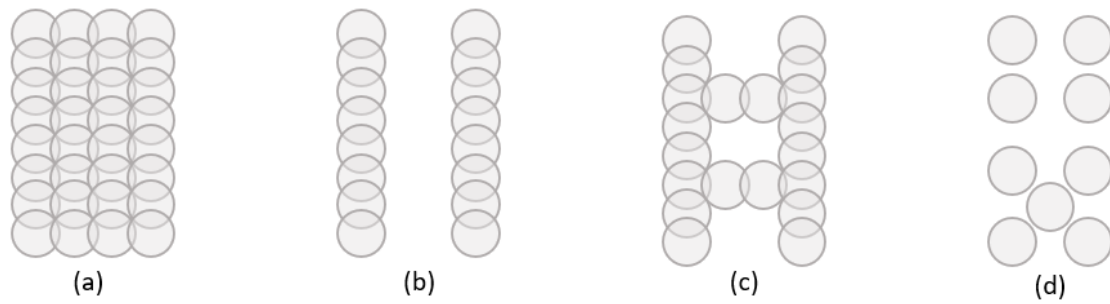


Figure 2.2: Installations patterns. (a) Block. (b) Walls/panels. (c) Grid. (d) Group.

There are a number of commonly used patterns for deep mixing, see Figure 2.2. These pattern usually have different applications which will be described in the following paragraphs.

When installing the columns in block type pattern, all the columns in the stabilized soil are overlapped, creating a big mass of improved soil (Kitazume & Terashi, 2013). This pattern is the most stable option, but it is an expensive process and the installation period is longer than for the other patterns. This pattern is often applied for very heavy structures that are permanent. Block type installation can also be used to prevent leaching from disposal area since the improved soil has a large width and impermeable properties.

Wall or panel type of installation consists of long walls of overlapping columns (Kitazume & Terashi, 2013). The panels are installed perpendicular to the most critical failure plane (Evans et al., 2021). They bear and transfer the weight of superstructures, or external load of other kind, down to deeper stiffer layers (Kitazume & Terashi, 2013). This pattern is applied to support embankments, sheet pile walls and also increase the stability of earth retaining systems. Compared to block type pattern this pattern requires a smaller volume of improved soil and is thereby also less costly.

Grid type installation pattern is a kind of mix between the panel and the block type (Kitazume & Terashi, 2013). The overlapping technique is used and a grid is created to improve the soil. This is a highly stable pattern and the cost is less than for block type installation but more than the panel type. This pattern is often used for marine structures when it is desired to increase the bearing capacity and stability of the ground. This type of pattern has also been proven efficient for preventing liquefaction in sandy soil. Also to prevent pore water pressure to be generated and to stop shear strength deformations within the grid of columns during earthquakes.

For the group type of installation pattern isolated columns of stabilized soil is installed in rows with either triangular or rectangular arrangements, bottom and top in Figure 2.2(d) respectively, to reduce settlements and increase stability (Kitazume & Terashi, 2013). Since this is the only pattern that does not use overlapping columns the required volume of improved soil is smaller and the installations process is quite

quick in comparison to the other patterns. However, the acquired horizontal resistance is not so high for the isolated columns. This pattern is usually applied for foundation of relatively low embankments and structures with light weight.

Another type of deep mixing is mass stabilization. This method is used in soils that are extremely soft, such as peat and mud that have high organic content and compressibility (Dahlström, 2012). Mass stabilization is mainly used for projects involving roads and parking areas or for low embankments (0.5-3m) and to a maximum depth of 5m. In Figure 2.3 the installation process for mass stabilization is shown.

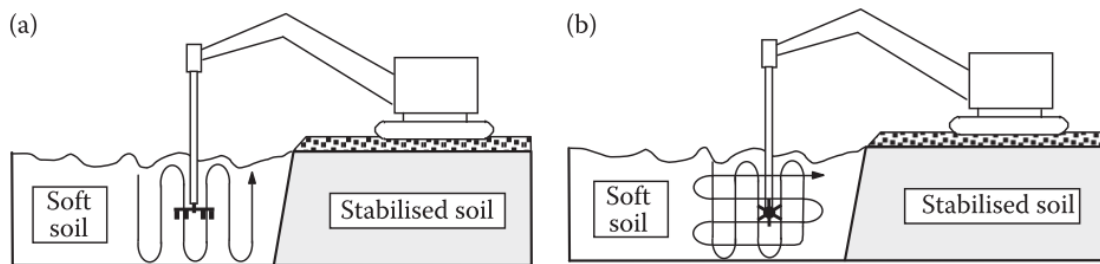


Figure 2.3: Mass stabilization procedure (Topolnicki, 2012). (a) Vertical mixing. (b) Both vertical and horizontal mixing.

2.2.2 The installation process

According to Topolnicki (2012) there are no significant differences in the mixing processes when similar binders are applied in dry or slurry form, looking at the engineering properties of soils. The installation process consist of the following four steps:

1. First the mixing shafts are positioned above the location where the deep mixing is planned to be installed (Topolnicki, 2012).
2. In the next phase, the penetration phase, the mixing tool is brought down to the desired depth at which the deep mixing is supposed to start (Topolnicki, 2012). Depending on which method and type of binder different mediums is used to support the mechanical drilling. For the dry method compressed air is used, for wet mechanical mixing it is slurry and, a third option is hybrid mixing where high-velocity jetting with slurry or water and air is used.
3. Following the penetration phase is the bottoming phase, which means that the mixing tool rotates at the bottom for about 0.5-2 minutes (Topolnicki, 2012). The aim with this phase is to ensure adequate contact between the deep mixing columns and the bearing subsoil.
The injection of the stabilizing binder can be done using two different methods, and can take place during penetration, withdrawal and restroking (Topolnicki,

2012). The two different methods for injection is the penetration injection method and the withdrawal injection method. The former is a top-bottom process which is typically used with the wet method for on-land applications. The latter is on the contrary, a bottom-top process which typically is used for the dry method. It is also possible to use a combination of penetration and withdrawal, if there is a very high binder concentration. If so, parts of the injection can be done during the penetration phase and the rest during withdrawal.

4. Withdrawal, which is done to ensure an effective treatment in the zone between the bearing soil and the soft soil being treated (Topolnicki, 2012). This step can be completed in a variety of ways, which can be seen in Figure 2.4. The different techniques includes continuous upstroke, with or without bottoming, bottom restroking, full-depth restroking or stepped restroking during the withdrawal. The execution could also be done with reversals during penetration.

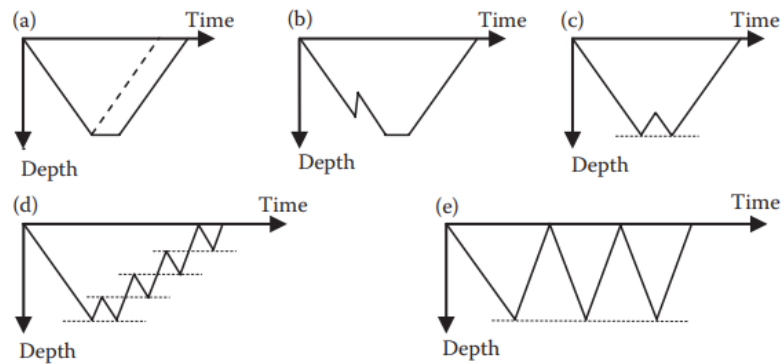


Figure 2.4: Execution procedure of deep soil mixing (Topolnicki & Wehr, 2008). (a) Executed by continuous upstroke, with or without bottoming. (b) With reversals during penetration. (c) With restroking at the bottom. (d) With restroking during withdrawal in steps. (e) With full-depth restroking.

2.2.3 Binders and soil properties

The type of binder and soil properties are important aspects when considering deep soil mixing as a ground improvement method. Lime and cement are the most common binders to consider for deep mixing columns (Dahlström, 2012). However, slag, fly ash, gypsum and other secondary products and compounds are also frequently used as binder for the deep mixing columns. Some tests have been carried out for other binders, however these are not extensively used in actual projects. Since lime and cement are the most common binders, the deep mixing columns will for now on be referred to as lime/cement (LC) columns.

While there are numerous similarities between dry and wet soil mixing techniques,

notable differences exist in the suitability of soil types for each method. The dry method is typically effective for cohesive soil with a moisture content of at least 30% (Dahlström, 2012), as the presence of sufficient amount of water in the ground is essential for initiating the chemical reactions. Conversely, the wet method is better suited for soil with a moisture content below 60%, such as soft clay, silt, or fine sand (Muttuvel et al., 2021).

2.3 Stone Columns

Stone columns is a well-know and commonly used in-situ method for ground improvement in soft soils (Castro et al., 2014). The method uses load-bearing semi-circular columns made of crushed stone or gravel to reduce the total and differential settlements in the soft soil, to improve slope stability and bearing capacity, to speed up the consolidation and to reduce the potential for liquefaction.

Using crushed aggregate as the filler material opens up the possibility of utilising recycled material, and promotes sustainable options (Foda et al., 2023). There is a potential to reuse materials within a single project. For example, when constructing a tunnel in bedrock, the excavated material could be used as filler material in the stone columns. In this way, both the environment and the economy will be favored. Ultimately, according to Foda et al. (2023), the choice of fill material is based on three key factors: availability, suitability and economy.

According to Guetif et al. (2007), there are three factors that contributes to the improvement of the soft soil when using stone columns. The first factor is the inclusion of a stiffer column material; next is the densification of the surrounding soil, which improves the stiffness and increases the unit weight in the soft clay. The last factor of improvement lies in the greater permeability of stone columns compared to the surrounding soil, allowing them to function as vertical drains and release excessive pore pressures (Castro, 2017). The vertical drainage effect contributes to the consolidation process being speeded up, increasing the settlement during the construction period (Kalantari & Mokhtari, 2012), thus leading to decreased amount of settlement after construction.

2.3.1 Application

There are several areas of application including supporting the soil underneath isolated footings, raft foundations and embankments (Kalantari & Mokhtari, 2012). Beena (2010) states that stone columns are also well-suited for landslide stabilization and reducing liquefaction potential of fine sands. The pattern used when installing stone columns are group type as described in Chapter 2.2.1 (Chan & Poon, 2021).

2.3.1.1 Design parameters

Key design parameters for stone columns include their diameter (D) and length (L), the cc-spacing, and whether they are designed as end-bearing or floating. Column diameters range from 0.6 to 1.1 meters, with smaller diameters typically used in harder clay and larger ones in softer clay (Foda et al., 2023). In Sweden, a diameter of 0.8 meters is commonly assumed for stone columns.

End-bearing stone columns, which extend down to a firm stratum, are the most commonly implemented design, although floating columns, terminating in soft clay, are also used (Ng & Chew, 2021). Special considerations, such as long-term settlements, must be addressed due to the untreated area beneath the toe of floating columns. Figure 2.5 illustrates the concepts of end-bearing and floating columns.

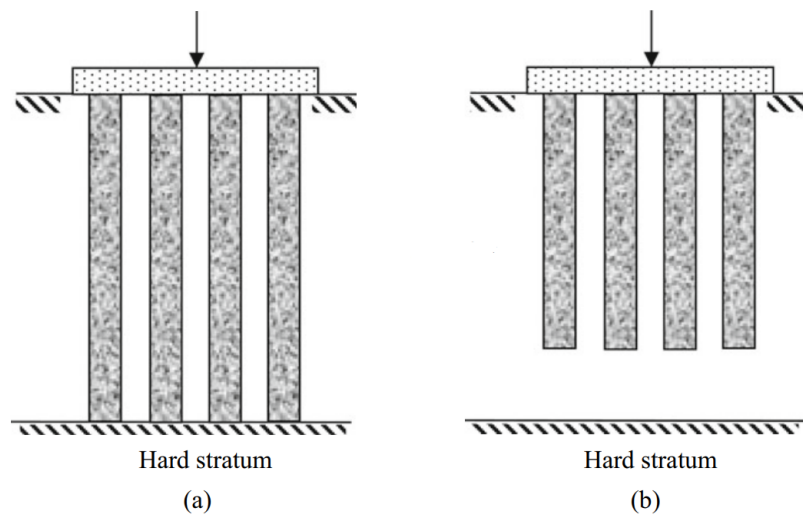


Figure 2.5: (a) End bearing stone columns (Ng & Chew, 2021). (b) Floating stone columns .

The column length is a critical design element (Foda et al., 2023). Various standards exist for determining the optimal column length. For instance, Dash and Bora (2013) indicates that column length should be larger than five times the diameter, whereas Ghazavi and Nazari Afshar (2013) and Naderi et al. (2018) suggest a ratio of $L/D > 4$ to prevent column bulging. Nazariafshar et al., 2022 proposes that a ratio greater than five is necessary to avoid punching shear in the columns.

Column spacing is typically influenced by specific site conditions and generally ranges from two to three times the column diameter (Foda et al., 2023). These conditions include the installation pattern, column factors, and soil properties, among others.

2.3.1.2 Encased stone columns

In very soft soils, the effectiveness of the stone columns is questionable due to their continuity, stability and shape (Castro, 2017). Typically, the undrained shear strength serves as a crucial parameter for determining the feasibility of the stone columns. The absence of lateral restraints can cause increased settlements and reduced radial drainage due to clay particles being clogged around the stone columns (Beena, 2010). Another limitation with stone columns in very soft soils, and soils with a high sensitivity, is the risk of aggregates drifting away in the surrounding soil (N. Yildiz Helvacoglu, personal communication, January 8, 2024).

In order to address these challenges, geotextile or geosynthetics can be used to encase the stone columns. Geotextile or geosynthetics can be described as a permeable fabric that, in this case, improves the lateral confinement and minimize the losses in strength and compressibility (Beena, 2010). A conceptualization of this phenomenon is shown in Figure 2.6.

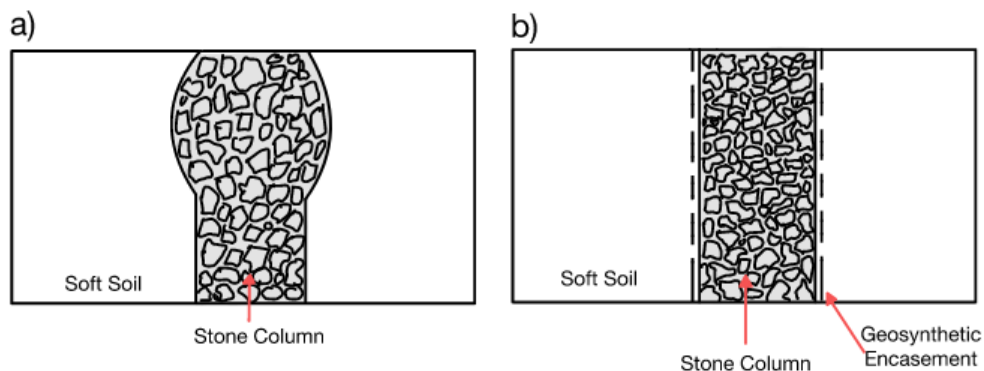
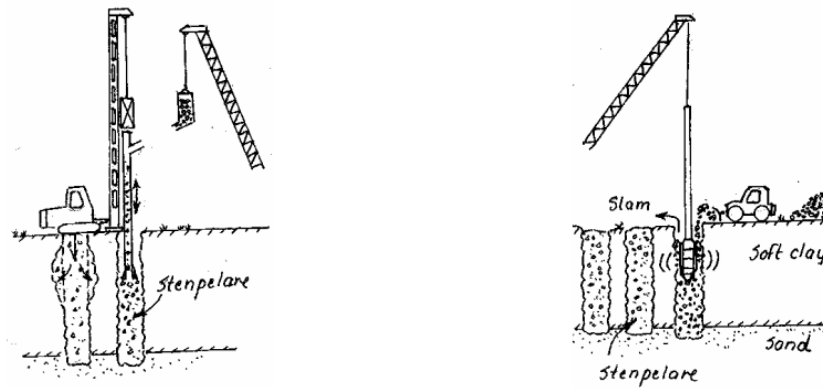


Figure 2.6: (a) Regular stone column in a very soft soil. (b) Encased stone column with geosynthetic.

Research conducted by Malarvizhi and Ilamparuthi (2004) shows reduced settlements when using encased stone columns. Furthermore, findings indicate a direct correlation between the stiffness of the encasement material and settlement levels, where greater stiffness leads to less settlement.

2.3.2 The Installation Process

Several different methods are being employed for installation of stone columns. These methods encompass variants either with or without using jetted water and a variant called the case-borehole or rammed columns method (Foda et al., 2023).



(a) Displacement method

(b) Replacement method

Figure 2.7: Installation process for stone columns (Svenska Geotekniska Föreningen, n.d.).

The fundamental principle for the method employing jetted water, Figure 2.7(b), also known as the vibro-replacement method, is that parts of the in-situ soil are replaced with fill material which forms stone columns (Foda et al., 2023). To achieve this, jetted water is used to displace the soil and thus create a hole in the soil (Chan & Poon, 2021). When the hole has been created, crushed aggregates are added, often from the ground surface (top feed), and then compacted with a vibrator. This technique is well suited for soft soils with a high ground water level (Foda et al., 2023).

The vibro-displacement method, Figure 2.7(a), is a more conventionally used method for installing stone columns (Chan & Poon, 2021). For this method a deep vibrator using compressed air is employed to laterally displace the soil to create a hole for the stone column (Kalantari & Mokhtari, 2012). Thereafter, crushed aggregates can be inserted either by the top- or bottom feed procedure. This technique is more suitable for firmer soils, when the ground water table is low.

The third installation method, case-borehole or rammed stone columns method, is based on prebored holes that are filled with filler material by ramming which forms the stone columns (Foda et al., 2023).

2.3.3 Installation effects

The impact of installation is crucial for precise stone column design, irrespective of whether these impacts are beneficial or detrimental. Nevertheless, such impacts are frequently overlooked, with designs typically relying on rigid inclusion instead of considering how the columns influence the properties of the adjacent natural soft clay (Castro & Karstunen, 2010). Watts et al. (2000) verifies that the characteristics of soft soil are enhanced through the installation of stone columns, which in turn enhances the load displacement behavior of the reinforced soil.

Elshazly et al. (2006) showed in their finite element analysis that the soil state of stress is substantially altered when installing vibro stone columns. By utilizing post-installation soil parameters and load-settlement records from a field load of full-scale the ratio of horizontal to vertical soil stress, K^* , is determined. K^* ranges between 1.1-2.5. If these radial changes in stress ratio were to be neglected when predicting the settlements, the capacity of stone columns would be underestimated and thereby the settlements overestimated. In another paper Elshazly et al. (2008) shows that K^* is decreased when the inter column spacing increases.

Further, Guetif et al. (2007) demonstrates in their finite element analysis that Young's modulus is significantly improved because of the consolidation caused by the installation of stone columns. In a drained case the effective mean stress is increased about four times in comparison with soft clay. F. Kirsch (2009) researched global and individual installation effects of stone columns of a group of 25 columns. Using cavity expansion both effects were numerically modelled. This showed an increase in the stress state and an improvement of the soft soil characteristics. Castro and Karstunen (2010) explores through a numerical analysis the installation effects of stone columns and found that the installation increases horizontal stresses. It is also shown that the lateral earth pressure coefficient is increases by a factor of 1.4 from the initial value at rest after the excess pore pressure has dissipated.

2.4 Advantages and Disadvantages

In this chapter, the results from the literature study will be presented. The results focuses on the advantages and disadvantages with the methods included in Chapter 2. The advantages and disadvantages are divided into different tables based on the aspects environment, economy, installation and others assessments, see Tables 2.1-2.4.

Table 2.1: Economy

Ground improvement technique	Advantage/Disadvantage
Deep soil mixing	<p>Both lime and cement are considered relatively expensive (Barisoglu et al., 2023).</p> <p>Allows for high productivity, hence it is cost effective (Topolnicki, 2016).</p>
Specific for wet or dry method	<p>The dry method does not need to be pre-mixed with water before it is injected to the ground, which allows for a higher rate of construction (K. Kirsch & Bell, 2012).</p>
Vibrated stone columns	<p>Relatively easy to find local fill material, to keep the cost low (Chan & Poon, 2021).</p> <p>For the replacement process there is a lot of water needed and in case of disposal it is expensive to fix especially if the water is contaminated (Chan & Poon, 2021).</p> <p>Risk for significant heaving which can lead to soil having to be removed, if contaminated this process can be expensive (Chan & Poon, 2021).</p>

Table 2.2: Environment

Ground improvement technique	Advantage/Disadvantage
Deep soil mixing	<p>Lime has a high environmental impact, $1.45kgCO_2e/kg$ (Trafikverket, 2024).</p> <p>In general there is almost no spoil material (Muttuvel et al., 2021).</p>
Specific for wet or dry method	<p>For the wet method there is a risk for slurry overflow which can be contaminated, hence effect the environment and the surrounding nature negative (Muttuvel et al., 2021).</p>
Vibrated stone columns	<p>It is possible to use recycled material (Chan & Poon, 2021).</p> <p>Aggregates has a low environmental impact, $0.004kgCO_2e/kg$ (Trafikverket, 2024)</p> <p>Relatively easy to find material nearby, hence cut longer transport distances (Chan & Poon, 2021).</p>

Table 2.3: Installation

Ground improvement technique	Advantage/Disadvantage
Deep soil mixing	<p>It is possible with targeting treatment without effecting overlaying soils due to installation with withdrawal of the installation tool (K. Kirsch & Bell, 2012)</p> <p>No problems installing deep mixing in already built up areas since there is little vibration generated (Sexton et al., 2016)</p> <p>If obstacles are encountered, such as blocks, the installation can not proceed (Muttuvel et al., 2021)</p> <p>Relatively fast construction time compared to stone columns (Muttuvel et al., 2021)</p>
Specific for wet or dry method	<p>For the dry method it is possible to operate in low temperatures since compressed air is used to transport the binder (K. Kirsch & Bell, 2012)</p>
Vibrated stone columns	<p>If the columns are installed to close to each other there is a risk for lateral movements which can negatively affect adjacent infrastructure (Chan & Poon, 2021)</p> <p>Vibrations in the probe can cause a drop in the shear strength in sensitive soils (Chan & Poon, 2021)</p> <p>It is not suitable to install stone columns near sensitive buildings and/or utilities, due to vibrations when installing the columns (Chan & Poon, 2021)</p>

Table 2.4: Other assessment

Ground improvement technique	Advantage/Disadvantage
Deep soil mixing	<p>Lime has a limited improvement effect on organic soils (S. Larsson, 2021)</p> <p>Suitable for various types of soils, soft clay to loose sand, resulting in high shear strength and low compressibility (Arulrajah et al., 2009)</p> <p>If there are acidic conditions in the soil, there is a risk that the columns will not obtain full strength (Muttuvel et al., 2021)</p> <p>DSM is a widely known method for improvement of soft soils and have been proven to be successful in many projects in Sweden</p>
Specific for wet or dry method	<p>Dry soil mixing demands a smaller amount of binder compared to wet mixing (Dahlström, 2012)</p> <p>Wet mixing gives a more homogeneous column which is preferred when stabilizing slopes (Karstunen, 2014)</p>
Vibrated stone columns	<p>In clays with high sensitivity the radial drainage of the stone columns could be reduced due to clay particles getting clogged in and around the stone columns (Kalantari & Mokhtari, 2012)</p> <p>There are difficulties when back-analysing the method since many parameters are involved (Chan & Poon, 2021)</p> <p>There is a risk that the column could budge or fail (Chan & Poon, 2021)</p>

3

Constitutive models in Plaxis

When doing numerical analysis in Plaxis it is necessary to use a constitutive model to express the stress-strain relationship of the clay (Amavasai & Karstunen, 2017). This is due to the emerging stress path which is not possible to control, since it is a result of different aspects of the soil. The influencing factors for the emerging stress path are the initial state of the soil, the undrained shear strength and the effects on the mobilised stiffness and pore pressure due to the loading type and the loading rate. A constitutive model is a mathematical formulation that enables predictions of soil behavior under any given stress path using a unified set of model constants.

The choice of which constitutive soil model is used has consequences for the outcome of the geotechnical numerical analysis (Amavasai & Karstunen, 2017). However, it is not the only influencing factor, the quality of the soil samples and the performed tests are also of great significance for the results of the analysis. It is important to choose a representative soil model that includes all the relevant features. Parameters that can be derived based on the available data enable to avoid using of empirical values with a risk to end up to far from the actual scenario.

In this thesis, the Soft Soil Creep (SSC) model was used to model the soft clay layers and the lime/cement columns were modelled with the Hardening Soil (HS) model. For the remaining materials; the sand layer, the embankment fill and the stone columns, the Mohr-Coulomb (MC) model was used. The decisive factor, for which soil models that were used, was the available laboratory tests. This effected which parameters that could be derived, hence limited the choice of soil model. In addition to that, the availability of parameters in literature was another decisive factor.

The three relevant soil models, SSC, HS and MC, will be presented in the following Chapters 3.2, 3.4 and 3.5, along with the initial Chapter 3.1, that describes the Mohr-Coulomb failure criteria that all of the included soil models are based on. In order to show the impact of creep on the vertical displacements, the analysis was also made using the Soft Soil (SS) model instead of SSC for the clay layers. Hence, in Section 3.3, the SS model is described in general.

3.1 Mohr-Coulomb failure criteria

The Mohr-Coulomb failure criterion is a mathematical framework that describes the condition during which an isotropic material will fail (Labuz & Zang, 2012). The

criterion focuses on the major and minor principal stresses, σ_1 and σ_3 respectively, while disregarding the effect of the intermediate principal stresses, σ_2 . To express this criterion one can either use the principal stresses or normal stress, σ , and shear stress, τ , on the failure plane. The model is based on Mohr's and Coulomb's failure conditions. Mohr's condition which is a foundational aspect of the Mohr-Coulomb criterion, assumes that failure depends solely on the major and minor principal stresses, it provides a graphical representation of stress states using Mohr circles, see Figure 3.1. While Coulomb's condition relies on a failure envelope which is linear to determine the critical stress combination leading to failure.

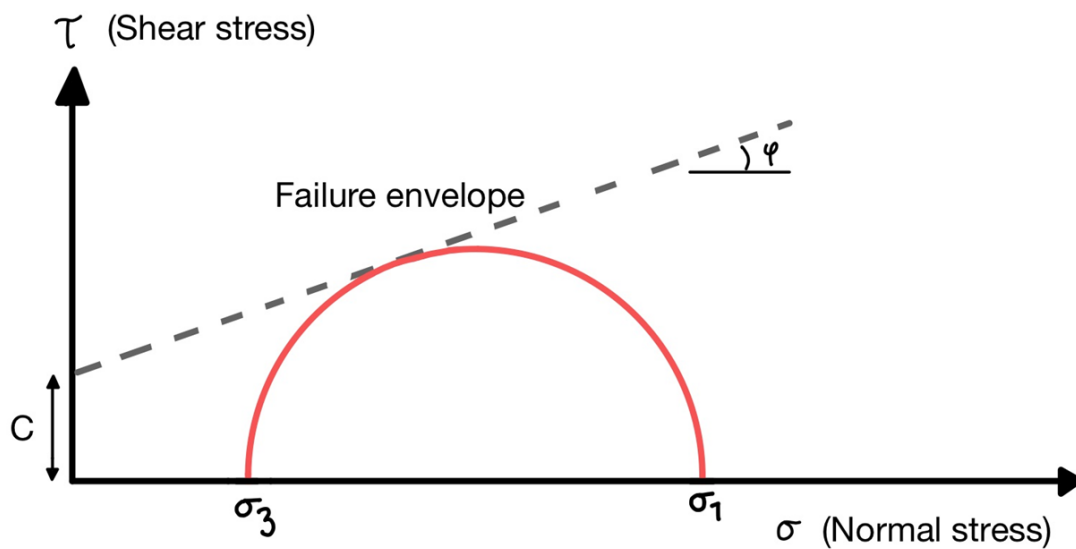


Figure 3.1: Mohr circle.

3.2 Mohr-Coulomb model

The Mohr-Coulomb model is an elasto-plastic perfectly-plastic model, which means that until failure is reached there is a purely linear elastic response (Amavasai & Karstunen, 2017). For soft clay that are normally consolidated (NC) or lightly over-consolidated (OC), MC is not the most suitable model to describe the stress-strain behaviour, since these types of soft clay are prone to considerable contraction, thus not exhibiting perfect plasticity. The MC model includes five input parameters: two for soil elasticity - Young's modulus (E) and Poisson's ratio (ν) and three for soil plasticity: cohesion (c), friction angle (φ) and dilatancy angle (ψ).

3.3 Soft Soil model

The SS model is a model which was inspired by the Modified Cam Clay model (Amavasai & Karstunen, 2017). It is a simple elastoplastic model which can be used for soft, normally consolidated and over consolidated soil. The yield surface, defined by Equation 3.1, of the model is an ellipsoidal cap, which size, p'_0 , is dependant on the over consolidation ratio (OCR) or pre overburden pressure (POP).

$$f_c = \frac{q^2}{(M^*)^2} + p'(p' - p'_0) \quad (3.1)$$

The value of the parameter related to the aspect ratio of the ellipsoid, M^* , is calculated based on K_0^{nc} , the coefficient of earth pressure at rest in normally consolidated region.

The SS model assumes associated flow in the cap yield surface (Amavasai & Karstunen,

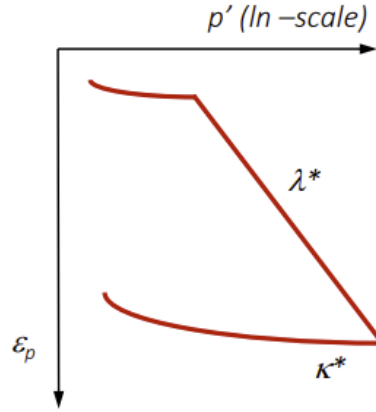


Figure 3.2: Definition of the modified compression and swelling index (Amavasai & Karstunen, 2017).

2017). Additionally, the modified swelling index, κ^* , and modified compression index, λ^* , serve as stiffness parameters. They are described as the gradient of the curve illustrating the relationship between the natural logarithm of variations in mean effective stress and volumetric strain, see Figure 3.2.

3.4 Soft Soil Creep model

The SSC model (Vermeer & Neher, 1999; Vermeer et al., 1997), is a further development of the SS model, that introduces rate-dependant features (Amavasai & Karstunen, 2017). It adheres MC failure criterion and utilizes Modified Cam-Clay as a reference surface (Olsson, 2010). Inspired by the Dutch ABC model, the total strain is formulated as follows in Equation 3.2.

$$\varepsilon = \varepsilon^e + \varepsilon_{dc}^c + \varepsilon_{ac}^c = A \cdot \ln \left(\frac{\sigma'}{\sigma'_0} \right) + B \cdot \ln \left(\frac{\sigma_{pc}}{\sigma_{p0}} \right) + C \cdot \ln \left(1 + \frac{t'}{\tau_c} \right) \quad (3.2)$$

The formulation distinguishes between elastic and visco-plastic creep components denoted by superscript e and c respectively (Olsson, 2010). The part which is visco-plastic is subdivided into consolidation and post-consolidation phase. The variables A and B is described in Figure 3.3 and C is described in Figure 3.4.

Multiple preconsolidation stress states are used with σ_{p0} corresponding to before

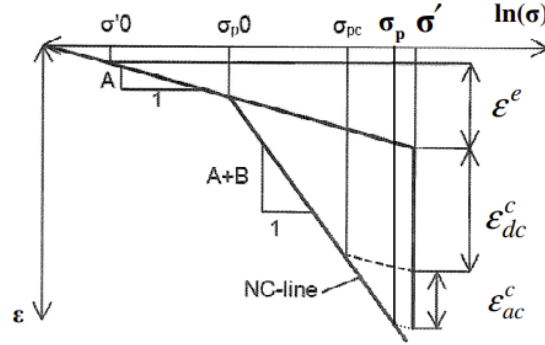


Figure 3.3: An idealization of a stress-strain curve which strain increment is divided into an elastic and a creep component from an oedometer test (Olsson, 2010).

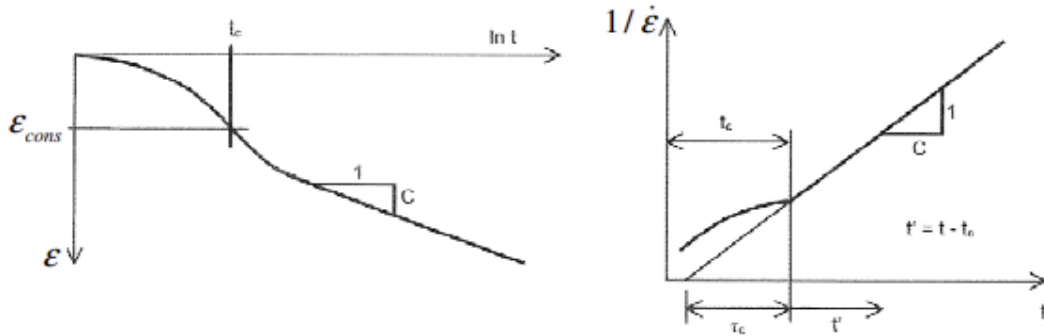


Figure 3.4: Creep behaviour and consolidation in a standard oedometer test (Olsson, 2010).

loading and σ_{pc} represents the preconsolidation stress at the end of the consolidation state (Olsson, 2010). The visco-plastic component is assumed to be purely creep, ε^c . Additionally, according to the SSC model, the accumulated creep strain over time fully determines the preconsolidation stress. This can be seen in the following expression, Equation 3.3 for total strain.

$$\varepsilon = \varepsilon^e + \varepsilon^c = A \cdot \ln \frac{\sigma'}{\sigma'_0} + B \cdot \ln \frac{\sigma_p}{\sigma_{p0}} \quad \text{where} \quad \sigma_p = \sigma_{p0} \cdot \exp\left(\frac{\varepsilon^c}{B}\right) \quad (3.3)$$

A, B and C can also be used to describe the model parameters λ^* , κ^* and μ^* , see Equation 3.4 (Olsson, 2010). Where κ^* is the modified swelling index, λ^* is the modified compression index and μ^* is the modified creep index.

$$\kappa^* \approx \frac{3 \cdot (1 - \nu_{ur})}{(1 + \nu_{ur})} \cdot A, \quad \lambda^* = B + \kappa^*, \quad \mu^* = C \quad (3.4)$$

The SSC model employs a Normal Compression Surface (NCS) to differentiate between small and large creep strain, instead of a yield surface, see Figure 3.5 (Amavasai & Karstunen, 2017). The model deems that the creep strain is irrecoverable, with the NCS presumed to show constant volumetric creep, this assumption is however erroneous. The model computes the incremental volumetric creep strain according to Equation 3.5. Where μ^* is derived for semi-logarithmic space, Figure 3.6.

$$\delta\varepsilon_p^c = \frac{\mu^*}{\tau} \left(\frac{p'_{eq}}{p'_p} \right)^\beta \quad \text{with} \quad \beta = \frac{\lambda^* - \kappa^*}{\mu^*} \quad (3.5)$$

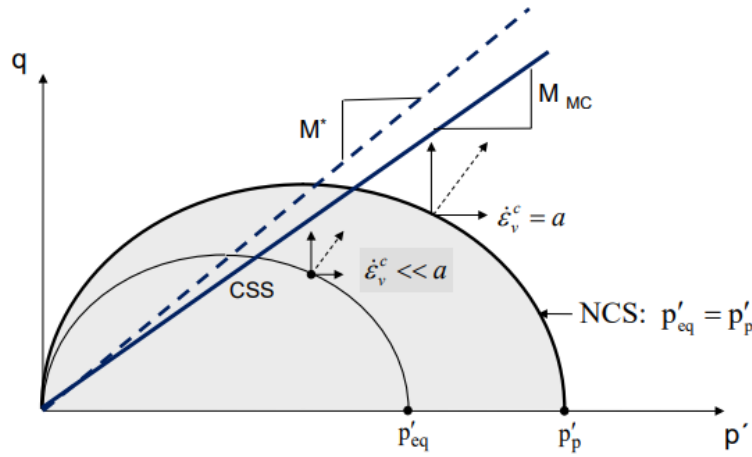


Figure 3.5: Soft Soil Creep model (Amavasai & Karstunen, 2017).

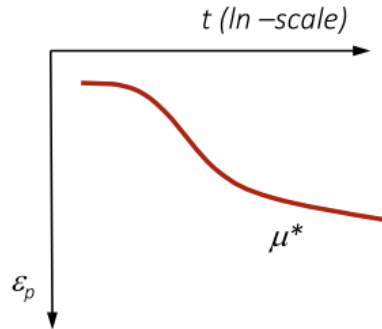


Figure 3.6: Definition of the modified creep index (Amavasai & Karstunen, 2017).

When conducting a conventional incremental loading (IL) test, the load is maintained for a constant period $t_c + t' = \tau$, where τ , in the SSC model, is equal to one day (Olsson, 2010). A normal consolidation line with $\sigma_p = \sigma'$ is obtained for this kind of IL. By combining equations 3.2 and 3.3, a simplified expression for the time dependency of the preconsolidation stress can be derived as in Equation 3.6. Further, the differential equation for the total strain is derived according to Equation

3.7.

$$\tau_c = \tau \cdot \left(\frac{\sigma_{pc}}{\sigma_p} \right)^{\frac{B}{C}} \quad (3.6)$$

$$\dot{\varepsilon} = \dot{\varepsilon}^e + \dot{\varepsilon}^c = A \frac{\dot{\sigma}'}{\sigma} + \frac{C}{\tau} \left(\frac{\sigma'}{\sigma_p} \right)^{\frac{B}{C}} \quad (3.7)$$

Moreover, the model predicts creep strains in both NC and OC regions (Amavasai & Karstunen, 2017). There are however some restrictions on stress states exceeding the MC failure criterion and therefore the model is not well-suited for highly over-consolidated clays.

3.5 Hardening Soil model

The Hardening Soil model (Schanz et al., 1999) was developed because there were some limitations correlated to the overconsolidation region in the existing models for soft soil (Amavasai & Karstunen, 2017). This relatively complex model has three parts: a volumetric cap yield surface, a shear hardening cone and a separate failure yield surface.

The failure yield surface uses the Mohr Coulomb failure condition (Amavasai & Karstunen, 2017). The initial size of the cap yield surface is defined by the OCR or POP, depending on which is used and expands with the plastic volumetric strain. Further, an associated flow rule is assumed on the cap surface. The initial size of the shear hardening cone is related to the coefficient of lateral earth pressure at rest for normally consolidated state, K_0^{nc} , which is defined using Jaky's formula. Unlike the cap surface, the shear hardening cone has no associative flow and instead the ultimate dilatancy angle, ψ , is used as a input. The dilatancy angle is assumed to be zero for soft clays.

The parameters for stiffness are the stress-dependent reference stiffnesses calculated based on a non-linear relationship of Ohde-Janbu-type by Equation 3.8 where $a = c' \cot(\varphi')$ (Amavasai & Karstunen, 2017). If the effective cohesion is assumed to be zero and the modulus exponent m is set to one, which it often is for soft soils, the stress-strain relation becomes semi-logarithmic.

$$E'_i = E_i^{ref} \left(\frac{\sigma'_i + a}{p_{ref} + a} \right)^m \quad (3.8)$$

A secant modulus, E'_{50} , and a elastic unload-reload modulus, E'_{ur} , represents the elasto-plastic stiffness under triaxial shearing (Amavasai & Karstunen, 2017). These can be calculated with Equations 3.9 and 3.10 in cases where $c'=0$ and $m=1$. The superscript 'ref' indicates that these are reference values of the corresponding E' modulus and σ'_3 is the cell pressure.

$$E'_{50} = E_{50}^{ref} \left(\frac{\sigma'_3}{p_{ref}} \right) \quad (3.9)$$

$$E'_{ur} = E_{ur}^{ref} \left(\frac{\sigma'_3}{p_{ref}} \right) \quad (3.10)$$

A third modulus is also needed for the model, the tangential oedometer modulus, E'_{oed} , which is defined by Equation 3.11 (Amavasai & Karstunen, 2017). If the E'_{oed} is set to be the compression modulus, M_L , σ'_1 is set to be the preconsolidation pressure.

$$E'_{oed} = E_{oed}^{ref} \left(\frac{\sigma'_1}{p_{ref}} \right) \quad (3.11)$$

There are, however, some limitations to the Hardening Soil model. For example there are some difficulties with defining the stiffness parameters in the laboratory testing programs typically used in Sweden, since it does not include drained triaxial testing (Amavasai & Karstunen, 2017). Further, there are some trouble to get a feeling for typical values because of the arbitrary stress level that the reference moduli are based on. Also some internal limitations in correlation to the reference moduli are present and this makes it difficult to use input values that are typical for in Swedish soil.

4

Numerical analysis

To conduct the numerical analyses, Plaxis 2D was used. Plaxis 2D is a geotechnical finite element analysis software to model, simulate and analyse different kinds of geotechnical problems (Plaxis, 2024). In this chapter the outlines and the method of the numerical analysis will be described, as well as how the parameters have been derived and used input parameters will be presented. The numerical analysis was conducted at two different sites, the two sites will be referred to as Östra berg and Väg 160.

4.1 Method

The chosen approach for the numerical analysis in this thesis is a unit cell model, which is widely recognized for its efficiency in theoretical analysis, it is well suited for analysis on final settlements and consolidation (Castro, 2017). The model is an axisymmetric model, using 6-noded triangular elements. The unit cell radius and the consequent width of the soil within each model is derived as half of one centre to centre spacing (cc-spacing). Thus, the width of the soil in each model is the half of one cc-spacing subtracted by the radius of one column, since rectangular pattern is assumed for the grid of the columns. A conceptualization of the used unit cell model is illustrated in Figure 4.1, the model in the figure is not true to scale.

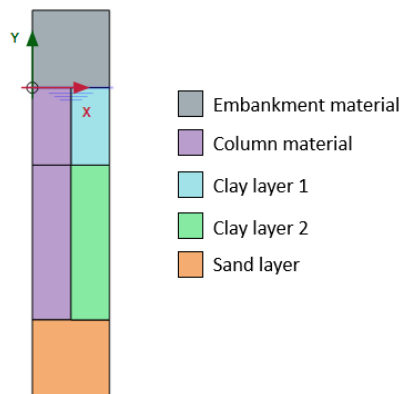


Figure 4.1: Conceptual figure of unit cell model

Proceeding to the boundary conditions of the model, the hydraulic boundaries are closed on all sides except the top. Regarding the deformation conditions it is set to normally fixed on the sides and free at the top. The bottom is set to fully fixed and the ground water table is at the ground surface.

The analysis is designed to be able to test mainly four different aspects accordingly:

- The impact of different embankment heights.
- The impact of different cc-spacing.
- The impact of different parameter sets of the clay.
- How well deep mixing resp. stone columns performed as ground improvement compared to each other.

To model the impact of different embankment heights and cc-spacings, four embankment heights was tested as well as 6 cc-spacings. The different heights were 1.5m, 2m, 3m, and 5m. The different spacings were 1m, 1.4m, 1.8m, 2.2m, 2.6m, and 3m. The diameter of the deep mixing and stone columns were assumed to be 0.6m and 0.8m respectively, since this is considered standard dimensions in Europe (Carlsten, 1996; M. Karstunen, personal communication, January 24, 2024). To analyse different sets of parameters, two sites have been used to derive parameters. The two sites are referred to as Östra berg and Väg 160 and are described more in detail in Chapter 4.3. The calculations are limited to only include end-bearing columns since the thickness of the clay layers are less than 15m.

4.1.1 Staged construction

In Table 4.1, the phases of the staged construction and their duration is presented. Every phase starts from the phase before. Initially a K0-procedure is completed with just the soil layers to generate the initial conditions, hence, the initial effective stresses. Thereafter, the column is installed (by changing the material), followed by the construction of the embankment. When the construction of the embankment is finished it is left to consolidate for 100 years. The consolidation phase is divided into different phases to be able to analyse the results easier, the different phases can be seen in Table 4.1.

Table 4.1: Calculation phases for staged construction in Plaxis 2D.

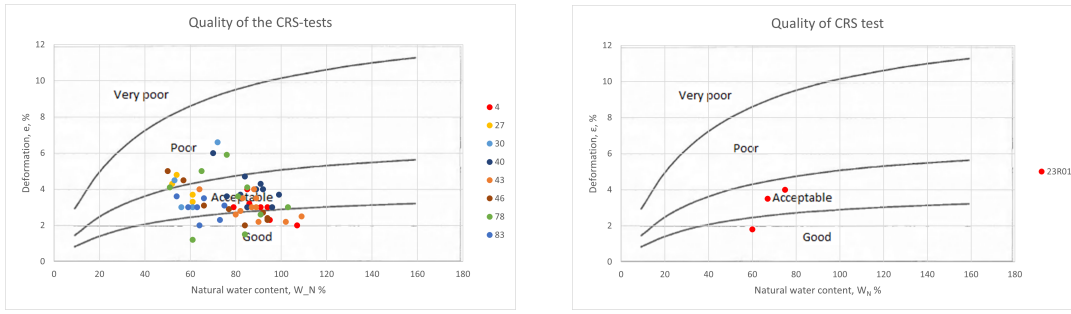
Analysis type	Phase	Duration [Days]
K0 - procedure	Initial phase	-
Consolidation	Clay	1
Consolidation	Column installation	1
Consolidation	Embankment	20
Consolidation	Consolidation 100 days	100
Consolidation	Consolidation 1 year	265
Consolidation	Consolidation 5 years	1 460
Consolidation	Consolidation 10 years	1 825
Consolidation	Consolidation 50 years	14 600
Consolidation	Consolidation 100 years	18 250

4.1.2 Mesh

A convergence study for the mesh was performed to guarantee that an ideal mesh was used for the numerical study. The initial mesh was generated as fine and the mesh were then refined until the results converged. The results from the mesh sensitivity analysis and the final mesh is found in Appendix A.1.

4.2 Parameter determination

In this section it is described how the parameters were derived, the input data used can be found in Section 4.3. The initial stress parameters were evaluated from a set of constant rate of strain (CRS) tests from each site. Initially the quality of the data from each borehole were evaluated according to R. Larsson et al. (2007), see Figure 4.2. For the site at Östra berg there were only one borehole available in the relevant cross section and after the quality check it was evaluated to be acceptable to use the data. Each dot within the same borehole represents a different depth.



(a) Väg 160

(b) Östra berg

Figure 4.2: The results from the quality check of the CRS tests conducted in Väg 160 and Östra berg.

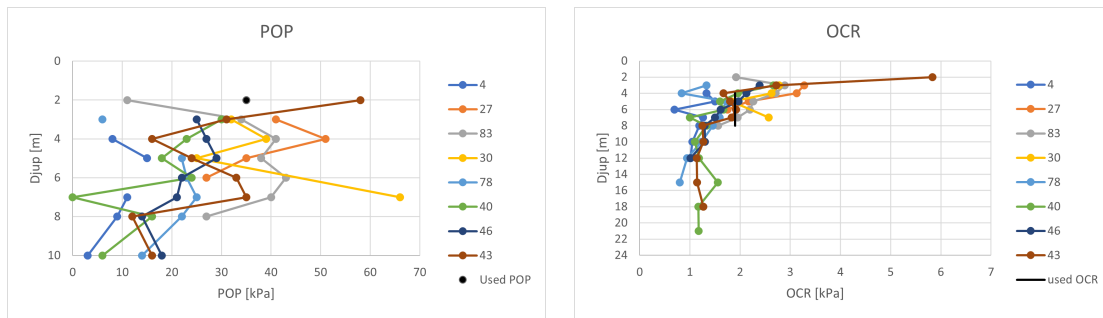
4.2.1 Initial stress state parameters

The preconsolidation pressure σ'_c was evaluated in the the already conducted CRS test. OCR and POP was calculated with Equation 4.1 and Equation 4.2, respectively.

$$OCR = \frac{\sigma'_c}{\sigma'_v} \quad (4.1)$$

$$POP = \sigma'_c - \sigma'_v \quad (4.2)$$

OCR and POP was later plotted against depth to find the most suitable value for the corresponding layer, see Figure 4.3. Since only one borehole was used in Östra berg, each OCR corresponding to a relevant depth in a clay layer was used, see Table 4.7.



(a) POP in Väg 160.

(b) OCR in Väg 160.

Figure 4.3: POP and OCR plotted against depth in Väg 160.

Poisson's ratio for unloading/reloading, ν_{ur} , for the clay layers have been assumed according to Amavasai and Karstunen (2017). Amavasai and Karstunen (2017) claims that ν_{ur} for soft soils often is assumed to have a constant value between 0.1-0.2. Further, K_0^{nc} is calculated with Jaky's formula, see Equation 4.3.

$$K_0^{nc} = 1 - \sin(\varphi') \quad (4.3)$$

4.2.2 Stiffness parameters

The modified swelling index, κ^* , and modified compression index, λ^* , was evaluated at the different depth that a CRS test have been conducted. The used κ^* and λ^* was a mean from all the relevant depth corresponding to each clay layer.

4.2.3 Permeability

The permeability of the clay was evaluated based on conducted CRS tests. The used permeability for each clay layer was a mean value from relevant soil depths. The permeability for the LC-columns were assumed to be the same as for the surrounding clay and for the stone columns, a value recommended from Keller was used. The embankment is assumed to be fully drained.

4.2.4 Soil test

After the parameters were derived as described in the earlier section a soil test was conducted with the soil test tool available in Plaxis 2D, to enhance the approximation of the parameters. The aim with the simulated soil test was to see how well the selected set of parameters corresponds to how the soil behaves in reality. The parameters used for Väg 160 was compared with the available CRS test and the parameters λ^* and κ^* were calibrated until the graphs aligned, an example can be seen in Figure 4.4.

Since λ^* , κ^* and OCR are crucial factors when calculating settlements during a numerical analysis, the main focus when conducting the soil test was to ensure alignment in the parts of the graph correlated to these values.

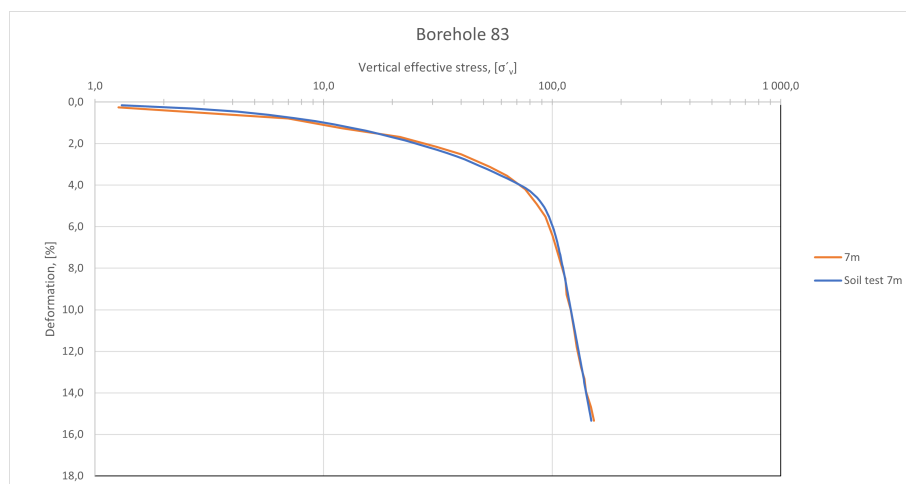


Figure 4.4: Comparison of soil test result and CRS test results, stress in log-scale plotted against strain in percent.

4.3 Input parameters

In this section the parameters used as input in Plaxis 2D will be presented for each material and associated layers.

4.3.1 Stone columns

Presented in Table 4.2 is the parameters used for stone columns. $\gamma_{sat/unsat}$, ν_{ur} and c' have been collected from Castro et al. (2014) while the rest of the parameters have been assumed with the help from Keller (N. Yildiz Helvacoglu, personal communication, March 18, 2024).

Table 4.2: Parameters used for the stone columns in Plaxis 2D.

Parameter	Value	Unit
γ_{unsat}	16	kN/m^3
γ_{sat}	19	kN/m^3
E'_{ref}	55 000	kN/m^2
ν	0.2	-
c'_{ref}	0.1	kN/m^2
ϕ'	38	°
ψ	8	°
k_x, k_y	864	m/day

4.3.2 Lime/cement columns

Table 4.3 shows the parameters used for the LC columns. The parameters used are collected from Bozkurt et al. (2023).

Table 4.3: Parameters used for the deep mixing columns in Plaxis 2D.

Parameter	Value	Unit
$\gamma_{sat/unsat}$	17	kN/m^3
E_{50}^{ref}	67 200	kN/m^2
E_{oed}^{ref}	67 200	kN/m^2
E_{ur}^{ref}	180 000	kN/m^2
ν_{ur}	0.2	-
c'_{ref}	20	kN/m^2
k_x, k_y	Equal to surrounding clay	m/day
ϕ'	37	°
POP	200	kN/m^2
K_0^{nc}	default	-

4.3.3 Embankment

The parameters for the embankment was partly obtained from the project Väg 160 and the remaining parameters which could not be obtained from the project were collected from Castro et al. (2014), the parameters collected from Castro et al. (2014) were E'_{ref} , ν and c'_{ref} . The parameters used is shown in Table 4.4.

Table 4.4: Parameters used for the the embankment in Plaxis 2D.

Parameter	Value	Unit
γ_{unsat}	19	kN/m^3
γ_{sat}	22	kN/m^3
E'_{ref}	40 000	kN/m^2
ν	0.3	-
c'_{ref}	10	kN/m^2
φ'	34	°
k_x, k_y	Default	m/day

4.3.4 Parameters for site Väg 160

In the table below the parameters derived for the clay layer used for the site at Väg 160 is presented. In Table 4.6 the derived values for the friction layer is presented. However, c'_{ref} , E_{50} and ν_{ur} were collected from Khan and Abbas (2014), since it was not possible to obtain these from the project.

Table 4.5: Parameters used for Clay 1 and Clay 2 in project Väg 160 in Plaxis 2D.

Parameter	Clay 1	Clay 2	Unit
$\gamma_{unsat/sat}$	16	16	kN/m^3
λ^*	0.148	0.273	-
κ^*	0.019	0.016	-
μ^*	0.002	0.002	-
ν_{ur}	0.15	0.15	-
c'_{ref}	5.33	5.33	kN/m^2
φ'	30	30	°
k_x, k_y	$1.58 * 10^{-4}$	$2.07 * 10^{-4}$	m/day
K_0^{nc}	Default	Default	-
POP	35	-	kN/m^2
OCR	-	1.9	-

4.3.5 Parameters for site Östra berg

In Table 4.7 the parameters used for the Soft Soil Creep model for Östra berg is shown. All parameters have been derived from site investigations. In Table 4.8 the parameters used for the friction layer in Östra berg is presented. The unit weight

Table 4.6: Parameters used for the friction layer in project Väg 160 in Plaxis 2D.

Parameter	Value	Unit
γ_{unsat}	19	kN/m^3
γ_{sat}	22	kN/m^3
E'_{ref}	30 000	kN/m^2
ν	0.3	-
c'_{ref}	1	kN/m^2
φ'	34	°
k_x, k_y	0.5	m/day

Table 4.7: Parameters used for Clay 1, 2 and 3 in project Östra berg in Plaxis 2D.

Parameter	Clay 1	Clay 2	Clay 3	Unit
$\gamma_{unsat/sat}$	16.5	16.5	16.5	kN/m^3
λ^*	0.234	0.195	0.205	-
κ^*	0.012	0.012	0.011	-
μ^*	0.002	0.002	0.002	-
ν_{ur}	0.15	0.15	0.15	-
c'_{ref}	5.75	5.75	5.75	kN/m^2
φ'	30	30	30	°
k_x, k_y	$5.52 * 10^{-5}$	$1.12 * 10^{-4}$	$5.96 * 10^{-5}$	m/day
K_0^{nc}	0.5	0.5	0.5	-
OCR	3.3	3.0	3.1	-

both saturated and unsaturated and the friction angle have been derived from the site investigations while the rest are from Khan and Abbas (2014).

Table 4.8: Parameters used for the friction layer in project Östra berg in Plaxis 2D.

Parameter	Value	Unit
γ_{unsat}	20	kN/m^3
γ_{sat}	20	kN/m^3
E'_{ref}	30 000	kN/m^2
ν	0.3	-
c'_{ref}	1	kN/m^2
φ'	38	°
k_x, k_y	0.5	m/day

5

Results

In this chapter the results from the numerical analysis will be presented. The results are based on the vertical displacement over time, in which the embankment is set to consolidate and creep. Included results are the difference in stone columns and deep mixing, the differences of the behaviour in the two sites, as well as the impact of different embankment heights and cc-spacing.

To be able to compare how the two ground improvement techniques performed in the different sites and compared to each other, one embankment height and one cc-spacing have been chosen, when it is relevant. In this case the selected embankment height is 2m and a cc-spacing of 1.8m was chosen. Hence, when the impact of different embankment heights and cc-spacing is analysed a cc-spacing of 1.8m and embankment height of 2m is chosen respectively as well. Lastly the results from the sensitivity analysis is presented. All the results will be discussed in Chapter 6.

5.1 Embankment without improvement

Initially a simulation was made with no ground improvement to get a baseline and to identify the need of ground improvement when implementing different embankment heights. As shown in the Tables 5.1 and 5.2, there is no need of ground improvement then using a 1.5m high embankment, according to the demands of total settlements from the Swedish transport administration (Karlsson & Moritz, 2016) of 30 cm settlements for the reference velocity of 100-120 km/h. When implementing the other embankment heights, on the other hand, ground improvement is necessary. Further, the initial calculations shows that Väg 160 is more prone to settle when no ground improvement is used.

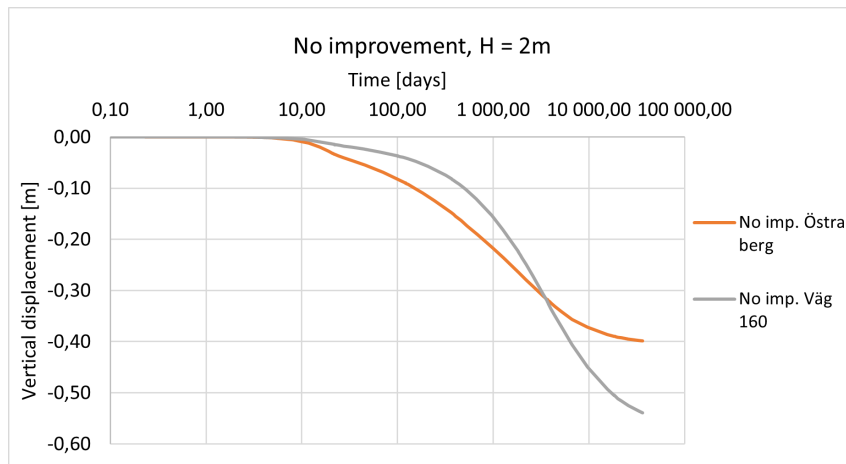
Table 5.1: Vertical displacement of embankment without improvement in Östra berg.

Embankment height [m]:	1.5	2	3	5
Vertical displacement [m]:	0.275	0.399	0.655	1.2

Table 5.2: Vertical displacement of embankment without improvement in Väg 160.

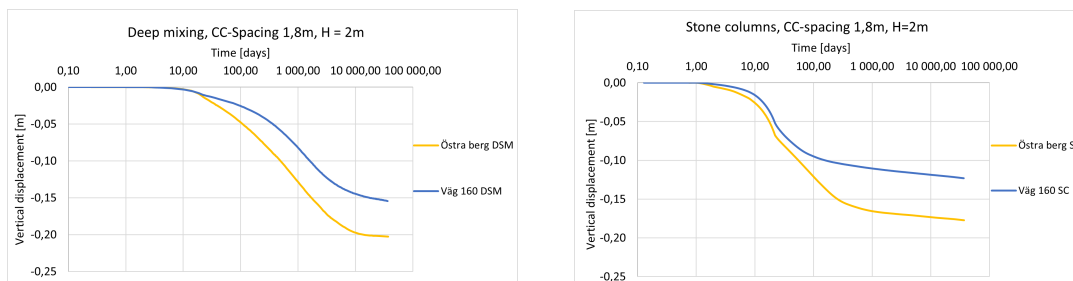
Embankment height [m]:	1.5	2	3	5
Vertical displacement [m]:	0.292	0.54	1.02	1.988

In Figure 5.1 it is shown how the vertical displacement progress over time. The soil in Väg 160 has larger total settlements after 100 years of consolidation, however the site in Östra berg settles faster, creating larger vertical deformation in the near future after construction.

**Figure 5.1:** Comparison of 2m embankment without improvement in Östra berg and Väg 160.

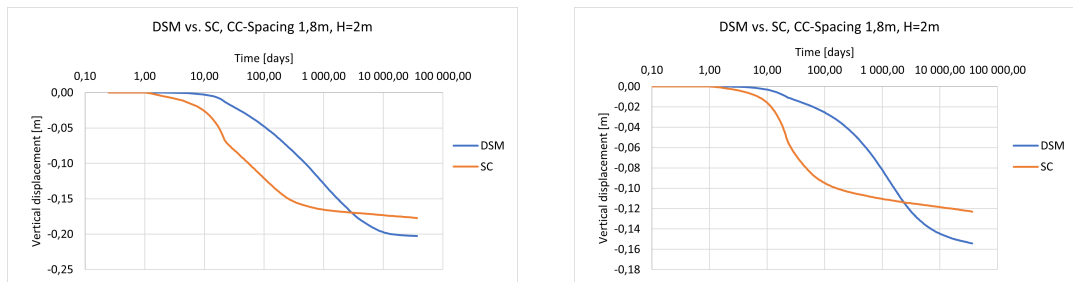
5.2 Differences in the two sites

In Figure 5.2 a comparison between the two sites are presented. Both deep mixing and stone columns reduces the settlements more in Väg 160. To strengthen the case even further, improvement factors have been evaluated. The improvement factor is defined as the ratio between the final settlements with and without improvement. In Östra berg the improvement factors are 2 and 2.3 for deep mixing resp. stone columns, corresponding numbers in Väg 160 are 3.5 resp. 4.5.

**(a)** Deep mixing.**(b)** Stone columns.**Figure 5.2:** Comparison between Östra berg and Väg 160 implementing deep mixing and stone columns respectively.

5.3 Stone columns vs. Deep mixing

The differences of the simulations with deep mixing and stone columns are presented in Figure 5.3. The same behaviour can be seen in both Östra berg and Väg 160, that the stone columns contributes to a larger settlement reduction after 100 years of consolidation. It is clear from Figure 5.3 that the time of consolidation has a large impact on the outcome when comparing deep mixing to stone columns. For stone columns 70-80% of the settlement occurs after only 100 days of consolidation, compared to deep mixing which only develops 15-30% of the total settlements in the same time.

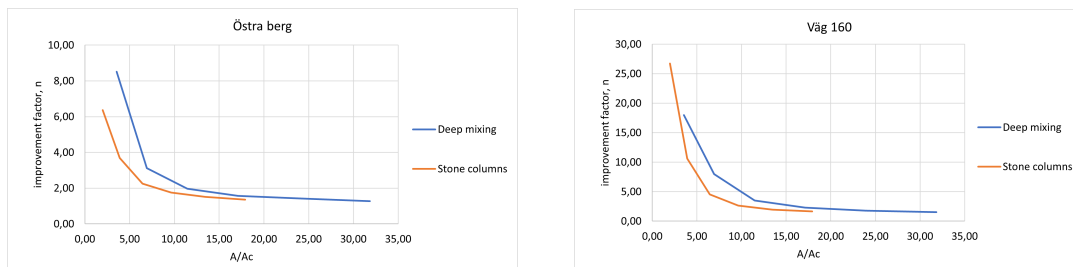


(a) Östra berg.

(b) Väg 160.

Figure 5.3: Comparison between deep mixing and stone columns in Östra berg and Väg 160.

Since the chosen diameter is smaller for the deep mixing columns compared to stone columns, the improvement factor have been plotted against the area ratio, (A/A_{col} - ratio), being the ratio between the total area of the unit cell and the area of the column. The improvement factor is based on the final settlements after 100 years of consolidation. In contrast to when looking at Figure 5.3, in Figure 5.4, deep mixing appears to be the option with best improvement, when the same area ratio is applied.



(a) Östra berg.

(b) Väg 160.

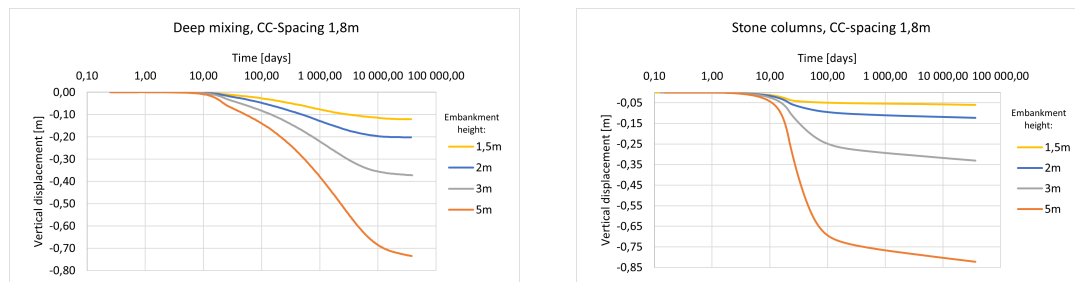
Figure 5.4: Improvement factor based on the ratio between the area of the column and the total area of the unit cell.

5.4 The impact of different embankment heights and cc-spacing

In this section the effect of different embankment heights and cc-spacing is presented. As described in Chapter 4, all combinations of four different embankment heights and six different cc-spacings, have been simulated with deep mixing and stone columns in the two sites Östra berg and Väg 160.

Since several results were similar to each other, a selection of diagrams has been chosen to be presented in this chapter which are shown in Figures 5.5 and 5.6 with accompanying sub-figures. The remaining Figures are presented in Appendix B.

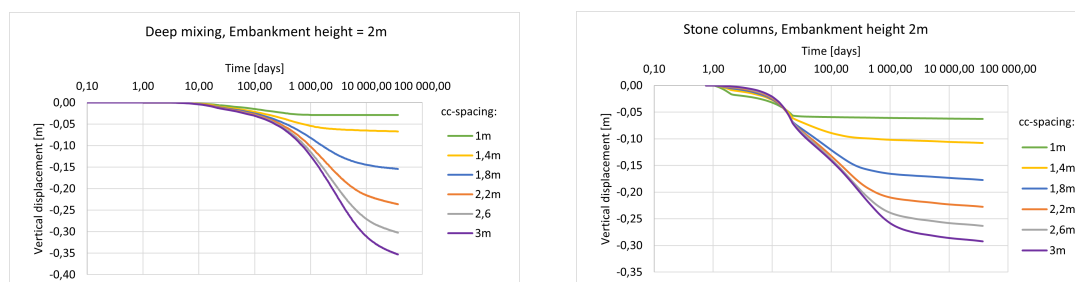
The only result that stood out were the impact of cc-spacing when stone columns were used, which can be seen in Figure 5.6b. It is shown that a smaller cc-spacing leads to the vertical displacement developing faster.



(a) Deep mixing in Östra berg.

(b) Stone columns in Väg 160.

Figure 5.5: The impact of different embankment heights in Östra berg and Väg 160.



(a) Deep mixing in Väg 160.

(b) Stone columns in Östra berg.

Figure 5.6: The impact of different cc-spacings in Väg 160 and Östra berg.

5.5 The impact of creep

To find the impact of creep additional simulations were conducted using the Soft Soil model to see how the final settlements were affected. In these simulations the same input data were used as for the other simulations but without the creep as it

is not an input parameter for the SS model. From these simulations it was found that the creep is approximately 40-50% of the total settlements in Väg 160, with exception for 1m cc-spacing. In Table 5.3 the impact of creep is shown in percent. Without ground improvement the creep showed to be 57% of the total settlements for Väg 160.

Table 5.3: Percentile impact of creep on the vertical displacement

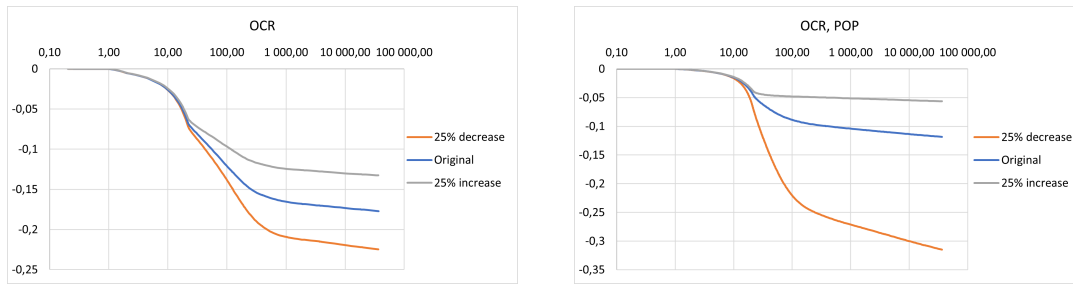
CC-spacing [m]	Creep [%]	
	Deep mixing	Stone columns
1	19	8
1.4	38	40
1.8	49	45
2.2	49	40
2.6	51	45
3	53	48

5.6 Sensitivity analysis

In order to assess the influence on the vertical displacements that the different parameters used in the soil model have, a sensitivity analysis was performed. The sensitivity analysis was conducted for both sites with cc-spacing 1.8m and embankment height 2m for both DSM and SC. Similar results were found for each of the parameters using both DSM and SC, hence only one case is presented for each parameter in this section, the remaining results can be found in Appendix C.

5.6.1 Sensitivity to OCR and POP

Figure 5.7 shows the achieved results from the sensitivity analysis on OCR and POP. It is clear that the model is sensitive to the preconsolidation pressure and further to the OCR and POP values. This, since a 25% increase/decrease shows changes in settlement of around 25% for Östra berg. For Väg 160 changes of -50% and +160% for the settlements are presented for increase/decrease, respectively. Similar results were found for DSM as ground improvement, see Appendix C.



(a) Östra berg.

(b) Väg 160.

Figure 5.7: The influence of OCR and POP on vertical settlements using stone columns as ground improvement.

5.6.2 Sensitivity to the modified compression and swelling index

In Figure 5.8 the results from the analysis of the modified compression index, λ^* , is presented. For the site Väg 160 a change of 25% resulted in a change in settlements of 5-6%. For the Östra berg site the settlement is increased a bit further and a decrease and increase by 25% results in a change in settlements of -12% and +7%, respectively for the stone columns.

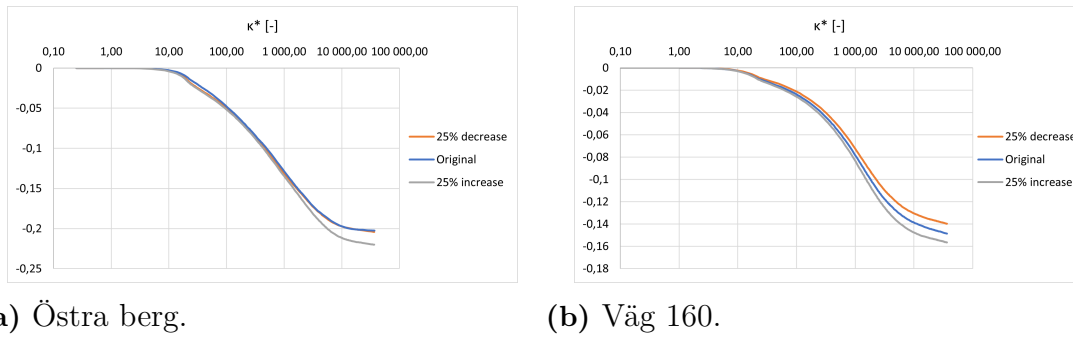


(a) Östra berg.

(b) Väg 160.

Figure 5.8: The changes in vertical displacements after a 25% increase and decrease of the modified compression index, λ^* , when using SC as ground improvement.

The impact of the modified swelling index, κ^* is shown in Figure 5.9. Here, changes around $\pm 7\%$ when κ^* is increased or decreased by 25%.

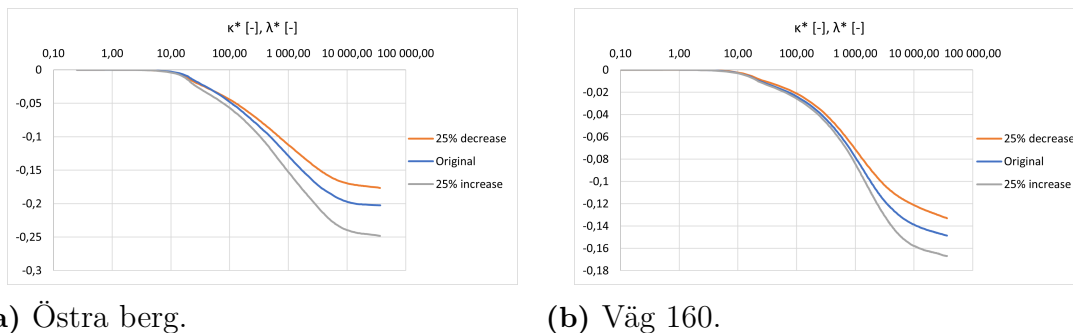


(a) Östra berg.

(b) Väg 160.

Figure 5.9: The impact of κ^* using DSM as ground improvement.

A 25% increase/decrease of κ^* and λ^* is tested, and the combined impact of the two parameters is presented in Figure 5.10. It is clear that the combined change has a larger impact on the vertical displacements. For both methods an increase of 25% at Väg 160 results in approximately 10% increase of settlements and likewise decreases with approximately 10% when λ^* and κ^* is decreased. For Östra berg however, the changes are slightly higher and an increase in κ^* and λ^* results in a increase of approximately 20%.



(a) Östra berg.

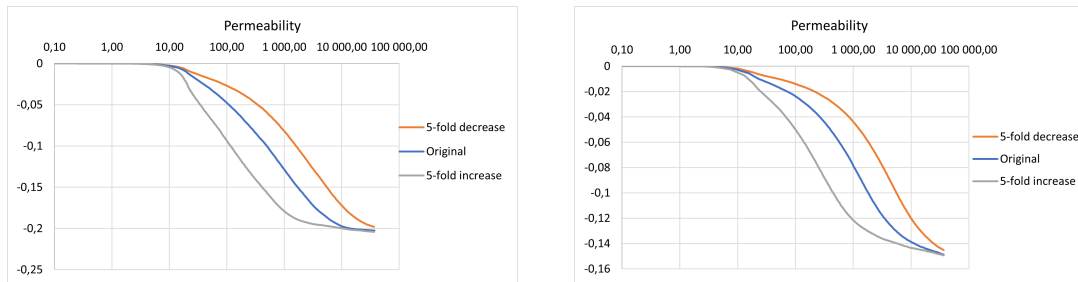
(b) Väg 160.

Figure 5.10: The merged impact of the modified compression and swelling index on vertical displacements with DSM.

5.6.3 Sensitivity to permeability

The impact of the permeability in the soil is shown in this chapter. It is clear that at the site in Väg 160, the permeability does not have a large impact on the results. A five-fold increase/decrease in decrease results in settlement changes of 2% or lower. For the site in Östra berg the permeability has a slightly larger impact and show settlement changes of up to 6%, see Figure 5.11.

However, it is clear when looking at the graph in Figure 5.11 that the changes in permeability changes when during the consolidation the displacements are developed. When increasing the permeability, the vertical displacements develop earlier and when decreasing it the displacements are developed later.



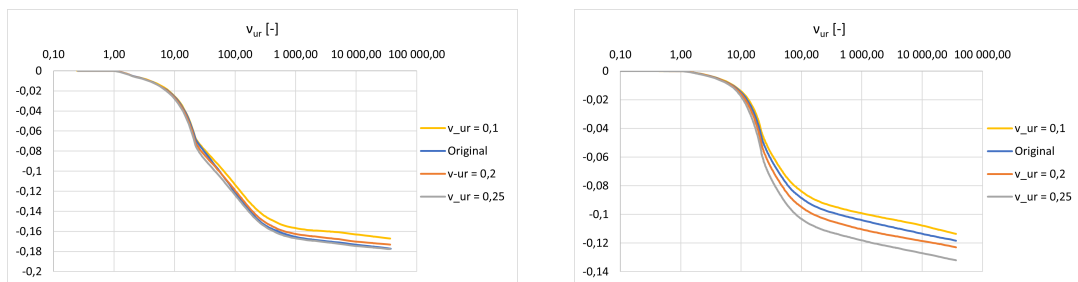
(a) Östra berg.

(b) Väg 160.

Figure 5.11: The influence of a 5-fold change of the permeability, using DSM.

5.6.4 Sensitivity to Poisson's ratio

Poisson's ratio for unloading and reloading shows to have a similar impact at Väg 160 for both DSM and SC, where a increase/decrease of 33% results in a +4/-4% change respectively, whereas a 66% increase of the parameters results in a change in vertical displacement of +11 % change for SC respectively. For the site in Östra berg the results vary between +5 and -5% changes. This is shown in Figures 5.12.



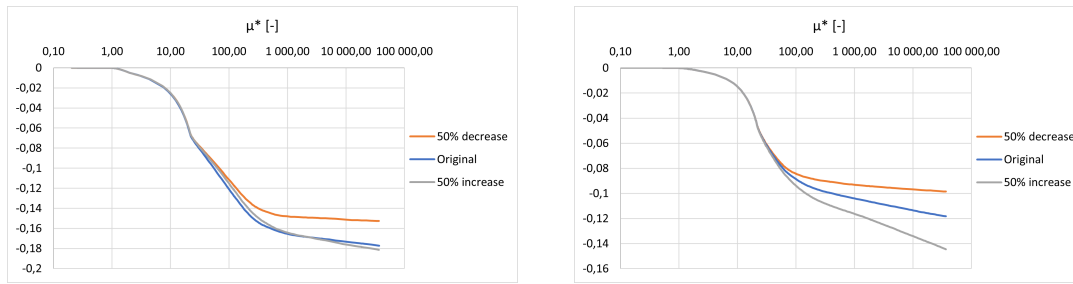
(a) Östra berg.

(b) Väg 160.

Figure 5.12: The impact of Poisson's ratio on vertical displacements, using stone columns.

5.6.5 Sensitivity to modified creep index

The modified creep index is changed by 50% and the impact of it is shown in Figure 5.13. It shows that this parameter has bigger impact on the site at Väg 160, where a decrease give changes in settlement of -17% and a increase shows changes of +22% for SC.



(a) Östra berg.

(b) Väg 160.

Figure 5.13: Impacts of the modified creep index, μ^* , when using stone columns.

5.7 CO_2 calculations

The emission calculations are limited to only look at the CO_2 – *equivalents* for the material while transport and installation etc. are excluded. All the calculations have been made on a single column and the column dimensions in Väg 160 was used. To calculate the emissions, the mass of the column was multiplied with emission factors from the carbon calculator from the Swedish Transport Administration (Trafikverket, 2024). The emission calculations shows that the deep mixing columns emits approximately $6133.3 \text{ kg}CO_2 - eq$ which is roughly 173 times as much $CO_2 - eq$ as the stone columns, which emits about $35.4 \text{ kg}CO_2 - eq$ (Trafikverket, 2024). The calculations can be found in Appendix D.

6

Discussion

In Figure 5.1 the results from the settlement predictions when no improvement is installed are presented. It is shown that the site in Väg 160 is more prone to generate large settlement, however the site in Östra berg settles faster. One of the main reasons that it is less settlements in Östra berg is likely because of the over consolidation ratio. It is known that the clay in Östra berg has a higher OCR compared to the clay in Väg 160, hence have a lower compressibility, and according to the sensitivity analysis, OCR is a parameter that the model is sensitive to. Another parameter that contributes to the settlements being larger in Väg 160 is the modified compression index, λ^* , which is known to have a larger value in Väg 160. Which also indicates that the site in Väg 160 have a larger compressibility.

In contrast to the result when no improvement is implemented, in Figure 5.2 it is shown that when lime/cement columns or stone columns are installed, it is the site in Väg 160 that generates smaller vertical deformation. This result is however unexpected since the differences of parameters in the sites indicates that it should be the other way around, as for the case without ground improvement. This may be due to multifaceted relationships between pore water pressure, preconsolidation pressure and the emerging stiffness. There is no significant difference of the rate in which the settlements develop. This is probably because the ground improvement has a larger effect on the rate of consolidation, rather than what site is considered.

Moreover, in Figure 5.4 where improvement factors are plotted against area ratio it is shown that for the case with deep mixing there is a higher improvement factor when the same area ratio is applied. With a larger difference for smaller cc-spacing, especially in Östra berg. In Figure 5.3 where vertical displacement is plotted against time it seems that the stone columns perform better as a ground improvement technique since it gives smaller total settlements, hence would be the better alternative, but the reason for that is because the diameter is larger. Looking at the improvement factor compared to area ratio instead, deep mixing performs better, this is probably because the stiffness is higher in the LC-columns. Nevertheless, the stiffness values for the LC-columns in this thesis are significantly higher than those recommended by the Swedish design guidelines, implying that the results might be reversed if the guideline values were applied. Despite the findings of this thesis, stone columns can still be regarded as the better choice, although they require a greater quantity of material to achieve the same level of improvement. Additionally, since the stone columns have a much smaller impact on the environment, this could be preferable anyways.

The most significant difference between deep mixing and stone columns are the rate at which settlements develop. In Figure 5.3 it is clearly evident that when stone columns are used the settlements take place much earlier in the consolidation phase in comparison to when deep mixing is used. The reason for this is simply because the material in stone columns and how they are constructed give rise to a very high value for the permeability. The permeability in the stone columns are about $4 * 10^6$ times the permeability of the lime cement columns and the surrounding soil, thus, limiting the post-construction settlements.

When analysing the impact of different embankment heights and cc-spacing only one case stood out. Which was for the case with stone columns, when it was observed that the rate of consolidation was affected by the cc-spacing used. A smaller cc-spacing seemed to slow down the consolidation rate, until the embankment was fully constructed. The reason for this is because when a smaller cc-spacing is used the ratio of stone column material versus clay is larger, hence, a bigger part of the ground has a higher permeability. This is more clearly showed from the results in Östra berg due to the lower permeability in the clay, compared to Väg 160, but the same behaviour is noted in Väg 160 as well.

6.1 Sources of error due to assumptions and choices

One main obstacle when conducting this study is the parameter determination for PLAXIS 2D. In order to use the more advanced models laboratory data of both one and multidimensional stress conditions are required. Due to a lack of these types of test in the projects used, parameters such as μ^* and ν_{ur} had to be assumed. In addition oedometer-test using incremental loading would have been a trustworthy test to use when checking the compression behaviour in the soil. However, only CRS oedometer-tests were conducted for these sites.

The preconsolidation pressure were later shown, in the sensitivity analysis, to have a large impact on the vertical displacements. These two parameters could have had a great impact on the final settlements. Based on this, it would have been preferable to have more tests to rely on. On the other hand the same uncertainties are present for both SC and DSM.

Additionally, from the parameter sensitivity analysis it was found that the permeability had a large impact on at which rate the vertical displacements developed, especially for the LC-columns. This is likely because a lower permeability slows down the rate of consolidation leading to the settlements developing slower, and the other way around if the permeability is increased. For the stone columns the effect is not as significant since the stone columns have a permeability many magnitudes higher than the surrounding soil, so it is reasonable that a 5-fold increase or decrease in the clay layers would not change the outcome as much. It was also noted that the permeability was not significant for the final settlements since the change in final settlements were really small.

Further, another obstacle in the numerical analysis was the inability to model the limitations of the stone columns. It was not possible to model the fact that the aggregates in the stone columns could drift away in the soft clay if the clay is sensitive enough. Apart from this we chose not to model the installation effects of the stone columns either. Since the installation effects, according to the findings in the literature study, mainly lead to the soil around the columns being improved, this could have effected the improvement with stone columns negatively.

The soil model employed in the numerical analysis is another factor that might have influenced the results. The SSC model was utilized, and to determine the effect of creep on the vertical displacements, analyses with the SS model were also performed. It was shown that the creep accounted for roughly 57% of the settlements when no improvement were implemented and approximately 40-50% when ground improvements were applied. However, for a 1m cc-spacing, it was shown that the impact of creep was less significant, being under 20%. This indicates that the creep has a substantial effect on the final results. Conversely, the effect of creep was similar for both methods, suggesting that it should not significantly impact the results when comparing DSM and SC.

From the conducted $CO_2 - eq$ calculation it is clear, based on the parameters included, that the stone columns are far more sustainable. Even though the calculations made were simplified it is still safe to say that the stone columns contributes to less CO_2 -equivalents than the lime/cement columns. Additionally, the lime/cement columns does often require longer transports than the stone columns, since it quite easy to find aggregates nearby the construction site, which indicates that the difference in $CO_2 - eq$ between the two methods would be even greater if transport would be included in the calculations. Also the fact that it is possible to use recycled aggregates in the stone columns is a good indicator for this.

6.2 Further investigations

From the results and discussion of this thesis some further studies are recommended to furthermore compare the deep mixing method to the vibrated stone columns. The further investigations are mainly based on the limitations of this thesis, but there are also some suggestions based the sources or error regarding assumptions etc., that this thesis contains.

A more detailed numerical analysis could be conducted where, for example, a more advanced soil model is used. This could show how for example the sensitivity in the soil affects the performance of the ground improvement methods, since a more advanced soil model, such as Creep-SCLAY1S consider more parameters, than Soft Soil Creep that was used in this thesis. This would be interesting since, like it was described in the literature study, the stone columns have been proved to preform poorer in sensitive soil and quick clay. Further, it would be interesting to include

encased stone columns in the study as well, to be able to compare these with DSM and regular stone columns. This would eliminate the disadvantage of stone column that occurs in fine soils when clay particles get clogged around the column, reducing radial drainage. This type of analysis would furthermore enhance the understanding of in which condition what method is better suited.

Additionally, to be able to compare the real improvement effect of stone columns and deep mixing one would have to include the installation effects of the methods in the numerical analysis. As mentioned in Section 2.3.3 in the literature study, the installation can have both positive and negative effects regarding how much and in what way the ground is improved.

A numerical analysis in 3D instead of 2D could show how the columns works as a group, grid, block etc, as well as differences in rectangular or triangular pattern. That would show further how the different applications of the columns perform against each other and give a better understanding of which method to use for different cases.

7

Conclusion

The ground improvement methods deep soil mixing and stone columns were reviewed from literature, and through a numerical analysis. From the numerical analysis one can tell that it is not the ability to reduce final settlements that is the main difference, but how the two types of columns effects the time of consolidation. Hence, how fast the majority of the total settlements occur. When stone columns are used it is shown that the vertical displacements develop faster, which means that less settlements are created post-construction, which is preferred.

In the initial results stone columns seem to reduce the settlements more, which is due to their larger diameter. However, when comparing the two methods improvement factors against the area ratio it can be concluded that the deep mixing columns are more effective for reducing final settlements.

As for the methods main advantages and limitations from the literature review. The stone columns have a clear advantage over the deep mixing regarding the environmental impact. While the main disadvantage with the stone column method is the lack of reference projects where stone columns have been successful and also the lack of experience with the method in the sector. This on the other hand would be the deep mixing's biggest advantage; that it is a widely known and proven to be successful in many projects in Sweden and also worldwide.

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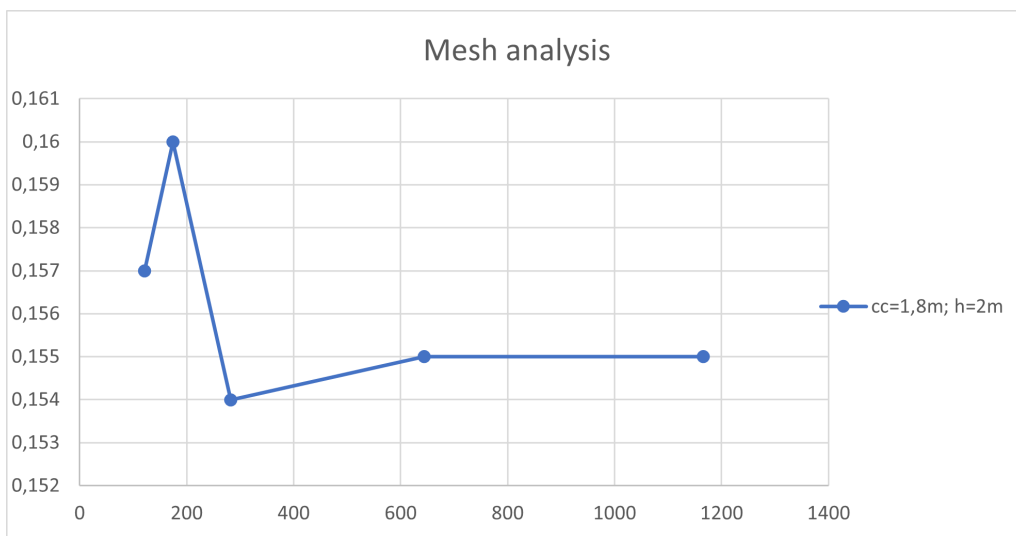
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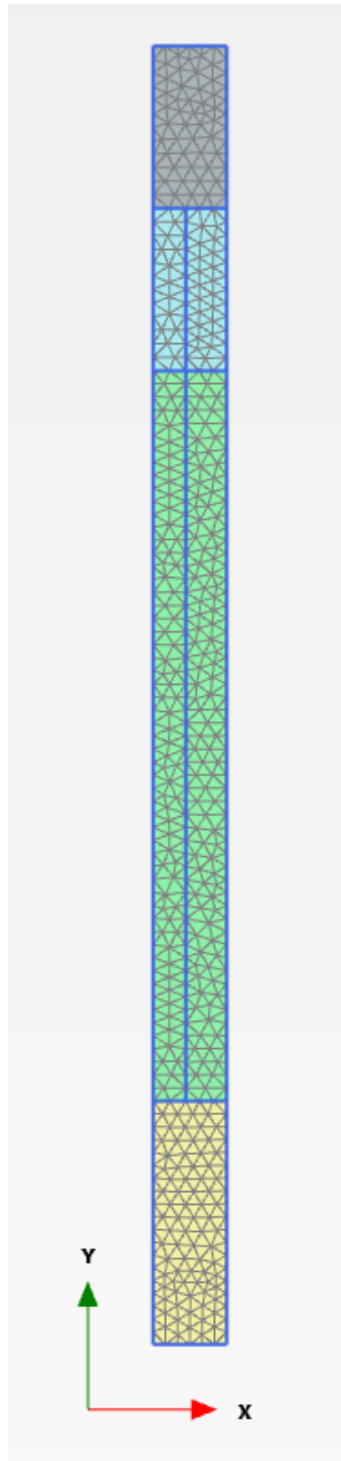
A

Appendix A

A.1 Mesh analysis



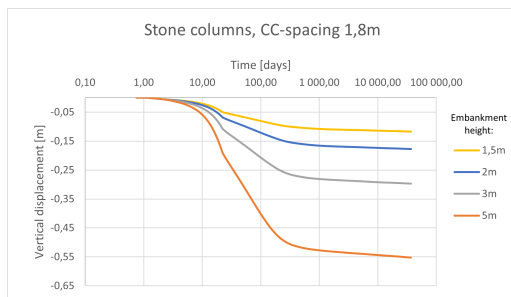
Final mesh



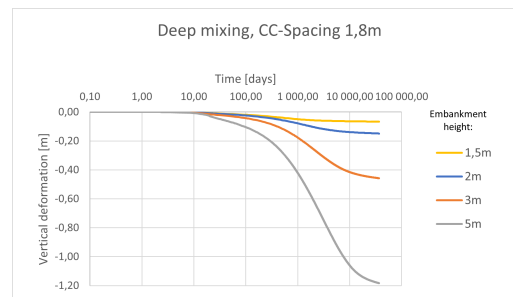
B

Appendix B

The impact of different embankment heights in Östra berg and Väg 160



(a) Stone columns in Östra berg.

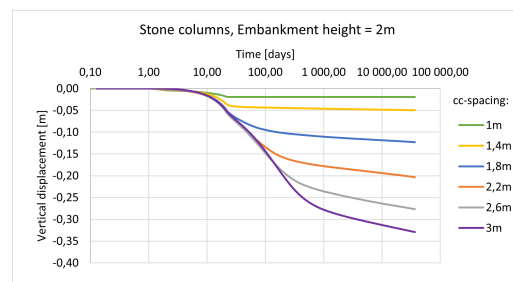


(b) Deep mixing in Väg 160.

The impact of different cc-spacings in Östra berg and Väg 160



(a) Deep mixing in Östra berg.



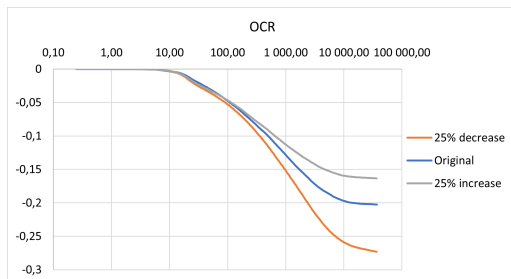
(b) Stone columns in Väg 160.

C

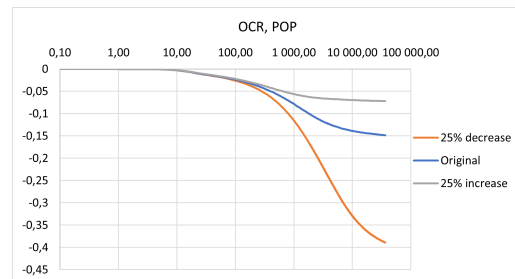
Appendix C

Sensitivity analysis

The impact of a 25% increase/decrease of OCR and POP using deep mixing columns as ground improvement.

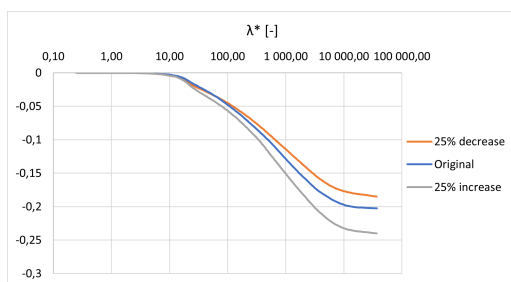


(a) Östra berg.

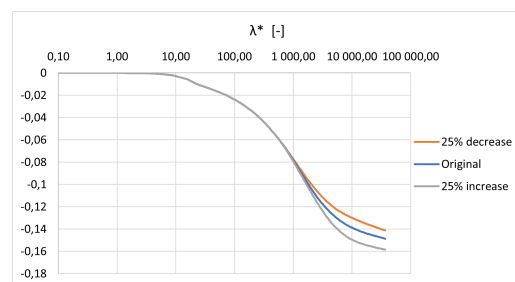


(b) Väg 160.

The impact of λ^* on settlements using DSM.

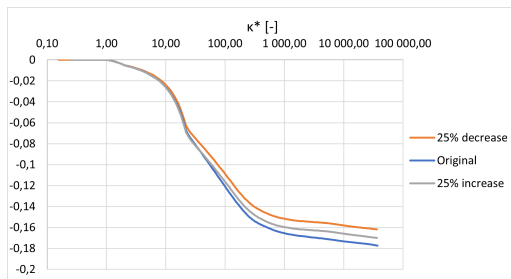


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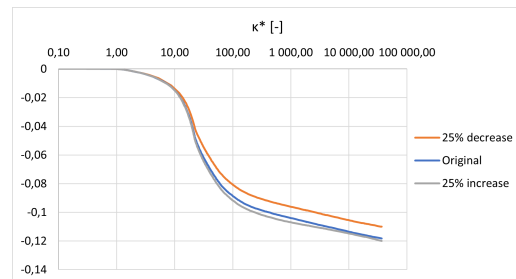


(b) Väg 160.

The influence of the modified swelling index, κ^* , using SC.

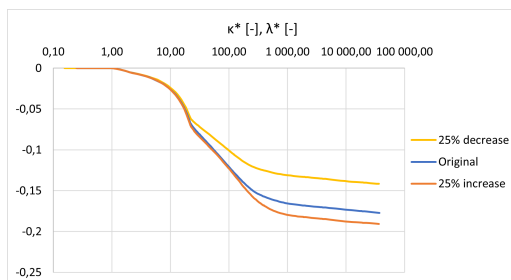


(a) Östra berg.

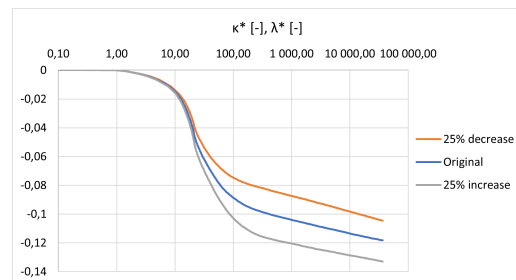


(b) Väg 160.

The combined influence of λ^* and κ^* on vertical displacements using SC as ground improvement.

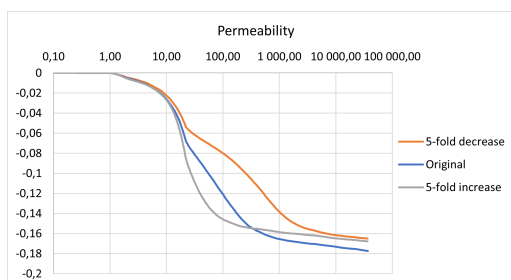


(a) Östra berg.

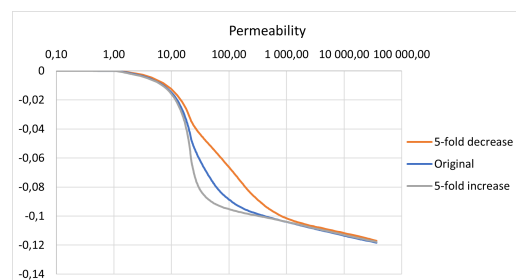


(b) Väg 160.

The permeability's impact on settlements, using SC.

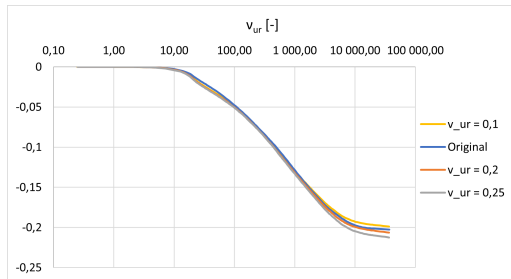


(a) Östra berg.

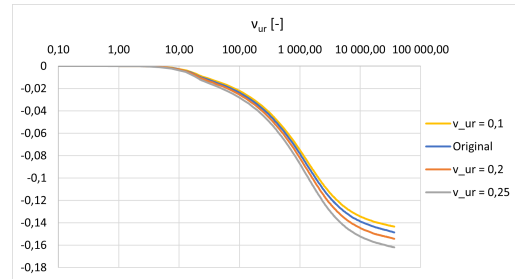


(b) Väg 160.

How changes in Poisson's ratio influences settlements when using deep mixing columns.

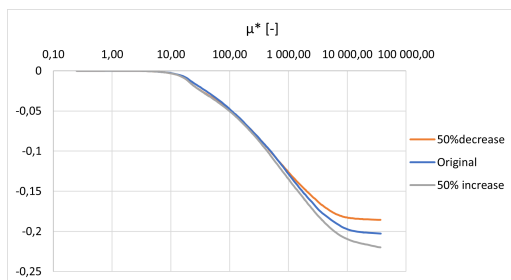


(a) Östra berg.

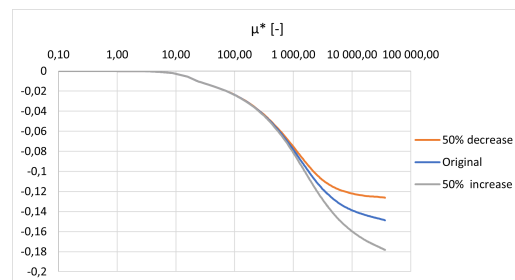


(b) Väg 160.

Influences of μ^* when using DSM.



(a) Östra berg.



(b) Väg 160.

D

Appendix D

CO₂ Calculations

CO2 calculations:		
<u>Lime/Cement Columns</u>		
r =		0,3 m
h =		11 m
V =		3,1 m ³
ρ =		1,7 t/m ³
m =		5287,3 kg
Lime (emission factor) =		1,45 kg co ₂ e/kg
Cement (emission factor) =		0,87 kg co ₂ e/kg
Emission =		6133,3 kg co ₂ e/column
<u>Stone Columns</u>		
r =		0,4 m
h =		11 m
V =		5,5 m ³
ρ =		1,6 t/m ³
m =		8846,7 kg
Aggregate (emission factor) =		0,004 kg co ₂ e/kg
Emission =		35,4 kg co ₂ e/column