Safety against quick clay landslides

An area stability assessment in accordance with NVE veileder 1/2019

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Preface

Denne masteroppgaven er utarbeidet ved institutt for Ingeniørvitenskap som en del av masterprogrammet for Byggdesign ved Universitetet i Agder. Oppgaven er skrevet i emnet Byg508 og utgjør det fjerde og siste semesteret. Hensikten med denne masteroppgaven har vært å gjennomføre en vurdering av områdestabilitet for et område bestående av kvikkleire.

Jeg ønsker å benytte anledningen til å takke mine to dyktige veiledere: Paul Ragnar Svennevig ved Universitetet i Agder og Emilie Laache fra Agder Fylkeskommune. Takker ydmykt for de erfaringer som er delt og den støtte og oppfølging dere har gitt meg.

Summary

In recent years, there has been a growing focus on landslide hazards and risks due to the gradual increase in landslide occurrences in Norway attributed to climate and climate change. This has led to heightened attention to the safety and stability of areas prone to sensitive and quick clay formations. In these clay-rich areas, landslides typically occur on steep slopes when the clay loses its strength over time and collapses under certain loading conditions. Currently, terrain modifications through construction activities are one of the main causes of such landslides, along with erosion in streams or rivers, although weather conditions often trigger the actual landslide event. The danger associated with this type of landslide, particularly quick clay landslides, is that they often initiate small but propagate over a larger area, potentially leading to catastrophic consequences for affected communities.

The focus of this master's thesis is to assess the safety against quick clay landslides. This research problem is of particular interest due to climate projections suggesting that extreme weather events may affect the stability of quick clay areas in the future. Therefore, there is an increased need to ensure the safety of such areas. By applying the GeoSuite software, stability analysis has been performed for the current situation in the area and after appropriate measures. The analysis has been supported by interpretation and assessment of data, simple calculations as well as literature review. By applying NVE guideline 1/2019 as the foundation for the thesis, the findings demonstrated a consistent safety factor across all analyses, satisfying the requirements.

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1 Introduction

The technical development of the country of Norway, with population growth and increased infrastructure, has made society more vulnerable to natural disasters than in the past [1, p. 9-10]. Since its inception at the beginning of the 20th century, this development has increased steadily. The amount of infrastructure buildings, transport, electricity supply, and other communication systems is very different today than it was about 100 years ago. Even though many vulnerable areas have been secured, and several landslide-prone regions have been evacuated, landslide events are expected to have more significant consequences today than in the past [1, p. 9]. Based on historical experience, several types of landslides may occur during the next 100 years.

Landslides are the natural hazard that causes the most damage and causes the most loss of human life in Norway, according to Stortingsmelding, *Meld. St. 15 (2011-2012)* [2, p. 14-16]. Landslides are typically triggered in areas with steep terrain, except mudslides in lowland areas below the marine boundary [1]. Terrain conditions are decisive for landslide hazards, while the weather is one of the most critical factors for triggering landslides [3, p. 76]. In rainy and cool seasons, landslides are more likely to occur, subsequently in warmer and drier periods, landslides are less likely. Recent research shows clear trends in terms of precipitation and temperature changes. The amount of precipitation in Norway has gradually increased over the past 100 years. Climate projections show that this trend is likely to continue, up to 20% by 2100. In addition, more frequent periods of extreme weather are expected [4].

Norway's landscape was formed shortly after the last ice age. In Western Norway and in Northern Norway, and in some high mountain areas, valleys, and fjords were eroded. The hillsides became "too steep," and the valleys became deep. Furthermore, large parts of the sides of the hills became unstable where the ice had melted away. In addition, moraine masses were left behind in the steepest terrains. Based on this development, different types of landslides began to occur. The sea deposited significant amounts of clay at the end of the last Ice Age. This took place in the lowlands of Eastern Norway and Trøndelag. Large mudslides in the clay deposits thus characterized these areas as these emerged during the land subsidence after the Ice Age. During this period, there was little cultivated forest and other vegetation to stabilize and support the soil masses. Therefore, it can be assumed that the landslide events in this period were highly demanding to handle. A decisive factor for more frequent landslide triggering shortly after the last ice age was the rapid land uplift that led to earthquakes, and as a result, steep hillsides were caused by ice erosion [1, p. 5-6]. Today, landslides occur regularly in Norway, although the frequency varies.

All factors combined make it evident that constriction and infrastructure are vulnerable for climate change. As such, it will become increasingly important in the years to come to assess and evaluate ground conditions to ensure human safety. This is the focus of this thesis.

[1] Marine boundary - highest sea level since the last ice age

2 Societal Impacts

In recent years, Norway has experienced several major landslides, with the quick clay landslide in Ask municipality in Gjerdrum in 2020 being the most recent. In total, the landslide resulted in the loss of 11 lives, including an unborn child, the evacuation of over 1600 people, and significant material destruction. Table 2.1 provides an overview of the costs incurred by the landslide, excluding the loss of life. This highlights the extensive and severe consequences that a landslide can entail [5].

Table 2.1: Costs related to the quick clay landslide in Gjerdrum [5]

Kostnadselement	Anslag, mill. kroner	Kilde
Skader på blant annet bygninger, innbo, kostnader ved rivning og opprydding	875	Norsk Naturskadepool
Andre materielle skader (blant annet kjøretøy)	–	–
Redningskostnader	–	–
Krisehåndtering, evakuering av innbyggere	25	Gjerdrum kommune
Skade på lokaler for kommunale tjenester (midlertidige lokaler og nybygg)	123	Gjerdrum kommune
Skade på kommunal infrastruktur – vann, avløp, veger (midlertidige løsninger og nyanlegg)	57	Gjerdrum kommune
Opprydding og sikringstiltak i skredgropa	78	Gjerdrum kommune
Personlige kostnader og ulemper ved evakuering	–	–
Merkostnader kollektivtrafikk, skader fylkesveg, kostnader Viken fylkeskommune	35	Viken fylkeskommune
Sikringsarbeider, NVE	200	NVE
Tap av liv (følger VSL, se avsnitt 4.3.1)	322	–
Personskader	–	–
Psykiske ettervirkninger	–	–
Omkjøringskostnader	90-120	Statens vegvesen
Skader på annen infrastruktur	–	–
Skader på natur, miljø, biologisk mangfold og friluftsliv	–	–
Negative ringvirkninger for næringsliv og offentlige tjenester	–	–

Evaluations and investigations conducted subsequently revealed that the landslide originated from a slope that had been subjected to erosion caused by a stream. The landslide developed backward and sideways and from Holmen and further to the Nystulia residential area as illustrated in Figure 2.1. Further assessments indicated that the slope had been highly susceptible for an extended period, and the erosion had further weakened its stability. It was determined that the erosion was primarily induced by human interventions, which had impeded the flow of water in several areas of the stream, resulting in increased water velocity and subsequent erosion [5].

Further evaluations highlights that a decisive factor that potentially contributed to this landslide was the abnormal mild autumn season observed in 2020. An extended duration of intense rainfall led to elevated pore pressure within the clay deposits at the bottom of the slope. Moreover, the high-water pressure in the stream immediately prior to the occurrence of the landslide likely worsened the overall stability conditions [5].

Figure 2.1: The area of Ask municipality in Gjerdrum after the quick clay landslide [5]

Some studies highlight that quick clay landslides are mainly triggered by either human interventions or natural causes [6, 7]. Natural causes are typically elevated pore pressure or erosion along watercourses. Human interventions include excavations at the bottom of slopes, fillings at the top, and other terrain loads that can worsen stability. According to Heureux et al [8], 90 % of the landslides from 2010 to 2018 have been triggered by human interventions as illustrated in Figure 2.2.

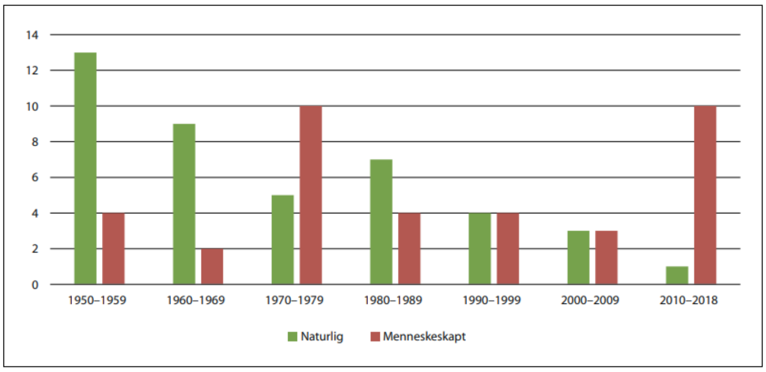


Figure 2.2: Distribution of landslide based on natural causes and human interventions [5]

Since 1900, 62 people have died due to quick clay landslides [5]. Based on location of damage Figure 2.3 illustrates that 37 of these deaths are related to construction. This reflects a need for increased understanding of the performance and the importance of safety measures in areas exposed to quick clay.

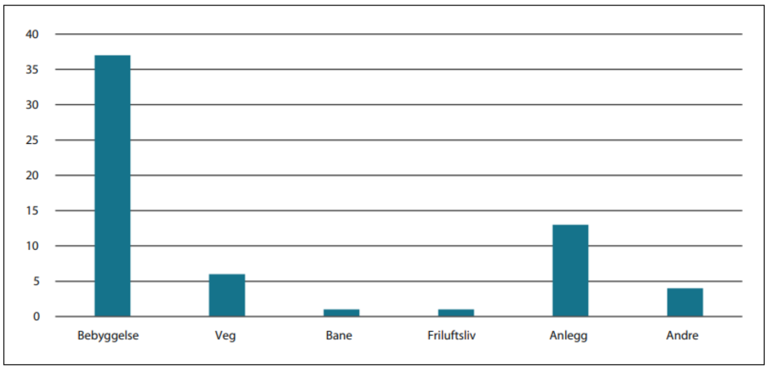


Figure 2.3: The number of deaths in quick clay landslide from 1900-2020 [5]

Recently, researchers, in collaboration with public agencies and private businesses, have completed the innovation project *Klima2050*, which started in 2015. The project aimed to promote reasonable solutions for dealing with increased rainfall. More than 50 innovations have been carried out through the project. In addition to 16 pilot projects and zero-emission buildings. The project has contributed to transferring and developing new knowledge to young, newly graduated masters and doctoral students. In addition to scientific communication, the project has ensured that the inside is made available and disseminated to the industry and the public. Hence, the project has the potential of to facilitate the achievement of UN`s sustainability goal number 17 [9]: Collaboration to achieve the goals. To be prepared for landslides, advanced measuring equipment has been installed to improve the methods for landslide warnings triggered by rainfall. [10].

The process of sharing and developing knowledge among newly graduated students through participation in innovative projects and the development of zero-emission buildings, facilitating industry access to information and the public as well as the development of equipment and methods for early warning of landslide hazard can help to increase the safety according to future landslides. Therefore, the project may be seen in context with UN`s sustainability goal number 13 [11]: stop climate change, including sub-goals 13.1 and 13.2. Subgoal 13.1 aims to strengthen the ability to resist and adapt to climate-related dangers and natural disasters. Subgoal 13.2 promotes incorporating climate change into policy, strategies, and planning at the national level [11].

In the construction industry, CO_2 emissions are a well-known challenge [12, 13]. From 1990 to 2016 these emissions increased with 58% [13]. Data from Statistisk Sentralbyrå (SSB) shows that this industry in 2021 corresponded to more than two tons of CO_2 equivalents [14]. In accordance with the Paris Agreement [15], Norway, along with most other countries worldwide, has committed to reducing CO2 emissions by at least 50% to 55% by 2030. This is a crucial goal to pursue if Norway is to become a low-emission society by 2050. The actions taken in the years ahead will be crucial in achieving this target. This forms the foundation for achieving UN`s sustainability goal number 13 about stopping climate change [11].

Lime-cement piles are the most used method for securing quick clay areas [16, 17]. Today this method typically requires tons of binder to make the ground stable. This process improves the ground conditions and increases the safety, but at the same time, contributes to significant CO_2 emissions. A typical mixing ratio is 150 kg lime-cement per m^3 clay. When producing one ton of cement, roughly the same amount of CO_2 is emitted [18].

That is why the Norwegian Public Roads Administration (NPRA) in 2020, in collaboration with Bane Nor and Statsbygg, established the innovation project *KlimaGrunn*. This project intends to develop new technology that will contribute to fewer emissions, saving money and improving the quality of the lime-cement piles.To achieve this, results from laboratory tests and actual conditions in the field must match better so that it is possible to perform design more securely while reducing the amount of binder. The project uses 45kg/m^3 of binder in the piles instead of the 100 kg/m^3 as recommended in the current guidelines, and two-thirds reduction in the cement content [16, 17, 19]. The reduction in the cement will contribute to lower CO_2 emissions and can therefore be seen in the context of the UN`s sustainability sub-goals 13.1 and 13.2 [11].

Usually, piles cast in areas consisting of quick clay achieve higher strength than tests conducted in the laboratory. Because results from the laboratory tests have priority, there is a risk that an unnecessary amount of binder is applied to achieve sufficient strength. Multiconsult, an additional collaborator in the *KlimaGrunn* project, has developed new methods for compacting binder-stabilized tests. This enables the attainment of homogeneous tests with the appropriate density. Moreover, the tests are hardened in molds to avoid volumetric changes during the hardening phase. There is an optimistic outlook that this approach may evolve into a new standard method, subsequently giving rise to the development of a comprehensive handbook.

At the same time, it is highlighted that laboratories today use different methods which are neither repeatable nor provide a basis for comparison. This is because the hardening process in the laboratory tests is typically performed at 4-5 degrees, which corresponds well with the soil's thermal conditions. In contrast, neither the development of heat during the hardening process nor the soil's moisture is considered. Therefore, NPRA conduct this hardening at 20 °C and humidity equal to 100 % road construction [17].

Therefore, NPRA, in collaboration with contractors, advisers, universities, and representatives from Sweden, has initiated the pilot project *E18 West Corridor*, which resulted in becoming an Innovation Norway project. The project intends to test the method with a reduced amount of lime cement on a stretch along the *E18 West Corridor* to reduce emissions related to road construction. With current practices it has not been possible to document the quality well enough. Therefore, the project seeks to implement seismic and sensor-based monitoring techniques to assess and observe the stabilization quality throughout the entire depth over a period of time [17].

In April 2023 the *Klimagrunn* project successfully achieved its target which involves developing an innovative technology that enables the reduction of CO_2 emissions and costs in accordance with ground reinforcement as well as increased control during the construction phase. The methodology involves the installation of temperature sensors and seismic monitoring devices within the piles, facilitating accurate monitoring of strength development over time. Continuous temperature monitoring within the piles allows for observation of the hardening phase, and the collected measurements are applied to anticipate the advancement of the strength. Field measurements are subsequently compared with laboratory measurements to confirm the achievement of preferred stability in the quick clay [12].

By applying this technology for areas consisting of quick clay, the CO_2 emissions will potentially be reduced by 50%. This an important step towards reaching the commitment in the Paris Agreement of reducing the emissions with 55% by 2030, and thus the project is a great contributor to meet UN`s sustainability goal number 13 [11]: Stop climate change.

The method developed through *Klimagrunn* allows for the optimization of binders during the design and construction phases. As a result, CO2 emissions and costs are minimized, while the development of strength can be continuously observed and controlled throughout the construction period. Furthermore, the method enables the determination of achieved strength and stiffness over the relevant time period for the intended purpose [12].

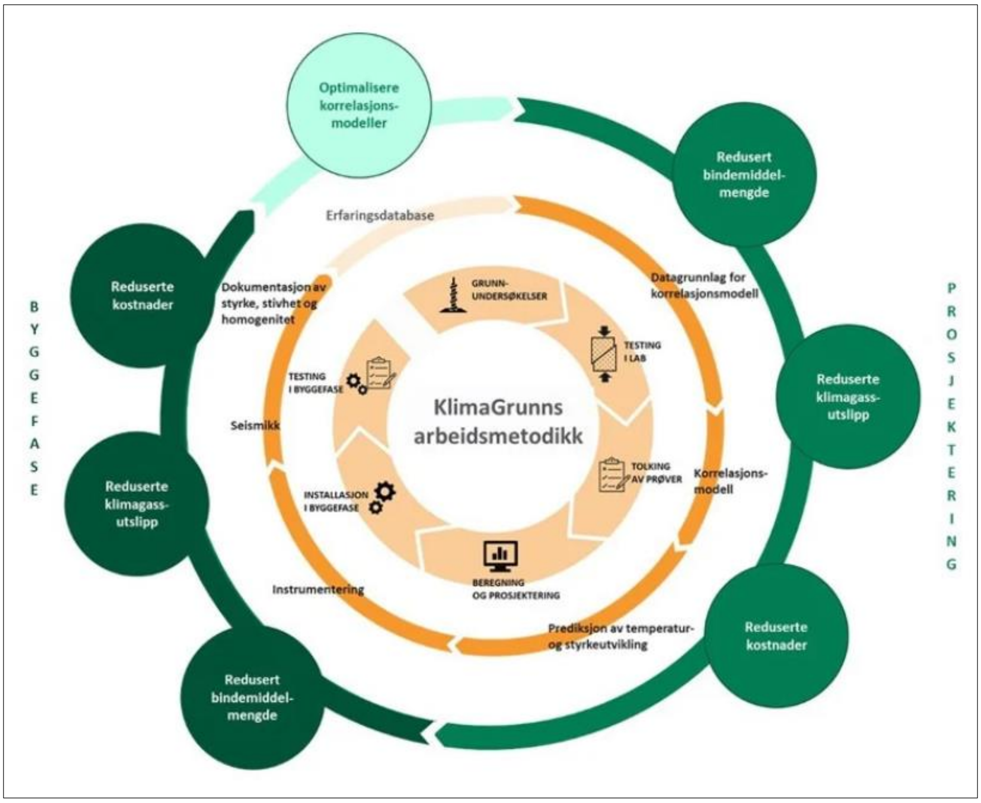


Figure 2.4: The methodology for ground stabilization developed through Klimagrunn [12]

Cost savings can have implications for future development projects. It is now feasible to apply areas designated for construction purposes in quick clay vulnerable regions that were previously left untouched due to the high cost of ground stabilization in these areas. With the increased development of roads, railways, and buildings, ground stabilization will now be considered in quick clay vulnerable urban areas [12]. This new approach may be seen in context with UN`s sustainability goal number 11 [20]: Sustainable cities and communities, including sub-goal 11.5: By 2030, achieve a significant reduction in the number of deaths and the number of people affected by disasters, including water-related disasters, and substantially decrease the direct economic losses in global gross domestic product (GDP) caused by such disasters, with a focus on protecting the poor and those in vulnerable situations.

3 Theoretical background

This chapter presents relevant and essential theories to clarify the problem area. Additionally, standards, handbooks, guidelines, and regulations applied in this thesis will be presented.

3.1 Area landslide

Area landslide is a common term for landslide caused by quick clay and other soil types with brittle fracture properties. A landslide of this type is usually caused by a small event, such as a small slip along a stream or fill along the top of a slope that can lead to a landslide that spreads over a larger area as demonstrated in Figure 3.1. When the masses resulting from a landslide loosen and disperse unrestrictedly, they can pose considerable challenges to manage. Depending on the location of the initial overload, the landslide can propagate in different directions: laterally and backwards (referred to as retrogressive or progressive) or forwards (progressive) [6].

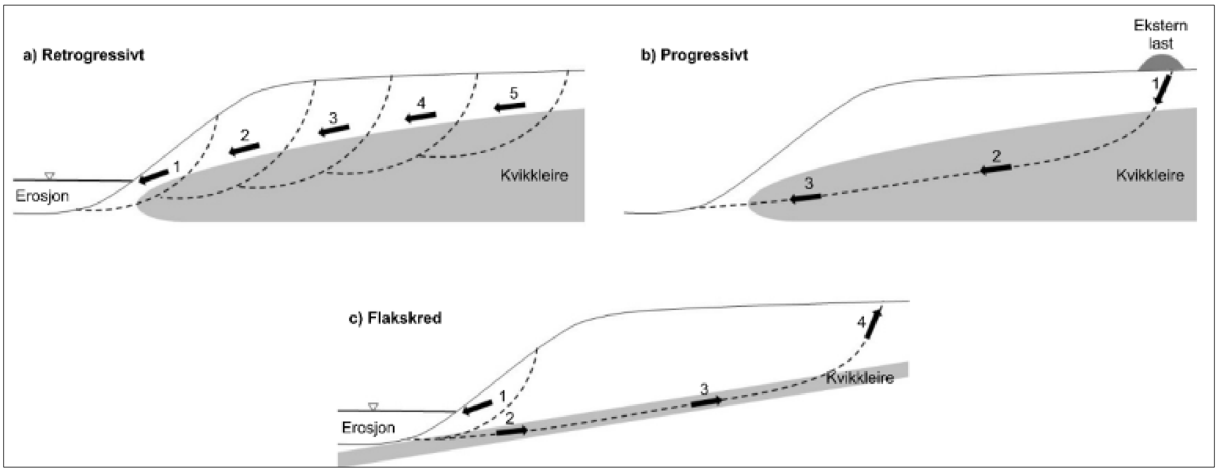


Figure 3.1: Landslide development - backwards (a & c) and forward (b) [21]

Documentation of landslide safety is normally expressed in terms verifying sufficient area stability. This process involves identifying potential hazard zones (areas of caution), which often necessitates further investigations and stability calculations to assess the actual risk. Stability calculations quantify a slope's capacity to resist failure and withstand planned or unforeseen changes in the landscape. When a slope is subjected to overloading, it may experience fracturing, whereby the soil undergoes pressures beyond its normal levels, resulting in a reduction of its shear strength. Factors contributing to overloading can include excessive excavation or deep digging, the addition of external masses, or excessive filling. Erosion and high pore pressure can also contribute to overburden. In clay terrains, localized overloads typically serve as the primary triggers for slope failures [6].

3.2 Quick clay

About 5,000 square kilometres of Norway is covered by soft marine clay deposits. Nearly 20 % of this area consists of highly sensitive or quick clay [22]. Quick clay is normally defined as clay which, when overloaded, loses its solid structure, becomes a thin flowing liquid and may trigger landslides [5]. Therefore, professor at NTNU Lars Grande has stated the following:

"The most important thing you can do to avoid quick clay landslides is, first of all, to find out where the quick clay is located and then to inform everyone concerned about how to prevent triggering landslides [6]*."*

In geotechnical context, quick clay is normally defined as stirred clay with a shear strength less than or equal to 0,5 kPa [5]. In recent years, a term has been established for the practical evaluation of quick clay landslides, referring to lose masses of silt or clay as brittle fracture material. According to NVE veileder 1/2019 [6], soil containing brittle fracture materials is characterized by a stirred shear strength less than 2 kPa. Low amount of salt and high sensitivity are indications of quick clay.

Quick clay is typically formed in pits or in layers in the ground, and on slopes down towards rivers or streams [5]. It exists only in areas below the marine boundary and landslide is normally triggered by human interventions in terrain or as a result of erosion in rivers and streams [3, p. 76].

3.3 How to achieve sufficient safety against area landslide

illustrates the hierarchy and the methodology. In this chapter the hierarchy will be explained while the methodology will be explained in the methodology chapter.

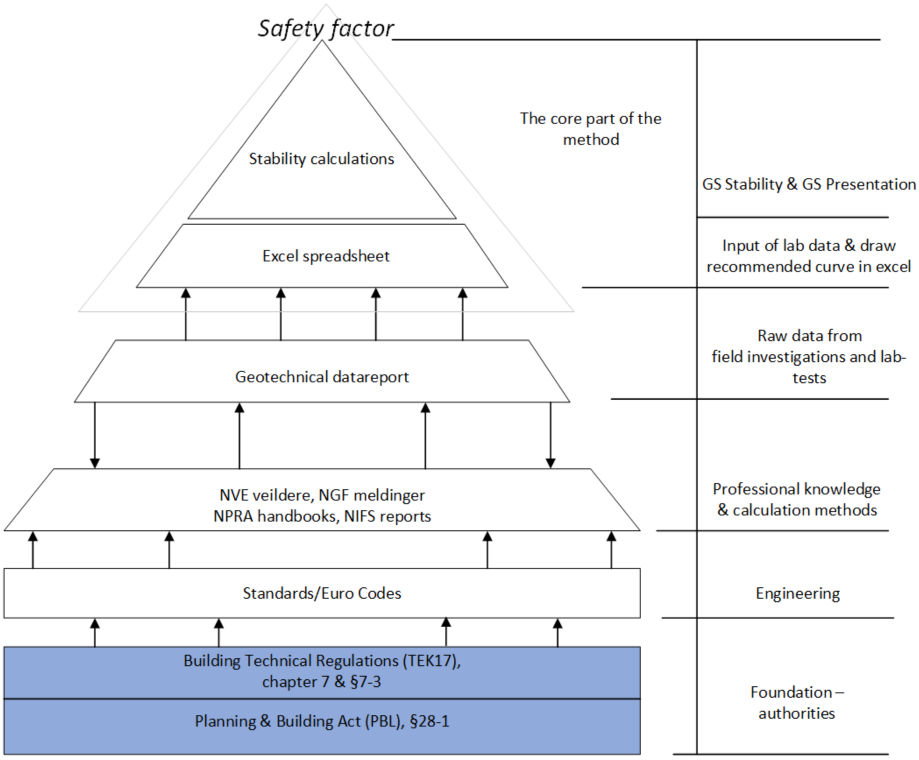


Figure 3.2: Hierarchy (left) & methodology (right). Self-made by author

The foundation of all geotechnical engineering is based on provisions and regulations outlined in laws and regulations such as the Planning and Building Act and TEK 17. In consideration of these, standards, guidelines, and manuals have been developed to reference professional knowledge and methods for carrying out geotechnical work related to construction and infrastructure, among other areas. For further planning, a geotechnical data report is typically utilized. Such a report usually includes information from both field measurements and laboratory tests for the relevant planning area. Today, there are ready-made Excel spreadsheets available that allow for interpretation and input of information from laboratory tests and field measurements. Additionally, various software applications and digital tools exist for stability calculations.

3.3.1 Standards, handbooks, guidelines & reports

Firstly, in this chapter, the handbooks, guidelines, and reports used in this thesis will be introduces. It should be noted that these are not identified in English version, therefore Norwegian names are used in this thesis. In addition, an introduction of geotechnical data report and an Excel CPTu spreadsheet will be provided.

NGF melding nr. 11 Veiledning for prøvetaking

The Norwegian Geotechnical Society (NGF) has developed a guideline for sampling. This guideline describes disturbed, partially disturbed, and undisturbed sampling, and provides instructions for normal application classes for different types of sampling, as well as recommendations regarding the use and control of equipment, sampling procedures, and sample handling [23].

NVE veileder 1/2019 Sikkerhet mot kvikkleireskred

The Norwegian Water Resources and Energy Directorate (NVE) has developed a guidance that describes how landslide hazards in quick clay areas and other soil types with similar properties must be investigated and considered in area planning and building matters. Further, the guide provides information about the security required for construction in such areas, requirements for field investigations, and stability calculations. In addition, it explains how the requirements can be met by provisions in *Byggteknisk forskrift* (TEK 17), and *NVE`s retningslinjer "Flaum- og Skredfare i arealplanar."* [6].

NIFS report 77/2014 Valg av karakteristisk C_uA – profil basert på felt- og laboratorieundersøkelser

This report is prepared by NVE in collaboration with the Norwegian Public Roads Administration (NPRA) and Bane Nor and describes how to choose a characteristic C_uA profile based on field and laboratory investigations [24].

NVE report 14/2014 En omforent anbefaling for bruk av anistropifaktorer i prosjektering norske leirer

This NVE report contains established guidelines for a consensus recommendation/practice that can be used by the geotechnical design community [25].

N200 Vegbygging

N200 Vegbygging contains requirements and guidelines for geotechnical and geological design, solutions, and construction methods for substructures, road embankments, cuttings, and slopes.

Dimensioning, material selection, and implementation for the management of surface water and drainage. Dimensioning, material selection, and implementation of road pavement. Road equipment and environmental measures [26].

V220 Geoteknikk i vegbygging
V220 prepared by NPRA presents the technical basis, principles for calculation, and guidance for geotechnical engineering [21].

Geotechnical data report
The Norwegian Geotechnical Society (NGF) describes a geotechnical data report as follows [27]: "Description of the basic investigations carried out with a quality-assured presentation of raw data by current standards and guidelines."

The report is thus an important starting point for further planning and works with strength and stability assessments in soil masses, foundations of support structures, evaluation of filling/replacement of masses, etc. The report usually contains results from both field investigations and laboratory tests. Suppose an area lies below the marine boundary. In that case, it is usually required to perform a geotechnical assessment in line with *TEK17 §7-3 Sikkerhet mot skred*, which involves an evaluation of area stability in accordance with *NVE veileder 1/2019* [28].

Excel CPTU spreadsheet
NPRA has developed a separate spreadsheet in Microsoft office Excel to interpret and present results from pressure sounding *CPTU* measurements. This spreadsheet is published on NPRA`s websites [29].

3.3.2 Investigations of risk of landslides

There is a requirement for documentation regarding secure land development by the Planning and Building Act (PBL) Section 28-1 and Chapter 7 of TEK17 for area planning, construction case processing, implementation of building measures, and mass relocation [6].

NVE veileder 1/2019 [6, p. 20-23], has developed a step-by-step procedure to be used when assessing and investigating risk of landslides. Step 1-3 is about mapping possible areas of caution for area landslides based on available knowledge. Further, step 4-11, explains how measure of risk (zone assessment) must be carried out when there already exist identified quick clay zones in the area, or when the risk of an area landslide cannot be ruled out after the review of step 1-3. Step 1 and 4-6 is the focus of thesis.

3.3.3 Choice of measure category

To be determined with regard to the requirements in TEK17 § 7-3 and based on the consequences for the measure in the event of landslides according to table 3.2 in NVE veileder 1/2019 [6]. The table is divided into five measure categories K0-K4 with associated descriptions. The choice of category depends on the consequences for the measure in the event of a landslide.

3.3.4 Principles and requirements for safety

The aim is to ensure that the measure does not lead to landslides or that the measure is exposed to landslides that have occurred elsewhere. Furthermore, there is a requirement that safety against area landslides must always be documented. The requirement depends on the measure category, degree of danger and the measure`s impact on the stability of the slopes. Safety requirements for each measure category is described in Table 3.1 and NVE veileder 1/2019 [6, p. 24-27].

Table 3.1: Requirements for measures. Self-made based on content in NVE 1/2019 [6]

	Condition	
Description of measure	**Undrained**	**Drained**
Measures that cause worse stability	Requirement: $\mathrm{F_c} \geq 1{,}40 * f_s$ (*)	Requirement: $\mathrm{F_{c\varphi}} \geq 1{,}25$

(*) It is required that measures which cause the stability to become worse, or which do not worsen the stability of the slope, must have an absolute safety factor f_s equal to 1,15 in addition to the effect of brittle fracture being taken care of.

A calculated value indicating safety against landslides, where F_s < 1,0 indicates failure. The safety factor F_s represents the relationship between stabilizing forces and driving forces along a potential sliding surface and is in stability analyses expressed by undrained $\mathrm{F_c}$ and drained $\mathrm{F_{c\varphi}}$ safety factor. The calculated safety factor is influenced by uncertainties in values of soil mechanical properties, density, pore pressure conditions, as well as uncertainties in the terrain model itself and calculation models [6].

Increasing the stability up to the absolute safety factor in these cases can have disproportionate consequences or be technically demanding to implement. It is therefore possible to plan and implement measures so that the stability of the slope increases or remains unchanged compared to the situation before development. Furthermore, it is assumed that the stability never deteriorates compared to the original state, and that all factors that could lead to breakage or landslides are avoided. Documentation in accordance with the determination of the safety level must be submitted [6].

3.3.5 Shear strength parameters

The behavior of soil is normally dependent on the stress conditions when these are expressed in terms of effective stresses [30]. The undrained shear strength of soil, C_u [kPa], is typically determined using methods such as fall-cone test, uniaxial and triaxial compression tests, or direct shear test in the laboratory, as well as pressure sounding or field vane test in field investigations.

The interpretation of shear strength parameters from in-situ soundings should typically be anchored according to local conditions. It is common practice to perform this by integrating relevant results from soundings, sampling, and pore pressure measurements [6].

Cone-fall test

This is a test normally performed to determine the shear strength C_{ufc} and C_{urfc} for homogeneous, fine-grained soil types with plastic properties based on respectively undisturbed and stirred samples [31].

Unaxial pressure test

Unaxial pressure test is an unconfined compression test C_{uuc} for determining the undrained shear strength of homogeneous, fine-grained soil types with plastic properties. C_{uuc} is the maximum shear strength at fracture determined by maximum axial stress divided by 2 [31].

Triaxial pressure test

The application of consolidation stresses in the triaxial test C_{uC} is crucial as it enables the investigation of the soil's undrained behavior, which is particularly relevant in scenarios where drainage is limited or absent, such as in cohesive soils as clay or during rapid loading conditions. By establishing a stress situation that closely resembles the in-situ conditions, the test provides valuable insights into the soil's strength characteristics, allowing for a comprehensive evaluation of its undrained shear strength and related properties [24, 31, 32].

3.3.6 Choice of characteristic profile

The expected behaviour of the active undrained shear strength C_uA in relation to depth defines a characteristic profile. Experienced data and measurements are normally used to determine this. The chosen profile will in many cases be a weighted mean value based on different data sources. To establish strength profiles, it is necessary to have detailed knowledge concerning the geological and topographic conditions, historical area information, existing basic investigations, simple soundings, in situ measurements, sampling, routine investigations, advanced laboratory investigations and known values from experience. All these data sources will contain some sort of uncertainties. To achieve an interpretation with less uncertainties it is necessary to interpret all data sources regarding each other. Despite this, certain methods are emphasized stronger when determining values for design of undrained shear strength.

Figure 3.3 illustrates an example of a sample plot with recommended curve for a C_uA profile based on different methods and data sources. A sample plot of available data for a given borehole serves as a reference for choice of a characteristic profile and is normally done manually. The curve includes data from CPTU, cone-fall test, uniaxial test and triaxial-and SHANSEP interpretation [33].

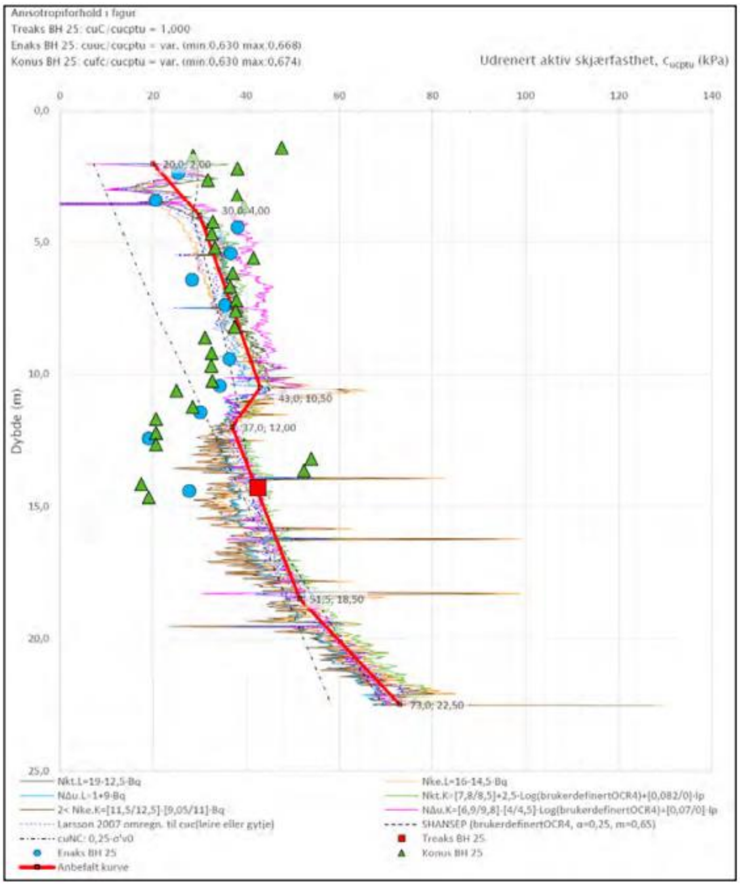


Figure 3.3: Example plot of characteristic profile [33]

3.3.7 Ranking of available data

To determine the factor of safety F_s in stability analyses, the choice of characteristic undrained active shear strength C_uA is a decisive factor. The C_uA profile is normally selected as the most probable active shear strength for the soil material in accordance with NIFS rapport 77/2014 [24]. The report proposes the following ranking when assessing the available data for choosing this profile [24, s. 23]:

1. Triaxial tests of good quality (quality class 1)
2. CPTU
3. Experienced values
4. Field vane/Cone/Unaxial

The ranking is based on which data are experienced as most credible when determining shear strength.

3.3.8 Laboratory investigations

From measurements in the laboratory and/or in the field, soil parameters applied in geotechnical context can be determined. By utilizing various sampling methods, soil samples are collected for classification and analysis in the laboratory to determine their composition, as well as strength and deformation properties. The soil samples collected are typically categorized and ranked based on their quality, and then divided into respective sampling categories. These categories are as follows [21]:

Category A: Undisturbed samples
Category B: Disturbed samples
Category C: Stirred samples

A sample is classified as undisturbed when its material structure and water content closely resemble the natural conditions of the soil.

Quality of sample
As stated in NVE veilder 1/2019 [23], it is recommended that samples applied in laboratory analysis should exhibit a high level of quality. Sample quality is typically dependent on the type of clay. Less deformable and quick clays are more susceptible to sample disturbance than plastic, deformable, and less sensitive clays. As described in V220 [21], experience indicates that the probability of disturbed samples increases with increasing sample depth. This is because stresses normally increase with depth. There are several criteria for assessing sample quality, such as water extrusion, change in volume strain, or void ratio. Experience suggests that void ratio is normally a more favorable parameter for assessing sample quality compared to changes in volume strain. The changes in void ratio can be applied to assess the quality of a sample according to the criteria presented in Table 3.2

Table 3.2: Criteria for assessment of sample quality - reproduced from NGF melding nr.11 [23]

OCR	$\Delta e/e_0$			
	Very good to excellent	Satisfactory to acceptable	Poor	Very poor
1 – 2	< 0,04	0,04 – 0,07	0,07 – 0,14	> 0,14
2 – 4	< 0,03	0,03 – 0,05	0,05 – 0,10	> 0,10
4 – 6	< 0,02	0,02 – 0,035	0,035 – 0,07	> 0,07

For further assessment of sample quality, it is common practice to perform a visual evaluation of the sample by studying the strain level and/or curve shape.
The curves in Figure 3.4 from triaxial test illustrates how quality of samples can be assessed in terms of stress paths (curve shape). In the figure the stress paths are classified as (i) good quality, (ii) disturbed sample and (iii) disturbed sample.

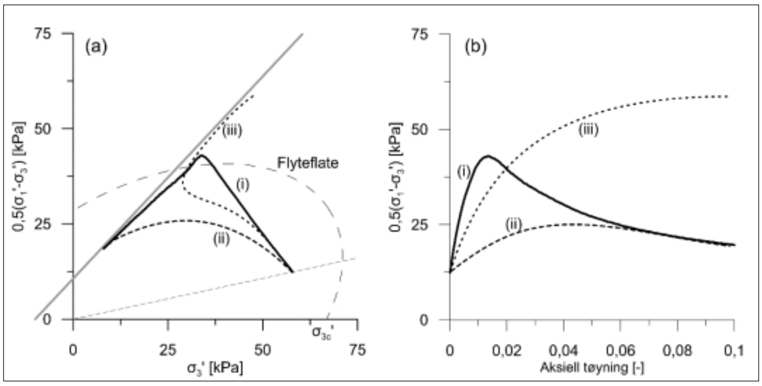


Figure 3.4: Examples of stress paths from tests of good and poor sample quality [21]

Consistency limits

The liquid limit and plastic limit, also known as the consistency limits, are parameters that are only applicable to plastic (deformable) soil types, i.e., cohesive materials in clay and fine silt fractions. The water content of a soil type where the soil consistency changes from plastic to liquid in a stirred state is defined as the liquid limit W_L. This limit is normally determined in a cone test. Further, when the water content in a stirred state ceases to be plastic and is further dried, it is referred to as the plastic limit W_p [21].

W_p is determined by rolling a clay string until it crumbles, gradually reducing its water content. These limits are used to indicate the behavior of soil in contact with water. For further material characterization, the plasticity index I_p, which is the difference between W_L and W_p, is normally used. I_p indicates the extent of the plastic range and how quickly the soil reacts if the water content changes. I_p is also related to strength and compressibility [21].

3.3.9 Analysis methods

Total and effective stress analysis

There is a requirement to perform stability calculations for both short time stability and long-time stability. Short-time stability is defined as stability calculations in undrained conditions with total stress analysis F_{cu} and describes what the slope might be able to withstand from rapid load changes in situations such as excavation or filling before fracture. In addition, it may also express the slopes strength and bearing capacity. In this analysis method soil-layers with brittle fracture behavior are modelled with undrained strength, while other materials are modelled with effective stress parameters. Long time stability is stability in drained conditions with effective stress analysis $F_{c\varphi}$ and can be performed for a static situation without expected load changes. The purpose of carrying out stability calculations in both conditions is because the shear strength of clay is time dependent and therefore dependent on the speed of the load change. These two analysis normally give clearly different results [6].

3.3.10 Stratification and loads

Stratification

Normally, an interpretation of the ground is performed based on the results from soil investigations. This is done to establish a soil profile for use in stability analyses. During this interpretation process, it is crucial to be precise in defining the layers based on distinct strengths and failure characteristics. Attention should be given to identifying potential weak layers that may trigger landslides. When cohesive soil layers with minor variations in properties are identified, it is common to evaluate whether a combination of these layers would provide the most representative average parameters [6].

Loads and partial factors

In stability calculation for traffic loads there is a requirement in N200 [26] to use an evenly distributed characteristic load F_{rep} of 15kPa across the entire width of the road. In addition, to cover for the uncertainties related to load size and effect, partial factors (safety factors) must be applied. For variable loads the partial factor is expressed as γ_Q and is equal to 1,3. If the load is favorable, γ_Q shall be 0. This provides the basis for determining the design characteristic load F_d which can be expressed by the following equation [21]:

$$F_d = \gamma_Q * F_{rep} \qquad \textit{Equation 3.1}$$

3.3.11 Soil parameters

Density

The density ρ which expresses the material`s mass per unit volume, is determined by samples with at least quality class 2 according to table 2-2 in v220 [21]. For mineral materials, the range of the density is typically 15 – 22 kN/m³.

Shear strength anisotropy

The clay in Scandinavia is very anisotropic in nature. This means that the undrained shear strength C_u for a material is different in all directions as illustrated in Figure 3.5, when exposed for active compression (A), direct shear force (D) and passive extension (P) load. The undrained shear strength of clay is influenced by the direction of the strain in relation to the in-situ stresses and the structural arrangement of clay minerals. Strength anisotropy plays a major role in many geotechnical contexts including slope stability analyses and bearing capacity for shallow and deep foundations. In undrained situations the value of direct and passive shear strength is typically lower than that of active shear strength, with the passive strength exhibiting the smallest value among these. Thus, circular shear surfaces` resistance to fracture will be determined by a weighted value of these factors. A considerable part of the shear surface will be subjected to direct shear in composite shear surfaces. Based on this ADP-factors has been increasingly common to use in stability analysis of an undrained situation to describe the strength anisotropy of sensitive clay [25, 34].

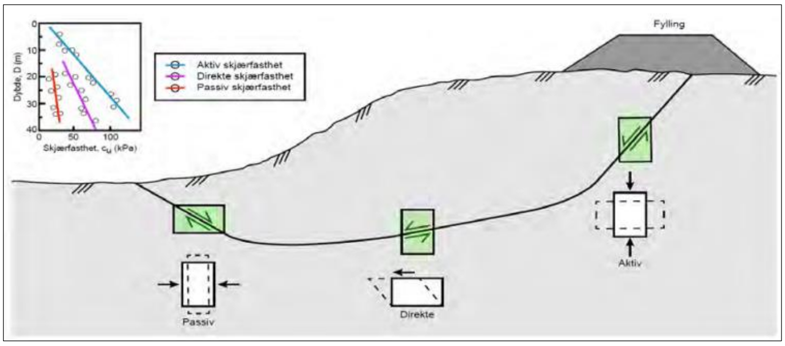


Figure 3.1: Anisotropic state of stress in critical slopes [33]

Choice of ADP-factors can be determined based on information about the plasticity index and according to the recommended values presented in Table 3.3. It is stated in NVE Report 14/2014 [25] that the current practice for selecting ADP factors with $C_{uD}/C_{uC} = 0{,}6 - 0{,}7$ and $C_{uE}/C_{uC} = 0{,}3 - 0{,}4$ is most likely not a direct cause of landslides or failures.

Table 3.3: Recommendation of ADP-factors – reproduced according to content in [25]

$\boldsymbol{I_p}$	$\boldsymbol{c_{uD}/c_{uC}}$	$\boldsymbol{c_{uE}/c_{uC}}$
$I_p \leq 10\%$	0,63	0,35
$I_p > 10\%$	0,63+0,00425*(I_p-10)	0,35+0,00375*(I_p-10)

3.3.12 Effective shear stress parameters

In the context of clay materials, the determination of effective stress parameters typically relies on the interpretation of triaxial test results. The purpose of the interpretation process is to ascertain the location of the fault line or fracture as illustrated in Figure 3.5 and subsequently assess the material's resistance capabilities and establish the boundary conditions. This investigation is essential to ensure that the design is sufficiently distant from the fracture zone, thus mitigating potential adverse effects [32, 35].

Attraction and angle of friction

Attraction a is a curve size and may also be defined as a negative tensile strength of a material [32, 35]. The linear shear strength in effective stress analysis is defined by Figure 3.5 and the linear relationship between cohesion $C`$, attraction $a$ and the angle of friction $\varphi`$, and can be further expressed by the following equation [21]:

$$C` = a * \tan \varphi` \qquad \textit{Equation 3.2}$$

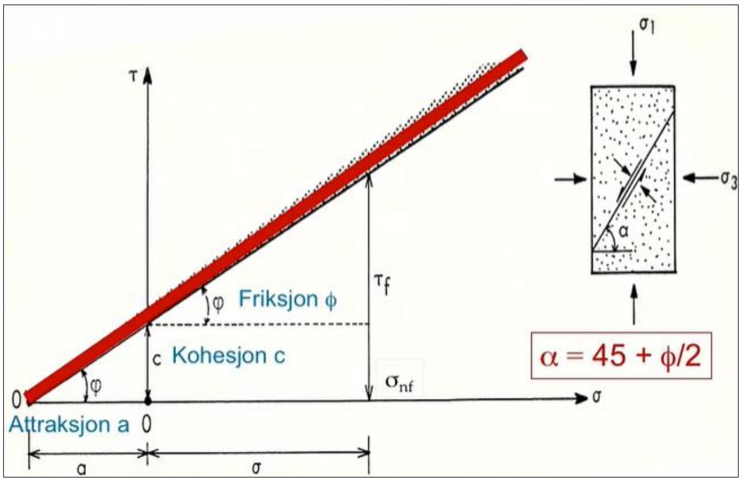


Figure 3.5: Shear strength in effective stress analysis – obtained from lecture at NTNU [32]

3.3.13 Stabilization measures

There are various stabilization measures to ensure safety and avoid landslides in quick clay areas [6]. Depending on measure category, there are measures such as erosion control, lime-cement stabilization, salt stabilization and topographical measures. In this thesis the focus lies on topographic measures.

Topographic measures (se kap 6.2.2 NVE 1/2019)

According to NVE veilder 1/2019 [6], a change in topographic conditions is the measure normally implemented to increase safety against area landslide.

Assessment of quality class

The choice of design parameters for stability calculations must be based on the achieved quality of the samples. This assessment of quality class is normally performed by following procedures described in NVE veileder 1/2019 [6].

4 Research question

Based on acquired knowledge from the subject BYG515 spesialiseringsprosjekt as well as literature presented in chapter 3, the following research question have been prepared:

"How to perform a geotechnical area stability assessment of Strengselva in accordance with NVE veileder 1/2019 Sikkerhet mot kvikkleireskred?"

To answer the research question, the following sub-questions will be investigated:

- *What is the current safety factor at Strengselva and how does this differ after appropriate measures?*

- *Which analytical and numerical methods are used for assessment of slope stability and could these be relevant in Norway?*

4.1 Limitations

In order to answer the research question, the following limitations have been made:

- All calculations in this thesis relate to the Lilleholt section/stretch. [2]
- Only landslide caused by quick clay is included in this thesis.
- Mapping of quick clay is not included. This information is already provided in the documents assisted by the client.
- Interpretation of shear strength is limited to triaxial pressure test.
- Service limit state is not included in this thesis.
- Interpretation and assessment of data from oedometer test is not included.
- Calculations of erosion control is not included.
- The author has been informed by the client that it is not common practice to conduct manual calculations for the determination of the safety factor $\mathrm{F_s}$ in the investigation of area stability. This is due to the extensive research and documentation that underpins the selection of parameters and factors, in conjunction with experience, interpretations, assumptions, and analysis of laboratory and field data, which are utilized in digital tools to form the basis of stability calculations. As such, manual calculations have been excluded from this study, and only simple intermediate calculations relevant to solving the task have been performed.

[2] See elaboration in the Case chapter.

5 Case and materials

This chapter presents the case that has been chosen as a starting point to answer the research question and the materials used to solve this case.

5.1 Case

Objektiv presentasjon av hele casen:
Strengselva in Tvedestrand municipality is a tributary that starts/runs from Jordstadvannet and 5.5 km through a landscape marked/consisting of agriculture with associated weak natural masses such as clay, silt and sand, before it flows into the Storelva as illustrated in Figure 5.1 Strengselva bears the hallmarks of a poor ecological condition, the main cause of which is agriculture and road development as described in Appendix A. In parts of the stretch, the river lies alongside the road embankment, which ensures a certain stability, but at the same time there are challenges associated with layers of weak masses between the road embankment and the riverbed. The river is exposed to erosion along the riverbanks and bottom [36, 37], which results in limited visibility and sedimentation with the following poor habitat conditions for riverine organisms. Based on this, thorough restoration/restoration/of the river is necessary, the purpose of which is to improve the habitat conditions and the state of erosion.

Figure 5.1: Overview map with location of Strengselva – modified by author [38]

The author has chosen to immerse himself within the stretch of river around the area of Lilleholt, a stretch of around 230 meters as illustrated in Figure 5.2 The area around this section mostly consists of cultivated land.

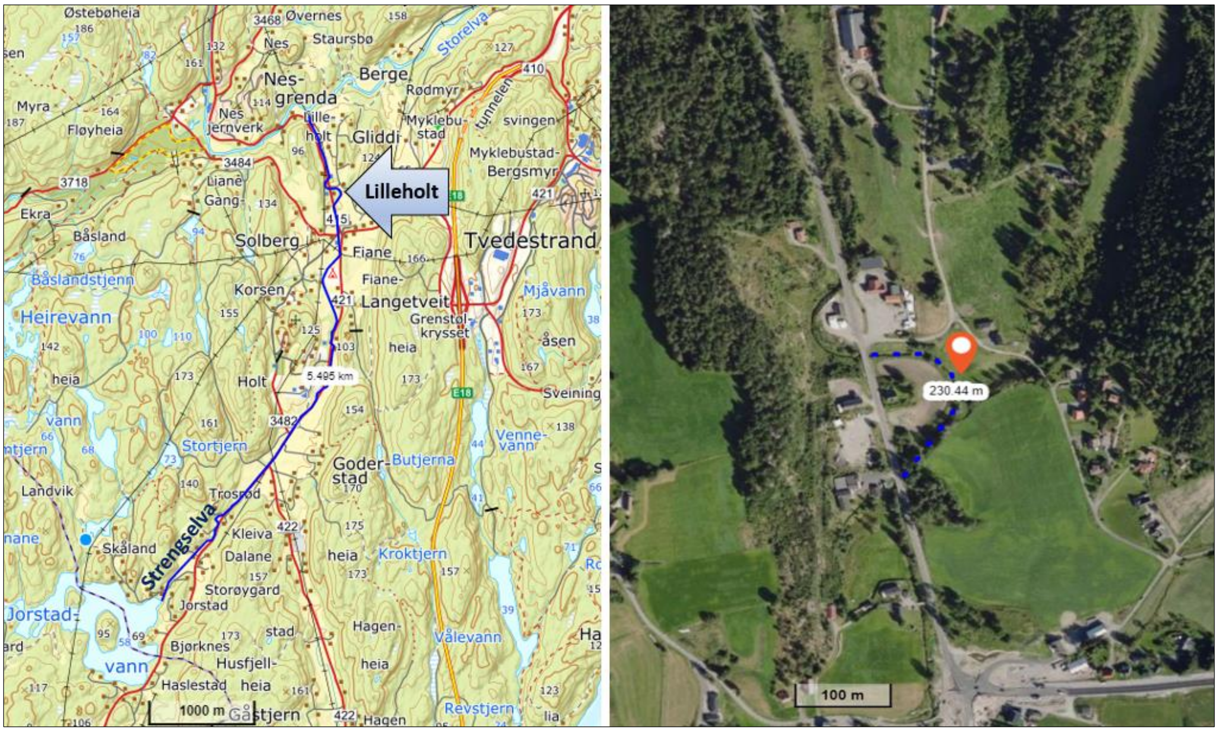


Figure 5.2: Basic map (left) [38] & aerial foto (right) [39] – Modified by author

The area between Lilleholtveien and the river, as demonstrated in Figure 5.3 (left), is only designed for grazing and thus unsuitable for the use of machinery. Here the slope is steep from the road and towards the river, but of less importance in an agricultural context. In general, the entire river stretch is characterized by erosion and the existing protection against erosion has slipped into the river course. The riverbed contains of weak masses such as silt and clay as shown in Figure 5.3 (right). Furthermore, the stretch is significantly affected by erosion damage to the sides and bottom as shown in which causes the river to excavate new side courses which in turn can cause damage to the terrain.

Figure 5.3: Raster map (left) & part of riverbed (right) [37, 40]

5.2 Digital tools

The digital tools used to solve the case in this thesis are presented here.

Novapoint GeoSuite (GS) Toolbox

GS Toolbox is a collection of software modules that Geotechnical engineers need for design, engineering, drawing production and 3D modelling. All these modules can be handled in a unified model with complete BIM interoperability. This ensures that geodata can be shared regardless of location. GS is a Finite Element Method (FEM) based program [41].

Based on this Toolbox, the following modules are used in this thesis [41]:

Novapoint GeoSuite (GS) Presentation

GS Presentation is the core element of GS Toolbox. The starting point for all geotechnical activities is to investigate the properties of the ground. These investigations are presented as diagrams such as single boreholes, plans or as cross-sections and profiles. GS Presentation has well-adapted functions for managing and presenting data [41].

Novapoint GeoSuite (GS) Stability

GS Stability is a powerful tool for stability analysis that uses cross-section as templates to ensure safe and fast modelling of the ground's geometry. Profiles for modeling undrained shear strength and pore water pressure provide great opportunities to simulate any condition in the ground. Enhanced control of input effectively reduces the risk of incorrect models. Various solution methods are supported. In addition to graphical presentation of the results, a comprehensive report can also be selected. [41].

Autodesk Civil 3D

Autodesk Civil 3D is a software solution for all types of infrastructure projects and a powerful tool for construction, analysis, and documentation with a workflow adapted to BIM. This means that it is possible to create models with actual data from map data and carry out analyses, and calculations to find the best engineering plan in a given area [42].

6 Methodology

The methodology for answering the research question is in this thesis based upon two different approaches as illustrated in Table 6.1. The purpose of the literature review in this thesis was to establish sufficient theoretical background, to formulate a research question and to identify case studies presented in the result chapter. Further, assessment of area stability was performed by applying handbooks, reports, and guides, and serves as basis for the stability analyses. Finally, stability analyses were conducted by applying GeoSuite software.

Table 6.1: Illustration of the methodology approaches

Chapter	6. Methodology
6.1	Literature review
6.2	Assessment of area stability

6.1 Literature review

In this chapter a hierarchical structure, as illustrated in Table 6.2, is applied to define an overall strategy. The first step involves fundamental themes and focuses on geological conditions, while the subsequent two steps encompass desirable subject matter, namely climate-related stresses, and sensitive clays. This strategy forms the basis for the development of relevant search terms to be used in the search for pertinent literature in this thesis.

Table 6.2: Hierarchical strategy to develop relevant search terms

Theme	Keywords
1.Geology	Marine deposits/sea level/last ice age
2.Climatic stresses	Rainfall/frost
3.Sensitive clay	Landslide/slopes

6.1.1 Unstructured searched

Popular science sources

Popular science sources as Google were applied to obtain general information, instead of academic search engines, as the goal was not identifying thorough and advanced research.

Snowball method

Parts of the literature presented in chapter 1-3 of this thesis was obtained using the "snowball method" [43]. The snowball method is defined as a search method for tracking down references in documents. This method was used towards NVE veileder 1/2019 Sikkerhet mot kvikkleireskred [6]. The purpose was to identify references within relevant chapters in this thesis regarding selected headings.

Table 6.3 presents the themes covered in the respective chapters. As the table describes, in the theoretical background covered themes correspond to headings in the sub-chapters.

Table 6.3: Overview of themes included in chapter 1-3

1.Introduction	2.Societal impacts	3.Theoretical background	
Theme		**Sub-chapter**	**Theme**
Climate in Norway	CO_2 - emissions	3.1	Area landslide
Climate projections			
Climate change			
Climate adaption	Stabilization of quick clay	3.2	Soil with brittle fracture properties
Landslide hazard			
Security measures	Sosio- economic consequences	3.3	Slope stability
Landslide in Norway			

Other necessary information

Client from Agder fylkeskommune, Emilie Laache has also assisted with relevant and necessary information, and these include the following documents:

Appendix A - Strengselva_Tiltaksbeskrivelse
Appendix B - Geoteknisk datarapport
Appendix C - FV.421 Borplaner_Strengselva_Tvedestrand
Appendix D - Rådata grunnundersøkelser
Appendix E - Laboratorieundersøkelser
Appendix F - Cptu-2021_borpunkt 2
Appendix G - Terrengprofiler
Appendix H - Korrigerte grafer fra treaksiale trykkforsøk
Appendix I - Situasjonsplan med profiler 1A og 1B

From appendix I, only profile 1A is used in this thesis.

6.1.2 Structured searches

Identification of literature via databases

Firstly, a literature search was performed by applying the Oria and Google Scholar databases, which provided a flexible approach to refine search criteria according to the thesis. Furthermore, both databases enabled direct import of bibliographic records into an Endnote source library. Initially, a set of pre-defined criteria and relevant search terms according to Table 6.2 were applied to identify articles pertaining to the thesis, with time frame limited to the last 10 years. However, the results from this search proved to be insufficient in yielding relevant articles.

As such, a new search strategy as presented in Figure 6.1 was implemented to broaden the scope of the search and improve the chance of identifying relevant literature. This search strategy was conducted with revised limitations and criteria, and relevant search terms and phrases were still applied in relation to the thesis and according to Table 6.2. A selection of the search terms and phrases used in this strategy are presented in Table 6.4. The time frame was defined to no more than the last 20 years, as research literature on the subject is scarce, except for countries such as Norway and Sweden, and some places in Finland, Russia, Canada, and Alaska. Furthermore, it was specified that the selection of articles be limited to peer-review English language publications with open access. A logbook applied to structure and support the search process is provided in Appendix M.

Table 6.4: Search terms and phrases

A selection of search terms applied in the search strategy	
Slope stability	Lime cement
Landslide	Soil stability
Sensitive clay	Safety factor
Marine deposits	Quick clay
Stabilization of clay	Critical slopes
Stability analysis	Landslide hazard
Assessment	Area stability
Methods	Slope investigations
Safety	Stability investigations

The initial search yielded a total of 108 hits, from which a subset of 14 articles was selected based on their abstract's relevance to the thesis. Subsequently, six of these articles were subjected to a full-text review, resulting in a final selection of two articles. In an attempt to identify more articles, three more searches were performed by applying the same specified criteria and search strategy. The first search yielded a total of 2971 hits from which 10 were selected based on their abstract`s relevance to the thesis. Three of these articles were full text reviewed, resulting in a final selection of one article. The second search yielded a total of 2241 hits from which 10 were selected based on their abstract`s relevance to the thesis. Three of these articles were full text reviewed, resulting in a final selection of two articles. The third and final search yielded a total of 1928 hits from which four were selected based on their abstract`s relevance to the thesis. One of these articles were full text reviewed, resulting in a final selection of one article. A total of six article were identified through this strategy and summary of all selected articles will be presented in the results chapter and will contribute to the discussion chapter, as well as addressing the second sub-question.

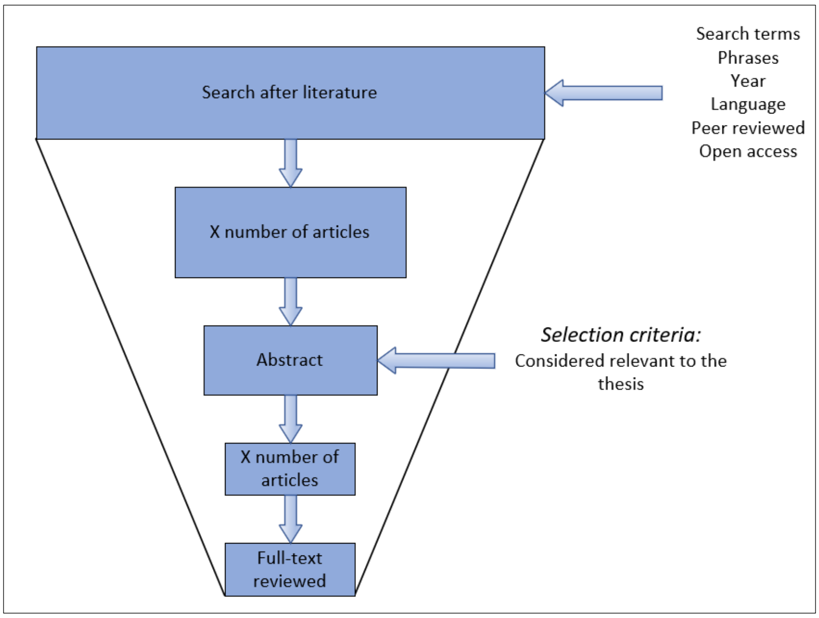


Figure 6.1: Search strategy

6.2 Assessment of area stability

At the onset of the project, relevant chapters in the NVE veileder 1/2019 [6] were reviewed to facilitate an improved understanding of the key considerations and factors that are pertinent to the assessment of area stability. This chapter presents the key considerations and factors relevant for this thesis through themes as illustrated in Table 6.5.

Table 6.5: Themes for assessment of area stability

6.2 Assessment of area stability	
Sub-chapter	Theme
6.2.1	Choice of sub-area
6.2.2	Identification of critical slopes
6.2.3	Stratification
6.2.4	Parameters of effective stress analysis
6.2.5	Choice of measure category
6.2.6	Quality and class
6.2.7	Interpretation of shear strength
6.2.8	Loads and ADP-factors
6.2.9	Analysis in GeoSuite

6.2.1 Choice of sub-area

In this thesis, the section is limited to Lilleholt, which is described in chapter 5.1. Lilleholt is selected as case mainly because the area consists of marine deposits and the entire stretch is prone to erosion, and according to the measure description included in Appendix A, topographical changes, and erosion control must be carried out along the entire stretch. However, in this thesis the focus is on topographical changes.

6.2.2 Identification of critical slopes

Critical slopes in the planning area were identified using Norgeskart prepared by the Mapping Authority [44]. These slopes are characterized by being the steepest and highest. This was performed by using height profile drawing function in Norgeskart and conducted in order to draw terrain profiles for implementation in Novapoint GS.

6.2.3 Stratification

To determine the appropriate parameters for the correct depth intervals of the various layers used in stability analyses, it is necessary to assess the characteristics of the layers. This assessment was conducted by studying the behavior of the borehole resistance and the stress-strain curve for boreholes 1 and 2, as provided in Appendix B.

6.2.4 Parameters of effective stress analyses

The angle of friction φ for sand was not provided in the triaxial test included in appendix B, thus experience parameters from table 2-21 in V220 [20] were obtained as presented in Table 6.10.
To compute the drained shear strength C`, it was necessary to interpret the angle of friction φ and attraction a of clay and quick clay from the curves in triaxial test included in Appendix B.

This knowledge was not previously acquired. In an attempt to cover for this, the methodology of interpretation is mainly based on lecture materials from *Flexible learning in geotechnics "Modul B – Lab og Felt – Treaksialforsøk, del 1 og del 3"* taught at The Norwegian University of Science and Technology (NTNU), by Rolf Sandven in 2015 [32, 35]. Firstly, value of *a* was interpreted, followed by simple trigonometrical functions to determine φ. Finally, C` was calculated by applying the following equation (reproduced from chapter 3.3.12):

$$C` = a * \tan \varphi`$$

In the final stages of this approach, an error was discovered in the relationship between the shear stress ($\sigma_a` - \sigma_r`/2$) along the x-axis and the effective radial stress $\sigma_r`$ along the y-axis. Specifically, the distance between the same values was different along the x- and y-axes, resulting in incorrect interpretations. As a result of this, corrected graphs were requested from the client, Emilie Laache at Agder fylkeskommune, and was provided by the responsible party for conducting the triaxial tests, Geo Strøm AS. The corrected graphs are presented in Appendix H and the interpreted version of these are included in Appendix K.

6.2.5 Choice of measure category

Choice of category

The choice of measure category is conducted in accordance with the guidelines outlined in chapter 3.3.3 and table of the NVE veileder 1/2019 [6]. The decision is based on the descriptions of measures provided in Appendix A and the document titled "*Plan for erosjonssikring og miljøforbedring av Strengselva - Tvedestrand kommune" by Terrateknikk AS* [36]. The measures primarily involve small-scale topographical modifications, such as the transportation and addition of masses. This consitutes a crucial factor in the choice of measure category.

Stabilization measures

In accordance with the measure description in Appendix A, it has been chosen to improve the stability in the case-area through the implementation of riverbanks slopes with new river profiles at 1:2 and 1:3 ratios, as illustrated in Figure 6.2. It should be noted that this figure is used as inspiration for det performance of new river profiles.

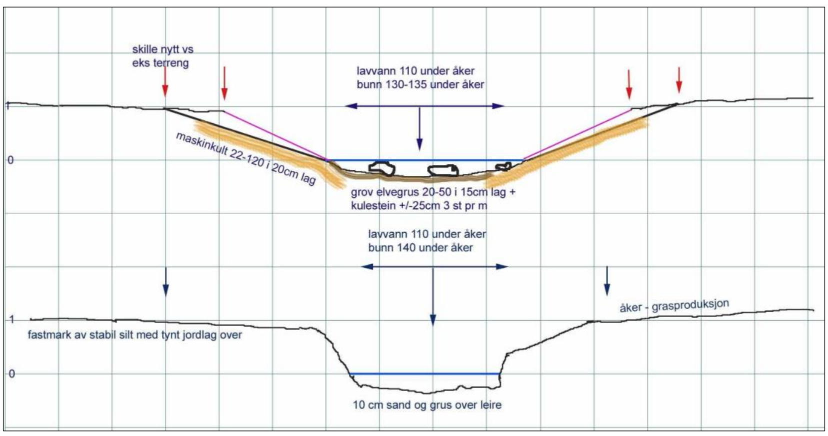


Figure 6.2: New improved river profiles – illustration performed by Terrateknikk AS [36]

6.2.6 Quality and class

The sample quality was initially assessed based on the stress-strain curve obtained from the triaxial tests included in Appendix B and in consideration of Figure 3.4. Building upon this a further evaluation of the sample quality was conducted by examining the available data for the void ratio change ${}^{\Delta e}/_{e_0}$. These data indicated ${}^{\Delta e}/_{e_0}$ values of 0,016 and 0,032, which were assessed according to the following table (reproduced from chapter 3.3.8):

OCR	$\boldsymbol{\Delta e/e_0}$			
	Very good to excellent	Satisfactory to acceptable	Poor	Very poor
1 – 2	< 0,04	0,04 – 0,07	0,07 – 0,14	> 0,14
2 – 4	< 0,03	0,03 – 0,05	0,05 – 0,10	> 0,10
4 – 6	< 0,02	0,02 – 0,035	0,035 – 0,07	> 0,07

6.2.7 Interpretation of shear strength

NVE veileder 77/2014 [24] deals with interpretation of shear strength. This was read through to gain a basic understanding and overview of how the interpretation is to be carried out. The CPTu Excel spreadsheet included in Appendix F, was applied for interpretation of shear strength. This chapter presents a review of how the interpretation process was performed.

Excel CPTu spreadsheet

In charge of conducting CPTu soundings, Geo Strøm AS has only carried out this task for borehole 2, in Appendix B. Therefore, the CPTu interpretation spreadsheet provided in Appendix F is only applied to this borehole. To define the in-situ stress conditions in the ground the groundwater level (GW-level) had to be assumed due to lack of information about this level. Further, density values were obtained followed by values of shear strength C_{uuc} from unaxial test and C_{ufc} from cone test, and plasticity. All tables in this chapter presents input values applied in the spreadsheet, obtained from Appendix B.

Table 6.6: Density values

Layer	Depth [m]	Density ρ [kN/m³]
Clay	0,8	17,4
Clay	3,3	17,4
Quick clay	4,6	16,9
Quick clay	7,4	16,0
Quick clay	9,5	16,4

Table 6.7: Shear strength values of uniaxial test

Depth [m]	Shear strength C_{uuc} [kPa]
3,3	30,3
4,6	16,7
7,4	13,1
9,5	13,9

Table 6.8: Shear strength values of undisturbed sample from cone test

Depth [m]	Shear strength C_{ufc} [kPa]
3,2	26,9
3,7	16,01
4,3	13,23
4,7	15,56
7,2	13,95
7,7	13,58
9,2	13,95
9,6	19,15

Table 6.9: Other index parameters

Depth [m]	Plasticity [%]	
	Plastic limit W_p	Liquid limit W_L
4,1	15,7	23,4
9,4	21,4	38,8

The active undrained shear strength C_{uC} was determined through interpretation of the shear stress curve from triaxial test included in Appendix B. This interpretation involved noting the maximum shear stress at fracture point and axial deformation for depths of 4.5m and 9.4m, respectively. To ensure a clear margin from fracture these values were adjusted to remain well below the limit of the fracture point. The evaluation is presented in Appendix M.

Based on the interpretations, tables and information described above, a recommended curve of characteristic profile C_uA was drawn. This resulted in input values of C_uA for further use in stability analyses.

6.2.8 Loads and ADP-factors

For traffic load a design characteristic load F_d was calculated using the required value of an evenly distributed characteristic load F_{rep} equal to 15 kPa and a partial factor γ_Q equal to 1,3 presented in chapter 3.3.10, and by applying the following equation (reproduced from chapter 3.3.10):

$$F_d = \gamma_Q * F_{rep}$$

To determine input data of the ADP-factors, the plasticity index I_p had to be evaluated. This was achieved by obtaining data for consistency limits W_L and W_p presented in Table 6.9. I_p was then evaluated by subtracting W_p from W_L and applying this in accordance with Table 3.3. This resulted in input data of ADP-factors.

6.2.9 Analysis in GeoSuite

Stability analyses have been conducted by applying the GS Toolbox, which includes two modules: GS Stability and GS Presentation. The methodology applied for the analyses is illustrated in Figure 6.1 and a review of this methodology will be provided in the following sections. The purpose is to calculate the safety factor F_s through undrained (total stress analysis, F_{cu}) and drained (effective stress analysis, $F_{c\varphi}$) analyses of the current situation and after measured have been implemented. This is to verify if the value of F_s meets the requirements in Table 3.1 and to document the safety of the slopes that are nearby the river in the selected case-area. It should be noted that the applied modules are interdependent, meaning that both modules are required to carry out the analyses as illustrated in the figure below.

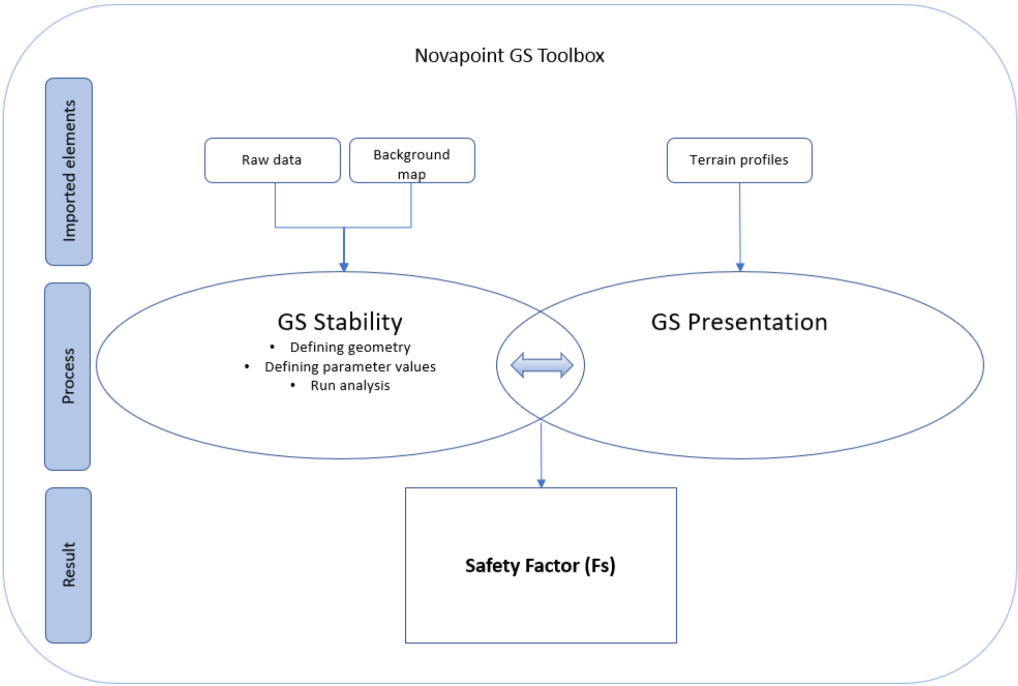


Figure 6.1: Methodology for Stability analyses in GeoSuite

Imported elements

The initial modelling process started with the import of a terrain profiles in GS presentation for situation 1A included in Appendix G. Further, raw data included in Appendix D and background map provided in Appendix G was imported in GS stability.

Process

The process illustrated in Figure 6.1 is performed by defining the geometry which includes defining actual layers based on chapter 6.2.3, adding assumed GW-level, editing soil surface and creating a soil surface hatch. Further, a characteristic profile C_uA was defined by applying the values from Table 7.8 followed by adding the values for density ρ and sub-merged density ρ` from Table 6.10 and Table 7.5. For sand value of $\rho$, attraction $a$ and angle of friction $\varphi$ was obtained from empirical data in V220 [21], and presented in Table 6.10. Finally interpreted values of friction angle $\varphi$, attraction $a$, drained shear strength C` were added from Table 7.2 as well as ADP-factors and traffic load parameters from Table 7.7.

Table 6.10: Obtained density value ρ of all layers and φ of sand

Layer	ρ [kN/m³]	φ [°]	a [kPa]
Clay	17,4	*	*
	17,4		
Quick clay	16,9	*	*
	16		
	16,4		
Sand	17	33	0
	17		

*The values will be determined by interpretation

In addition to conducting analyses of the current situation and after the implementation of measures in drained and undrained situation, further analyses were performed of drained situation $F_{c\varphi}$ by replacing the interpreted value of φ in Table 7.2 with recommended empirical values from table 2-21 in V220 [21]. The reason for selecting this parameter is that there was a significant difference between the empirical and interpreted values. The empirical input values applied are presented in Table 6.11. It should be noted that the empirical value of quick clay is not presented in table 2-21 [21], therefore the empirical value of clay is selected as representative for both clay materials in these analyses. The GW-level in the slope was adjusted to 2m below the terrain surface to check the impact on the results.

Table 6.11: Empirical values of attraction φ -based on content in V220 [21]

Material	Angle of friction φ [°]
Clay	26
Quick clay	26

The purpose of these additional analyses is to examine the impact that adjustments to the factors may have on the results. This will also contribute significantly to the discussion and provide insights for addressing the research question. A total of 12 analyses have been conducted in this thesis.
The ground water level was drawn one meter below the terrain surface.
The input data of characteristic active undrained shear strength profile C_uA for borehole 2 is based on the interpretation methods and data sources described in chapter 6.2.7 and the recommended curve drawn in the Excel CPTu spreadsheet provided in Appendix F. When determining input data for C_uA profile this borehole is used as reference for both boreholes applied in this thesis. These input data are presented in Table 7.8 and included in Appendix J.

The last step in this methodology is the result, which is the safety factor F_s in undrained and drained conditions. This will be presented in the result chapter.

6.3 Formatting

All figures and tables in this thesis without clear references, are self-made by author. Microsoft programs such as Word, Visio and PowerPoint have been applied in the creation. Different visualization methods based on maps have been applied to provide a more easily interpreted sense of the problem area. These maps have been obtained from Norgeskart [38-40, 44] and customized by using its integrated clippings and drawing functions.

7 Results

7.1 Previous research

This chapter provides a summary of the articles obtained through a systematic literature search. These selected articles play a significant role in addressing the second sub-question of this thesis and contribute significantly to the discussion chapter as well.

F. Göktepe and I. Keskin [45], conducted in 2018 a comparison study between traditional Limit Equilibrium Methods (LEM) and Finite Element Methods (FEM) for slope stability evaluations. Slope stability was evaluated by considering a failure mechanism named as plastic limit condition (e.g. Mohr-Coulomb`s Criteria). Safety factor F_S was calculated using different numerical analysis with FEM and LEM. In the article slope angle is mentioned as key component in slope stability analysis. If the slope angle increase, it may normally affect the increase of shear stress in the soil or other unconsolidated materials. The authors argue on modelling slopes with a degree of very high realism and greater demonstration of soil deformations as examples of advantages using FEM in stability calculations. High realism means modelling with complex geometry, loading sequences, the presence of material for reinforcement, the effect of water, laws for complex soil behaviour etc. Experienced values from laboratory tests and in-situ test for slope stability analysis served as a basis for analyses in slope stability softwares as Plaxis, Geoslope and Slide.

With computational solution using LEM and FEM combined with software's the methods presented the results illustrated in Figure 7.1.

- The obtained results for LEM and FEM are similar.

Table 7.1: Results from LEM and FEM – reproduced based on content in [45]

Method		F_S	
		Slope/W	Slide
LEM	Fellenius	1.275	1.161
	Bishop	1.279	1.164
	Janbu	1.246	1.125
FEM	c-Ø reduction	1.115	

"An analysis of critical slopes stability in Bendosari Village, Pujon Malang due to the effect of rainwater infiltration" by I. Nindia Rizky, S. Eko Andi and R. Arief [46], investigates the stability of critical slopes in accordance with rainwater infiltration. The article seeks to identify the possible risks and supply with suggestions for slope stability measures in the area. In this study data from field and information concerning the geological conditions such as slope geometry, and rainfall regimes as well as laboratory tests, were performed to evaluate soil properties and to conduct slope stability analyses by applying conventional approaches. The slope stability analysis was performed by applying GeoStudio 2D software SEEP/W and SLOPE/W. SLOPE/W was used to determine the safety factor for slopes under dry and rainy conditions.

The findings validated that rainwater in both conditions may reduce the safety factor because of the content of water in the soil that increased pore water pressure. The soil shear strength was reduced due to high content of water in the soil. To improve the critical slopes in the area, the researchers propose to apply soil reinforcement at the bottom of the slope and vegetation reinforcement at within the slope profile.

"Computer-Aided Slope Stability Analysis of a Landslide - A Case Study of Jhika Gali Landslide in Pakistan" by M.N. Amin, M.U. Ashfaq, H. Mujtaba, S. Ehsan, K. Khan and M.I. Faraz [47], investigates the stability of a landslide by performing computer-aided slope stability analysis. Data concerning the geological and geotechnical properties of the slope, soil composition, slope geometry and groundwater conditions was used to conduct the analysis. Based on this data a numerical approach was developed by applying LEM and GeoStudio software. The LEM methods used were the Bishop method and the Spencer method, which consider factors such as soil shear strength and slope geometry to evaluate the safety factor.

The findings pointed out that the landslide area was critically unstable because this safety factors proved to be below the threshold for stable slopes. Further, the analysis identified the potential failure surfaces along the slope. The researchers propose various appropriate measures to stabilize the slope to reduce the risk of future landslides. These measures include slope reinforcement approaches such as soil nailing, retaining walls, and drainage systems to control groundwater. The study emphasizes the effectiveness of digitally supported slope stability analyses in assessing landslide risk and identifying suitable measures for improving the stability.

In "Geographic Information Systems-Based Three-Dimensional Critical Slope Stability Analysis and Landslide Hazard Assessment" M. Xie, T. Esaki, G. Zhou and Y. Mitani [48], applied geographical information systems (GIS) and a hydrologic analysis and modelling tool with a column-based 3D slope stability analysis model, to develop a new GIS grid-based 3D deterministic model for slope stability analysis. The study seeks to apply GIS technology to improve the understanding of slope stability and assess the potential risk of landslides. Firstly, the authors collected various data such as topographic data, geological information, soil properties and rainfall data. Further the data was implemented into a GIS platform, in order to establish a 3D model of the slope and to conduct stability analysis. The analysis was performed by applying numerical methods such as limit equilibrium method (LEM) and the finite element method (FEM) to further evaluate the safety factor and identify potential zones of failure within the slope.

The findings of this study yielded significant contributions to the understanding of critical slope stability and landslide hazard within the study area. The improvement of a 3D GIS model facilitated a comprehensive assessment of slope behavior, revealing areas susceptible to potential instability. Furthermore, the analysis clarified the complex relationship between rainfall patterns and landslide occurrences, thereby facilitating the identification of zones with heightened risk.
The authors argue that the implementation of various data combined with 3D visualization of the model improves the precision and efficiency of the analysis.

P. Goeroeg and A. Toeroek [49], conducted a case study in Budapest focusing on the slope stability assessment of weathered clay. This was performed by a combination of field data and computer analysis. The researchers used data from field investigations, soil samples, laboratory tests and in-situ measurements. In the study numerical methods such as FEM and LEM were applied to compute soil behavior while the slope geometry, soil properties and boundary conditions was analyzed by using software's as Geo4 and Plaxis. These software applications enabled the evaluation of the slope stability and the determination of the safety factor. The findings indicates that the stability is affected by factors such as soil properties, groundwater, and slope geometry. FEM and LEM offered insights into the locations of critical failure surfaces and identified potential failure mechanisms within the slopes. By determining the safety factor, areas of potential instability were identified facilitating the necessity for slope stability measures. Based on the findings, the article emphasizes the significance of conducting thorough geotechnical investigations, including soil sampling and laboratory testing, to obtain precise data for software analyses. Applying field data in software analyses improves the comprehensive understanding of slope behavior and facilitates knowledge-based decisions regarding slope stability.

Rotaru, F. Bejan and D. Almohamad [50], conducted a study which seeks to explore the suitability and limitations of various methods applied in slope stability analysis from a sustainability perspective. Based on this, the aim of the study is to establish a new approach for determining the safety factor, the shape, and the centre of the critical slip surface (CSS), providing an improved estimation of slope probability of failure, which can represent a remarkable component in a more accurate risk assessment. The researchers performed a comprehensive review of previous literature and studies on slope stability analysis methods, which includes identifying and analyzing common methods such as LEM (Fellenius, Bishop, Janbu, Morgenstern-Price and Spencer), FEM (shear strength reduction, SSR and Upper-Bond limit UBL). These methods were evaluated based on their applicability, accuracy, computational efficiency, and ability to incorporate sustainability factors.

The results from the five LEM were performed by using Slide Rocscience Program and compared with the FEM results conducted by applying Plaxis software. The researchers argue for importance of including sustainability factors, such as environmental impact, social aspects, and economic feasibility, in addition to traditional engineering factors. The study proposes that a comprehensive framework incorporating both technical and sustainability dimensions is necessary for achieving sustainable slope stability analysis. Furthermore, the study emphasizes the essential of improving the combination of data derived from varied disciplines, including geotechnical engineering, hydrology, ecology, and socioeconomics, to facilitate a comprehensive and sustainable assessment of slope stability. The importance of interdisciplinary collaboration and application of methodologies such as geographic information systems (GIS) and remote sensing techniques for seamless data integration and analysis is also underscored.

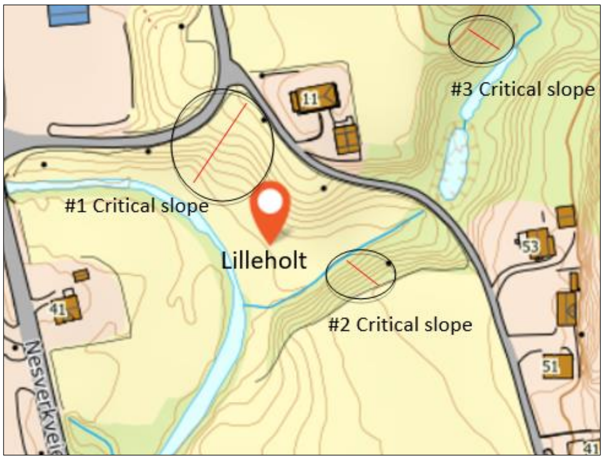
In conclusion, the study highlights the need for a more sustainable approach to slope stability analysis. By critically evaluating existing methods and proposing a framework for sustainable analysis, the research provides valuable insights for researchers and practitioners involved in slope stability assessments. The results from the FEM analysis pointed at slope geometry, mesh size and density as parameters affecting the safety factor. When comparing the results from all methods, there was a close agreement that the Safety factors was closely connected to the height of the slope, when the height decreased the safety factor increased.

7.2 Assessment of area stability

7.2.1 Identification of critical slopes

Figure 7.1 present identified critical slopes

Figure 7.1: Overview of identified critical slopes [44]



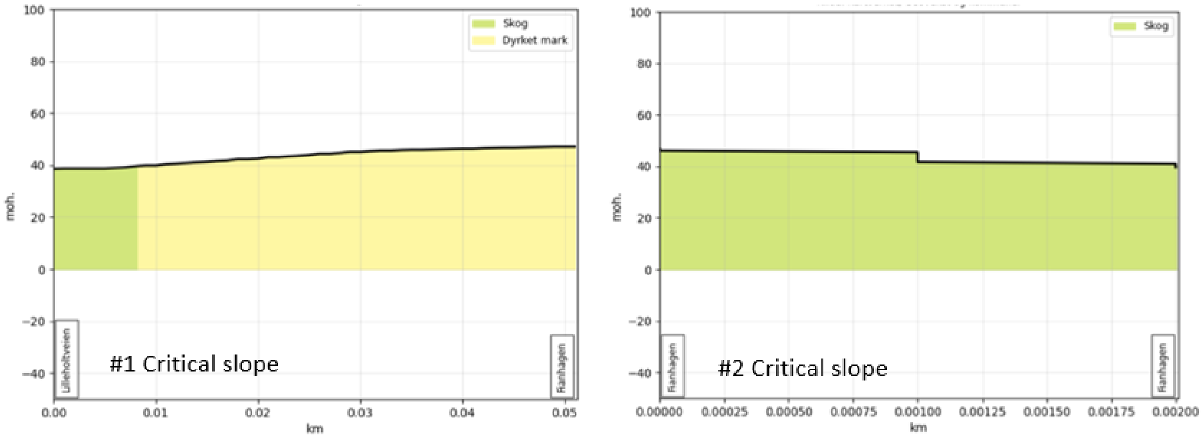
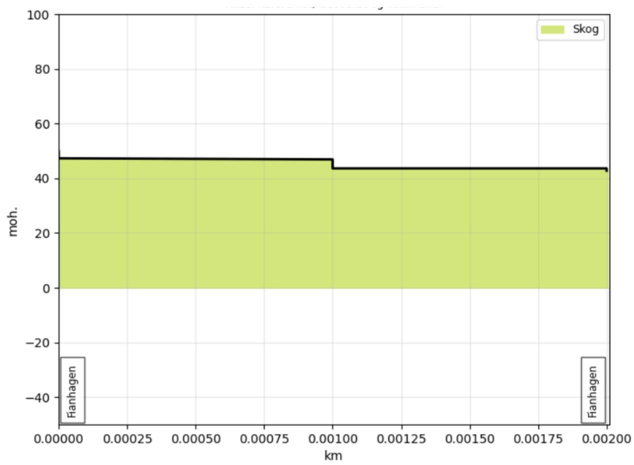


Figure 7.2: #1-3 Height difference in the critical slopes [44]



7.2.2 Stratification

Figure 7.3 and Figure 7.4 illustrates assessment of stratification of borehole 1 and 2. Based on curves proved in Appendix B.

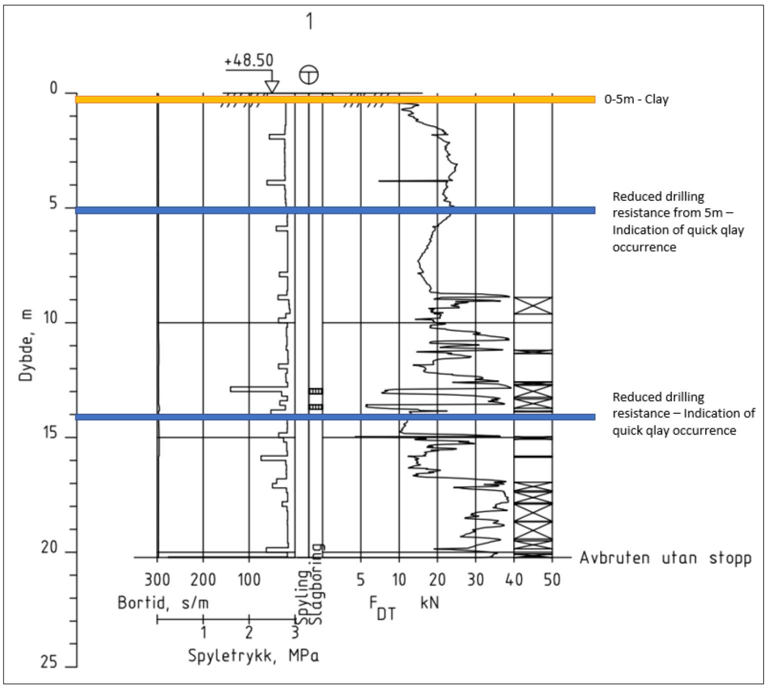


Figure 7.3: Stratification for borehole 1

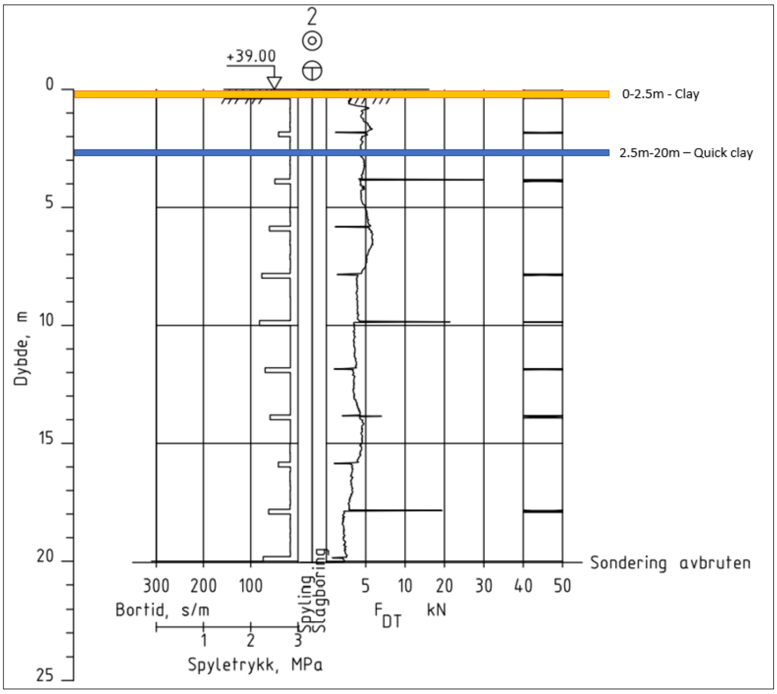


Figure 7.4: Stratification for borehole 2

7.2.3 Quality and class

K2 is chosen as measure category for the case-area based on the method described in chapter 6.2.5. The sample quality was assessed to be of high quality based on the curve shape of the triaxial test. The data for the void ratio further confirms this assessment.

7.2.4 Parameters of effective stress analysis

Table 7.2 presents interpreted parameters for effective stress analysis in GeoSuite. Based on chapter 6.2.4.

Table 7.2: Interpreted parameters for effective stress analysis

Depth [m]	Attraction a [kPa]	Angle of friction φ [°]	Shear strength C`[kPa]
4,5	4,0	41,8	3,6
9,4	8,0	34,8	5,6

7.2.5 Interpretation of shear strength

Table 7.3 presents assumed groundwater level.

Table 7.3: GW-level

GW-level	
Depth [m]	1,00

Table 7.4 presents interpreted active undrained shear strength C_{uc} and axial deformation ε_a.

Table 7.4: Interpretation of shear strength C_{uc} and axial deformation ε_a

Depth [m]	Shear strength C_{uc}[kPa]	Axial deformation ε_a [%]
4,5	32	0,8
9,5	42	1,4

Table 7.5 presents the mean density value ρ of the quick clay layer and the sub-merged values ρ` for all layers, based on Table 6.6.

Table 7.5: Mean and sub-merged value of density

Layer	Density ρ [kN/m³]	Sub-merged ρ`[kN/m³]
Clay		7,4
Quick clay	16,4	6,4
Sand		7,0

7.2.6 Loads and ADP-factors

Table 7.6 presents the plasticity index. Based on content in chapter 6.2.8 and Table 6.9.

Table 7.6: Plasticity index I_p

Depth [m]	I_p [%]
4,5	7,7
9,4	17,4

In Table 7.7, calculated characteristic traffic load F_d and ADP-factors are presented. Based on chapter 6.2.8.

Table 7.7: Traffic load F_d and ADP-factors

F_d [kPa]	Layer	Aa [-]	Ad [-]	Ap [-]
19,5	Clay	1,00	0,63	0,35
	Quick clay	1,00	0,66	0,38
	Sand	1,00	1,00	1,00

Figure 7.5 illustrates the recommended $\mathrm{C_uA}$ profile and. Based on tables in chapter 6.2.7 and 7.2.5.

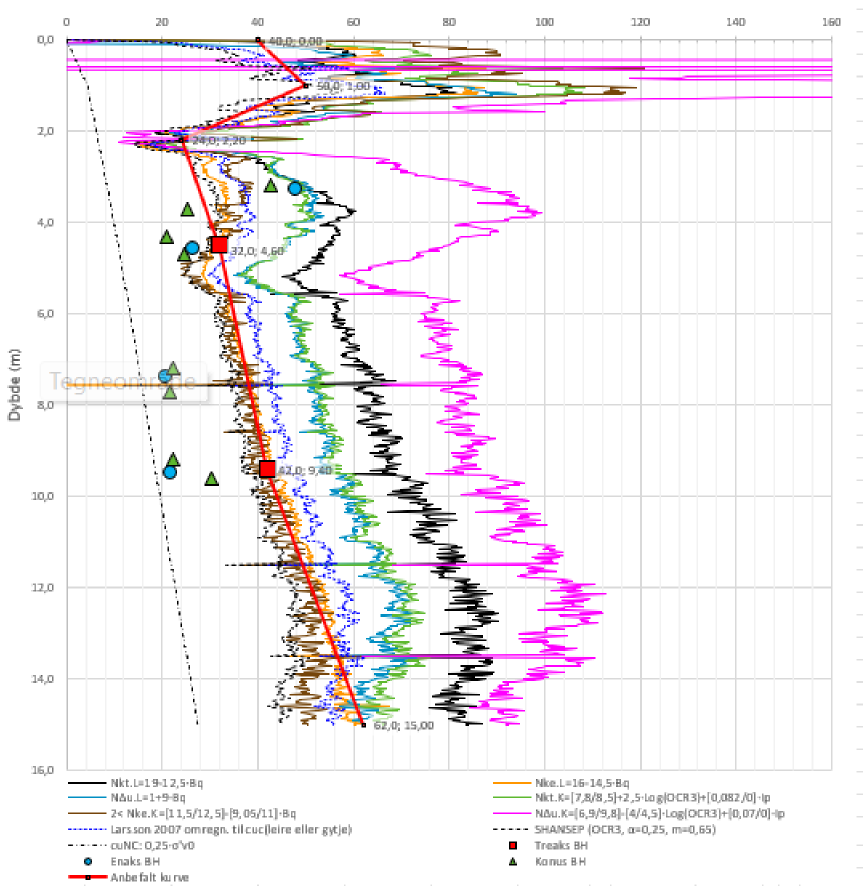


Figure 7.5: Recommended C_uA profile

Table 7.8 presents the input data for the recommended C_uA profile in GeoSuite.

Table 7.8: Input data for recommended C_uA profile

Active undrained shear strength profile C_uA	
Depth Z [m]	Undrained shear strength C [kPa]
0,00	40,00
1,00	50,00
2,20	24,00
4,60	32,00
9,40	42,00
15,00	62,00

7.2.7 Analysis in GeoSuite

By applying the process described in this thesis for assessment of area stability, safety factor F_s expressed by total F_c (undrained) and effective $F_{c\varphi}$ (drained) stress analysis was identified. Table 7.9 present 8 analyses in drained conditions and 4 in undrained conditions.

Table 7.9: Identified safety factors through $F_{c\varphi}$ and F_c analyses

Profile	Situation	Condition	Safety factor	Notes
1A	Current situation	Drained/undrained	1,84/1,21	With interpreted values of φ
	Current situation	Drained	1,29	With empirical values of φ
	Current situation	Drained/undrained	2,09/1,21	With interpreted values of φ & adjusted GW
	Current situation	Drained	1,48	With empirical values of φ & adjusted GW
	Improved river profile*	Drained/undrained	1,87/1,19	With interpreted values of φ
	Improved river profile*	Drained	1,32	With empirical values of φ
	Improved river profile*	Drained/undrained	1,99/1,19	With interpreted values of φ and adjusted GW-level
	Improved river profile*	Drained	1,40	With empirical values of φ and adjusted GW-level

*River profile with slope inclination 1:3 & 1:2

8 Discussion

In this chapter the results will be discussed, and potential weaknesses and limitations will be highlighted.

8.1 Assessment of area stability

8.1.1 Identification of critical slopes

The critical slopes that were mapped and presented in the results chapter are of minor significance in this study, particularly in relation to the stability analyses in GeoSuite. It was found that the area consisted only of one large critical slope. However, it has been important as part of the site assessment to attempt to identify critical slopes as described in the NVE Guideline 1/2019. This is a common practice to perform before conducting stability calculations, as the identified critical slopes are typically plotted in Novapoint for use in stability analyses.

8.1.2 Stratification

Assessing the layering can be somewhat imprecise due to a lack of prior knowledge. To achieve reasonably accurate layering, some guidance from the client was necessary. In cases where the drilling resistance was significantly reduced, it was assumed to be quick clay, while an alternating curve could indicate frictional material.

8.1.3 Parameters of effective stress analysis

The interpreted values of the friction angle φ from the triaxial test were slightly high, as presented in Table 7.2, compared to empirical values. This could be due to errors in the interpretation process of cohesion or in the calculations of the angle itself. Another possible source of error could be a high degree of overconsolidation (OCR) when stresses were applied during the triaxial test.

8.1.4 Interpretation of shear strength

There were significant uncertainties associated with assessing the groundwater level for both stability analyses and the Excel CPTu interpretation sheet. Since no data were available, the groundwater level had to be assumed. Based on the results of the stability analyses in GeoSuite, it can be argued that the groundwater level may not have a significant impact on the results, as the area is relatively small and the measures to be implemented were minor, which did not significantly affect the results. The density γ of the quick clay layer was based on an average value representative of the entire layer of quick clay. The difference between the highest and lowest values was not significantly large, allowing for the assumption that the calculated average value was representative enough for the entire quick clay layer.

8.1.5 Analysis in GeoSuite

The availability of groundwater level information in the provided data from the ground investigations conducted by the client was limited. This was due to the absence of pore pressure measurements during the initial investigations. As a result, the depth of the groundwater level had to be assumed by the author in consultation with the client, both in the excel CPTu interpretation sheet and in GeoSuite.

To assess the significance of this limitation, the author performed several analyses in GeoSuite with adjusted groundwater levels to observe the impact on the factor of safety. Specifically, the groundwater level beneath the critical slope was adjusted from 1m below ground level to 2m. It should be noted that if the selected study area had been larger, it would have likely involved conducting more CPTu tests than the single test performed for borehole 2. Considering the cost implications of conducting ground investigations and the relatively small size of the area with minor measures, the results from borehole 2 were used as a reference for both boreholes. However, this approach may have provided a relatively limited basis for the calculations. If the area had been larger with significant measures, it would likely have required additional CPTu tests. In such cases, the project would have fallen into a higher category than K2, potentially necessitating a more rigorous zone assessment as per NVE guideline 1/2019, which imposes stricter requirements than K2.

The results of the analyses in GeoSuite demonstrate that the factor of safety in undrained conditions (total stress analyses) remains almost identical in all cases, even after adjusting friction angle and groundwater level. The first two analyses of the current situation in drained conditions (effective stress analyses) using interpreted values for friction angle and a GW level of 1m below ground level, compared to using empirical values for friction angle and the same GW level, showed a significantly higher factor of safety when using the interpreted values. This difference may be attributed to the initially much higher interpreted values for the friction angle from the triaxial test compared to the empirical values, with a difference of 0.55.

The next two analyses of the current situation were performed in the same manner as the previous ones, except that the GW level was adjusted to a depth of 2m below ground level. The results showed a similar trend to the earlier analyses, with the interpreted values for friction angle and the adjusted GW level resulting in a substantially higher factor of safety compared to the empirical values and the adjusted GW level. Here, the difference was 0.61. After drawing a new river profile, four additional effective stress analyses were conducted. The first two were performed using interpreted and empirical values for friction angle, respectively. The results exhibited the same trend as the preceding analyses, indicating a 0.55 higher factor of safety when using interpreted friction angles. The last two analyses followed the same approach as the previous ones, except that the GW level was adjusted to 2m below ground level. The results remained consistent with the preceding analyses, with a higher factor of safety favoring the interpreted friction angles. The difference observed was 0.59.

In summary, all the analyses consistently demonstrate that the interpreted values yield a higher factor of safety compared to the empirical values, regardless of whether the GW level is adjusted or not. The difference ranges from 0.55 (minimum) to 0.61 (maximum). This suggests that the GW level has minimal influence on the factor of safety in these analyses. In the study conducted by P. Goeroeg and A. Toeroek chapter 7.1, the role of groundwater (GW) as a contributing factor to stability was identified. Consequently, one can argue that in the specific case of the Lilleholt area, characterized by its small size and limited scope of implemented measures, the changes in GW level may not significantly impact the safety factor. However, the obtained results suggest that the friction angle plays a substantial role in determining the factor of safety. This can be attributed to the application of the highest interpreted friction angle of 41.8 degrees in the analyses, in contrast to the interpreted value of 26 degrees. It is worth noting that the interpreted data derived from the triaxial test is typically considered as the most reliable, instead of relying only on empirical values. The thorough assessment of the triaxial test data, including the examination of void ratio variations and classification of sample quality as high, further supports the use of interpreted friction angle values as the reference when presenting the main findings of the study.

8.1.6 Previous research

In the study conducted by F. Göktepe and I. Keskin, numerical methods such as FEM (Finite Element Method) and LEM (Limit Equilibrium Method) were applied, along with software programs such as Plaxis, Geslope, and Slide, to calculate the safety factor F_s. Their results consistently showed similar safety factors in all the analyses. Additionally, they found that the slope angle played a crucial role in stability analyses. The researchers emphasized that as the slope angle increases, the shear stress in the soil or other unconsolidated materials also tends to increase. By using FEM in their analyses, the authors gained a better understanding of the deformation characteristics of the soil. This advantage is not provided by GS stability, which relies on LEM and applies force equilibrium, BEAST, simplified Bishop, and modified Bishop methods.

In the GS analyses conducted by the author of this this master`s thesis, the slope angle was not considered, as GS did not require this factor for performing the analyses. All the software programs incorporate calculation models within their platforms. However, the number of models included may vary depending on the software and the approach it is based on, whether it is LEM, FEM, or other methods. The results of their study suggest that all the programs can be used for conducting stability analyses. Overall, the study highlights the effectiveness of numerical methods and software programs in calculating the safety factor. The consideration of slope angle and the availability of different calculation models in the software programs contribute to a comprehensive analysis of slope stability.

In their study, P. Goeroeg and A. Toeroek aimed to compare the results of slope stability analysis using two different software applications: Plaxis, which utilizes integrated FEM modelling, and Geo4, which employs conventional calculation methods such as Bishops. The objective was to assess the similarities and differences between these two approaches. Interestingly, the findings of the study revealed that the results obtained from both software applications were almost similar, despite some variations. Notably, the Plaxis software yielded a more conservative safety factor compared to Geo4. This outcome raises captivating questions and justifies further discussion. Plaxis, being a widely used FEM software, offers the advantage of modeling complex slope morphology and identifying the weakest point of the slope. However, it does not always generate a geologically feasible slip surface automatically.

On the other hand, Geo4 relies on conventional methods where the slip surface is typically manually entered. The fact that the results from Geo4 were comparable to Plaxis implies that conventional methods may still have relevance and effectiveness in assessing slope stability, even when dealing with complex slope geometries. The study highlights the importance of considering both software applications and conventional methods in slope stability analysis. While Plaxis provides advanced capabilities, the findings suggest that the conventional methods employed by Geo4 can yield reliable results and be as effective as software that allows for more complex geometry. This discussion opens possibilities for further research and emphasizes the need to carefully evaluate different approaches to slope stability analysis, considering both the accuracy of the results and the feasibility of the slip surface identified.

In the study conducted by M. Xie, T. Esaki, G. Zhou, and Y. Mitani, a different approach was employed for conducting stability analyses. By combining numerical models such as FEM and LEM with a GIS grid-based 3D deterministic model, the authors claimed that this approach increased the precision and efficiency of the analyses. This methodology differed from the methods used in other articles identified. The authors discovered that this 3D approach not only provided a better understanding of soil behavior but also identified unstable areas.

The combination of 3D modeling with LEM and FEM is not widely used but the results of their study indicate that this method could be relevant in the future and in other countries, including Norway. This is particularly significant since Norway has many areas below the marine limit and is prone to quick clay occurrences. Implementing such a 3D approach in Norway would be advantageous as it is assumed to be effective and accurate while providing detailed information about hazardous areas and soil behavior. Although 3D-based methods for stability calculations exist in Norway today, it is uncertain whether the method developed in this study is currently being utilized in the country. The current debate revolves around the utility of performing 3D-based stability analyses. Geotechnicians have raised concerns about the potential drawbacks of employing 3D tools in certain situations. One prominent concern is that applying these tools may result in an overestimation of the safety factor F_s, which could misleadingly indicate that terrain intervention is unnecessary when, in fact, it is required.

In the study conducted by A. Rotaru, F. Bejan, and D. Almohamad, density was identified as a significant factor influencing the safety factor in their stability analysis. However, in the GS stability analysis performed by the author, density values were not adjusted. It would have been beneficial to investigate the potential impact of adjusting the density values on the safety factor and determine if any notable changes would occur. By examining the effects of density adjustments, further insights could have been gained into the stability characteristics of the analyzed system.

8.2 Critique of the method

8.2.1 Literature review

Unstructured searches

The “snowball method”

One drawback of this method is its retrospective nature, whereby each subsequent source found is likely to be older than the preceding one.

Structured searches

Online databases

If general searches are performed through Google Scholar, a disadvantage might be that the sources can vary a lot in quality. Variation between high quality high peer-review articles to conference paper or unpublished work of lower quality. Another limitation is that it does not have any function for selecting only relevant hits and discard. By limiting the choice of databases to Google Scholar and

Oria, useful information may have been missed. The search for relevant research literature in accordance with the task was challenging due to the limited availability of studies addressing the specific issue of quick clay in selected countries. Some of the articles that were identified had a notable absence of cross-references, despite being selected based on the literature search method. Initially, the search process included the criterion that identified articles should have at least 20 cross-references. However, this criterion had to be dropped as relevant literature matching the thesis requirements was not identified. Articles with few cross-references typically provide a significantly weaker foundation in terms of credibility compared to articles with a higher number of cross-references. Cross-referencing plays a crucial role in academic research as it allows for the integration of diverse perspectives, verification of claims, and validation of findings through the support of existing literature.

8.2.2 Assessment of area stability

Identification of critical slopes

Critical slopes were drawn by applying Norgeskart and should have been applied to draw terrain profiles for implementation in Novapoint GS. In lack of time this was skipped, and client assisted with these profiles.

Interpretation of shear strength

The interpretation of oedometer tests should have been conducted. This is a common practice to perform in combination with interpretations from triaxial tests to provide a more comprehensive basis for interpretation prior to conducting stability analyses. However, due to the scope of the thesis, this step had to be omitted.

Stability analyses in GeoSuite (GS)

The author of this thesis acknowledges that prior to undertaking this research, he did not have the necessary knowledge and expertise in conducting an area stability assessment or applying data software like GeoSuite for stability analysis. However, recognizing the significance of these skills in achieving the research question, the author actively tried to acquire the required knowledge through various means. Collaboration with the client played a crucial role in covering the knowledge gap. Working closely with the client, the author gained insights into the practical aspects of performing an area stability assessment. The client's expertise and guidance provided valuable insights and direction throughout the learning process. Additionally, the author took the initiative to improve their understanding by extensively using available resources such as Novapoint GeoSuite's own tutorials and supplementary videos and tutorials accessible on the internet. These resources offered guidance and explanations on how to effectively apply the software for stability analysis.
To conduct stability analysis in GeoSuite, undrained shear strength values are typically selected and interpreted based on practical experience. However, as the author lacked such experience, the chosen values may be unfavorable, potentially constituting a weakness, even though the resulting factor meets the necessary requirements according to Table 3.1. The safety factor $\mathrm{F_s}$ should also have been calculated manually in order to validate the findings in the stability calculations. This may have given greater feedback if the chosen soil parameters were favorable or not. Even though the value of the calculated safety factor itself may confirm that the parameters used are reasonable or not if compared with the requirements.

9 Conclusion

In this thesis, it has been desirable to investigate how to secure areas with quick clay properties in the ground, in order to avoid triggering area landslides. Literature has been used as qualitive method and findings and theories have been presented related to this method. In addition, it has also been desirable to gain a better understanding of the theoretical described solutions and solutions in practical context. To solve this, two sub-questions have been a part of the thesis from the initial phase. Therefore, these questions will be answered first, followed by the answer to the research question.

9.1 Main findings

The findings in this thesis are separable based on both literature review and practical analyses.

Stability analysis in GeoSuite has made a significant contribution in addressing the first sub-question:

What is the current area safety factor at Strengselva and how does this differ after appropriate measures?

- In total stress analysis (undrained) the current safety factor is 1,21.
- In total stress analysis (undrained) after appropriate measures the factor is 1,19.
- In effective stress analysis (drained) the current safety factor is 1,84.
- In effective stress analysis (drained) after appropriate measures the factor is 1,87.

The literature review has contributed to answer the second sub-question:

Which analytical and numerical methods are used for slope stability, and could these be relevant in Norway?

- *Plaxis – a finite element software (FEM)*
- *GeoStudio softwares SEEP/W & SLOPE/W*
- *Geo4 – a finite element software (FEM)*
- *Slide – 2D based with LEM*
- GIS grid-based 3D deterministic model – with Limit Equilibrium Method (LEM) and FEM
- Shear Strength Reduction (SSR) with Mohr-Coulomb Criterion
- All methods are considered relevant for slope stability analysis in Norway

The combination of theoretical and practical approach in this thesis contributed to answer the research question:

***"How to perform a geotechnical area stability assessment of Strengselva in accordance with NVE veileder 1/2019 Sikkerhet mot kvikkleireskred*?"**

- Identify critical slopes within the planning area
- Assess soil layers and their properties
- Determine the sample quality and measure category
- Interpret and identify effective stress parameters
- Define GW-level and interpret shear strength
- Compute loads and determine ADP-factors
- Perform stability analysis in GeoSuite or other suitable softwares
- Control achieved safety factor against requirement in NVE 1/2019

10 Recommendations

It would be beneficial to validate the findings of this study by conducting manual calculations to determine the factor of safety in the area. Additionally, it would be interesting to further explore a more realistic and comprehensive assessment of site stability by engaging in practical field investigations and laboratory testing, aiming to gain an expanded understanding of the interplay between various aspects related to geotechnical investigations. In this study, the GeoSuite software program was applied for stability analyses. Therefore, it would be valuable to employ alternative software to both validate the findings and assess stability using different approaches.

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12 Appendix

Appendix A - Strengselva_Tiltaksbeskrivelse
Appendix B - Geoteknisk datarapport
Appendix C - FV.421 Borplaner_Strengselva_Tvedestrand
Appendix D - Rådata grunnundersøkelser
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