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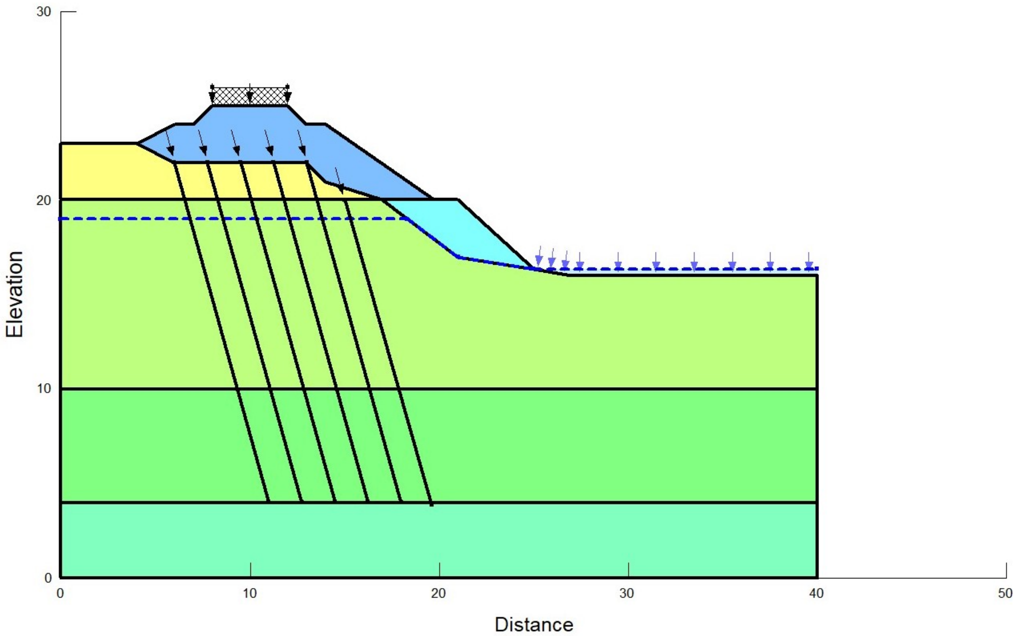

Slope Stability Study at Lärjeån

A stability study to evaluate the risk of a landslide occurring for a part of Angeredsbanan located at Lärjeån

Master's thesis in Infrastructure and Environmental Engineering

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Master's thesis ACEX30
Gothenburg, Sweden 2021

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Cover: An illustration of the model from SLOPE/W.

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Abstract

The purpose of this Master's thesis is to conduct a slope stability study regarding the risk of a landslide occurring, using different calculation methods and a sensitivity analysis. The calculations are performed using the Direct method and SLOPE/W. Historical research will be done on how the slope was constructed, together with its effect on the stability. Based on the slope stability as it is today, various factors that could affect the stability will be studied to find the most critical changes that could trigger a landslide. Additionally, actions that can be taken to secure the slope will further be evaluated.

The slope stability is investigated for two sections (840/875 and 875/886) of Angeredsbanan which is located on a slope, leading down to Lärjeån. The data that is evaluated is retrieved from field tests conducted by Sweco at the request of Gothenburg city.

For both the Direct method and SLOPE/W, the calculated factors of safety are compared with the required factors of safety which are set up by the Commission on slope stability. This is done to evaluate if the slope is safe or not. For the Direct method, three historical scenarios are set up as different models. The results show that the slope is unsafe for all scenarios. In SLOPE/W, five historical scenarios are set up. The results show that when excavating the natural slope it loses some of its stability, going from a stable to an unstable slope. Further piles were inserted and the slope increased its stability again, reaching the required values for the undrained analysis for both sections. Lastly, the construction of the embankment was performed which resulted in a slight reduction of the stability, still reaching the required factor of safety for the undrained but not for the combined analysis.

When performing a sensitivity analysis to see what possible changes that the slope stability is most sensitive towards, the natural changes: groundwater level, water level, erosion and the human changes: embankment height, slope angle, pressure bank and tram load were studied. The results show that the slope is most sensitive towards erosion when studying the factor of safety and the rate of change, meaning that this factor is most likely to trigger a landslide on the site.

Comparing the different actions that could be done to secure the slope, the most profitable implementation would be to add a pressure bank. This is mainly due to the relatively easy implementation process connected with the flexible geometry and usage of material together with the high impact on the stability.

Keywords: Slope stability, Angeredsbanan, Lärjeån, Direct method, SLOPE/W, Sensitivity analysis, factor of safety.

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ALICE JOHNSSON & IRMA MAGNUSSON, Gothenburg, June 2021

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1 Introduction

The event of a landslide is today a worldwide problem causing death to human lives, making people lose their homes and contributing to enormous losses of money (Sassa & Canuti, 2009). The costs will both include the direct costs which are the costs needed to rebuild and replace damaged facilities and the indirect costs which are connected to the lost productivity and disturbance of utility and transportation systems. Between the years 1903 - 2004, America had above 200 000 fatalities and spent around 1300 million dollars on damages due to landslides. Between the same years, Europe had above 150 000 fatalities and spent around 1700 million dollars on damages due to landslides which shows the severe consequences. In Sweden, about five percent of the land area is made out of clay and four percent of these soils are prone to cause landslides (SGU, 2020). In Figure 1.1 the tendency of a landslide in Sweden to occur is shown. The figure shows that the west coast is part of one of the highest risk areas in the country and consists of mainly clay soils. Accidents caused by landslides cost about 200 million Swedish crowns a year and a lot of infrastructures are affected since the constructions often are placed on clay soils.

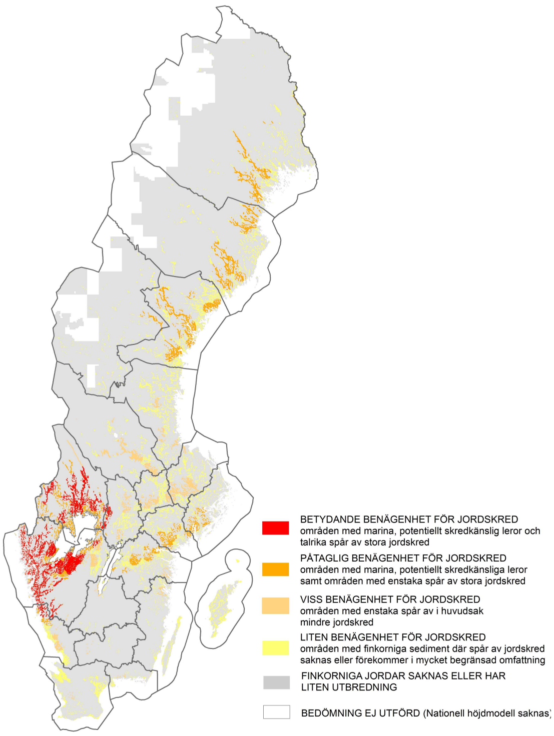


Figure 1.1: National overview in Sweden of the tendency of landslides in fine-grained soils. Note. SGU. (2020). Riksöversikt över finkorniga jordars skredbenägenhet. Retrieved from www.sgu.se/samhallsplanering/risker/skred-och-ras/riks oversikt-over-finkorniga-jordars-skredbenagenhet/

When infrastructure is placed on clay soils, it is necessary to evaluate the risk of a landslide occurring. The occurrence of a landslide is due to the fact that a landmass loses its state of equilibrium, causing movement of the mass (SGI, 2019; SGI, 2012). The disturbance of the equilibrium state can depend on various factors such as erosion, changes in land use, climate change, changes in the water level in connection with the soil and the presence of quick clay. To further anticipate where the risks for landslides are higher and to prevent the slide from happening, extensive site investigations and calculations have to be done. This will provide information about soil properties, stability and how exposed the area is for the risks leading to a landslide. Something that is of high importance to avoid damages.

In the future, problems concerning both climate change and change in land use can be expected (Australian Academy of Science, 2021). The population is steadily increasing, which increases the need to expand the infrastructure and the housing on earth, something that will lead to that unprofitable areas with less good stability may be needed to be used for this purpose. The hotter climate and more frequent rains will also contribute to higher risks for a landslide since this to a high extent affects the erosion and the stability of the soil. Those aspects as previously mentioned, will increase the risk for the soil to lose its equilibrium and cause a landslide.

1.1 Background

The area around Göta Älv is located at the west coast of Sweden and is highly sensitive. A landslide in the area would lead to catastrophic consequences (SGI, 2018). Important societal functions like access to drinking water, roads, railways and electricity supply are a few functions that would be highly affected. A contributing factor to the high risk of landslides is the presence of quick clay and the erosion happening in the Göta Älv river. In the area of Göta Älv, Lärjeholm and the stream Lärjeån is located which is the area of interest for this thesis. If a landslide would happen here, it would have dire effects on the infrastructure and could potentially affect human lives. In history, there has been six larger landslides in the area of Göta Älv since the 12th century, where one of these was located at Lärjeån (SGU, w.y.).

At the site which is investigated for this thesis, the tramway Angeredsbanan is located near the stream Lärjeån. The project of the tramway is under Trafikkontoret´s (TK) responsibility, where other consulting companies help with the geological investigations to evaluate the safety for the tramway. The purpose of the investigations is to analyse the slope in connection with the tramway which further leads down to Lärjeån, in order to determine the risk of a landslide, which factors that could cause a landslide and what efforts that can be done to prevent it from happening.

1.1.1 Angeredsbanan

Angeredsbanan is a part of the Gothenburg tram network which stretches between Drottningtorget in the center of Gothenburg and Angereds centrum. This network was the first in Gothenburg to be used as a high-speed tramway (Göteborgs Spårvägar, w.y.). Before 1969 the tramway was used as a railway called Västgötabanan. The railway was mainly used to transfer goods into Gothenburg. After 1969 the railway was closed and rebuilt into the tramway that can be seen on the site today. The tramway was built to meet the increased demand for public transportation in the growing parts of the city. (Historiskt, 2005).

1.1.2 Site description

At the site, Angeredsbanan is located near the stream Lärjeån. The stream empties into Göta älv river, which is located on the west side of the area, see Figure 1.2. In the south-west of the area, allotments are located between the tramway and highway E45. South from the area, the ground is mostly flat and to the east, the landscape consists of more rock. North of the area the district of Hjällbo is located and in the south, the district of Gamlestaden can be found. When studying the embankment where the tramway is located, the chosen area has a very steep slope, which stretches down towards Lärjeån. On the slope, a lot of trees and vegetation can be seen, see Figure 1.3. At the toe of the slope, a resisting embankment reinforcement has been placed. Next to the reinforcement, Lärjeån is flowing with what it seems like, a high velocity. On the other side of Lärjeån, signs of erosion and small landslides can be seen on the slopes stretching down into the stream, see Figures 1.4 and 1.5. Figures 1.3, 1.4 and 1.5 are taken from a site visit and where these motives are located can also be seen in Figure 1.2.

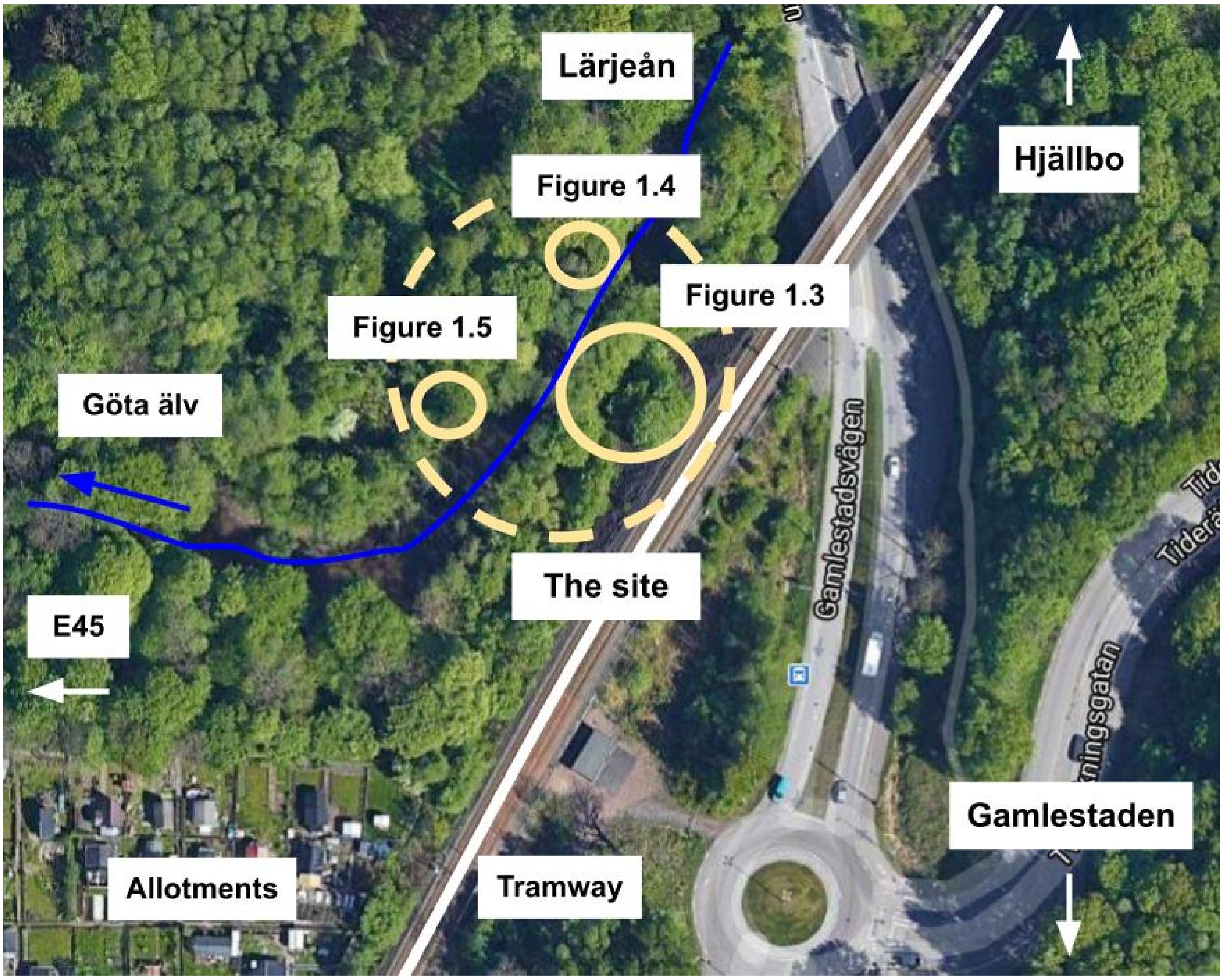


Figure 1.2: Locations for the site, surrounding infrastructure and the pictures taken on site. Modified picture from Google Maps.
Note: Google.(w.y.). Lärjeholm. Retrieved from
www.google.se/maps/@57.7598679,12.008195,219m/data=!3m 1!1e3

Figure 1.3: Photo of the Tramway embankment and the slope leading down to the stream Lärjeån.

Figure 1.4: Erosion of the steambank on the site.

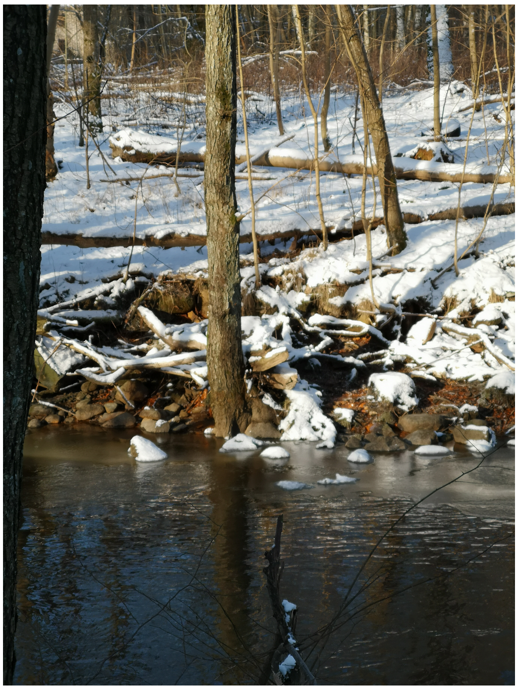


Figure 1.5: Small landslide at the streambank on the site.

To further understand the risks on the site, SGIs mapping of the landslide risk areas along the Göta Älv river is used. From their mapping tool, it can be seen that the area that is being analysed in this thesis has a high risk of a landslide occurring, see Figure 1.6. SGI has assessed that the probability class is at an S5 and that the consequence class is at a K5. The classification system is developed by SGI and is based on their report “Metodbeskrivning sannolikhet för skred: Kvantitativ beräkningsmodell”. The site has the highest class in both probability and consequence, meaning that the risk of a landslide occurring is high and that the consequences if the landslide happens will lead to catastrophic damages.

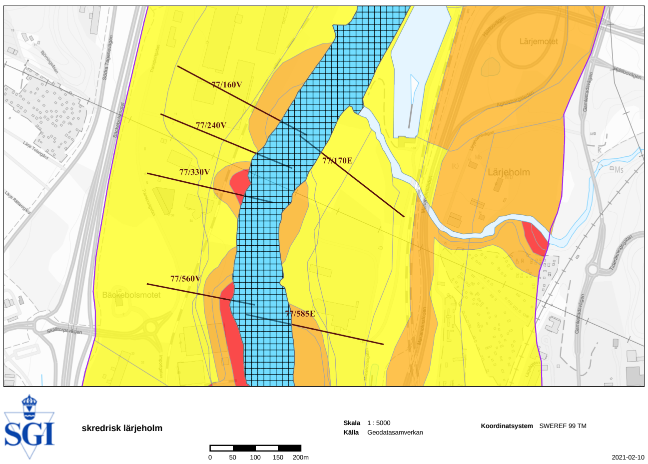


Figure 1.6: Landslide risk mapping from SGI, where red color indicates the highest risk, followed by orange and lastly yellow. The site for this thesis is on the right side in red, indicating that it is a high risk area. Note: SGI. (w.y.). Skredrisk Lärjeholm. Retrieved from gis.swedgeo.se/skredriskkarteringar/

1.2 Aim

The aim of the project is to do a risk assessment regarding the risk of landslides occurring on the site, including the impacts from surrounding events and factors. The factors and surrounding events include information regarding previously mentioned geological properties, site-specific information and tramway properties, which can affect the equilibrium and cause a landslide. Certain factors which will affect the stability on the site will be chosen to see the sensitivity of the stability in the slope when changing those factors. The aim will further be to study different historical scenarios for the development of the slope. When the different scenarios are simulated, a factor of safety will be provided. The factor of safety will show if the landslide is assumed to be unstable. If this is the case, recommendations will be given on how to reduce the risks and ensure the safety of the tramway.

1.3 Limitations

The scope will be limited to the area in Lärjeholm where the tramway is located. The reason for this is to see the effects of a landslide on the tramway and not on how it affects other areas. The stability will both be based on the current situation in the slope and on historical changes in connection with the geometry. Even though the slope will be affected by climate change in the future, these parameters will not be studied in detail but will be discussed on a general level. Expected events in the future will only be based on possibilities and assumptions and could therefore vary from reality.

The risk assessment will only be based on the risk of a landslide to occur and risks including settlements e.g will not be taken into account. When doing the study, cost-benefit analysis will neither be included. However, cost differences can be expected when comparing possible measures to avoid a landslide and the alternative if no measures were taken.

Further, the calculations on the stability will only be based on the direct method and simulations in Slope/W. More extensive simulations in other programs with numerical modeling will not be performed due to the time limitation.

1.4 Research questions

With the aim as a background, following research questions will be answered:

- How have the historical changes of the slope geometry affected the stability?
- How stable is the slope today?
- What factors will be the most critical to trigger a landslide?
- What actions can be taken to secure the slope from failure?

2

Theory

In this chapter, information about the analysed site is described considering both geography, geology, and hydrogeology. Basic information about geotechnical parameters, calculation methods, and the models used for the calculations and simulations are described for further understanding of the analysis. Also landslides that have happened in the past are described and international methods used for slope stability are analysed.

2.1 Slope stability performance

To further get a broader understanding of the problems that can occur due to a landslide, it is important to understand why landslides happen and how the slope stability investigation is done to evaluate the risks.

2.1.1 Reasons for landslide

The normal state, when no landslide occurs and the slope is laying still, is at a state where the soil is at equilibrium (SGI, 2019). Meaning that the driving forces and the resisting forces on the soil are equal and the soil is stable, see Figure 2.1. The stability can over time be changed in the soil due to changes in the soil´s strength and due to additional loads on the soil. The strength of the soil highly depends on different factors and properties in the different soil layers.

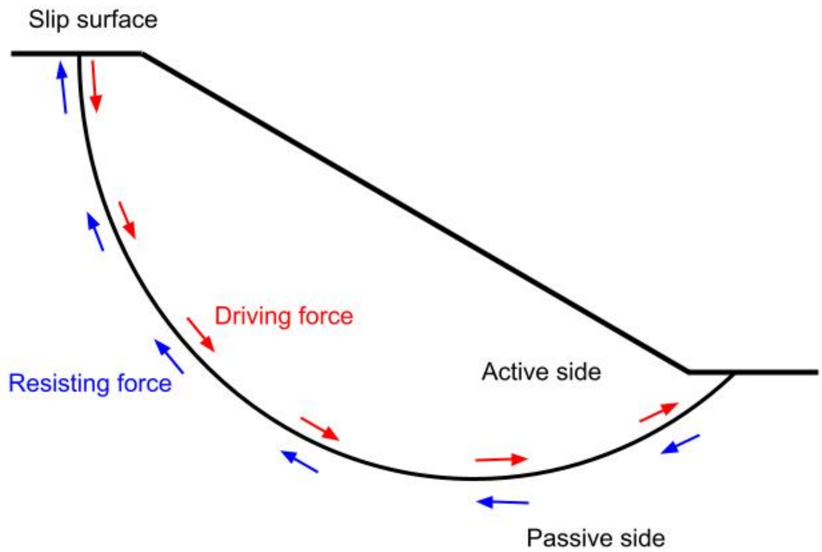


Figure 2.1: The resisting- and driving forces working on the landslide.

Important factors for the stability will be the water content and the water pressure in the soil (SGI, 2005). This is something that can be expected to change and be a critical factor in the future due to climate change and an increased amount of rainfall. The groundwater level is another important factor when studying the stability. The groundwater level can be heightened due to an increased amount of rainfall and lowered due to human activities such as water usage. Changes in the groundwater level can result in a reduction of the strength in the soil, mainly if the level gets too high. A too high level will result in higher pore pressure which will create greater separation of the soil particles which further will reduce the shear strength and lower the stability. A too low level will result in changes in the soil behaviour, mainly concerning settlements which are not focused on in this thesis.

If the slope is located in connection with a waterbody, the change of the water level or the effect of a landrise could lead to a reduction in the resisting forces which are important to prevent the slope from failure (SGI, 2019). The flowing water will also play an important role for the erosion of the slope which can lead to changes in the resisting force and increase the inclination of the slope so that the angle exceeds the friction angle. Which further can cause failure of the slope.

Increased loading in connection with the slope can lead to an increased driving force which will elaborate with the equilibrium that keeps the slope stable.

2.1.1.1 Quick clay

One main problem that to a high extent can be seen in the Western parts of Sweden, and is of importance when studying the slope stability is the amount of quick clay that is present (SGI, 2004). The common method to determine whether or not quick clay is present is by taking undisturbed samples and making fall-cone tests in the states when the clay is undisturbed and remoulded.

The properties of a quick clay are formed due to the leaching of salt in marine clays, where the composition of the Na+ and K+ ion are of importance for the sensitivity (SGI, 2004). The areas where the quick clays are found are therefore areas that once were glaciated and the sediments were deposited in seawater under the highest shoreline or in some cases in fresh and brackish water. During the leaching processes, the salt content will decrease. This leaching process can occur in three different ways: water percolation through the soil, diffusion of the salt between zones of different concentrations and water flowing upwards due to artesian pressure.

If the soil in some way is remoulded due to vibrations or loading, it is harder for the clay particles to connect back to each other since the clay now is lacking the presence of the link-binding saltwater. When leaching occurs there will also be seen increased compressibility when the soil is exposed to loading. With other words, the lack of link-binding salt together with increased compressibility will reduce the water holding forces in the clay. Further, this will lead to lower shear strength and a higher sensitivity of the soil.

2.1.2 Assessment of strength properties

As mentioned earlier, one important factor when studying slope stability is the strength in the soil (Knappett and Craig, 2012). The strength in the soil is divided by drained and undrained shear strength. When performing calculations three different analyses can be used: undrained-, drained- and combined analysis. The undrained investigation assumes that the shear deformation happens quickly and that the water is not leaving the soil. If the shear deformations are happening slowly, the pore pressure changes will have time to equalize and the drained state is used. For the combined investigation the lowest value for either drained or undrained shear strength is used (SGI, 2007). In general, the drained shear strength will therefore be dimensional for the friction soil and used for long time stability for slopes in cohesive soil. The undrained shear strength will in general be dimensional for the cohesive soil. The combined analysis is a method which is only used in Sweden and as mentioned before the analysis considers slip surfaces that go through both drained and undrained soils and will be dimensioned based on which has the lowest shear strength and will therefore give the lowest factor of safety (SGI, 2007).

When looking at the Mohr-coulomb model (Knappett and Craig, 2012), the failure of the soil will occur when the combination of the effective normal stress and the shear stress is critical. Failure will happen when the mohr coulomb circle touches the failure envelope. Figure 2.2 shows the Mohr-Coulomb failure criterion for the different analysis.

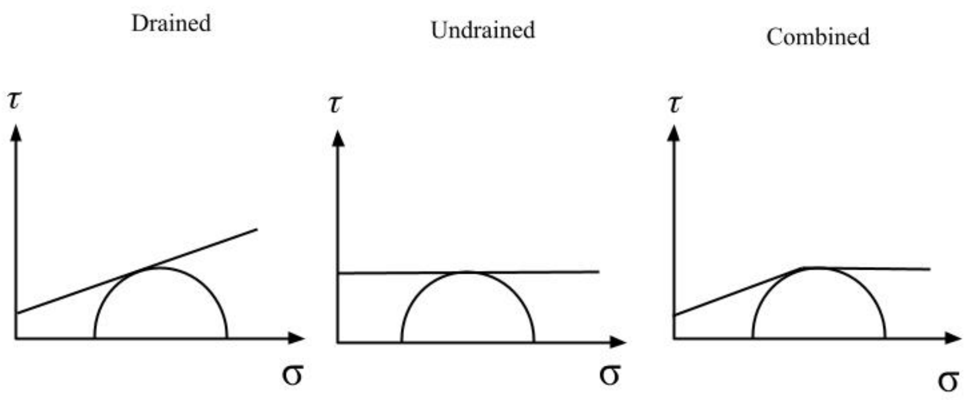


Figure 2.2: Mohr-Coulomb criterion for driand, undrained and combined analysis.

2.1.2.1 Vegetation effects on soil strength

Vegetation and roots in the soil can affect the strength and slope stability in various ways (Wieder and Shoop, 2018). In many soils, the roots work as a reinforcement and act as a fiber network which will stabilize the slope. The roots are especially useful in saturated soils where the strength in tension often is assumed to be less. There are many different factors that will affect how efficiently the roots work as reinforcement. This could for example be the number of roots, the age of the vegetation, the area of the roots and the vegetation mix.

It is also found that the degree of vegetation on a slope affects the erosion process (Wieder and Shoop, 2018). When studying the hydraulic effects, the vegetation will reduce the soil erosion rate due to the intake of raindrops, the increased infiltration, the addition of organic matter to the soil and the addition of surface roughness.

2.1.3 Pore pressure and water levels

As seen above the pore water pressure is of high importance for the strength and the stability of the soil, mainly since it controls the frictional resistance in the slope (Commission on slope stability, 1995). The higher pore water pressure is seen when an already saturated soil is stressed. The porosity in the soil can not increase and the pore water cannot escape or expand. When studying the pore pressure, it is important to be aware of the natural changes that can be seen over time. If the pore pressure variation is measured during a whole year, those natural changes are noted and a more reliable forecast for the dimensioned pore pressure for the slope stability can be done. It is often difficult to get those long time changes. Similar changes in the water pressure can be seen in surrounding groundwater reservoirs and can therefore be helpful to understand the pore pressure changes over time. The amplitudes will not be the same but the pressure variations over time will match. The pore pressure is also complex since the soil normally is not homogeneous. This means that the permeability differs with depth due to the soil layering.

Before starting the calculations for the slope stability, the observations of the groundwater and pore water pressure are compiled. The maximal water pressure is then transformed into equipotential lines to be able to see the variations in the slope. If no pore pressure measures have been made, a free groundwater level or a zero pressure level has to be defined as a minimum.

2.1.3.1 Erosion risks

Different types of erosions pose a risk for the slope stability. If the slope is in connection with a waterbody, heavy rains and an increased flow will lead to an increased risk for erosion and further a reduced stability of the slope (SGI, 2012). The erosion in this case will remove material and reduce the resisting force and change the inclination of the soil. Different types of materials are more or less sensitive against erosion and this degree of sensitivity is important to know. One method that SGI is using to study the sensitivity for erosion is a modified version of the Coastal vulnerability index (CVI). The method divides the classification areas into three different indexes which are stated as (SGI, 2019)):

1. Natural conditions
2. Impacts
3. Societal values

The natural conditions are about the specific system and its sensitivity against erosion (SGI, 2019). Parameters that are included are geology, the slope angle, ongoing erosion, land use and erosion protection. Important here is to know the soil type and its grain sizes since different soil materials are differently sensitive against erosion. For friction soils, the coarse silt and fine sand are the grain sizes that erode the easiest. For this type of soil, the erosion risk decreases with both increased and decreased grain size. The cohesion soil is in general harder to erode, but even for this soil, the slow process of erosion creates stability problems forward in time.

The impacts describe the fluvial processes which the system is affected by and how they change with a changing climate. Those processes are mainly the water velocity and the change in water flow. This information can for example be found at SMHI.

The societal values describe the value and importance of possible affected structures in the area, such as buildings and infrastructure (SGI, 2019). Each of the indexes have different parameters which will be given a value between low and high depending on their vulnerability.

If data is missing or is unclear the highest value on vulnerability is picked. The values are weighted and a final mean value is found for the erosion index. This index will give an idea of how sensitive the slope is towards erosion.

2.1.4 Calculations

When all the conditions are met, and enough data is gathered to be able to make calculations for the slope stability, the stability will be calculated for the assumed most critical loading (Commission on slope stability, 1995). All permanent loadings should be studied together with the varied loadings.

Calculations can be done both by hand and in simulation softwares. What method that is used depends on what information that is needed and the complexities of the information needed. It is often relevant to use both calculations to strengthen the results. The choice of the calculation method and the sliding surface is to a certain extent dependent on the topography in the area. In this thesis, calculations will be done with the Direct method followed up by simulations in the software Slope/W, which further will result in a factor of safety. The calculated factor of safety for the two methods will be compared to the required factors of safety from the Swedish guidelines.

2.1.4.1 Direct method

One method that normally is used is the Direct Method. This method assumes circular-cylindrical sliding surfaces and is used for pretty simple geometries, soil layering and pore pressure conditions (Commission on slope stability, 1995). The method can only be used when the soil is assumed to be totally drained or undrained. The method is further used for general calculations in an early stage.

2.1.4.2 Limit equilibrium method

When using this method, the sliding surface is normally penetrating through more than one layer (Sällfors, 1994). These layers can include both friction soil and cohesion soil where the strength parameters can vary with depth. The sliding mass is then divided into different laminas where the strength properties are similar along the sliding surface in each lamina. When simpler calculations are made the sliding surface can be assumed to be circular-cylindrical.

2.1.4.3 Simulation methods

Traditional numerical methods such as the finite element method are often used to analyse the slope response in the pre-failure and failure stages under the assumption of deformations (Wang et al, 2020). In comparison to the hand calculations, the simulation methods can give a factor of safety based on the problematics shown from a 2D/3D point of view, something that the hand calculations are not taking into account. The simulations will also show the soil failure mechanisms and show how the failure occurs in multi-stages. The simulation methods also help to estimate the most critical sliding surface and can also detect all feasible probabilistic slip surfaces. This assumption is not needed beforehand. This in combinations makes the simulations more complex and more details and information can be provided than from the hand calculations.

SLOPE/W is a software tool from Geostudio that uses the limit equilibrium method to analyse the stability of earth structures (GEO-SLOPE International Ltd, 2004). SLOPE/W also uses a CAD tool to create the model. The software uses a variety of methods to calculate the factor of safety. SLOPE/W can model heterogeneous soil types, complex geometries and variable pore water pressure conditions. The limitation that comes with using SLOPE/W is that it does not consider strain and displacement compatibility.

2.1.4.4 Swedish guidelines

In Sweden, bank piling is designed by the general technical description conducted by the Swedish Road Administration, called “ General description of bank piling”. The classification used for the bank piling is the safety class 3 and for geoconstructions the safety class is either GK2 or GK2 (to read more about these safety classes see BKR 94, chapter 4:21.) (Commission on slope stability, 1995).

In “Commission on slope stability” from 1995, guidelines on how to evaluate if a slope is stable or not are presented. The evaluation depends on the investigation phase, land use and soil analysis type which leads to a required factor of safety which the slope needs to achieve. The required factor of safety for the different soil conditions and investigation phases will be affected by the degree of detail for the parameters.

In this thesis, the calculations for the Direct Method will be assumed to contribute to the investigation phase called “geotechnical inspection and general calculations”. The land use for scenario 1 will be assumed as natural land. The land use for scenarios 2 and 3 will be assumed as existing buildings and facilities.

When moving forward to the simulations in Slope/W the investigation phase is assumed to be contributing to the more “detailed investigations”. The land use for scenario 1 will be assumed as natural land. The land use for scenarios 2, 3, 4 and 5 will be assumed as existing buildings and facilities.

Table 2.1, shows the required factors of safety for the scenarios above.

Table 2.1: Modified from “Commission on slope stability” showing the required factor of safety for the different phases and land use of the slope. The required factor of safety for the different analysis are presented where: F_c is for the undrianed, $F_{c\phi}$ is for the drained and F_{KOMB} is for the combined.

Investigation phase	Land use	
	Existing buildings and facilities	Natural land
General calculations	$F_c < 2$ or $F_{c\phi} < 1.5$	F_c and $F_{c\phi} < 1$
Detailed investigation	$F_c < 1.5$ or $F_{KOMB} < 1.35$	F_c and $F_{KOMB} < 1$

2.1.4.5 Sensitivity analysis

A sensitivity analysis is a method that analyses how different variables and factors of uncertainties affect the impact on the stability of a slope (Pichery, 2014). The method helps to understand the predictions of the model. It can also help to better understand how different variables interact or to understand how a model reacts to changes in independent input values. The sensitivity analysis can help to pinpoint the values that have the greatest impact on the model and target where improvements are needed by also reducing the overall uncertainties.

2.2 Stratification

To further understand the problems on site, and to be able to do simulations of the landslide, the stratification is necessary. The soil layers can have different greatness and properties over an area and will therefore behave differently and affect a slip that could lead to a landslide. It is therefore important to use different methods and to get all data needed to properly determine the stratification on the site.

2.2.1 Soil layering

The stratification is based on the layering of soils. The soil layers are formed from depositions that are transported by wind, water or ice. The depositions are formed from weathering of bedrock, chemical precipitation or organic materials. Both the transportation, how the deposition was formed and the sedimentation environment affect the properties of the soil layers (NE, 2020).

In Sweden, almost every earth deposit comes from the Quarter time. Weichsel is one of the ice ages during the Quarter time which highly has affected the earth's deposit of soils. During and after this ice age, most of the earth deposits in Sweden were formed (SGU, 2020). After Weichsel, the landmass that was under the ice sheet was and still is rising. Due to the land uplift, some of the deposits that before were below the sea level are now above the sea level. The structure of the stratification will also depend on if the land has been located above or below the highest coastline, see Figure 2.3. If an earth deposit was deposited above or below the highest coastline is important because it will affect the properties of the soil (SGU, 2020).

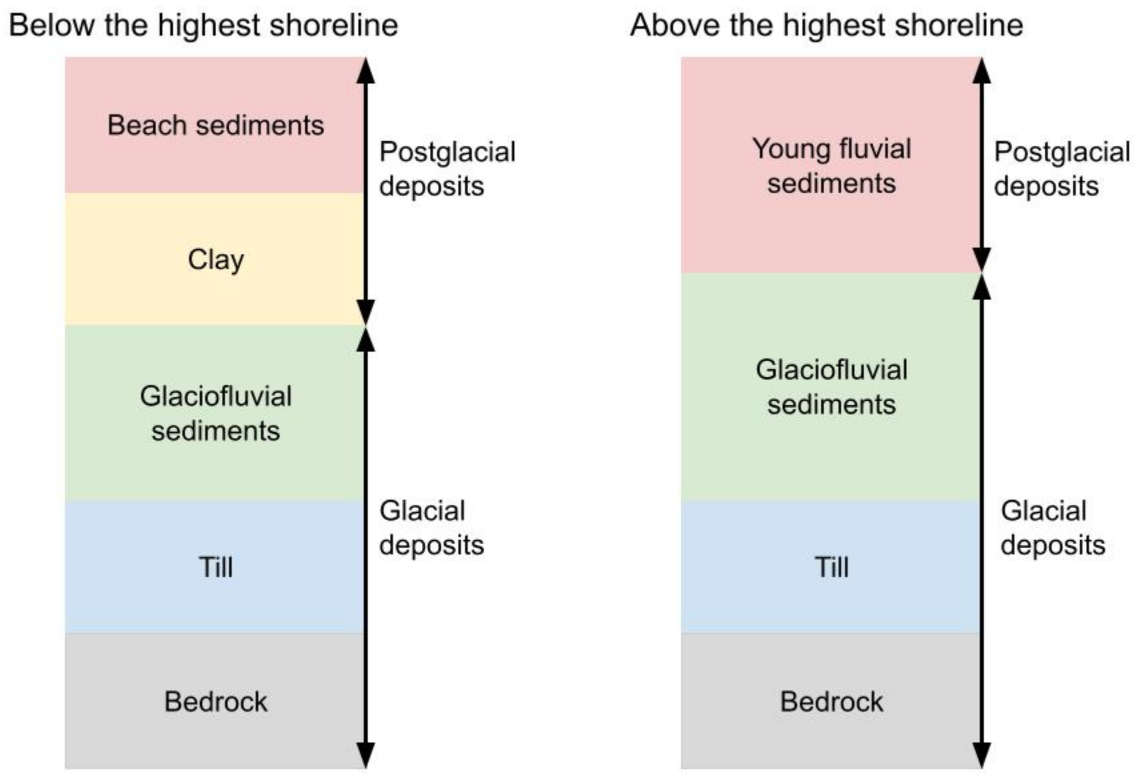


Figure 2.3: General soil layering below and above the highest shoreline.

2.2.2 Soil testing methods

Different tests can help to determine the properties of the soils and the stratification. Depending on what is needed for the investigation and what the conditions are on the site, different tests and combinations of tests can be used. The tests can be conducted on-site or in laboratories. When conducting tests and samples of soils, they need to be representative of the whole soil layer. If the soil conditions vary a lot on a site, more tests need to be performed (SGI, 1984).

The soil sampling can also vary in quality, and therefore the test can be divided into undisturbed, disturbed and stirred tests (SGI, 1984). The undisturbed sampling is used in tests for strength- and deformation properties. Also, the undisturbed samples are the most preferred due to that those samples correlate the most with reality. If undisturbed samples can't be taken, disturbed samples can be used to determine what type of soil that is present and further be used for laboratory testing. When using disturbed samples in laboratory testing, the grain size, plasticity, sensitivity to frost and water content can be obtained. When a sample is a stirred sample, the mechanical properties and the structure of the soil will be altered. The stirred sample can be used to determine the same properties as the disturbed sample if the stirred sample is homogeneous enough.

2.2.2.1 Cone Penetration Test

A Cone Penetration Test (CPT) is an in-situ testing method to determine the soil stratification and the soil properties. The test is performed by a cone, with a specific diameter and opening angle, that is pushed into the soil with a constant penetration rate. The data that is received from the CPT is the cone resistance, sleeve friction and pore water pressure. By analysing the data from the test the stratification and soil properties can be conducted (SGI, 2018).

2.2.2.2 Vane Shear Test

Vane Shear Test (VST) is a Swedish invention to determine the undrained shear strength in-situ. This testing method is mostly used for fully saturated clays (Knappett & Craig, 2012). For the method, a vane is driven into the earth to the specific depth of the soil that is going to be tested. After that, the vane is loaded with a torque that causes a fracture to occur in the soil, at the surface of the vane that can be registered in a measuring instrument connected to the vane. By knowing the maximum torque that is needed for the soil to fracture, a corresponding shear strength can be received (SGI, 1984). Although this test is not suitable for non-homogeneous clays, for example, clays that contain sand or layers of silt (Knappett & Craig, 2012).

2.3 Site investigation

To be able to calculate and simulate the slope stability in an area, the site investigation is vital. If a site investigation is done thoroughly, it can minimize the risk of the model lacing important information and lead to a more realistic output (Yang et al, 2019).

The site investigation which is done in this thesis is mainly based on data gathered from Sweco´s report (Göteborgs Stad Fastighetskontoret, 2014). Data is available for 3 of the sections, A, B and C, which are connected to boreholes 1301, 1303, and 1305, see Figure 2.4. CPTs are done for all of the boreholes and VSTs are done for boreholes 1303 and 1305. The tramway which is studied in this thesis is located closer to section D. Since no data is available for this section, information will mainly be gathered from borehole 1305.

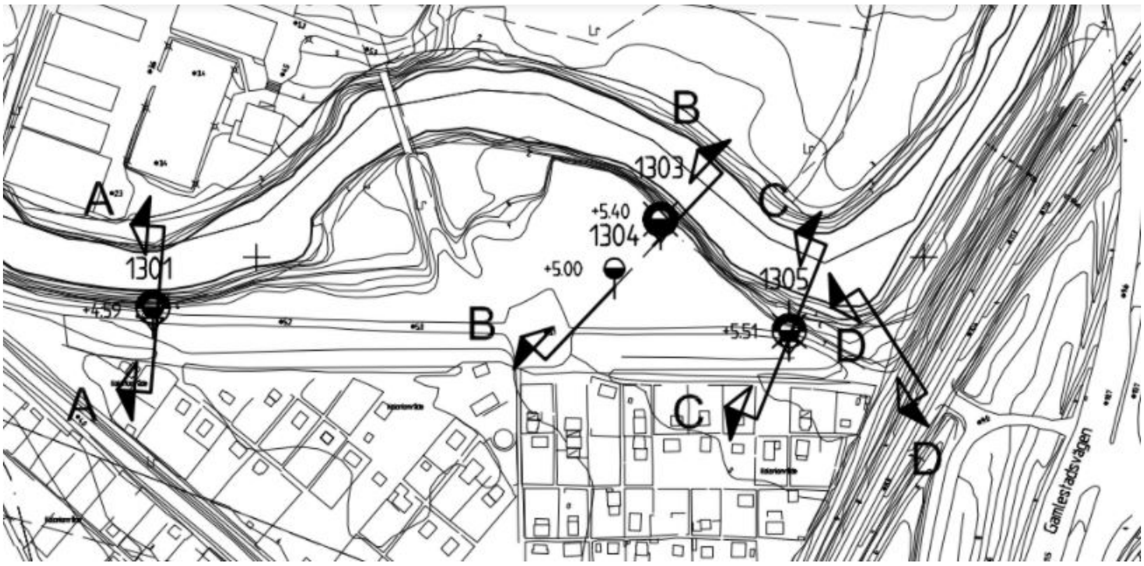


Figure 2.4: Overview of the division for the different sections and the boreholes, provided by SWECO. Note: Göteborgs Stad fastighetskontoret. (2014). Borrplan.

2.3.1 Angeredsbanan construction

Angeredsbanan is reinforced with bank piling. According to the drawings for Angeredsbanan, the tramway is mostly reinforced by pointed concrete piles that are constructed with the classification K400 which correlates to C32/C40 concrete. The section furthest south is unlike the other parts reinforced by cohesion piles of wood. The bank piling is constructed in six rows of piles with the center to center distance of 2.55 meters, where the pile plates have the measurements of 1.2 x 1.2 meters (Göteborg stad, 2014), see Figure 2.5. This drawing will further also be used to set up the models for the Direct method and SLOPE/W.

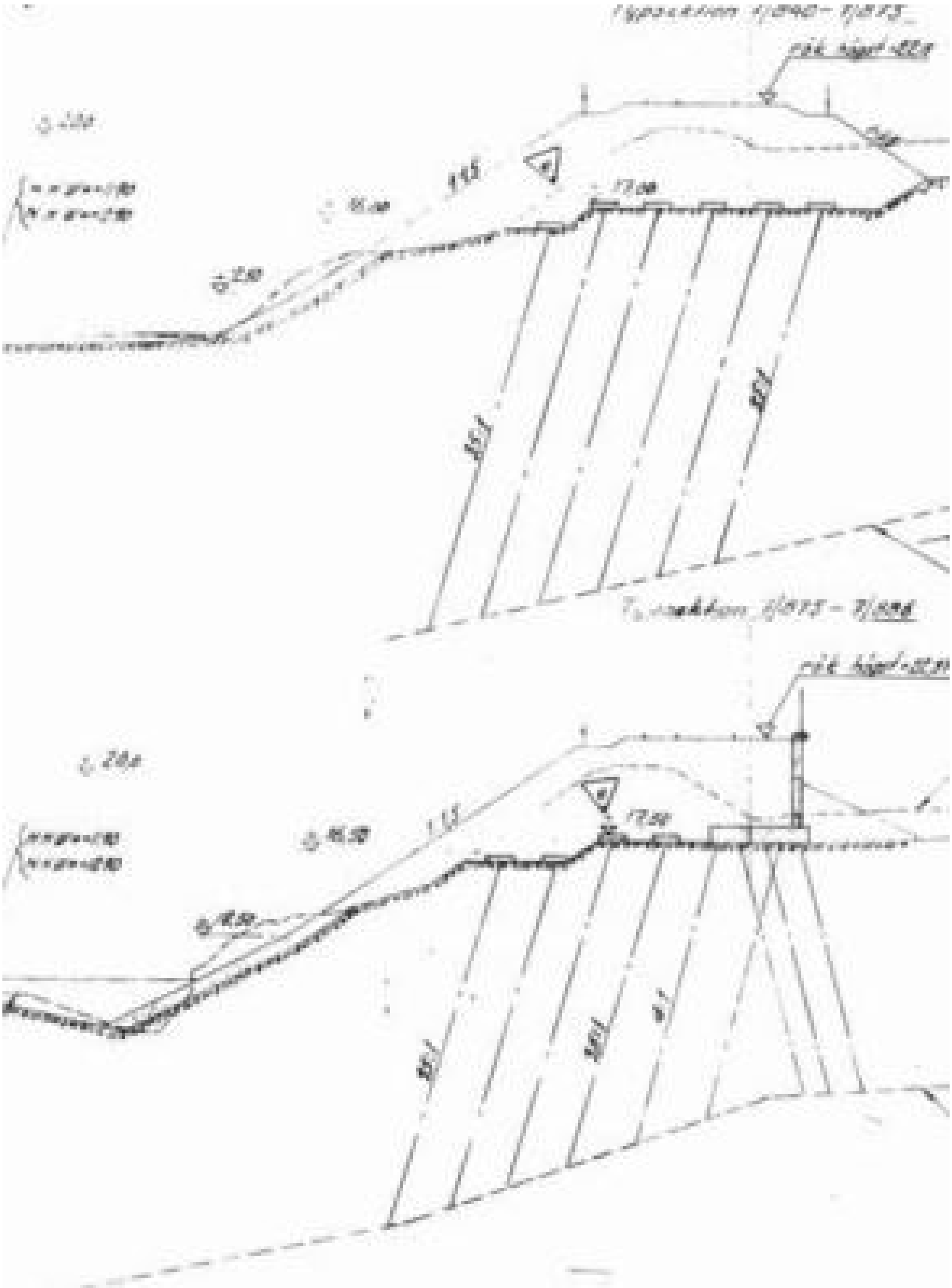


Figure 2.5: Drawings over Angeredsbanan by Lärjeån. Note: Trafikkontoret. (w.y.). Named 1480-3047-1. Retrieved from BatMan.

2.3.2 Stratification for Lärjeholm

To get an overview of the stratification at Lärjeholm, the soil type map from SGU is used, see Figure 2.6. From this map, it can be seen that the site is located below the highest shoreline, due to the soil deposits which earlier were mentioned. This statement is done according to Figure 2.3. At the site, there is till, beach sediments in the form of sand and post-glacial silt, and post-glacial clay. The bedrock at the site consists of granite which is one of the most common rock types in Sweden.

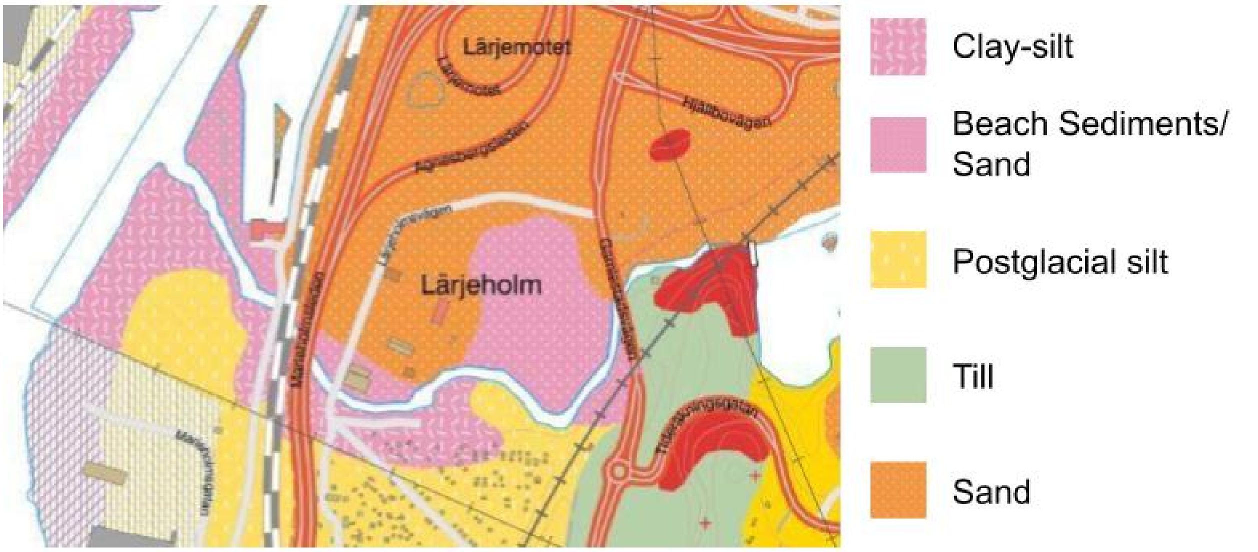


Figure 2.6: Altered soil type map from SGU for Lärjeholm. Note: SGU. (w.y.). Soil type map. Retrieved from sgu.se/kartvisare/kartvisare-jordarter-25-100.html.

To prove that the site has been located below the highest shoreline, by not only looking at the soil types at the site, SGU has a map that simulates the position of the shoreline up to 16 000 years ago. From the map, it can be seen that 14 000 years ago the shoreline was at its highest point at the site, to see the shoreline changes see Appendix A. Due to the site being located below the highest shoreline there is a risk of quick clay.

2.3.2.1 Field Test Lärjeholm

From the CPT and VST for borehole 1305, the undrained shear strength is obtained and is presented in Table 2.2. To see full CPT and VST data see Appendix B.

Table 2.2: The undrained shear strength at a certain depth at borehole 1305.

Depth [m]	Undrained shear strength [kPa]
0	-
2	-
4	-
6	20
8	21
10	22
12	22
14	25
16	26
18	32
20	36

2.3.2.2 Soil profile

Based on the stratification and the CPT from borehole 1305, a soil profile and the density for the soils are generated, see Table 2.3.

Table 2.3: Soil layering and density for the different soil layers.

Soil type	Density [ton/m^3]	Depth [m]
Sand	2	0-3
Clay 1	1.8	4-14
Clay 2	1.7	15-21
Till	2.1	21-24

2.3.3 Water- and Groundwater Level

Data in connection with groundwater level have been obtained from SWECOs CPTs. Wet soil was found at a depth of 3.5 meters in section C. Altho the pore pressure measurements show that the groundwater can be found further down, see Appendix B. From the field testing it was obtained that the groundwater level on the site correlates with the water level in Lärjeån stream and will therefore be used as the referred level in this thesis.

The water level in Lärjeån stream is assumed to confirm the water level in Göta Älv river according to SWECOs stability study of the site from 2014. The water level that SWECO used from SMHI is presented in Table 2.4.

Table 2.4: Characteristic water levels (RH2000) in Göta älv from SMHI.

MW (Mean water)	+ 0.1
MLW (Mean low water)	- 0.58
LLW (50 years) (Low low water)	- 0.9

The flow in Lärjeån varies a lot, an investigation conducted by the city planning office in Gothenburg reports the average flow and the highest flow in Lärjeån, which will affect the amount of erosion on the site (Göteborgs Stad, 2014), see Table 2.5.

Table 2.5: Flow in Lärjeån, mean- and high flow.

MQ [m^3/s]	2.1
HQ [m^3/s]	17.4

2.4 Similar problems worldwide

The problems caused by landslides are not only an issue that can be seen in Sweden. Countries such as Norway and Canada have a soil that is assumed to be the most similar to the soil we can find in Sweden (SGI, 2004). The majority of the landslides that have been detected in Sweden, Norway and Canada have been a consequence of the often present quick clay. The explanation for this type of clay is as previously mentioned the deposit of the clay under the highest coastline during the last deglaciation.

2.4.1 Sweden

In Sweden, there has been detected over 55 large landslides, affecting over 1 ha of land during the past 100 years (Center for climate adaptation, 2021). Landslides which together have affected many human lives. The most hazardous zones for landslides in Sweden are mainly around lake Värnen, Göta älv river, the whole east coast and the eastern of Svealand.

2.4.1.1 Tuve Landslide

The Tuve landslide was located in Gothenburg and happened in 1977 (Larsson & Jansson, 1982). The landslide affected around 270 000 m^3 and caused 9 human lives, injured about 60 people and made around 600 people homeless. The soil involved in the landslide was assumed to be soft clay, including quick clay, see Figure 2.7. The reason for the landslide was probably due to the increased flow in the stream which was connected to the toe of the slope which further led to erosion and triggered the landslide. Heavier rain during quite some time also showed an increased level of groundwater in the area. Something that increased the artesian pore pressure and unstabled the soil in the area.

Figure 2.7: Aerial view of Tuveraset. Note. From Tuveraset [Photograph], by Åke Hillefors, 1977, Wikipedia (sv.wikipedia.org/wiki/Tuveraset/media/Fil:Tuveraset GNM7033-002.jpg). CC BY 4.0.

2.4.2 Norway

In Norway, there has been registered more than 33 000 landslides which together have caused over 2000 deaths during the past 150 years (Kalsnes et al, 2016). The most hazardous zones for landslides in Norway are along the western and northern coastlines.

2.4.2.1 Finneidfjord Landslide

The landslide in Finneidfjord, located in the north of Norway along the coast happened during 1996 (Vardy et al, 2012). The landslide moved $1 \times 106\ m^3$ of sediments and caused 4 human lives, see Figure 2.8. After hand when using different geophysical and geotechnical techniques to conduct a reason for the landslide, the region where the landslide occurred consisted of clay-rich layers. Further a 3D morphology and stratification show signs of a quick clay landslide, which also has been detected on similar sites in Norway. The investigation states that the landslide and the failure of the quick clay may depend on the groundwater artificially pore pressure raise in combination with human triggers such as placement of fill along the shore and/or blasting for road works.

Figure 2.8: View of landslide at Finneidfjord. Note. Nettavisen Nyheter. [Photograph], by NTB, 1996, Retrieved from www.nettavisen.no/nyheter/se-oversikt-her-har-kvikkleire-alarmen-gatt-i-norge-de-siste-40-arene/s/12-95-3424068074

2.4.3 Canada

In Canada around thousands of landslides are detected every year (Perreaux, 2018). The majority of these landslides are small and larger ones occur around once each 10:th year. Around 700 people have died due to landslides when looking back to the 18th century . The most damaging landslides happen in the mountain areas of Alberta and British Columbia, but also in the St. Lawrence Lowlands of Ontario and Quebec.

2.4.3.1 Saint-Jude Landslide

The landslide happened in 2010 in the municipality of Sain-Jude along the Salvail river, belonging to Quebec in Canada (Locat et al, 2017). The landslide caused four human lives and affected 520 000 m^3 of land. To understand what caused the landslide and to further learn about the failure mechanism a detailed investigation was done. For a given stratification, the soil involved in the landslide was a slightly overconsolidated sensitive clay. The assumption could after the investigation be made that the trigger for the landslide was the degree of erosion at the toe of the slope together with an increased artesian pore pressure, see Figure 2.9.

Figure 2.9: View of the Saint-Jude landslide. Note. From Saint-Jude landslide [Photograph], by Grafcom., 2010, Wikipedia (commons.wikimedia.org/wiki/File:Landslide, Saint-J ude, Les Maskoutains, Quebec, Canada - 20100511.jpg). CC BY 2.0.

2.5 International methods

Since the soil properties are similar for Sweden, Norway and Canada, interest was growing to study whether or not the methods used for slope stability are the same for the different countries.

2.5.1 Hazard zonation

To determine whether or not areas pose a risk for a landslide, it is important to in an early stage use a hazard zonation method (Lundström & Andersson, 2008). This will help to understand if further investigations and measurements have to be considered for the studied area.

2.5.2 Sweden

For Sweden, two different methods are used depending on whether a coarse-grained soil is studied or if a clay/silty soil is studied. Since the majority of the landslides occur in clays, this method will be further studied.

In the first step, a preparation study has to be done to know which areas to evaluate more closely (Lundström & Andersson, 2008) (SGI, 2007). This preparatory study involves studying geological and topographical maps, field inspections, inventory of aerial photographs, previous geotechnical investigations and in some cases soundings are made. The area is thereafter divided into one of the three different stability zones l-lll, independent of if there is a prerequisite or not for a landslide.

Stability zone 1 is covering land areas which could be primarily affected by initial slips or slides. Stability zone 11 is covering areas which are not prerequisites for initial landslides but could be affected secondarily. Stability zone 111 is covering areas with bedrock outcrops and soils which do not include clay or silt. The stability zone is shown on a map together with earlier slides, inclinations over 1:10 and erosion risks. After this, general calculations are made based on the gatherings on the map. These calculations will mainly be based on field and laboratory investigations. When the calculations are done, the areas will be assumed to have either satisfactory stability or unsatisfactory stability.

2.5.3 Norway

For Norway, the mapping is done in two stages (SGI, 2007). The first stage is to map the marine clay areas and the quick clay areas with prerequisites for a landslide based on basic topographical criterias. The quick clay is also determined by using a rotary pressure sounding, which is a Norwegian method to get the penetration force towards the depth. In a quick clay soil, the penetration force will stay constant or decrease slightly. After this, a risk zonation is done according to the mapped quick clay areas. The areas are further divided into smaller areas, assuming that each smaller zone could potentially be affected by a landslide. The smaller zones are classified due to a soil criteria and a topographical criteria. Slopes and gullies with a difference in height over 10 m or with a sloping inclination above 1:15 are also marked to investigate further.

The zones will further be given engineering scores, which will be based on local conditions, engineering judgment, geotechnical parameters and the properties and persons that will be exposed (SGI, 2007). The engineering scores include hazard class (low, medium or high), which will be dependent on OCR, the thickness of the clay layers, human activity, earlier sliding, pore pressure, erosion, sensitivity and height of the slope. Where high pore pressure, human activity and erosion are assumed to be most critical. The score also includes the consequence class (not severe, severe or highly severe), which will be based on persons in industrial buildings, power lines, the consequence of flooding, number of dwellings, number and the importance of roads and railways and the surrounding building's value. Where persons in industrial buildings and closely placed dwellings are most critical.

The final risk will then be calculated as:

$$risk = hazard \times consequence$$

Where hazard weighted score is in percentage of the maximum value, and the consequence weighted score is also in percentage of the maximum value.

The mapped areas will from this calculation be divided into five risk groups (SGI, 2007). Whereas stated "Risk class 1-2 = no more soil investigations needed, the area is assumed to be stable. Risk class 3 = further investigations are needed and risk class 4-5 = further investigations are needed, stability analyses are required and remediation to get the area more stable is probably needed".

2.5.4 Canada

For Canada, the first step is to map areas which are already built on, areas which are attractive for building and areas which are planned for use (SGI, 2007). The main goals are to get a hazard map and a map for land use management. The information is further gathered in two maps, the information and the susceptibility map.

The information map consists of data on heights and the inclination of the slope, soil type, scars of landslides, location of boreholes, erosion, limits of the slope, topography, slope classes and field inspections (SGI, 2007). Aerial photographs and CPT tests are used to identify the soil, and piezometers detect the groundwater conditions. Clay and silty slopes with a height higher than 5 m and an inclination above 14 degrees are also mapped to study further. For steep slopes with an inclination above 20 degrees, landslides are assumed to be able to occur naturally.

The susceptibility map is dividing the areas into different hazard zones (SGI, 2007). The susceptibility will mainly depend on the geomorphological, geological and geotechnical properties of the soil and the site. The zones will be divided into low susceptibility zones and moderate to high susceptibility zones. These zones will further be divided into ten different subzones depending on the inclination of the slope, erosion, type of soil, presence of sensitive clay and the type of danger. The sensitive soil is assumed to be most critical and is detected by aerial photography and by using boreholes and laboratory tests. The classification of hazard zones is then following a specific flow chart.

2.5.5 Similarities and differences

As a summary, the three different methods have many similarities due to the often very similar geology in the risk areas. It can anyhow be seen that the soil in the different countries has some different properties. For example, there is a variation in the accepted inclination of the slope. This is a result of the variations of shear strength in the soils, where Canadian soils often can take more strength and accept higher inclination. In Norway, the method is only applied in areas where quick clay is present which is not the case for the Swedish and Canadian methods. In Sweden, there are no requirements for studying the presence of quick clay or the sensitivity of the soil.

The division of hazard zones also differs between the methods. The Norwegian method divides into larger zones which pose a risk for landslides, the Canadian method divides into smaller zones which pose a risk for landslides and the Swedish method divides zones showing whether or not more investigations are needed, not necessarily meaning that the zone is a potential landslide zone. For the Canadian and Swedish methods, geotechnical investigations are needed but for the Norweigan method only rotary pressure soundings are needed. The Norweigan method is both taking into account the consequences and the risks, not only the hazard, which is unique when comparing the three methods.

2.6 Slope stabilisation methods

When slope stabilisation is assumed to be a problem, the need for complementary actions is important to ensure safety and reduce the risk of failure. The various stabilisation techniques can be divided into different categories which are set to (Das Braja, 2014) :

1. Soil stabilisation
2. Removal and protection
3. Water drainage
4. Support stabilisation

The soil stabilisation methods focus on the processes which will strengthen the soil's mechanical properties, mainly increasing the shear strength. Some commonly used techniques are chemical stabilisation such as using fly ash, lime and cement e.g. and mechanical stabilisation such as dewatering, compaction and mixing. To increase the stability in the soil, water drainage to control the groundwater level and to decrease the amount of water entering the system is also a commonly used method.

The focus in this thesis will be on the support stabilisation methods. Which involves structural support which helps to increase the stability. Techniques that are used can in this case be piling, retaining walls, soil nailing, geosynthetic reinforcement and prestressed anchors. Two methods that are highly used are soil nailing and retaining walls, mainly due to their fast construction, cost effectiveness and easy revision (Chen & Liu, 2007).

2.6.1 Retaining wall

Retaining walls are often constructed at the toe of the slope to create a resisting force and prevent small size and secondary landslides from occurring. The wall is mostly placed vertically with a small inclination towards the top of the slope. Depending on the size and the different needs on the location, different materials such as concrete, wood or rock can be used in the retaining wall. Some walls may include drains in the design to make a way for the groundwater to escape and keep the sturdiness of the retaining wall. For large-scale soil movement, a similar method called crib walls may be more efficient (Chen & Liu, 2007)(Tangent technologies, 2020).

A similar method used to create a resisting force by the toe of the slope is the use of a pressure bank. The pressure bank has the same purpose as the retaining wall. The only difference is that the material used in this case are different soils and grain sizes instead of concrete for e.g.

2.6.2 Piling

The purpose of piling in connection with a construction work is to transfer the loads from the construction through the loose layers of soil down to solid rock or stronger, more compacted soils. This method will increase the stability of the area and is mainly a method to avoid landslides or settlements (Fleming, K et al., 2009).

Depending on what construction that is built, the soil bearing capacity and properties, in combinations with the surrounding, different types of piling methods and materials are used. The main methods that are used are driven or bored piles (Fleming, K et al., 2009).

Driven piles can generally also be called displacement piles which means that the soil is displaced continuously as the pile penetrates the soil. Granular soils will normally become more compact and clay soils tend to heave during the displacement process. This method will cause a lot of vibrations and is therefore not suited to be used in areas surrounded by buildings. The method is well suited in soils with high water tables, on contaminated sites and in non-cohesive soils. There are two different methods to insert driven piles. Either pre-formed piles are driven into the soil or a displacement method is used to create a void that further will be filled with concrete. The main types of materials that are used for the pre-cast piles are concrete, steel, wood, timber or a combination of those (Alén, C., 2012).

Bored piles can generally also be called non-displacement piles which mean that the soil is extracted and not displaced as for the driven piles. The hole that is created is mainly filled with concrete or steel. When the soil is extracted, the lateral stress is reduced in the soil. The problems that can be seen from the displacement of the soil are therefore removed. This method will also produce spoil which can be expensive to remove from the site, which can be problematic for contaminated sites. This method is more suitable for soils with low water tables, for cohesive soils and in surroundings with buildings due to the lower intensity of vibrations (Alén, C., 2012).

2.6.2.1 Pile axial capacity

When a pile is loaded at the pile head with a vertical force, shear stresses will be mobilized in the ground. If the shear strength of the soil is exceeded, failure will occur. Depending on if the pile is dominated by end-bearing capacities or shaft bearing capacities, the shear stresses will mobilize differently, see Figure 2.10. For end-bearing piles, the shear stresses will mobilize in the soil around the pile toe and for shaft bearing piles the shear stress will mobilize at the interface of the pile and in the soil along the pile shaft (Fleming, K et al., 2009).

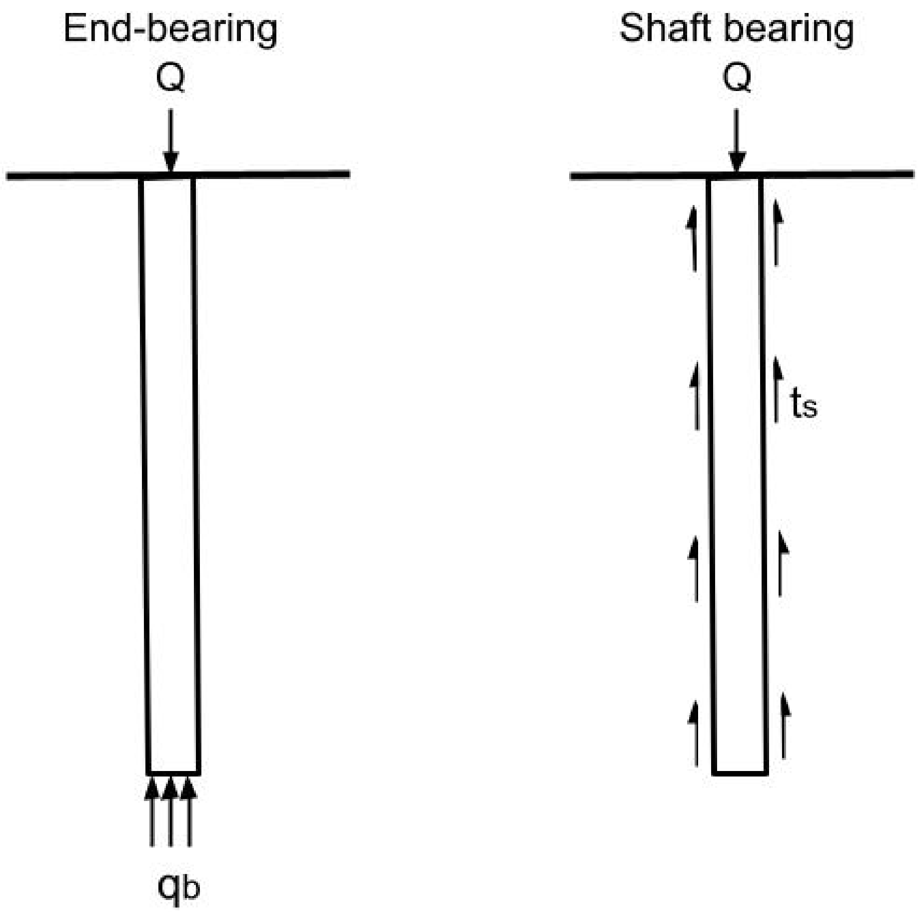


Figure 2.10: Axial capacity of piles. Modified from Fleming et al. (2009).

The end-bearing capacity, qb, for piles in cohesive soils long-term will be greater for drained than for undrained scenarios. Despite this, the settlements for drained capacity will be too great and not acceptable for most structures. The piles must also have enough immediate load-carrying capacity so that they could possibly stop short-term failure from happening. With previously stated information as a background, the end-bearing capacity in cohesive soils is obtained from the undrained shear strength together with a bearing capacity factor, Nc, to correct the force at the bottom of the pile. A majority of the piles in cohesive soils generate almost all the capacity along the shaft. The shaft friction, ts, can be evaluated from the undrained shear strength and an empirical factor, α , is used to correct the force from the shaft friction (Fleming et al., 2009).

3

Methods

In this chapter, the evaluation of the input data gathered from the site investigation, the motivation for the model's setup followed by a description of the calculation processes are obtained for both the Direct method and SLOPE/W. Further, the pile capacity is evaluated to be able to perform the calculations in Slope/W. A description of how the sensitivity analysis is conducted will also be included.

3.1 Data

To be able to do calculations and simulations, data for the soil parameters are needed. In Table 3.1 the drained- and undrained parameters for the soil are presented. All parameters are obtained from the CPTs that Sweco conducted. The undrained parameters are including the undrained shear strength Cu and the friction angle for all material except the clays. The drained parameters are including the cohesion c´ and the friction angle for all materials. For the combined analysis, the undrained shear strength and the friction angle for all materials are used. The reason for the ballast having a unit weight at almost zero is because when using SLOPE/W all of the weight from the ballast is assumed to be taken by the piles.

Table 3.1: Soil parameters used in the drained-, undrained- and combined analysis.

Material	Unit weight [kN/m^3]	Cu [kPa]	c´[kPa]	ø [°]
Ballast	0.01			42
Sand	20			35
Clay 1	18	23	2.3	30
Clay 2	17	24+2z	2.4+0.2z	30
Filling material	20			38
Till	21			36

3.2 Direct method

The Direct method is a commonly used method which is used to calculate the factor of safety for slopes. This method is chosen to get general results of the safety and the risks for the slope in an early stage. The geometric and soil layering for the slope are assumed to be pretty simple and is therefore also an argument for the chosen method. The results will further be compared with results from more detailed and complex simulations and also compared with required factors of safety from the “Commission on slope stability”.

3.2.1 Model

To get a broader understanding of the model which will be set up for the calculations, a basic description of the involved parameters is done. See Table 3.2 for used parameters, for the used models.

H - Height from the top of the layering to the bottom of the inclination
D - Height from bottom of inclination to bottom of layering
d - D/H
H_w - Height of external water level
H'_w - Height of internal water level
b - Inclination of slope
β - Slope angle
q - Load from trams and embankment

Figure 3.1 shows the parameters´ connection to a general slope.

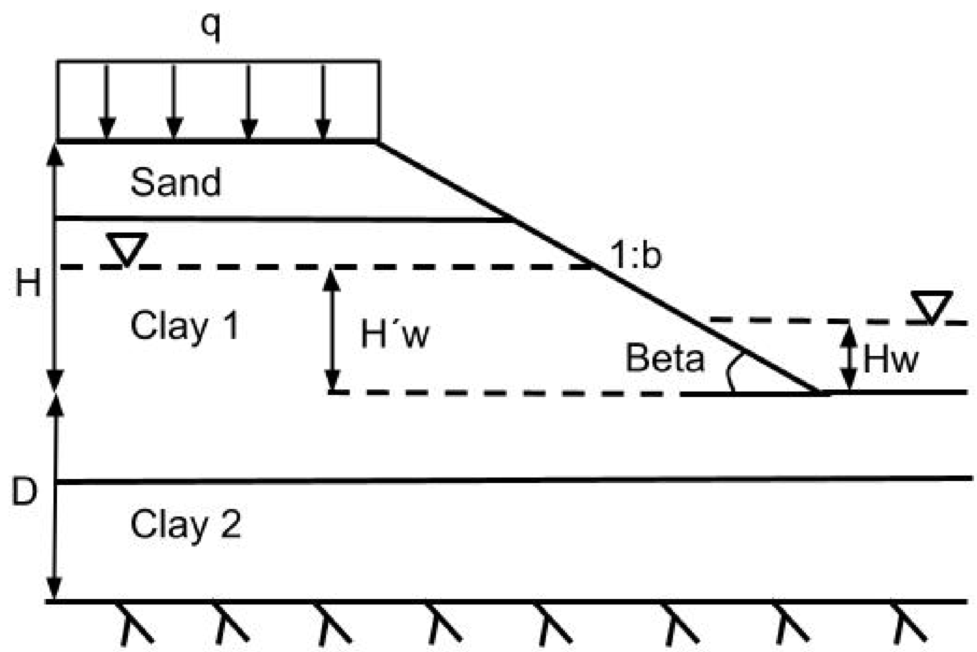


Figure 3.1: General model and its relevant parameters used for the Direct method.

The models which are used for the calculations are simplifications and are general models which in this basic state are relevant for both sections 840/875 and 875/886, see Figure 2.5.

Three different scenarios, each one set up as a separate model were used for the Direct method. The scenarios are set up to be able to compare the stability for different historical states of the slope. The historical states are following the development of the construction of Angeredsbanan as if reinforcement was not added. Piles will not be included in the calculations due to the limitations of making calculations with piles for this method. Before construction, there was a natural slope that was undisturbed. After this state, the preparation of the construction started with the excavation of the natural slope. Further, the construction of the embankment was made, leading to the tram load being added.

Following scenarios are set up and can be seen in Figures 3.2-3.4:

1. Natural slope
2. Excavated slope
3. Excavated slope with load

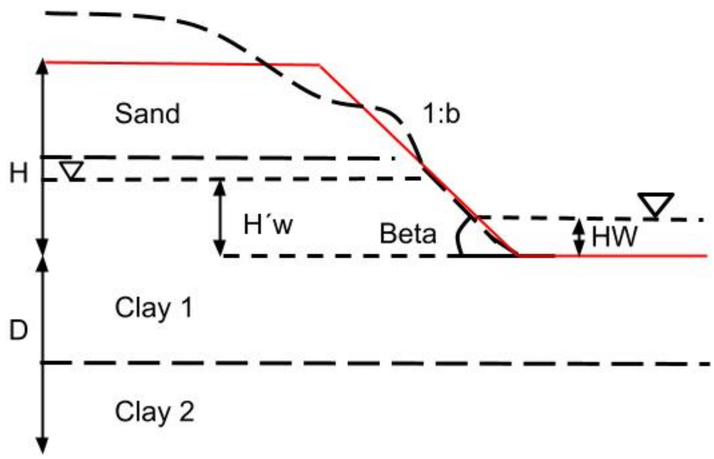


Figure 3.2: Natural slope [1]. Before the excavation is done and the embankment with the connected tram load is added. The black line showing the slope´s original state and the red line showing the simplification of the slope. Data for the parameters can be found in Table 3.2.

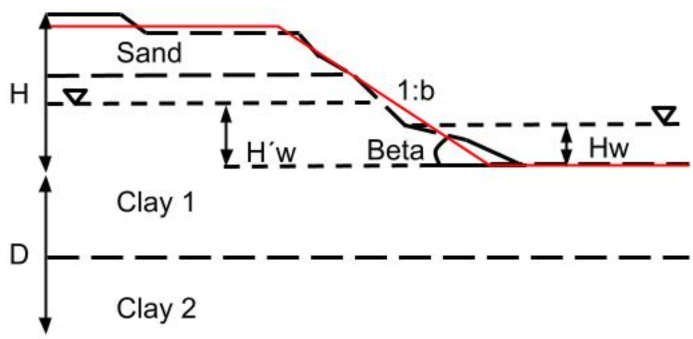


Figure 3.3: Excavated slope [2]. Before the embankment and tram load are added, after the excavation is done. The black line showing the slope´s original state and the red line showing the simplification of the slope. Data for the parameters can be found in Table 3.2.

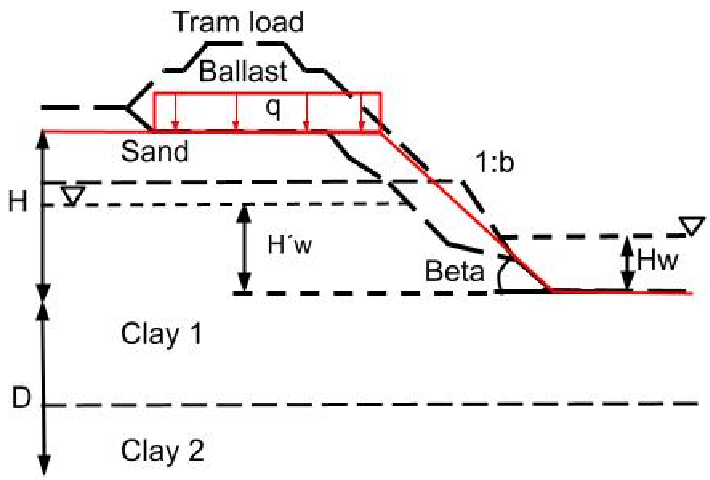


Figure 3.4: Excavated slope with load [3]. After the embankment and tram load is added. The black line showing the slope´s original state and the red line showing the simplification of the slope. Data for the parameters can be found in Table 3.2.

The soil properties and groundwater level for the different layers are gathered from CPT 1305. Further, the inclination, slope angle and other dimensions are gathered from the drawings of Angeredsbanan and the water level in Lärjeån is gathered from SMHI, see Figure 3.1. Hw was obtained by calculating the water level from SMHI´s guide for the RH2000 system (SMHI, 2019). At the time, the current water level in Göta älv was acquired from SMHI and was +7.6 cm, see Appendix C for calculation. The load (q) is consisting of the load from the trams (15 kN/m) based on Trafikkontorets ”Teknisk handbok, Tekniska anvisningar avsnitt 2HA1.7” and the ballast where the density is assumed to be 2.6 ton/m3 for granite. The assumption about the ballast is based on the material consisting of crushed rock, where the most common rock type in Sweden as mentioned before is granite. By using SGU´s Ballast map the density of the rock is obtained.

Following data, see Table 3.2, is used based on findings described in Figure 3.1 for the three different scenarios:

Table 3.2: Soil parameters used in the drained- and combined analysis.

Parameters	Natural slope [1]	Excavated slope [2]	Excavated slope with load [3]
H	8	6	6
D	15	15	15
d	1.875	2.5	2.5
N_0	5.6	5.5	5.5
q	-	-	46
μ_q	-	-	0.94
μ_w	1	1	1
μ'_w	0.95	0.94	0.94
H_w	0.076	0.076	0.076
H'_w	3.5	3.5	3.5
b	1.5	1.5	1.5
β	34°	34°	34°

3.2.2 Undrained

To be able to calculate the factor of safety for the undrained conditions, correction factors and stability factors (see eq 3.1 and 3.2) are derived from tables found in the document called "Commission on slope stability". The driving pressure and the factor of safety are then calculated, for full calculations see Appendix C.

$$p_d = \frac{\gamma H + q - \gamma H_w}{\mu_w \mu_w'} \tag{3.1}$$

$$F_c = N_0 \frac{\tau_{fu}}{p_d} \tag{3.2}$$

where:

γ - Unit weight soil
γ_w - Unit weight water
μ_q - Correction factor for external load
μ_w - Correction factor for external water levels
μ_w' - Correction factor for internal water levels
q - Load
p_d - Driving pressure
N_0 - Stability factor
τ_{fu} - Shear strength

3.2.3 Drained

The Direct method in drained conditions is only intended for toe circles. For the calculations used for the drained conditions, the correction factors and the stability factor (see eq 3.3, 3.4 and 3.5) are derived from tables found in the document called "Commission on slope stability". Further, the driving pressure is calculated followed by the factor of safety, for full calculations see Appendix C.

$$p_e = \frac{\gamma H + q - \gamma H_w}{\mu_w'} \tag{3.3}$$

$$\lambda_{\phi c} = \frac{p_e \times tan\phi'}{c'} \tag{3.4}$$

$$F_{\phi c} = N_{cf} \frac{c'}{p_d} \tag{3.5}$$

Where the parameters are the same as for undrained conditions exept for:

p_e - Parameter
$\lambda_{\phi c}$ - Parameter
N_{cf} - Stability factor
c' - Shear strength

3.3 Slope/W

SLOPE/W is a software used to do more detailed analysis of slopes. This method is chosen to get more detailed results of the safety and the risks for the slope. The stability will be interpreted by studying factors of safety resulting from the simulations and comparing it with the required factor of safety from “Commission on slope stability”. The calculations for this project are following the total calculation philosophy and are not following Eurocode.

3.3.1 Model

Two different sections along the tramway in Angered are chosen to further be simulated in SLOPE/W. The models of the sections are based on the drawings of Angeredsbanan and the soil properties and groundwater level for the different layers are gathered from CPT 1305. Information is also gathered from SMHI regarding the water level in Lärjeån. The parameters for the piles are based on the drawings of Angeredsbanan and can be seen in Table 3.3.

Table 3.3: Parameters for the piles.

Inclination	1:3.5
Center to center	2.55 m
Diameter	0.28 m

Five different scenarios are simulated in SLOPE/W. All five scenarios are simulated for both combined and undrained conditions. This is done for two sections of the slope (840/875 and 875/886, see Figure 2.5) to be able to study the stability differences between the sections and the different scenarios.

The different scenarios are based on the historical development of the slope in the same way as for the Direct method, but in this case piles will be added after the excavation. Also, a simulation without the piles when the embankment is built and the tram load is added will be made to see what effects the piles have on the stability of the slope.

Following scenarios are simulated and can we seen in Figures 3.5-3.14:

1. Natural slope
2. Excavated slope
3. Excavated slope reinforced with piles
4. Excavated slope with load, reinforced with piles
5. Excavated slope with load

Scenarios 1, 2 and 5 are the same scenarios that are calculated for the Direct method. The different scenarios will give an understanding of the different risks with and without reinforcement, historically before and after different loads are added and in the long and short term.

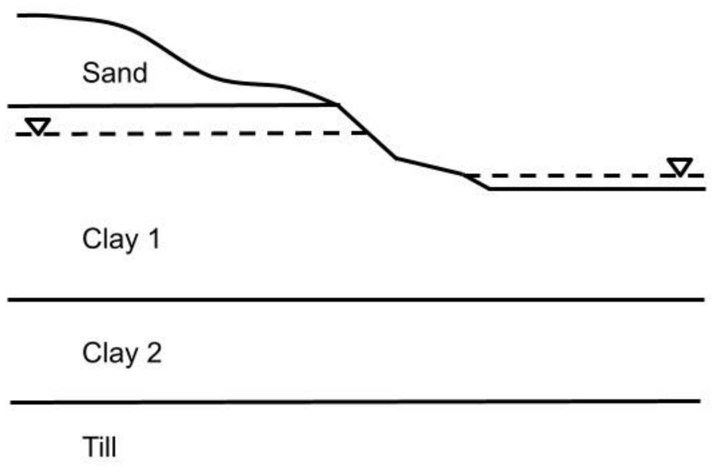


Figure 3.5: Model of section 840/875, Natural slope [1].

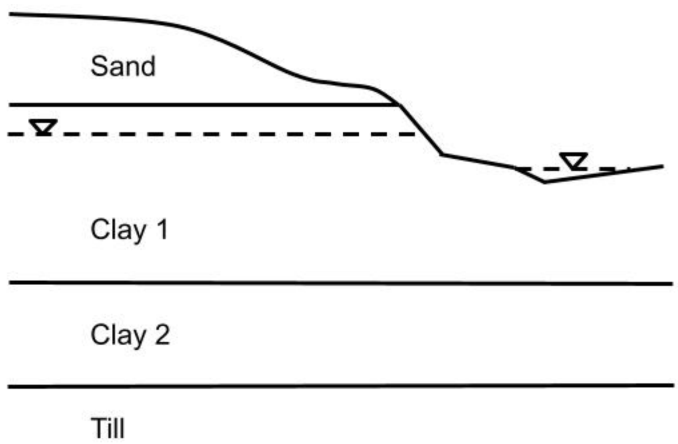


Figure 3.6: Model of section 875/886, Natural slope [1].

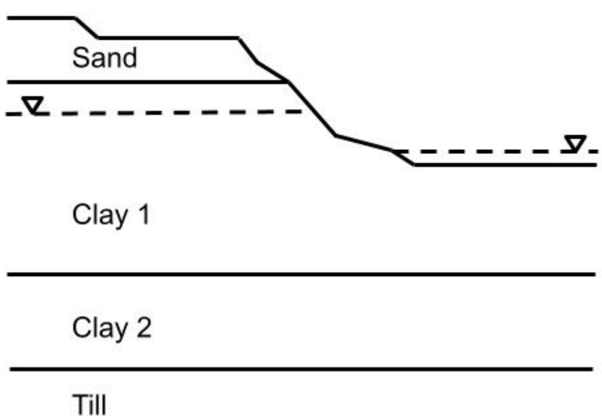


Figure 3.7: Model of section 840/875, Excavated slope [2].

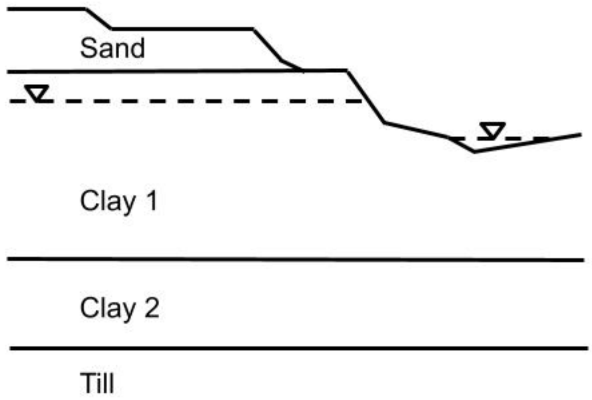


Figure 3.8: Model of section 875/886, Excavated slope [2].

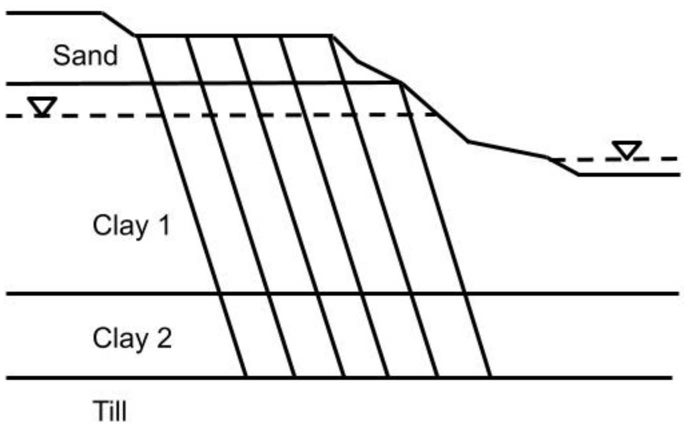


Figure 3.9: Model of section 840/875, Excavated slope reinforced with piles [3].

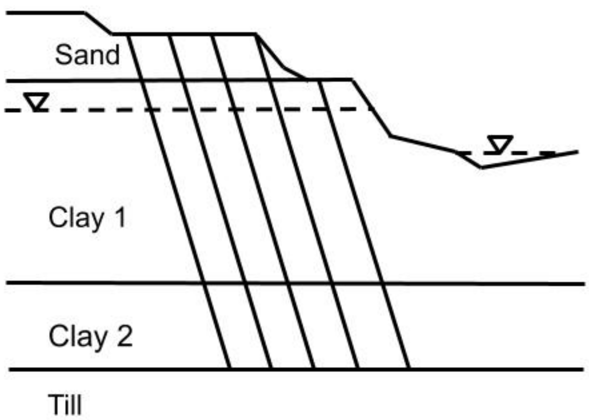


Figure 3.10: Model of section 875/886, Excavated slope reinforced with piles [3].

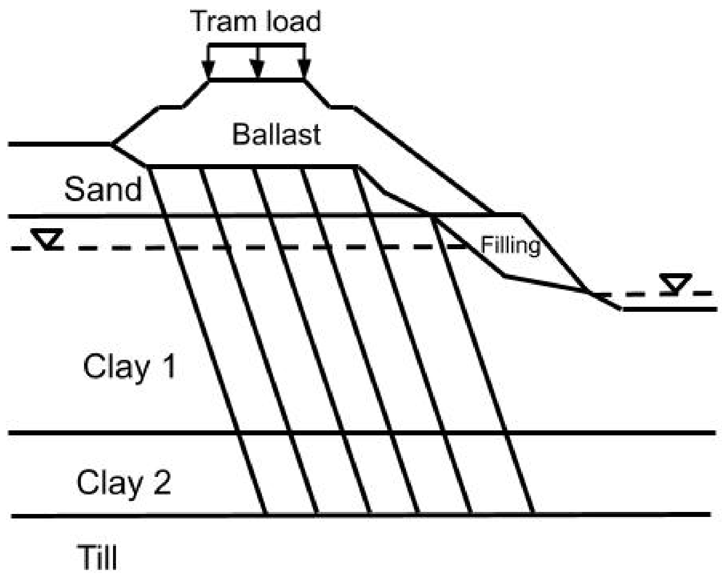


Figure 3.11: Model of section 840/875, Excavated slope with load, reinforced with piles [4].

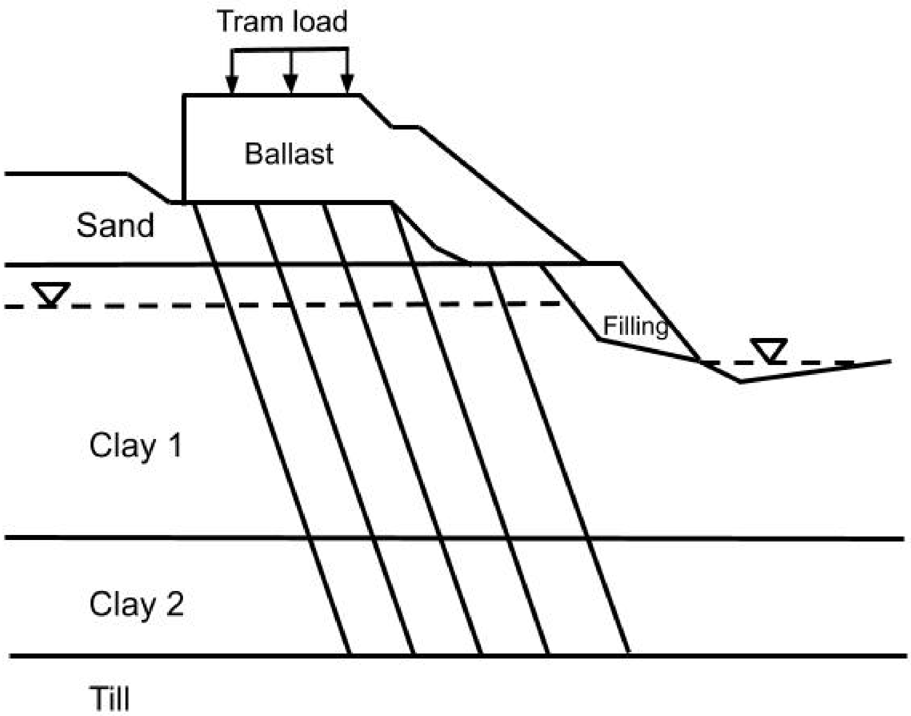


Figure 3.12: Model of section 875/886, Excavated slope with load, reinforced with piles [4].

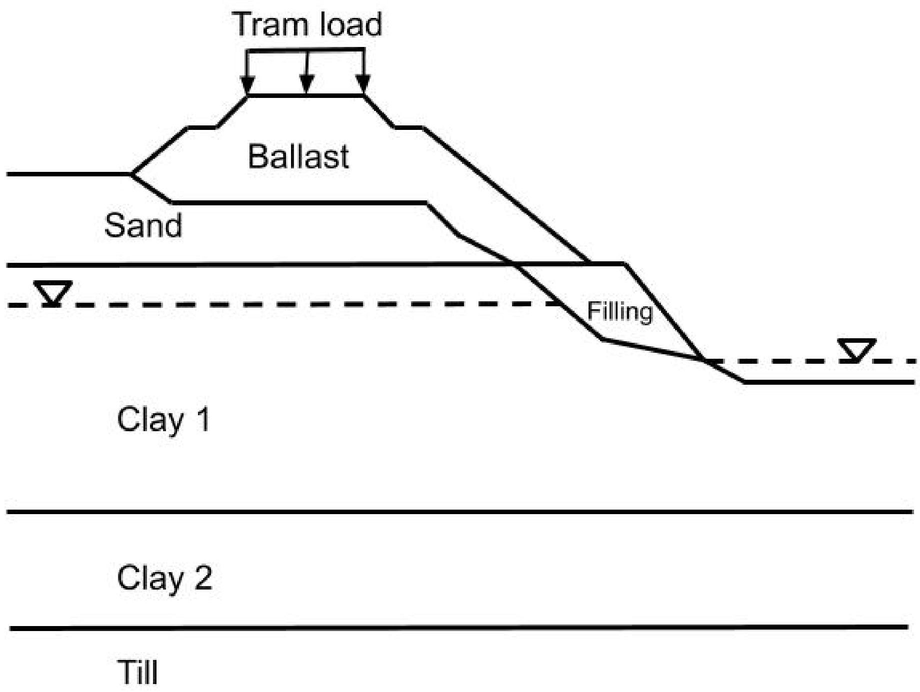


Figure 3.13: Model of section 840/875, Excavated slope with load [5].

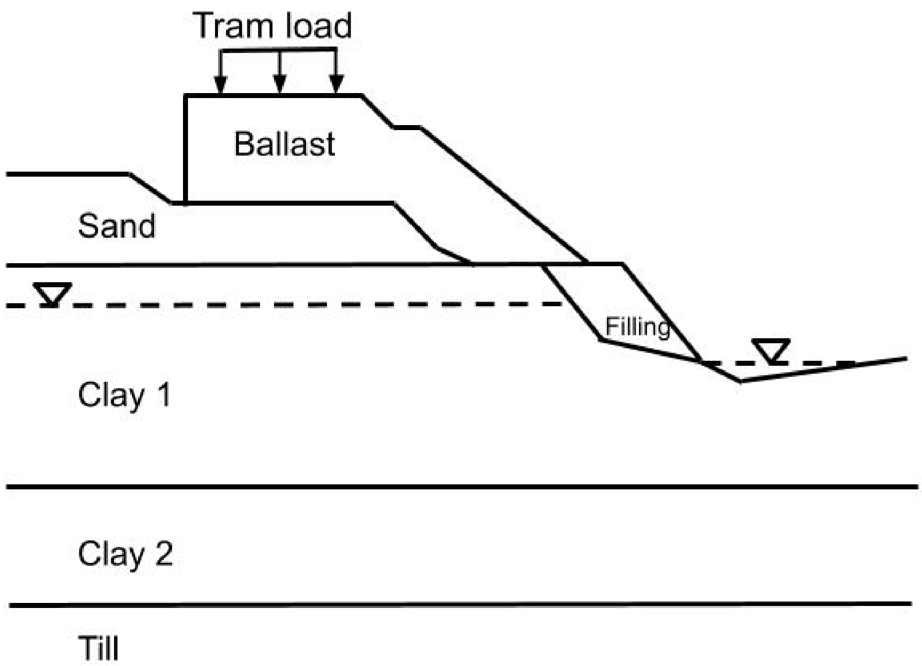


Figure 3.14: Model of section 875/886, Excavated slope with load [5].

3.3.2 Pile capacity

The most critical slip surface without reinforcement is produced in SLOPE/W to be able to see where the later inserted piles intersect with the slip surface. This is done to be able to see which length of the pile that is located above the slip surface. A new critical slip surface will be produced when inserting the piles´ capacity and properties. See Appendix D for pile capacity calculations and for the insertion into SLOPE/W.

The calculation process is based on the book “Piling Engineering” written by Fleming et al. (2009). Figure 3.15 is modified from “Piling Engineering” and describes the length of the pile to the critical slip surface, which is the part of the pile that is most exposed to the driving force of the landslide, see Appendix D. The figure also includes the forces working on the pile and the ultimate capacity on the pile. Which further is important to calculate to see the effects from the piles.

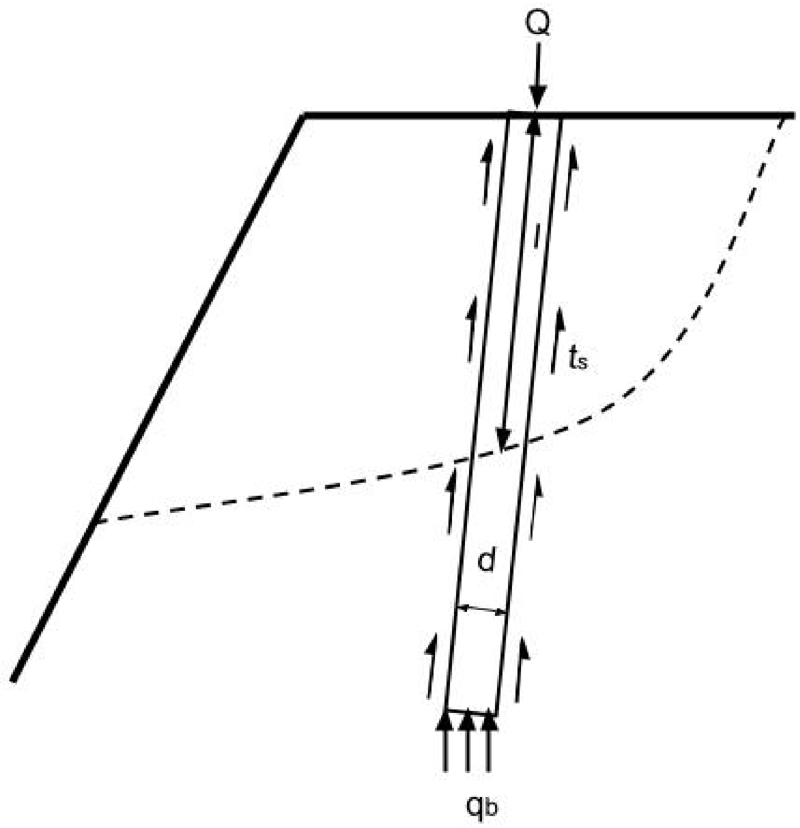


Figure 3.15: Model to calculate the pile capacity. Modified from Fleming et al. (2009).

In Figure 3.15 the following parameters are shown:

d - Pile diameter
l - Pile length to slip surface
τ_s - The limiting shear stress down the pile shaft
q_b - The end-bearing pressure
Q - Ultimate capacity

Following calculations which are used to understand the capacity of the piles, are based on the capacity in cohesive soils. To see the exact calculations and how the implementation of the capacity was inserted in Slope/W, see Appendix D. As mentioned before in the chapter “Pile axial capacity” from the Theory, the piles must work short- and long-term which depends on drained and undrained conditions. The base capacity of the piles is based on the undrained shear strength of the clay, cu, and a bearing capacity factor, Nc. The shaft friction around the pile shaft is obtained from the undrained shear strength of the soil and the empirical factor, α , is based on the function of the strength ratio, $cu/\sigma\prime_v$. The following equations are used to calculate the ultimate capacity:

$$\tau_s = \alpha \cdot cu \tag{3.6}$$

$$q_b = N_c \cdot cu \tag{3.7}$$

$$Q_s = \tau_s \cdot A_s \tag{3.8}$$

$$Q_b = q_b \cdot A_b \tag{3.9}$$

$$Q = Q_s + Q_b \tag{3.10}$$

where:

cu - Undrained shear strength
α - Empirical factor ($\frac{cu}{\sigma'_v}$)
N_c - Bearing capacity factor
A_s - Area of the pile shaft
A_b - Area of the pile base
Q_s - Shaft capacity
Q_b - Base capacity

3.4 Sensitivity analysis

A sensitivity analysis is carried out in SLOPE/W and Excel to be able to see how different parameters will affect the stability of the slope. The analysis is done for one of the studied sections (840/875) and for both combined and undrained conditions. Section 875/886 is not studied since the results from the sensitivity analysis are assumed to follow the same trend for both sections.

The change for the factor of safety will be carried out when changing the data for chosen parameters. Three of the changed factors are assumed to be natural changes and the other four are assumed to be changes made by humans. To see how the changes were done in Slope/W, see Figure 3.16 and Figure 3.17. Those values will be compared to the original conditions which can be seen in the slope today, see Appendix G. This will give an understanding of how the stability of the slope will change in connection to the different parameters, and which factors will be the most critical for the stability. The following parameters are assumed to affect the stability and could change over time in reality.

3.4.1 Natural changes

The natural changes which primarily are not assumed to be affected by humans are:

- Change of groundwater level (in blue)
  - The analysis of the groundwater level is conducted by raising and lowering the groundwater level by one meter at the time.
- Change of water level in Lärjeån (in green)
  - The analysis of the water level in Lärjeån is performed by raising and lowering the water level by one meter at the time.
- Erosion of filling material (in purple)
  - The analysis of the erosion is conducted by removing a portion of the filling material by the toe of the slope. The portion of the filling material is removed horizontally by one meter at a time.

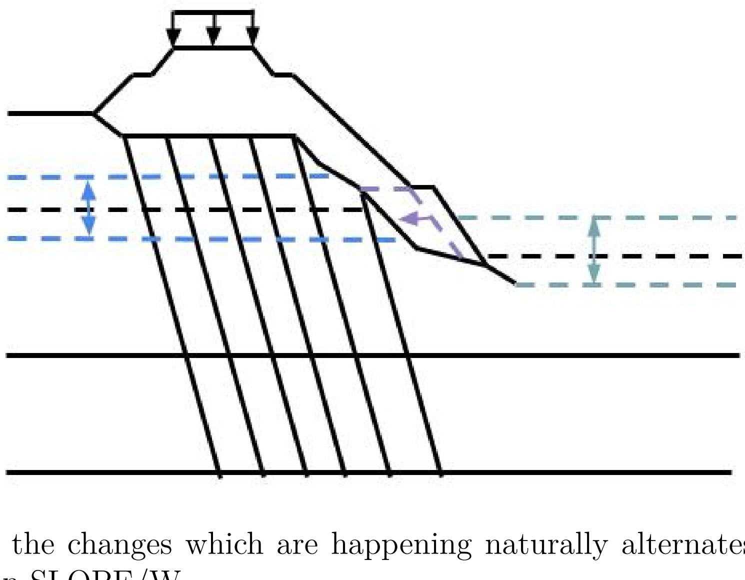


Figure 3.16: How the changes which are happening naturally alternates for the sensitivity analysis conducted in SLOPE/W.

3.4.2 Human changes

The changes which primarily are assumed to be caused by humans are:

- Change of slope angle (in blue)
  - The analysis of the slope angle is performed by adding or removing material in the slope, to get a change of the toe angle. The material is removed horizontally by one meter at a time.
- Heavier tram load (in pink)
  - The analysis of the tram load is conducted by increasing the load on the slope.
- Installation of pressure bank (in green)
  - The analysis of a pressure bank is performed by increasing the amount of filling material horizontally by one or two meters at a time.
- Lowering the embankment (in purple)
  - The analysis of the embankment height is performed by lowering the height of the embankment by one meter at a time.

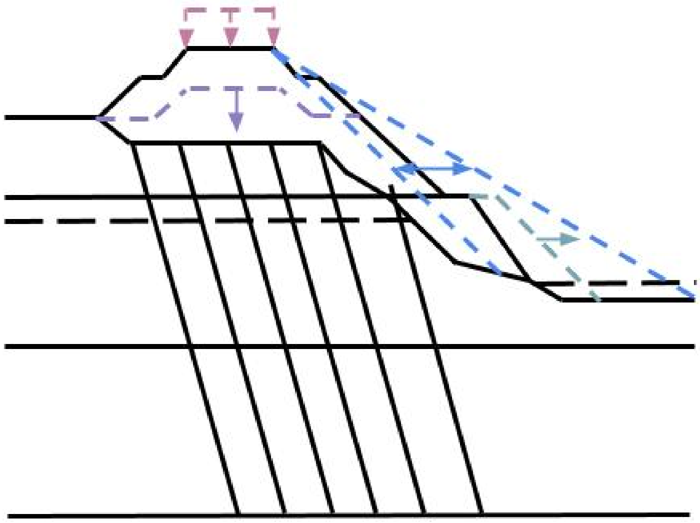


Figure 3.17: How the changes that are caused by humans alternates for the sensitivity analysis conducted in SLOPE/W.

4

Results

In this chapter the results from calculations conducted by the Direct method, SLOPE/W and the Sensitivity analysis are presented.

4.1 Direct method

From the calculations for the Direct method, see Appendix C, the factors of safety are obtained as follows in Table 4.1. The results show that none of the scenarios accomplish the required factor of safety. Also, most of the results for the drained conditions are lower than for the undrained conditions, however, the drained FS is in relation to the required FS closer to being acceptable than for the undrained.

Table 4.1: Calculated factor of safety for the three scenarios compared with required factor of safety according to "Commission on slope stability".

Analysis	Calculated FS	Required FS
Natural slope [1]		
Drained	0.85	>1
Undrained	0.84	>1
Excavated slope [2]		
Drained	0.91	>1.5
Undrained	1.11	>2
Excavated slope with load [3]		
Drained	0.85	>1.5
Undrained	1.04	>2

4.2 Slope/W

The results for SLOPE/W are divided into showing the most critical slip surfaces and the factors of safety for all scenarios, this is done for both sections and for combined- and undrained conditions.

4.2.1 Slip surfaces

The most critical slip surfaces after inserting the pile capacity together with the soil properties for the existing slope can be seen in Figure 4.1-4.4. For remaining, slip surfaces see Appendix E. For the combined condition in section 840/875, the most critical slip surface is shallower and fewer piles are contributing to the stabilisation of the slope. For the combined conditions, either the undrained or drained parameters will be active along the slip surface. When the undrained parameters are active, friction strength will be generated in the soil. When the drained parameters are active, there will be no friction strength in the soil. To see how the active parameters are varying along the slip surface see Appendix H.

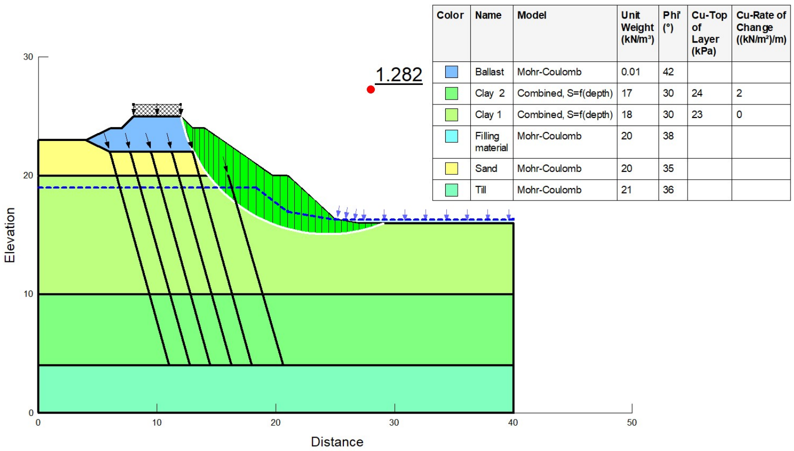


Figure 4.1: Critical slip surface for section 840/875 during combined conditions.

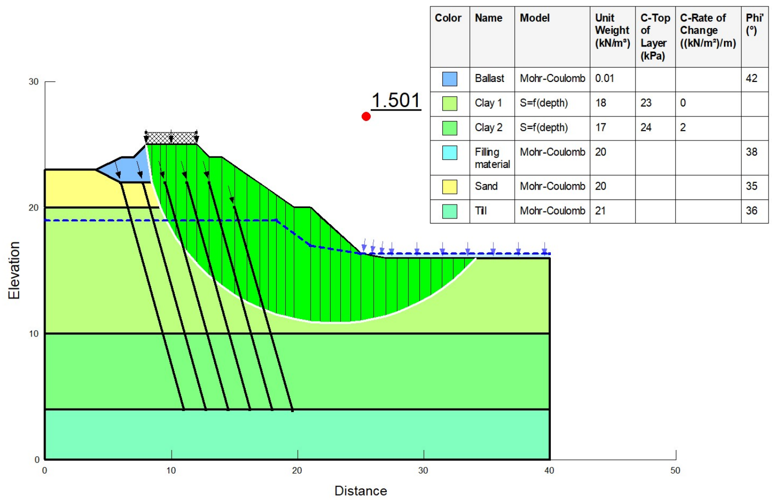
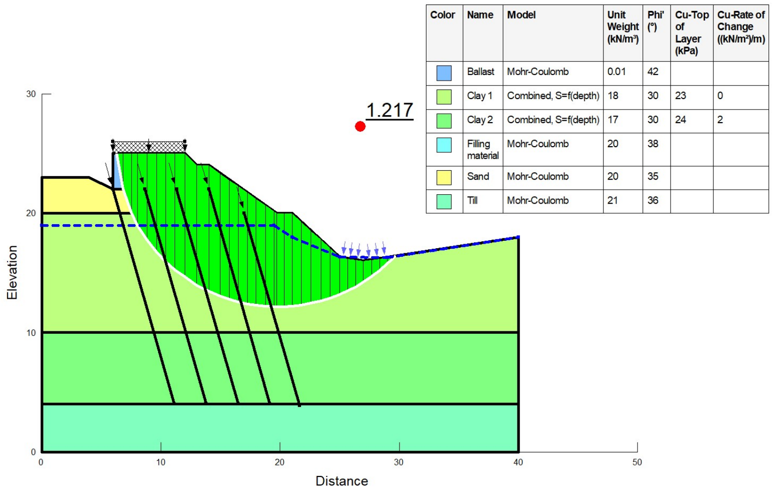
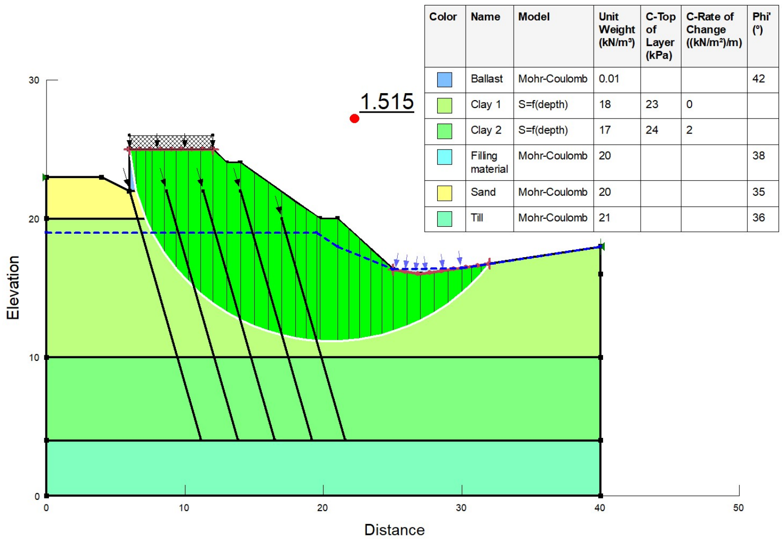


Figure 4.2: Critical slip surface for section 840/875 during undrained conditions.



Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Cu-Top of Layer (kPa)	Cu-Rate of Change ((kN/m²)/m)
	Ballast	Mohr-Coulomb	0.01	42		
	Clay 1	Combined, S=f(depth)	18	30	23	0
	Clay 2	Combined, S=f(depth)	17	30	24	2
	Filling material	Mohr-Coulomb	20	38		
	Sand	Mohr-Coulomb	20	35		
	Till	Mohr-Coulomb	21	36		

Figure 4.3: Critical slip surface for section 875/886 during combined conditions.



Color	Name	Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	Phi' (°)
	Ballast	Mohr-Coulomb	0.01			42
	Clay 1	S=f(depth)	18	23	0	
	Clay 2	S=f(depth)	17	24	2	
	Filling material	Mohr-Coulomb	20			38
	Sand	Mohr-Coulomb	20			35
	Till	Mohr-Coulomb	21			36

Figure 4.4: Critical slip surface for section 875/886 during undrained conditions.

4.2.2 Factor of safety

Following factors of safety are obtained from the simulations and are then compared with the required factors of safety according to “Commission on slope stability”, see Table 4.2. In general, it can be seen that the factors of safety are not reaching the required level. When inserting piles an improvement can be seen. For both sections 840/875 and 875/886, the slope is assumed to be safe for the undrained conditions where the excavated slope with load, reinforced with piles and where the excavated slope reinforced with piles are set up in the models.

Table 4.2: Simulated factor of safety for the five different scenarios, for both sections, compared with required factor of safety according to “Commission on slope stability”.

Analysis	FS Section 840/875	FS Section 875/886	Required FS
Natural slope [1]			
Combined	1.008	0.949	>1
Undrained	1.039	1.005	>1
Excavated slope [2]			
Combined	0.680	0.721	>1.35
Undrained	1.089	1.136	>1.5
Excavated slope reinforced with piles [3]			
Combined	1.121	1.015	>1.35
Undrained	1.615	1.597	>1.5
Excavated slope with load, reinforced with piles [4]			
Combined	1.282	1.217	>1.35
Undrained	1.501	1.515	>1.5
Excavated slope with load [5]			
Combined	1.072	1.054	>1.35
Undrained	1.199	1.165	>1.5

4.3 Sensitivity analysis

The results from the sensitivity analysis are presented in the chapters below based on the human- and natural changes.

4.3.1 Natural changes

The following results are obtained for the changes of the groundwater level, water level in Lärjeån and the amount of erosion.

4.3.1.1 Groundwater level

The original groundwater level is set at a level 0 and is marked in blue, see Table 4.3. The results show that lowering the groundwater level leads to an increased factor of safety. It can be seen that the combined scenario reaches a constant value when lowering the level, this is due to the fact that the critical slip surface looks the same and isn't reaching the groundwater level for these changes. The change will therefore not lead to a factor of safety above 1.35. The undrained scenario is not studied since there will be no change for the factor of safety in this case.

Table 4.3: Results from changing the groundwater level and the corresponding factor of safety.

Change of GWL [m]	Change in FS Combined
+2	1.108
+1	1.181
0	1.282
-1	1.305
-2	1.303

4.3.1.2 Water level

The original water level is set at a level 0 and is marked in blue, see Table 4.4. The results show that the factor of safety increases when rising the water level. The combined scenario reaches a stable state when increasing the level 2 meters with a factor of safety at 1.465. The undrained scenario is already safe at its main state and reaches a slightly smaller value when removing all the water in Lärjeån, but it still reaches the required factor of safety.

Table 4.4: Results from changing the water level and the corresponding factor of safety.

Change of WL [m]	Change in FS Combiend	Change in FS Undrained
+2	1.465	-
+1	1.303	-
0	1.282	1.501
-0.076	1.273	1.459

4.3.1.3 Erosion

The original amount of filling material before erosion is at 0 and is marked in blue, see Table 4.5. To see the amount of material that is removed for each changed meter, see Appendix G. The results show that the factor of safety decreases when the amount of filling material decreases due to erosion. The factor of safety decreases a bit faster for the undrained scenarios than for the combined scenario when removing some of the filling material.

Table 4.5: Results from erosion and the corresponding factor of safety.

Erosion [m]	Change in FS Combined	Change in FS Undrained
0	1.282	1.501
-1	1.197	1.215
-2	1.100	1.162
-3	0.818	1.095

4.3.2 Human changes

Following results are obtained for the changes of the slope angle, tram load, pressure bank and embankment height.

4.3.2.1 Slope angle

The original slope angle is at 34 degrees and is set to 0, see Table 4.6. The results show that the stability for the combined condition increases when lowering the slope angle. For the combined scenario, when the width of the slope is decreased by 5 m and the slope has an angle of 29 degrees, the factor of safety reaches an acceptable variable at 1.35. For the undrained scenario, the stability is not very affected.

Table 4.6: Results from changing the slope angle and the corresponding factor of safety.

Changing the Slope angle [m]	Change in FS Combined	Change in FS Undrained
-5 [29°]	1.345	-
-3 [31°]	1.327	-
0 [34°]	1.282	1.501
+3 [37°]	1.258	1.515
+8 [42°]	0.754	1.517

4.3.2.2 Tram load

The original tram load is at 15 kN/m2 and is marked in blue, see Table 4.7. The results show that the stability of the slope is decreasing when increasing the tram load. How much the factor of safety is affected by the added load will also depend on how big the slip surface is and how much of the soil that is working as a resisting force.

Table 4.7: Results from changing the tram load and the corresponding factor of safety.

Changing the Tram load [kN/m^2]	Change in FS Combined	Change in FS Undrained
15	1.282	1.501
20	1.270	1.460
40	1.154	1.312
60	1.048	1.185
80	0.953	1.075
100	-	0.982

4.3.2.3 Pressure bank

The original width of the filling material is set at 0 m and is marked in blue, see Table 4.8. To see the amount of material that is added for each changed meter, see Appendix G. Due to the fact that the undrained scenario is safe at its original state, only the combined scenario is analysed. The results show a big increment of stability when adding more material to the pressure bank. When adding 3 m to the width of the original filling material the factor of safety reaches an acceptable variable at 1.356.

Table 4.8: Results from adding filling material and the corresponding factor of safety.

Adding Pressure bank [m]	Change in FS Combined
0	1.282
+2	1.317
+3	1.356

4.3.2.4 Embankment height

The original height of the embankment is at 3 meters and is in the table set at level 0, see Table 4.9. The results show an increment of the factor of safety when lowering the embankment height, but not enough to reach the required factor of safety. The undrained scenario is already safe and is therefore not studied in this case.

Table 4.9: Results from changing the embankment height and the corresponding factor of safety.

Change of Embankment height [m]	Change in FS Combined
0	1.282
-1	1.301

4.3.3 Rate of change

To easier get an understanding of the effects of the change for the various factors, a trend line is set for all the scenarios. The results from the rate of change can be seen in Table 4.10. To see the graphs and calculations for the rate of change see Appendix F.

The most critical factor in connection with the stability, which happens naturally in the slope is for the combined conditions the change due to erosion, and for the undrained conditions the change of the water level. When looking at the action that can be made by humans, the change of the pressure bank will give the highest increased stability for the combined conditions. The other changes will not affect the stability significantly. The change that will affect the stability the least is the change of slope angle.

Table 4.10: Rate of change for the results from the sensitivity analysis.

Factor	Rate of change Combined	Rate of change Undrained
Natural changes		
Groundwater level	-0.0512	-
Water level	0.0855	0.5523
Erosion	0.1489	0.1271
Human changes		
Slope angle	-0.0426	-0.0018
Tram load	-0.0052	-0.0062
Pressure bank	0.0439	-
Embankment height	-0.019	-

5 Discussion

5.1 Site investigation

From the site investigation, it is clear that there is no quick clay in the slope. Previous events and studies have shown that the area around Göta älv has a lot of quick clay, something that has led to extreme damages. It is therefore important to be extra careful and observant. If a landslide occurs at the site it could possibly trigger a quick clay landslide in another part at Göta älv. If that would happen it could lead to dire consequences, like loss of life and infrastructure.

It is also known that there is a large area which today is used as an allotment area, which is located next to the studied tramway. The area with the allotments is fairly unutilized at the moment and can therefore be seen as an area with the potential to be used for something else in the future. This statement is mainly based on the fact that the area of Gamlestaden is expanding and more people are assumed to live in those areas in the future. If the allotment area would be used for another purpose in the future, that could affect the conditions at the site of the slope, such as change of the groundwater level. Building in the area will also lead to heavier loading and impacts on the soil.

5.2 Application of international methods

For this thesis, the method used has mainly been based on the Swedish method. The process for the study has to a large extent been following the guidelines from SGI since a lot of information regarding this topic is easily accessed. SGI is an expertise institution under the Swedish government and is known as a strong and reliable source which is another reason for mainly following their guidelines. During the years at Chalmers, the teaching has been based a lot on the Swedish methodology and is something that comes naturally even in this thesis.

When comparing the Swedish methodology with the methods used in Norway and Canada, there are some differences that could be useful even in Sweden. In Norway, the method takes into consideration the consequences if a landslide would occur, such as a number of dwellings, effects of flooding in the area, importance of roads e.g and is something that is not done in the Swedish methodology. This is something that could be useful even in Sweden since the consequences could be more or less severe in different areas depending on the location, something that could help with how to prioritise money and time for different investigations.

In Norway, the presence of quick clay has to be investigated which is not the case in Sweden. In the western parts of Sweden, the main reason for landslides occurring is the large amounts of quick clays in the region. For these parts of the country, the investigation connected with the presence of quick clay could therefore be mandatory. Different parts of the country with different properties of the soil, could therefore potentially have different methodologies to some extent.

In the Canadian methodology, the hazard zonation is divided into smaller zones than in Sweden. The division into smaller zones is something that could be more appropriate in terms of getting the correct results for the area. When larger areas are investigated, the probability of getting a larger variation of the data increases. Something that could further increase the uncertainty and reduce the accuracy in the area, possibly leading to an incorrect result for the site.

5.3 Data and reliability

The drawings which are used for the project were drawn in 1930. Changes in the area such as the inclination of the slope, slope angle and the height from the top of the layering to the bottom of the inclination could have changed slightly over time and could therefore be a source of error.

In geotechnics, a big part is being capable of gathering a lot of data, analysing the data and transforming it into useful results to further be able to make correct assumptions and decisions. This can to some extent be hard to do exactly according to reality without some human errors. If the knowledge, understanding and the amount of correct data is good enough, these human errors should get smaller and not have a huge impact on the results.

The amount of data used, the quality of the data and how well the data corresponds to our sections can be questioned. The data which is used in this thesis is only based on one CPT for borehole 1305, which is located close to section C, see Figure 2.4. The sections which are studied in this thesis are located closer to section D. The data which is used is therefore assumed to correspond to the data gathered closer to section C which to some extent could be incorrect for this case. To be able to get a more correct understanding of the reality and to get the correct data for the used sections, more CPTs and laboratory testing would be needed.

Additional data which could be relevant for the study could for example be the preconsolidation pressure, more strength profiles and the water level variation in Lärjeån. Some factors that could have a positive impact on the slope, are not possible to be taken into consideration when conducting the calculations and simulations. Such factors are for example, trees and other vegetation growing in the slope, where the root system could act as reinforcement in the slope when the critical slip surface is shallow. Another factor that could decrease the stabilisation in the slope which is not studied is the effect of pollution on the site.

5.3.1 Direct method

As mentioned earlier in the thesis, the direct method is a general way of getting a first understanding of the safety of a slope. The method takes into consideration a simple geometry with general properties. To make the geometry as general as the method requires, simplifications need to be done to be able to make the calculations properly, see Figure 3.1. This method is also based on user driving parameters from tables which could lead to human errors if not driving these parameters properly. This can further affect the results and diverge from reality.

5.3.2 SLOPE/W

The calculations of the piles' effect on the stability, and the input data used in SLOPE/W for the simulations with the piles, is in general connected with a lot of complexity. If using piles as reinforcement in SLOPE/W, the only parameter which can be inserted in the program is the shear resistance. The shear force depends on many different factors such as anisotropic properties including permeability and drainage conditions, which is information that is lacking for the site. This type of calculation will for this thesis therefore not be possible to perform. The user-defined reinforcement is in this case better to use, to be able to calculate the axial capacity in the slip surface to then be inserted into the software. This will define the reinforcement and will represent the piles´ effects on the stability.

When using the entry-exit method in SLOPE/W, which is a way of defining between which points the slip surface is allowed, it is very important how the entry and exit segments are drawn since this will affect the factor of safety. It is important to have an understanding of where in the slope it is reasonable for the slip surface to start and end in both the combined and undrained scenarios. Exactly where the slip surface should start and end is in this case assumed and can vary a bit from reality and therefore affect the results.

Instead of using the entry-exit method in SLOPE/W, the grid and radius option is an alternative. The grid and radius option is conducted by drawing a grid over the slope where the slip surface is considered to occur and then specifying potential center points for the circular slip surfaces. This option also depends on the user specifying critical input to obtain the factor of safety, which could also lead to some uncertainties for the outcome. Meaning that both choices of the method still rely on assumptions and could affect the results.

The limitation with using SLOPE/W is that the Limit Equilibrium analysis does not consider strain and displacement compatibility. These limitations together with the user-based problematics could be eliminated by using a Finite Element method-based program.

5.3.3 Sensitivity analysis

When analysing the rate of change, it is important to survey how many possible changes that can be carried out for every factor that can have an impact on the stability of the slope. The number of changes that can be implemented on the factors which are used in this thesis varies a lot. Some factors can only have one or two reasonable changes, for example changing the height of the embankment, while other factors can have many more. The fact that the rate of change is based on a trend line, it matters if the trend line represents the changes between two points or more. If there are more points, a more realistic rate of change will be reached.

It is also important to understand what the rate of change represents for the different factors. The rate of changes may be high for some factors, for example changing the water level in Lärjeån, but even if there would be no water in Lärjeån the slope would be safe during undrained conditions and the factor of safety would not be considerably lower for the combined conditions. This means that the high rate of change in this case doesn't necessarily indicate that the slope is sensitive towards this change. It is therefore crucial to not only look at the rate of change to see which factors that the slope is more sensitive towards.

5.4 Results from calculations

5.4.1 Direct method

The results from the calculations performed with the Direct method show that none of the scenarios are stable enough to reach the required factor of safety which are conducted from Skredkommissionen. For all three scenarios, the undrained factor of safety is higher than for the drained. This is mainly assumed to depend on the fixed state and the constant level of water in the soil for the undrained scenarios. When calculating the drained scenarios, the water is "allowed" to move more freely and the variation will therefore lead to more uncertainty and contribute to a lower factor of safety.

Even though none of the three scenarios are relevant for how the slope and its stability are today, the purpose of the calculations was to see the historical development of the stability and whether or not the implementation of reinforcement could be motivated. The calculations would also show whether or not more detailed calculations would be needed.

The results for the drained scenarios, which are assumed to correspond to the long-term situation, show that a landslide would most probably occur for all three scenarios since the factor of safety is below one. This in combination with that none of the required factors of safety are reached, motivates that furthermore detailed calculations are needed and indicates that the reinforcement is motivated, which corresponds to the assumptions. In consideration of the above-mentioned statements, the purpose of the calculations was reached.

5.4.2 SLOPE/W

When simulating the natural slope, a factor of safety above one is reached for all scenarios except for the combined scenario for section 875/886. The factor of safety is in this case almost one and can therefore also be considered safe. When the construction of Angeredsbanan begins and the natural slope is excavated, the slope goes from a stable to an unstable state when the excavation is completed. This is reasonable since the soil and the slope are disturbed and could have lost some of their strength.

When installing the piles in the excavated slope, the factor of safety is increased and the slope is considered safe for the undrained condition but not for the combined condition for both sections. In the last step of the construction of Angeredsbanan, the embankment consisting of the ballast and the tram load is added. The slope is for the finished construction safe for the undrained condition and unsafe for the combined condition in both sections, altho the combined factor of safety is close to the required factor of safety. The results are considered reasonable due to the degree of detail for the diffident analyses.

If Angeredsbanan would be constructed without piles, the factor of safety would be considerably lower than with piles and would be far from safe. By looking at the different stages of the construction for Angeredsbanan and the factor of safety during the different stages, it is clear why the piles are installed as a reinforcement and that the piles have a significant impact on the factor of safety in the slope.

5.4.3 Comparison of Direct method and SLOPE/W

The only difference when comparing the Direct method and SLOPE/W, is that the natural slope is considered safe in SLOPE/W and not safe when calculating with the Direct method. The reason for the different outcomes is likely due to the simplifications conducted when using the Direct method. It is more favorable that the results from the Direct method are lower than those from SLOPE/W since the results from the Direct method indicate whether or not more extensive simulations in SLOPE/W or other programs using numerical modeling e.g are needed. Having lower FS for the Direct method indicates that the risk for making mistakes regarding whether or not to conduct more calculations/simulations is minimised. The results from SLOPE/W are assumed to be more correct due to the lower degree of simplifications.

5.4.4 Sensitivity analysis

In the sensitivity analysis, the effect on the stability for each factor is studied separately. It would also be of interest to study some of the factors in combination to see how they affect each other and how the factor of safety would change. In reality, it would be reasonable to think that some changes of the factors could happen simultaneously.

To be able to make a fair analysis, the probability for the changes needs to be taken into account. Practical and economical reasons can for example be aspects that make the change less probable. Also regarding the possibility for humans to affect the change needs to be taken into account.

As mentioned in the results, there are two different categories of factors that will change the stability of the slope. The first one is the factors that happen naturally in the slope. These factors can indirectly be connected with human activity but can also depend on other aspects. The second category of factors is the often direct human-affected factors. These factors can directly be changes that could be performed by humans to stabilise the slope or decrease the stability.

5.4.4.1 Groundwater level and Water level

In the thesis, the change regarding groundwater- and the water level is done by one meter at a time. A change of the water level with one and two meters is not assumed to be very realistic but is in the thesis done to see the rate of change for the factor of safety and to see how much change is needed to reach a safe or an unsafe state.

The factor of safety increases slightly when lowering the groundwater level. The value only reaches around 1.305 even if the groundwater level is decreased further, and will therefore not reach the required factor of safety. The marginal change of the factor of safety when lowering the level from -1 to -2 could depend on the fact that the total tension is a bit higher on the active side, and therefore the factor of safety gets minimally lower.

The rate of change for the undrained condition when changing the water level in Lärjeån only compares the level between the original condition and if there would be no water in Lärjeån. Even if the rate of change is high for the undrained condition, the factor of safety is still considered safe when there is no water in Lärjeån and therefore this condition is not assumed to be sensitive towards this change. For the combined condition to reach the required factor of safety, the water level would have to be increased by 1,25 meters.

An increased groundwater level and water level in Lärjeån will often depend on an increased amount of rainfall, something that humans cannot affect but indirectly can depend on a higher environmental impact caused by humans. Lowering of the two water levels can depend on a higher degree of drinking water extraction in the area, something that more directly is caused by humans.

The previous discussion concerning the unutilised allotment area in connection with the higher degree of urbanization are factors that strengthen a theory that the groundwater level and the water level in Lärjeån can follow a decreasing trend in the future. On the other hand, the earth is today facing an extensive environmental change leading to more extreme weathers, including heavier rains and rougher winds. This is further something that strengthens a theory that an increasing groundwater level and water level in Lärjeån could be seen in the future. If an increased or decreased level is to be expected in the future is therefore hard to estimate from this thesis. Further studies are needed to ensure the result.

5.4.4.2 Erosion

In the thesis the change of stability due to erosion is done by removing the filling material horizontally for 1 meter at a time, starting from the toe of the slope. A reduction of the filling material width with 3 m is not likely to be removed in the near future but is done to see the rate of change and to see how much material that would have to be removed to reach an unsafe state $FS < 1$.

From the sensitivity analysis, the most crucial change when looking at the factor of safety is erosion. Altho it would take more than half of the filling to be eroded for the factor of safety to reach below one for the combined conditions, the change is the fastest compared to the other factors in the sensitivity analysis.

The geometry of how the filling material would be removed due to erosion was very limited in the model. Only larger parts of the filling material could be taken away for each step of the analysis. How the filling material would be eroded and the geometry of it may not be completely in line with reality, but the analysis still shows that the erosion is a contributing factor for the factor of safety decreasing in the slope.

An increased erosion will mainly be caused by a higher flow and a higher water level in Lärjeån. The flow in Lärjeån is today assumed to be pretty high and small landslides have also been seen along the streambanks, something that could be the effect of erosion. Due to the environmental change with heavier rain and more wind, the problems connected with erosion are assumed to continue in the future as well. Decreased amounts of erosion on the site are therefore not very likely.

5.4.4.3 Slope angle

When alternating the slope angle, the factor of safety changes to safer values for the smaller angles and less safe for the wider angles during combined circumstances, which are the results that can be expected. When the angle is 29 degrees, the slope is safe during combined conditions. The slope is secure at the original slope angle for the undrained circumstances but when the slope angle is wider the safety factor gets marginally larger, which is the opposite of what happens for the combined conditions. The marginal change of the factor of safety could depend on the fact that the total tension is less on the active side, and therefore the factor of safety gets minimally higher.

The action of changing the slope angle in the slope is possible to perform by adding extra material and possibly move the stream a bit in connection with the slope. The material which would have to be removed would work well as the additional material needed to create the correct slope angle. The process could though be seen as a bit complicated since the exact angle is hard to get and the space on the site is limited.

5.4.4.4 Tram load

When changing the load from the trams, the sensitivity analysis indicates that an increased tram load contributes to the lowest rate of change. For the factor of safety to reach below one, it would take the load from the trams to be five times bigger than today. If new and heavier trams would be installed with a load of 20 kN/m2 it would mean an increased weight for the tram at about half a ton, but that would only provide a decrease in the factor of safety with about a hundredth unit for both combined- and undrained conditions.

How much the factor of safety is affected by the added load will depend on how big the slip surface is and how much of the soil that is working as a resisting force. If the slip surface is smaller, the change in the safety factor will be greater if more load is applied, this is due to less resisting force. The slip surface generated from increasing the tram load changes when increasing the load.

When analyzing the change of the tram load, the rate of change in connection with the stability is very small. The action of changing the weight would therefore probably be too expensive and complicated for its effect on the stability. The need for heavier trams could in the future be affected by a possible increased need for transporting more people to and from the area. From an improved environmental impact point of view, the need for using less and lighter materials could increase in the future. To what extent this would happen is hard to specify at this point.

5.4.4.5 Embankment height

If the embankment height would be decreased as an action to secure the slope, the factor of safety would increase, but not enough to secure the slope.

The change of the embankment height is seen to be an effective way to improve the stability of the slope. In the analysis, the embankment height is lowered by one meter. That reduction for the embankment height is not realistic due to the embankment being directly connected to a bridge at one end of the slope, and would therefore need alteration of the bridge as well if this would be seen as a solution to secure the slope. The struggles with the connection of the different parts could also be seen as a very costly process.

5.4.4.6 Pressure bank

When adding a pressure bank as reinforcement for the combined analysis, the factor of safety increases. A pressure bank that is three meters wide would make the slope safe. The rate of change for the pressure bank is almost the same as for the slope angle and embankment height.

The increased stability due to inserting more material in a pressure bank is assumed to be very effective. The implementation is assumed to be pretty easy since the geometry of the material is flexible and can be placed where there is space. If material is needed to be removed from the stream, this material could be used for the pressure bank which would be an advantage.

5.4.4.7 Reinforcement alternatives

The results and the discussion together strengthen that the piles which are used as reinforcement are of importance for the stability of the slope. It is also assumed that a large amount of vegetation in the area, which could not be inserted for the calculations, has an impact and further will increase the stability slightly. How much the vegetation is contributing to an increment is hard to estimate.

The results for both sections show that the slope as it is formed today is safe for the undrained conditions but not for the combined conditions. With this as a background, the assumption is made that extra reinforcement is of importance to secure the slope. Due to the already inserted piles in the slope today, some reinforcement alternatives have no possibility to be inserted to support the slope further. This is for example soil nailing and anchoring, which capacity therefore is not tried as an alternative in SLOPE/W. Another reinforcement alternative that possibly could increase the stability is the commonly used retaining wall. The implementation of the wall in SLOPE/W is hard to conduct due to limitations in the software and is therefore not done. Since erosion is one of the most critical changes in connection with stability, some protection against erosion could be implemented but can not neither be done in the software.

When studying the effects of the changed factors in the sensitivity analysis, the assumption is made that the most appropriate stability increment which could be performed by humans is the reinforcement with a pressure bank. The effects of the pressure bank work similarly as the retaining wall. When studying the rate of change and the factor of safety when adding a pressure bank, the values are high and the implementation is assumed to help stabilize the slope enough to be stable. The implementation is also pretty easy to conduct due to the flexible geometry and material possibilities, something which further can have economical advantages.

6
Conclusion

Events have shown that landslides are a severe occurring problem in today's society. The problem creates damages, both causing human lives and destroying infrastructure leading to extensive costs. This is something that will continue to be a problem in the future due to the continued need for infrastructure, increased amount of human activities, and climate change.

The results of the thesis show that the slope and its geometry have changed over time due to the expansion of Angeredsbanan. When excavating the natural slope, it can be seen that the slope loses some of its stability, going from a stable to an unstable slope when studying the factor of safety. Further piles were inserted and the slope increased its stability again, reaching the required values for the undrained analysis for both sections. Lastly, the construction of the embankment was performed which resulted in a slight reduction of the stability, still reaching the required factor of safety for the undrained but not for the combined analysis. The outcome indicates that the piles are important for the stability of the slope and are motivated as reinforcement both historically and today.

When performing a sensitivity analysis to see what possible changes that the slope stability is most sensitive towards, the natural changes: groundwater level, water level, erosion and the human changes: embankment height, slope angle, pressure bank and tram load are studied. The results show that the slope is most sensitive towards erosion, meaning that this factor is most likely to trigger a landslide on the site. The site investigation confirmed that erosion is happening at the site and is most likely to continue in the future. The amount of erosion could possibly be controlled by adding some sort of erosion protection.

Comparing the different analyses on which actions that could be done to secure the slope, factors such as the implementation process, which impact the change has on the safety factor and on the rate of change for the slope has to be taken into consideration. With this as a background, the most profitable implementation would be to add a pressure bank. This is mainly due to the relatively easy implementation process connected with the flexible geometry and usage of material together with the high impact on the stability. A pressure bank together with the already existing vegetation will make both sections for the combined- and undrained analysis safe following the required factors of safety conducted by the Commission on slope stability.

7 Further research

To further develop this thesis, more comprehensive simulations could be carried out in numerical modeling softwares. Numerical modeling softwares are better designed for implementing piles and would most likely be more in line with reality. To get more information on the piles, their capacity and their failure mechanisms, such software would be useful and an interesting further study of this slope.

This thesis has slightly touched on the effects on slope stability due to the future expected climate changes. These effects will to a high extent be increasing and are therefore of high interest to study more in detail. Investigations on possible future events would give an understanding of how the slope would behave in the future and would be of importance for long-term stability.

For the sensitivity analysis in this thesis, different factors that affect slope stability have only been studied separately. In reality, the changes of these factors could be happening simultaneously. The optimal case would be to study these changes with each other, to get a better understanding of the reality and the effects they bring on the slope stability.

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A

Appendix: Highest Shoreline historically

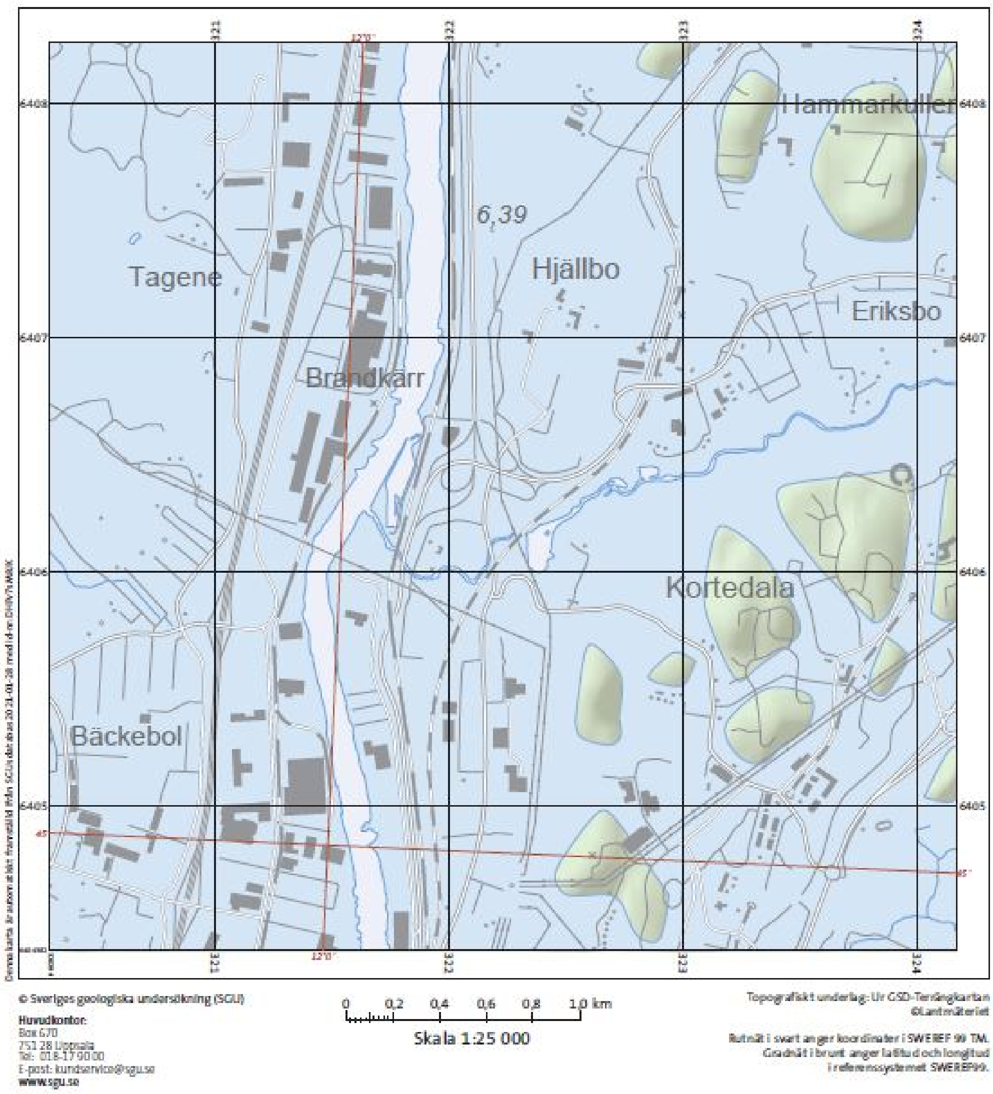


Figure A.1: SGU shoreline map, simulation of the shoreline from 15 000 years ago. Note: SGU. (w.y.). Shoreline map. Retrieved from http://apps.sgu.se/kartgenerator/maporder sv.html.

B

Appendix: Field test conducted by SWECO

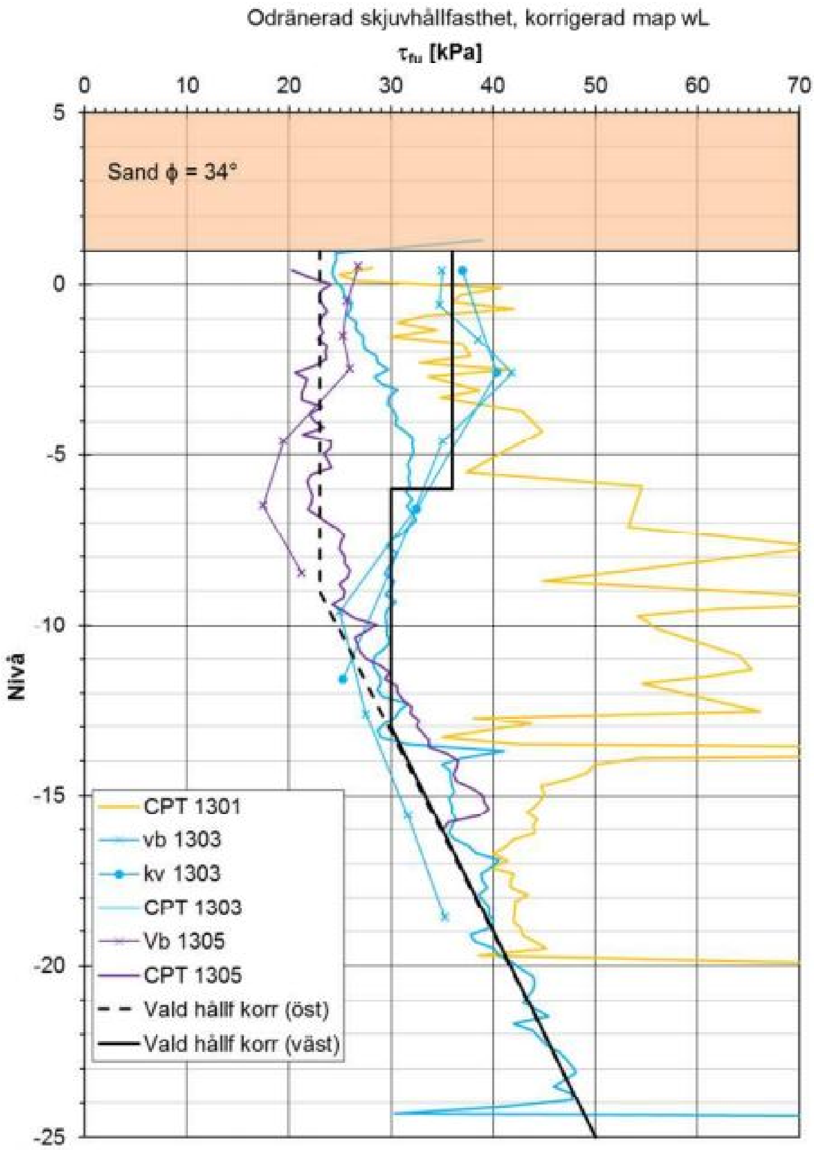


Figure B.1: Compilation of SWECOs CPT and VST tests and the undrained shear strengths. Note: Göteborgs Stad fastighetskontoret. (2014). Sammanställning och utvärdering av odränerad skjuvhållfasthet.

B.1 2.2 CPT

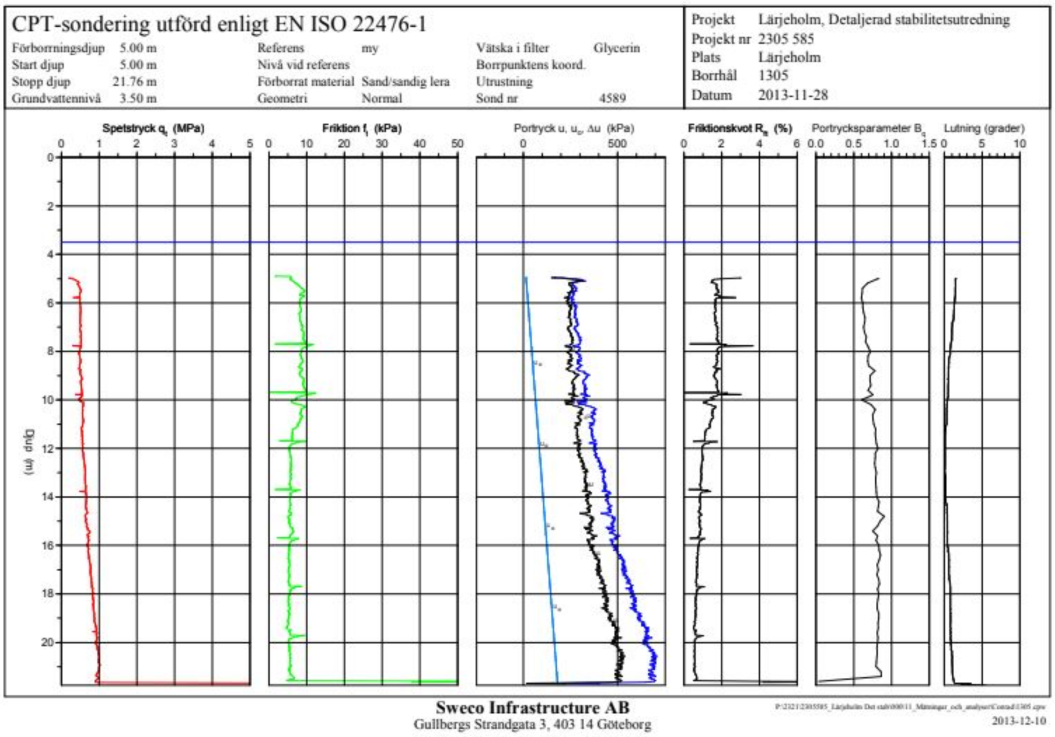


Figure B.2: SWECOs CPT for borehole 1305. Note: Göteborgs Stad fastighetskontoret. (2014). Sammanställning och utvärdering av odränerad skjuvhållfasthet.

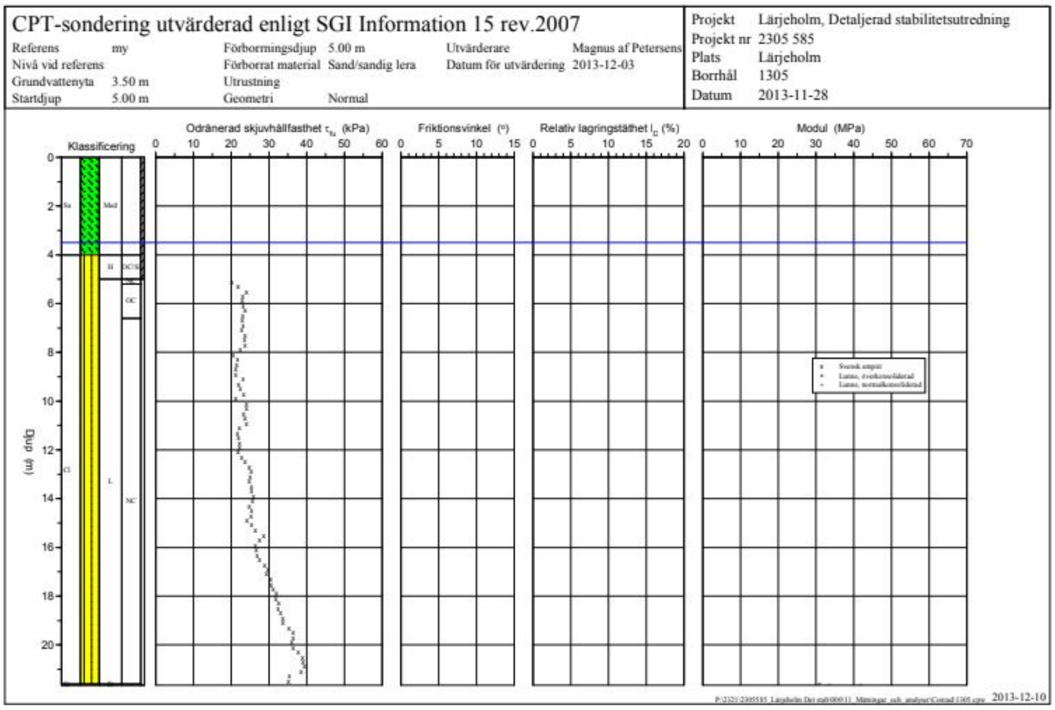


Figure B.3: SWECOs CPT for borehole 1305. Note: Göteborgs Stad fastighetskontoret. (2014). Sammanställning och utvärdering av odränerad skjuvhållfasthet.

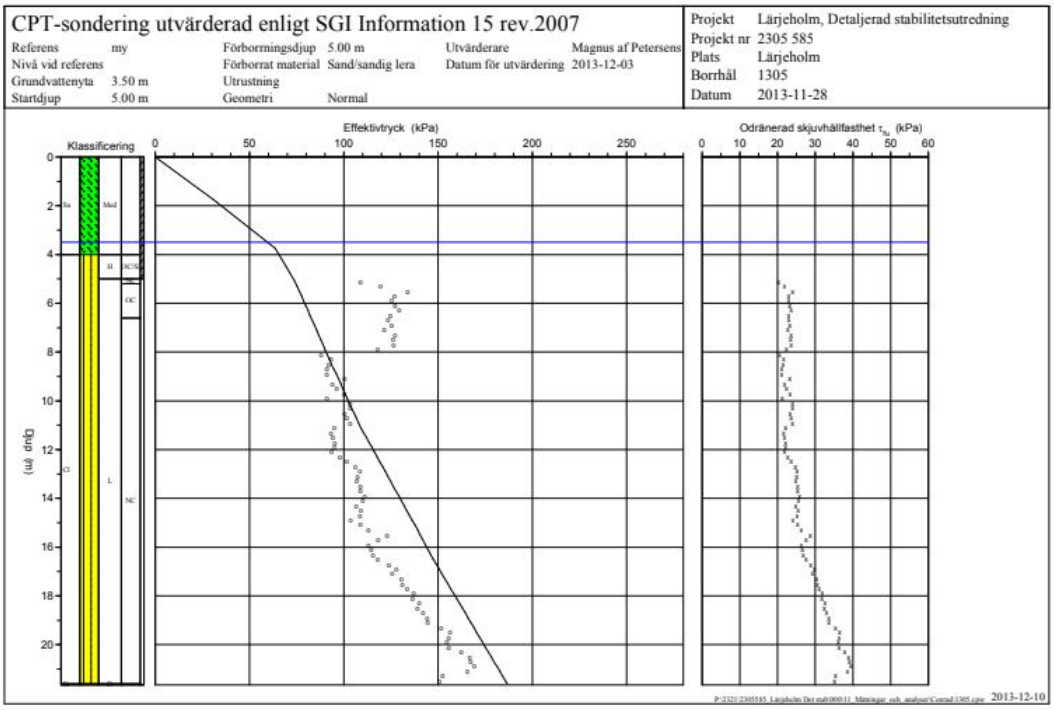


Figure B.4: SWECOs CPT for borehole 1305. Note: Göteborgs Stad fastighetskontoret. (2014). Sammanställning och utvärdering av odränerad skjuvhållfasthet.

B.2 2.3 VST

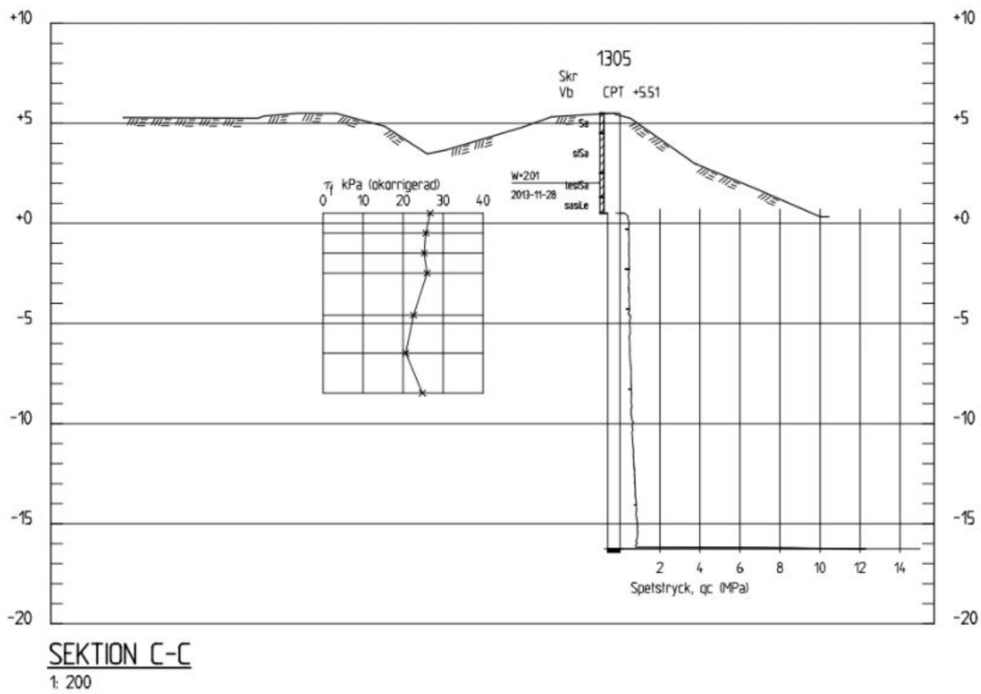


Figure B.5: SWECOs VST for borehole 1305. Note: Göteborgs Stad fastighetskontoret. (2014). Sammanställning och utvärdering av odränerad skjuvhållfasthet.

C

Appendix: Direct method calculations

Table C.1: Calculation of water level in Lärjeån according to RH 2000.

Current average water level RH2000	MW	HW
7.7 cm	0.1 cm	7.6 cm

Table C.2: Load calculations.

Load ballast	Load tram	Total Load
51 kN/m	15 kN/m	66 kN/m

Table C.3: Factor of safety calculation for undrained- and drained soil parameters.

	Natural slope	Excavated slope	Excavated slope with load
Undrained			
p_d	150.78	126.85	134.95
F_c	0.85	1.00	0.94
Drained			
p_e	116.00	77.66	77.66
$\lambda_{c\phi}$	29.11	19.49	19.49
p_d	150.78	126.85	134.96
N_{cf}	55	45	45
$\mathrm{F}_{c\phi}$	0.84	0.82	0.77

D

Appendix: Piles axial capacity calculations

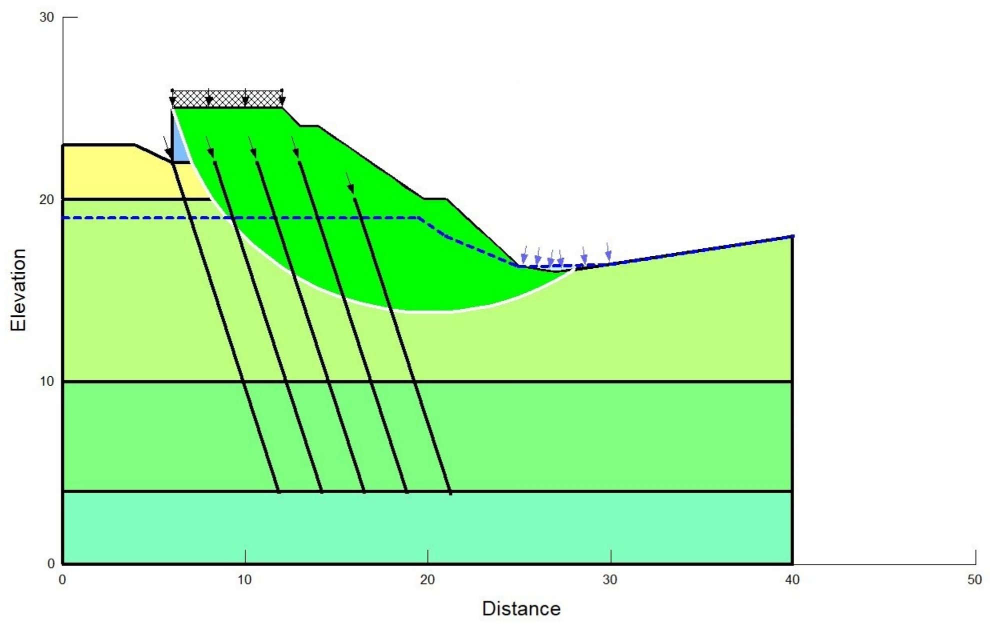


Figure D.1: The most critical slip surface together with supporting piles for the combined scenario to see the length of the pile to the critical slip surface, in section 875/886.

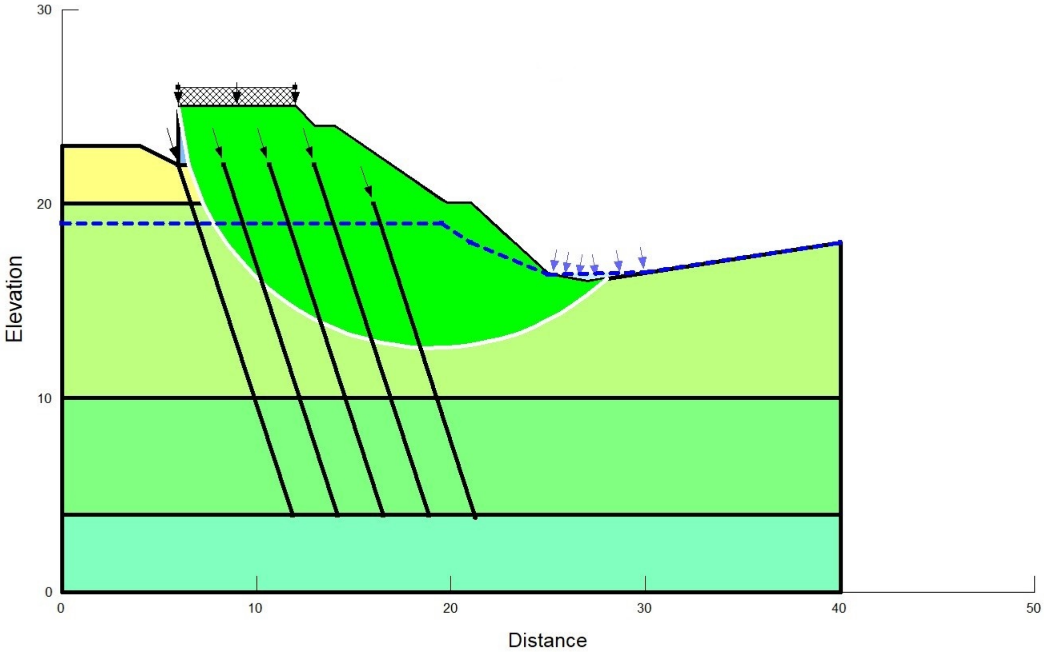


Figure D.2: The most critical slip surface together with supporting piles for the undrianed scenario to see the length of the pile to the critical slip surface, in section 875/886.

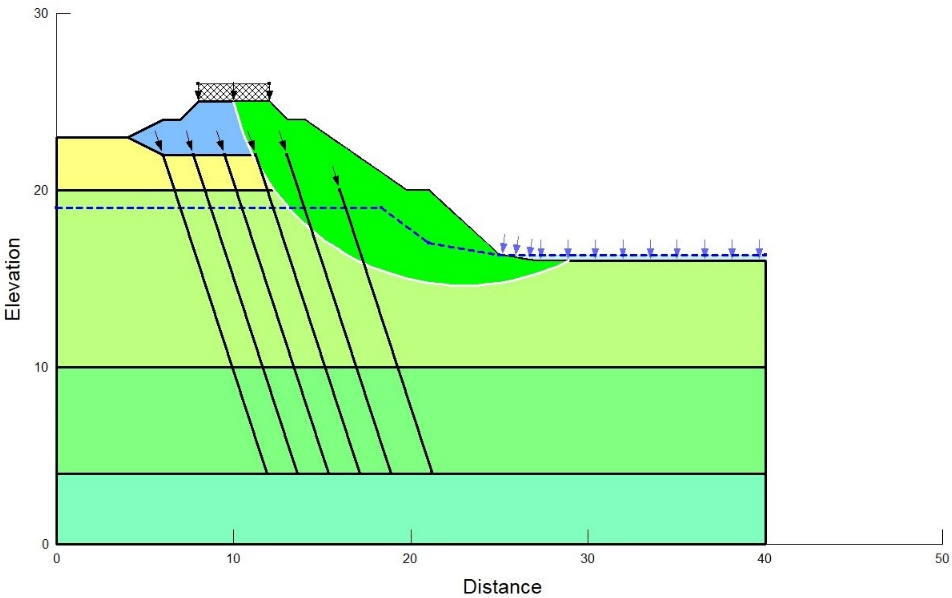


Figure D.3: The most critical slip surface together with supporting piles for the combined scenario to see the length of the pile to the critical slip surface, in section 840/875.

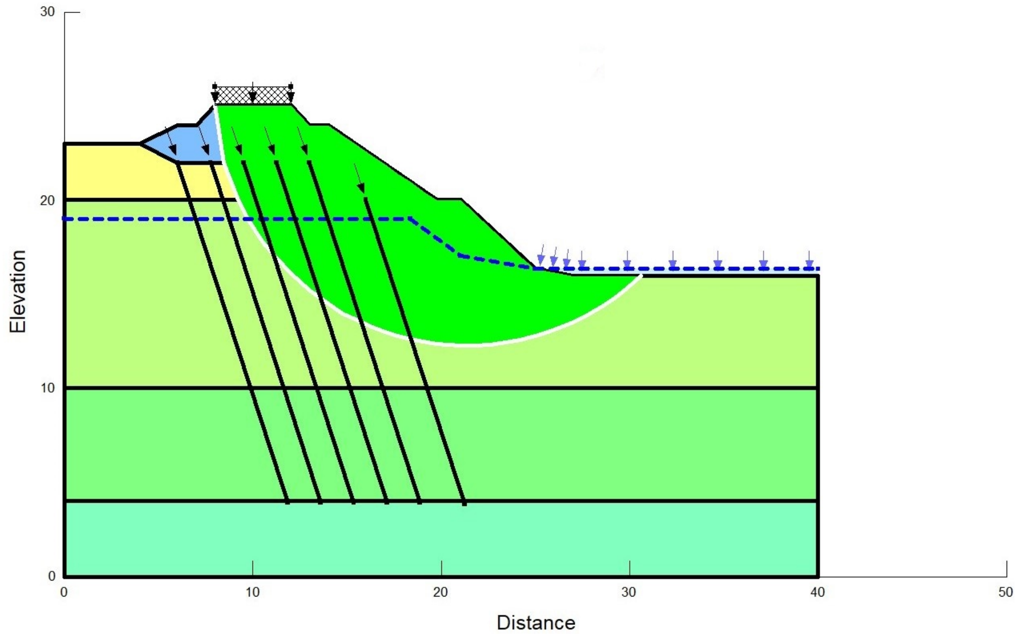


Figure D.4: The most critical slip surface together with supporting piles for the undrained scenario to see the length of the pile to the critical slip surface, in section 840/875.

D.1 Pile capacity calculations

Table D.1: In-data parameters for the different soil layes to calculate the piles capacity.

Depth [m]	Material	Unit weight $[kN/m^3]$	Cu [kPa]	σ_0 $[kN/m^2]$	u [kPa]	σ'_v $[kN/m^2]$	α [-]
1	SAND	20		20	0	20	0
2	SAND	20		40	0	40	0
3	SAND	20		60	0	60	0
4	CLAY 1	18	23	72	5	67	0.3
5	CLAY 1	18	23	90	15	75	0.3
6	CLAY 1	18	23	108	25	83	0.3
7	CLAY 1	18	23	126	35	91	0.3
8	CLAY 1	18	23	144	45	99	0.2
9	CLAY 1	18	23	162	55	107	0.2
10	CLAY 1	18	23	180	65	115	0.2
11	CLAY 1	18	23	198	75	123	0.2
12	CLAY 1	18	23	216	85	131	0.2
13	CLAY 1	18	23	234	95	139	0.2
14	CLAY 1	18	23	252	105	147	0.2

Table D.2: Axial capacity calculation for piles in section 875/886 combined.

Cohesive soil	Pile 1	Pile 2	Pile 3	Pile 4
N_c [-]	9	9	9	9
cu [kPa]	23	23	23	23
$\alpha = cu \cdot \sigma' v$[-]	0.2	0.2	0.3	0.3
$q_b = N_c \times cu$[kPa]	207	207	207	207
$\tau_s = \alpha \cdot cu$[kPa]	4.04	4.94	6.37	6.37
d [m]	0.28	0.28	0.28	0.28
L [m]	8	10	8	6
$A_b = \pi \times d^2/4$	0.062	0.062	0.062	0.062
$A_s = \pi \times d \times L$	7.04	8.80	7.04	5.28
$Q_b = q_b \times A_b$	12.75	12.75	12.75	12.75
$Q_s = q_s \times A_s$	28.42	43.49	44.85	33.64
Q=Q_b+Q_s	41.16	56.24	57.60	46.38

Table D.3: Axial capacity calculation for piles in section 875/886 undrained.

Cohesive soil	Pile 1	Pile 2	Pile 3	Pile 4
N_c [-]	9	9	9	9
cu [kPa]	23	23	23	23
$\alpha = cu \cdot \sigma' v$[-]	0.2	0.2	0.2	0.3
$q_b = N_c \times cu$[kPa]	207	207	207	207
$\tau_s = \alpha \cdot cu$[kPa]	4.30	4.30	4.60	5.81
d [m]	0.28	0.28	0.28	0.28
L [m]	9	10	9	7
$A_b = \pi \times d^2/4$	0.062	0.062	0.062	0.062
$A_s = \pi \times d \times L$	7.92	8.80	7.92	6.16
$Q_b = q_b \times A_b$	12.75	12.75	12.75	12.75
$Q_s = q_s \times A_s$	34.05	37.83	36.42	35.79
Q=Q_b+Q_s	46.79	50.58	49.16	48.54

Table D.4: Axial capacity calculation for piles in section 840/875 combined.

Cohesive soil	Pile 1	Pile 2
N_c [-]	9	9
cu [kPa]	23	23
$\alpha = cu \cdot \sigma'v$[-]	0.3	0.3
$q_b = N_c \times cu$[kPa]	207	207
$\tau_s = \alpha \cdot cu$[kPa]	6.90	6.90
d [m]	0.28	0.28
L [m]	5	4
$A_b = \pi \times d^2/4$	0.062	0.062
$A_s = \pi \times d \times L$	4.40	3.52
$Q_b = q_b \times A_b$	12.75	12.75
$Q_s = q_s \times A_s$	30.35	24.28
Q=Q_b+Q_s	43.09	37.02

Table D.5: Axial capacity calculation for piles in section 840/875 undrained.

Cohesive soil	Pile 1	Pile 2	Pile 3	Pile 4
N_c [-]	9	9	9	9
cu [kPa]	23	23	23	23
$\alpha = cu \cdot \sigma'v$[-]	0.2	0.2	0.2	0.3
$q_b = N_c \times cu$[kPa]	207	207	207	207
$\tau_s = \alpha \cdot cu$[kPa]	4.60	4.60	4.60	6.90
d [m]	0.28	0.28	0.28	0.28
L [m]	9	10	9	7
$A_b = \pi \times d^2/4$	0.062	0.062	0.062	0.062
$A_s = \pi \times d \times L$	7.92	8.80	7.92	6.16
$Q_b = q_b \times A_b$	12.75	12.75	12.75	12.75
$Q_s = q_s \times A_s$	36.42	40.46	36.42	42.49
Q=Q_b+Q_s	49.16	53.21	49.16	55.23

D.2 Piles input SLOPE/W

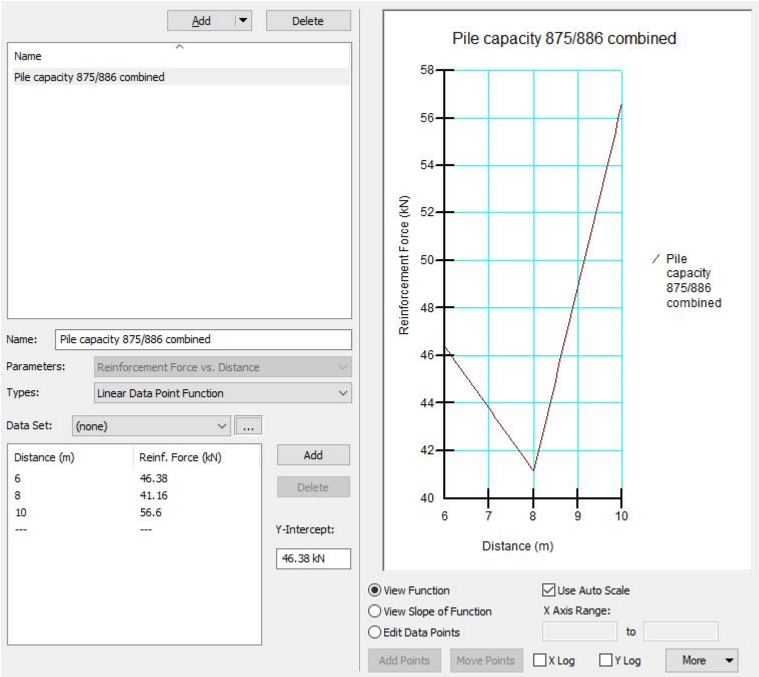


Figure D.5: Axial capacity for the piles put in SLOPE/W for section 875/886 combined.

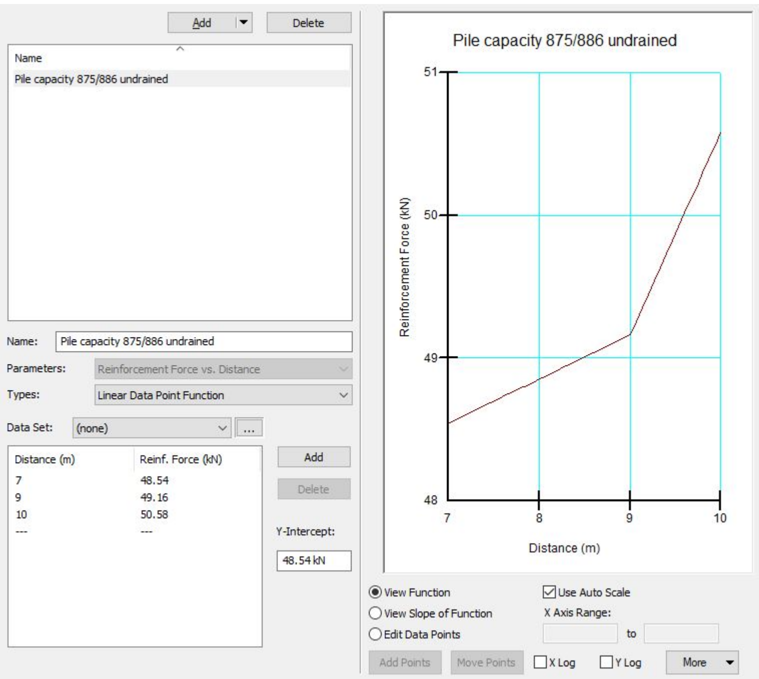


Figure D.6: Axial capacity for the piles put in SLOPE/W for section 875/886 undrianed.

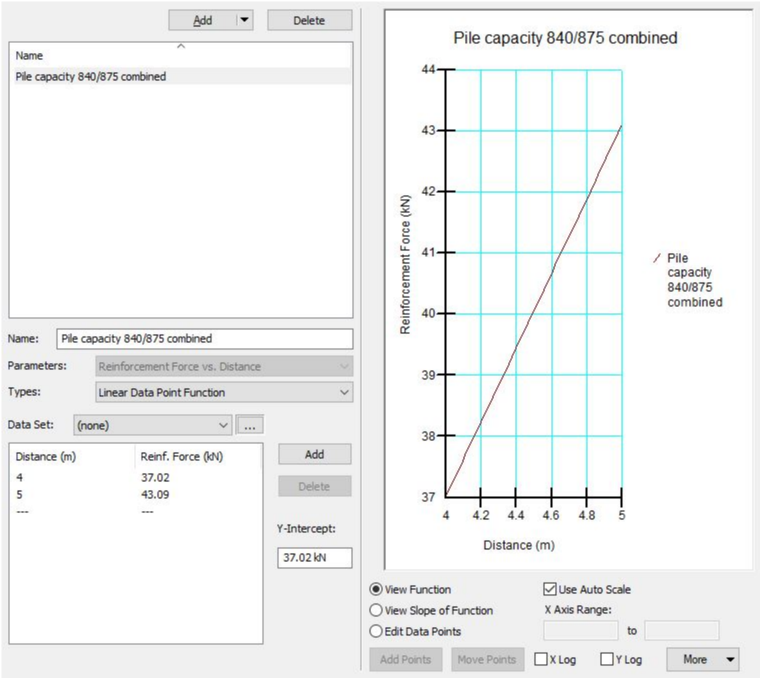


Figure D.7: Axial capacity for the piles put in SLOPE/W for section 840/875 combined.

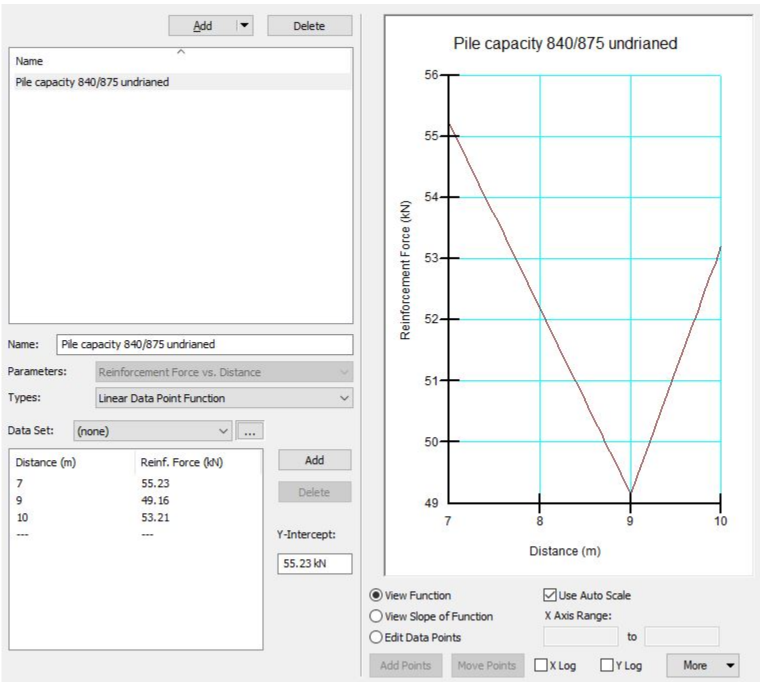


Figure D.8: Axial capacity for the piles put in SLOPE/W for section 840/875 undrianed.

E

Appendix: Slip surfaces

E.1 Natural slope

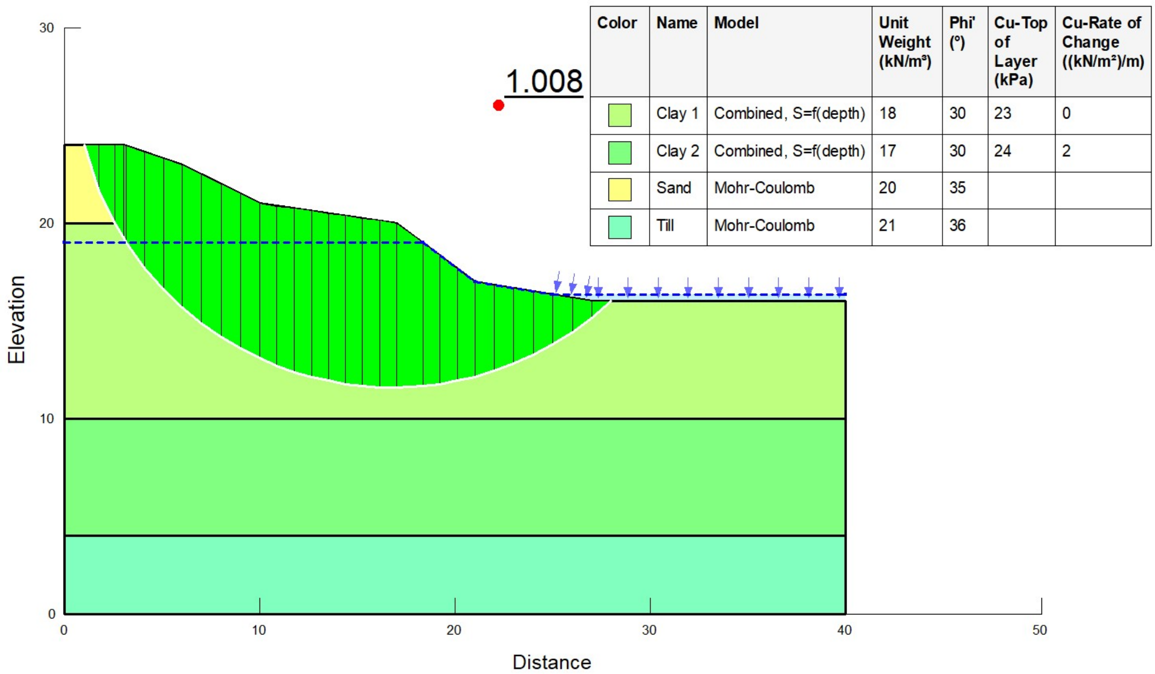


Figure E.1: Natural slope slip surface section 840/875 combined.

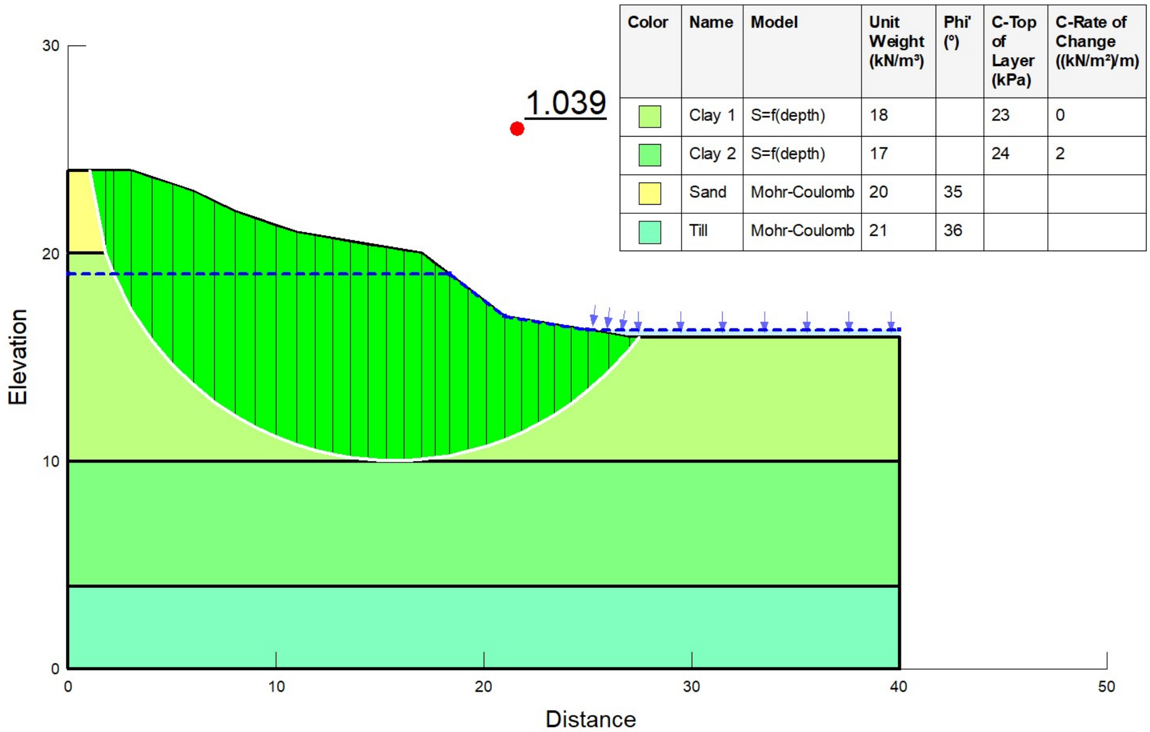


Figure E.2: Natural slope slip surface section 840/875 undrained.

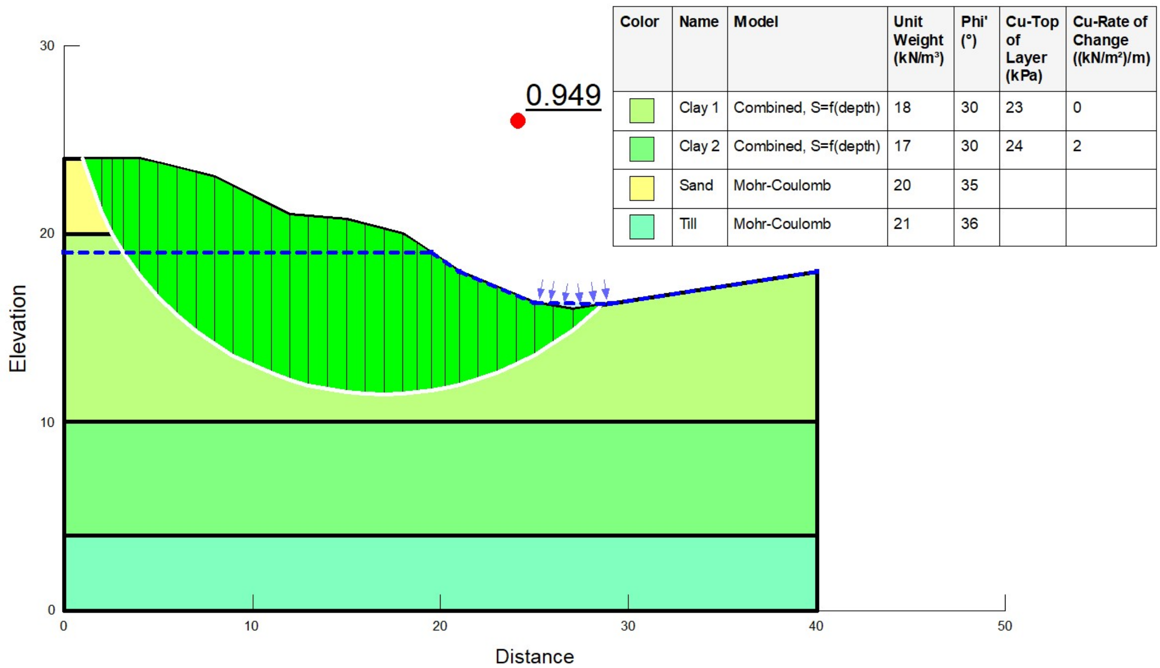


Figure E.3: Natural slope slip surface section 875/886 combined.

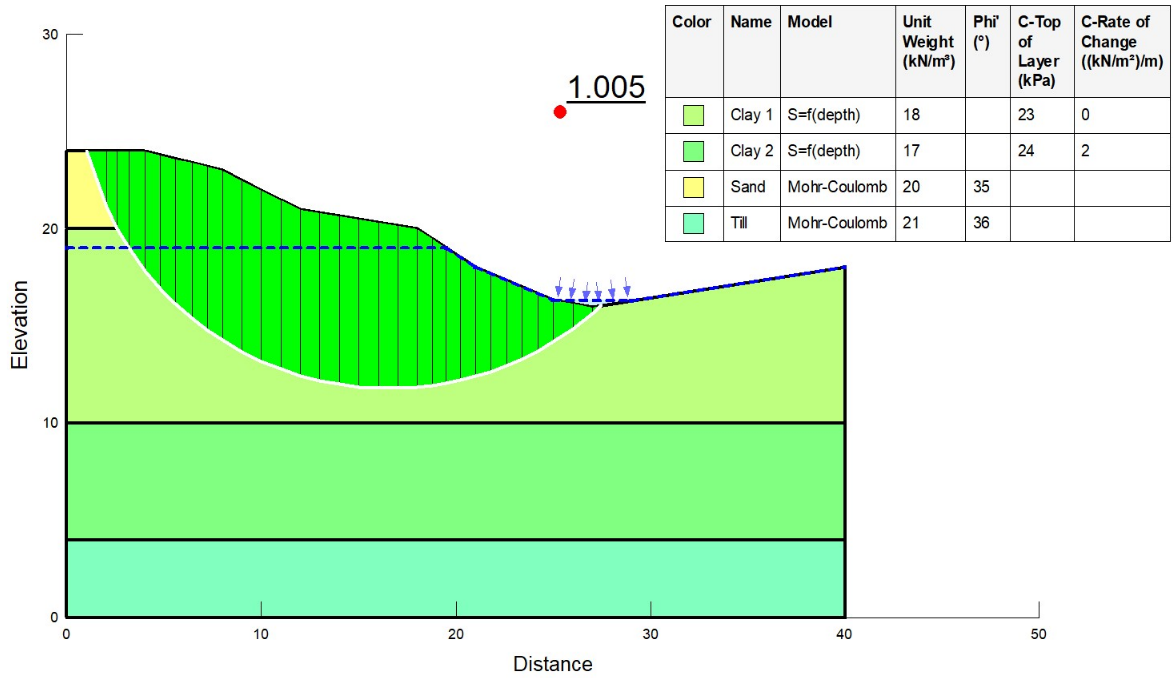


Figure E.4: Natural slope slip surface section 875/886 undrained.

E.2 Excavated slope

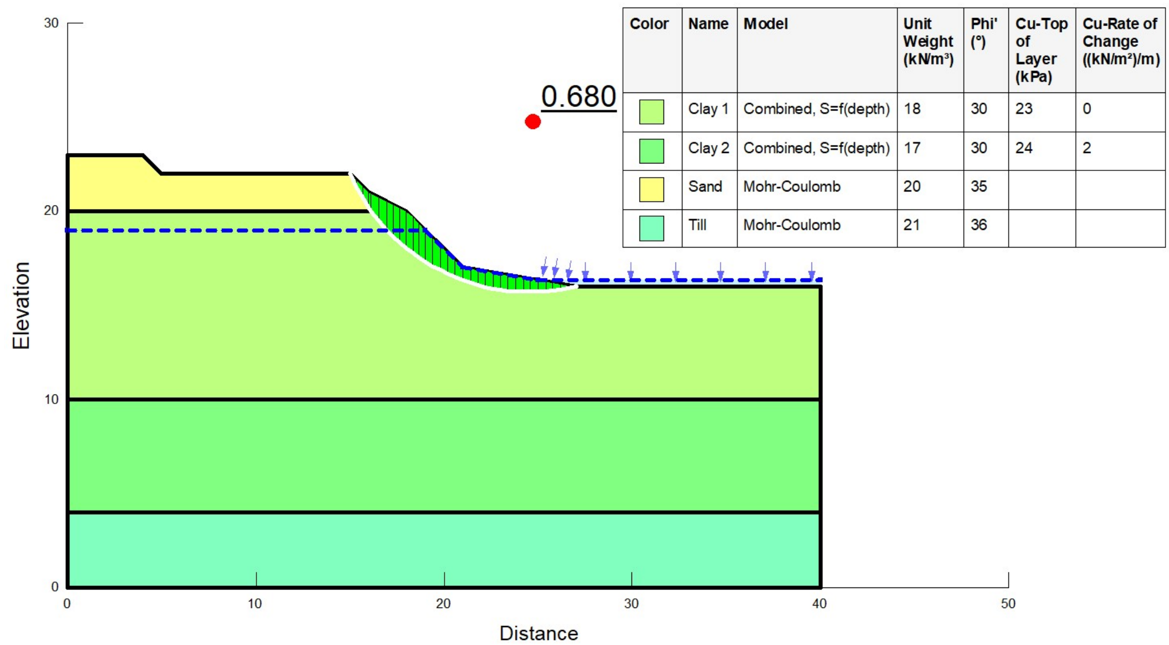


Figure E.5: Excavated slope slip surface section 840/875 combined.

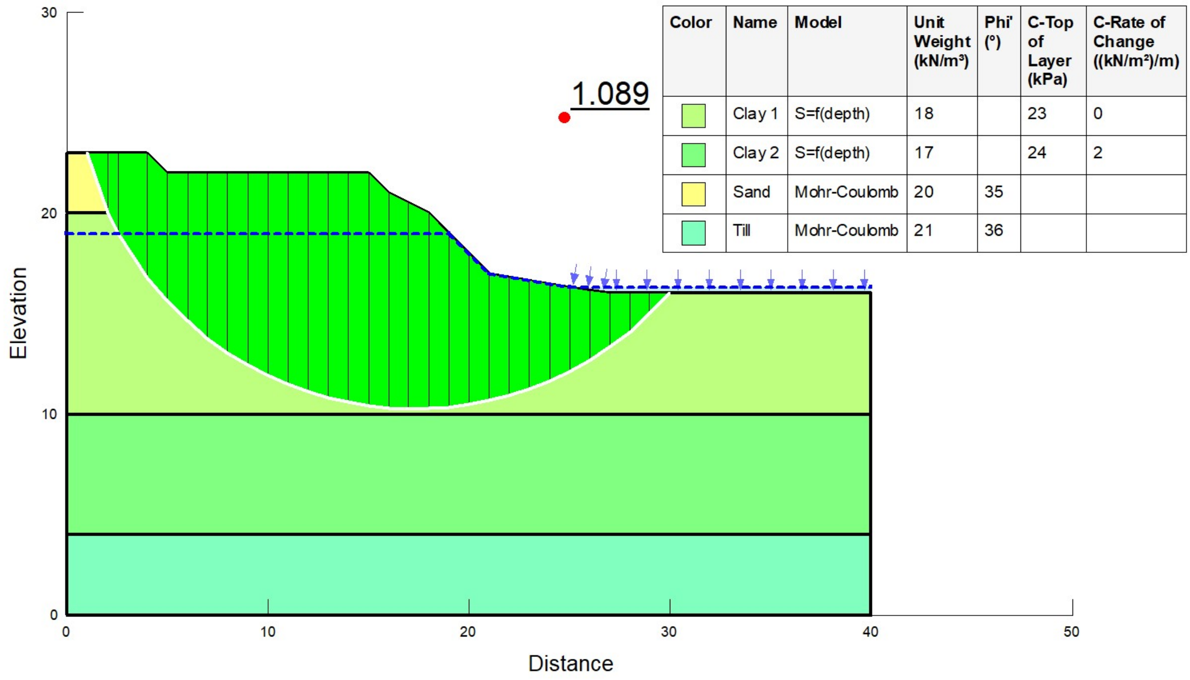


Figure E.6: Excavate slope slip surface section 840/875 undrained.

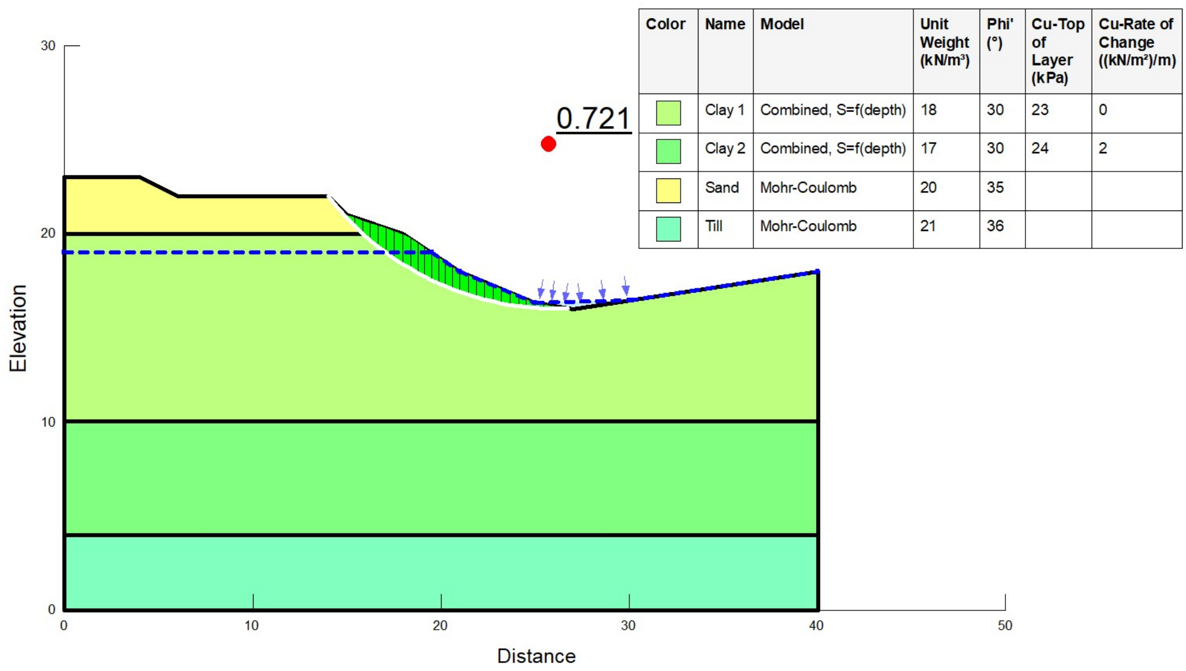


Figure E.7: Excavated slope slip surface section 875/886 combined.

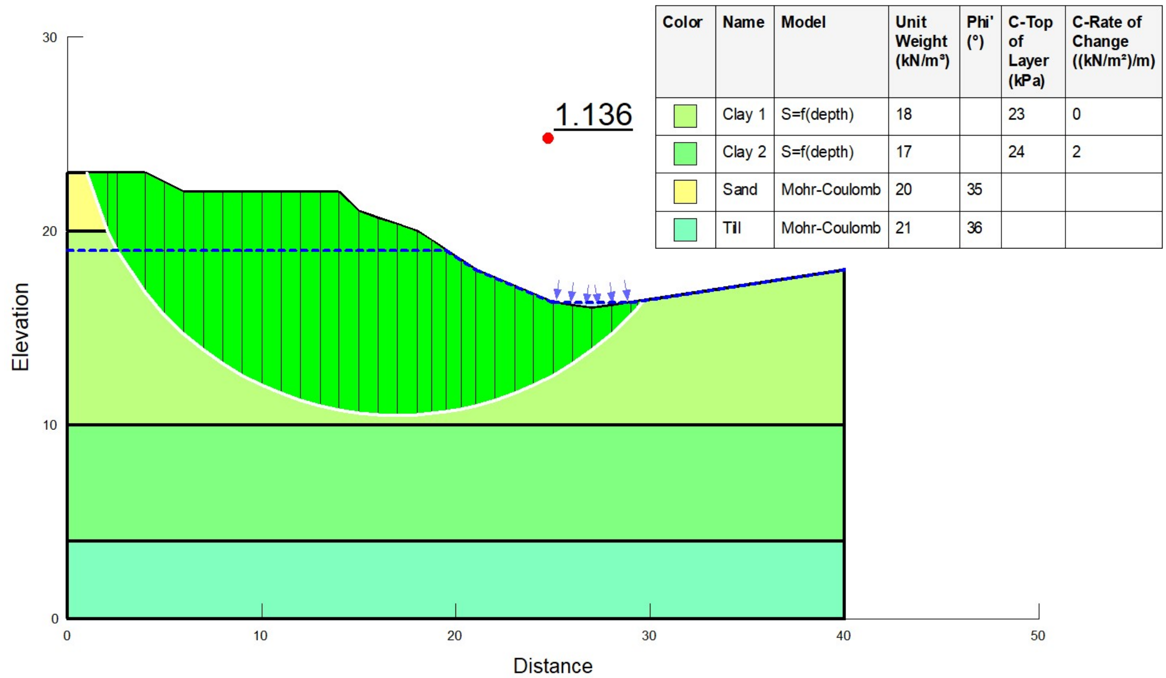


Figure E.8: Excavated slope slip surface section 875/886 undrained.

E.3 Excavated slope reinforced with piles

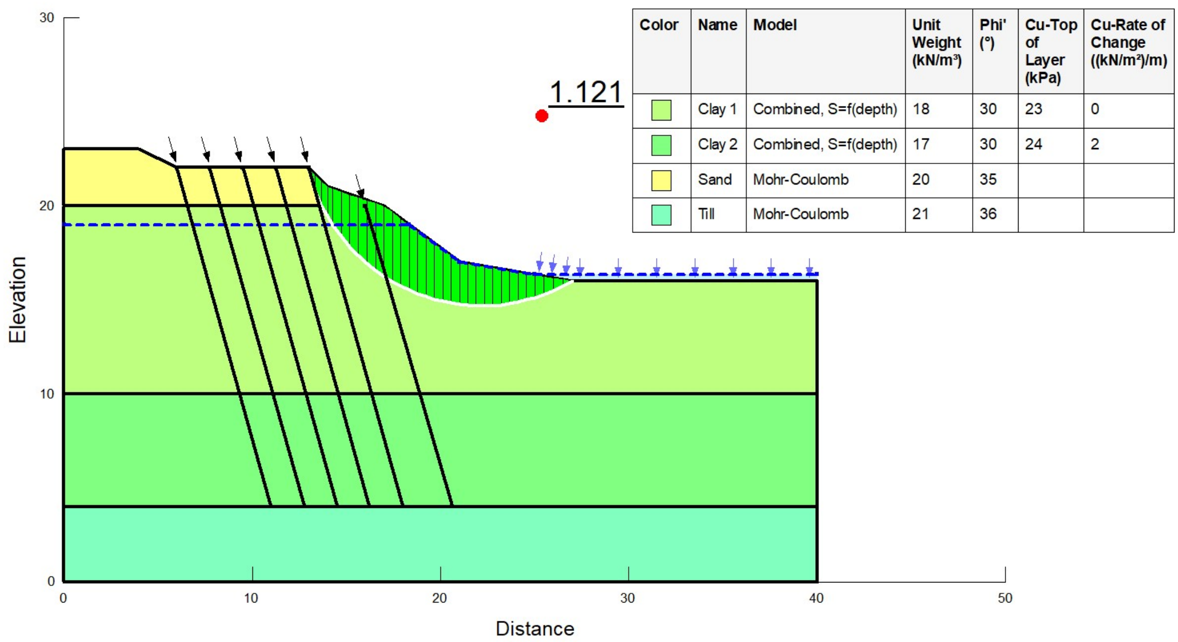


Figure E.9: Excavated slope reinforced with piles slip surface section 840/875 combined.

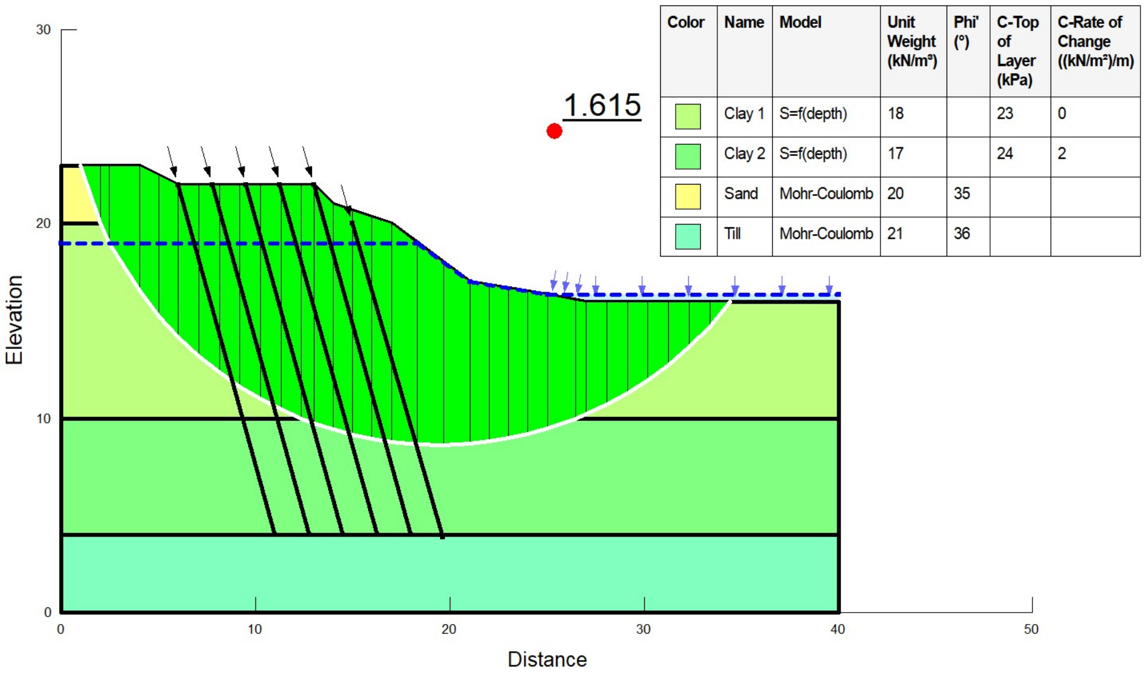


Figure E.10: Excavate slope reinforced with piles slip surface section 840/875 undrained.

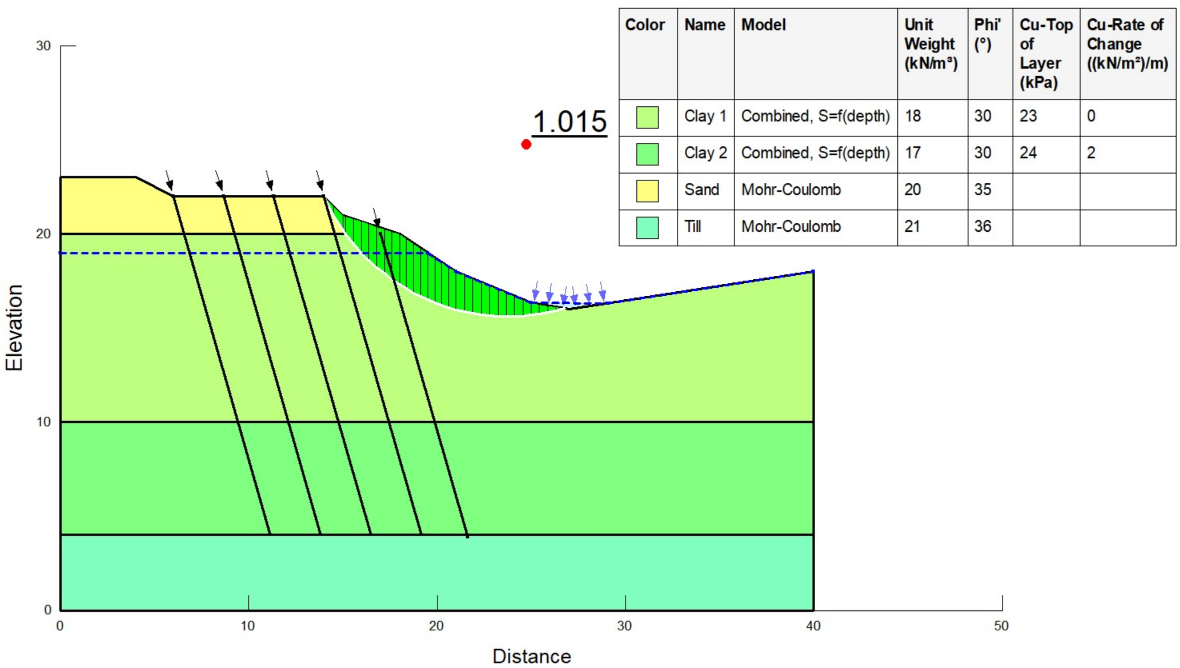


Figure E.11: Excavated slope reinforced with piles slip surface section 875/886 combined.

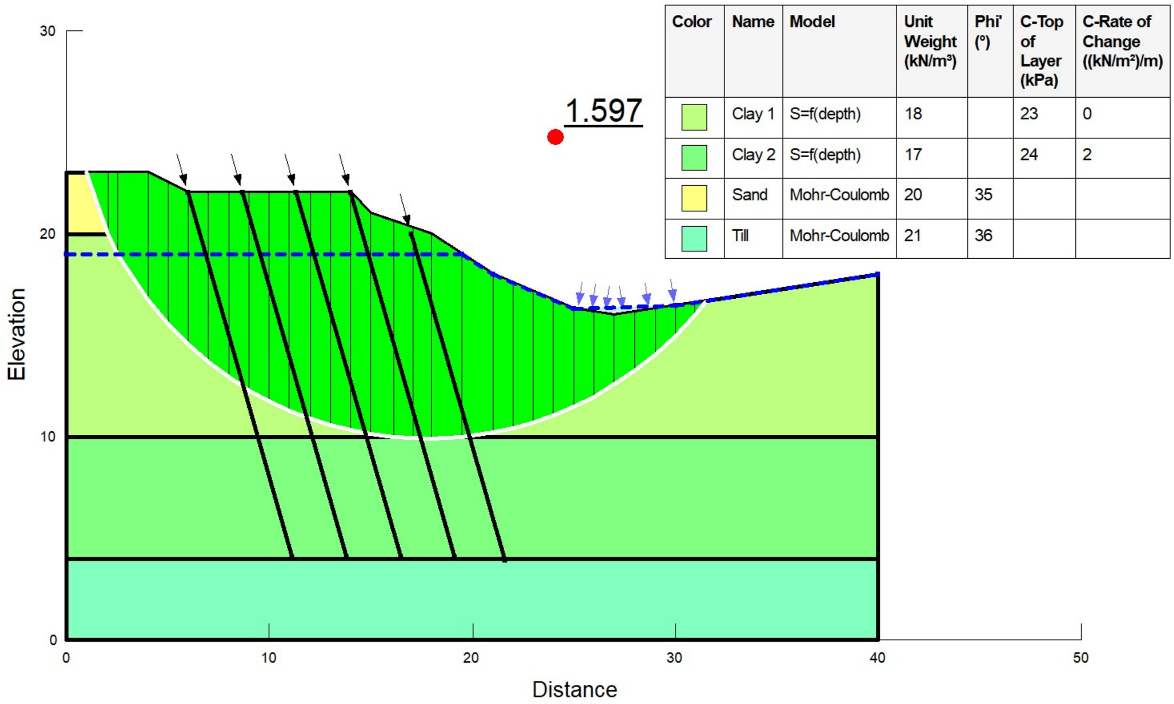


Figure E.12: Excavated slope reinforced with piles slip surface section 875/886 undrained.

E.4 Excavated slope with load

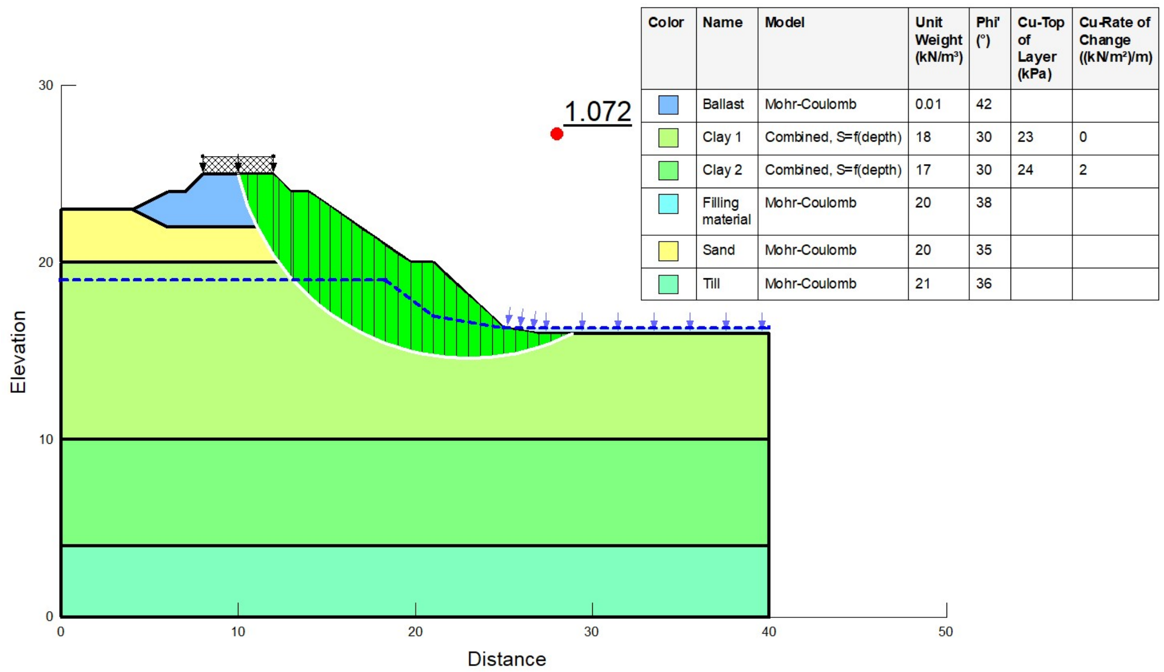


Figure E.13: Excavated slope with load slip surface section 840/875 combined.

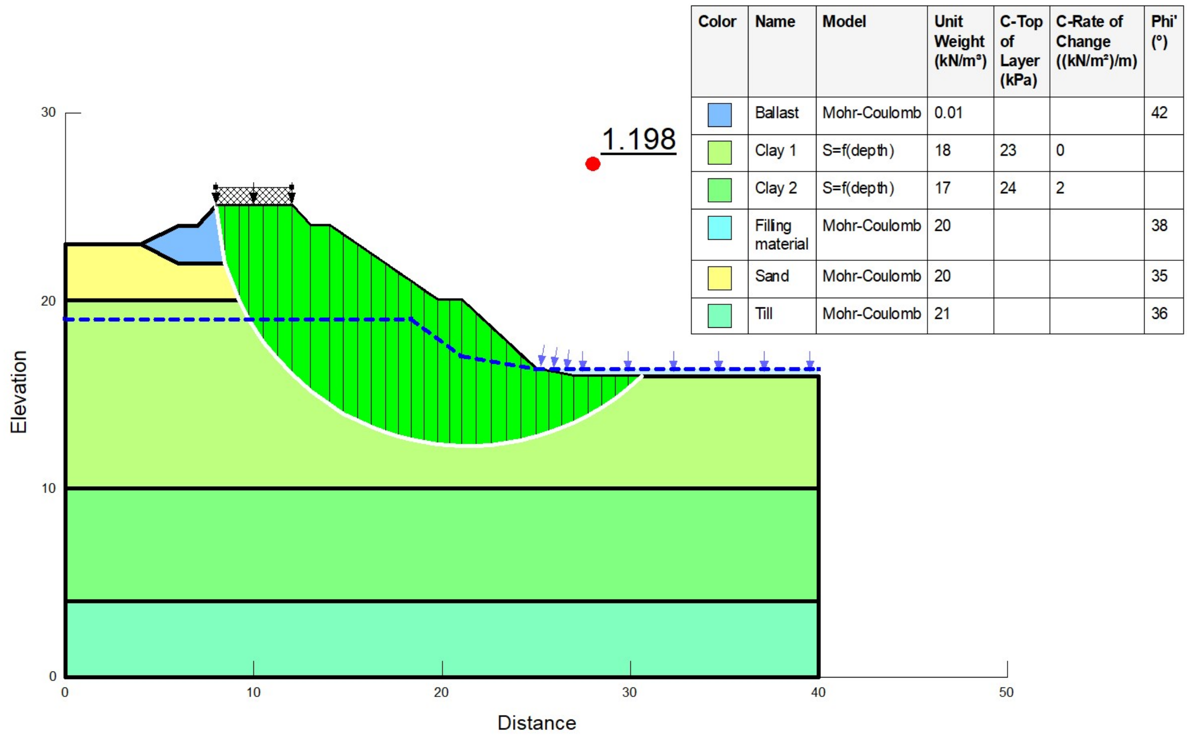


Figure E.14: Excavate slope with load slip surface section 840/875 undrained.

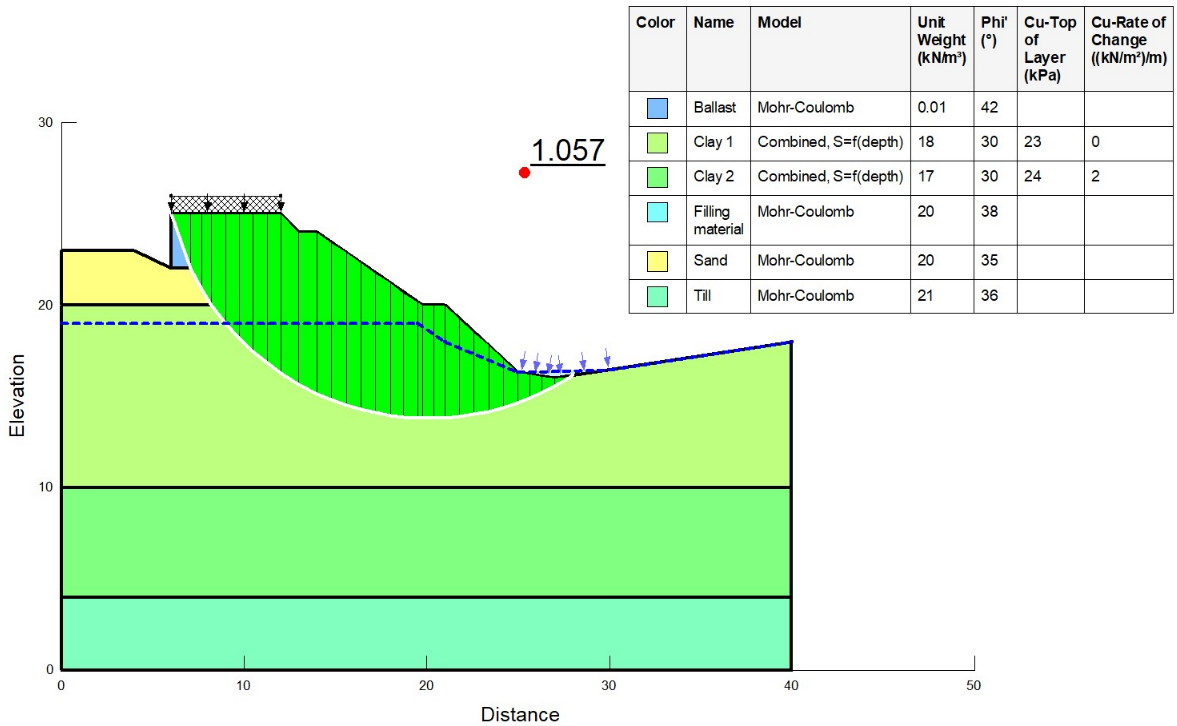
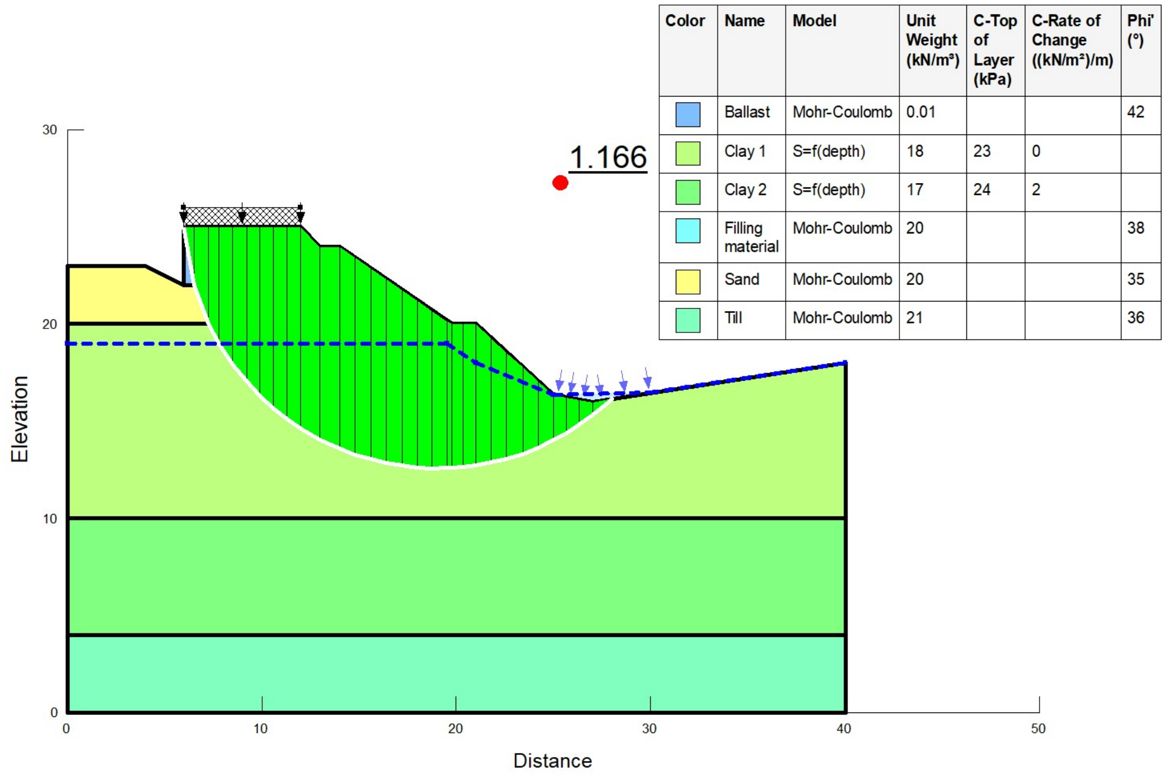


Figure E.15: Excavated slope with load slip surface section 875/886 combined.



Color	Name	Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	Phi' (°)
	Ballast	Mohr-Coulomb	0.01			42
	Clay 1	S=f(depth)	18	23	0	
	Clay 2	S=f(depth)	17	24	2	
	Filling material	Mohr-Coulomb	20			38
	Sand	Mohr-Coulomb	20			35
	Till	Mohr-Coulomb	21			36

Figure E.16: Excavated slope with load slip surface section 875/886 undrained.

F

Appendix: Rate of change

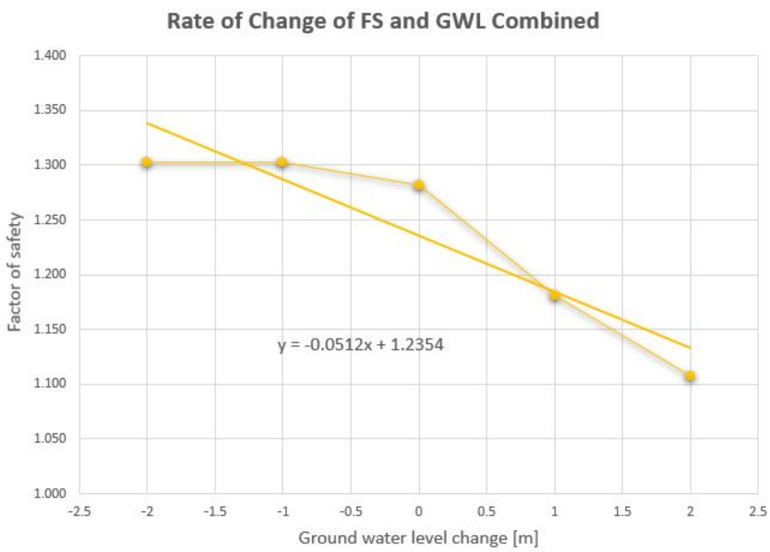


Figure F.1: Rate of change for the groundwater level and the factor of safety, combined.

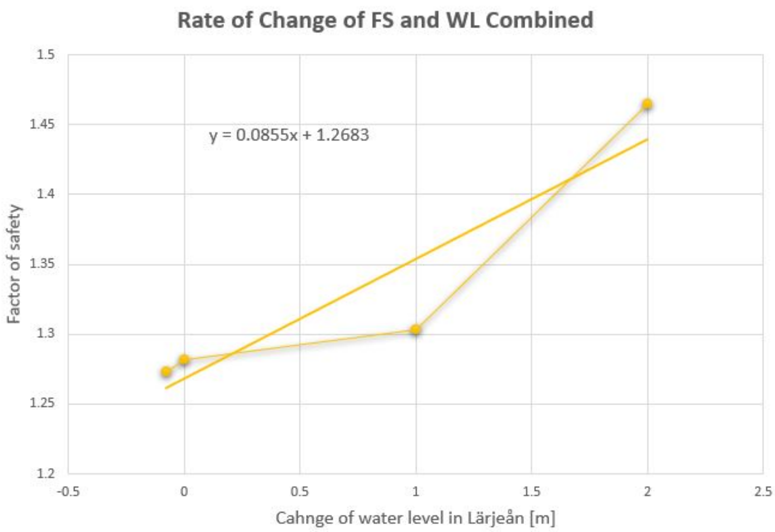


Figure F.2: Rate of change for the water level in Lärjeån and the factor of safety, combined.

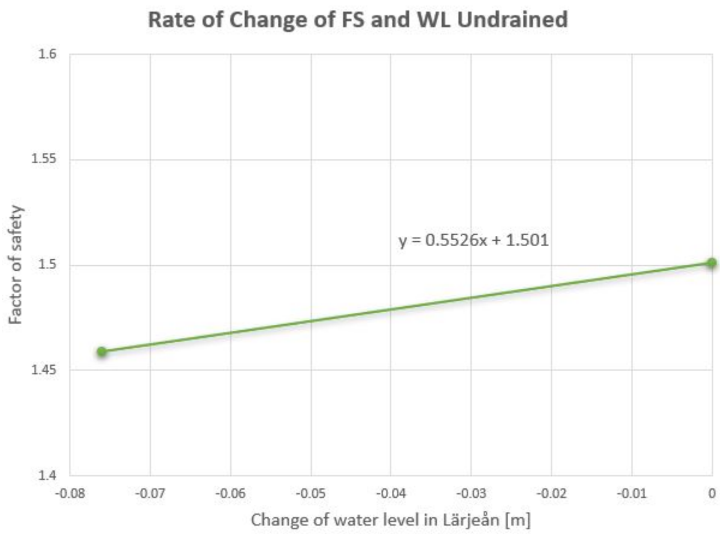


Figure F.3: Rate of change for the water level in Lärjeån and the factor of safety, undrined.

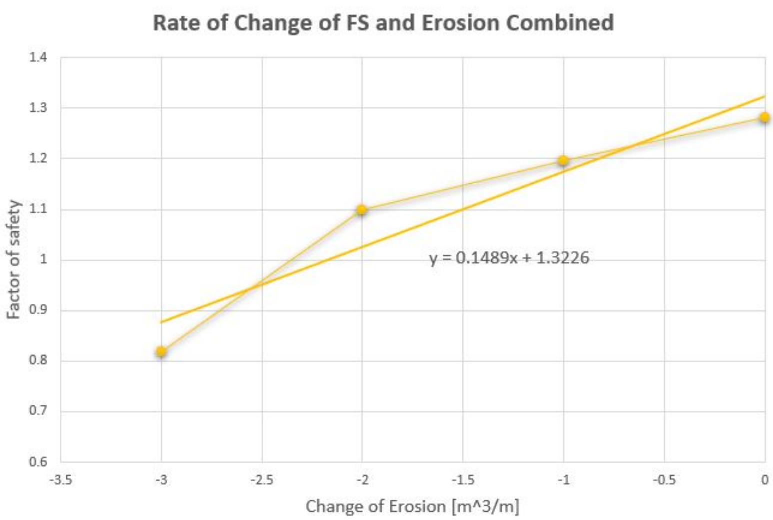


Figure F.4: Rate of change for the erosion and the factor of safety, combined.

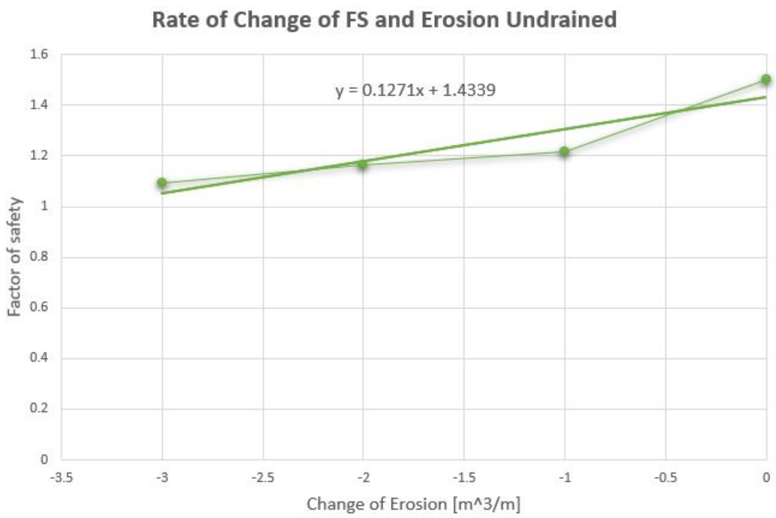


Figure F.5: Rate of change for the erosion and the factor of safety, undrined.

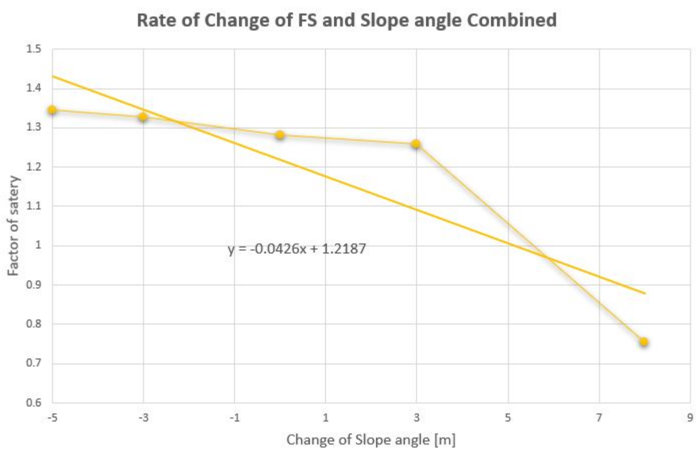


Figure F.6: Rate of change for the slope angle and the factor of safety, combined.

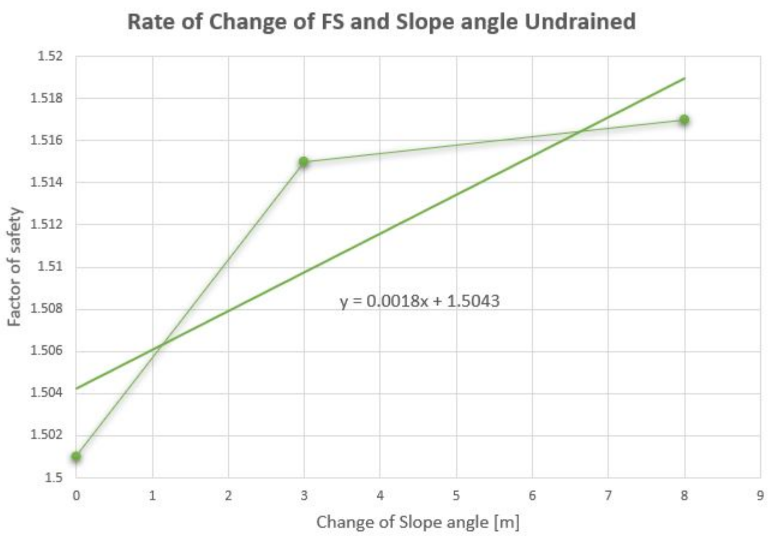


Figure F.7: Rate of change for the slope angle and the factor of safety, undrined.

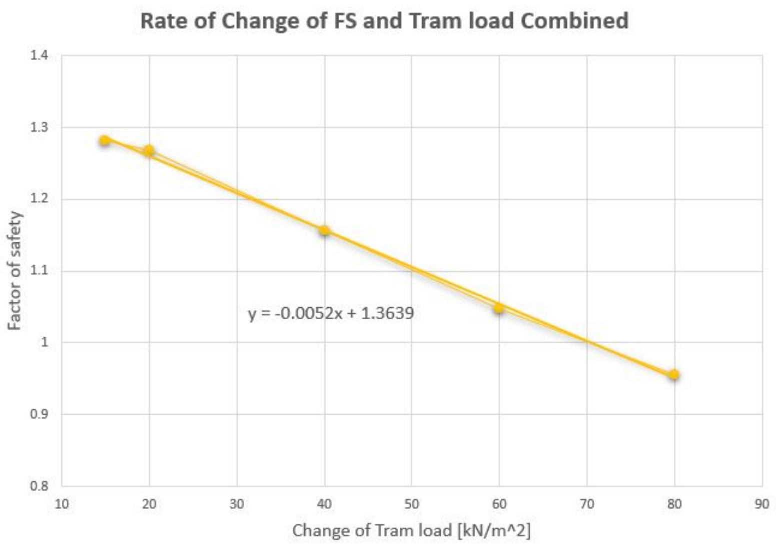


Figure F.8: Rate of change for the tram load and the factor of safety, combined.

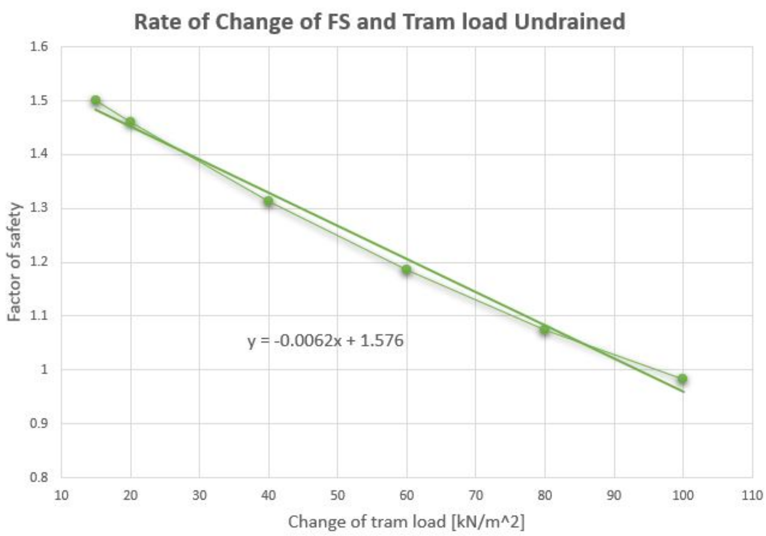


Figure F.9: Rate of change for the tram load and the factor of safety, undrined.

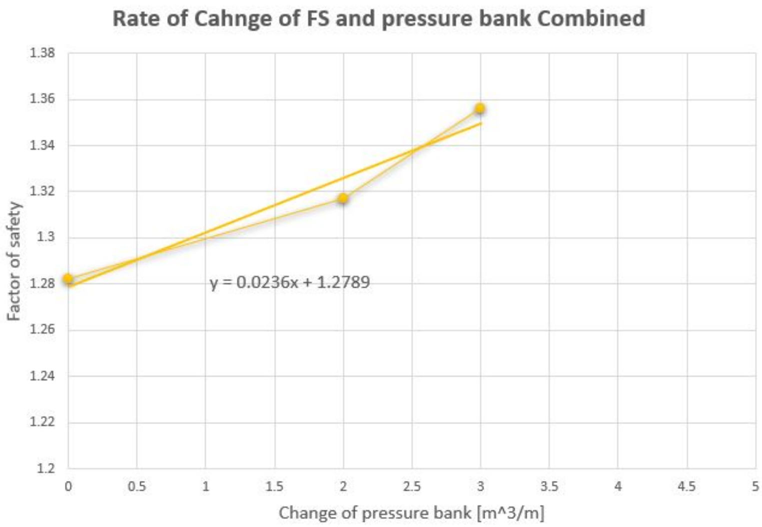


Figure F.10: Rate of change for the pressure bank and the factor of safety, combined.

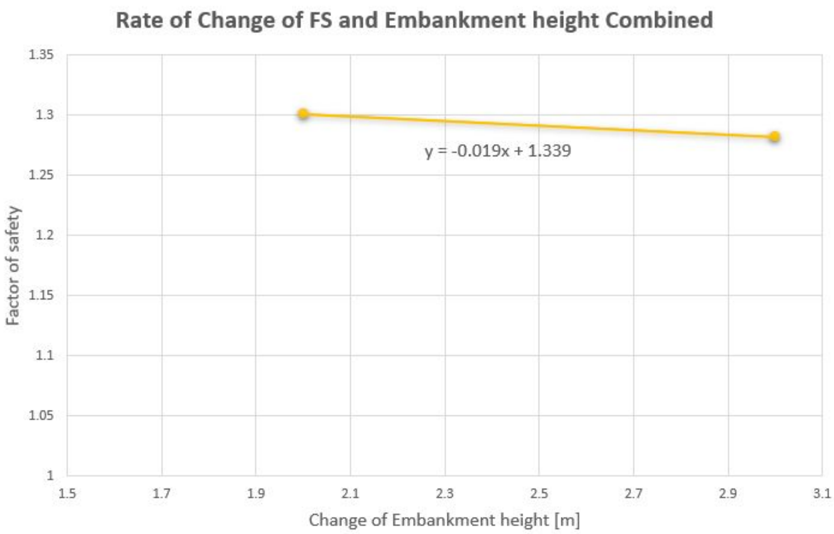


Figure F.11: Rate of change for the embankment height and the factor of safety, combined.

G

Appendix: Sensitivity analysis

G.1 Original conditions

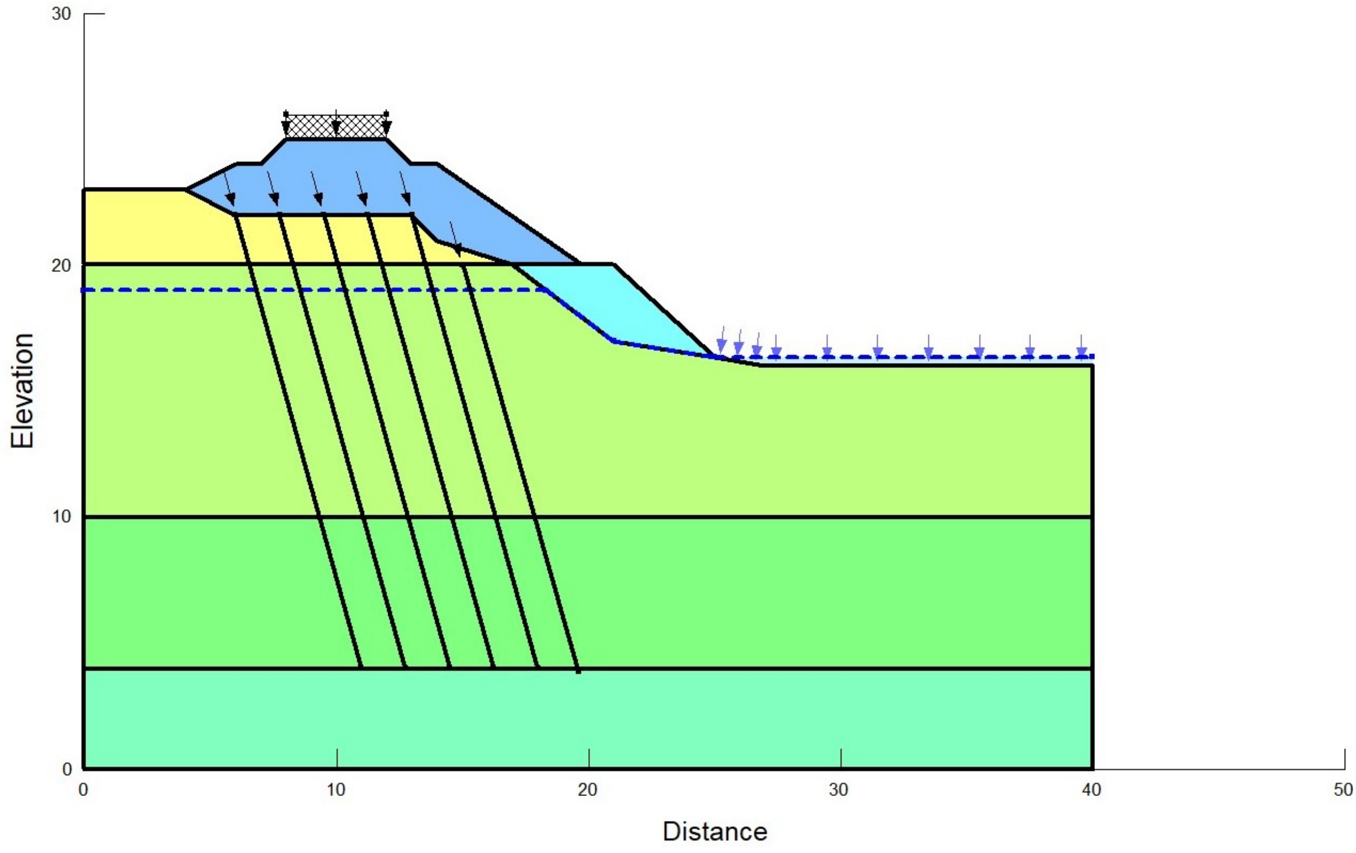


Figure G.1: Original conditions for the sensetivity analysis

G.2 Erosion

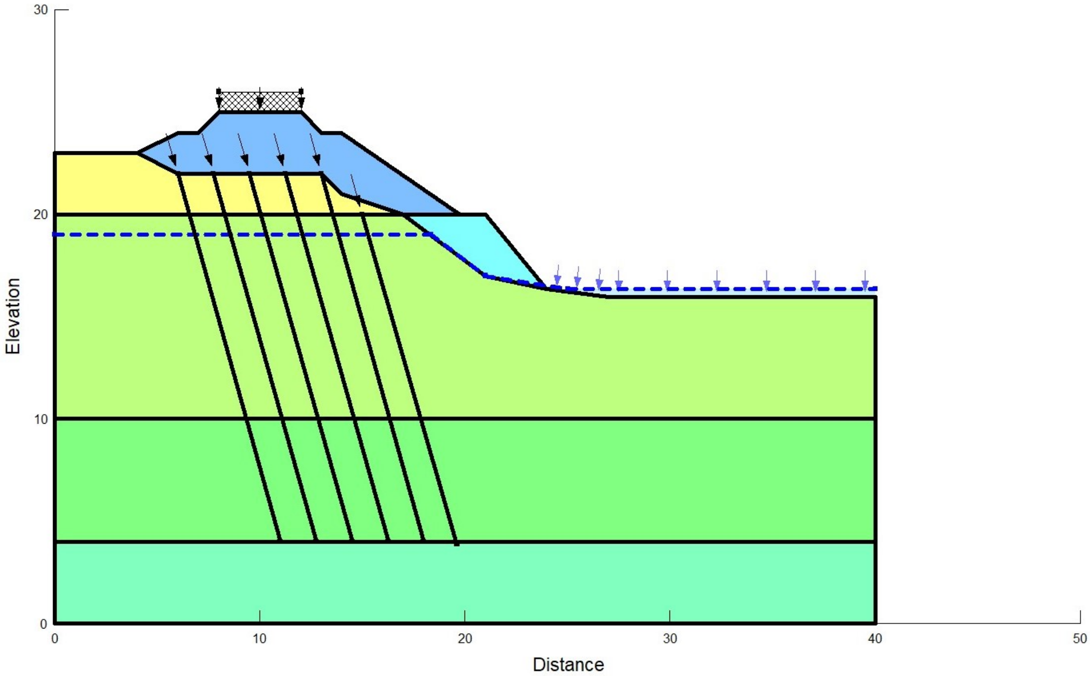


Figure G.2: Model after -1 m erosion.

sectionErosion

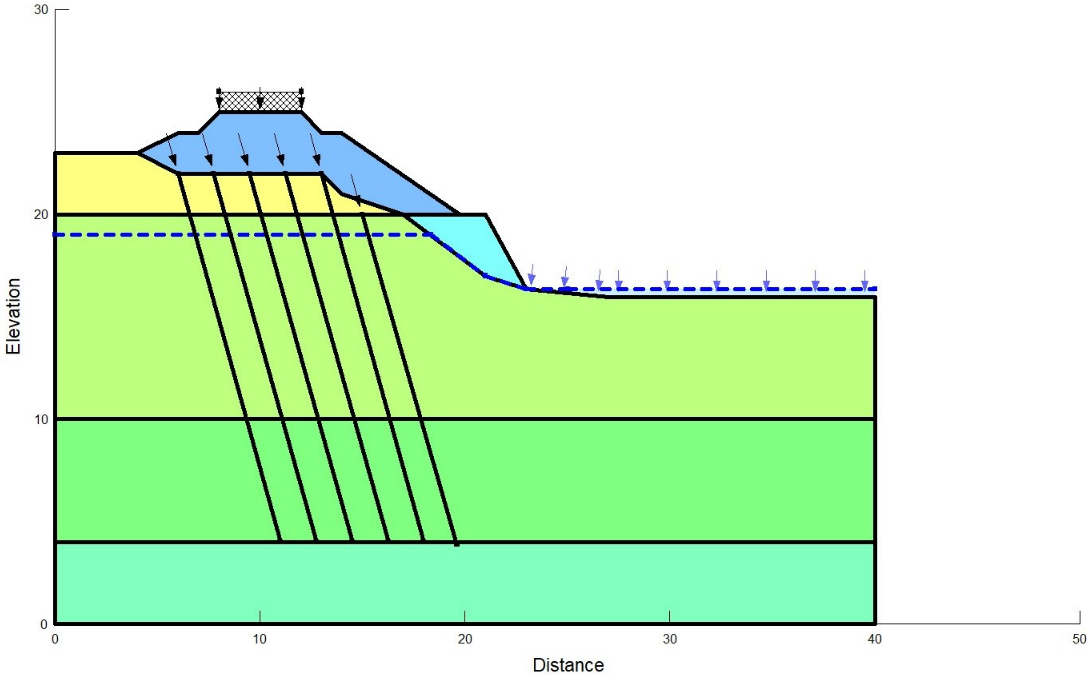


Figure G.3: Model after -2 m erosion.

sectionErosion

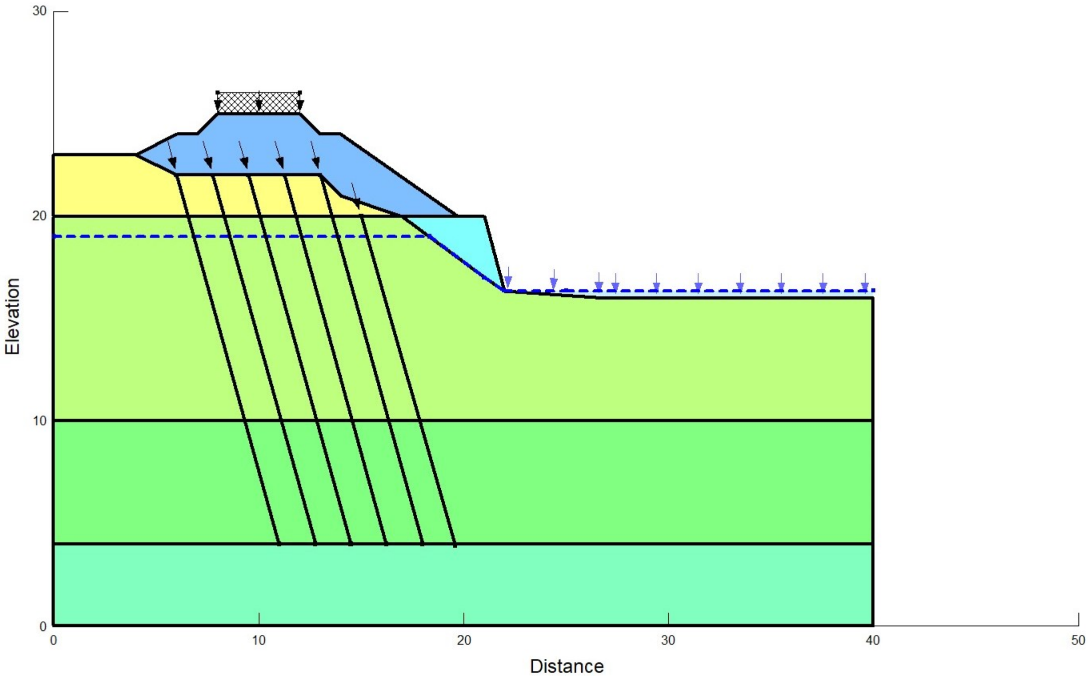


Figure G.4: Model after -3 m erosion.

G.3 Pressure bank

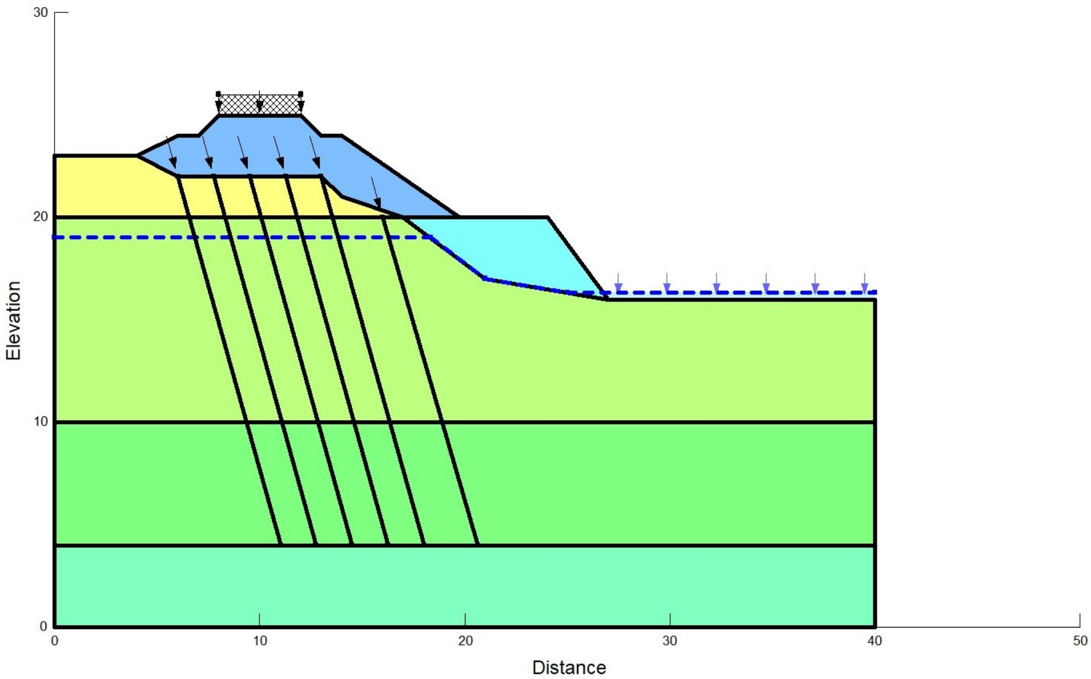


Figure G.5: Model after +2 m pressure bank.

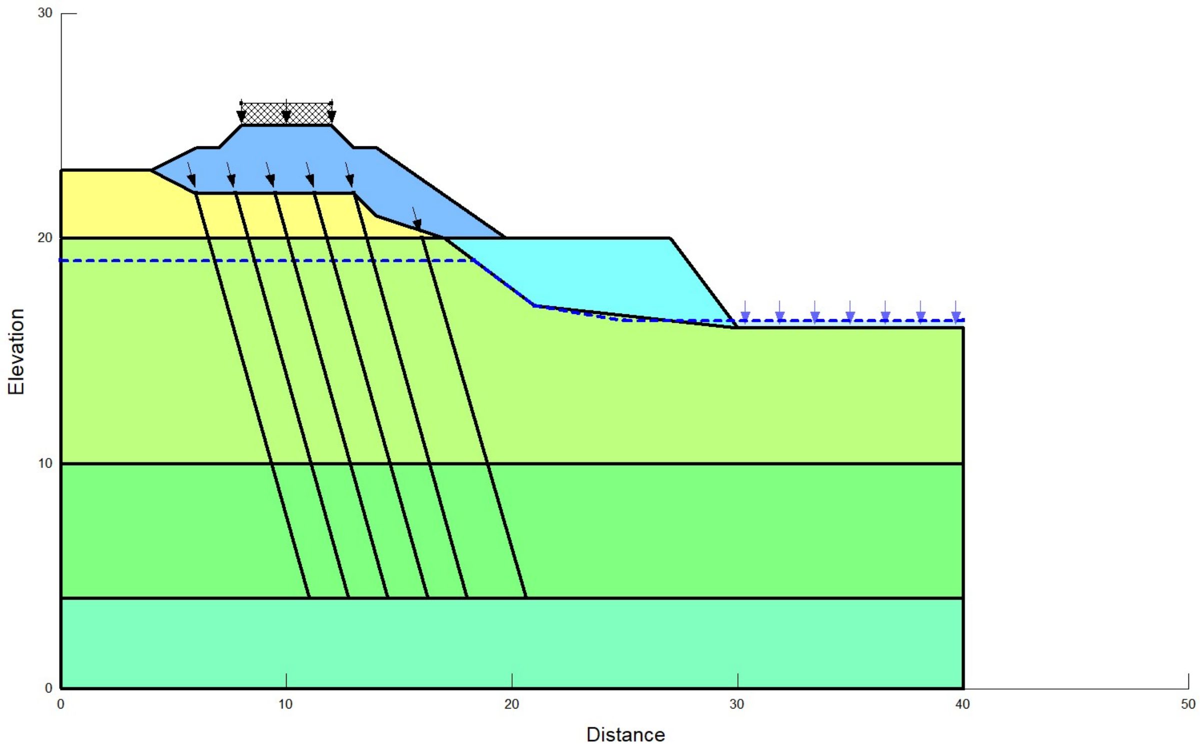


Figure G.6: Model after +3 m pressure bank.

H
Appendix: Analysis dimensioning strength

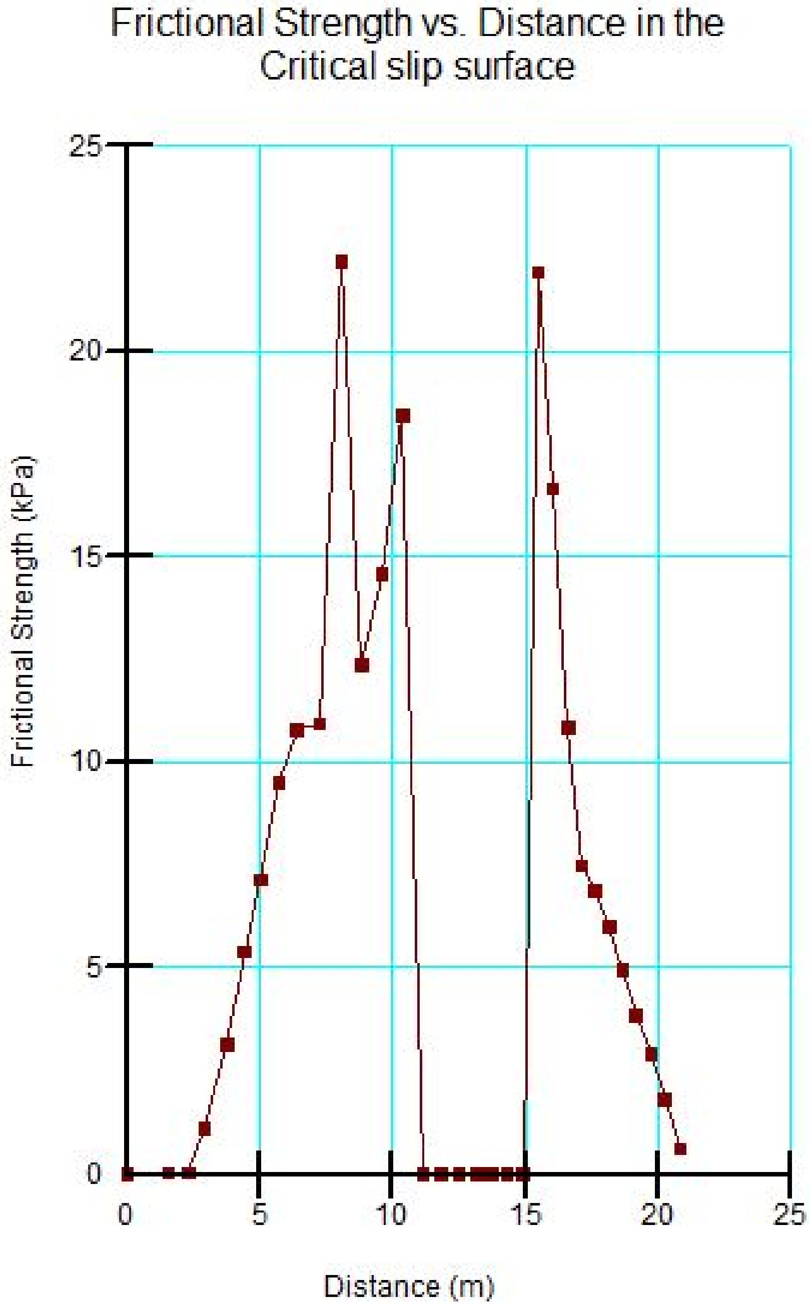


Figure H.1: Graph over frictional strength vs. distance in section 840/875 to determine the dimensioning strength in combined analysis.

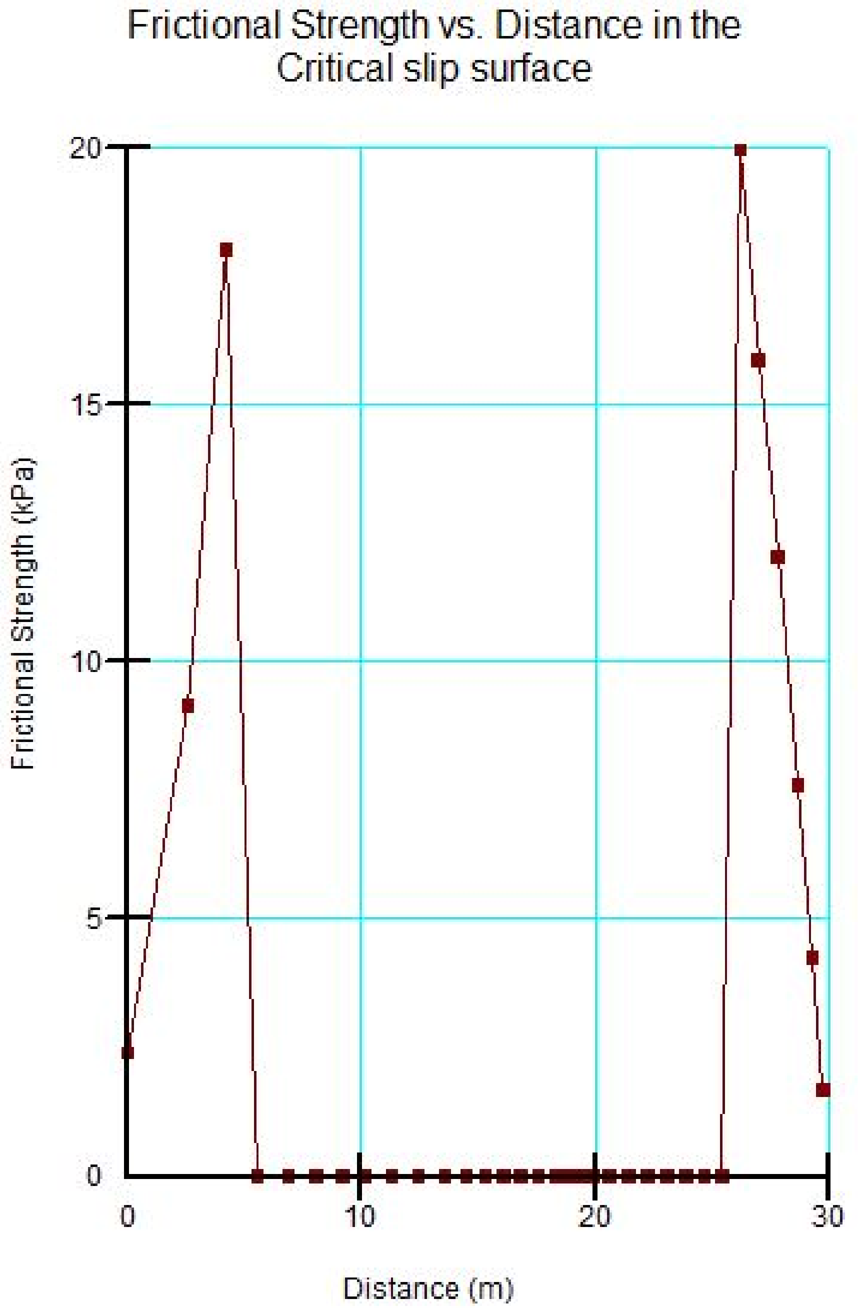


Figure H.2: Graph over frictional strength vs. distance in section 875/886 to determine the dimensioning strength in combined analysis.