Eulerian Based Large Deformation Finite Element Modeling of Strain Softening Clay Slopes

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Abstract

Large scale landslides in quick and sensitive clays have the tendency toward retrogressive failure, sometimes with catastrophic consequences, such as in the 2020 Gjerdrum landslide. Modeling retrogressive slope failure using traditional modeling techniques is challenging due to problems arising with large deformation and the strain softening nature of sensitive and quick clays. The coupled Eulerian-Lagrangian (CEL) finite element (FE) modeling technique addresses many of these shortcomings. A method for implementing the CEL technique for analyzing slope stability in the commercially available software package Abaqus is presented. Results using this technique are compared to recent studies, showing similarities in observed failure mechanisms. Profiles from the Gjerdrum landslide are analyzed. Simulation results of the Gjerdrum landslide show failure patterns recognized in quick and sensitive clay landslides although correlation to post slide investigations is weak.
Preface

Since learning about quick clay landslides after moving to Norway, I have been fascinated by their processes and very unique mechanics. I am grateful to the Department of Geosciences at the University of Oslo for the opportunity to study this subject in greater detail. I would like to thank my advisors for their support: Karen Mair and Olivier Galland from the University of Oslo, Amanda Johansen DiBiagio, Håkon Heyerdahl, and Hans Petter Jostad from NGI have all provided valuable support throughout the process. I would like to also thank Khoa D.V. Huynh and Kjetil Bakke for their technical help with the Abaqus software package, it has proven to be much more involved than one would expect. Finally I would like to thank my wife Vilde Aall Rosendahl for her patience over the past years.
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1 Introduction

To mitigate the risks associated with landslides in sensitive clays, researchers and engineers are actively studying the behavior of these soils and developing methods for identifying areas at risk of failure. Sensitive and quick clays are known for their complex behavior such as strain-softening and retrogressive failure geometry, complicating traditional modeling methods. Due to the strain softening properties of quick clay the traditional methods for assessing slope stability have significant shortcomings when analyzing slopes where retrogressive failure occurs. Limit equilibrium methods assume soil behaves as a perfectly plastic material, not accounting for softening, and finite element (FE) methods using a purely Lagrangian framework can be quite good at modeling the initiation but struggle to handle large deformation as a landslide mobilizes. A recent focus has been on using the coupled Eulerian-Lagrangian (CEL) finite element (FE) modeling technique to address many of the shortcomings of these traditional methods. Researchers have been using the CEL technique in the Abaqus FE software to model landslides in sensitive clays, investigating various factors affecting the stability of slopes, such as soil properties, geometry, and water content. With the studies focused on improving accuracy and efficiency of the CEL technique, they do not give a easily understandable method for creating models in the commercially available software. Furthermore, most of the current research requires additional coding to run simulations. The main goal of the thesis presented here is (1) to provide a method for implementing the CEL technique for analyzing slope stability using the commercially available Abaqus software package, without the need for coding additional subroutines, (2) compare results of this technique with recently published studies of sensitive clay slope simulations and (3) simulate slope failures of the Gjerdrum landslide comparing simulation results with post slide investigations.

1.1 Current research

Recent studies have explored many applications of the CEL finite element approach to large deformation problems involving sensitive soils. Studies investigating sensitive clay landslides in Canada (Dey et al., 2014, 2015; Saha et al., 2022; Wang & Hawlader, 2017; Wang et al., 2016; Wang et al., 2022; Wang et al., 2015) using the CEL FE approach provide a foundation
for implementation in the Abaqus finite element software. The CEL finite element approach using Abaqus was first presented by Dey et al. (2013), with following studies primarily focused on understanding failures in sensitive clay slopes using idealized models. A parametric study modeling a sensitive clay slope with simple geometry (Dey et al., 2015) show that depending upon geometry and soil properties, toe erosion could cause three types of shear band formation; (a) only a horizontal shear band without any global failure, (b) global failure of only one block of soil, (c) global failure of multiple blocks of soil in the form of horsts and grabens. FE simulations of three major types of large-scale landslides in sensitive clay (Wang & Hawlader, 2017) attempt to explain the initiation of failure, formation of shear bands and global failure of the slopes using an idealized model for each case. More recent studies (Saha et al., 2022; Wang et al., 2022) explore the effects of undrained shear strength profiles, slope geometry and soil properties on retrogressive failures in sensitive clays. Saha et al. (2022), examines the effects of the undrained shear strength and softening rate in progressive failures using FE simulations for an idealized model with two sensitive clay layers. Conclusions from the study suggest a slower rate of soil softening below the toe level could cause shallower failure of soil blocks with larger retrogression distance and a relatively higher initial undrained shear strength of the soil in the lower clay layer could increase retrogression distance, noting the effects of softening appear to be more influential than shear strength increase on retrogression distance. Wang et al. (2022) identify conditions leading to flow slides or spreads, examining strain-softening and strain-rate effects on the undrained shear strength of sensitive clays using idealized simple slope models. Although the majority of CEL FE studies use idealized models, Wang and Hawlader (2016), employed the same approach to model the 2010 Saing Jude landslide in Quebec successfully modeling shear band formation closely matching results from post slide geotechnical investigations.

1.2 Quick Clay Overview

Quick clay is a marine clay that can be found in the northern hemisphere at latitudes above the extent of the last glacial maximum. Due to isostatic land uplift following glacier retreat, large amounts of marine clay rose above sea level and can be found at elevations up to 220 meters above sea level (Ramberg et al., 2013). In Scandinavia some of the most ideal land for living and farming sits on these marine clay deposits (NGI, 1978). As fresh ground water
leaches through the marine clay from the surface and fissures in the bedrock, salt concentration in the marine clay is reduced (Figure 1).

![Figure 1: Principle diagram of freshwater leaching through marine sediment with salt pore water](Quick Clay Landslides: Slippery Subject, 2021)

In Norway, Quick clay is defined as having an undrained remolded shear strength lower than 0.5 kPa. (Janbu, 1970). The sensitivity values, a material property utilized to define the ratio between the undisturbed and the remolded shear strength of a soil material in quick clay are often higher than 16. Scandinavian marine clays often become quick clays when the pore water concentration dips below 2 g/l (Torrance, 1979). The Norwegian Water Resources and Energy Directorate (NVE) publishes interactive maps of known quick clay deposits in Norway. Figure 2 shows areas in Norway where quick clay hazards have been mapped. Mapped quick clay deposits north of Oslo, according to risk classes, can be seen in Figure 3.
Figure 2: Mapped quick clay areas in Norway, (NVE Atlas, 2023).
Figure 3: Quick clay zones to the north east of Oslo, mapped according to risk (NVE Atlas, 2023), with risk class 5 representing the highest danger.

1.3 Gjerdrum Landslide

The Gjerdrum 2020 quick clay landslide occurred on December 30, 2020, in the village of Ask in Gjerdrum, Norway. The landslide occurred slightly before 04:00 in the morning, killing 11 people including an unborn child, causing more than 1600 people to be evacuated and resulting in extensive property damage (Ekspertutvalg, 2021). Ask is approximately 25 km northeast of Oslo (Figure 4), with an population of under 7000 people.
The landslide occurred in multiple phases (Figure 5) with erosion from the Tistil stream assumed to cause the initial failure in Phase 1 of the landslide (Ekspertutvalg, 2021). This erosion lowered the streambed, Figure 6, which, combined with other factors resulted in the slope becoming unstable. As the quick clay liquefied it flowed downstream, leaving slopes to fail retrogressively in the following phases.
Figure 5: Overview of the estimated main phases of the landslide, with 0 being the estimated initiation location, (Ekspertutvalg, 2021).
Figure 6: Examples of two cross profiles in Tistil stream showing the terrain in 2007, 2015 and 2020. Erosion between 2007 and 2015 has led to the stream being lowered by up to 2.5 meters, with erosion between 2015 and 2020 mainly occurring laterally (Ekspertutvalg, 2021).
2 Theoretical Background

2.1 Slope Stability Concepts

In geotechnical engineering, slope stability refers to the ability of a slope to withstand external forces such as, water, and earthquakes without undergoing failure or collapse. There are several key concepts used in slope stability analysis.

2.1.1 Landslide and Failure Types

Several main types of slope failure relevant to geotechnical engineers are shown in Figure 7:

- Rotational: Rotational failure occurs when a section of the slope rotates about a pivot point. These failures can be circular as well as non-circular.

- Translational: Also known as a slide failure, occurs when a section of the slope moves along a planar surface.

- Complex failure: A complex failure is a combination of two or more types of failures. A compound slip failure is a combination of a circular failure and a translational failure.

Figure 7: Types of slope failure, figure taken from Craig (2004).
Slope failures are typically triggered by factors such as erosion of the slope, changes in soil properties, Human activity, weathering, and changes in water content or ground water pressure. These types are not mutually exclusive, one failure can lead to another, creating a complex failure. Knowing the failure type it is possible to calculate the Factor of Safety of a slope. The Factor of Safety (FoS) is the ratio of the available shear strength to the applied shear stress, generally calculated by comparing all the driving forces and resisting forces present. A FoS of greater than 1 indicates that the slope is stable, while a FoS less than 1 indicates an unstable slope (Terzaghi, 1943).

The two primary landslide types observed in sensitive and quick clays are flowslides and spread landslides. A spread landslide is a type of slope failure where the soil or rock mass moves along a relatively planar surface. In a spread landslide, the movement of the material is dominated by translational or rotational sliding, and typically does not deform significantly during the movement. Flowslides are a type of landslide where the soil moves as a viscous fluid. In a flowslide, the material deforms significantly during the movement, and the flow behavior is typically controlled by the material properties and the slope geometry.

FE simulations from Wang et al. (2022) examining the effects of geometry and soil properties on type and retrogression of landslides in sensitive clays illustrate progressive failure in a typical flowslide and progressive failure in typical spread, Figure 9 and Figure 10. Figure 9 (c) shows the failure surface of a sensitive clay flowslide to be successive circular failures. A more angular failure surface producing the horst and graben structure characterizes a spread
2.1.2 Limit Equilibrium Methods (LEM)

Limit equilibrium methods are based on the assumption that the failure surface of a slope can be divided into a series of wedge-shaped blocks, which are assumed to be in equilibrium under the influence of various forces and moments acting on them. There are several variations of the limit equilibrium method used to analyze slope stability, including, Bishop's...
method (Bishop, 1955), Janbu's method (Janbu, 1954), Spencer's method (Spencer, 1967) and the Morgenstern and Price's method (Morgenstern & Price, 1965). A variety of slope stability analysis software that can be used to perform LEM analyses such as Slope 2, SLOPE/W, Rocscience, and GEO5.

2.2 Geotechnical Investigation methods

2.2.1 CPTu Soundings

The Cone Penetration Test with pore pressure measurements (CPTu) is a widely used geotechnical investigation method providing detailed information about the strength of soil. A specially designed cone-shaped probe is driven into the soil at a constant rate using a hydraulic ram. As the cone penetrates the soil, it measures the resistance of the soil to the penetration, providing information about the soil's strength and stiffness. In addition to the cone resistance, the CPTu test also measures pore water pressure. Pore water pressure is measured using a pressure transducer located behind the cone tip. This measurement provides information about the water content and the permeability of the soil. Local friction on the probe is measured using a friction sleeve located behind the pressure transducer. This measurement can also provide information about the soil's shear strength.

2.2.2 Triaxial Tests

The triaxial testing method is used to examine the stress-strain behavior of soil under different loading conditions. A cylindrical soil specimen is placed in a cell with a permeable membrane at the top and bottom. The cell is then filled with fluid to simulate the in-situ stress conditions. The sample is subjected to a confining pressure applied to the cell, an axial load applied to the top of the sample, and a back pressure applied to the fluid in the cell. The confining pressure is applied to the sample from all sides to simulate in-situ stresses the soil would experience in the ground. The axial load is applied to the top of the sample to induce failure. Back pressure is used to maintain a constant volume of the fluid in the cell and to measure pore water pressure. The axial load is increased gradually until the sample fails. Several types of triaxial tests can be performed on soil samples:
1. Unconsolidated-Undrained (UU): A soil sample is rapidly loaded without allowing consolidation, often used to determine undrained shear strength. Pore water in the sample is not allowed through the permeable membrane at the top and bottom.

2. Consolidated-Undrained (CU): A soil sample is allowed to consolidate under a constant confining pressure, once consolidated the sample is then loaded to failure. Pore water in the sample is not allowed through the permeable membrane at the top and bottom. This is the most commonly used triaxial test on clay samples.

3. Consolidated-Drained (CD): A soil sample is allowed to consolidate under a constant confining pressure and then loaded. Pore water in the sample is allowed through the permeable membrane at the top and bottom.

4. Stress Path Triaxial Test: A soil sample is subjected to a series of loading and unloading cycles, often used to determine the soil's stress-strain behavior under different loading paths.

Triaxial test data is useful for calculating the shear strength of soil and its stress-strain behavior. A example of triaxial data from a Gjerdrum clay sample can be seen in Figure 11, where shear stress is plotted against axial strain.
2.3 Soil Modeling

In soil mechanics, constitutive models are used to describe the behavior of soil when it is subjected to shear stress. Generally these models fall into two categories, total stress models and effective stress models. Both describe the stress state of soil under different loading conditions. Total stress models assume soil particles and water behave as a single unit, where strength and deformation characteristics are governed by the total stress state. Total stress analyses are usually performed in situations where the primary interest is undrained loading and response. Sensitive clays are known for low permeability where the undrained response is almost always of interest. The undrained shear strength, $s_u$, of a soil is therefore of primary interest total in stress analyses, such as those presented here.

Effective stress models treat the soil as a two-phase system consisting of solid particles and pore water. The effective stress in a soil is the stress transmitted through the soil skeleton only.
and is the difference between the total stress and the pore water pressure. The relationship can be described mathematically as:

\[ \sigma = \sigma' + u \]  

Where \( \sigma \) is the total stress acting on a soil element, \( \sigma' \) is the effective stress acting on a soil element and \( u \) is the pore water in the element. The pore water pressure is the pressure exerted by water on the soil particles, dependent on a variety of in situ factors. Effective stress analyses are more commonly used in soil mechanics as they can accurately represent the stress state of a soil under all loading conditions.

### 2.3.1 Principal stresses and principal axes of stress

Common geotechnical convention denotes compression as positive and extension as negative. Considering all the stresses acting on a cubical soil element (Figure 12) we can denote principal stresses as stresses acting on a plane where no shear stress occurs, thus giving the principal directions as the directions of the normal vectors to these planes.

![Stresses acting on a soil element](Nordal, 2022)

Given the stress components in matrix form,

\[
\sigma = \begin{bmatrix}
\sigma_{11} & \sigma_{13} & \sigma_{13} \\
\sigma_{21} & \sigma_{22} & \sigma_{23} \\
\sigma_{31} & \sigma_{32} & \sigma_{33}
\end{bmatrix}
\]

it is possible to use Cauchy’s law to set up the following eigenvalue problem, where:

\[ I_1 = \sigma_{11} + \sigma_{22} + \sigma_{33} \]
\[ I_2 = \sigma_{11}\sigma_{22} - \sigma_{11}\sigma_{33} - \sigma_{22}\sigma_{33} + \sigma_{12}^2 + \sigma_{13}^2 + \sigma_{23}^2 \]  

\[ I_3 = \sigma_{11}(\sigma_{22}\sigma_{33} - \sigma_{23}^2) - \sigma_{12}(\sigma_{12}\sigma_{33} - \sigma_{13}\sigma_{23}) + \sigma_{13}(\sigma_{12}\sigma_{23} - \sigma_{13}\sigma_{22}) \]

From which it is possible to obtain the principal stresses, \( \sigma_1 \), \( \sigma_2 \) and \( \sigma_3 \), by solving the third order equation:

\[ \sigma^3 - I_1\sigma^2 - I_2\sigma - I_3 = 0 \]

### 2.3.2 Active, Shear and Passive Stresses

Low-plastic clays, such as most Norwegian clays with brittle fracture properties are generally more anisotropic than highly plastic clays (Lunne et al., 2006). When determining representative values for undrained shear strength in clays with brittle fracture properties it is particularly important to take the shear strength anisotropy into account. Shear strength values in compression, direct shear and tension vary with undrained shear strength greatest under active conditions and lowest during passive conditions, see equations [8] and [9] in 3.1.2. Figure 13 shows where the three stress conditions should be used in a typical slope failure.

![Figure 13: Anisotropic state of stress in slopes, showing active direct and passive conditions, (NIFS, 2014)](image)

### 2.3.3 Tresca Criterion

The Tresca criterion is a material failure theory used to predict the yield strength of a ductile material under multiaxial loading conditions. According to the Tresca criterion, a material will fail when the maximum difference between any two principal stresses is equal to or greater than the yield stress of the material. Yielding occurs when the difference between the
maximum and minimum principal stresses exceeds a certain value. The Tresca criterion is a total stress criterion, as total stresses are applied requiring no knowledge of effective stress. The Tresca criterion may can be denoted the \( s_u \)-criterion and can be expressed mathematically as:

\[
\tau_{\text{max}} = \frac{1}{2} (\sigma_1 - \sigma_3)
\]

where \( \sigma_1 \) and \( \sigma_3 \) are the maximum and minimum principal stresses, respectively, and \( \tau_{\text{max}} \) is the maximum shear strength. The Tresca criterion is isotropic with shear strength, \( s_u \), the same on all planes independent of orientation. mean stress independent with shear stress capacity in compression and tension the same. This can be shown graphically, Figure 14, where the failure envelope does not expand. Neither isotropic compression or isotropic tension causes failure. The Tresca criterion is a special case of the Mohr-Coulomb criterion where the friction angle is zero.

**FIGURE 5.1. The Tresca Criterion in principal stress space**

*Figure 14: The Tresca criterion in principal stress space. Taken from NTNU PhD course BA8304 lecture notes by Steinar Nordal (Nordal, 2022).*
2.4 FEM Overview

2.4.1 Finite Element Method

Finite Element modeling technique is a numerical method for solving differential equations and analyzing the behavior of physical systems. In geotechnical engineering, FE methods can be used to simulate the behavior of a soil, predicting deformation, stresses and flow. FE methods can be used to evaluate the stability and safety of slopes, embankments, and other earthworks, and to predict the behavior of soil and rock under different environmental conditions. FE analyses are well-suited to model advanced slope stability problems for several reasons:

1. Discretization: Slope stability problems involve large areas of soil or rock, which can be difficult to model using traditional analytical methods. FEM allows for the discretization of the slope into smaller, more manageable elements, so they can be modeled and analyzed individually. This also helps with problems arising from complex geometries.

2. Non-linearity: Slope stability problems often involve non-linear behavior, such as plastic soil deformation and failure. FEM is well-suited to handle non-linearity and provide predictions of the behavior of the slope under different loading conditions.

3. Sensitivity analysis: FEM allows for sensitivity analysis, which is useful in understanding how different variables affect the stability of the slope.

2.4.2 Abaqus Overview

Abaqus is a general-purpose finite element analysis software that can be used to simulate the behavior of a wide range of physical systems, including structures, fluids, soils and rock. It is widely used for applications such as design optimization, structural analysis, and impact analysis and can accommodate more complex simulations involving earth materials. Abaqus is not widely used by geotechnical engineers, who favor programs specifically created for geotechnical analysis. Most FE software developed for slope stability analysis use the Lagrangian framework (e.g. Griffiths and Lane (1999) and Loukidis et al. (2003)). A main
disadvantages of a Lagrangian framework model is that a significant mesh distortion occurs around the failure planes. Using the coupled Eulerian-Lagrangian framework available in Abaqus solves mesh distortion issues. Following mechanical engineering practice, Abaqus denotes compression as negative and extension as positive, so careful attention must be paid to the sign conventions. The basic steps of using Abaqus to perform a finite element analysis include:

1. Model creation: The first step is to create a model of the system being analyzed by defining the geometry, boundary conditions, and loading conditions. Different regions in a model can be assigned different material properties. The model can be created using the built-in CAD tools or imported from other CAD software.

2. Discretization: The model is discretized into a finite element mesh. The elements can be of different shapes, such as triangular, quadrilateral, or hexahedral, and can have different properties, such as material properties, boundary conditions, and loading conditions.

3. Analysis: Abaqus uses a variety of numerical techniques, including the finite element method, to solve the equations of motion for the system. The software can handle both linear and nonlinear problems, and can include a variety of effects such as large deformations, contact between many parts, and thermal effects.

4. Post-processing: After the analysis is complete, the results can be post-processed to obtain information such as displacements, strains, and stresses. Abaqus includes a variety of post-processing tools, including visualization and data extraction tools for analyzing results.

2.4.3 Explicit Dynamic Analysis

In Abaqus, an explicit central-difference time integration rule is used to solve the equations of motion for the system being analyzed. Compared to the direct-integration dynamic analysis procedure available in Abaqus/Standard, each time increment is computationally relatively inexpensive. This approach is different from implicit analysis, where the equations are solved using an iterative method. At the beginning of each time increment, $t$, the explicit central-difference operator satisfies the dynamic equilibrium equations then the accelerations
calculated at time $t$ are used to advance the velocity solution to time $t + \Delta t / 2$ and the displacement solution to time $t + t\Delta$ (Abaqus Documentation Collection, 2021). The explicit time integration scheme used in Abaqus is suitable for modeling problems with large deformations and nonlinear material behavior.

2.4.4 CEL and Eulerian Analysis

Traditional Lagrangian analysis uses fixed nodes within the material, meaning elements deform as the material deforms. Lagrangian elements are always 100% full of a single material, making the material boundary consistent with the element boundary. In an Eulerian analysis, nodes are fixed in space, allowing material to flow through elements that do not deform, (Abaqus Documentation Collection, 2021). Eulerian elements are not required to be 100% full of material, allowing for void space in the element, meaning the Eulerian material boundary must be computed during each time increment and not necessarily corresponding to an element boundary. The Eulerian mesh works best as a simple rectangular grid of elements constructed to extend past the Eulerian material boundaries, giving the material space to move and deform. As Eulerian material moves outside the Eulerian mesh, it is lost from the simulation. Interactions with Lagrangian elements is possible through Eulerian-Lagrangian contact; simulations with this type of contact are referred to as coupled Eulerian-Lagrangian (CEL) analyses. All of the simulations presented in this thesis use CEL analyses. Using CEL analyses it is possible to simulate material changing from a solid to liquid form, allowing simulations to more accurately capture all the phases of a flowslide.
3 Methods

The following method uses Abaqus/Explicit commercial finite element program for all analyses without the need for coding additional subroutines. The main challenge addressed here relates to implementing the variation of undrained shear strength, \( S_u \), with depth and accumulated plastic shear strain. Other studies implemented this in Abaqus by writing a program in FORTRAN via user subroutine. Simulations of the 4 profiles presented in this thesis are implemented in Abaqus as described in the following section with some variations between the preliminary simulations (base case and Saint Jude Landslide) and the Gjerdrum simulations (Profile 1 and Profile 357).

3.1 ABAQUS Implementation

The following section will aim to provide a thorough guide for how the simulations were set up and implemented in the Abaqus/Explicit commercial software package. The workflow of creating a model in Abaqus generally follows the structure seen in Figure 15.

![Figure 15: Abaqus modules showing the general workflow for creating and running simulations in ABAQUS.](image)

Model optimization was not conducted, leaving significant room for improved efficiency in future works. Simulations were run using the Abaqus standard explicit model. Abaqus simulations do not include units, leaving unit consistency up to the user. The international
system of units (SI) was followed using base units (seconds, meters and kilograms) in all simulations.

### 3.1.1 Part

All simulations are made up of parts. Different part types are available, the primary type used here is a Eulerian part. AutoCAD line drawings of the slope geometry were imported and used in creating the Eulerian part. For CEL simulations the slope must be modeled as one part, portioned into multiple sections. Each section can be assigned a material or void space. This allows material to move within the model. In Abaqus/Explicit, the Eulerian approach is only available for three-dimensional elements. The simulations performed here are therefore only one element length in the out-of-plane direction to simulate 2-dimensional plane strain conditions. Following previous studies (Dey et al., 2015; Saha et al., 2022; Wang et al., 2016), an 0.5 meter element thickness is used in all simulations. A 3 dimensional view of Profile 1 from the Gjerdrum landslide is shown in Figure 16.

![Figure 16: 3-Dimensional view of Profile 1 part.](image)

To decrease computational time, small modifications to the profile geometry were made to reduce the number of acute angles.

### 3.1.2 Property

Material properties for each section are created and then assigned to a region, in this case the entire Eulerian part is one section with different regions corresponding to different soil layers. All soil layers (materials) are assigned a density, elastic parameter, and plastic parameters. An
example of material behaviors assigned to the clay layer used in Profile 1 can be seen in
Figure 17 where a density of $1900 \text{ kg/m}^2$, Youngs modulus of $5e6 \text{ Pa}$ and Poisons ratio of
0.495 is assigned.

The Mohr-Coulomb plasticity criterion is used for all simulations presented here. For the
Gjerdrum simulations the crust is given a friction angle of $30^\circ$ and a dilation angle of $0.1^\circ$,
with no cohesion. For clay and quick clay materials, plasticity is modeled with zero angle of
friction and $0.1^\circ$ dilation, varying cohesion as the primary parameter, Figure 18. The Tresca
criterion can be implemented by using the Morh-Coulomb model with no friction angle. The
variation of yield strength ($= 2\sigma_2$ in Abaqus FE analysis) is defined as a function of plastic
strain, calculated assuming the shear band thickness is equal to the element thickness. The
strain softening behavior for clay and quick clay materials in the Gjerdrum simulations are
based on triaxial tests from borehole samples (Figure A 1, Figure A 2), plotted in Figure 19.
The clay material does not soften past 20% strain while the quick clay softens linearly from
the 20 % strain value to 0.5 kPa at 200% strain.
Cone penetration test with pore water measurement (CPTu) soundings provide a basis for estimating active shear strength at increasing depths. Using the Norwegian Public Roads Administrations (NPRA) spreadsheets (NPRA, 2021) to interpret CPTu soundings from
locations along each profile, initial shear strength vs depth profiles for the Gjerdrum profiles were generated, Appendix A. Results from triaxial tests and CPTu interpretations give active shear strength values. Since materials in the simulation are modeled using isotropic shear strengths, a correction factor is applied to the active shear strengths. Recommended anisotropy factors for design in Norwegian clay with low plasticity (NIFS, 2014) are:

\[ direct = active \times 0.63 \]

\[ passive = active \times 0.35 \]

An average of active, direct and passive shear strength is used as the isotropic shear strength of each element (isotropic shear strength = 0.66 active). For ease of comparison with CPTu profiles and triaxial data, all strength vs depth profiles presented show active shear strength. Initial simulations for the Gjerdrum profiles assign isotropic shear strength values, calculated using CPTu borehole interpretations. Later simulations adjust these values with the goal of simulating slope failure closer to what was observed in the Gjerdrum landslide.

Each region of the Eulerian slope is assigned a material (Figure 39, Figure 46) where the shear strength vs depth curve chosen determines cohesion values. Due to the topography of each profile, the depth below ground surface varies in relation to both the x and y coordinate. To assign elemental shear strength values based on depth below ground surface, each element references a field variable. To obtain a field variable to be used as a proxy for depth, a simulation of the profile with an artificially strong stiff clay for the entire soil mass is run, producing an output database with stress components in the vertical direction (SVAVG22). SVAVG22 values for Profile 1 can be seen in Figure 20. Elements in subsequent simulations can use SVAVG22 from the output database to assign field variable to each element. This allows a strength profile to be assign to each element based on depth below the ground surface.
Figure 20: Model results showing volume averages stress components in the y direction (SVAVG22) for Profile 1. The simulation here, assigns an artificially high strength to the entire soil mass to ensure no deformation. The output database values for vertical (y-direction) stresses in each element can be used as a proxy for depth.

Figure 18 shows where the cohesion yield stress, absolute plastic strain and field variables are assigned to a material. The SVAVG22 parameter calculated for each element does not give an exact correlation with depth. Correlation of SVAVG22 values with depth was done by manually inspecting elemental values at different locations. Stress strain curves are assigned to a field variable approximating 1 meter of depth. The stress strain values shown in the blue box in Figure 21 follow the normalized stress strain curve for quick clay shown in Figure 19 for a depth of 6 meters. An example element shown in Figure 21, lying at a depth of approximately 6 meters deep, is assigned a Field 1 variable of -113427. Elements with field variable numbers between -107000 and -128000 are assigned a stress strain curve, corresponding to a depth of 6 meters with a max shear strength of 52.8 kPa, softening to a strength of 15.7 kPa at 20 % strain, then further softening (liquefying) to a strength of 0.5 kPa at 200 % strain (shown in the blue box in Figure 21).
Figure 21: Example of how cohesive strengths are assigned to each element. Element 24065, lying at a depth of approximately 6 meters deep, is assigned a Field 1 variable of -113427 from SVAVG22 values. Elements with field variable numbers between -107000 and -128000 are assigned the stress strain curve shown in the blue box, corresponding to a depth of 6 meters.
In combination with the normalized stress strain curve, depth vs strength profiles for all clay and quick clay materials are used to create the input cohesive shear strengths. Figure 22 shows active shear strength vs depth profiles used for selected simulations in Profile 1.

![Figure 22](image)

*Figure 22: Active shear strength vs depth curves for the QC1 quick clay layer in Profile 1. Profile 1 CPTU values follow from boreholes 2020-121 interpretations.*

### 3.1.3 Assembly

A model contains one main assembly, composed of instances. Each instance can contain one or more parts. Even though most of the slope profiles examined here were modeled using a
single Eulerian part, an assembly consisting of just one instance with one part needs to be created. It is also helpful to assign a global coordinate system with the origin at a convenient location in here.

### 3.1.4 Step

The step module creates and defines the analysis steps as well as associated output requests. The step sequence allows for implementation and changes of applied loads, boundary conditions, addition or removal of parts or other changes during the course of the analysis. The analyses presented here use 3 primary steps, Figure 23; Initial, Add Gravity, and Runout, with some analyses adding a place holding step after the initial and a Reduce Toe step for the Saint Jude Landslide simulations. The initial step defines boundary conditions, predefined fields, and interactions that are applicable at the beginning of the analysis. All Abaqus analyses have an initial step which can not be re-named, changed or deleted. For CEL analyses only dynamic explicit analysis is available, and must be used in every step. Step-1 is created a placeholder step, allowing analyses to be restarted from the following steps. The add gravity step increases the gravitational load linearly over the course of the step. Increasing gravity gradually dampens the elastic response of the material. Initial simulations where gravity was applied instantaneously resulted in the slope bouncing. The runout step keeps the gravitational load applied allowing for the slope to slide and/or flow. The methodology used in Wang and Hawlader (2016) used a step to reduce the shear strength of a toe region to trigger a larger landslide. Simulations are set to model a time period relevant to actual landslide events. Total simulation time often varies to save computational time and storage space.

![Figure 23: Step manager window showing the analysis steps created in the step module.](image)
Outputs written to the output database (.odb file) are specified by creating output requests in each step. An output request defines which variables will be saved from an analysis step, from which region of the model they will be output, and at what rate they will be output. Many different output variables have been explored with the primary output variables being:

- **SVAVG**: Stress components, computed as a volume fraction weighted average of all materials in the element.
- **PEAVAVG**: Plastic strain components, computed as a volume fraction weighted average of all materials in the element.
- **PEEQAVAVG**: Equivalent plastic strain, computed as a volume fraction weighted average of all materials in the element.
- **U**: Displacement components.
- **V**: Velocity components (both translation and rotation).
- **EVF**: Eulerian volume fraction. Output includes volume fraction data for each material defined in the Eulerian section, plus the volume fraction of void.
- **FV**: Field variable, showing the field variable assigned to each element.

For the simulations presented here, output variables were written at one second intervals during each time step. Larger time intervals were investigated as they reduced computational time and output database size, but sacrificed quality.

### 3.1.5 Load

Creating initial conditions, boundary conditions, and loads can be done in the load module. Two initial conditions are created in the initial step. Material is assigned to each region and vertical stress field values are assigned to each element. Using the Eulerian Volume Fraction (EVF) tool, soil and void spaces are created in Eulerian part by assigning material in a predefined field. The procedure for assigning the vertical stress field to each element can be seen in Figure 24. Boundary conditions are also assigned in the initial step. Zero velocity boundaries are assigned to the sides and ends of the model, not allowing for movement normal to the side plane. The bottom boundary does not allow for movement in any direction. The downslope boundary above the soil layers is kept open, allowing for material to flow out of the model. A gravity load is applied during the add gravity step, increasing linearly, over the 100 second step. Increasing gravity slowly limits artificial elastic movements from sudden
loading as well showing at what percent of gravity failure occurs. A geostatic load is available in Abaqus, used to create a geostatic stress field removing the need to apply gravity incrementally. Previous studies (Dey et al., 2015) apply a geostatic load in the first step, although it is unclear how specifically this was done. In the Saint Jude landslide simulations the strength of the toe material was reduced by changing the field variable in the reduce tor step.

Figure 24: Procedure for creating a predefined vertical stress field using a field variable.

3.1.6 Mesh

The soil is modeled as Eulerian material using EC3D8R elements, which are 8-noded linear brick, reduced integration elements with hourglass control. A finer mesh 0.5 m mesh is used where the slip surface is expected to occur, increasing to a maximum of 0.5 x 5 m. Hex element shapes were used. The mesh for Profile Phase 3-5-7 is shown in Figure 25.

Figure 25: Profile Phase 3-5-7 mesh.

Mesh sensitivity analysis on soil slopes of similar magnitude (Dey et al., 2015) show mesh size has significant influence on shear strain in post-peak softening behavior of soil. The majority of the current studies mentioned in section 1.1 have settled on using a mesh size of 0.5 x 0.5 m, although a few analyses have reduced the mesh size further to 0.25 x 0.25 meters.
3.1.7 Job

Each simulation is submitted for analysis by creating a job. Double precision was used in the explicit analysis to help remove roundoff errors. Parallelizing into 4 domains, most simulations took between 2 and 10 hours to complete. The mesh geometry was the largest factor contributing to simulation efficiency.

3.1.8 Visualization

The visualization module displays simulation results, with many powerful tools for examining output variables when displaying results. Contour plots showing stress and strain components can be animated, showing deformation processes as failure progresses. Other useful tools are available here to overlay different results on the model, such as shown in Figure 35.
4 Preliminary Simulations

To compare with other studies, preliminary simulations attempt to replicate Dey et al. (2015) and Wang et al. (2016) simulations. With the post-peak degradation of undrained shear strength, or strain softening, of the soil determining much of the mechanics, careful attention is paid to implementation in the model. Using a strength degradation equation modified from Einav and Randolph (2005) current studies (Dey et al., 2015; Wang & Hawlader, 2017; Wang et al., 2016; Wang et al., 2022; Wang et al., 2015) have settled on using the equation:

\[ S_u = [1 + (S_t - 1) \exp \left( -\frac{3\delta}{\delta_{95}} \right)] S_{uR} \]

Where \( S_u \) is the mobilized undrained shear strength at displacement \( \delta \), \( S_t \) is the sensitivity of the soil with \( S_t = \frac{s_{up}}{S_{uR}} \), \( \delta_{95} \) is the value of \( \delta \) at which the undrained shear strength of the soil is reduced by 95 % and \( S_{uR} \) is the residual shear strength value. The entire stress strain curve, including the elastic portion is shown in Figure 26.

![Figure 26: Shear strength reduction curve using equation Error! Reference source not found. from Dey et. al. (2015).](image-url)
4.1 Base Case Slope

Simulations of the base case slope follow FE modeling of sensitive clay slopes presented by Dey et al. (2015). The current simulations presented here replicate as closely as possible all the parameters used in the published study. A difference in how the material properties are implemented should be noted. Simulations presented here assign material properties in a simplified process as is outlined in 3.1.2. Since peak undrained shear strength is held constant throughout the sensitive clay layer there is no need to use a field variable to correlate elements with depth. Dey et al. (2015) assign material properties using the von Mises yield criterion, although it is not clear how it is specifically implemented.

4.1.1 Model Parameters

The base case slope geometry can be seen in Figure 27. The eroded block is displaced to the left at a velocity of $0.1 \, \text{m/s}$. To increase efficiency, small geometric changes were made for the simulations presented here, where the slope is increased in 0.5 meter steps.

![Base Case Slope Diagram](image)

*Figure 27: Geometry of base case slope, (Dey et al., 2015). The slope is modeled in 3 Dimensions with the z thickness of the model being 0.5 meters.*

Material parameters used for the base case simulation are given in Table B-1. Equation [10] is used to define the shear strength reduction curve shown in Figure 26 for the sensitive clay layer. It can be noted the peak undrained shear strength ($s_{up}$) used in the sensitive clay layer is constant, and does not increase with depth.

4.1.2 Results
Base case simulation results showing plastic strain can be seen in Figure 28. Shear band formation follows a circular pattern without exhibiting continued horizontal propagation along the bottom of the sensitive clay layer. Element velocity at the start of the runout step, given in Figure 29 shows circular movement at the initiation of failure.

![Diagram](image)

**Figure 28:** Base case 1 simulation results showing plastic strain components (PEAVG) at 5 (a), 17 (b), 50 (c) and 100 (d) seconds into the runout step. The change in position of the toe at a 0.1 m/s displacement velocity is 0.5 m in (a), 1.7 m in (b) 5 m in (c) and 10 m in (d).
4.2 Saint Jude Landslide

Simulations of the Saint Jude landslide follow FE modeling of sensitive clay slopes presented by Wang and Hawlader (2016). The first simulation presented here aim to replicate as closely as possible all the parameters used in the published study. The subsequent two simulations vary the sensitivity and shear strength vs depth profile. A difference in how the material properties are implemented should again be noted. Material properties are assigned as outlined in 3.1.2. The variation of undrained shear strength with depth and accumulated plastic shear strain is implemented Abaqus by writing a program in FORTRAN via user subroutine in the Wang and Hawlader (2016) study.

4.2.1 Model Parameters

The Saint Jude landslide geometry as used by Wang and Hawlader (2016) can be seen in Figure 30. Equation [10] is used to define the shear strength reduction curve shown in Figure 26 for the sensitive clay layer. Sensitivity values and peak undrained shear strength profiles of the sensitive clay layer for simulations of the Saint Jude landslide are given in Error! Reference source not found. To initiate failure, the shear strength of the circular toe section is reduced at the beginning of the runout step, such that it flows out of the toe.
Figure 30: Saint Jude landslide geometry (a) and shear strength profile (b) from Wang and Hawlader (2016). The slope is modeled in 3 Dimensions with the z thickness of the model being 0.5 meters. The variation of undrained shear strength with depth implemented in the model follows the CPTu results presented in (b).

4.2.2 Results

Using material parameters outlined in Table B 1 and Error! Reference source not found., s imulation Saint_Jude_01 aims to replicate the Saint Jude landslide as close as possible to simulations presented by Wang and Hawlader (2016). Plastic strain components (PEVAVG) in Abaqus is used to show the shear bands formation during the landslide. The initial failure plane, Figure 31 (a), presents 4 seconds into the runout step, exhibiting a mostly circular failure plane. At the interface of the sensitive clay and stiff clay layers, a horizontal shear band with the failure surface staying in the sensitive clay layer. At 13 seconds into the runout step a second, smaller, retrogressive failure, Figure 31 (b), is seen in the upper 10 meters of the sensitive clay layer. The final runout of the simulation is shown in Figure 31 (c). The shear band is not observed to propagate horizontally past the circular slip surface during the simulation. Looking at the resultant velocities 4 seconds into the runout step, Figure 32, circular elemental velocities can be seen during the initial failure.
Figure 31: Plastic strain components (PEAVG) showing the initial failure surface at 4 seconds into the runout step (a), the first retrogressive failure at 13 seconds into the runout step (b) and the final runout for simulation Saint_Jude_01, 25 seconds into the runout step.

Figure 32: Velocity vectors for the Saint_Jude_01 simulation showing resultant element velocity during the initial failure, 4 seconds into the runout step. The colors displayed in the legend correspond to the vector arrows.

Using the same simulation parameters as Saint_Jude_01 but increasing the sensitivity value of the sensitive clay layer, from 6 to 12, Saint_Jude_02 simulation results (Figure 33) show an increase in retrogressive failures. The initial circular failure plane occurs at the beginning of the runout step, Figure 33 (a). The first retrogressive failure, Figure 33 (b), exhibits a triangular failure pattern with successive retrogressive failures, Figure 33 (c) and (d), having a more circular fail plane. The final runout for the Saint_Jude_02 simulation, Figure 33 (e), shows a larger retrogression distance when compared with Saint_Jude_02. A mostly circular elemental velocity pattern can be seen during the initial failure in the Saint_Jude_02 simulation, Figure 34. Plastic strain components plotted on top of elemental velocity during
the first retrogressive failure shown in Figure 35 both exhibit the only non-circular failure pattern in the simulation.

Figure 33: Plastic strain components (PEAVG) for simulation Saint_Jude_02 showing the initial failure surface 1 second into the runout step (a), the first retrogressive failure at 8 seconds into the runout step (b). Multiple retrogressive failures follow, some of which are shown in (c) and (d), with the final runout for simulation Saint_Jude_01, 55 seconds into the runout step (e).
Simulation Saint_Jude_06 uses a constant undrained shear strength for the entire sensitive clay layer, 25 kPa, with the rest of the parameters the same as the Saint_Jude_01 simulation. The initial failure is seen to present earlier, 77 seconds into the add gravity step, Figure 36 (a), with a larger first retrogressive failure presenting 83 seconds into the add gravity step, Figure 36 (b). Two more smaller circular retrogressive failures occur Figure 36 (c) and (d) in the simulation during the runout step. Similar to the other Saint Jude simulations, a circular elemental velocity pattern can be seen during the initial failure, Figure 37.
Figure 36: PEAVG for the Saint_Jude_06 simulation. The initial failure surface presents 77 seconds into the add gravity step (a), the first retrogressive failure 83 seconds into the add gravity step (b). Retrogressive failures shown in (c) and (d) occurring at the beginning and 7 seconds into the runout step, with the final runout for simulation ending 20 seconds into the runout step with the same failure surface as shown in (d).

Figure 37: Velocity vectors for Saint_Jude_06 simulation showing resultant element velocity during the initial failure, 77 seconds into the add gravity step.
5 Gjerdrum Simulations

Two profiles are presented here from the Gjerdrum 2020 landslide, Profile 1 and Profile Phase 3-5-7. Profile 1 examines the initiation of the Gjerdrum landslide (Figure 38(a)), while profile phase 3-5-7 examines the retrogression of the landslide northward (Figure 38 (b)), through phases 3, 5 and 7.

Figure 38: Location of Profile 1 (a) and Profile Phase 3-5-7 (b) of the 2020 Gjerdrum landslide (Ekspertutvalg, 2021).

5.1 Profile 1

The first simulation presented here uses available geotechnical data to simulate the initial slope failure of the Gjerdrum landslide. The subsequent two simulations presented vary the undrained shear strength vs depth profiles.

5.1.1 Model Parameters

Slope geometry, Figure 39, is estimated from geotechnical reports (Multiconsult, 2021; NGI, 2021b) with uncertainties, as limited data is available from before the landslide. Technical drawings for Profile 1, including borehole logs, can be seen in Figure B 8. A quick clay layer
lies between two clay layers with a thin 2 meter crust layer on top. A portion of the crust at the base of the slope is removed to simulate the initial erosion of the toe failure. The Profile_1_CPTU simulation uses shear strength vs depth values for the quick clay and quick clay layers based on CPTu interpretations from borehole 2020-121 and 2020-180. Material properties are assigned as outlined in 3.1.2. Both the quick clay and clay layers are modeled with softening curves, with undrained shear strength profiles given in Appendix B. The crust is assigned an 30° angle of friction with no cohesion.

5.1.1 Results

The first failure plane in the Profile_1_CPTU simulation can be seen clearly 62 seconds into the add gravity step (Figure 40 (a)). The first retrogressive failure plane can be seen 90 seconds into the add gravity step (Figure 40 (b)) with the second retrogressive failure plane occurring 6 seconds into the runout step (Figure 40 (c)). It should be noted that between the initial failure at 62 seconds and first retrogressive failure at 90 seconds the material in the initial erosions blocks briefly slide back toward the slope. This movement is evident in the simulation although not shown in Figure 31. The shear band in the first retrogressive failure primarily propagates along the boundary between the clay and quick clay interface dipping into the bottom clay layer briefly. The second retrogressive failure propagates along the top boundary of the quick clay boundary before transitioning into a circular slope failure. A compound slip failure can be seen from the elemental velocities of the initial failure in Figure 41.
Figure 40: Profile 1 failure surfaces for simulation Profile 1 CPTU, shown using elemental strain component (PEAVG). The initial failure plane (a) can be seen at 62 seconds into the add gravity step, the first retrogressive failure (b) can be seen at 90 seconds into the add gravity step and the second retrogressive failure (c) can be 6 seconds into the runout step. The post slide surface elevation is shown in red in (c).

Figure 41: Velocity vectors for Profile 1 CPTU simulation showing resultant element velocity during the initial failure, 62 seconds into the add gravity step.

Results from simulation Profile 1 A32 show a single circular slope failure without any further retrogressive failures (Figure 42), also observed in resultant elemental velocities, Figure 43, during the slide.
Figure 42: Failure surfaces for simulation Profile 1 A32, shown using elemental strain component (PEAVG). The initial failure plane (a) can be seen at 82 seconds into the add gravity step, no further retrogressive failure occurred, with the final runout shown in (b).

Figure 43: Velocity vectors for Profile 1 A32 simulation showing resultant element velocity during the initial failure, 87 seconds into the add gravity step.

Looking at the Profile 1 A35 simulation, a horizontal shear band initially propagates along the bottom boundary of the QC1 layer before transitioning up through the layer and propagating along the top boundary of the layer, Figure 44 (a) and (b). Element velocities in the initial failure, Figure 45, show movement in is a quasi-circular pattern. Animations of the simulation show spread type sliding on the horizontal shear band during the runout step.
Figure 44: Failure surfaces for simulation Profile 1 A35, shown using elemental strain component (PEAVG). The initial failure plane (a) can be seen at 81 seconds into the add gravity step, the first retrogressive failure (b) can be seen at 85 seconds into the add gravity step with the final failure surface still slowly creeping at the end of the simulation 100 seconds into the runout step.

Figure 45: Velocity vectors for Profile 1 A35 simulation showing resultant element velocity during the initial failure, 81 seconds into the add gravity step.

5.2 Profile Phase 3-5-7

Simulations of Profile Phase 3-5-7 break up the quick clay layer into different sections, approximately corresponding to phases of the Gjerdrum landslide. CPTu borehole interpretations from locations in each phase of the landslide provide a basis for assigning shear strength profiles.
5.2.1 Model Parameters

The profile modeled runs approximately north south along the Gjerdrum landslide. Relevant borehole locations along the profile are shown in Figure 38. From Figure 5, phase 1 of the landslide slides in a westward direction, perpendicular to the north-south profile phase 3-5-7. The profile geometry, Figure 46, accounts for this by removing the soil assumed to have slid out of the profile phase 3-5-7 plane, thus also acting as a trigger for initiating failure in the QC3A region. The bottom boundary of the model is assumed to be bedrock. Material properties are assigned as outlined in 3.1.2. Both the quick clay and clay layers are modeled with softening curves, with undrained shear strength profiles given in Appendix B. The crust is assigned an 30° angle of friction with no cohesion.

![Figure 46: Profile phase 3-5-7 geometry. QC1 denotes the quick clay layer sliding in phase 1. Sliding in phase 3 of the landslide occurs in layers QC3A and QC3B. QC5 and QC7 denote quick clay layers sliding in phases 5 and 7 respectively.](image)

The different quick clay regions in Profile Phase 3-5-7 allow for lateral variation of the shear strength vs depth curves. These regions are drawn as a rough approximation to the phase (Figure 5) of the Gjerdrum landslide. The void space (top two cells in Figure 46) contains part of the slope previously eroded in phase 1 of the landslide. There are 5 quick clay sections denoted by QC(Phase x), a non-quick strain softening clay layer and a crust.

5.2.2 Results

The first failure plane in the Profile 357 CPTU simulation occurs in the QC3A region with a circular failure into the eroded QC1 region at 76 seconds into the add gravity step, Figure 47 (a). A second retrogressive circular failure is seen 81 seconds into the add gravity step, Figure
47 (b) before transitioning into a flow slide beginning 88 seconds into the add gravity step and continuing through the runout step, Figure 47 (c) and (d). As shown in Figure 48, the initial failures do not propagate into quick clay regions upslope.

Figure 47: Failure surfaces for simulation Profile 357 CPTU, shown using elemental strain component (PEAVG). The initial failure plane (a) can be seen at 76 seconds into the add gravity step, the first retrogressive failure (b) can be seen at 81 seconds into the add gravity step. Further flowslide runout can be seen in (c) at 88 seconds with the final runout of the simulation shown in (d) at 100 seconds into the runout step.
Larger, global failures can be seen in the Profile 357 03 simulation. Initial shear band formation occurring 71 seconds into the add gravity step propagates from the base of the model up through the QC3A region, Figure 49 (a). Shear bands along the bottom boundary of the QC3A, QC3B and QC5 regions can be seen 85 seconds into the add gravity step, Figure 49 (b). Continued shear band development occurs in all of the quick clay regions 92 seconds into the add gravity step, Figure 49 (c). At the beginning of the runout step, a global spread failure of the entire profile can be seen, Figure 49 (d), primarily sliding on shear bands at the bottom boundary of the model. Further into the runout step many of the blocks have deformed as the landslide continues to runout, Figure 49 (e).
Figure 49: Failure surfaces for simulation Profile 357 03, shown using elemental strain component (PEAVG). The initial failure plane (a) can be seen 71 seconds into the add gravity step, propagating from the bottom boundary in the QC3A region. 85 seconds into the add gravity step shear bands develop into the QC3B and QC5 regions (b). 91 seconds into the add gravity step, shear band propagation continues along the top of the QC7 region (c). The global failure plane can be seen in (d) at 1 second into the runout step with further runout shown in (e) 19 seconds into the runout step. The post slide surface elevation is shown in red in (e).
6 Interpretation

6.1 Base Case

We can categorize retrogressive failures into three groups based on the formation of the failure surfaces, similar to Saha et al. (2022): flowslides, spread and a combination of the two. The initial circular failure initiating the successive curved failure surfaces characterizing flowslides is seen in the base case simulation. The simulation ended at a block displacement of 10 m, had the simulation allowed for a larger displacement it is possible further retrogressive failures would occur. The mechanism triggering slope failure in the base case can is seen to successfully trigger the slope failure. No plastic deformation is seen in the add gravity step. Once the toe block is displaced away from the slope plastic strain can be seen along the failure plane, where the shear band forms.

6.2 Saint Jude Landslide

The initial compound failure and first retrogressive failure in the Saint Jude 01 simulation are consistent with a flowslide, with the first retrogressive failure surface a step up from the initial failure surface. Although the initial failure occurred in the runout step in the simulation, the shear band is observed to propagate under the toe of the slope and curve upward past the toe of the slope (Figure 31 (a)) suggesting the simulated erosion did not cause the initial global failure. The slope failing at the beginning of the runout step, with gravity fully applied suggests the slope had a factor of safety (FoS) close to 1.

Failure surfaces in the Saint Jude 02 simulation suggest a combination of a flowslide and a spread. The horizontal shear band propagates along the bottom of the sensitive clay layer before angling up into the surface of the slope in the initial failure, Figure 33 (a). Formation of the triangular shaped horst and graben structure is visible in the first retrogressive failure Figure 33 (b), with velocity vectors at the same step time, Figure 35, exhibit spread failure mechanisms. The second retrogressive failure, Figure 33 (c), exhibits circular failure with the final retrogressive failure transitioning back to a triangular failure pattern. Similar to Saint Jude 01, failure in the Saint Jude 02 simulation initiated at the beginning of the runout step, also suggesting the slope had a FoS close to 1. Since both simulations are assigned the same shear strength vs. depth profile, similar FoS’s should be expected. The primary difference
between the simulations is the rate at which the slope fails and the retrogression distance. The Saint_Jude_02 simulation assigns a sensitivity of 12, twice Saint Jude 01. With higher sensitivity, deformation occurs faster as seen when comparing the runout of the two simulations. Higher sensitivity is also seen to increase retrogression distance, where Saint Jude 02 retrogresses significantly further with a higher sensitivity.

Keeping the same sensitivity as the Saint Jude 01 simulation, Saint Jude 06 assigns a constant shear strength (25 kpa) to the sensitive clay layer. This reduces the overall strength of the layer, and is evident when looking at the timing of the first failure, 77 seconds into the add gravity step as seen in Figure 36 (a). A horizontal shear band develops along the bottom boundary of the sensitive clay layer before transitioning into a quasi-circular failure upward toward the slope surface. Elemental velocity during the initial failure, shown in Figure 37, show mostly rotational movement of the initial failure block, although horizontal movement can be seen parallel to the bottom horizontal shear band. The first retrogressive failure, Figure 36 (b), is interpreted to have a non-circular shape with a triangular shape shear band formation, stepped up from the initial failure surface. As the failure propagates from the first to second retrogressive failures, Figure 36 (b) to Figure 36 (c), the triangular blocks break up. The last retrogressive failure in the Saint Jude 06 simulation exhibits a more clear circular slip surface. The formation of triangular blocks and instances of translational sliding are indicative of a spread failure, while features of a flowslide such as circular failures and stepped failure surface present, the global retrogressive failure can be interpreted as a spread/flowslide combination. Similar to the other Saint Jude simulations, the toe erosion did not facilitate the initial failure in the Saint Jude 06 simulation, as failure occurred before the shear strength in the toe was reduced.

**6.3 Profile 1**

Using values interpreted from CPTu profiles for the initial Profile 1 simulation, failure is observed 62 seconds into the add gravity step. This suggests the FoS for the Profile 1 CPTU simulation was much less than 1. It is also worth noting the erosion material from the initial failure had stopped flowing before the first retrogressive failure. The first retrogressive failure, observed at 90 seconds can be attributed to an increase in gravity, in a slope with a new geometry. The failure progression shown in Figure 40 could be seen as a combination of a flowslide and a spread, with the intact blocks sliding along a horizontal shear band with
successive curved failure surfaces. Elemental velocities shown in Figure 41 of the initial failure support a combination of a flowslide and spread, showing a non-circular movement not necessarily as angular as would be expected in a spread. Since much of the shear band is on the bottom boundary of the model or on a layer contact, an outsized influence from the boundary conditions is suspected.

Significant shear strength increases of both the clay and quick clay layer in the Profile 1 A32 simulation yield an initial slope failure, without any retrogression. A clear circular failure is present here, evident by elemental velocities shown in Figure 43. With the only failure occurring where slope geometry is steepest, an increase in shear strength with depth can be interpreted to effect the propensity for retrogressive failures to develop.

In the Profile 1 A35 simulation, initial horizontal shear band development can be seen along the bottom the quick clay layer before transitioning to propagation along the top of the layer. A possible reason for the horizontal shear band to transition from the bottom to the top of the sensitive clay layer as seen in Figure 44 (b) and (c) could be due to the constant shear strength of the quick clay (shown in Figure B 1) assigned between 10 and 20 meters depth. The horizontal shear band is also thought to facilitate the spread failure runout pattern observed in the simulation.

**6.4 Profile Phase 3-5-7**

With the quick clay layer divided into 5 different regions, split up by phase, Profile Phase 3-5-7 simulations introduce many more material boundaries, and notably, vertical boundaries between quick clay layers. The initial simulation, Profile 357 CPTU, based off CPTu interpretations from boreholes in each region did not produce a global failure, with a local failure observed 81 seconds into the add gravity step where the phase 1 material is assumed to have slid out during the initial failure in the Gjerdrum landslide. These local failures exhibit properties of a flow slide, and only occur in the QC3A region as seen in Figure 47. The runout material seen flowing in Figure 48 also supports a flowslide with almost complete remodeling.

Keeping the clay and crust layers the same, simulation Profile 357 03 quick clay layers were assigned different strength profiles (see Appendix B), resulting in a global failure. With many parameters changed, it would be hard to justify a reliable comparison between the Profile 357 CPTU and Profile 357 03. As the Profile 357 03 global failure largely follows the bottom
boundary condition, chances for artificial effects to be introduced increase. Although these large uncertainties are acknowledged, many of the observed effects are noteworthy. The Global failure shows large blocks sliding along shear bands (Figure 49 (c) and (d)) before breaking up with the material becoming completely remolded (Figure 49 (e)). Where the geometry of the bottom boundary dips down at the boundary of QC5 and QC7, the shear band does not continue to propagate along the bottom of the quick clay, suggesting bottom geometric effects might influence shear band propagation.
7 Discussion

7.1 Methodology

The method presented here is able to implement the CEL FE technique for analyzing slope stability using the commercially available Abaqus software package, without the need for coding additional subroutines. Progressive failure is simulated in soil slopes where degradation of undrained shear strength with plastic strain controls failure mechanisms.

Modeling material behavior could arguably be viewed as the most significant aspect when modeling sensitive and quick clay. As the Tresca criterion assumes isotropic shear strength, and the undrained shear strength of clay is known to be highly anisotropic (AAS, 1967; Soydemir, 1976) significant improvements could be gained by implementing models such as the NGI-ADP model (Grimstad et al., 2012) where anisotropy can be accounted for. An alternative option could also be to create an anisotropic yield function in Abaqus (Abaqus Documentation Collection, 2021). Both of these options would introduce the need for additional subroutines.

The stress components SVAVG22 is not calculated strictly from the weight of the overlying material, thus using it as a proxy for depth is not perfect. Looking closely at the SVAVG22 values used in Profile 1 simulations (Figure 50), higher vertical stresses can be observed at the toe of the slope resulting in elements at the toe of the slope being assigned higher shear strengths compared to elements under horizontal ground at the same depth. The impact of this could be considered minimal however, as many of the simulations have a crust layer or the upper shear strength profile is constant, as is the case with many of the quick clay layers in the Profile Phase 3-5-7 simulations.

Figure 50: An enlarged section of Figure 20 showing SVAVG22 values of Profile 1 higher near the toe of the slope.
The normalized softening curves used in all the Gjerdrum simulations were based off a single quick clay sample and single clay sample. Large differences can be seen in softening curves from triaxial tests of Gjerdrum samples (NGI, 2021a, 2021b). Assigning softening curves from localized sampled would significantly increase model accuracy. Significant increases in model efficiency could be obtained by investigating various meshing techniques. Although it was not formally investigated, models with many acute angled elements ran significantly slower, prompting re-meshing for subsequent simulations. Many meshing techniques for improving efficiency are available (Abaqus Documentation Collection, 2021) and could be employed in future methodologies.

### 7.2 Preliminary Simulations

Simulations of the base case slope and the Saint Jude landslide can be directly compared to the Dey et al. (2015) and (Wang et al., 2016) studies. The base case simulation presented here does not exhibit horizontal shear band development with spread failure as is seen in results from Dey et al. (2015), shown in Figure 51. The eroded soil block at 5 meters shown in Figure 28 (c) generally correlates with the M1 soil mass seen in Figure 51(c).

![Figure 51: Horizontal shear band formation of the base case simulation published by Dey et al. (2015). It can be noted the equivalent plastic shear strain (PEEQVAVG) is displayed.](image-url)
A parametric study on the base case slope from Dey et al. (2015) show increasing $S_{95}$ (used in equation [10] to calculate strength degradation) values reduced the likelihood for the formation of horst and graben structures. Further simulations from this parametric study using a high $S_t$ show a failure pattern similar to the base case 1 simulation presented here. The overall conclusions of the parametric study show toe erosion does not always create the horst and graben type structure with their formation depending on soil properties and geometry. Future parametric studies using the methodology presented here might be able to determine what geometries and soil properties lead to the horst and graben type spread failure.

Simulation of the Saint Jude landslide using the same parameters did not produce horizontal shear bands propagating from the base of the toe in the sensitive clay layer as shown in Figure 52 by Wang et al. (2016). While the Saint Jude simulations presented here did exhibit some horizontal shear band propagation, this followed along the base of the sensitive clay layer. Similarities in shape can be noted between the initial failure surface in the Saint Jude 02 simulation and those seen in Figure 52 (b). Resultant velocities shown in Figure 52 (c) are also largely similar to those shown in Figure 34.

*Figure 52: Formation of failure surfaces of the Saint Jude landslide simulation by Wang et al. (2016) showing a spread type landslide*
One of the conclusions drawn from the Dey et al. (2015) study states “For low values of St of the sensitive clay layer, only a small horizontal shear band is formed by toe erosion. On the other hand, only one block of soil near the slope fails when St is very high. For intermediate values of St (5 and 7 in the present study) a number of horst and grabens are formed and the failure is extended over a large distance”. This conclusion suggests sensitivity plays a large role in determining failure mechanisms, and could be a possible explanation for the observations seen when comparing retrogression distance of Saint Jude 02 to Saint Jude 01 where the St is doubled.

In the Saint Jude 02 and 03 simulations, observed failure geometries show similarities with the Wang et al. (2022) study. The stepped surface failure geometry observed in flowslides (Figure 9 (c)) can be seen in both simulations. The spread failure pattern observed in Figure 35 is similar to the spread failure pattern given in Figure 10.

With many aspects of the preliminary simulations not quite in agreement with recently published works, additional studies are recommended to further develop this approach for practical applications. Although not the focus of the thesis presented here, the general CEL FE methodology presented can be used to analyze large deformation of slopes without complex material parameters for a variety of situations.

### 7.3 Gjerdrum Simulations

Initial Profile 1 simulations using CPTu values show an unstable slope, failing before 62 percent of gravity has been applied. This is likely due to variations of shear strength with depth in the layer and brittleness of the clay. Similar to the Saint Jude landslide simulations, the intended mechanism to trigger failure in Profile 1 simulations was not applicable with none of the failures being triggered by the removal of a section of soil at the toe of the slope. More calibrating of the soil parameters is necessary to ensure the slope stable before any erosion should be introduced.

All the simulations allowed material to flow out of the model. For a meaningful comparison of simulation results to post slide terrain the model should include the entire runout area. The longer the simulations were allowed to run the more material would continue to flow, or slide along the shear bands, removing slide debris. The Profile_1_CPTU simulation retrogresses
further than post slide terrain, but this should be viewed critically as the failure in this simulation exhibits large post slide blocks not seen in the post slide elevation profile.

A conclusion from Wang et al. (2022) study states: “For an average initial undrained shear strength of sensitive clay ($s_u0$), the retrogression distance in a flowslide reduces with an increase in the gradient of $s_u0$ with depth”. Although only three simulations are presented here for Profile 1, the only profile not to fail retrogressively (Profile 1 A32) had the largest increase in gradient of $s_u0$ with depth, supporting the Wang et al. (2015) conclusion.

With many different material regions complicating Profile 3-5-7 geometry, significant future improvements could be made by removing these separate regions and changing soil properties to vary gradually within the soil layer. Examination of the CPTu profiles show the debris material of the Gjerdrum landslide to extend 4 to 7 meters deep. The failure surface seen in Profile_1_CPTU, Profile_1_A35 and Profile_357_03 simulations extend much deeper.

Failure patterns in all the Gjerdrum simulations show aspects of both flowslides and spreads as documented in other studies conducting CEL FE simulations on sensitive clay slopes (Dey et al., 2015; Saha et al., 2022; Wang & Hawlader, 2017; Wang et al., 2016; Wang et al., 2022; Wang et al., 2015). A major deficiency in the Gjerdrum simulations as well as the Saint Jude simulations is the inability to model horizontal shear band propagation originating at the toe of the slope. Further parametric studies of simplified models using the methodology would go a long way to address this issue.
8 Conclusions

The main goal of the thesis presented here is to provide a method for implementing the CEL technique for analyzing slope stability, compare results of this technique with recently published studies and simulate slope failures of the Gjerdrum landslide comparing simulation results with post slide investigations. A working methodology was presented, although refinement is needed before any practical use. Comparison with recent studies showed many similar failure patterns observed in sensitive clay spreads and flowslides, but the overall ability to reproduce these studies is not present. Simulations of the Gjerdrum landslide again showed failure mechanics consistent with sensitive and quick clay landslides although correlation to post slide investigations is relatively poor. Further calibration of the procedure should be undertaken for simulations to more closely match post slide investigations. Strain softening parameters are known to have impacts on failure mechanics ((Dey et al., 2015; Wang et al., 2022; Wang et al., 2015) and are not sufficiently explored here. Further investigation softening curves effect failure mechanics when implemented with this method would likely improve model accuracy.
9 Proposals for Future Works

There are many areas where significant improvements the methodology could be made, possibly in combination with more analysis of the Gjerdrum landslide. Following are recommendations for future works:

- Implementation of anisotropic material models when assigning material properties would further improve model accuracy.

- Conducting a parametric study using the methodology presented on an idealized slope to further understand how model parameters effect shear band development, specifically in regards to softening curves.

- Improvements to how strength parameters are assigned to soil elements, would help reduce boundary effects, possibly in combination with improving meshing methods.

- Implementation in 3 dimensions would allow for runout to be accurately modeled, greatly improving correlation with post slide investigations.

Abaqus Documentation Collection. (2021). Retrieved 30.03.2023 from help.3ds.com


DiBiagio, A. J. (2022). In: NGI.


NGI. (2021b). *Grunnundersøkelser - Datarapport*.


Appendix

A. Geotechnical Data

Triaxial tests conducted on borehole samples showing shear stress vs. axial strain are used to create normalized softening curves for clay and quick clay materials.

Figure A 1: Triaxial test results from borehole 2020-31 quick clay sample taken at 17.35 meters depth.
Figure A 2: Triaxial test results from borehole 2020-36 clay sample taken at 10.45 meters depth.

All CPTu interpretations were generated assuming ground water level 1 meter below the surface with hydrostatic conditions.
Figure A 3: Interpreted CPTu profile of undrained active shear strength vs depth for borehole 2020-121. Red line shows handpicked values used as reference for simulations.
Figure A 4: Interpreted CPTu profile of undrained active shear strength vs depth for borehole 2020-125. Red line shows handpicked values used as reference for simulations.
Figure A 5: Interpreted CPTu profile of undrained active shear strength vs depth for borehole 2020-140. Red line shows handpicked values used as reference for simulations.
Figure A 6: Undrained shear strength profile for borehole 2020-141, received as a personal communication and interpreted by Amanda Johansen DiBiagio (DiBiagio, 2022).
Figure A 7: Interpreted CPTu profile of undrained active shear strength vs depth for borehole 2020-162. Red line shows handpicked values used as reference for simulations.
Figure A 8: Interpreted CPTu profile of undrained active shear strength vs depth for borehole 2020-167. Red line shows handpicked values used as reference for simulations.
Figure A 9: Interpreted CPTu profile of undrained active shear strength vs depth for borehole 2020-170. Red line shows handpicked values used as reference for simulations.
Figure A 10: Interpreted CPTu profile of undrained active shear strength vs depth for borehole 2020-174. Red line shows handpicked values used as reference for simulations.
Figure A.11: Interpreted CPTu profile of undrained active shear strength vs depth for borehole 2020-180. Red line shows handpicked values used as reference for simulations.
Figure A 12: Interpreted CPTu profile of undrained active shear strength vs depth for borehole 2020-187. Red line shows handpicked values used as reference for simulations.
Figure A 13: Interpreted CPTu profile of undrained active shear strength vs depth for borehole 2020-189. Red line shows handpicked values used as reference for simulations.
Figure A 14: Interpreted CPTu profile of undrained active shear strength vs depth for borehole 2020-190. Red line shows handpicked values used as reference for simulations.
### B. Simulation Data

**Table B 1**: Parameters used for modeling the base case sensitive clay slope presented by Dey et al. (2015).

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Values</th>
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</thead>
<tbody>
<tr>
<td>Undrained Young's modulus, (E_u) (MPa)</td>
<td>Sensitive clay</td>
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<td>Poisson ratio, (\nu_u)</td>
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<tr>
<td>Peak undrained shear strength, (s_{ud}) (kPa)</td>
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<td>Undrained shear strength, (s_{ud}) (kPa)</td>
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<td>(\delta_{uc}) (m)</td>
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<tr>
<td>Submerged unit weight, (\gamma) (kN/m³)</td>
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*Crust is modelled as elastic–perfectly plastic without softening.

**Table B 2**: Parameters used by Wang and Hawlader (2016) for modeling the 2010 Saint Jude Landslide.

<table>
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<th>Parameters</th>
<th>Values</th>
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<tr>
<td>Undrained Young’s modulus, (E_u) (MPa)</td>
<td>Sensitive clay</td>
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<td>10</td>
<td>10</td>
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<tr>
<td>Poisson’s ratio, (\nu_u)</td>
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<tr>
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<td>Soil unit weight, (\gamma) (kN/m³)</td>
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*Crust and stiff layer is modeled as elastic–perfectly plastic without softening.

Simulations of the Saine Jude landslide varied sensitivity values and peak undrained shear strength profiles of the sensitive clay shown in **Error! Reference source not found.**
Normalized values for the softening curves used to define the clay and quick clay material properties used in Profile 1 and Profile Phase 3-5-7 simulations can be found in Table B 3:

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<th>Absolute Plastic Strain</th>
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Table B 3: Normalized stress strain relationships used to define the softening curves for the clay and quick clay material properties for Profile 1 and Profile Phase 3-5-7 simulations. Quick clay softening curves are modeled based on triaxial test results from borehole 2020-31.
Figure B 1: Active shear strength vs depth profiles of the quick clay layer (QCl) for Profile 1 simulations.
Figure B 2: Active shear strength vs depth profiles of the clay layer for Profile 1 and Profile 357 simulations. Profile 357 used Clay CPTU in both simulations. Profile 1 CPTU used Clay CPTU and Profiles 1 357 A32 and A35 used Clay Profile 1 A32/A35.
Figure B 3: Active shear strength vs depth profiles of the quick clay layer (QC1) for Profile 357 simulations.
Figure B 4: Active shear strength vs depth profiles of the quick clay layer (QC3A) for Profile 357 simulations.
Figure B 5: Active shear strength vs depth profiles of the quick clay layer (QC3B) for Profile 357 simulations.
Figure B 6: Active shear strength vs depth profiles of the quick clay layer (QC5) for Profile 357 simulations.
Figure B 7: Active shear strength vs depth profiles of the quick clay layer (QC7) for Profile 357 simulations.
Figure B.8: Profile 1 geometry (DiBiagio, 2022).
Figure B 9: Profile phase 3-5-7 geometry (DiBiagio, 2022).