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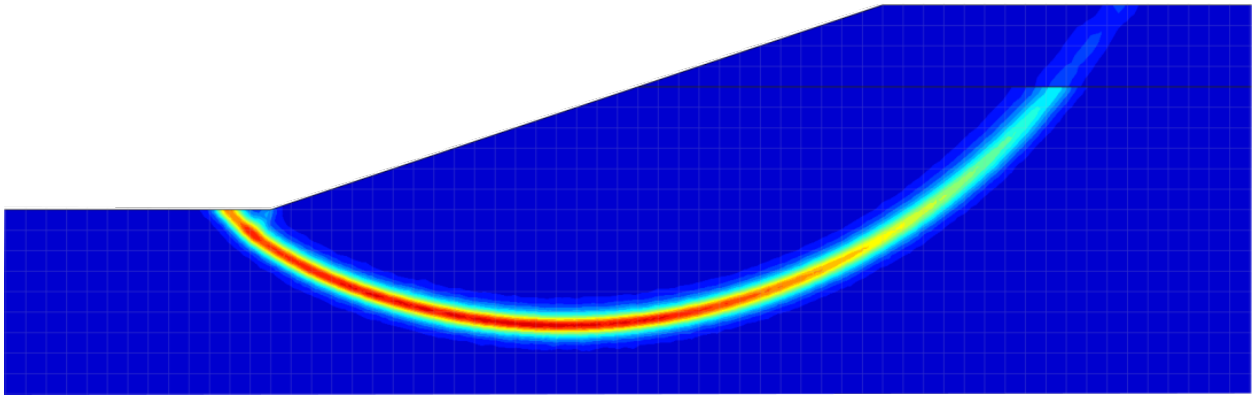

Clay slopes and their stability: An evaluation of different methods

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

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CHALMERS UNIVERSITY OF TECHNOLOGY
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Cover: The shear dissipation along the failure surface in a model of a slope.

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Abstract

This thesis compare and evaluate slope stability and the methods used to determine it. Slope stability is already an important aspect when it comes to infrastructure projects and with the climate changes and increasing amounts of precipitation will this become even more important in the future. Slope stability is today evaluated with a Factor of Safety (FOS) that is determined with numerical computer code in Geotechnical programs. How these programs work will be looked at in detail. The thesis is limited to clay slopes, 2D models and only the ultimate limit state of the slopes is analysed.
A literature study was first conducted to see which methods exist to determine the FOS and to understand the theory behind them. Three geotechnical programs that employs three different methods (Limit Equilibrium Method LEM, Discontinuity Layout Optimisation DLO and Finite Element Limit Analysis FELA) to determine the FOS was then evaluated and compared to each other. First was the methods used to calculate a FOS for one case with a closed-form solution and one case where true solution has been narrowed done to a very small bracket to determine which program had the best accuracy. Then was the result from the method with the most accurate result used as the benchmark to compare the results from the other two methods against for cases that are more complex and realistic. As a final step was the method used as the benchmark used to determine the FOS for a slope with data and parameters from a real case.
Of the three methods was the results from the FELA closest to the closed-form solution and was therefore used as the benchmark method. In the more complex cases was it found that the DLO results overall had a smaller discrepancy compared to FELA solutions than the results from LEM. The program using FELA was also able to model seepage and suction and show how the suction could raise the stability of a slope. In the case based on real data and topography was anisotropy also implemented into the model to show how the FOS could be higher than first expected.
All three methods was able to determine FOS in a satisfying manner and produced results that were comparable to each other. For the more complex models did the FELA combined with the upper and lower bound meshes it uses show that it holds great potential for geotechnical modelling, since it gives the possibility to bracket the solution and thereby give a margin of error for the model. There are some parts in the program using FELA that should be analysed further in more detail, especially when it comes to including the anisotropy of soft clays into models.

Keywords: Slope stability, Slopes, Clay, Limit Equilibrium Method (LEM), Finite Element Limit Analysis (FELA), Factor of Safety (FOS), Slope/w, Optum G2, Limistate:GEO, Discontinuity Layout Optimisation (DLO).

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Hannes Hernvall, Gothenburg, June 2017

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Abbreviations

AUS	Anisotropic Undrained Shear
DLO	Discontinuity Layout Optimisation
DSS	Direct Simple Shear
FE	Finite Element
FELA	Finite Element Limit Analysis
FOS	Factor Of Safety
LA	Limit Analysis
LEM	Limit Equilibrium Method
SLS	Serviceability Limitstate
SR	Strength Reduction
SRF	Strength Reduction Factor
SRM	Strength Reduction Method
ULS	Ultimate Limitstate
vCPU	Virtual processors

Greek symbols

α	Slice base inclination
$\alpha[m^1]$	Model parameter related to air pressure entry
β	Slice base length
γ	Unit weight
γ_{dry}	Unit weight dry
γ_{sat}	Unit weight Saturated
ϕ	Friction angle
ϕ'	Effective friction angle
ω	Geometric parameter

Lower cases

c	Cohesion
c'	Effective cohesion
d	Geometric parameter
f	Geometric parameter
h	Height
n	Model parameter related to the rate at which water is extracted from the soil once the air entry pressure has been exceeded
s	Undrained Shear Strength
s_u	Undrained Shear Strength
s_{uc}	Undrained Shear Strength in triaxial compression
s_{us}	Undrained Shear Strength in simple shear
s_{ue}	Undrained Shear Strength in triaxial extension
x	Geometric parameter

Upper cases

D	Concentrated load point
N	Base normal
Q_γ	Loading parameter
R	Geometric parameter
U	Pore water pressure
W	Slice weight

1
Introduction

1.1 Background

Slopes and their stability have for a long time been a big part of geotechnical engineering, this includes both natural and man-made slopes used for different kinds of infrastructure. To complicate things further many of these slopes are also consisting of clay. The Geological Survey of Sweden estimates that landslides causes damages for around 200 MSEK for the society each year (Geological Survey of Sweden, 2017).

The climate changes will make the stability for these slope even more important since the increasing precipitation decreases the stability and increases the risk for landslides. This can cause problems in the Western part of Sweden which is an area that already is prone to landslides as the majority of the landslides in Sweden have occurred there. For example there have been several large landslides in the past along and close to the Göta river, two of these happened in Surte (MSB, 1950) and Tuve (MSB, 1977).

An important tool to determine the slope stability is geotechnical modelling, today often performed using computer codes. On the market today there are several numerical codes for geotechnical engineering that have different models and methods implemented in them to solve geotechnical problems. Since these programs uses different methods they have different advantages and disadvantages that an engineer has to take in to consideration when calculating slope stability. Methods based on limit equilibrium have for a long time been the dominating choice to use for determination of the stability and factor of safety (FOS) for slopes. While newer methods to determine slope stability such as the Discontinuity Layout Optimisation (DLO) (Smith and Gilbert, 2007) and finite element based limit analysis (Sloan, 2012) have been developed, they still do not have the same widespread usage as methods and programs based on limit equilibrium.

Several of the new geotechnical numerical codes used to determine the stability of slopes are not designed to calculate smaller deformations associated with serviceability limit state (SLS) such as settlements and creep. These types of deformation can of course already cause problems but they are also an indication that a slope is at risk to collapse. Since a collapse of a slope is a failure event it is the ultimate limit state (ULS) that is of interest to determine the stability and safety margin to failure.

1.2 Aim & Objectives

The aim of this Thesis is to get a deeper understanding for the different aspects of clay slopes and for the methods and programs used to analyse them.

- Compare limit equilibrium, discontinuity layout optimisation and finite element methods based on limit analysis for slopes of different complexity.
- Analyse the influence of the different layers in a slope.
- See how the dry crust and the soil anisotropy of a slope influence the stability.

1.3 Limitations

- The modelling of the slopes in the programs will only be done in 2D.
- Only the Ultimate Limit State of the slope will be analysed.

2
Limit analysis

2.1 Limit equilibrium

2.1.1 Slice methods

The first method to determine slope stability based on Limit Equilibrium was developed by Wolmar Fellenius in 1927 and is referred to as either the Ordinary, Swedish or Fellenius method (GEO-SLOPE, 2012). Fellenius calculated stability by assuming a circular slip surface and divide the slope into several slices shown i Figure 2.1. Then he calculated the resisting force and driving forces for each slice and summarized both values for the slices and divided the resisting forces with the driving forces to get a ratio which is called the factor of safety (FOS):

$$
\begin{aligned}
Factor\ of\ safety &= \frac{\sum Resisting\ forces}{\sum Sliding\ forces} = \frac{\sum c\beta + N\tan\phi}{\sum W\sin\alpha} \\
c &= \text{Cohesion} \\
\beta &= \text{Slice base length} \\
N &= \text{Base normal} \\
\phi &= \text{Friction angle} \\
W &= \text{Slice weight} \\
\alpha &= \text{Slice base inclination}
\end{aligned}
\tag{2.1}
$$

The main disadvantage with this method is that it does not take into account all the interslice forces, which can lead to unrealistic FOS that is off by as much as 60% compared to other methods (Fredlund and Krahn, 1977). For this reason it is not really used today except for hand calculations to get a basic understanding for a project or for educational purposes. After the Ordinary method was developed a multitude of other methods have been developed that give a more realistic FOS. However, for most of them the basis for how the FOS is calculated is similar to the ordinary method.

There are methods that calculate factor of safety from either the force equilibrium or the moment equilibrium which depending on the circumstances can give an acceptable FOS in one case but not the other. Newer methods, for example Spencer (1967), calculate both the force and moment equilibrium too determine a factor of safety that satisfies both. A method that takes both equilibriums into account when determining slope stability is a requirement according to Eurocode 7 (Bond, Schuppener, Scarpelli, and Orr, 2013). The different methods can also be divided into

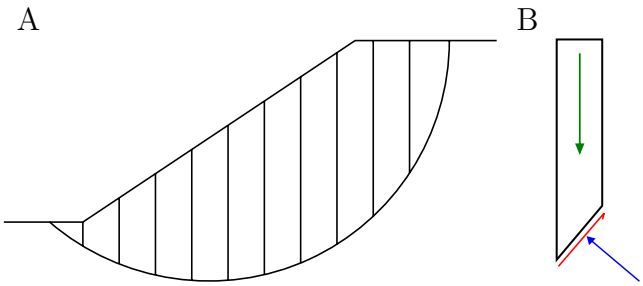

Figure 2.1: A: Slope with circular slip surface divided into several slices.
B: The forces acting on a slice that are taken into account with the Ordinary method.

the categories simplified or rigorous. Methods that are classified as simplified only take either force or the moment equilibrium into account while the one's classified as rigorous take both equilibriums into account when calculating the factor of safety. It is important not to be misguided by the name of the categories and believe that a rigours method always will give an accurate factor of safety, it will vary from case to case.

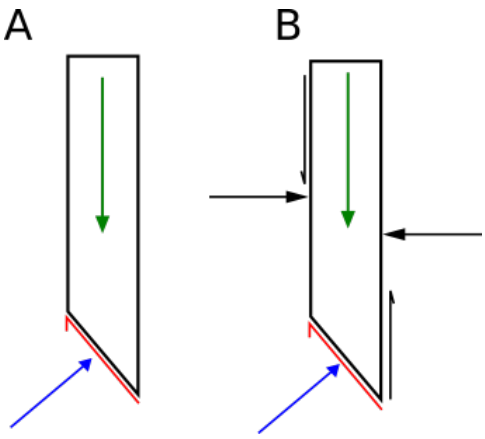

Figure 2.2: A: The forces taken into account for a slice with Fellenius method. B: The forces taken into account for a slice with Spencer's method.

The General limit equilibrium method was developed by Fredlund and Krahn (1977) and it incorporate the elements from other methods in a general form with one equation for the horizontal force equilibrium Equation 2.2 and another for the moment equilibrium Equation 2.3.

$$f_f = \frac{\sum(c'\beta\cos\alpha + (N - u\beta)tan\phi'\cos\alpha)}{\sum N - \sin\alpha - \sum D\cos\omega} \tag{2.2}$$

$$f_m = \frac{\sum(c'\beta R + (N - u\beta)R\tan\phi')}{\sum Wx - \sum Nf \pm \sum Dd}$$

$$\begin{aligned} c' &= \text{Effective cohesion} \\ \phi' &= \text{Effective angle of friction} \\ U &= \text{Pore-water pressure} \\ N &= \text{Slice base normal force} \\ W &= \text{Slice weight} \\ D &= \text{Concentrated point load} \\ \beta,\ R,\ x &= \text{Geometric parameters} \\ f,\ d,\ \omega &= \text{Geometric parameters} \\ \alpha &= \text{Inclination of slice base} \end{aligned} \tag{2.3}$$

The main disadvantage with methods based on limit equilibrium methods are that they require either a known or at least an assumed failure surface to be used. How modern methods determine the failure surface will be looked into next.

2.1.2 Slip surface

The first shape used for the slip surface in limit equilibrium calculations was circular, later also a logarithmic spiral was used when calculating the FOS. The difference in FOS between those surfaces have been found to be negligible for any practical usage (Duncan, 1996). Another example of the shape for slip surfaces is a straight line if the slope is very steep or a composite between a circle and line if there is a very hard or soft layer (a bedrock or a thin weak clay layer for example) in the slope that can be a part of the slip surface. It has been found that unless there are geological constraints it can be assumed that the slip surface will be circular in shape (Duncan, 1996). A common feature of the failure surfaces are that they are convex in their shape (Chen and Lau, 2014).

It is somewhat of a problem for limit equilibrium methods that they require a known slip surface to be used and some of the methods are even limited to a specific geometry as well. This can lead to an arbitrariness in the process in many cases since the slip surface has to be assumed. Modern geotechnical programs can at least partly mitigate this since they either have an iterative process that will refine the result in several steps to give a more realistic slip surface or test a lot of different slip surfaces and subsequently choose the one with the lowest FOS as the critical. Programs that are based on limit equilibrium can have a problem to determine a slip surface if it happens not to have a convex shape. During the last decade several different algorithms and methods have been developed to determine non-circular slips surfaces. These methods are based on either optimisation techniques or on Monte Carlo simulation, a couple of examples of these are the Artificial Bee Colony (Kang, Li, and Ma, 2013) and the Meeting Ant Colony Optimisation (Gao, 2016).

2.2 Limit Analysis

Limit analysis is in very simplified terms a combination of several methods to discover the actual collapse load for a body. It has the advantage compared to limit equilibrium method that it does not require a known or at least assumed critical slip surface to be used for calculations. The limit analysis is based on plasticity theorems that was developed in the early 1950s by Drucker and Prager. Two different theorems are used for the limit analysis, one based on a statically admissible stress field that give a lower bound solution and the other based kinematically admissible velocity field that gives an upper bound solution. Together they form a bracket in which the closed solution is located, so the upper and lower bound should be as close to each other as possible (Chen and Lau, 2014).

2.2.1 Lower-bound formulation

A statically admissible stress field (Chen and Lau, 2014) is one that has the following satisfied:

- equilibrium
- the stress boundary conditions
- the yield criterion

When a finite element limit analysis is conducted there are two equilibriums that need to be satisfied. The first one is for the stresses in all the elements of a mesh which most satisfy the equilibrium equations, this is referred to as the continuum equilibrium. The second equilibrium is for the stress discontinuities. A stress discontinuity is located between the nodes of different elements that share the same coordinates. For a discontinuity to be in equilibrium it needs its normal and tangential stresses to be equal in nodes on both sides. The tangential normal stress in discontinuities are however allowed to vary, so nodes that have the same coordinates do not need to have the same stress in them (Lynamin and Sloan, 2002a).

The conditions for stress boundaries are achieved when the stresses in a boundary node are equal to the prescribed surface tractions. Boundary conditions are often defined as the normal and tangential components of a stress vector located at a boundary edge. For a linear finite-element model the constraints must be applied to all edges with specified surface stresses to ensure that the boundary conditions are exactly satisfied (Sloan, 2012).

The yield condition will be satisfied as long as the yield function $f(\sigma)$ is convex and the stresses in the element vary linearly as long as each node is imposed with the inequality constraint $F(\sigma^i) \leqslant 0$ (Lynamin and Sloan, 2002a).

2.2.2 Upper-bound formulation

A kinematically admissible velocity field (Chen and Lau, 2014) is one that satisfies the following:

- velocity boundary conditions
- the plastic flow rule

Velocity boundary conditions are like the lower-bound stress boundary conditions often defined as the normal and tangential components of a velocity vector along a boundary edge. They must be satisfied in all boundary nodes with prescribed velocities(Lynamin and Sloan, 2002b).

The constraints set by an associated flow rule must be satisfied by the velocity field if it is going to be kinematically admissible. Similar to the equilibrium requirement for the lower-bound is the flow rule applied to both the continuum and the discontinuities between the elements. Another similarity is that the velocity can vary in nodes with the same coordinates since it is allowed to jump in the normal and tangential direction of discontinuities (Lynamin and Sloan, 2002b).

2.2.3 Non-linear optimisation problem

After both meshes and their constraints have been created and determined can both the upper- and lower-bound be solved as non-linear optimisation problems. When solving the lower-bound optimisation problem the load that is carried by unknown stresses and body forces is maximised under the constraints assembled. The upper-bound limit load is calculated, by minimising the internal power dissipation from plastic shearing minus the work done by the external forces. The bounds of a limit load should be as close to each other as possible to increase the accuracy (Sloan, 2012).

2.3 Discontinuity Layout Optimisation

Discontinuity Layout Optimisation (DLO) is another method to conduct limit analysis. Since it is based on a kinematically admissible velocity field it is an upper bound solution (Smith and Gilbert, 2007). The goal of the procedure is to discover the subset of discontinuities which require the least amount of internal energy dissipation. To get a understanding for the DLO can the discontinuities be seen as the bars in a truss problem.

The DLO process is divided into four steps, these are shown in Figure 2.3. In the first step a soil body and the forces acting on it is established. The body is then discretized with nodes in the second step, it is worth noticing that the placement of the nodes does not have to be uniform and can be adapted to get a better result. Discontinuities are then created to connect the nodes to each other in the third step. For smaller bodies there can be discontinuities between all the nodes while on larger bodies limiting length for the discontinuity is introduced to reduce the number of discontinuities somewhat, other limitations can also be imposed. In the fourth and final step the critical discontinuities are determined through optimisation to solve how the soil body fails (Smith and Gilbert, 2007).
Limit analysis performed with DLO is a bit less complicated than with finite element since the discontinuities are symbolised by the lines between the nodes instead of the edges between the elements in a mesh (Smith and Gilbert, 2007). This gives the advantage of a larger search space for the critical mechanism, a smaller search space that is restricted to the edges of the elements can lead to poor computed predictions for the collapse load.

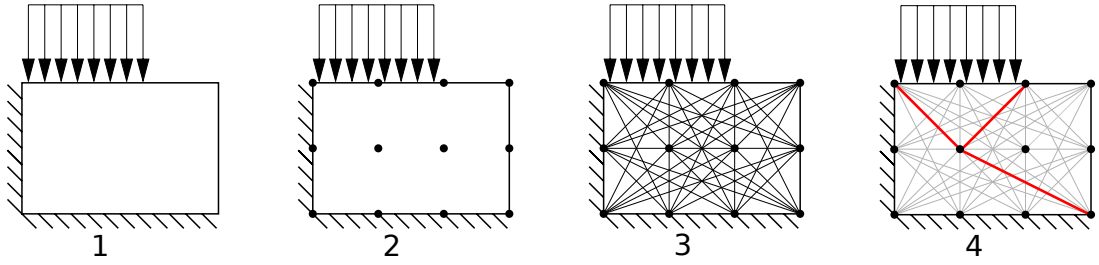

Figure 2.3: The four steps in the Discontinuity Layout Optimisation process. Figure is based on Figure in Gilbert, Smith, Haslam, and Pritchard (2010).

2.4 Strength Reduction

Strength Reduction is another type of method to determine slope stability that works by reducing the soils strength parameters and was first presented by Zienkiewicz (1975). A Strength reduction factor (SRF) is introduced to divide the friction angle ϕ and the undrained strength s_u, so a reduction of both parameters occur simultaneously see Equations 2.4 and 2.5. The SRF is incrementally increased until the slope collapse and the SRF at the moment of collapses is then used as the FOS. Strength reduction is used in conjunction with finite elements and has the advantage that it does not require a known slip surface.

$$s_{u.red} = \frac{1}{SRF} s_u \tag{2.4}$$

$$\phi_{red} = \arctan\left(\frac{1}{SRF}\tan\phi\right) \tag{2.5}$$

2.5 Software description

The methods to determine slope stability described in the thesis are implemented in three different geotechnical programs. A short description of each program and which method they employ to calculate the FOS as well as some of their other mechanisms will be given here.

2.5.1 Slope/W

Slope/W is one of the first programs that was available to determine slope stability, the first version was released in 1977 and was developed at the University of Saskatchewan by D.G. Fredlund (GEO-SLOPE, 2012). The program uses limit equilibrium methods to calculate FOS and most of the existing limit equilibrium methods are available to be used depending on which method is needed or preferred. In the new versions several methods are available to determined the slip surface, these include grid with radius, entry and exit, block specified and fully specified (GEO-SLOPE, 2012). Another feature in the newer version is the possibility to optimise the slip surface. In simplified terms this is done by dividing the

slip surface with the lowest FOS into several segments of straight lines which then are adjusted to further minimise the FOS (GEO-SLOPE, 2012).

2.5.2 Limitstate:GEO

Limitstate:GEO was developed at Sheffield university, UK and is based on discontinuity layout optimisation (Smith and Gilbert, 2007). There are a couple of different methods to determine a FOS for a slope with Limistate:GEO. They are all based on an adequacy factor that is applied to both the soil strength parameters or the load. The adequacy factor for loads is applied for loads that are placed on top of a soil body or on the soils own self weight. When the adequacy factor is applied to a load it is multiplied until the slope in this case collapses. For the strength parameters for a soil the opposite is done, the strength parameters are divided by the adequacy factor until the parameters are reduced enough to cause the slope to fail (LimitState, 2016).

2.5.3 Optum G2

Optum G2 is based on the research that was conducted at University of Newcastle in Newcastle, Australia. It is based on finite element limit analysis and at the moment has the unique ability to give both an upper and lower bound solution to geotechnical problems. Optum G2 has a couple of different method to determine a FOS for a slope. The first is just to multiply the load applied until a collapse is caused. The second method causes a collapse by increasing the gravity. A third method is with strength reduction in which the soil strength parameters are reduced until a failure occur.

3
Basic models

3.1 Introduction

All three programs need to be evaluated to see how they perform and how accurate their solutions are for different geotechnical problems. The basic evaluation of the methods will be done by using a problem with a known closed form analytical solution and one problem where the true solution has been narrowed done to a limited span. After this is done more complex and realistic geotechnical problems will be analysed.

3.2 Closed-form solution

The first model will be a simple footing on a homogeneous soil with a distributed load with the same width as the footing that has a closed-form solution that was calculated by Prandtl in 1920 (Merifield, Sloan, and YU, 1999). Three different cases with different unit weight γ and undrained strength s will be tested for this model, the parameters for for each case are and their solution are specified below in Table 3.1. It is only Case 1 that is a true Prandtl problem with the solution $2 + \pi$, but since the unit weight is so low in the other two cases and the influence by it so small is $(2+\pi)s_u$ close enough to be used as the solution for Case 2 and 3.

Table 3.1: The parameters for the soil strength and the solutions for the different cases.

Case	γ	s_u	Solution
1	0	1	$2+\pi$
2	1	1	$(2+\pi)s_u$
2	1	10	$(2+\pi)s_u$

3.2.1 Results closed-form solution

The result for the Prandtl footing can be seen in Table 3.2. Slope/W's LEM was not included in the first two cases since it was not possible to use a material with zero unit weight in the calculations. It was also only compared to the lower number of nodes and elements since the program is not really optimised for this kind of calculation and it would be unfair to compare to the other two programs. Limitstate:GEO DLO was inside 1% in all five cases and was off by less then 0.3%. The results for Optums finite elements limit analysis (FELA) with lower and upper-bound are inside 2% of the solution for the three cases with 2000 elements, as expected the results for the two cases with 10000 elements are more accurate by being off by

less then 1%. It is worth to notice that the average between Optums FELA upper and lower-bound are off by less then a tenth of a % in all five cases. Furthermore for both Limitstate:GEO's DLO and Optums upper-bound solution are the results above one for all five cases, which should be the case since both are upper-bound solutions.

Table 3.2: Results and how many percent the cases differ from the closed-solution for a footing on soils with different γ, s_u and nodal/element density.

	γ=0 & s_u=1			
Nodes/Elements	2000	% Disparity	10000	% Disparity
LEM	-	-	-	-
DLO	5.189	0.9	5.156	0.3
FELA Lower	5.068	-1.4	5.112	-0.6
FELA Upper	5.224	1.6	5.161	0.4
FELA Mean	5.146	0.1	5.1365	-0.1
	γ=1 & s_u=1			
Nodes/Elements	2000	% Disparity	10000	% Disparity
LEM	5.548	7.9	-	-
DLO	5.189	0.9	5.156	0.3
FELA Lower	5.053	-1.7	5.114	-0.5
FELA Upper	5.224	1.6	5.159	0.3
FELA Mean	5.138	–0.01	5.1365	-0.1
	γ=1 & s_u=10			
Nodes/Elements	2000	% Disparity		
LEM	55.6	8.1		
DLO	51.89	0.9		
FELA Lower	50.422	-1.9		
FELA Upper	52.272	1.7		
FELA Mean	51.347	-0.1		

3.3 Vertical cut

The second model will be a Tresca vertical cut which characteristics is based on equation 3.1, where γ is the unit weight, s_u is the undrained strength, h is the height and Q_γ is a loading parameter.

$$Q_\gamma = \frac{\gamma h}{s_u} \tag{3.1}$$

Although a closed-form solution for Q_γ is not yet available Pastor, Thai, and Francescato (2003) was able to narrow it to a bracket of 0.01 with the lower boundary at 3.772 and the upper boundary at 3.782. The mean of these bounds will be used to calculate the maximum height for three cases with different s_u and γ. These vertical slopes and their soil parameters will then be inserted and simulated in all three programs to get a FOS to compare. The FOS should be as close to one as possible, since the height of the slope is maximised. Any decrease or increase in strength parameters or loads lead to a collapse.

3.3.1 Results vertical cut

Three cases with different unit weights and undrained strength were evaluated. The results are presented in Table 3.3 with the shape of the slip surface displayed in Figure 3.1. Optum G2 was the program that performed best and was less then 1% off the true solution for both its upper and lower bound solution in all three cases. The average of the upper and lower-bound solutions ended up very close to the true solution, rather than to be off by 10ths of a percent it was down to be off by 100ths of a percent in two of the cases and was exactly on the solution in the third case. Limitstate:GEO's DLO also performed good and was within the 1% mark for two of the cases and just over it (1.1%) in the other. Slope/W did not perform as well as the two other programs but was only off by a maximum of 4.9% from the true solution.

Table 3.3: The FOS for all three cases and how many percent they differ compare to the expected result for all three vertical slopes with maximised height for the specified γ an s_u. Density of nodes/elements are 2000 for all three slopes.

Height	5.01m s_u=22 & γ=16.3		8.09m s_u=30 & γ=14		10.07m s_u=48 & γ=18	
		% disparity		% disparity		% disparity
LEM	1.048	4.8	1.051	5.1	1.046	4.6
DLO	1.011	1.1	1.009	0.9	1.008	0.8
FELA Lower	0.992	-0.8	0.994	-0.6	0.993	-0.7
FELA Upper	1.007	0.7	1.007	0.7	1.007	0.7
FELA Mean	0.9995	-0.05	1.0005	0.05	1	0

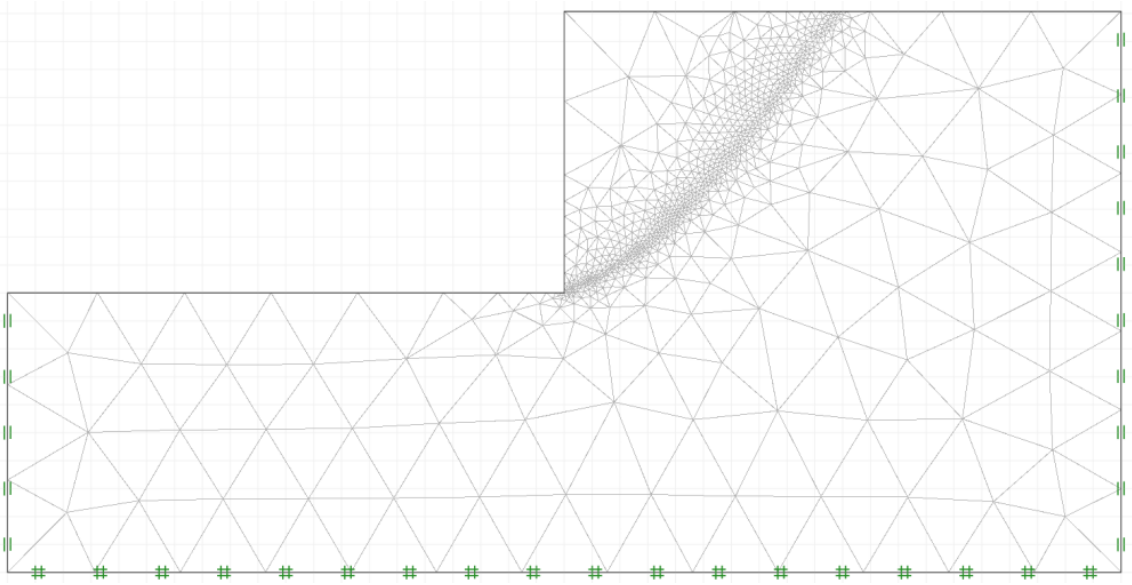

Figure 3.1: A picture of a lower-bound mesh and the failure surface for a case in Optum G2.

3.4 Discussion

Slope/W LEM performed worst in the five cases for both models used. This is probably partly due to the nature of the two models that were chosen for the basic evaluation of the three programs, since only the vertical slope is similar to cases that Slope/W is designed to evaluate. The FOS deviates less for the solution for the vertical slope which is similar to what the program is designed for than the solution for the Prandtl footing. It also highlights the limitation of the process that Slope/W uses to determine the slip surface for calculation of the FOS. It both requires understanding of the process and sometimes even requires luck to get a good result.

Optums FELA and Limistate:GEO DLO performed well in all the cases for both models and the results were very close to the true solution. As expected the accuracy of the solutions increased for both programs when the number of nodes and elements was increased in the Prandtl footing model. It is worth noticing that the upper-bound solutions (Limitstate:GEO and Optums upper) are above and Optums lower-bound is below the solution just as they should be. The most interesting finding from the results of the basic models is that the average of Optums upper- and lower-bound solution is very close to the solution for both the prandtl footing and the vertical slope. This indicates that the mean of Optums solutions can be treated as a quasi rigorous solution for further benchmarking. Because of this the mean of Optums FELA upper- and lower-bound will be used as a benchmark to which the other two programs methods will be compared for the next parts of this study when more complex slope models will be evaluated.

4

Complex models

4.1 Slope description

Instead of a vertical cut will slopes with an inclination be analysed and the complexity of the slopes will be increased in several steps. For the analysis the slope has three different inclinations 1:3, 1:2 and 1:1 of the slope will be used for the analysis. The top of slope is fixed on that same location regardless of the inclination, only the toe is moved to to get the desired inclination. The first case is a slope that consists of a single type of soft clay (Soil A) with an undrained shear strength that increases with depth, a model for Case 1 with 1:3 inclination can be seen in Figure 4.1. In the next case the top of the slope is replaced with another type of clay (Soil B) that is softer than Soil A. A water table is added in the third case, the water tables depths under the surface varies between one and three meters. In the final case a dry crust with a thickness of two meters added over the entire length of the slope model. What the differs between the cases included can be seen in Table 4.1.

Table 4.1: Overview of what is evaluated in the four cases.

Case	Inclination	Two layers	Water table	Dry crust
1	Yes	-	-	-
2	Yes	Yes	-	-
3	Yes	Yes	Yes	-
4	Yes	Yes	Yes	Yes

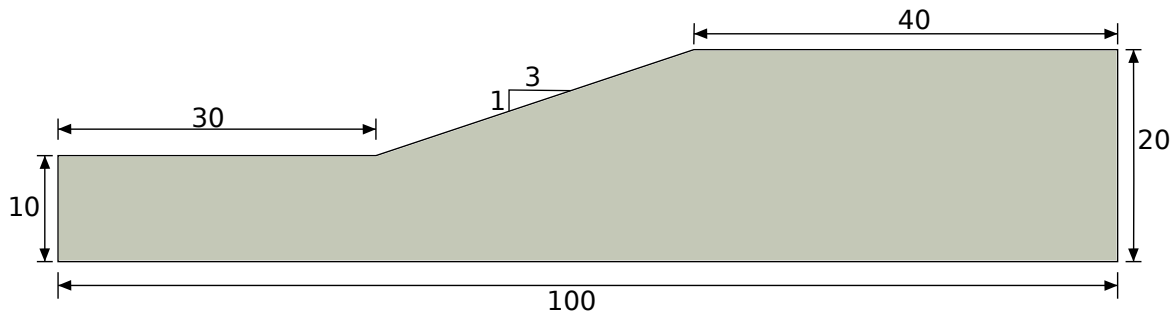

Figure 4.1: Figure of the basic slope at the inclination of 1/3 and the dimensions used for the models.

4.2 Soil parameters

The soil parameters used for all four cases are listed in Table 4.2. Soil A's s_u increases with depth and the start datum is set to 20 meter which is at the top of the slopes. The start datum remains there even when the top four meters of the slopes are changed to Soil B, so the lowest value of s_u is 22kPa in Case 2,3 and 4 instead of 16.8kPa as it is in Case 1.

Limitstate:GEO and Slope/W use Mohr-coulomb based failure envelopes for the analyses where undrained shear strength (s_u) and effective friction angle (ϕ) are used as the strength parameters. For Optum a Tresca based soil model is used to be able to perform a short term analyse in satisfying way. Tresca has many similarities to Mohr-coulomb, but it does not have the option to included a friction angle but has s_u as its only strength parameter. Optum has several different variables and properties, for example the E-modulus or the hydraulic model that can be adjusted depending on what is needed in the model. But since the other two program do not have similar options Optums standard values for the soil models always are used unless otherwise stated. For Optum are 5000 elements used in the mesh for the analyses, five adaptive iterations of mesh are performed for the limit analysis while only three adaptive iterations was used for the strength reduction since that required significant longer computing time. Slope/W used entry and exit method for to determine the slip surface together with slip surface optimisation for the analyses. Limitstate:GEO used 1000 nodes to determine the critical slip surface.

Table 4.2: The soil parameters used for the models in the evaluated programs.

Soil	γ_{dry} (kN/m^3)	γ_{sat} (kN/m^3)	s_u(kPa)	ϕ (°)
Soil A	16	17	16.8 +1.3/m	-
Soil B	16	17	10	-
Dry Crust	19	19	50	30°

4.3 Results for Case 1 to 4

4.3.1 Case 1

The calculated Factor of Safety (FOS) for Cases 1 are listed in Table 4.3. Slope/W LEM provided the most conservative FOS for all three inclinations, it also had the largest disparity compared to Optums limit analysis. Limitstate:GEO DLO results were the highest for all three inclinations and was above Optums limit analysis for all three inclination, this is in line with the earlier test results and that Limitstate:GEO uses an upper bound method. The FOS from limit analysis and strength reduction in Optum was basically equivalent, with only minor disparity compared to each other. Overall the general trend was for the FOS as expected, with the steepest inclination (1:1) having the lowest FOS and that it is higher for the lower inclinations.

Table 4.3: Result for Case 1; Disparity % is how much the results differ from the mean of Optums finite element limit analysis.

	Inclination	1:1	1:2	1:3
Finite Element				
Limit Analysis	Lower	0.9562	1.192	1.36
	Upper	0.964	1.198	1.365
	Mean	0.9601	1.195	1.3625
Strength Reduction	Lower	0.9576	1.19	1.36
	Upper	0.9665	1.198	1.366
	Mean	0.9621	1.194	1.363
	Disparity %	+0.2	-0.08	+0.04
DLO	Load	0.9997	1.244	1.411
	Strength	0.9997	1.244	1.411
	Disparity %	+4.12	+4.1	+3.56
LEM		0.932	1.102	1.268
	Disparity %	-2.93	-7.78	-6.94

4.3.2 Case 2

The introduction of a second soil layer on top of the first soil lead to a decrease of the FOS. This has to be expected since the second layer consists of Soil B which has a lower undrained strength then Soil A. As can be seen in Table 4.4 the general trends continues with Slope/W's LEM having the largest disparity on average compared to Optums FELA mean and Limitstate:GEO's FOS generally are above the limit analysis mean. Optums strength reduction appears to give a slightly higher FOS than the limit analysis on average, but since the differences between them are so small in these cases it probably does not have any practical significance.

Table 4.4: Results for Case 2 with two layers of soil.

	Inclination	1:1	1:2	1:3
Finite Element				
Limit Analysis	Lower	0.884	1.142	1.324
	Upper	0.8913	1.148	1.329
	Mean	0.88765	1.145	1.3265
Strength Reduction	Lower	0.8849	1.141	1.324
	Upper	0.8982	1.15	1.33
	Mean	0.8916	1.1455	1.327
	Disparity%	+0.44	+0.04	+0.04
DLO	Load	0.8849	1.152	1.33
	Strength	0.8849	1.152	1.33
	Disparity%	-0.31	0.61	0.26
LEM		0.902	1.06	1.237
	Disparity%	+ 1.62	-7.42	-6.75

4.3.3 Case 3

The introduction of a water table in Case 3 lead to an overall reduction in the FOS for all three programs, which has to be expected. Nothing unexpected really happened for the slopes in this case, the only thing that might be worth to notice is that Slope/W's disparity is smaller here compared to the first two cases.

Table 4.5: Result for Case 3 with two soil layers and a water table.

	Inclination	1:1	1:2	1:3
Finite Element				
Limit Analysis	Lower	0.8529	1.097	1.267
	Upper	0.8587	1.101	1.272
	Mean	0.8558	1.099	1.2695
Strength Reduction	Lower	0.8515	1.096	1.259
	Upper	0.8621	1.102	1.271
	Mean	0.8568	1.099	1.265
	Disparity%	+0.12	0	-0.36
DLO	Load	0.8666	1.117	1.287
	Strength	0.8666	1.117	1.287
	Disparity%	+1.26	+1.64	+1.38
LEM		0.899	1.059	1.237
	Disparity%	+5.05	-3.64	-2.56

4.3.4 Case 4

Case 4 has several interesting results which are summarised in Table 4.6. The first is that Optum was unable to calculate a lower bound for both limit analysis and strength reduction for the slope with an inclination of 1:1. The reason for this is that Optum could not form a stable lower bound mesh to perform the calculations. Because of this there is no disparity percentage for inclination 1:1. Limitstate:GEO's adequacy factor's for load and strength have in the cases before been equal to each other but here they differ with a relatively large margin.

Compared to the results in Case 3 the FOS increases with the introduction of a dry crust, except for the slope with 1:3 inclination analysed with Optum. The results for Optums limit analysis for that slope was below the corresponding results for Case 3 which is surprising. While the strength reduction FOS did increase from 1.265 in Case 3 to 1.266 in Case 4 the gain in FOS is still smaller than expected.

Table 4.6: Results for case 4 with two soil layers, water table and a dry crust.

	Inclination	1:1	1:2	1:3
Finite Element				
Limit Analysis	Lower	-	1.14	1.252
	Upper	1.015	1.143	1.258
	Mean	-	1.1415	1.255
Strength Reduction	Lower	-	1.15	1.262
	Upper	1.02	1.153	1.27
	Mean	-	1.1515	1.266
	Disparity%	-	+0.88	+0.88
DLO	Load	0.9533	1.213	1.361
	Strength	0.9705	1.173	1.306
	Disparity%	-	+2.76	+4.06
LEM		1.07	1.139	1.293
	Disparity%	-	-0.22	+3.03

4.4 Long term drained

For Optum all four cases are also evaluated in the long term perspective. The FOS is either equal or very close to the short term undrained FOS for most of the cases. The only exception is in case 4 for the slope with inclination 1:3 where the FOS increased for both the limit analysis and the strength reduction. The increased FOS in Case 4 meant that the FOS is in line with the general trend observed for all cases.

Table 4.7: The mean FOS for all four cases evaluated with both limit analysis and strength reduction with finite elements in the long term drained condition.

Mean FE Inclination		1:1	1:2	1:3
Case 1 Simple	Limit Analysis	0.9606	1.195	1.3625
	Strength Reduction	0.961	1.194	1.363
Case 2 Two Layers	Limit Analysis	0.8878	1.145	1.3265
	Strength Reduction	0.8916	1.1455	1.3275
Case 3 Water table	Limit Analysis	0.8555	1.0985	1.268
	Strength Reduction	0.8568	1.0995	1.265
Case 4 Dry crust	Limit Analysis	0.9798	1.2255	1.3515
	Strength Reduction	0.9817	1.196	1.3195

4.5 Discussion

There is a general trend for the FOS that remains stable through all four cases regardless of the method or inclination of the slope. The trend is that the FOS decreases in two steps in Case 2 and 3 to increase again in Case 4, but without reaching the same levels as in Case 1. These changes in FOS for the four cases are expected. In Case 2 for example the second layer consists of a clay that is weaker than the first clay so it is logical that the FOS is lower than in Case 1. The only exception from this trend was in the short term version of Case 4 for the slope with inclination 1:3 for the finite element evaluation were both the limit analysis and strength reduction delivered a lower FOS than expected. Why this happened is not entirely clear but it might have to do with the Tresca soil models used.

As mentioned earlier was Optum unable to create a stable lower bound finite element mesh for the slope with 1:1 inclination in Case 4, so the FOS for that slope is unknown. However, that a stable mesh cannot be formed might give an indication that the FOS could be on the lower side and possibly below one and therefore possible unsafe, this is partly supported by the fact that the FOS for the upper bound of that slope are just above one. So when a mesh cannot be formed in Optum it should been seen as an indicator that the slope can be unstable, even if there is no FOS to verify it and should therefore be treated with caution.

5

Suction and Anisotropy

5.1 Suction and Seepage

5.1.1 Introduction

Water is going to affect the factor of safety (FOS) for a slope and in Case 3 and 4 this was taken into consideration with a stationary water table. Since the water table was placed just below the surface it can be seen as a conservative or worst case but it is not necessarily a realistic case. To make the slope models more realistic seepage and the suction of the soil need to be integrated into the models to see how FOS is influenced. Optum can model seepage by adding a pressure head to one of the vertical boundaries in the model and a water table to mark what level the water should flow to in whatever path it wants. Suction can be modelled in Optum by employing the Van Genuchten hydraulic model together with Bishops option for effective stress. As in the earlier cases three inclination will be used and the results will then be evaluated against as reference slope which corresponds to Case 2 in the earlier sections.

5.1.2 Parameters and layout

Two parameters needs to be specified to use the Van Genuchten hydraulic model (K Krabbenhoft, 2016):

- $\alpha[m^{-1}]$ = model parameter related to air pressure entry
- n = Model parameter related to the rate at which water is extracted from the soil once the air entry pressure has been exceeded

For this case is n set to 1.50 and α to 0.9, which are values for the parameters that are specified for clay in Optums material manual. A pressure head is placed at the right vertical boundary of the model and set to 10 meters and a water table is placed four meter below the slopes toe until it reaches the models left vertical border as shown in Figure 5.1. The two soils that were used for Cases 1 to 4 will be used again with exception that instead of a Tresca based soil model a Mohr-coulomb based be used to able to simulate the suction. All soil parameters will remain the same for Soil A and B with the exception that a friction angle (ϕ) of 10 degrees will be added. 5000 elements and five adaptive iterations are used for the model. The dimensions of the slopes are the same as in previous chapter.

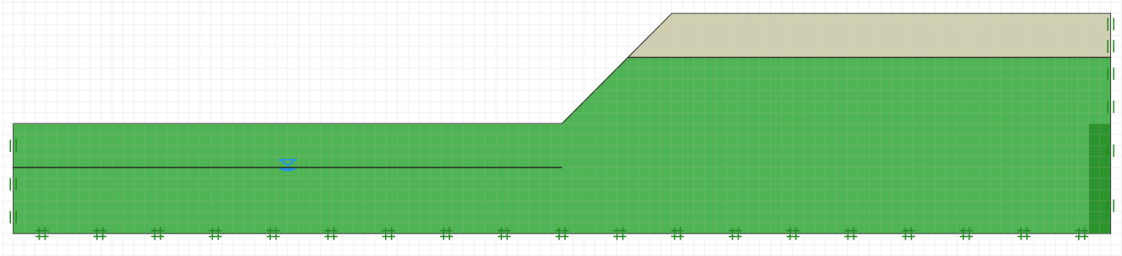

Figure 5.1: Layout of the slope with 1:1 inclination with water table and pressure head included.

5.1.3 Results seepage and water

With the introduction of suction and seepage did the FOS increase for all three slopes in both the short and the long term. The increase in FOS was significant for all three slopes in the long term with increments between 30 and 80 % in FOS, the results are shown in Table 5.1.

Table 5.1: The FOS for the slopes with seepage and suction compared to a reference slopes in both the short and long term.

	1:1		1:2		1:3	
Short term	Ref	Suction	Ref	Suction	Ref	Suction
Lower	1.225	1.321	1.616	1.696	1.94	1.96
Upper	1.235	1.336	1.626	1.706	1.961	1.968
Mean	1.23	1.3285	1.621	1.701	1.9505	1.964
% Increase	8		4.9		0.7	
Long term						
Lower	1.357	1.75	2.447	3.636	4.396	7.935
Upper	1.369	1.798	2.47	3.721	4.454	8.13
Mean	1.363	1.774	2.4585	3.6785	4.425	8.0325
% Increase	30.2		49.6		81.5	

5.2 Anisotropy

5.2.1 Introduction

The property that is referred to as anisotropy here is the anisotropic shear strength of a soil. This means that the soil strength varies depending on the loading acting on the soil body. To be able to take anisotropy into account Optum has been fitted with a specific soil model that is named Anisotropic Undrained Shear (AUS). With AUS the desired shear strength ratios and strain percentage for stiffness parameters for the soils can be inserted, everything else is the same as in Optums Tresca soil model. In Optum the anisotropic strength is governed by two ratios, the first is between the

undrained shear strength in triaxial extension (s_{ue}) and triaxial compression (s_{uc}). The second ratio is between undrained shear strength in simple shear (s_{us}) and s_{uc}. These two ratios have some limitations which are shown in Equations 5.1 and 5.2.

$$0.5 < \frac{s_{ue}}{s_{uc}} < 1 \tag{5.1}$$

$$\frac{4}{9}\left(1 + \frac{s_{ue}}{s_{uc}}\right) \leq \frac{s_{us}}{s_{uc}} \leq \frac{1}{2}\left(1 + \frac{s_{ue}}{s_{uc}}\right) \tag{5.2}$$

Equation 5.1 lower bound is the lowest the s_{ue}/s_{uc} ratio can be and still have a convex yield surface. The upper limit is due to Optum internal mechanisms where s_{uc} need to be at least equal or bigger than s_{ue}. Convexity of the yield surface is also the reason for the limitation in Equation 5.2 (K Krabbenhoft, 2016).

5.2.2 Parameters

The s_{ue}/s_{uc} ratio that is used for this model is set to 0.71, which is the mean of anisotropic strength ratios for Swedish clays (Won, 2013). To obtain a s_{us}/s_{uc} ratio the mean of the upper and lower limit when the s_{ue}/s_{uc} is set to 0.71 is taken, which gives a ratio of 0.8075. Since Optum has s_{uc} (strength in compression) as the main strength parameter instead of s_u when modelling with anisotropy it is necessary to take that into consideration. The reason for this is that the s_{uc} often is larger than s_u. For example if the s_u that is used for the reference slope would be used as s_{uc} the FOS for the slope would decrease since that gives a lower s_{us} and s_{uc}. This has been taken into account by assuming that s_{uc} is 1.75 times bigger on average than s_u the soils specific strength parameters can be seen in Table 5.2. Also in this case are 5000 elements and five adaptive iterations are used for the calculations. No water is included in this case.

Table 5.2: The undrained strength of the soil's in the reference slopes and the soil's undrained shear in compression for the anisotropic slopes.

	Soil A(kPa)	Soil B(kPa)
Reference	16.8 +1.3/m	10kPa
Anisotropic	30+ 2.25/m	17.5KPa

5.2.3 Results anisotropy

With the introduction of anisotropy the FOS increase did for all three slope compared to the three reference slopes as is shown in Table 5.3. This was to be expected since the peak shear strength was increased to simulate the s_{uc}. The FOS increased most for the slope (1:3) with lowest inclination which is in line of what to be expected. The Time scope did not have any real influence on the slopes stability in this case since the FOS basically remained the same regardless of inclination for both the long and short term time scope.

Table 5.3: The FOS for the slopes with the soil's anisotropy and the references slopes without the soil's anisotropy.

	1:1		1:2		1:3	
Short term	Ref	Aniso	Ref	Aniso	Ref	Aniso
Lower	0.884	1.206	1.142	1.588	1.325	1.859
Upper	0.8912	1.218	1.148	1.598	1.329	1.865
Mean	0.8876	1.212	1.145	1.593	1.327	1.862
% Increase	36.5		39.1		40.3	
Long term						
Lower	0.8839	1.208	1.142	1.589	1.325	1.858
Upper	0.8912	1.218	1.148	1.599	1.329	1.865
Mean	0.88755	1.213	1.145	1.594	1.327	1.8615
% Increase	36.7		39.2		40.3	

5.3 Discussion Suction and Anisotropy

The difference in the FOS between the long term and the short term cases are significant and shows how big influence that the suction of the soil can have on a slopes stability. However, since this is a model based on a limited amount of data should further studies be conducted to validate these results and analyse this in more detail.

For this case it was assumed that the s_{uc} is 1.75 times higher than s_u, in other and real cases this should be determined more thoroughly from several triaxial tests. If the ratio varies for different depths this should be taken into account by including several layers with appropriate values for each depths for the model. From the triaxial tests it would also be possible to get more site specific s_{ue}/s_{uc} and s_{us}/s_{uc} ratios and improve the accuracy of the model, these ratios could also be specified for different depths if needed.

At the moment it is not possible to use suction and anisotropy at the same time in Optum since they require different soil models. This is a bit sad since that might mean that the FOS is lower than it should be for certain cases depending on the slope and the soils properties, which means that the FOS in those cases are on a slightly conservative side. Hopefully will this be fixed in the future so it is possible to have models with both anisotropy and suction integrated that gives more accurate results.

6

Real case: E45 and Railway Expansion

6.1 Background

After the evaluation of several different slope models with finite element limit analysis it has become apparent that it should be used on a real case. Between 2007 and 2012 north of Gothenburg the E45 highway was expanded to motorway and the railway to dual tracks on the east bank of Göta river seen in Figure 6.1. Before and during the construction several site investigations and evaluations of the stability were conducted with a standard array of geotechnical tests. Overall a 650 meter long section of the shore was surveyed in close detail. The soil in the area mainly consists of relative soft clay, there are also pockets of quick clay with high sensitivity. Because of this there is also a risk for secondary landslides in certain areas. Secondary landslides are in this case a landslide that is triggered by a smaller landslide that occur in the underwater part of the slope.

Figure 6.1: Map of the section of E45 motorway and the railway, map taken from maps.google.com.

6.2 Soil parameters and lab results

There are several layers of clay both below the river and on shore, the properties of these differ until 15 to 18 meters below Göta rivers surface where they begin to equalize. The properties are specified in Table 6.3. The unit weight in the area is around 15 to 16 kN/m^3 and the undrained strength (s_u) is increasing between 1 to 2 kPa per meter with the depth. Seven different clay layers are overall evaluated and used for the model. A total of 32 soil samples from three different location were evaluated with lab tests, of these were 12 triaxial compression test, 12 triaxial extension test and eight Direct Simple Shear test (DSS). The results for these can be seen in Table 6.1 and are also plotted against the depth in Figure 6.2. All diagrams from which the data are taken can be found in Appendix A.

Table 6.1: Results from triaxial and DSS tests from three borehole in the surveyed area.

FB2201 Depth (m)	Compression s_u(kPa)	Extension s_u(kPa)	DSS s_u(kPa)
12	40	22	-
18	56	33	-
24	58	33	37.8
30	69	38	47.5
FB2202 Depth (m)	Compression s_u(kPa)	Extension s_u(kPa)	DSS s_u(kPa)
12	32	19	-
18	55	32	-
24	62	37	44
30	74	36	47
FB2203 Depth (m)	Compression s_u(kPa)	Extension s_u(kPa)	DSS s_u(kPa)
12	37	19	23
18	51	28	33.9
24	55	28	34
30	77	42	46.1

For some reason does the increase in the soils undrained shear strength decline between 17 and 25 meter before it resumes again so there is a zone where the undrained shear strength is lower than expected. The reason for this is not clear but might has something to do with how the soil was deposited.

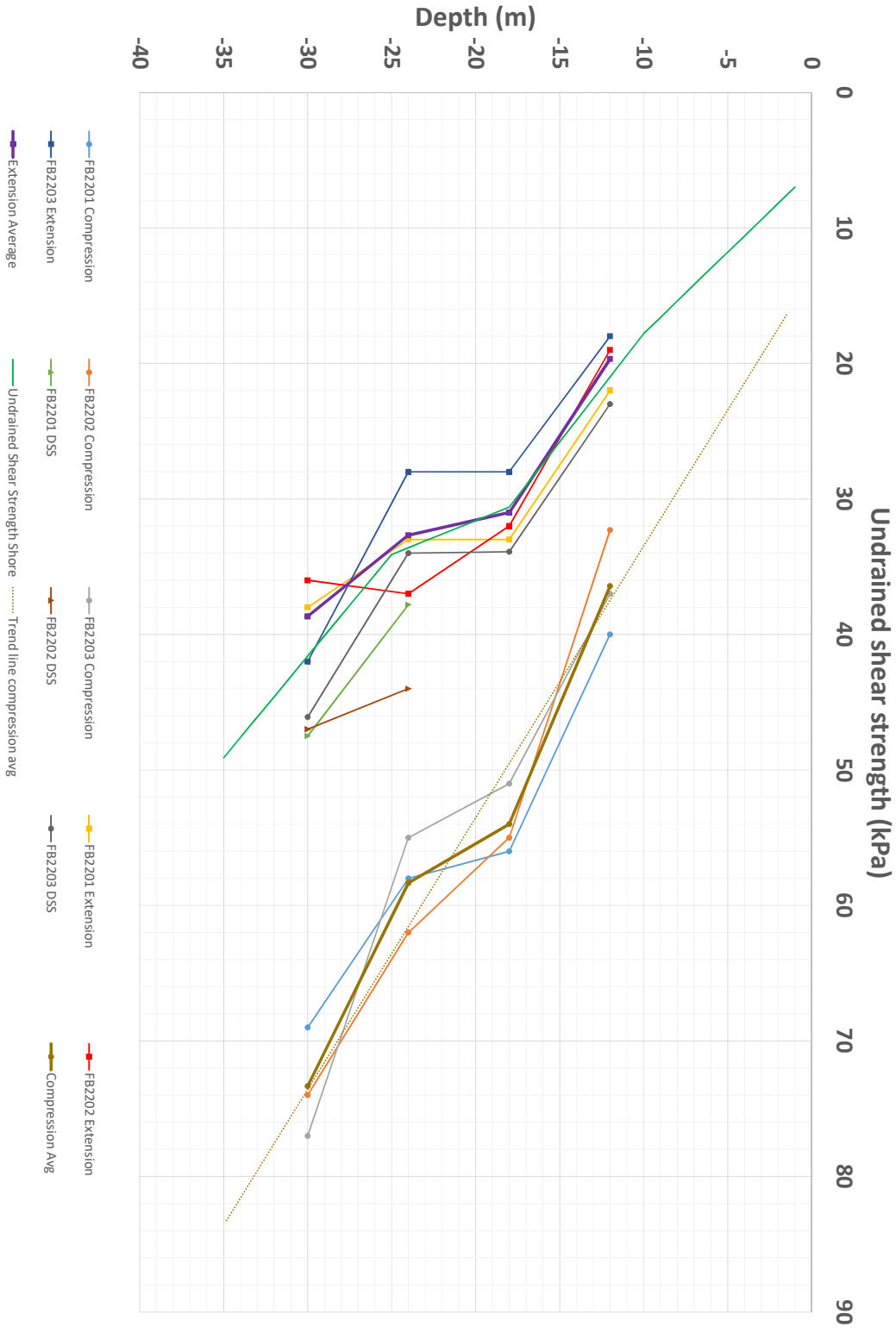

Figure 6.2: Diagram of the clays overall undrained shear strength together with the results from triaxial and DSS tests.

6.3 Optum model

In Optum is s_{uc} the governing parameter for the anisotropic soil model, because of this is the anisotropy value for compression in Table 6.2 that are of greatest interest for this case. Since there are not any common trends for the anisotropy it is hard to draw any real conclusion from these values. But to get an anisotropy factor that can be used to determine s_{uc} for the model in Optum the mean of all anisotropy values for compression in Table 6.2 will be used. This gives an anisotropy factor with a value of 1.75. This is a somewhat arbitrary method to determine s_{uc} and should be handle with very great care. The values that will be used for s_{uc} in the model are displayed in Table 6.3.

Table 6.2: Anisotropy of the clays, calculated by dividing s_{uc} from a triaxial test with s_u from the same depth as the sample for the triaxial test.

	Compression			Extension		
Depth	FB2201	FB2202	FB2203	FB2201	FB2202	FB2203
12	1.90	1.54	1.76	1.07	0.91	0.92
18	1.83	1.79	1.67	1.09	1.04	0.92
24	1.73	1.85	1.66	0.98	1.12	0.85
30	1.66	1.77	1.87	0.91	0.87	1.01
Mean		1.75				

Table 6.3: The clays parameters and their start level beneath the surface.

Clay	γ (kN/m^3)	s_u (kPa)	s_{uc} (kPa)	Datum(m)
River Clay 1	15.2	3	5.25	-
River Clay 2	15.7	3 +1.73/m	5.25 + 3.03/m	-5
Shore Clay 1	15.2	7	12.25	-
Shore Clay 2	15.2	7+ 1/m	12.25 +1.75/m	-1
Shore Clay 3	15.7	16+2/m	28 +3.25/m	-10
Shore Clay 4	16	32 +0.4/m	56 +0.7/m	-18
Organic clay	15	10	17.5	-

The ratios between s_{ue}/s_{uc} and s_{us}/s_{uc} from the triaxial and DSS tests are displayed in Table 6.4. s_{us}/s_{uc} mean from the lab results are 0.656, which unfortunately are below the lower limit of what is allowed in Optum due to the demand of convexity for the yield surface. With s_{ue}/s_{uc}=0.548 is the lower limit set to 0.688. So to be able to model this case Optums lower limit for s_{us}/s_{uc} will be used. The discrepancy between the actual s_{us}/s_{uc} and the lower limit is 4.87%, this would probably be a to big discrepancy to be acceptable under normal circumstances but it will be used for this model.

Table 6.4: The ratios between s_{ue}/s_{uc} and s_{us}/s_{uc}.

	s_{ue}/s_{uc}			s_{us}/s_{uc}		
Depth	FB2201	FB2202	FB2203	FB2201	FB2202	FB2203
12	0.550	0.588	0.522	-	-	0.622
18	0.589	0.516	0.549	-	-	0.665
24	0.569	0.597	0.509	0.652	0.710	0.618
30	0.551	0.488	0.545	0.688	0.637	0.599
Mean		0.548			0.656	

Two normal cases will be evaluated, the first case is a normal analysis were anisotropy not is included while anisotropy is included for the second case. Two special cases will also be done where the soils undrained strength varies stochastic. The parameters used in the model to get a stochastic distribution for the undrained shear strength are shown below.

- Mean of undrained shear strength in compression for each layer.
- Coefficient of variation = 15% (Nadim, 2007)
- Horizontal correlation length = 20m
- Vertical correlation length = 2m

The special cases will be evaluated with 500 upper and 500 lower bound Monte Carlo simulations to see how the FOS varies due to the stochastic distribution of undrained shear strength. The cross section used for all for both the normal and stochastic cases can be seen in Figure 6.3. Since it is Optum's Tresca and Anisotropic Undrained Shear (AUS) soil models that will be used for the cases are only a water table used to include water into the models. 10000 elements are used for all three cases while, the two normal cases will use 5 adaptive iterations for the mesh while the special stochastic case only uses 3 due to the long computational time it requires. The four cases are only evaluated with finite element limit analysis for upper and lower bounds.

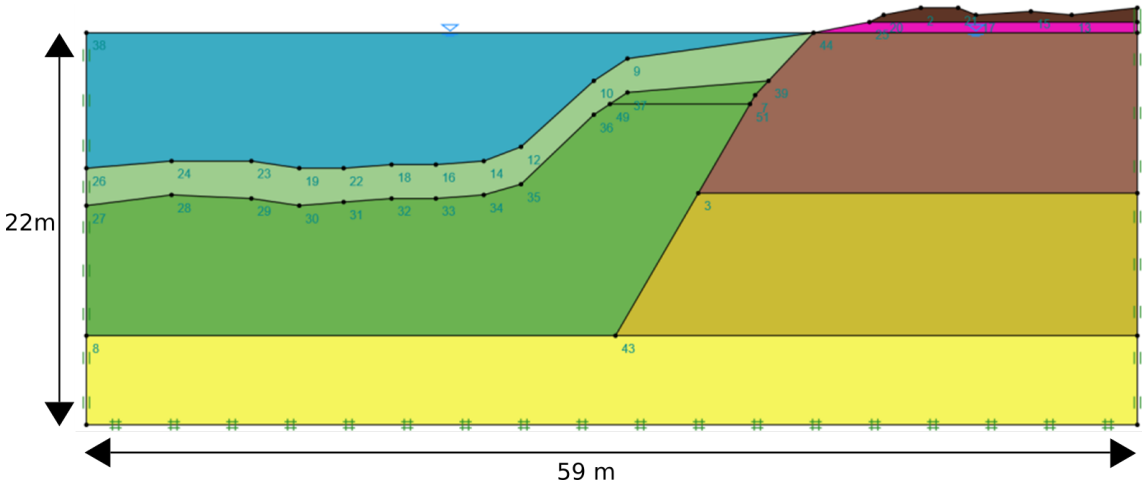

Figure 6.3: The cross section of the east bank of Göta river that is to be evaluated.

6.4 Results

6.4.1 Ordinary results

The results for the two ordinary cases can be seen in Table 6.5. Optum was unable to form a stable mesh for the lower bound short term case without anisotropy so no analysis was done for that case, but this probably does not really matter since all other results for between the long and short terms cases are equal to each other. So any results for the short term lower bound cases would probably either be equal or very close to the result in the long term case.

The failure surface can be seen in Figure 6.4 and is located in the weak layer of clay at bottom of Göta river. Without the inclusion of anisotropy into the model is the slope deemed to be unstable. However, with the soils strength anisotropy taken into account the FOS is above one. It is not the biggest safety margin but it shows why anisotropy should be included slopes with low FOW and shows why the slope not has collapsed despite the initial calculations.

Table 6.5: The results for cross section model for Göta river both with and without the anisotropy of the soil taken into account.

		Without anisotropy	With anisotropy
Short term			
	Lower	-	1.134
	Upper	0.9824	1.143
	Mean	-	1.1385
Long term			
	Lower	0.9741	1.134
	Upper	0.9824	1.143
	Mean	0.97825	1.1385

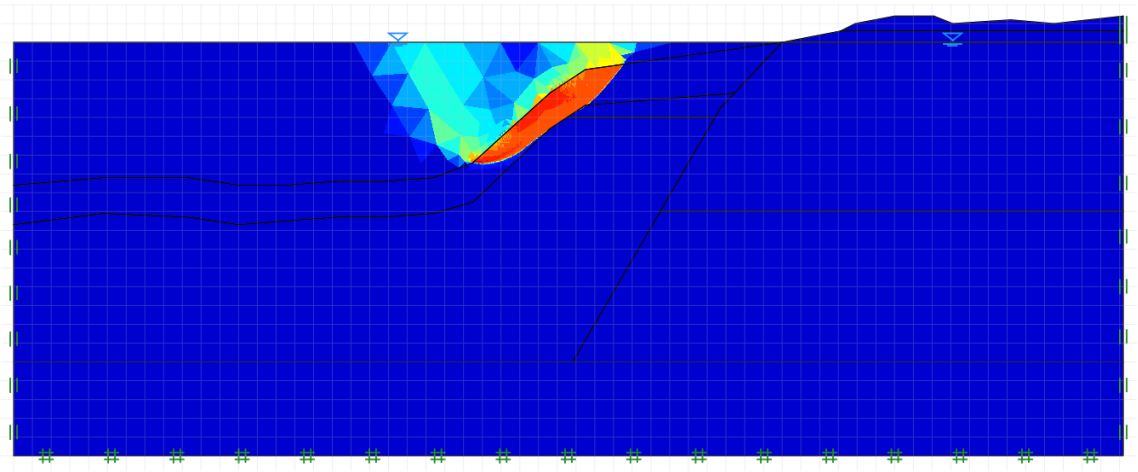

Figure 6.4: Failure surface for slope in the real case.

6.4.2 Results Monte Carlo simulations

When the undrained shear strength varies stochastically in the soils have a clear majority, around 80%, of both the lower and upper bound Monte Carlo simulations a FOS that are above one when the soils anisotropy is included in the model. Without the inclusion of the soils anisotropy are the FOS above one for a minority of the simulations. Only in around 30 to 35% of the simulations are the FOS above one, but it shows that the slope could be seen stable even without the inclusion of the soils strength anisotropy depending on the circumstances. How the FOS varies can be seen in Figure 6.6 and Table 6.6. The difference in how the undrained shear strength is distributed in the soil due to the stochastic parameters compared to the normal distribution can been seen in Figure 6.5.

Table 6.6: The minimum, mean, maximum and the standard deviation for the load multipliers/factor of safety from the Monte Carlo simulations.

	Minimum Load Multiplier	Mean load Multiplier	Maximum Load Multiplier	Standard Deviation
Without Anisotropy				
Lower bound	0.6837	0.9499	1.282	0.0947
Upper bound	0.6949	0.9655	1.302	0.09615
With anisotropy				
Lower bound	0.7956	1.107	1.496	0.1108
Upper bound	0.8087	1.125	1.518	0.1124

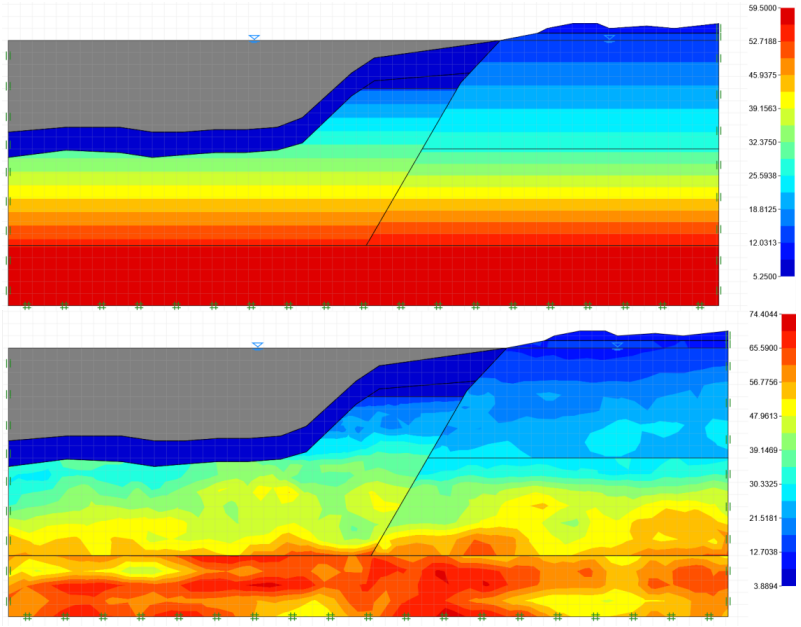

Figure 6.5: Top picture shows how the undrained strength normally is distributed while the bottom pictures shows the distribution of the undrained strength in one of the stochastic simulations.

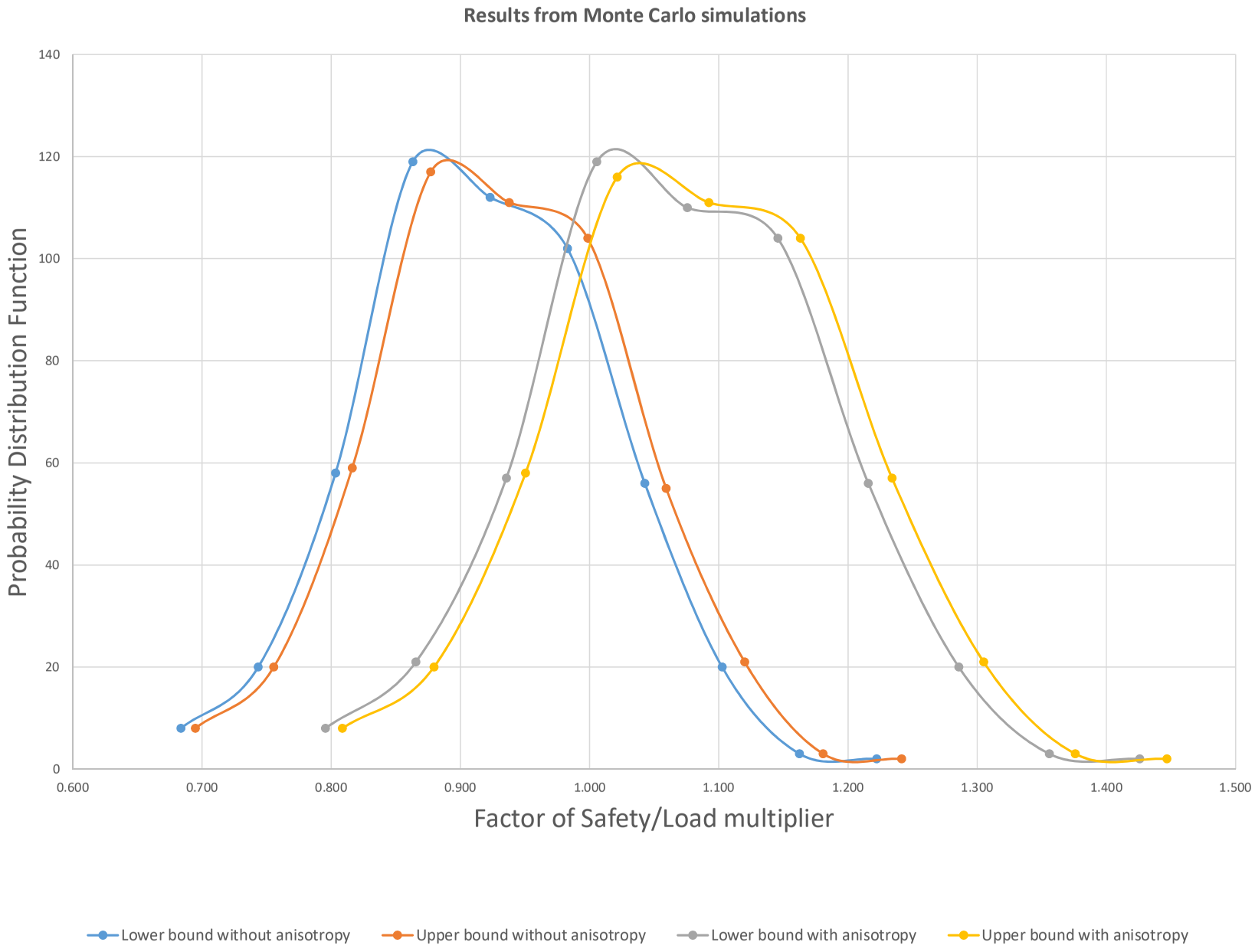

Figure 6.6: Results from the Monte Carlo simulations displaying the Load multiplier/ FOS and the Probability Distribution Function for the real case both with and without anisotropy included.

6.5 Discussion

It is concerning that the FOS is below one for the version of the model that does not included anisotropy. However, since the FOS is so close to one is it possible that it is inside the margin of error and actually could be stable. This is at least partly supported in the stochastic case without the inclusion of anisotropy were around 30 to 35 % of the simulations have FOS above one. So how the undrained shear strength varies spatially in the soil can have a big influence.

The introduction of the soils strength anisotropy into the model lead to an increase in FOS which indicate that the slope is stable. This is also shown in the Monte Carlo simulations were the FOS are above one in 80% of the cases. So overall can the slope probably be classified as stable.

The analyses done with Monte Carlo simulations gives very interesting and useful results, but the computational cost are unfortunately very high at the moment. On a laptop with quad core processor of 2.4 Hz and 12 GB RAM did 500 lower and 500 upper bound simulations take 36 hours. Even on a server with 16 virtual processors (vCPU) and 30 GB RAM did 500 lower and 500 upper bound Monte Carlo simulations take 15 hours.

Another aspect that needs to be taken into consideration are the statistical parameters used for the simulations. In these cases have values from literature been used to see how they can influence the case but they are not necessarily correct for this location. In an analysis for a real case they should be determined from a geotechnical survey of the location.

7
Conclusions & Recommendations

7.1 Conclusion

All three of the methods evaluated in this thesis was able to evaluated that stability of slopes in an adequately and satisfying way. The Limit Equilibrium Method showed why it has been used for stability analysis in close to 100 years. Finite element limit analysis and discontinuity layout optimisation are both interesting method with potential to develop further.

During the initial evaluation did the finite elements limit analysis perform very good and showed great accuracy in its results when compared to closed-form solutions. The excellent performance continued in the complex and real cases and it showed that it both has great potential and already can deliver good analyses.

The analysis done for the real case showed how useful it can be to integration anisotropy of the soils undrained strength into the models for slopes were an ordinary analysis shows a very low FOS, since it can give a FOS that is higher than the first calculated. A disadvantage with inclusion of anisotropy is that the method require data from triaxial tests which can lead to an increase in cost since this required test samples from in situ. However, since including anisotropy can show a FOS higher it can that less reinforcing measures are need for the slope and that way lead to a lower overall cost. So it is an cost benefit analysis that should be decided on a case to case basis.

Monte Carlo simulations for geotechnical problems can be a useful tool to determine a bigger margin of error. These simulations will probably also require a thoroughly conducted in situ geotechnical surveys to get accurate statistical useful parameters.

7.2 Recommendations

Areas from this thesis that can be researched in future are the following:

- Finite Element Limit Analysis should be studied and evaluated further with a wider range of problems and on frictions soils as well.
- Look more in more detail how the Finite Element Limit Analysis models suction and see if that can be integrated for models with soil strength anisotropy.
- Examine when it can be useful to use Monte Carlo Simulations for real cases.

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A

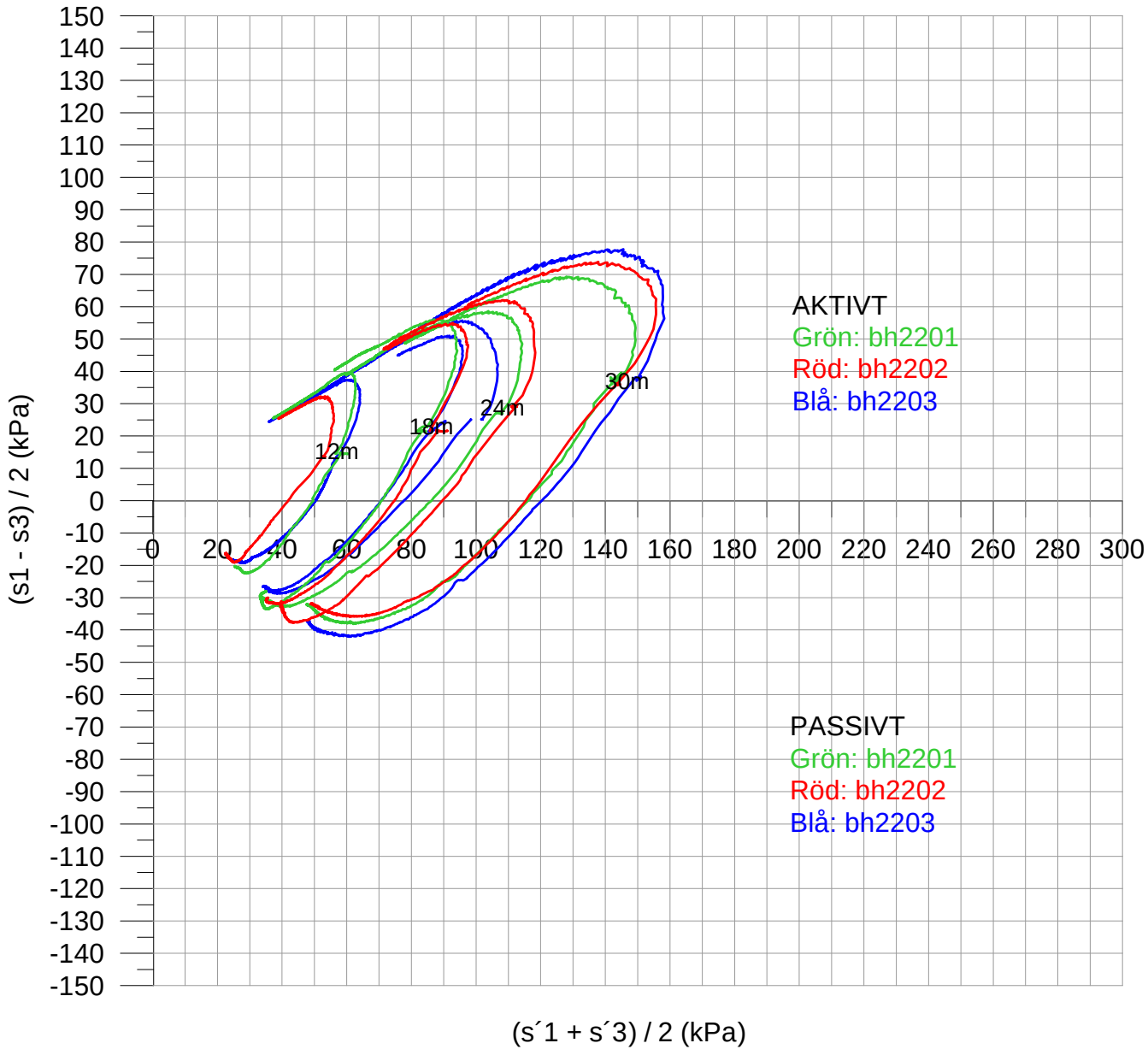
Appendix

Chalmers tekniska högskola
GEO

Triax försök, aktivt & passivt odränerat

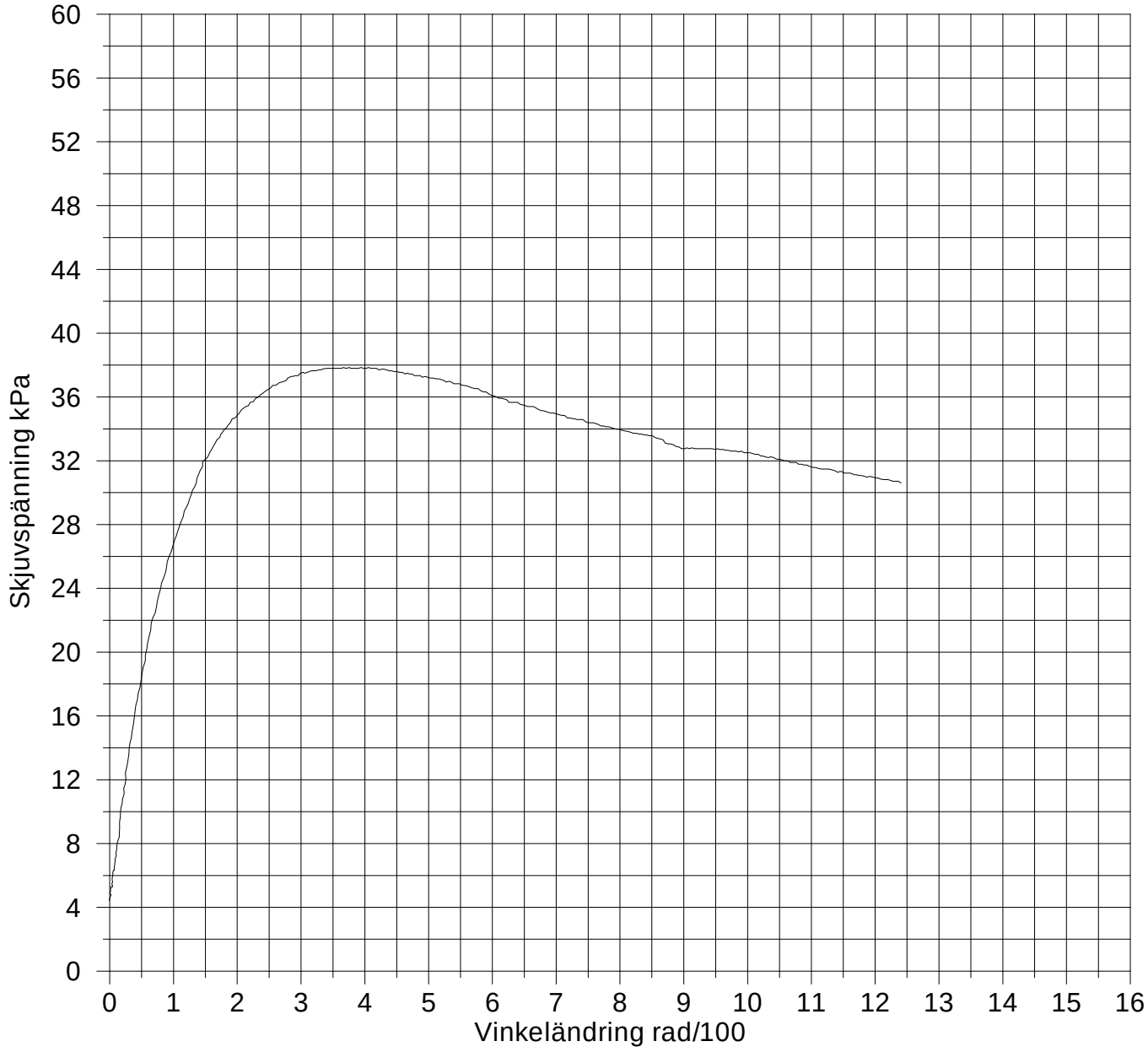
Provet har belastat upp till angivna konsolideringsspänningar och har sedan fått konsolidera i ca ett dygn.

Konsolidering:
Datum: 2009-10-28
Projekt: E45
Punktnummer: FB2201,FB2202, FB2203
Djup: 12m, 18m , 24m ,30m
Tub:
Utförare: Peter Hedborg



Chalmers tekniska högskola - Direkt skjuvförsök

Projekt	E45	**Belastningshastighet**	0,10mm/h
Datum	2009-10-21	**Försöksmetod**	odränerat
Borrhål	FB2201	**Konsolideringsspänning**	126(126)kPa
Beställare	FB	**Vattenkvot före**	
Uppdragsnummer		**Vattenkvot efter**	
Provtub	410	**Skrymdensitet**	
Djup:	24m	**Höjd**	15,4mm
Utfört av	Peter Hedborg	**Diameter**	50,0 mm



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Beställare	FB	**Vattenkvot före**	
Uppdragsnummer		**Vattenkvot efter**	
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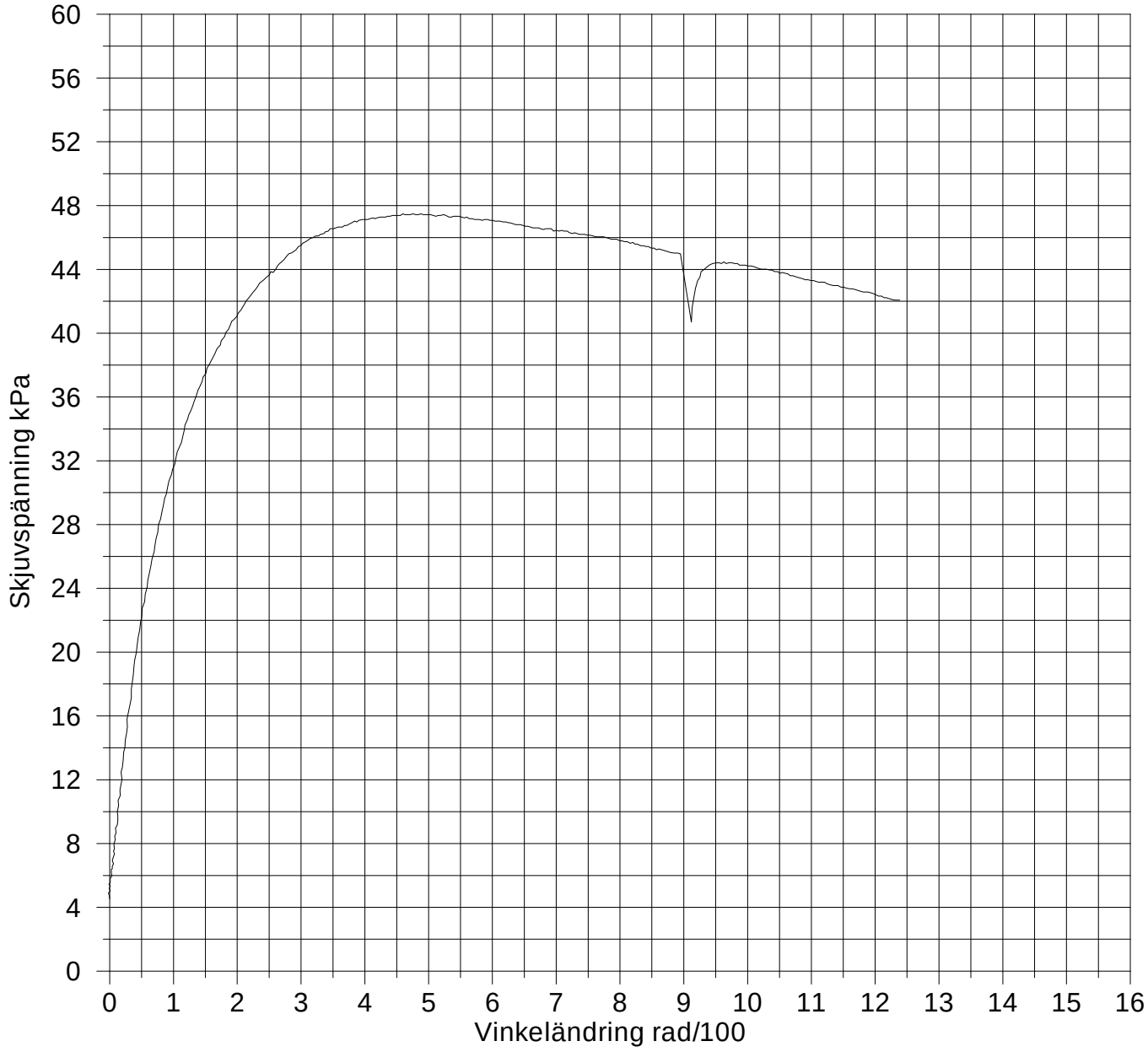
Djup:	30m	**Höjd**	15,1mm
Utfört av	Peter Hedborg	**Diameter**	50,0 mm



Chalmers tekniska högskola - Direkt skjuvförsök

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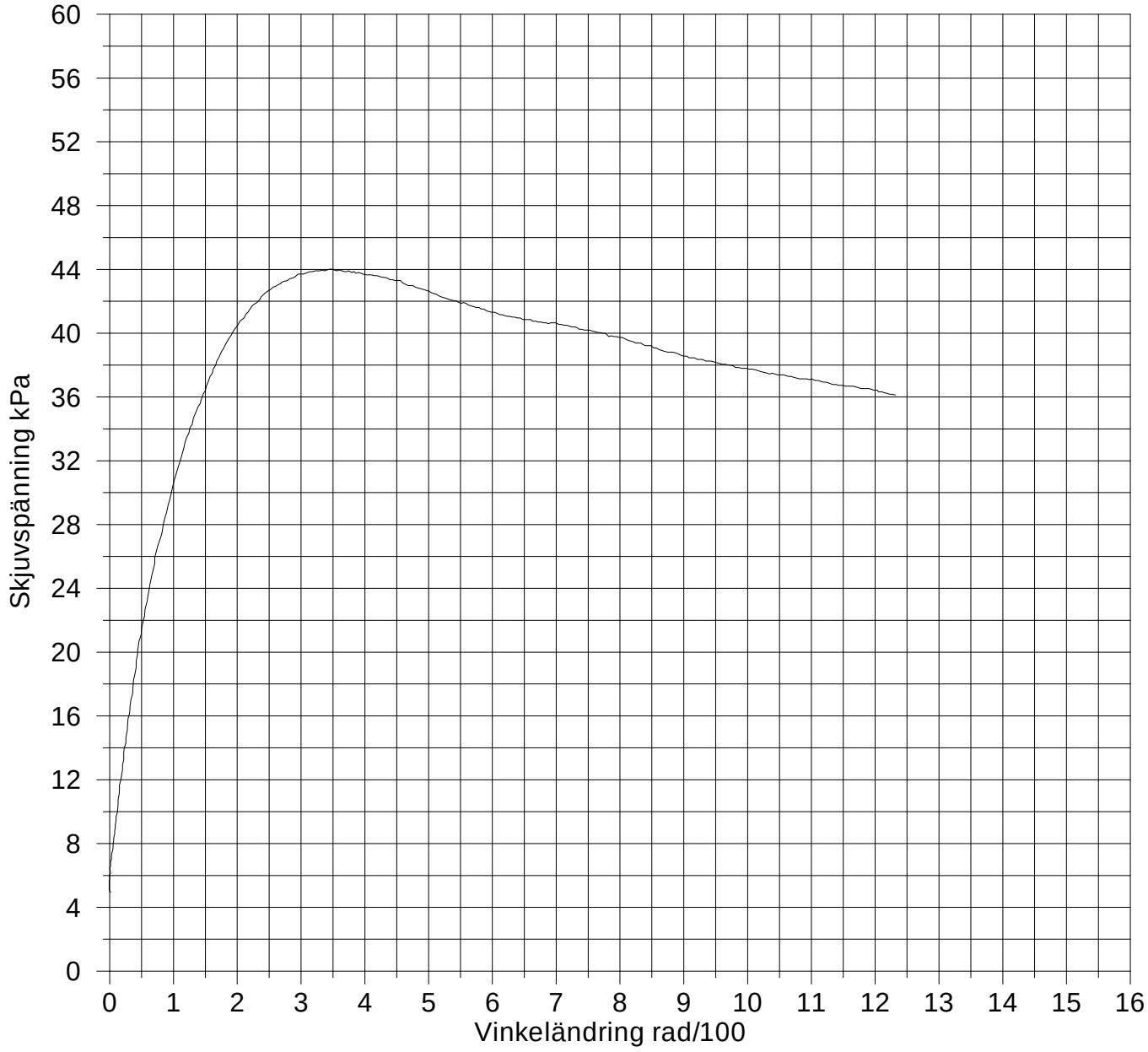
Datum	2009-10-23	**Försöksmetod**	odränerat
Borrhål	FB2202	**Konsolideringsspänning**	144(144)kPa
Beställare	FB	**Vattenkvot före**	
Uppdragsnummer		**Vattenkvot efter**	
Provtub	17	**Skrymdensitet**	
Djup:	24m	**Höjd**	15,0mm
Utfört av	Peter Hedborg	**Diameter**	50,0 mm



Chalmers tekniska högskola - Direkt skjuvförsök

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Beställare	FB	**Vattenkvot före**	
Uppdragsnummer		**Vattenkvot efter**	
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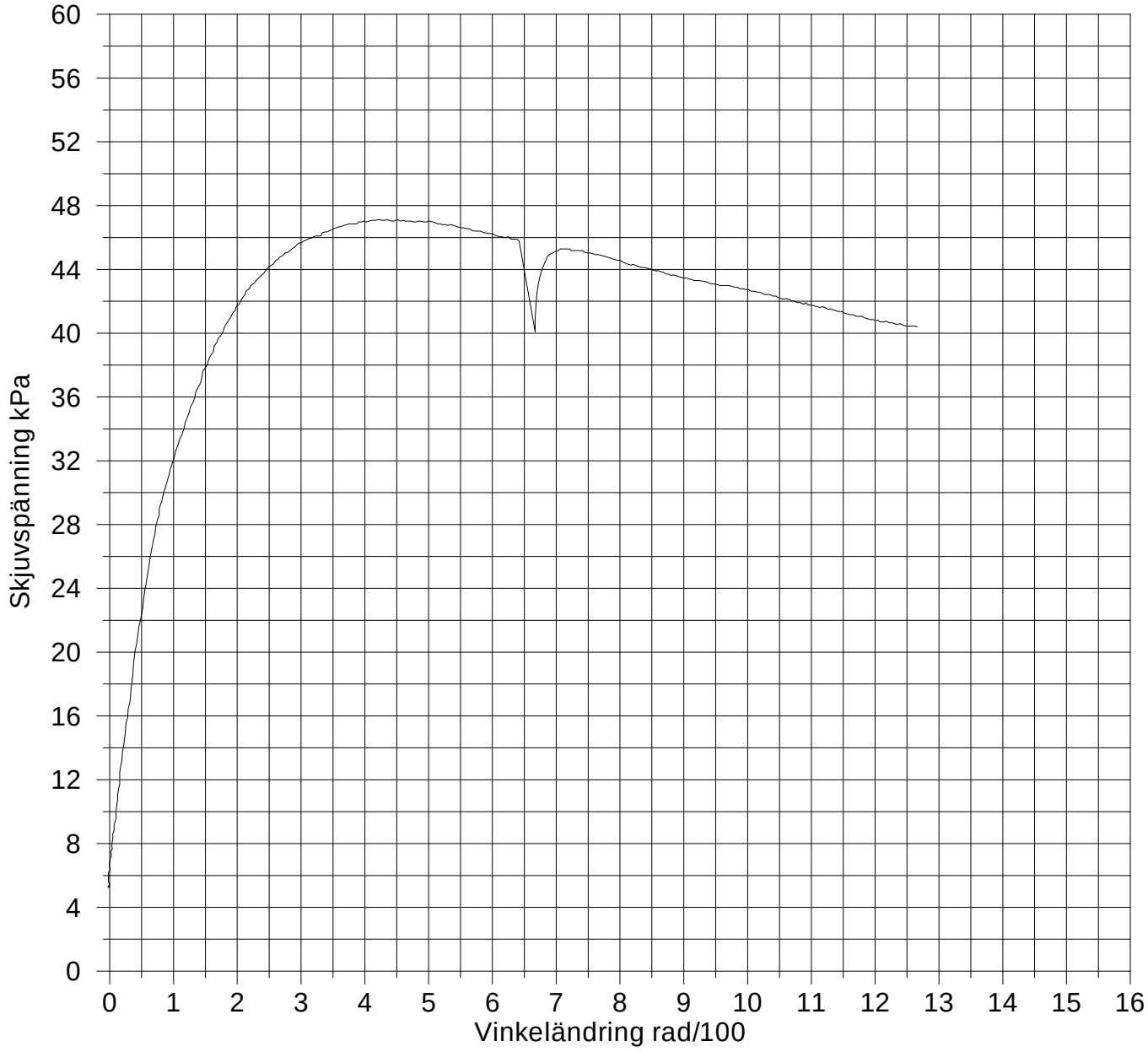
Utfört av	Peter Hedborg	**Diameter**	50,0 mm



Chalmers tekniska högskola - Direkt skjuvförsök

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Borrhål	FB2203	**Konsolideringsspänning**	82(72)kPa
Beställare	FB	**Vattenkvot före**	
Uppdragsnummer		**Vattenkvot efter**	
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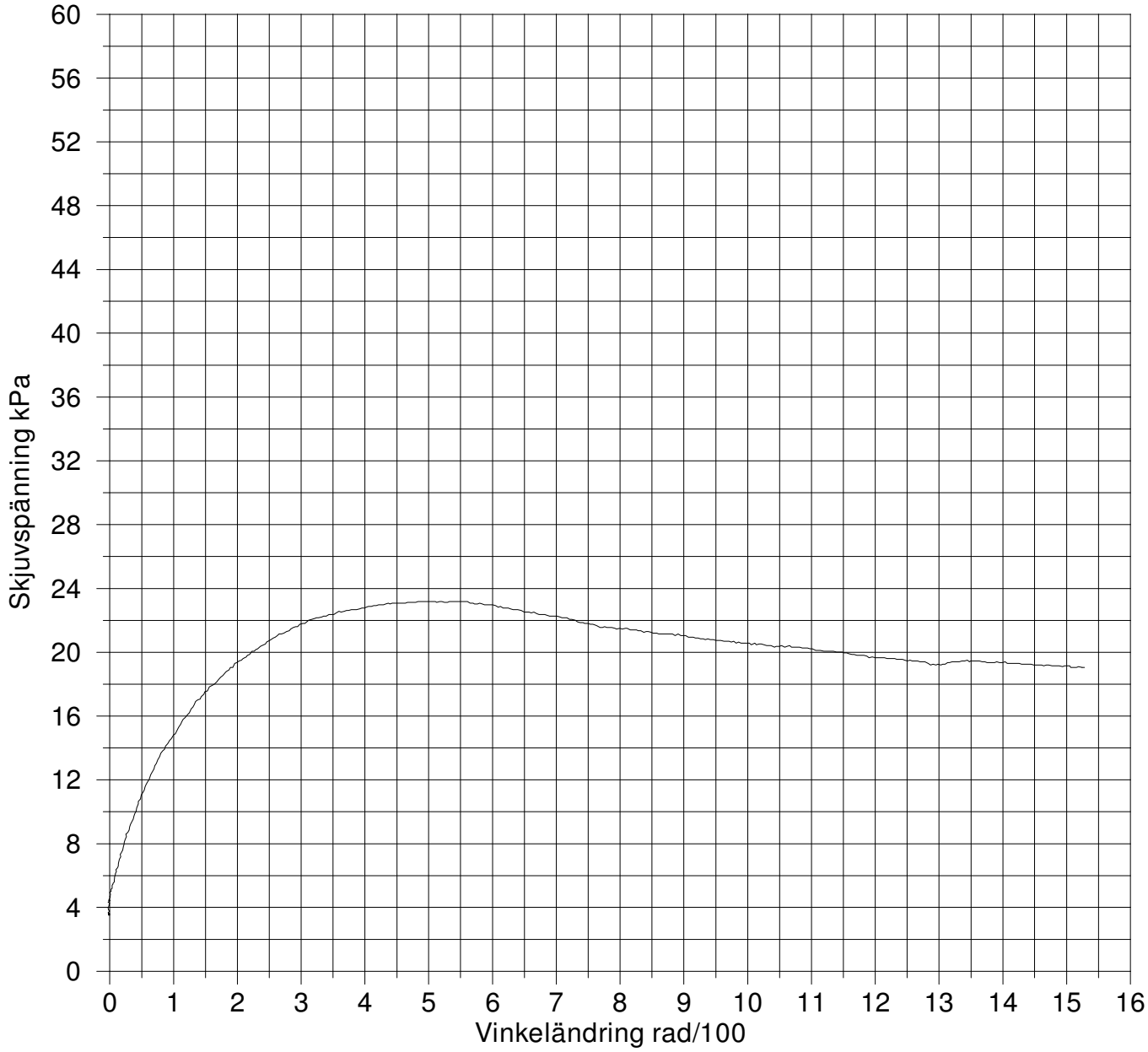
Utfört av	Peter Hedborg	**Diameter**	50,0 mm



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Beställare	FB	**Vattenkvot före**	
Uppdragsnummer		**Vattenkvot efter**	
Provtub	37	**Skrymdensitet**	
Djup:	18m	**Höjd**	14,5mm
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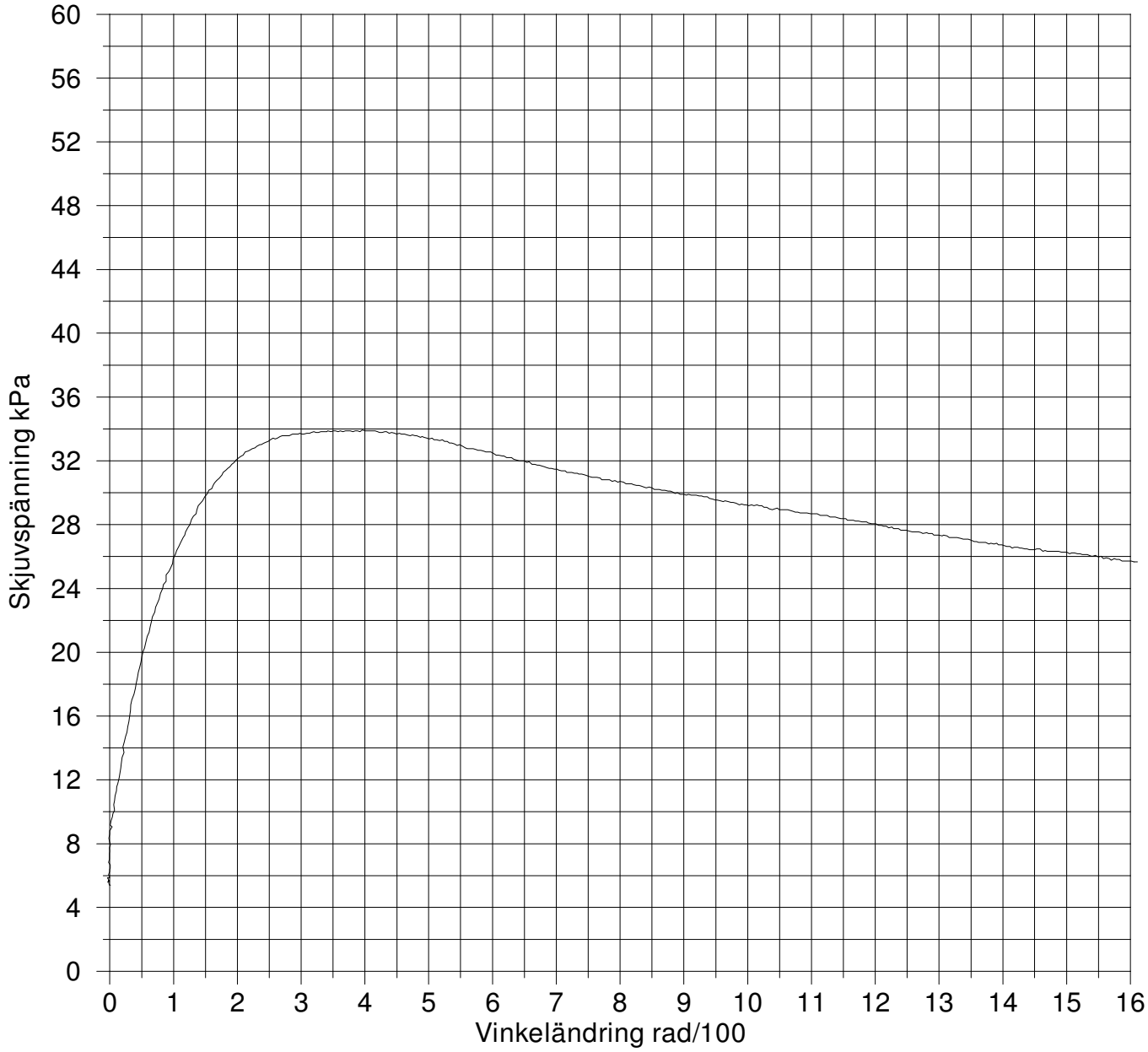

Chalmers tekniska högskola - Direkt skjuvförsök

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Borrhål	FB2203	**Konsolideringsspänning**	126(126)kPa
Beställare	FB	**Vattenkvot före**	
Uppdragsnummer		**Vattenkvot efter**	
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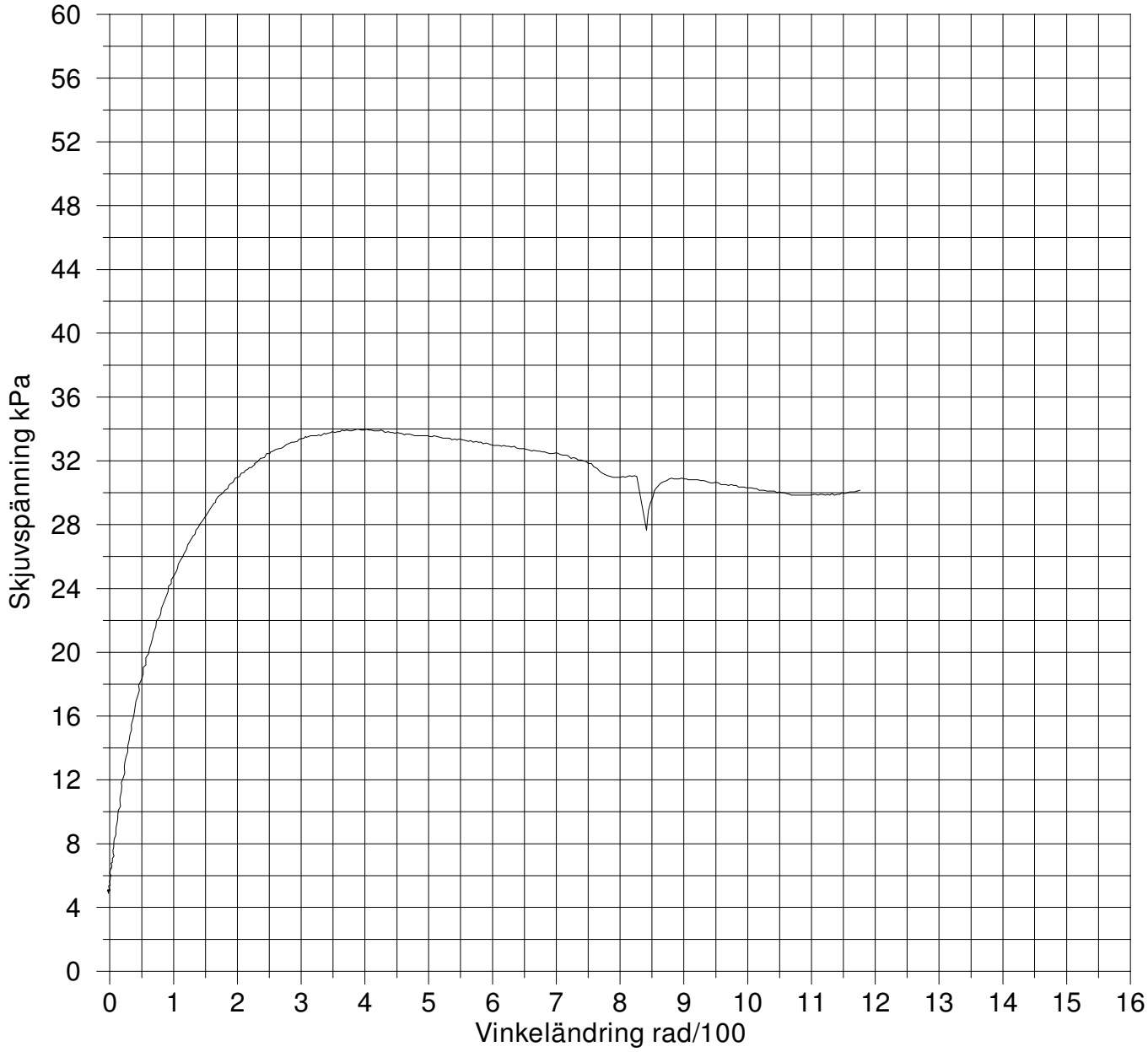
Utfört av	Peter Hedborg	**Diameter**	50,0 mm



Chalmers tekniska högskola - Direkt skjuvförsök

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Beställare	FB	**Vattenkvot före**	
Uppdragsnummer		**Vattenkvot efter**	
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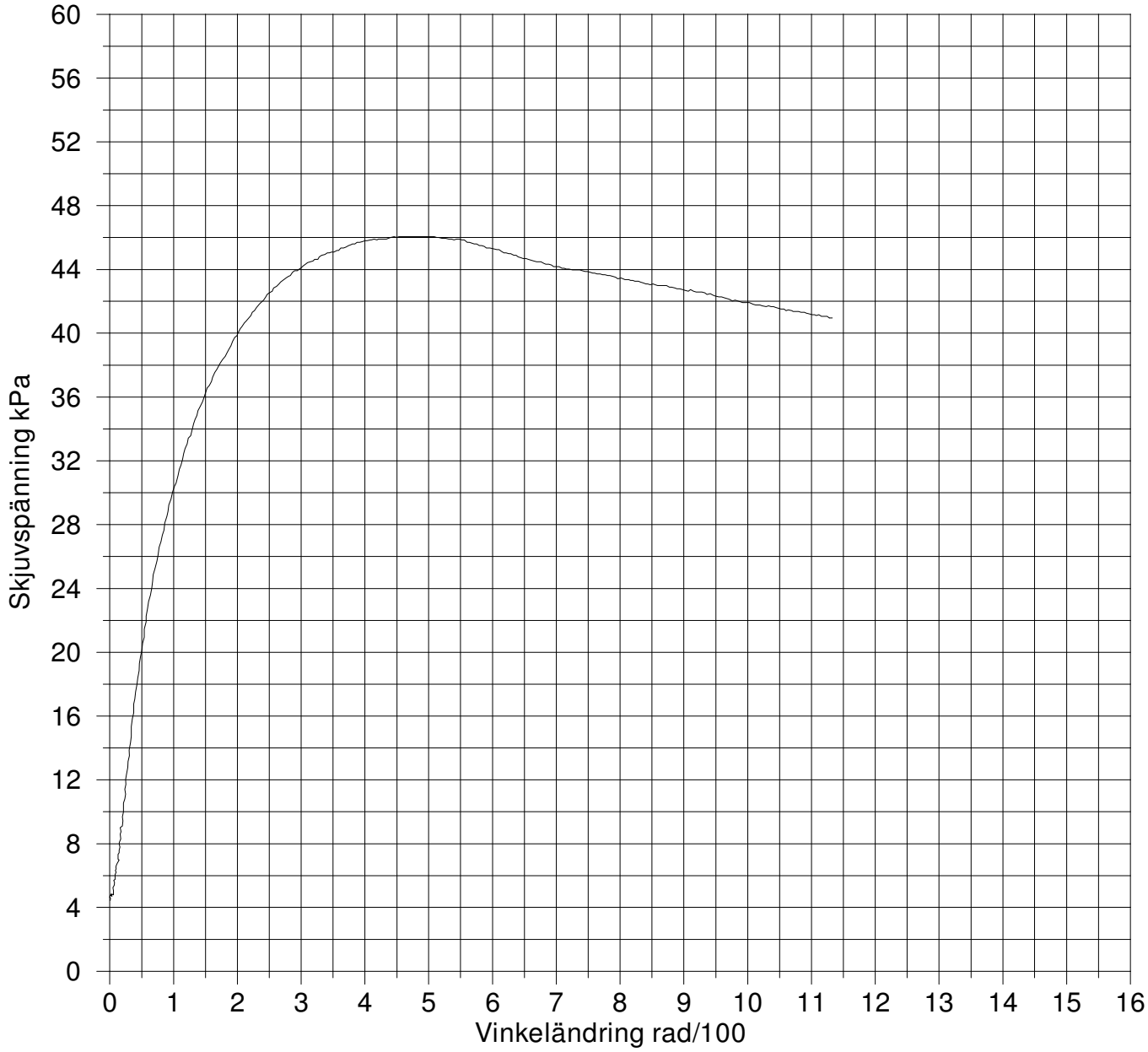

X