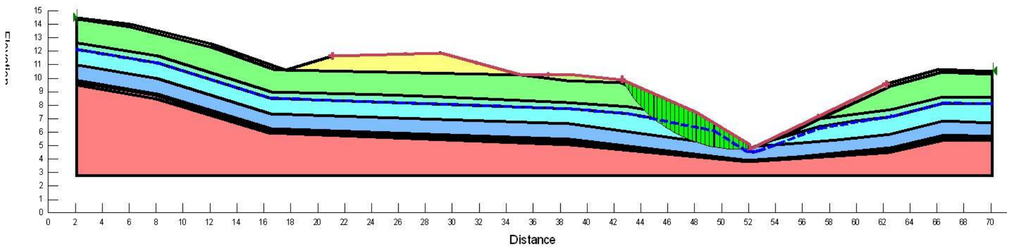
MASTER'S THESIS

Slope stability analysis and road safety evaluation

A case study on two roads located close to the Piteå river - in Sikfors and Nystrand



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SLOPE STABILITY ANALYSIS AND ROAD SAFETY EVALUATION

- A CASE STUDY ON TWO ROADS LOCATED CLOSE TO THE PITEÅ RIVER-IN SIKFORS AND NYSTRAND

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Preface

At first I express my praise to almighty that the project was completed in proper time. From my undergraduate studies at the Khulna University of Engineering & Technology (KUET), Bangladesh, I like Geotechnical Engineering. I have always loved latest computer software to solving civil engineering problem.

In November 2011, I met with Nicklas Thun and Göran Pyyny, Geotechnical specialist, Trafikverket, Luleå, Sweden for the first time and discussed about master thesis project. In that regard, I would like to thank Nicklas Thun and Göran Pyyny that gave me the opportunity to work with slope stability analysis with modern geotechnical software. I'm grateful to them for enduring advice, kin interest, directing me towards such an interesting problem.

I also, express my profound gratitude, indebtedness and heartiest thanks to the honorable teacher Dr. Hans Mattsson, Division of Mining and Geotechnical Engineering, Luleå University of Technology (LTU), Luleå, Sweden, for his valuable patient advice, sympathetic assistance, constant encouragements, guidance, co-operation, relatives interest, contribution to new ideas and supervision of all stages of this project work. His guidance has benefited me greatly.

This has been a wonderful year for me to have experienced studying in Sweden. This experience has been more colorful with many friends that support me during my study. Living in Luleå would never have been as wonderful without all of you.

Last but not least, I would like to express deep felt gratitude to my family members and one of my best friends for their continuous support.

Luleå, June 2012
MD. ZILLUR RAHMAN

Abstract

This master thesis focuse on natural slope and embankment slope stability analysis with the Limit Equilibrium Method computer program Slope/W and the Finite Element Method computer program Plaxis 2D and a comparison of the analyzed result. The finite element method needs additional information regarding the potential performance of a slope; just basic parameter information is needed when using limit equilibrium methods. The results indicate that it is important to use the effective shear strength characterization of the soil when performing the slope stability analysis. A distinction should be made between drained and undrained strength of cohesive materials. Shortly, drained condition refers to the condition where drainage is allowed, while undrained condition refers to the condition where drainage is restricted. Most likely the worst case scenario occurs when the river water level is increased rapidly, and then quickly drops while the water table in the embankment is retained on an extremely high level so that the low effective stresses might lead to failure. The existence of trivial failure surfaces is a large problem in stability analysis of natural slopes, especially in the Plaxis 2D program. We tried to avoid these types of failure. In Slope/W analysis we consider critical slip surface failure. If the critical failure is trivial, then we consider the secondary failure. In Plaxis 2D analysis, additional displacements are generated during a Safety calculation. The total incremental displacements in the final step (at failure) give an indication of the likely failure mechanism.

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1 INTRODUCTION

Evaluating the stability of slopes in soil is an important, interesting, and challenging aspect of civil engineering. Slope instability is a geo-dynamic process that naturally shapes up the geo-morphology of the earth. However, they are a major concern when those unstable slopes would have an effect on the safety of people and property. Concerns with slope stability have driven some of the most important advances in our understanding of the complex behavior of soils. Extensive engineering and research studies performed over the past 70 years provide a sound set of soil mechanical principles with which to attack practical problems of slope stability.

Over the past decades, experience with the behavior of slopes, and often with their failure, has led to development of improved understanding of the changes in soil properties that can occur over time, recognition of the requirements and the limitations of laboratory and in situ testing for evaluating soil strengths, development of new and more effective types of instrumentation to observe the behavior of slopes, improved understanding of the principles of soil mechanics that connect soil behavior to slope stability, and improved analytical procedures augmented by extensive examination of the mechanics of slope stability analyses, detailed comparisons with field behavior, and use of computers to perform thorough analyses. Through these advances, the art of slope stability evaluation has entered a more mature phase, where experience and judgment, which continue to be of prime importance, have been combined with improved understanding and rational methods to improve the level of confidence that is achievable through systematic observation, testing, and analysis.

This thesis provides the general background information required for slope stability analysis, suitable methods of analysis with the use of computers, and examples of common stability problems in the location of the places Sikfors and Nystrand in North Sweden.

2 BACKGROUND

Historically, landslides on the area Sikfors and Nystrand have occurred and the soil has progressed into the river. Two important places from a slope stability point of view along the Piteå river is Sikfors and Nystrand.

2.1 Sikfors

Along road 374 there exist several places close to Sikfors where the stability of the Sikfors is not satisfactory. The cause of the collapse was partly the ice-clogged Piteå River at a narrow passage which make water rose to abnormally high levels in the mode for the current object see the Figure 2.1.

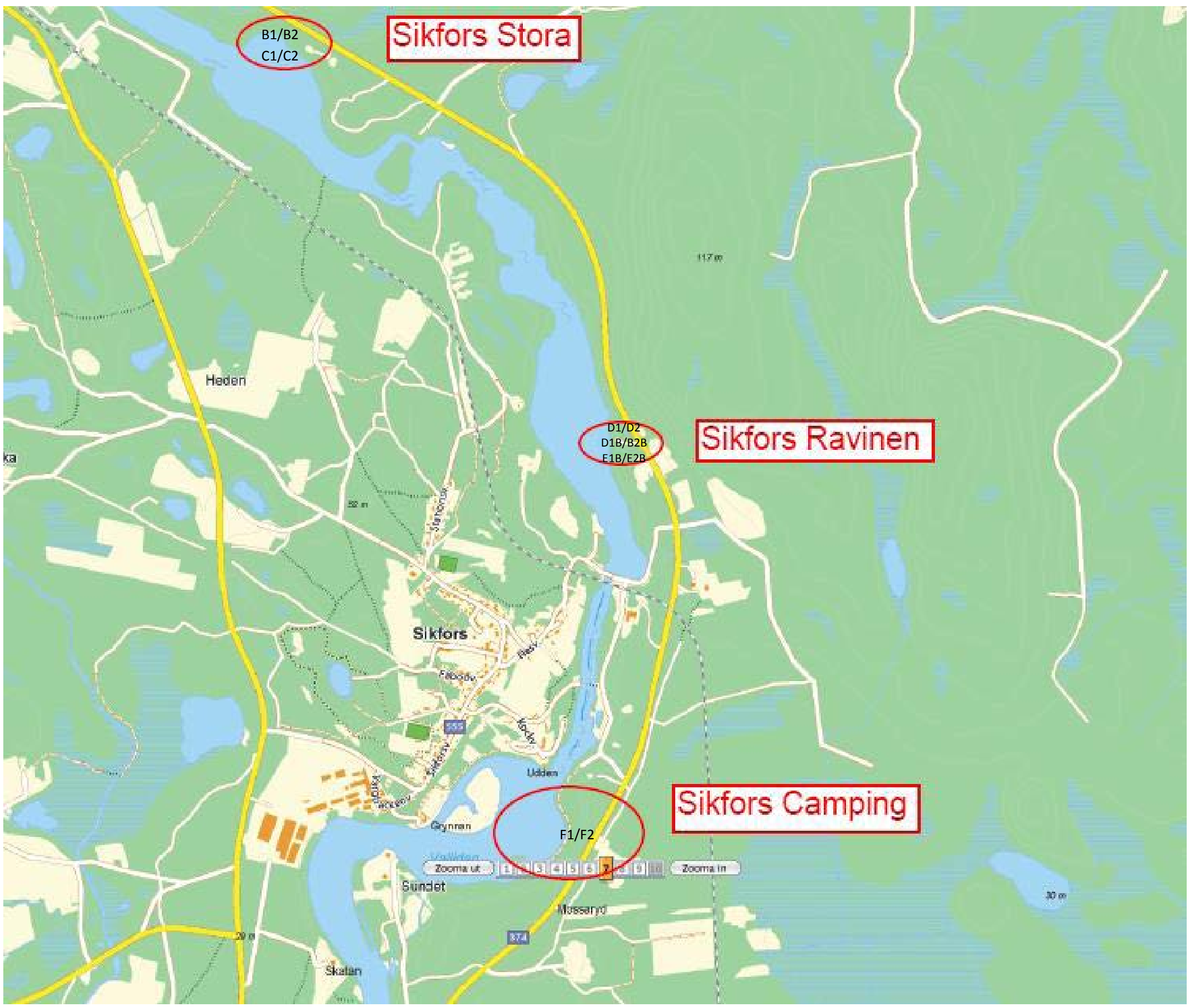

Figure 2.1: Road layout in Sikfors and approximate section location

The landslide took away about 25 m of the ground surface, which resulted in that the road were 40 meters closer to the river. After the landslide, inclinometers and extensometers were installed to control any further movement. Alarms have been linked via the GSM network. Reports have been sended directly to the control center (TIC). The stability is poor close to the river but the road is still expected to have a safe distance to the river. This applies to a direct landslide. However, it is difficult to predict what will happen if another ice plug occur.

2.2 Nystrand

Road 664 is located 15 to 20 meter from the slope crest at the riverbank at the locations N1 and N2 in figure 2.2. The riverbank area is more than 20 meters high and erosion progresses constantly at foot of the slope. The stability here might be very poor. The probability that the road shall be affected could be high. The expectation is that a primary landslide will not affect the road but eventual secondary landslides might bring the road into the river. The plan is that this road should be moved so it has a safe distance to the river. The planning work is going on and the goal is that the new road will be constructed this year. In order to have control on the current ongoing movements, inclinometers with alarms have been installed in the river bank.

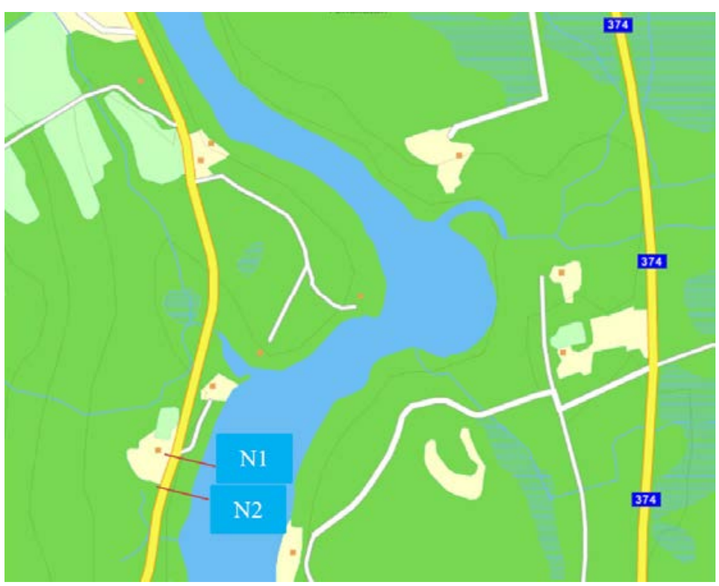

Figure 2.2: Road layout in Nystrand approximate section location.

3 OBJECTIVES

The primary purpose of slope analysis in many engineering applications is to contribute to the road safety analysis. Preliminary analyses assist in the identification of critical geological, material, environmental and economic parameters. Therefore the results are of value in planning detailed investigations of major projects. Subsequent analyses enable an understanding of the nature, magnitude and frequency of slope problems that may require to be solved. Evaluation of slope stability is often an inter-disciplinary effort requiring contributions from engineering geology, soil mechanics and rock mechanics. In this project the stability and the safety of a road will be evaluated at two different locations close to the Piteå river; Nystrand and Sikfors

3.1 Nystrand

At Nystrand the following points will be considered:

- Stability calculations with Slope/W and Plaxis 2D to find out factors of safety (FoS) compare the result with Novapoint GS Stability analysis
- Determination of critical failure surface of the slope
- Sensitivity analysis (cohesion and friction angle)

3.2 Sikfors

At Sikfors the followings point will be considered:

- Stability calculations with Slope/W and Plaxis 2D to obtain factors of safety (FoS) compare the result with Novapoint GS Stability analysis
- Determination of critical failure surface of the slope
- Determination of effects on slope stability in the case when the river water level goes up and then down but the embankment water level are constantly high

4 LITERATURE REVIEW

4.1 Ground Investigations

Before any further examination of an existing slope, or the ground onto which a slope is to be built, essential borehole information must be obtained. This information will give details of the strata, moisture content and the standing water level. Also, the presence of any particular plastic layer along which shear could more easily take place will be noted.

Ground investigations also include:

- In-situ and laboratory tests
- Aerial photographs
- Study of geological maps and memoirs to indicate probable soil conditions
- Visiting and observing the slope

For the study in this thesis, field investigations have been done by Tyrens AB and they used cone penetration test (CPT) for evaluation geotechnical parameters.

4.2 Geotechnical Parameters

Before a geotechnical analysis can be performed, the parameters values needed in the analysis must be determined.

4.2.1 Unit weight

Unit weight of a soil mass is the ratio of the total weight of the soil to the total volume of the soil. Unit weight, γ, is usually determined in the laboratory by measuring the weight and volume of a relatively undisturbed soil sample obtained from the field. Measuring unit weight of soil directly in the field might be done by sand cone test, rubber balloon or nuclear densiometer. We will use unit weights presented in a report by Tyrens AB.

4.2.2 Cohesion

Cohesion, *c,* is usually determined in the laboratory from the *Direct Shear Test.* Unconfined Compressive Strength S_{uc} can be determined in the laboratory using the *Triaxial Test* or the *Unconfined Compressive Strength Test*. There are also correlations for S_{uc} with shear strength as estimated from the field using *Vane Shear Tests*. Tyrens AB has already determined the cohesions for this project.

4.2.3 Friction Angle

The angle of internal friction, φ, can be determined in the laboratory by the Direct Shear Test or by Triaxial test. For our analysis we will use values determined by Tyrens AB.

4.2.4 Young's Modulus of Soil

Young's soil modulus, E_s, may be estimated from empirical correlations, laboratory test results on undisturbed specimens and results of field tests. Laboratory test that might be used to estimate the soil modulus is the triaxial test. For our analysis we will use values determined by Tyrens AB.

4.3 Type of soil

Geotechnical engineers classify soils, or more properly earth materials, for their properties relative to foundation support or use as building material. These systems are designed to predict some of the engineering properties and behavior of a soil based on a few simple laboratory or field tests

4.3.1 Sand

Soil material that contains 85% or more sand; the percentage of silt plus 1.5 times the percentage of clay does not exceed 15 (CSSC; USDA).

4.3.2 Clay

Soil material that contains 40% or more clay and 40% or more silt (CSSC; USDA).

4.3.3 Silt

Soil material that contains 80% or more silt and less than 12% clay (CSSC; USDA).

4.3.4 Silty clay

Soil material that contains 40% or more clay and 35% or more silt (CSSC; USDA).

4.3.5 Sandy clay

Soil material that contains 7 to 27% clay, 28 to 50% silt, and less than 52% sand (CSSC; USDA).

4.4 Basic Requirement for Slope Stability Analysis

Whether slope stability analyses are performed for drained conditions or undrained conditions, the most basic requirement is that equilibrium must be satisfied in terms of total stresses. All body forces (weights), and all external loads, including those due to water pressures acting on external boundaries, must be included in the analysis. These analyses provide two useful results: (1) the total normal stress on the shear surface and (2) the shear stress required for equilibrium.

The *factor of safety* for the shear surface is the ratio of the shear strength of the soil divided by the shear stress required for equilibrium. The normal stresses along the slip surface are needed to evaluate the shear strength: except for soils with $\varphi = 0$, the shear strength depends on the normal stress on the potential plane of failure.

In effective stress analyses, the pore pressures along the shear surface are subtracted from the total stresses to determine effective normal stresses, which are used to evaluate shear strengths. Therefore, to perform effective stress analyses, it is necessary to know (or to estimate) the pore pressures at every point along the shear surface. These pore pressures can be evaluated with relatively good accuracy for drained conditions, where their values are determined by hydrostatic or steady seepage boundary conditions. Pore pressures can seldom be evaluated accurately for undrained conditions, where their values are determined by the response of the soil to external loads.

In total stress analyses, pore pressures are not subtracted from the total stresses, because shear strengths are related to total stresses. Therefore, it is not necessary to evaluate and subtract pore pressures to perform total stress analyses. Total stress analyses are applicable only to undrained conditions. The basic premise of *total stress analysis* is this: the pore pressures due to undrained loading are determined by the behavior of the soil. For a given value of total stress on the potential failure plane, there is a unique value of pore pressure and therefore a unique value of effective stress. Thus, although it is true that shear strength is really controlled by effective stress, it is possible for the undrained condition to relate shear strength to total normal stress, because effective stress and total stress are uniquely related for the undrained condition. Clearly, this line of reasoning does not apply to drained conditions, where pore pressures are controlled by hydraulic boundary conditions rather than the response of the soil to external loads.

4.5 Drained and Undrained Strength

A distinction should be made between drained and undrained strength of cohesive materials. As cohesive materials or clays generally possess less permeability compared to sand, thus, the movement of water is restricted whenever there is change in volume. So, for clay, it needs years to dissipate the excess pore water pressure before the effective equilibrium is reached. Shortly, drained condition refers to the condition where drainage is allowed, while undrained condition refers to the condition where drainage is restricted. Besides, the drained and undrained condition of cohesive soils, it should be noted that there is a decline in strength of cohesive soils from its peak strength to its residual strength due to restructuring.

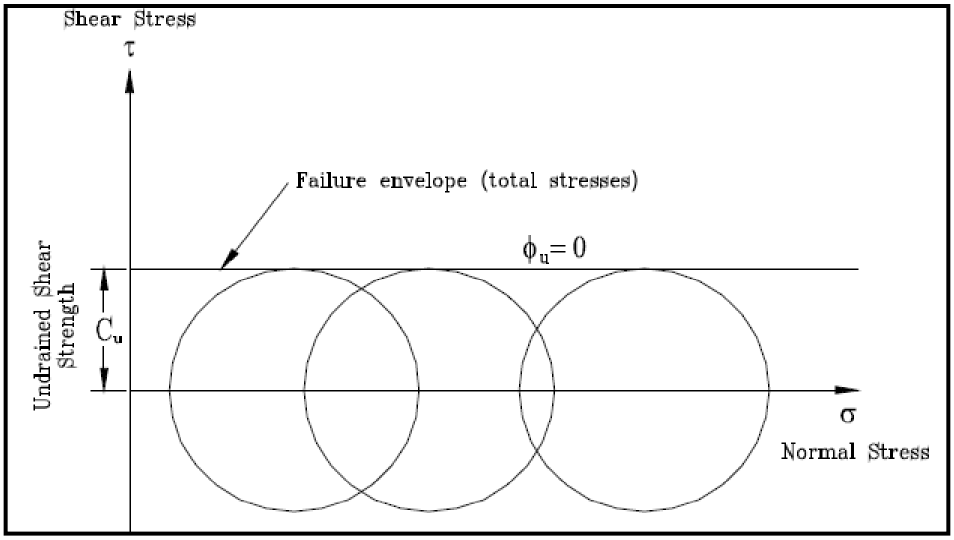

Figure 4.1a: Results of Triaxial Undrained Tests on Saturated Clay

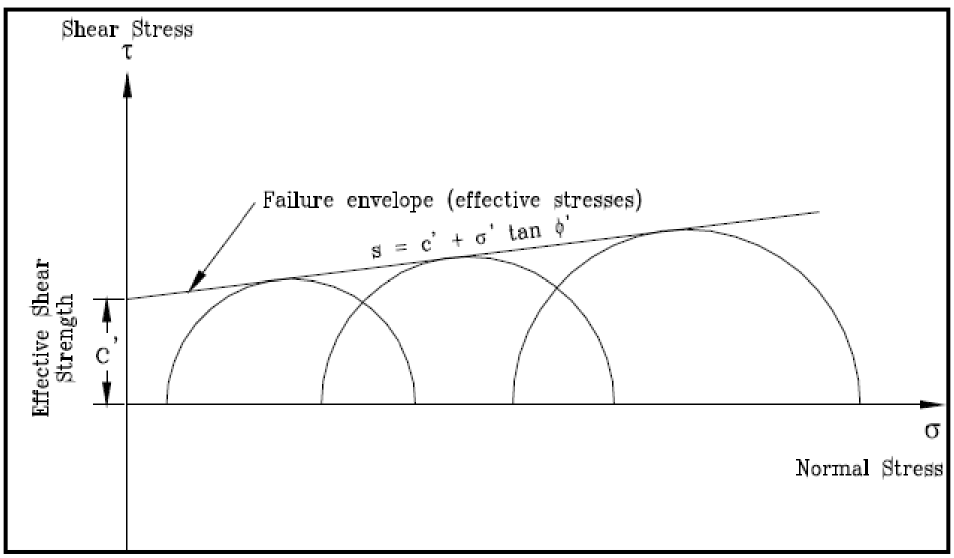

Figure 4.1b: Results of Triaxial Drained Tests on Saturated Clay

4.5.1 Analyses of Drained Conditions

Drained conditions are those where changes in load are slow enough, or where they have been in place long enough, so that all of the soils reach a state of equilibrium and no excess pore pressures are caused by the loads. In drained conditions pore pressures are controlled by hydraulic boundary conditions. The water within the soil may be static, or it may be seeping steadily, with no change in the seepage over time and no increase or decrease in the amount of water within the soil. If these conditions prevail in all the soils at a site, or if the conditions at a site can reasonably be approximated by these conditions, a drained analysis is appropriate. A *drained analysis* is performed using:

- Total unit weights
- Effective stress shear strength parameters
- Pore pressures determined from hydrostatic water levels or steady seepage analyses

4.5.2 Analyses of Undrained Conditions

Undrained conditions are those where changes in loads occur more rapidly than water can flow in or out of the soil. The pore pressures are controlled by the behavior of the soil in response to changes in external loads. If these conditions prevail in the soils at a site, or if the conditions at a site can reasonably be approximated by these conditions, an undrained analysis is appropriate. An *undrained analysis* is performed using:

- Total unit weights
- Total stress shear strength parameters

4.6 Short-Term Analyses

Short term refers to conditions during or following construction—the time immediately following the change in load. For example, if constructing a sand embankment on a clay foundation takes two months, the short-term condition for the embankment would be the end of construction, or two months. Within this period of time, it would be a reasonable approximation that no drainage would occur in the clay foundation, whereas the sand embankment would be fully drained.

4.7 Long-Term Analyses

After a period of time, the clay foundation would reach a drained condition, and the analysis for this condition would be performed as discussed earlier under "Analyses of Drained Conditions", because *long term* and *drained conditions* carry exactly the same meaning. Both of these terms refer to the condition where drainage equilibrium has been reached and there are no excess pore pressures due to external loads.

4.8 Pore Water Pressures

For effective stress analyses the basis for pore water pressures should be described. If pore water pressures are based on measurements of groundwater levels in bore holes or with piezometers, the measured data should be described and summarized in appropriate figures or tables. If seepage analyses are performed to compute the pore water pressures, the method of analysis, including computer software, which was used, should be described. Also, for such analyses the soil properties and boundary conditions as well as any assumptions used in the analyses should be described. Soil properties should include the hydraulic conductivities. Appropriate flow nets or contours of pore water pressure, total head, or pressure head should be presented to summarize the results of the analyses.

4.9 Soil Property Evaluation

The basis for the soil properties used in a stability evaluation should be described and appropriate laboratory test data should be presented. If properties are estimated based on experience, or using correlations with other soil properties or from data from similar sites, this should be explained. Results of laboratory tests should be summarized to include index properties, water content, and unit weights. For compacted soils, suitable summaries of compaction moisture–density data are useful. A summary of shear strength properties is particularly important and should include both the original data and the shear strength envelopes used for analyses (Mohr–Coulomb diagrams, modified Mohr–Coulomb diagrams). The principal laboratory data that are used in slope stability analyses are the unit weights and shear strength envelopes. If many more extensive laboratory data are available, the information can be presented separately from the stability analyses in other sections, chapters, or separate reports. Only the summaries of shear strength and unit weight information need to be presented with the stability evaluation in such cases.

4.10 Circular Slip Surface

Inherent in limit equilibrium stability analyses is the requirement to analyze many trials slip surfaces and find the slip surface that gives the lowest factor of safety. Included in this trial approach is the form of the slip surface; that is, whether it is circular, piece-wise linear or some combination of curved and linear segments. Slope/W has a variety of options for specifying trial slip surfaces. The position of the critical slip surface is affected by the soil strength properties. The position of the critical slip surface for a purely frictional soil (c = 0) is radically different than for a soil assigned untrained strength (φ = 0). This complicates the situation, because it means that in order to find the position of the critical slip surface, it is necessary to accurately define the soil properties in terms of effective strength parameters.

4.11 Factor of Safety

In slope stability, and in fact generally in the area of geotechnical engineering, the factor which is very often in doubt is the shear strength of the soil. The loading is known more accurately because usually it merely consists of the self-weight of the slope. The FoS is therefore chosen as a ratio of the available shear strength to that required to keep the slope stable. For highly unlikely loading conditions, accepted factors of safety can be as low as 1.2-1.25, even for dams e.g. situations based on seismic effects, or where there is rapid drawdown of the water level in a reservoir. According to TK Geo 11(Swedish Transport Administration requirements and guidelines) allowable limit for factor of safety is 1.5 for undrained analysis and 1.3 for combined or drained analysis.

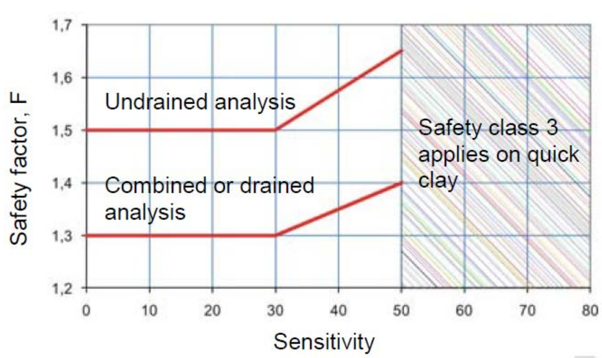

Figure 4.2: The minimum acceptable safety factor value for geotechnical structures on safety class 2 clay according to TK Geo 11

4.12 Traffic load

Traffic load refers to the action of the traffic on the carriageway or railway structure. Action distribution shall be taken into consideration using an elastic theoretical based method. Where there are low permeable soils the traffic load is to be reduced for drained and combined analysis. Normally the traffic load can be ignored for combined analysis and drained analysis in the above conditions. Account must be taken of the vehicles and other equipment used in the execution phase.

Design using partial factors. The characteristic surface load for traffic shall be:

- 15 kN/m^2 for design situations where the critical failure surfaces are short
- 10 kN/m^2 for design situations where the critical failure surfaces are long

Design using characteristic values. The characteristic surface load for traffic shall be:

- 20 kN/m^2 for design situations where the critical failure surfaces are short
- 13 kN/m^2 for design situations where the critical failure surfaces are long

4.13 Numerical analysis

Slope stability analyses can be performed using deterministic or probabilistic input parameters. Plaxis 2D and Geostudio (SLOPE/W) can model heterogeneous soil types, complex stratigraphic and slip surface geometry, and variable pore-water pressure conditions using a large selection of soil models.

5 METHODOLOGY

Many different solution techniques for slope stability analyses have been developed over the years. Analyze of slope stability is one of the oldest type of numerical analysis in geotechnical engineering. In this project we will use both Limit Equilibrium Method and Finite Element Method for our analysis. Two modern geotechnical software programs are utilized, i.e. Slope/W and Plaxis 2D.

5.1 Slope/W

5.1.1 Limit Equilibrium Methods

Modern limit equilibrium software is making it possible to handle ever-increasing complexity within an analysis. It is now possible to deal with complex stratigraphy, highly irregular pore-water pressure conditions, and various linear and nonlinear shear strength models, almost any kind of slip surface shape, distributed or concentrated loads, and structural reinforcement. Limit equilibrium formulations based on the method of slices are also being applied more and more to the stability analysis of structures such as tie-back walls, nail or fabric reinforced slopes, and even the sliding stability of structures subjected to high horizontal loading arising, for example, from ice flows.

5.1.2 Defining the Problem

A limit equilibrium analysis was carried out using the Slope/W software for the slope stability of the natural slope. The geometry was created in .dxt format and imported into the Slope/W program. The analysis type is then selected and it is determined that failure will follow a right to left path. The Morgenstern-Price analysis and half-sine function was selected but the software also gives the result of factor of safety for Ordinary, Bishop and Janbu analysis type.

5.1.3 Modeling

The most common way of describing the shear strength of geotechnical materials is by Coulomb's equation which is:

$$\tau = c + \sigma_n' \tan\varphi \quad \dots \dots \dots \dots \dots \dots \dots \ (5.1)$$

where, τ is shear strength (i.e., shear at failure), c is cohesion, σ'_n is normal stress on shear plane, and φ is angle of internal friction. The equation 5.1 represents a straight line on shear strength versus normal stress plot (Figure 5.1). The intercept on the shear strength axis is the cohesion c and the slope of the line is the angle of internal friction φ.

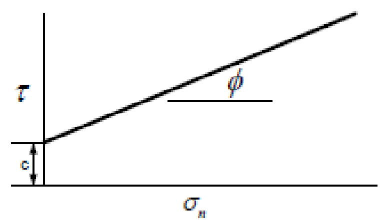

Figure 5.1: Graphical representation of Coulomb shear strength equation

The failure envelope is often determined from triaxial tests and the results are presented in terms of half-Mohr circles, as shown in Figure 5.2 and Figure 5.3, hence the failure envelope is referred to as the Mohr-Coulomb failure envelope.

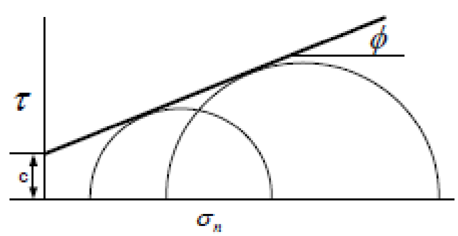

Figure 5.2: Mohr-Coulomb failure envelope

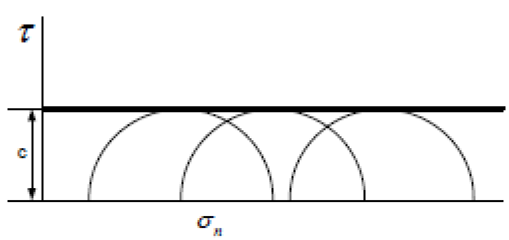

Figure 5.3: Undrained strength envelope

The strength parameters c and φ can be total strength parameters or effective strength parameters. Slope/W makes no distinction between these two sets of parameters. Which set is appropriate for a particular analysis is project-specific, and is something you as the software user, need to decide. The software cannot do this for you. From a slope stability analysis point of view, effective strength parameters give the most realistic solution, particularly with respect to the position of the critical slip surface.

5.1.4 Analysis Type

An analysis of slope stability begins with the hypothesis that the stability of a slope is the result of downward or motivating forces (i.e., gravitational) and resisting (or upward) forces. The resisting forces must be greater than the motivating forces in order for a slope to be stable. The relative stability of a slope (or how stable it is at any given time) is typically conveyed by geotechnical engineers through a factor of safety Fs defined as

$$Fs = \frac{\sum R}{\sum M} \dots \dots \dots \dots \dots \dots \dots \dots \dots (5.2)$$

The equation states that the factor of safety is the ratio between the forces/moments resisting (R) movement and the forces/moments motivating (M) movement.

5.1.4.1 Ordinary method of slices

This method neglects all interslice forces and fails to satisfy force equilibrium for the slide mass as well as for individual slices. However, this is one of the simplest procedures based on the method of slices (Fellenius, 1936). This method assumes a circular slip surface and it is also known as the Swedish Method of Slices or the Fellenius Method.

5.1.4.2 Simplified Bishop

The simplified Bishop method assumes that the vertical interslice shear force does not exist and the resultant interslice force is therefore horizontal (Bishop, 1955). It satisfies the equilibrium of moment but not the equilibrium of forces.

2.4.3. Janbu simplified method

This method uses the horizontal forces equilibrium equation to obtain the factor of safety. It does not include interslice forces in the analysis but account for its effect using a correction factor. The correction factor is related to cohesion, angle of internal friction and the shape of the failure surface (Janbu et al., 1956).

5.1.4.3 Spencer Method

This is a very accurate method which satisfies both equilibrium of forces and moments and it works for any shape of slip surface. The basic assumption used in this method is that the inclinations of the side forces are the same for all the slices.

5.1.4.4 Morgenstern and Price

Morgenstern and Price proposed a method that is similar to Spencer's method, except that the inclination of the interslice resultant force is assumed to vary according to a "portion" of an arbitrary function. This method allows one to specify different types of interslice force function (Morgenstern & Price, 1965).

5.1.4.5 General Limit Equilibrium

This method can be used to satisfy either force or moment equilibrium, or if required, just the force equilibrium conditions. It encompasses most of the assumptions used by various methods and may be used to analyze circular and noncircular failure surfaces (Ferdlund, Krahn, & Pufahl, 1981).

5.1.5 Slip Surface for Circular Failure Model

After the material input and pore pressure was assigned, a slip surface was defined. The analyses were performed for two failure models namely the circular failure model and block failure model. There were several methods for defining the slip surface for the circular failure but the entry and exit method was selected. One of the problems with the other methods is how to visualize the extents or the range of the trial slip surface. This difficulty is solved by the entry and exit method because it specifies the location where the trial slip surfaces should enter the ground surface and where they should exit.

5.1.6 Verification and Computation

When the slip surface has been specified, then Slope/W runs several checks to verify the input data using the verify/optimize data command in the Tools menu. When the verification is completed and there are no errors, then Slope/W computes the factor of safety using the method of slice selected. The minimum factor of safety is obtained for that particular analysis and its corresponding critical slip surface is displayed.

5.2 Plaxis 2D

5.2.1 Finite Element Modeling

The finite element program Plaxis 2D was used for evaluating the stability of natural slope. The natural slope cross-section utilized for the numerical model is presented in Appendix B.

5.2.2 Mesh Generation and Boundary Conditions

In this modeling, 15-node triangular elements were used; see figure 5.4 The mesh generation of PLAXIS version 8.0 used here follows a robust triangulation procedure to form 'unstructured meshes'. These meshes are considered to be numerically efficient when compared to regular 'structured meshes'. The powerful 15-node element provides an accurate calculation of stresses and failure loads. The two vertical boundaries are free to move, whereas the horizontal boundary is considered to be fixed as presented in Figure 5.4. The foundation soil was considered to be stiff and its stability is not considered in this analysis, therefore the bottom boundary is fixed.

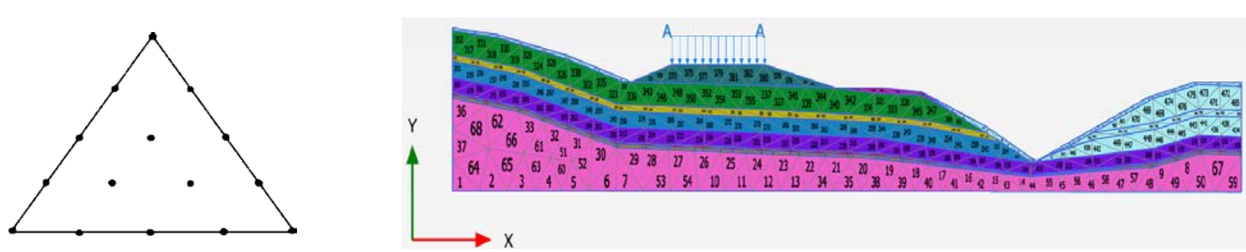

Figure 5.4: 15-nodded triangular element and cross section of generated mesh

5.2.3 Material Model

The Mohr–Coulomb model was used for this analysis. This model involves five parameters, namely Young's modulus, E, Poisson's ratio, ν, the cohesion, c, the friction angle, φ, and the dilatancy angle, ψ. In this case dilatancy angle was assumed to be zero, since it is close to zero for clay and for sands with a friction angle less than 38^0 (Lenita,T.).

5.2.4 Analysis Type

The factor of safety in PLAXIS was computed using *Phi-c* reduction at each case of slope modeling. In this type of calculation the load advancement number of steps procedure is followed. The incremental multiplier Msf is used to specify the increment of the strength reduction of the first calculation step. The strength parameters are reduced successively in each step until all the steps have been performed. The final step should result in a fully developed failure mechanism, if not the calculation must be repeated with a larger number of additional steps. Once the failure mechanism is reached, the factor of safety is given by (PLAXIS 2D manual)

$$SF = \frac{\text{Available Strength}}{\text{Strength at Failure}} = \Sigma \text{Msf value of Msf at failure} \ldots \ldots \ldots (5.3)$$

6 RESULT AND DISCUSSION

The stability of natural slopes ware analyzed for drained and undrained conditions by using both finite the element program Plaxis 2D and Limit Equilibrium Methods (LEM) slope stability software Slope/W. Results from slope stability analysis are presented in Appendix A and Appendix B. Appendix A shows the safety factors calculated by slope/W utilizing the Morgenstern-Price methods, Ordinary method, modified Bishop Method and Janbu method and Appendix B presents output of the total incremental displacements output from Plaxis 2D for both drained and undrained conditions. A distinction should be made between drained and undrained strength of cohesive materials. As cohesive materials or clays generally possess less permeability compared to sand, thus, the movement of water is restricted whenever there is change in volume. So, for clay, it takes years to dissipate the excess pore water pressure before the effective equilibrium is reached. Shortly, drained condition refers to the condition where drainage is allowed, while undrained condition refers to the condition where drainage is restricted. Besides, the drained and undrained condition of cohesive soils, it should be noted that there is a decline in strength of cohesive soils from its peak strength to its residual strength due to restructuring.

The existence of trivial failure surface is a large problem in stability analysis of natural slopes, especially in the Plaxis 2D program. We try to avoid these types of failure. That's why we sometimes cut some portions of the slope or use high strength soil parameters in exposed part, i.e. cohesion, $c' = 100$ Kpa, and friction angle, $\varphi = 45^0$. In Slope/W analysis we consider critical slip surface failure. If the critical failure is trivial, then we consider the secondary failure which present in Appendix A. In Plaxis 2D analysis, additional displacements are generated during a Safety calculation. The total displacements in the final step (at failure) give an indication of likely failure mechanism. The incremental displacement curve present in Appendix B. The shading of the total displacements indicating the most applicable failure mechanism of the embankment in the final stage.

In the calculations, according to Swedish road administration guideline (TK Geo 11), a traffic load of q = 20 kPa is used for design situations where the critical failure surfaces are short. Though, the road is relatively far from the river in all the analysed sections. So, traffic load impact is insignificant.

6.1.1 Sikfors Stora

Landslides on the slope in the area Sikfors Stora occurred very potently and the soil has progressed into the river. In this area we selected two section B and section C between the road 374 and the Piteå river for analysis. Soil properties were evaluated from by CPT sound test result presented in Table 6.1, Table 6.2, Table 6.3 and Table 6.4. The ground water table found in this area is approximately situated 6 meter below the ground surface at the crest in the spring season. In the autumn season we found that the groundwater levels the same as the river level.

Table 6.1: Geotechnical parameters of section B1/B2 for the different layers

Soil Layer	γ_{sat} (KN/m³)	γ_{unsat} (KN/m³)	Friction Angle, φ (⁰)	Undrained Shear Strength, τ (KPa)	Cohesion c' (KPa)	Young's Modulus E (MPa)	Poisson Ratio, v
saSi	17	11.11	34	-	10.0	3.00	0.33
Si	17	11.11	38	-	10.0	2.00	0.33

Table 6.2: Geotechnical parameters of section C1/C2 for different layers

Soil Layer	γ_{sat} (KN/m³)	γ_{unsat} (KN/m³)	Friction Angle, φ (⁰)	Undrained Shear Strength, τ (KPa)	Cohesion c' (KPa)	Young's Modulus E (MPa)	Poisson Ratio, v
Sa/Si	18	12.70	33	-	10	15.90	0.33
Si	15	7.94	28	-	10	19.20	0.33
MSa	18	12.70	34	-	-	19.40	0.33
Cl	15	7.94	28	15 - 80	1.5 - 8	2.00	0.33
saSi	18	12.70	35	-	10	39.25	0.33

Table 6.3: Geotechnical parameters of section D1/D2 for different layers

Soil Layer	γ_{sat} (KN/³)	γ_{unsat} (KN/m³)	Frictio Angle, φ (⁰)	Undrained shear Strength, τ (KPa)	Cohesion c´ (KPa)	Young's modulus *E* (MPa)	Poisson Ratio, *v*
Sa	18	12.70	32	-	-	15.00	0.33
saSi	17	11.11	29	-	10	19.00	0.33
MSa	18	12.70	30	-	-	14.00	0.33
saSi	17	11.11	29	-	10	8.95	0.33
Susicl	15	7.94	26	20-40	2-4	2.50	0.33
Sa	18	12.70	30	-	-	20.00	0.33
Si	17	11.11	26	70	7	4.80	0.33
siMSa	18	12.70	28	-	-	12.00	0.33

Table 6.4: Geotechnical parameters of section D.1B/D.2B for different layers

Soil Layer	γ_{sat} (KN/m³)	γ_{unsat} (KN/m³)	Friction Angle, φ (⁰)	Undrained Shear Strength, τ (KPa)	Cohesion c´ (KPa)	Young's Modulus *E* (MPa)	Poisson Ratio, *v*
Sa	20	15.882	32	-	1	15.00	0.33
saSi	17	11.118	29	-	1	19.00	0.33
MSa	18	12.706	30	-	0	14.00	0.33
saSi	17	11.118	29	-	0	9.00	0.33
Susicl	15	7.941	26	18-28	1.8-2.8	2.00	0.33
Sa	18	12.706	33	-	0	20.00	0.33
Si	17	11.118	26	19-38	1.9-3.8	3.00	0.33
siMSa	18	12.706	37	-	0	19.00	0.33

6.1.1.1 Section B1/B2

In section B1, it is a low ground water table in the slope and a low water in the river. In section B2 the pore pressures are high because of a high ground water table in the slope while the water level is low in the river. Most likely the worst case scenario occurs when the river water level is increased rapidly, and then quickly drops while the water table in the embankment is retained on an extremely high level so that the low effective stresses might lead to failure. The worst case scenario has been simulated in the calculation cases section B2 (Appendix A) and the results show that the slope computationally under these conditions is stable. The more favorable condition after drainage, when the groundwater table in the slope is on the same level as the water level in the river, has been simulated in the calculation case of section B1. Figure 6.1,

Figure 6.2 and Figure 6.3 shows the sensitivity analysis in section B1 under drained condition. In Table: 6.5 content different analysis result. In section C1 and C2, we find factor of safety below the allowable limit due to the trivial failure. The results of these calculations show that the slope is stable and meet with Swedish road administration guide line TK Geo 11.

Table 6.5: Safety factor for Sikfor Stora

Section Name	Slope/W		Plaxis 2D		Tyrens Analysis		TK Geo Allowable	
	D*	UD*	D*	UD*	UD*	C*	UD*	C*
B1	1.830	1.814	2.08	1.87	-	1.72	1.5	1.3
B2	1.355	1.146	1.50	0.88	-	1.30	1.5	1.3
C1	1.312	0.994	1.23	1.148	-	1.10	1.5	1.3
C2	0.879	0.747	1.22	0.60	-	0.62	1.5	1.3

*D = Drained, *UN = Undrained and *C = Combined Analysis

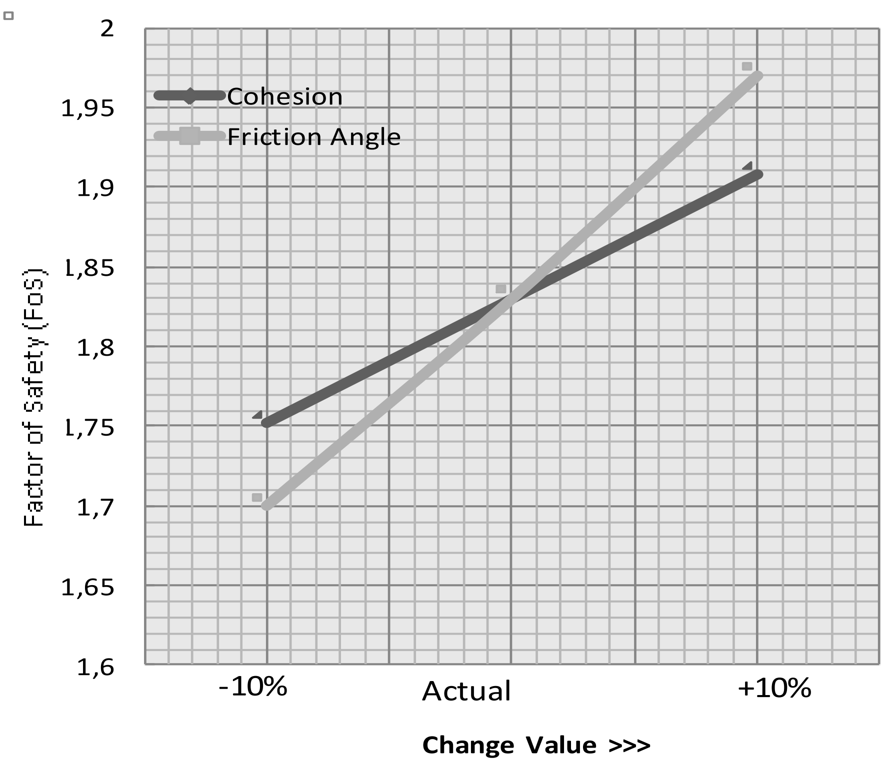

Figure 6.1: Section B1 sensitivity analysis for friction angle and cohesion in drained condition

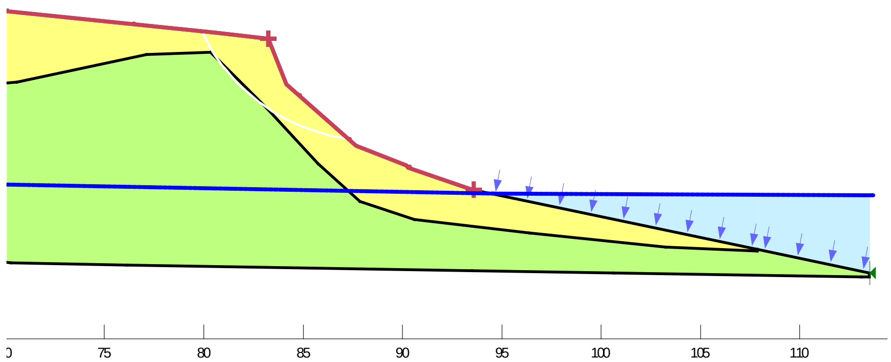

Figure 6.2: Safety map in drained condition at section B1

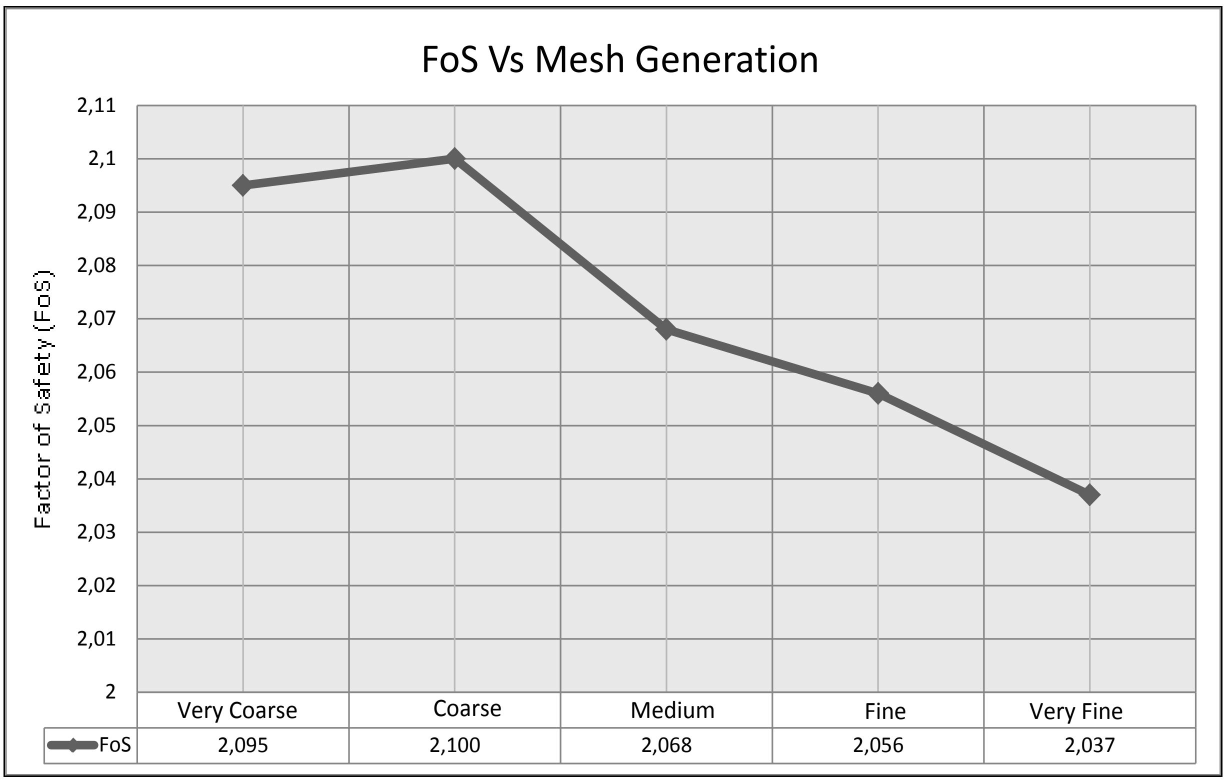

Figure 6.3: Factor of Safety variation due to Mesh Generation in Plaxis at section B1

In section B1/B2, we found that the soil is cohesive (i.e. Silt and sandy Silt). The cohesion of a clay soil changes significantly depending on the presence of water. In dry conditions clay soils can break up into lumps. If the soil is very dry and the lumps are small then a clay soil can behave (at least locally) very much like a frictional soil. In figure 6.1 show that, with friction angle, φ^0 and cohesion, c change constantly where the safety factor change linearly.

6.1.1.2 Section C1/C2

In section C1, it is a low groundwater table in the slope and a low water level in the river and in section C2; it is a high groundwater table in the slope and a low water in the river. In the same way as in section B1/B2 is the most likely worst case scenario simulated for section C1/C2.

In section C2 the pore pressures are high because of the high groundwater table in the slope. Most likely the worst case scenario occurs when the river water level is increased rapidly, and then quickly drops while the water in the embankment is retained on an extremely high level so that the low effective stresses might lead to failure. The slope is not smooth. At the bottom portions of the slope, the inclination is quite low (36^0) and at the top of the slope, the inclination are quite high (65^0). In this section we found different type of soil layer, i.e., sand, silt and medium sand. The worst case scenario has been simulated in the calculation cases section C2 (Appendix A) and the results show that the slope computationally under these conditions is stable. The silt layer is the most important factor to occur the failure of the slope. From Figure 6.4 we can find out the different slip surface. The more favorable condition after drainage, when the groundwater table in the slope is on the same level as the water level in the river, has been simulated in the calculation case of section C1. Here the drained calculations are more important. The results of these calculations show that the slope is stable and meet with Swedish road administration guide line TK Geo 11.

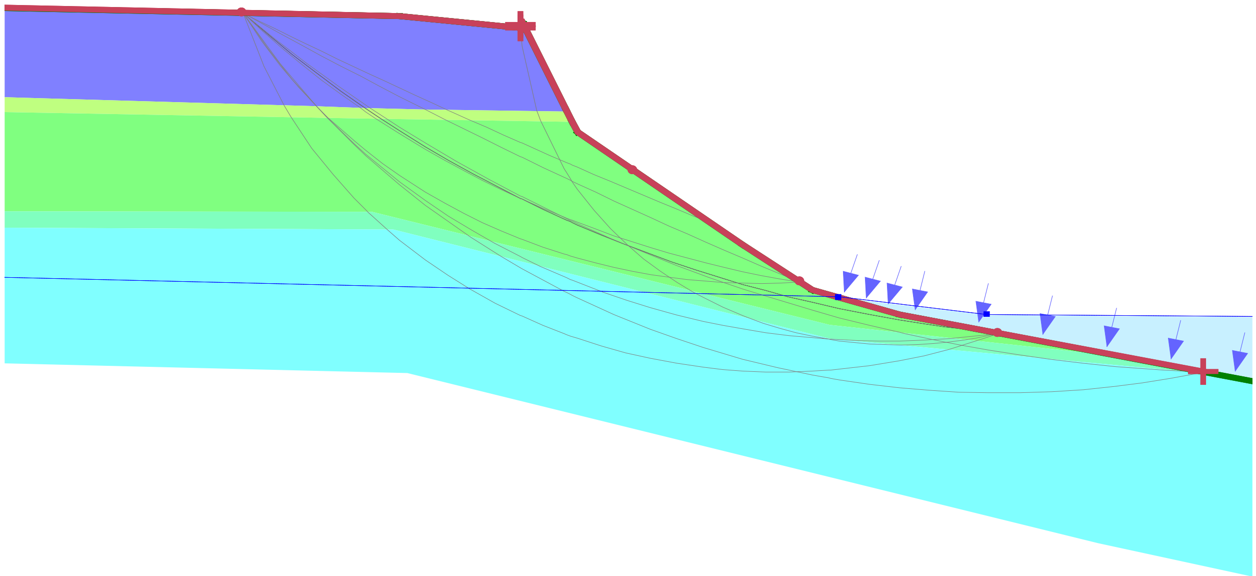

Figure 6.4: Multiple slip surfaces in drained condition at section C1

6.1.2 Sikfors Ravinen

The canyon forms a wedge-shaped area between the road and ravine, and the in this area, several changes occurred in the form of level differences a couple of millimeter along a suspected fracture. In the ravine, road 374 slope are clear signs of movement. A calculation case with increased pore pressure levels in the sand layer and higher pore pressures in the overlying sulphide-clay soils is studied. The sulphide-clay soil layer is situated approximately 2.5 m below from surface.

6.1.2.1 Section D1/D2

The stability calculations on the slopes of section D1 show that they are stable under both drained and undrained conditions. The road 374 is not stable in the Ravine area according to the conditions in the Swedish road administration guideline (TK Geo 11). In this region, an approximately 1.5 thick layer of suitable sulfide silty clay or clayey silty sulfide is situated 4m below the ground surface. This layer has a great impact on the slope stability, because the friction angle is low (26^0) and the density is also low which means that this is a low strength layer. The groundwater flow in the sand coming from the mountain to the east has been translated into pressure level +40.5m. These imply a raised pore pressure in the overlying cohesive soil. The sand layer above the cohesive soil is considered as drained. This scenario may be possible in spring and early summer when water infiltration is high into the soil. Figure 6.5 shows sensitivity analysis of friction angle in section D1 for the most dangerous slip surface. The results of these calculations in the Table: 6.6 show that the road 374 is stable but not within allowable limit according to TK Geo 11.

Table 6.6: Safety factor for Sikfors Ravinen

Section	**Slope/W**		**Plaxis 2D**		**Tyrens Analysis**		**TK Geo Allowable**	
Name	D*	UD*	D*	UD*	UD*	C*	UD*	C*
D.1	1.27/2.21	1.23/2.87	1.21	1.83	2.55	2.22	1.5	1.3
D.2	1.38/3.92	2.895	1.46	1.79	2.38	1.07	1.5	1.3
D.1B	1.04/2.26	1.45/2.37	1.21	1.83	1.88	1.75	1.5	1.3
D.2B	1.38/3.92	2.895	1.46	1.79	1.33	0.83/0.44	1.5	1.3
E.1B	1.14/2.00	2.85	1.85	1.80	2.71	1.96	1.5	1.3
E.2B	2.606	2.980	1.74	1.62	1.66	1.62	1.5	1.3

*D = Drained, *UN = Undrained and *C = Combined Analysis

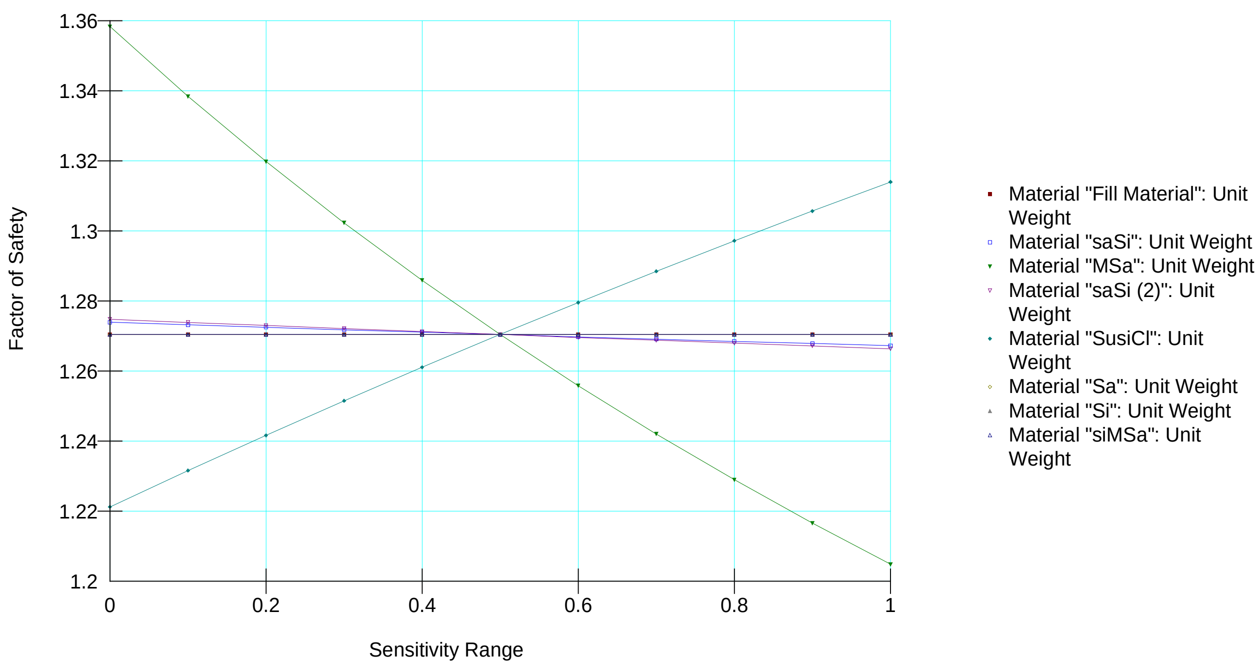

Figure 6.5: Sensitivity analysis of friction angle in section D1 for the most dangerous slip surface

6.1.1.2 Section D.1B/D.2B

The stability calculation performed in drained and undrained conditions with Slope/W and Plaxis 2D. The result (Table: 6.5) shows that the slopes are stable in both cases but the safety factors are not acceptable according to Swedish road administration guidelines TK Geo. The characteristics of these slopes are similar to section D1/D2. In this calculation the section D2B represents an elevated pressure level simulated in the top sand layer at the level of about +32m.

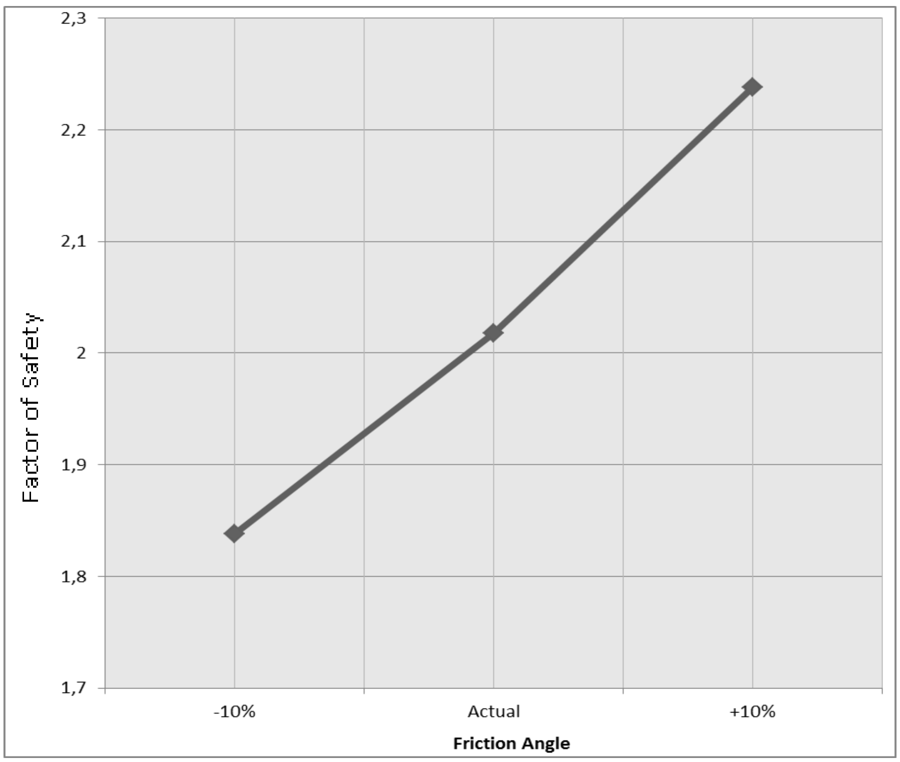

Figure 6.6: Sensitivity analysis of friction angle in section D1B/D2B for the most dangerous slip surface

In this region soil have approximate 1.5m thick sulfide silty clay or clayey silt sulfide layer 4m bellow from ground surface. This layer have a great impact in the slope stability, because this soil friction angle little bit low 26^0 and density are also low, means this layer is a low strength soil. The groundwater flow in the sand coming from the mountain to the east has been translated into pressure level +40.5m. This means raised pore pressure in the overlying cohesive soil. Sand layer is considering to be draining above cohesive soil. This scenario may be possible in spring and early summer when water infiltration is high into the soil. In section D2B we found some trivial failures in analysis with Slope/W and Plaxis2D analysis which are present in Appendix A and Appendix B. These trivial failures cannot cause any real danger for the stability of the slope.

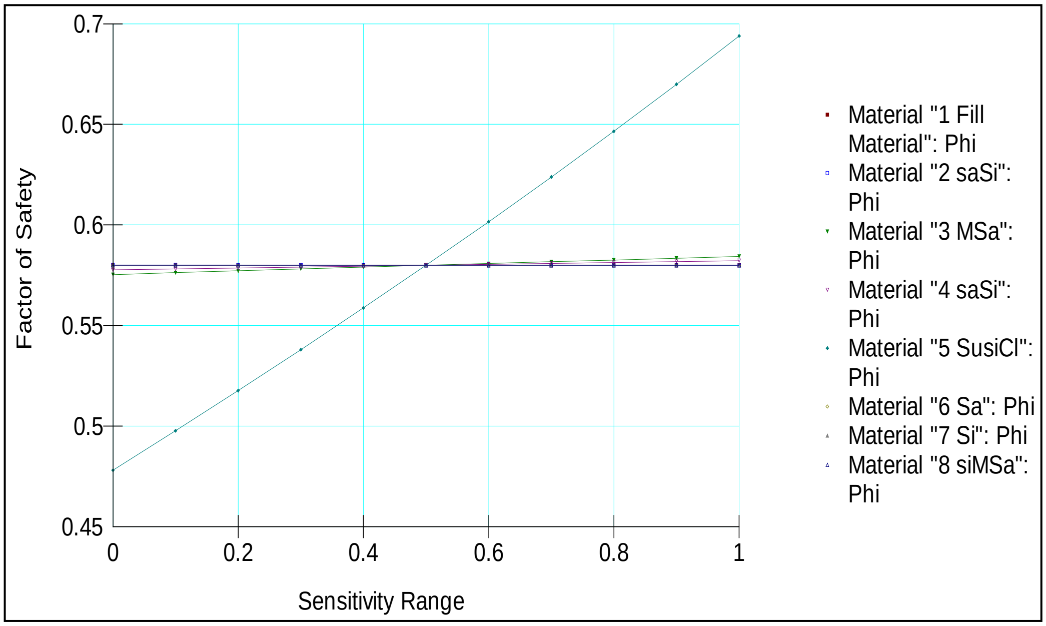

Figure 6.7: Sensitivity analysis of friction angle in section D.1B/D.2B considering individual materials for critical slip surface

6.1.2.3 Section E1B/E2B

The results of the performed stability calculations in the section E1B shows that the ravine slope can be assumed to be stable in the drained and undrained case. In the section E2B, an elevated pressure level is simulated in the thin layer of sand identified by the CPT sounding test at the level of about +32.5m. A groundwater flow in the sand from the hill to the toe is confirmed. This implies an increased pore pressure in the overlying cohesive soil. The sand layer above the cohesive soil is considered as drained. This scenario can be possible, for example in the spring and early summer when the water infiltration into the soil is high. The calculated factors of safety for those sections are presented in Table 6.6, the position of slip surfaces are presented in Appendix A and total incremental displacement presented in Appendix B.

6.1.3 Sikfors Camping

The slope is very steep between the road 374 and the Piteå River. The soil model for the cross section is constructed based on results from weight probes and CPT sounding tests conducted in this area. Soil properties of the materials were evaluated by means of interpreted probing results and empirical correlations based on TK Geo. Soil material properties are present in Table 6.6.

Table 6.7: Geotechnical parameters of section F1/F2 for the different soils in the profile

Soil Layer	γ_{sat} (KN/m^3)	γ_{unsat} (KN/m^3)	Friction Angle, φ (^{0})	Undrained Shear Strength, τ (KPa)	Cohesion c' (KPa)	Young's Modulus E (MPa)	Poisson's Ratio, ν
Filling (Sa)	18	12.70	32	-	-	14.00	0.33
Sa	18	12.70	34	-	-	20.00	0.33
suSi	15	7.94	29	100	10	3.00	0.33
suCl	15	7.94	28	100	10	2.00	0.33
Sa	18	12.70	34	-	-	20.00	0.33
Sa/Si	15	7.94	30	-	10	14.00	0.33
Sa	16	9.52	32	-	-	20.00	0.33
SaMn	20	15.88	32	-	-	19.00	0.33

6.1.3.1 Section F.1/F.2

The results of the stability calculations in the section F1/F2 show that the slope can be assumed to be stable both for drained and undrained conditions. In both analyses we found short slip surfaces with low safety factor (below 1.0) that do not have any direct effect on the road. In section F2, moderately higher pore pressures have been simulated in the sand layer between the levels +13 and +18.5. A groundwater flow in the sand has been assessed as pressure level +26. This implies an increased pore pressure in the overlying cohesive soil. The sand layer above the level of +23 is considered to be drained. This scenario may be realistic, for example, in the spring and early summer when water infiltration into the soil is high. The results of these calculations show that the unstable situations of road 374 exist during undrained conditions. The analysis results presented in Table 6.7

Table 6.8: Safety factor for Sikfors Camping

Section Name	Slope/W		Plaxis 2D		TyrensAnalysis		TK Geo Allowable	
	D*	UD*	D*	UD*	D*	C*	UD*	C*
F1	0.94/1.74	1.49/1.72	0.93	0.98/1.18	1.68	1.36/0.76	1.5	1.3
F2	0.67/1.36	0.87/1.18	0.51	0.59	1.20	1.17/0.30	1.5	1.3

*D = Drained, *UN = Undrained and *C = Combined Analysis

6.2 Nystrand

A limited geotechnical investigation was made on two sections between road 664 and Piteå River in Nystrand, in Älvsbyns municipality. The Approximate location of the sections N1 and N2 are presented in Figure 2.2. The distance between the edge of the road and the river slope crest in the sections is about 40 meters and the height from the crest of the river embankment of the water level in the river is 15 meters under the normal condition. The analyses have been performed to check the slope stability in the road According to the Tyrens AB report (Lenita, T. 2011). The major parts of the steep river embankment consist of sand and silty sand with inserted sulphide at depth soil layers. In section N1 has two different geometric models used in the calculations. In the section N1b, more thin layers of sand and sulphide have been used at depth in the soil profile for for our analysis for a more detailed inventory. Soil material properties are presented in Table 6.9, Table 6.10 and Table 6.11

Table 6.9: Geotechnical parameters of section N1 for the different soils in the profile

Soil Layer	γ_{sat} (KN/m^3)	γ_{unsat} (KN/m^3)	Friction Angle, φ (^{0})	Undrained Shear Strength, τ (KPa)	Cohesion c' (KPa)	Young's Modulus E (MPa)	Poisson's Ratio, ν
Filling (Sa)	20	15.882	32	100	0	14.00	0.33
Si	17	11.118	26	100	0	3.00	0.33
MSa	18	12.706	31	100	0	19.00	0.33
Si	17	11.118	29	100	0	5.00	0.33
siSa	18	12.706	29	100	0	14.00	0.33
clSi/(cl)Si	17	11.118	30	80, 20-80	8, 2-8	15.00	0.33
susiCl	16	9.529	29	60, 40-75	6, 4-7.5	2.00	0.33
MSa	18	12.706	33	100	0	14.0	0.33

Table 6.10: Geotechnical parameters of section N1b for the different soils in the profile

Soil Layer	γ_{sat} (KN/m^3)	γ_{unsat} (KN/m^3)	Friction Angle, φ (^{0})	Undrained Shear Strength, τ (KPa)	Cohesion c' (KPa)	Young's Modulus E (MPa)	Poisson's Ratio, *v*
Filling (Sa)	20	15.882	32	100	0	14.00	0.33
Si	17	11.118	26	100	0	3.00	0.33
MSa	18	12.706	31	100	0	19.00	0.33
Si	17	11.118	29	100	0	5.00	0.33
siSa	18	12.706	29	100	0	14.00	0.33
clSi/(cl)Si	17	11.118	30	80, 20-80	8, 2-8	3.00	0.33
susiCl	16	9.529	29	60, 40-75	6, 4-7.5	2.00	0.33
MSa	18	12.706	33	100	0	19.00	0.33
susiCl	16	9.529	29	100-150	10-15	5.00	0.33
MSa	18	12.706	33	100	0	19.00	0.33
siSa	18	12.706	29	100	0	14.00	0.33
MSa	18	12.706	35	100	0	19.00	0.33

Table 6.11: Geotechnical parameters of section N2 for the different soils in the profile

Soil Layer	γ_{sat} (KN/m^3)	γ_{unsat} (KN/m^3)	Friction Angle, φ (^{0})	Undrained Shear Strength, τ (KPa)	Cohesion c' (KPa)	Young's Modulus E (MPa)	Poisson's Ratio, *v*
Filling (Sa)	20	15.882	32	100	0	14.00	0.33
Si	17	11.118	28	100	10	3.00	0.33
MSa	18	12.706	31	100	0	19.00	0.33
vsiSa	18	12.706	29	100	0	12.00	0.33
Si	17	11.118	28	90, 20	9-2	2.00	0.33
sisuCl/siSuCl	15	7.941	29	45-66, 35-45	4.5-6.6, 3.5-4.5	3.00	0.33
Sa	18	12.706	35	100	0	20.00	0.33

6.2.1 Section N1/N1b

The results show that the embankment and the road 664 are not stable in both analyses for the circular-cylindrical sliding surfaces. From the Slope/W analysis we found the factor of safety for the critical slip surface to be 0.663 in drained analysis and 1.99 in undrained analysis respectively. The 20 kPa traffic load has been used where the failure surface are short.

In section N1b, we used more details thin geotechnical layer for analysis which evaluate by CPT sounding test. The results show that the road 664 is not stable for circular slip surfaces which develop under the road in a drained analysis factor of safety is 0.66 but allowable requirement is 1.5. In an undrained analysis is the road is stable factor of safety with safety factor of 1.96 compared with the requirement Fc = 1.3. The figure shows probability of failure under drained condition at section N1 The road is not directly affected due to trivial short circular slip surfaces but after such trivial failures the road might be unstable. The undrained shear strength has been evaluated from CPT sounding test results.

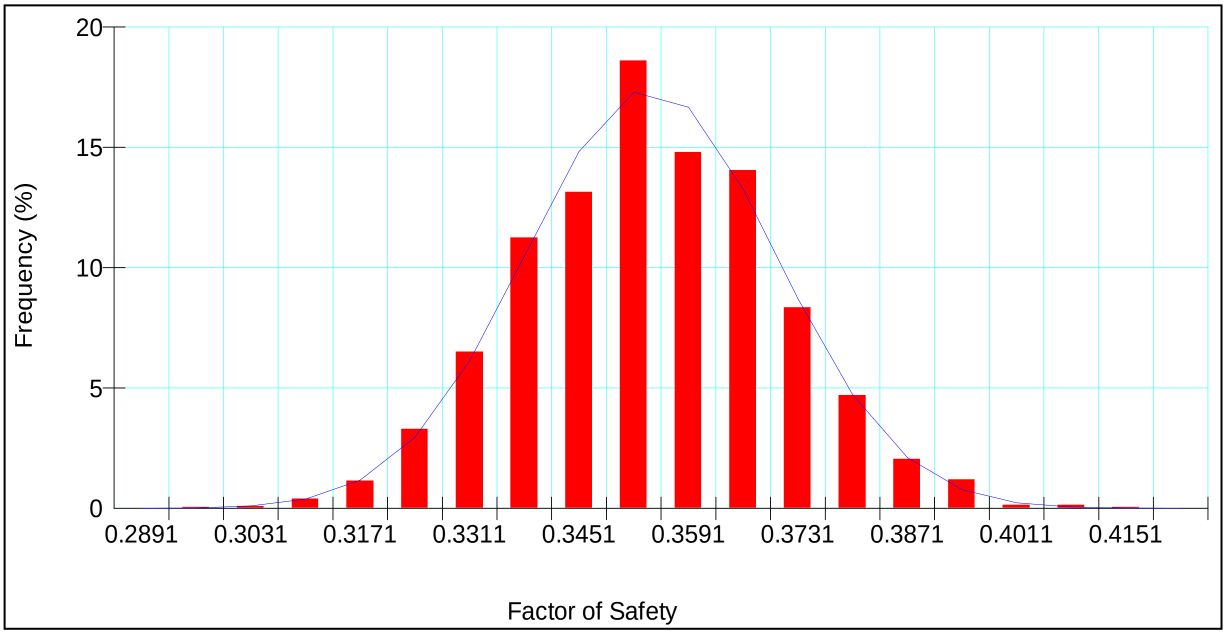

Figure 6.7: Probability of failure under drained condition at section N1

Table 6.10: Safety factors for Nystrand

Section Name	Slope/W		Plaxis 2D		Tyrens Analysis		TK Geo Allowable	
	D*	UD*	D*	UD*	UD	C*	UD*	C*
N1	0.66/1.16	1.92/1.95		0.24	0.65/1.13	0.70/1.16	1.5	1.3
N1b	0.66/1.44	1.96/2.04	0.60	0.40	0.73/1.40	0.61/1.30	1.5	1.3
N2	0.77/1.21	1.77/1.77	0.32	0.62	0.74/1.13	0.80/1.21	1.5	1.3

*D = Drained, *UN = Undrained and *C = Combined Analysis

6.2.2 Section N2

The results show that the present river embankment and road 664 is not stable for the circular-cylindrical sliding surfaces. In undrained analysis higher values of the safety factor were obtained. The undrained shear strength has been evaluated values from CPT sounding test results. CPT sounding test is not reliable all time. In this section we found very big difference between drained and undrained shear strength. So, the drained and undrained analysis is not comparable.

7 CONCLUSIONS

Natural slope instability is a major concern in the area of Sikfors and Nystrand where failures might cause catastrophic destruction on the surrounding area. The failures might be triggered by internal or external factors that cause imbalance to natural forces. An internal triggering factor is the factor that causes failure due to internal changes, such as increasing pore water pressure and or imbalanced forces developed due to external load.

Plaxis 2D is not good enough for natural slope stability analysis due to trivial failures and can not indicate exact slip surface location. On the other hand, with Slope/W it is easy to find out the position of the critical slip surface, safety map, probabilistic failure and exact factor of safety. The factor of safety computed from both Slope/W and Plaxis 2D decreases as the slope angle becomes larger. The Limit equilibrium method overestimated the factor of safety as compared to the Finite element method.

A distinction should be made between drained and undrained strength of cohesive materials. As cohesive materials or clays generally possess less permeability compared to sand, thus, the movement of water is restricted whenever the soil is located. So, for clay, it might take years to dissipate the excess pore water pressure before the effective equilibrium is reached. Shortly, drained condition refers to the condition where drainage is allowed, while undrained condition refers to the condition where drainage is restricted. Sensitivity analyses performed indicate that an increase in the friction angle and in the cohesion increases the factor of safety. Therefore the stability analysis is much more sensitive to changes in friction angle and cohesion than the unit weight of the layers. It was found that CPT sounding test result were not reliable all the time. Trivial failures do not directly influence the stability of the road but might progressively lead to failure. Results from this study indicates that Nystrand and Sikfors Stora are critical places from a slope stability point of view. Safety factors below the allowable limit have been obtained on these places.

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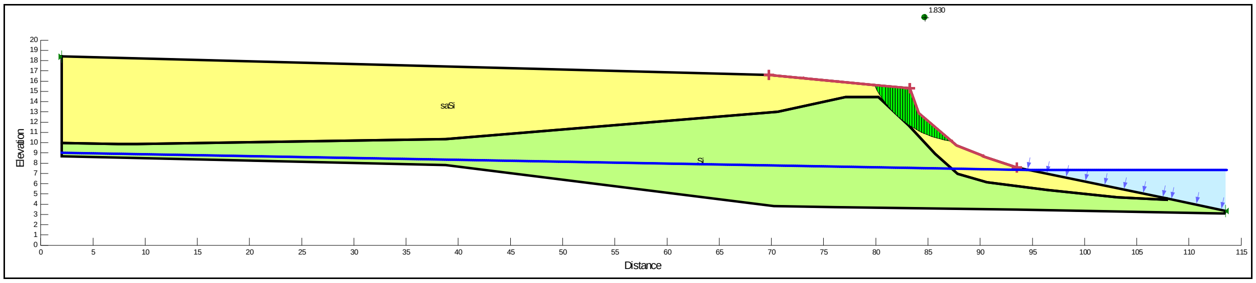
Appendix A

Result from Slope/W analyses presented in Appendix A both for drained and undrained conditions. People should be able to find them based on the information given.

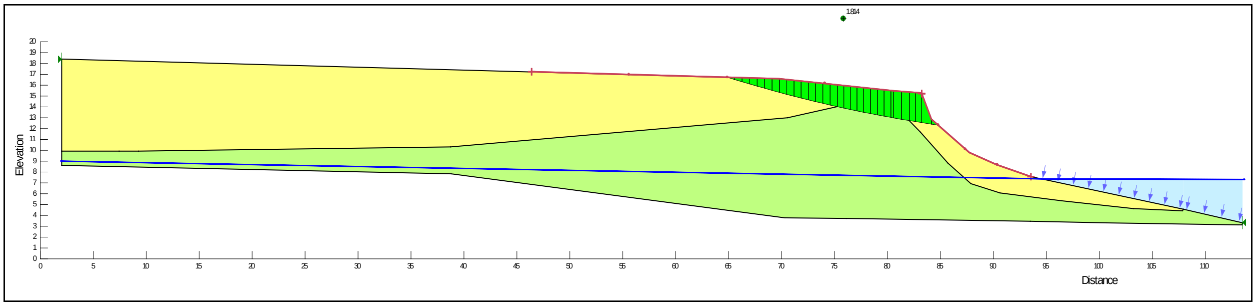
Section: B1 Drained



Factor of Safety (FoS):

- Morgenstern-Price - 1.830
- Ordinary - 1.782
- Bishop - 1.840
- Janbu - 1.783

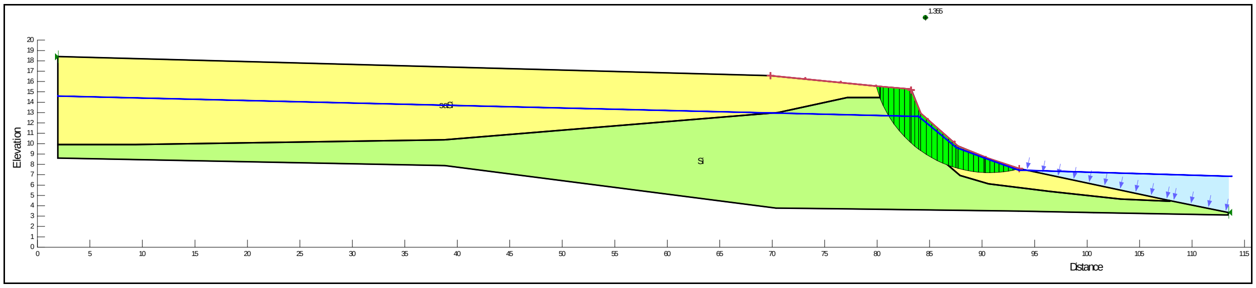
Section: B1 Undrained



Factor of Safety (FoS):

- Morgenstern-Price - 1.814
- Ordinary - 1.814
- Bishop - 1.814
- Janbu - 1.818

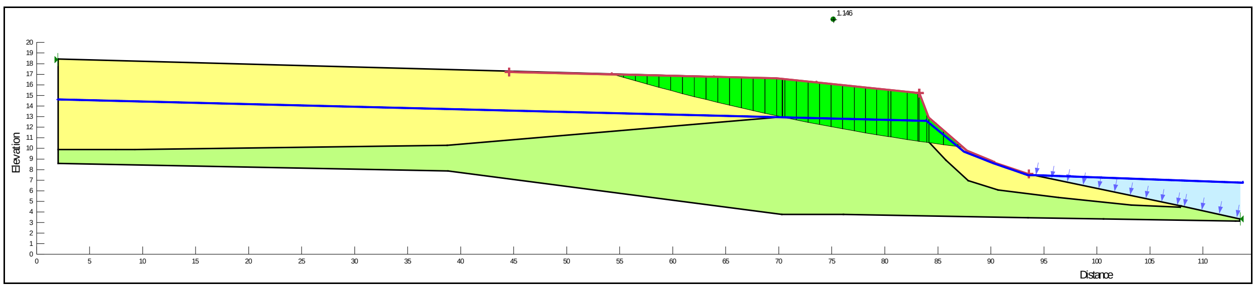
Section: B2 Drained



Factor of Safety (FoS):

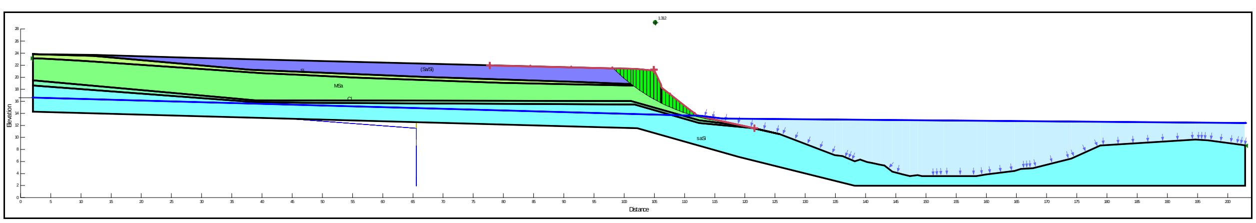
- Morgenstern-Price - 1.355
- Ordinary -1.187
- Bishop - 1.361
- Janbu - 1.250

Section: B2 Undrained



- Factor of Safety (FoS):
- Morgenstern-Price - 1.146
- Ordinary - 1.146
- Bishop - 1.146
- Janbu - 1.148

Section: C1 Drained

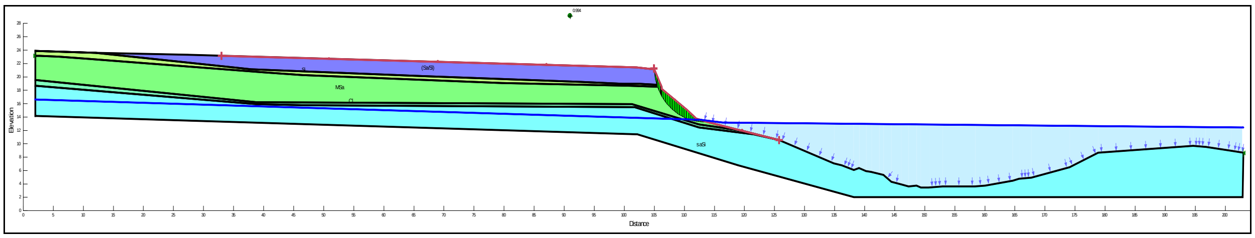


Factor of Safety (FoS):

- Morgenstern-Price - 1.312

- Ordinary - 1.306
- Bishop - 1.314
- Janbu - 1.309

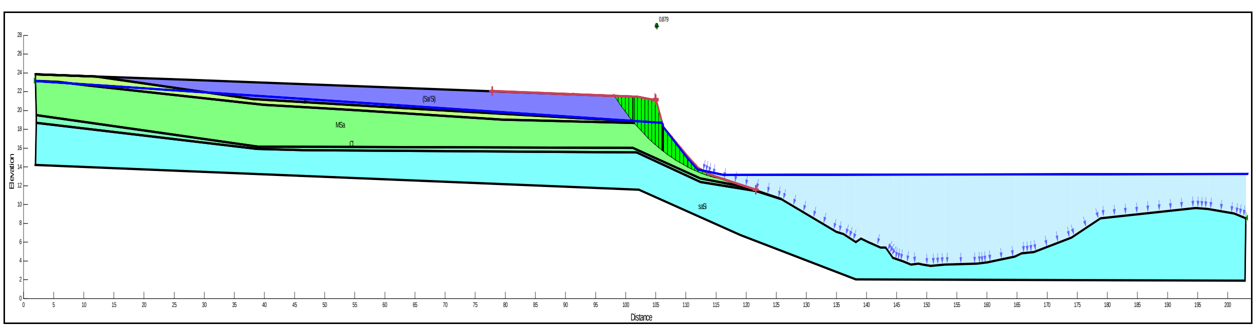
Section: C1 Undrained



Factor of Safety (FoS):

- Morgenstern-Price - 0.994
- Ordinary - 0.853
- Bishop - 0.996
- Janbu - 0.840

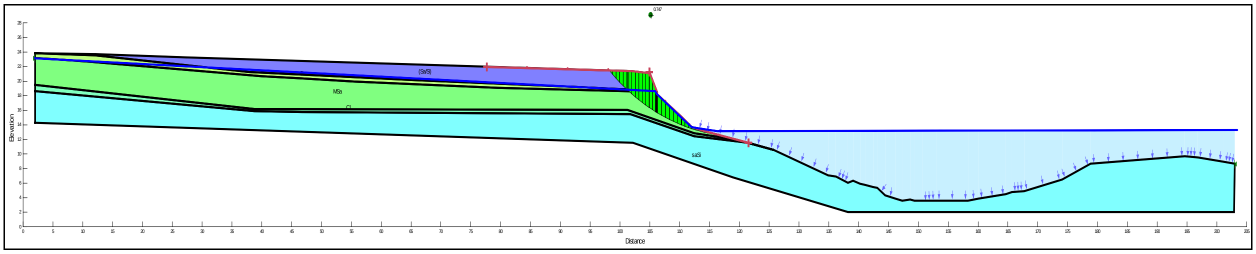
Section: C2 Drained



Factor of Safety (FoS):

- Morgenstern-Price - 0.879
- Ordinary - 0.897
- Bishop - 0.886
- Janbu - 0.899

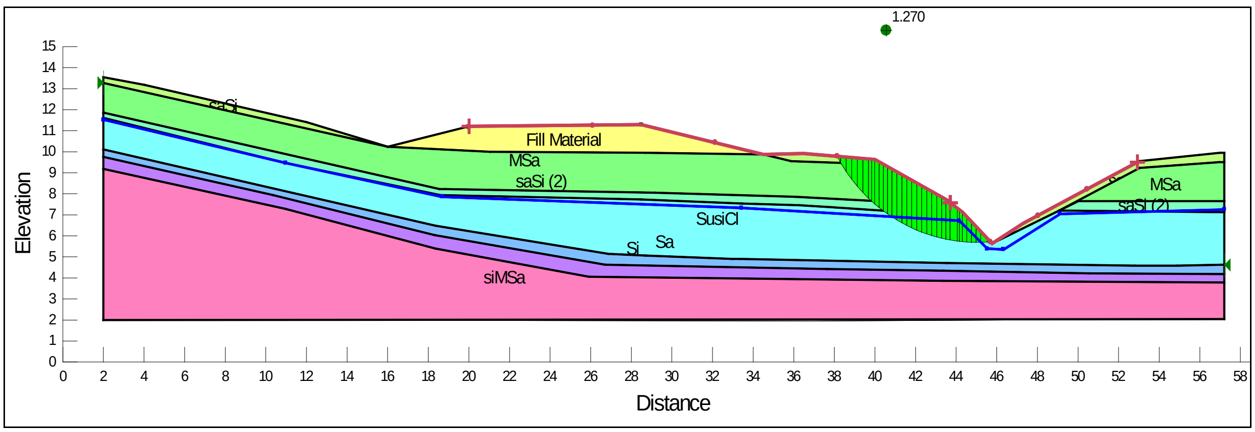
Section: C2 Undrained

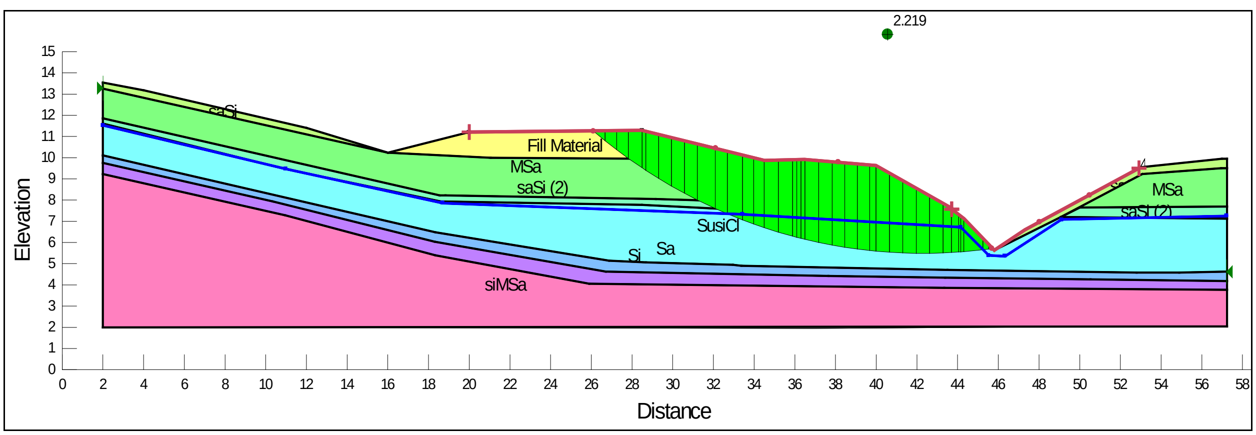


Factor of Safety (FoS):

- Morgenstern-Price - 0.747
- Ordinary - 0.745
- Bishop - 0.751
- Janbu - 0.746

Section: D1 Drained

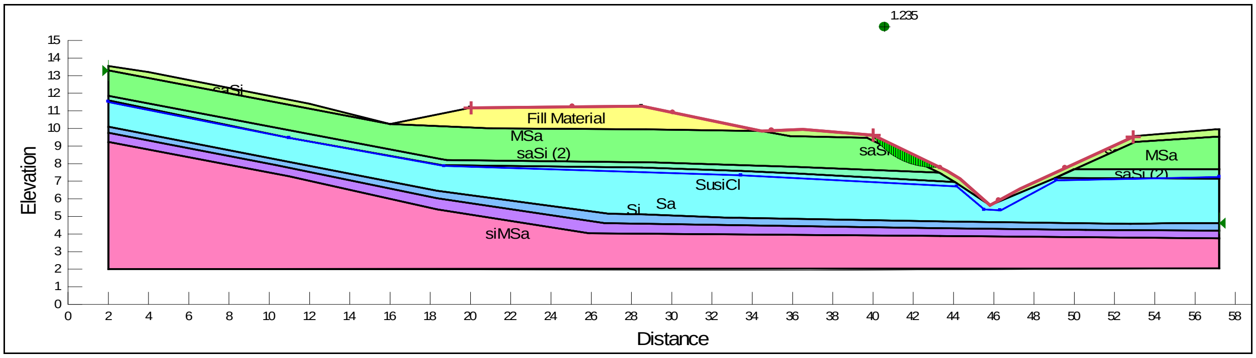


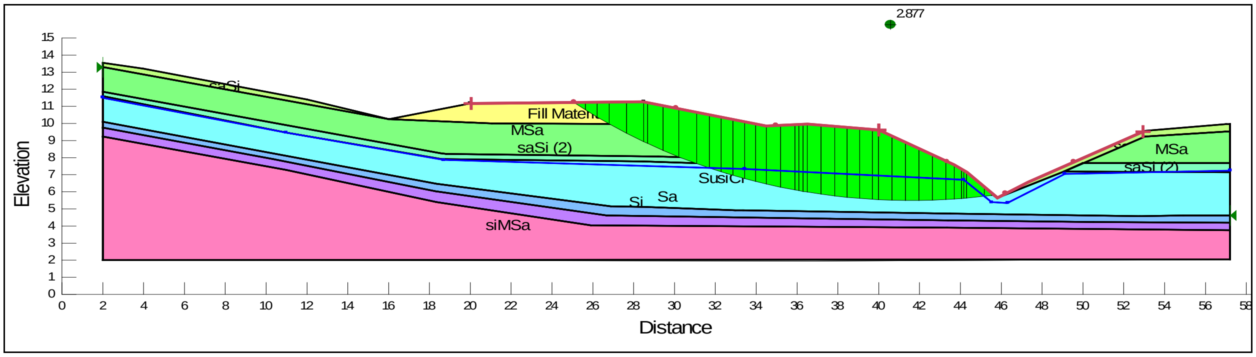


Factor of Safety (FoS):

- Morgenstern-Price - 1.272/2.219
- Ordinary - 1.206/2.156
- Bishop - 1.276/2.223
- Janbu - 1.210/2.161

Section: D1 Undrained

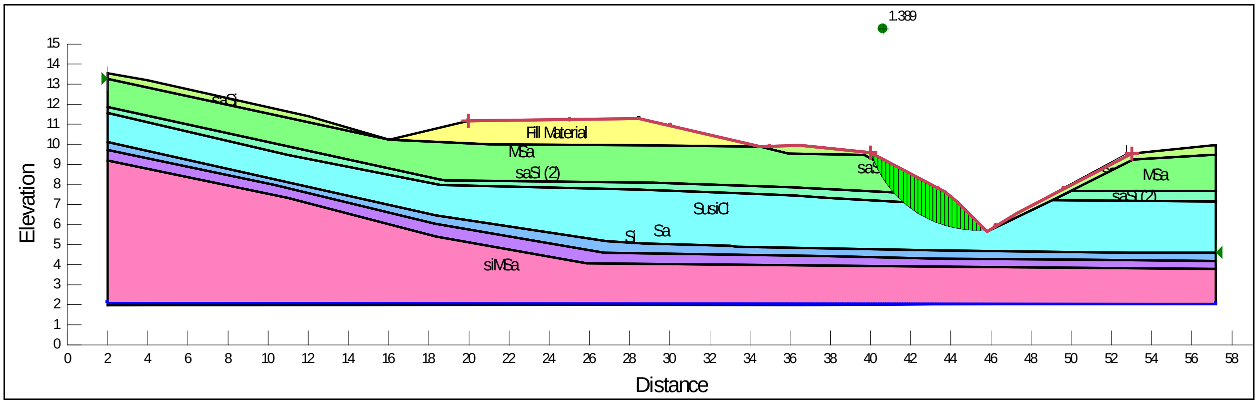


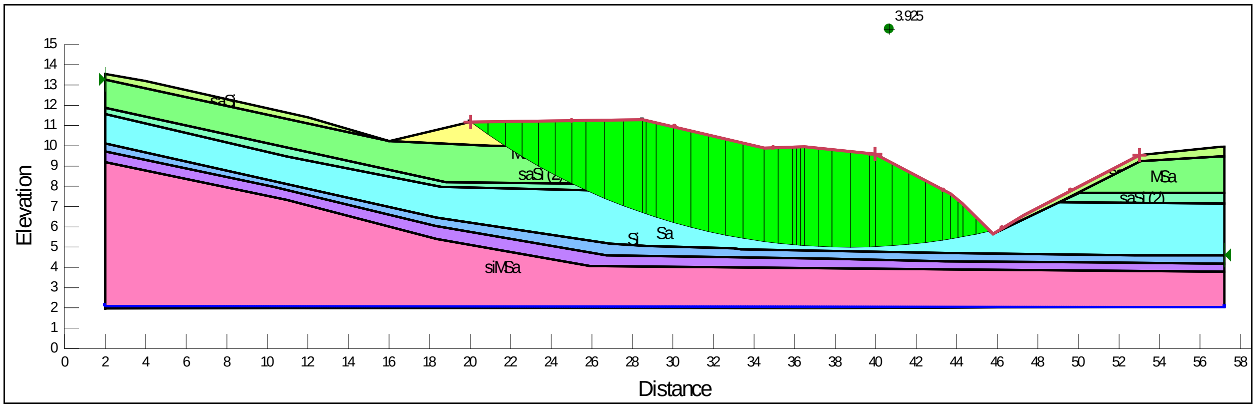


Factor of Safety (FoS):

- Morgenstern-Price - 1.235/2.877
- Ordinary - 1.144/2.843
- Bishop - 1.237/2.900
- Janbu - 1.140/2.808

Section: D2 Drained

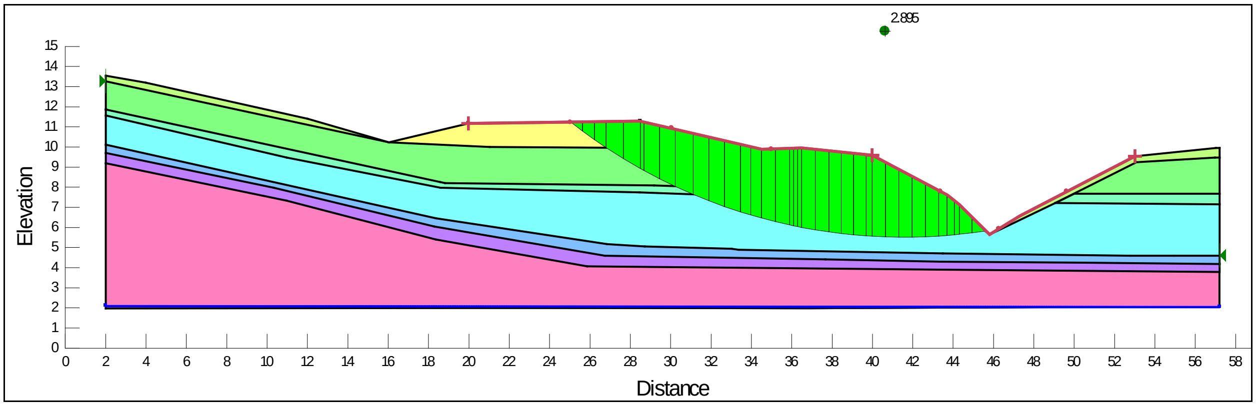




Factor of Safety (FoS):

- Morgenstern-Price - 1.389/3.925
- Ordinary - 1.316/3.810
- Bishop - 1.398/3.927
- Janbu - 1.314/3.797

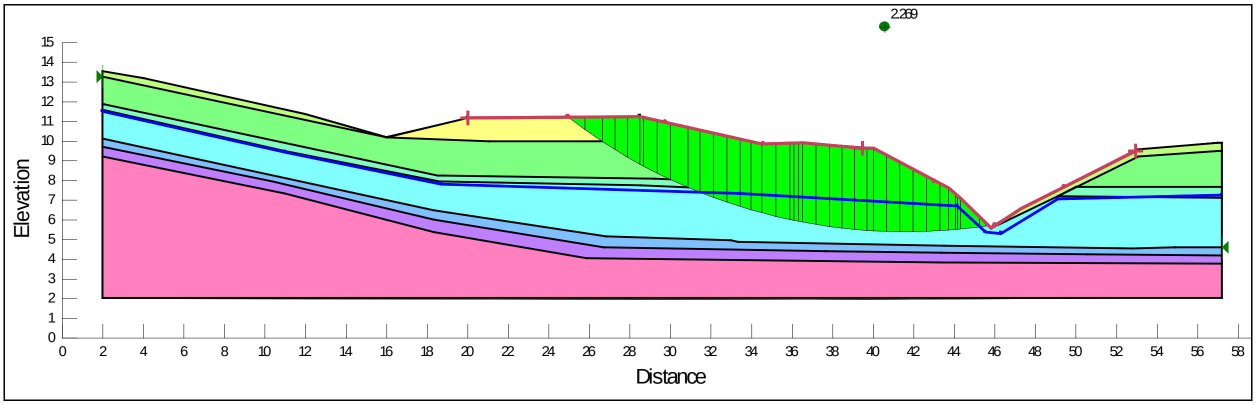
Section: D2 Undrained

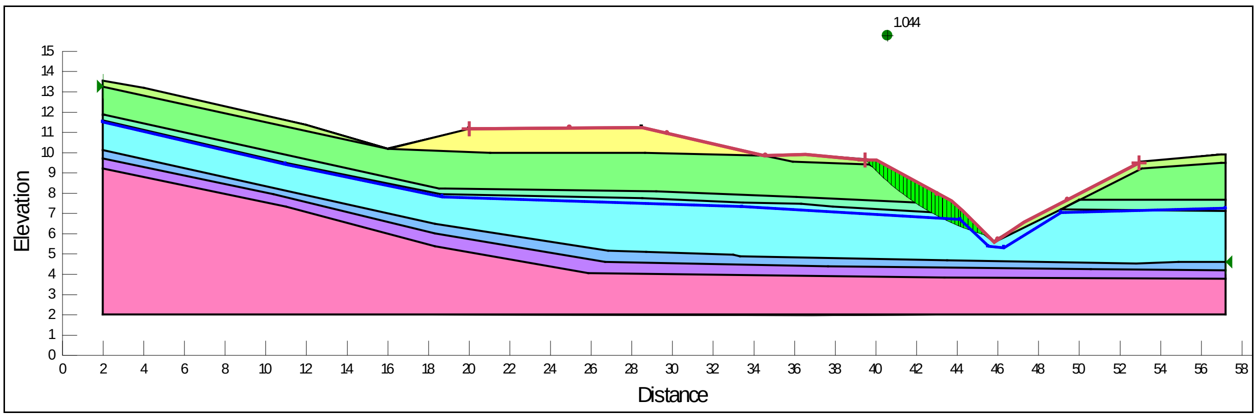


Factor of Safety (FoS):

- Morgenstern-Price - 2.895
- Ordinary - 2.861
- Bishop - 2.918
- Janbu - 2.826

Section: D1B Drained

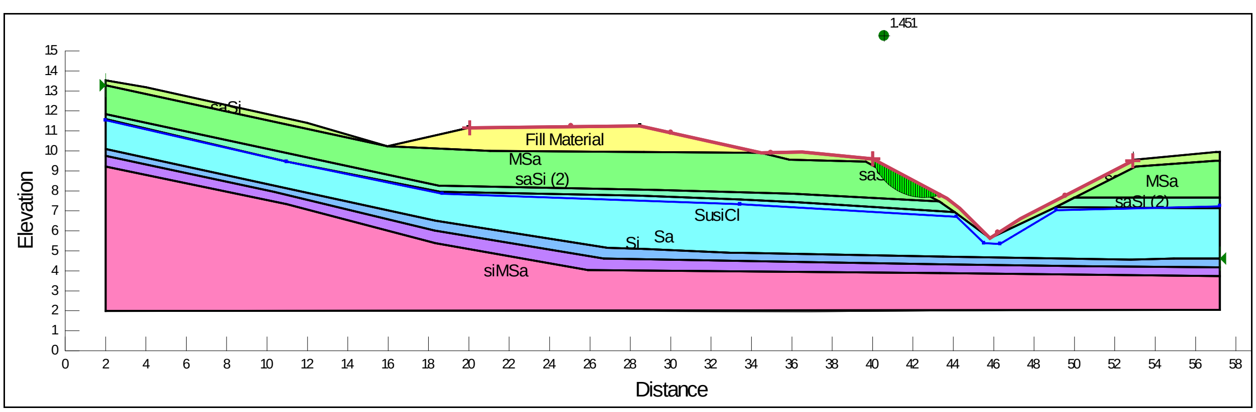


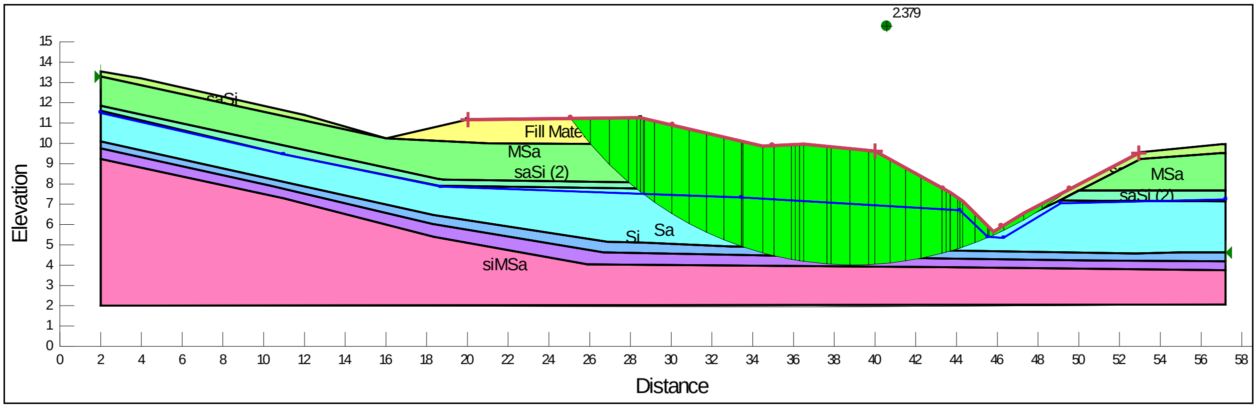


Factor of Safety (FoS):

- Morgenstern-Price - 1.044/2.269
- Ordinary - 1.019/2.202
- Bishop - 1.053/2.272
- Janbu - 1.022/ 2.212

Section: D1B Undrained

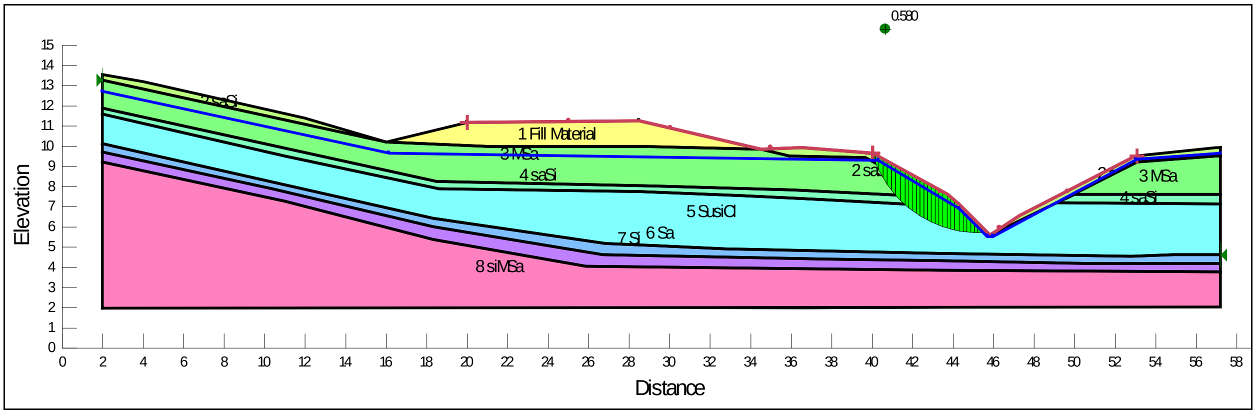


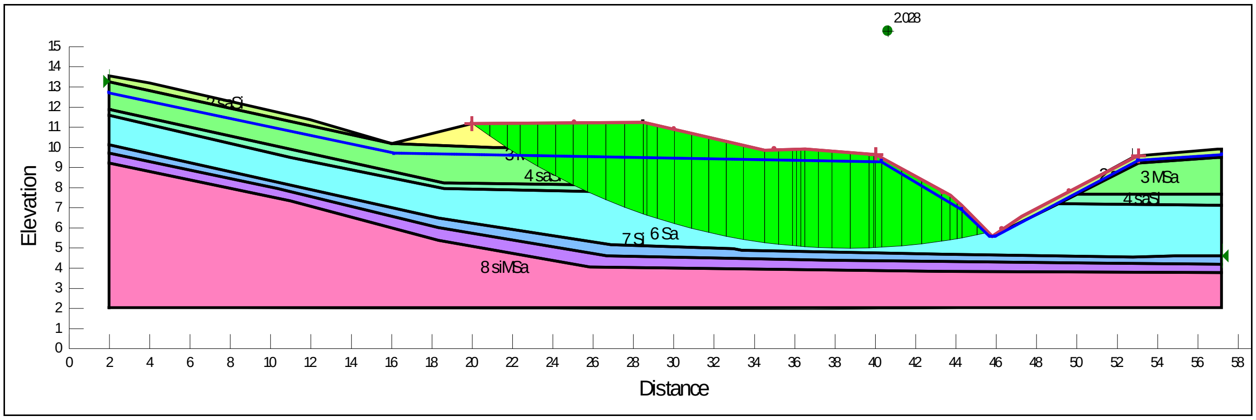


Factor of Safety (FoS):

- Morgenstern-Price - 1.451/2.379
- Ordinary - 1.262/2.292
- Bishop - 1.455/2.374
- Janbu - 1.249/ 2.208

Section: D2B Drained

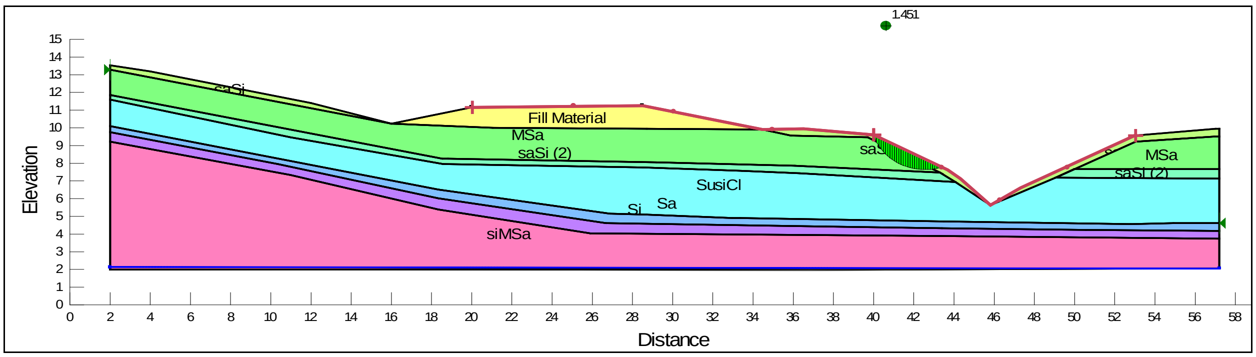


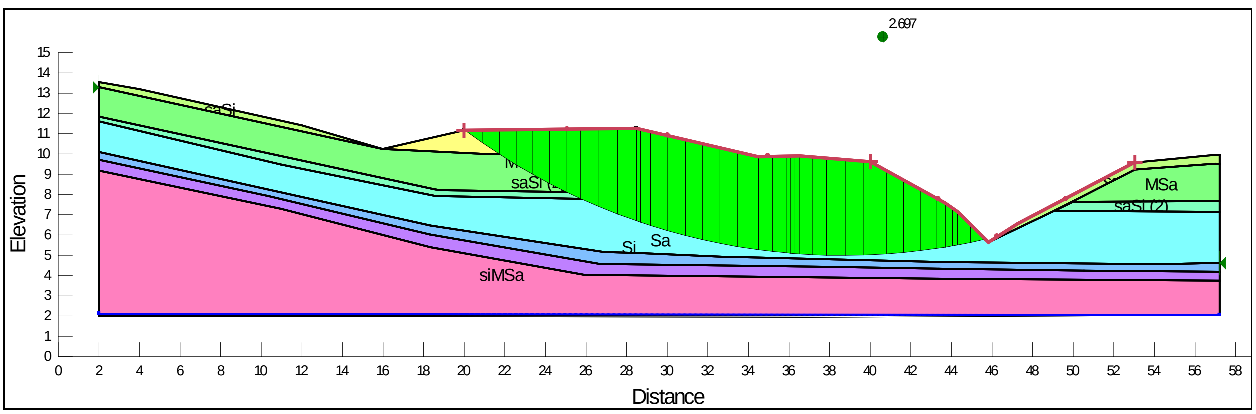


Factor of Safety (FoS):

- Morgenstern-Price - 0.580/2.028
- Ordinary - 0.403/1.920
- Bishop - 0.563/2.028
- Janbu - 0.474/ 1.966

Section: D2B Undrained

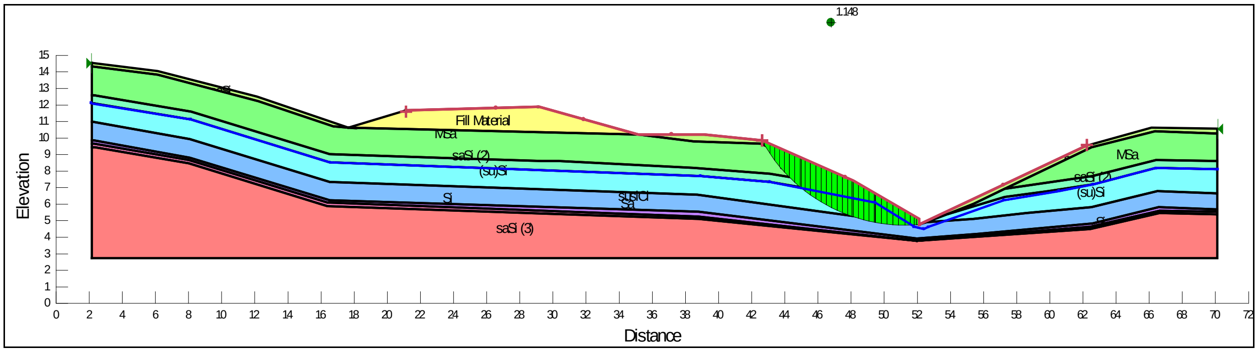


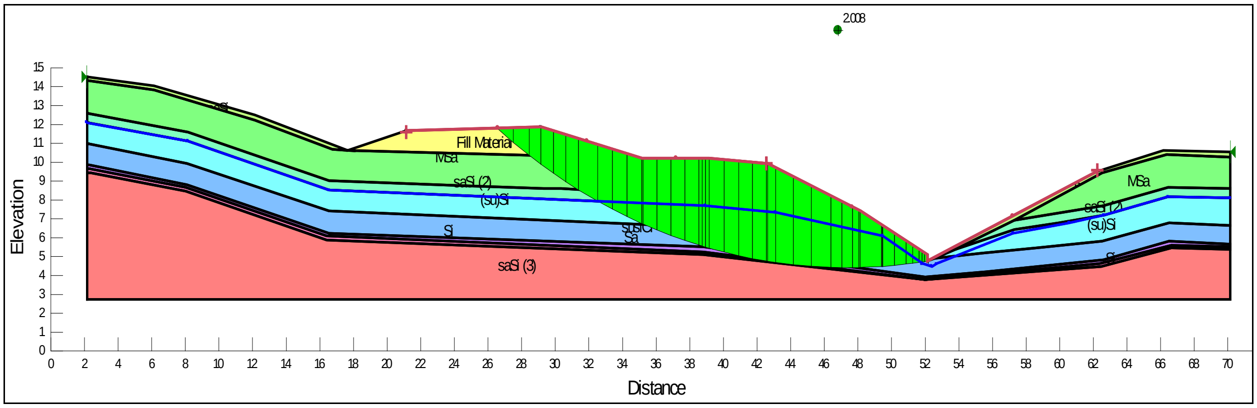


Factor of Safety (FoS):

- Morgenstern-Price - 1.451/2.697
- Ordinary - 1.262/2.671
- Bishop - 1.455/2.703
- Janbu - 1.249/ 2.626

Section: E1B Drained Condition

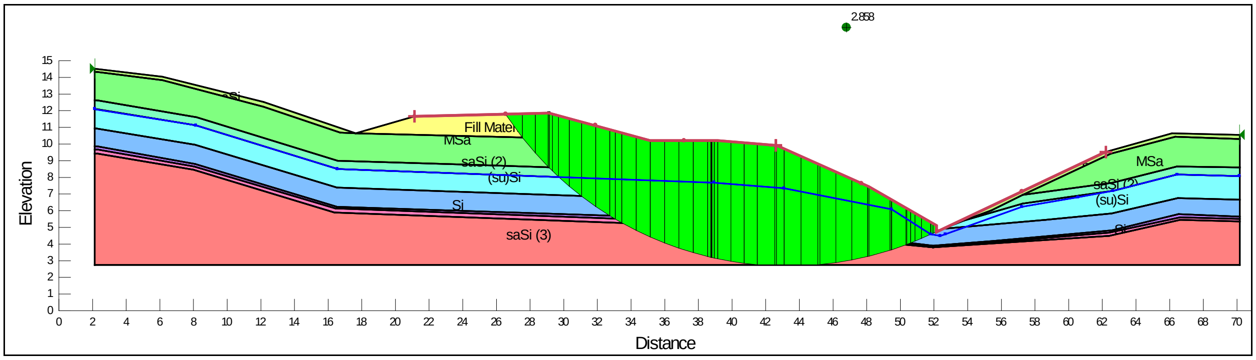




Factor of Safety (FoS):

- Morgenstern-Price - 1.148/2.008
- Ordinary - 1.059/1.937
- Bishop - 1.156/2.007
- Janbu - 1.070/ 1.946

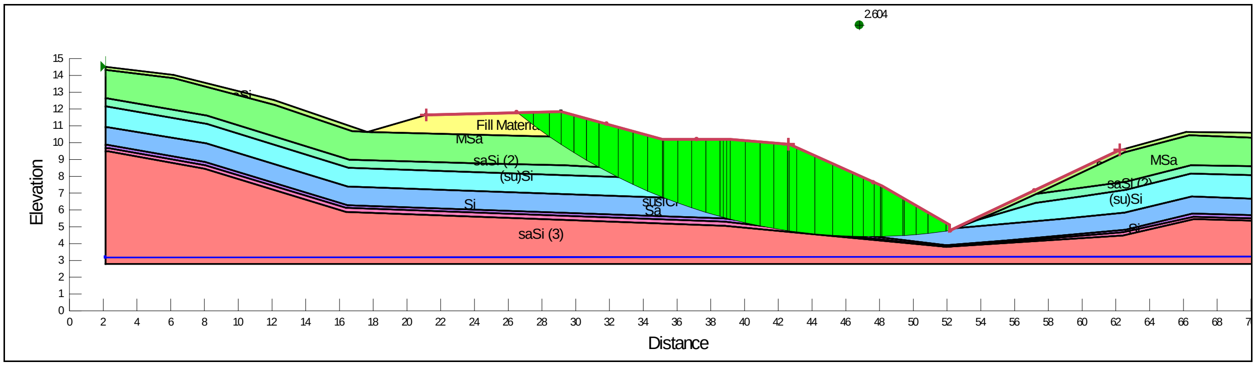
Section: E1B Undrained Condition



Factor of Safety (FoS):

- Morgenstern-Price - 2.858
- Ordinary - 2.606
- Bishop - 2.807
- Janbu - 2.539

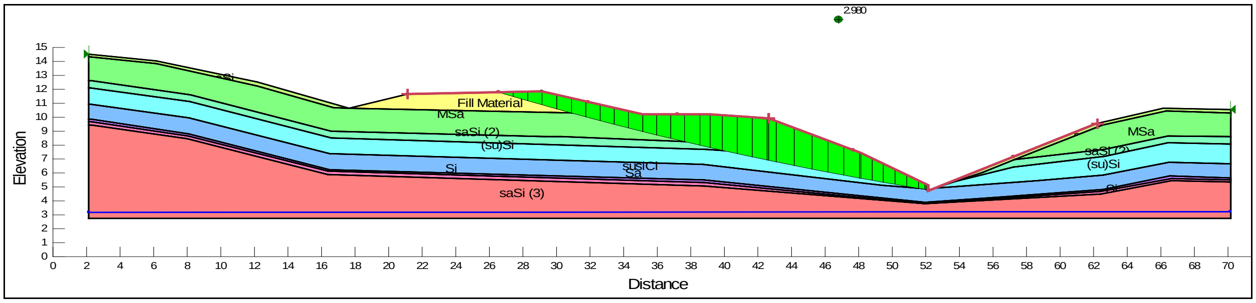
Section: E2B Drained Condition



Factor of Safety (FoS):

- Morgenstern-Price - 2.604
- Ordinary - 2.522
- Bishop - 2.604
- Janbu - 2.510

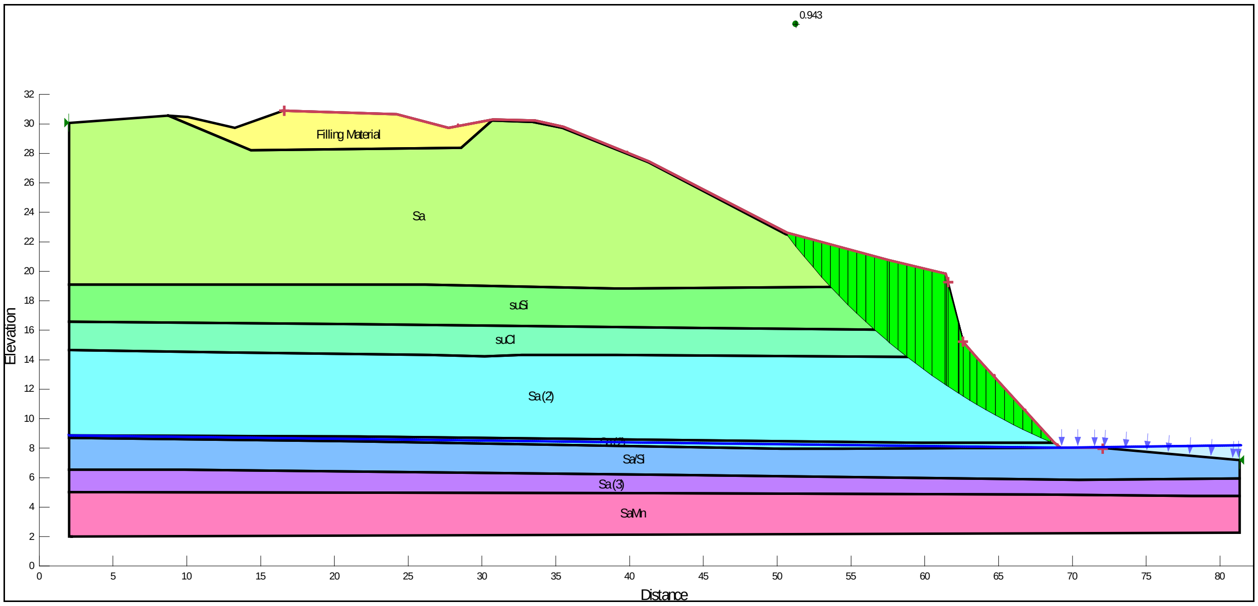
Section: E2B Undrained Condition

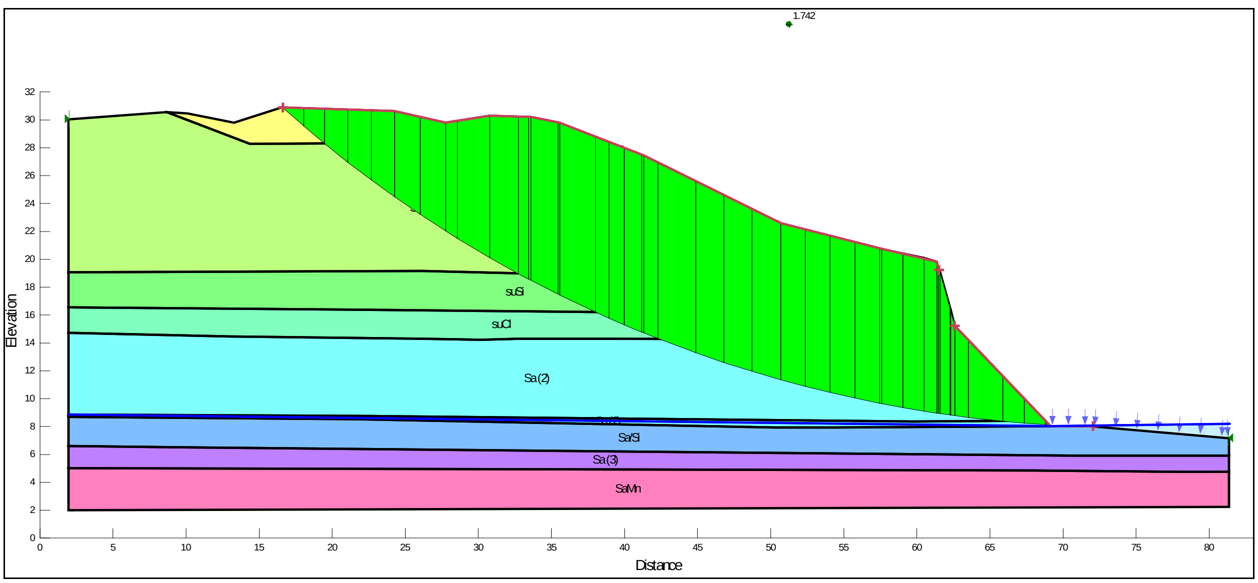


Factor of Safety (FoS):

- Morgenstern-Price - 2.980
- Ordinary - 2.969
- Bishop - 2.996
- Janbu - 2.984

Section: F1 Drained Condition

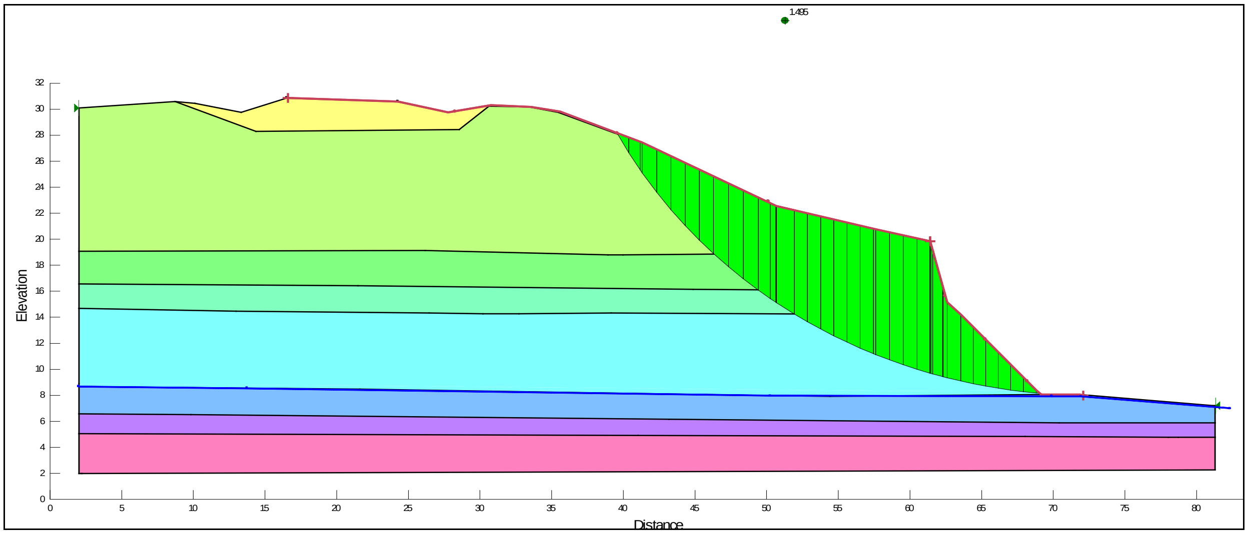


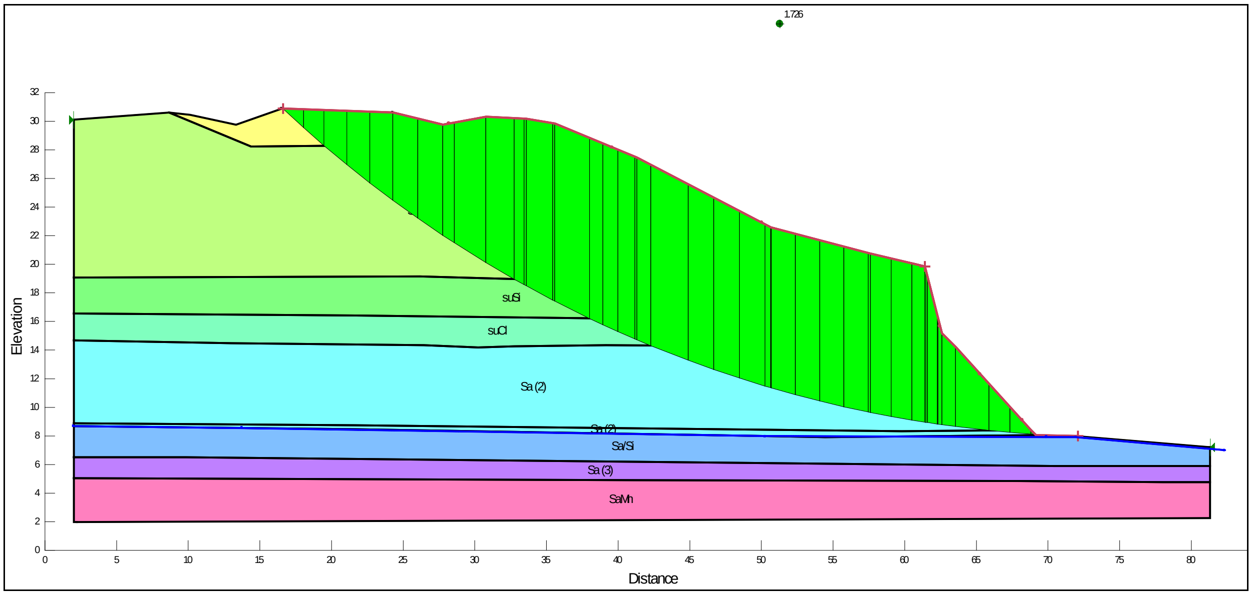


Factor of Safety (FoS):

- Morgenstern-Price - 0.943/1.742
- Ordinary - 0.930/1.696
- Bishop - 0.945/1.737
- Janbu - 0.927/1.692

Section: F1Undrained Condition

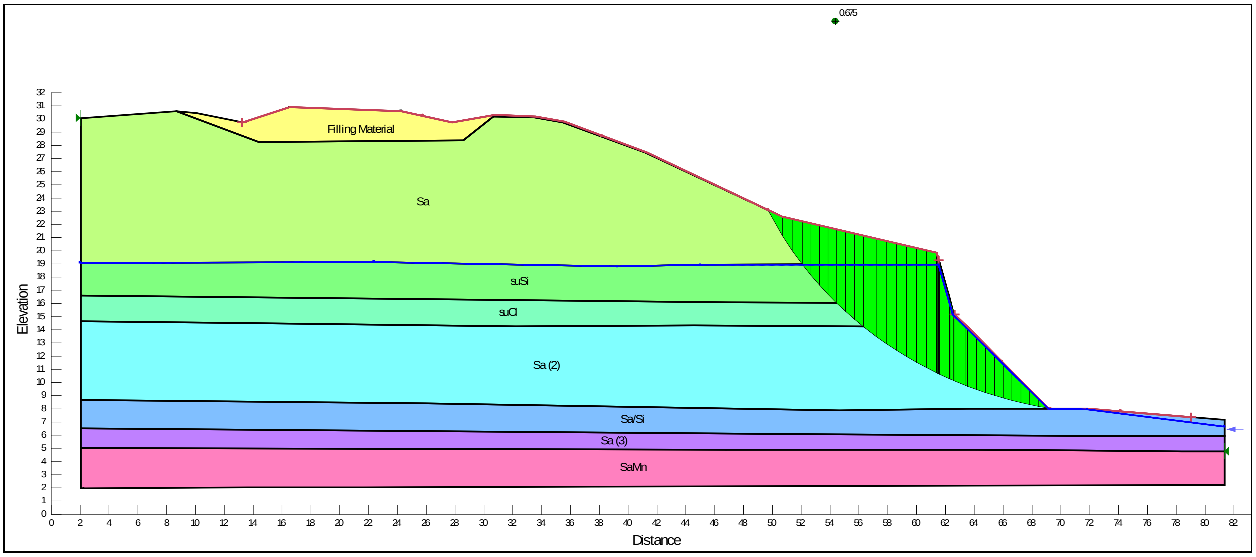


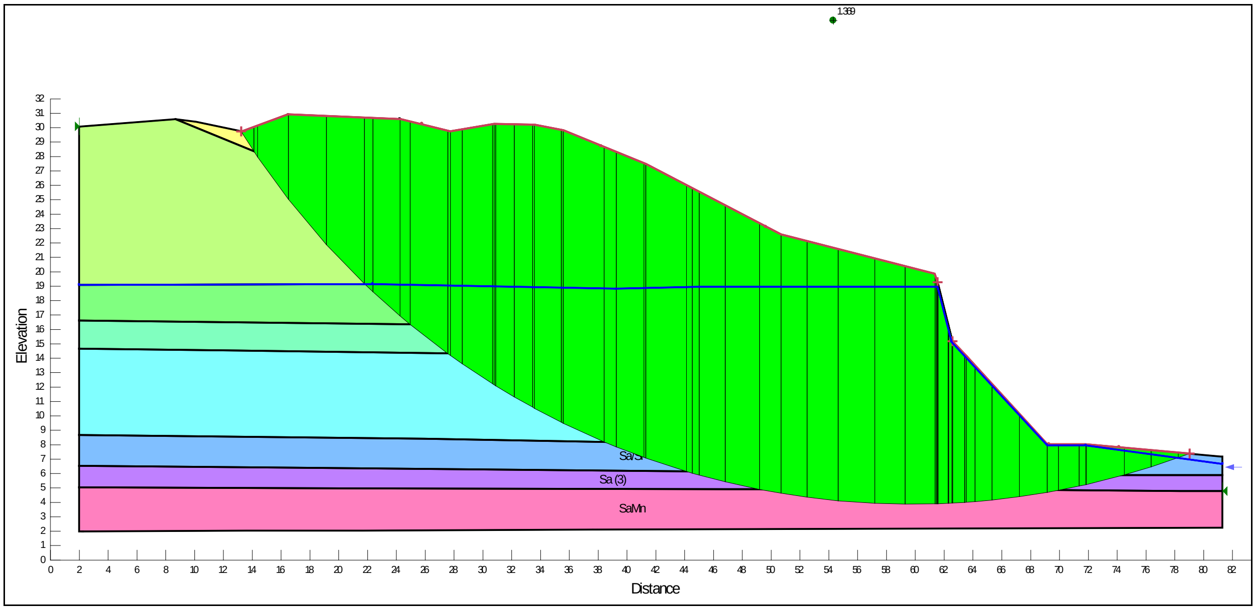


Factor of Safety (FoS):

- Morgenstern-Price - 1.495/1.726
- Ordinary - 1.430/1.677
- Bishop - 1.507/1.692
- Janbu -1.445/1.649

Section: F2 Drained Condition

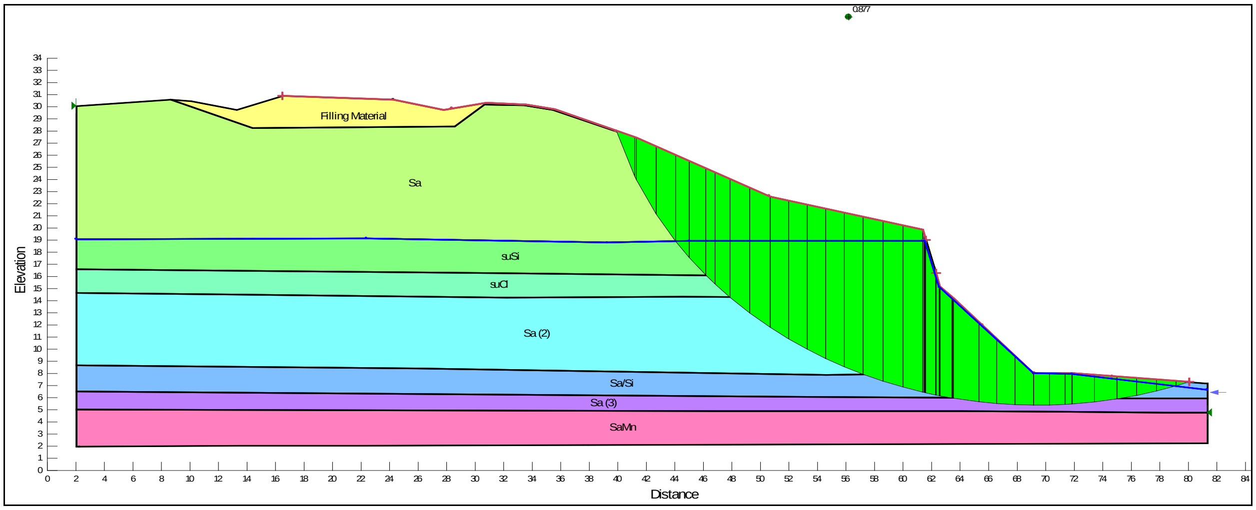


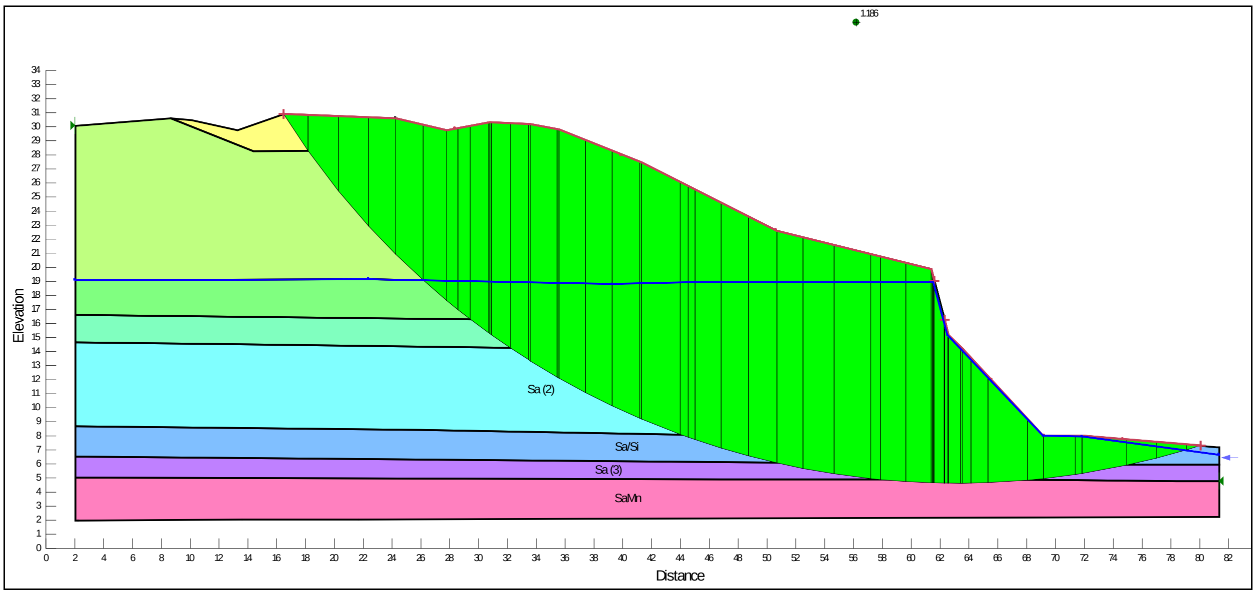


Factor of Safety (FoS):

- Morgenstern-Price - 0.675/1.369
- Ordinary - 0.676/1.248
- Bishop - 0.685/1.364
- Janbu - 0.695/1.285

Section: F2 Undrained Condition

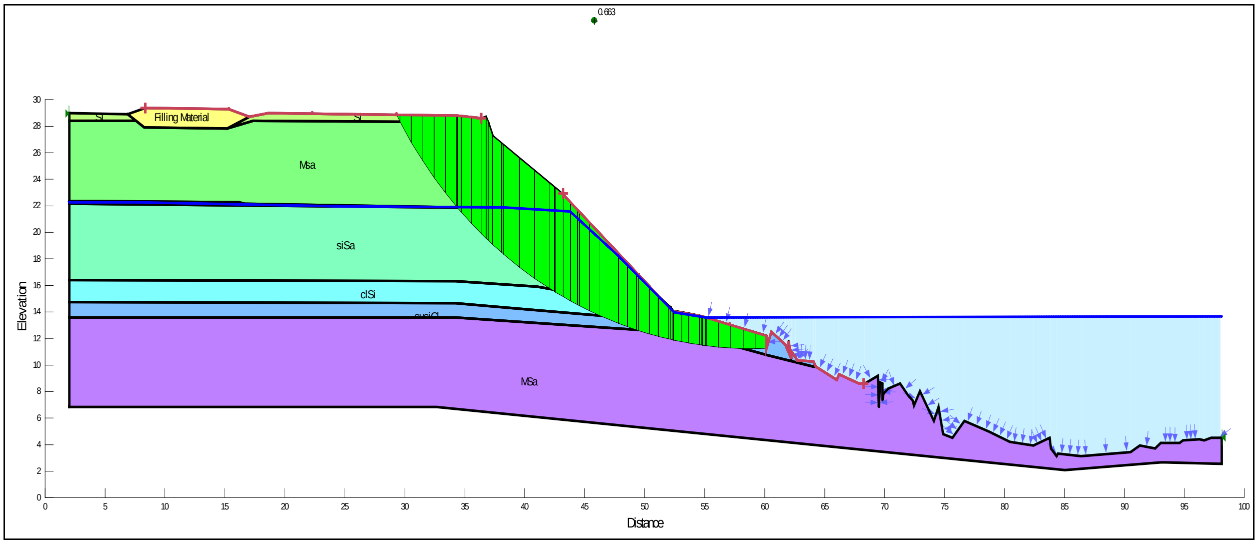


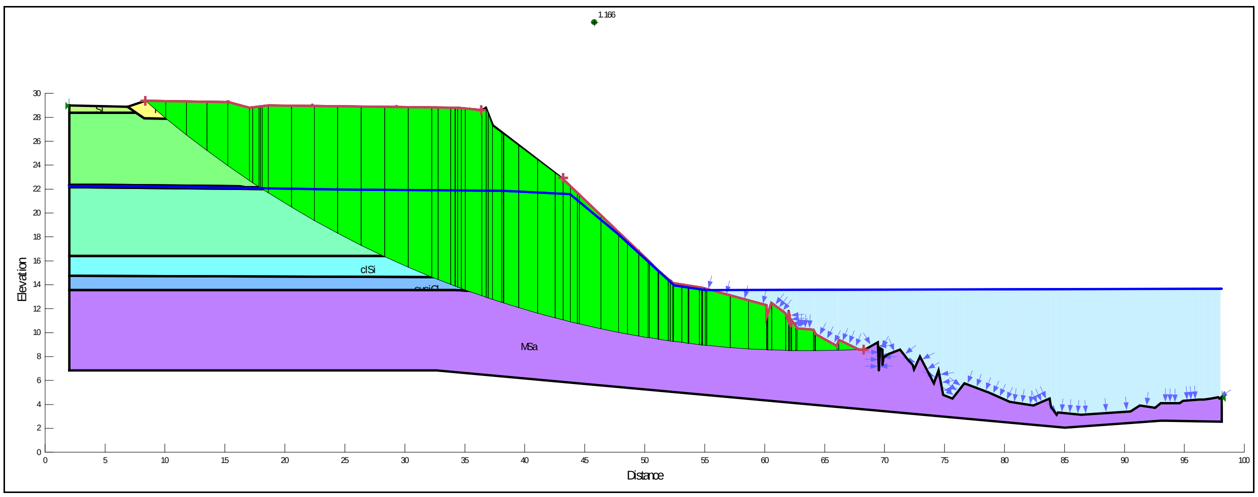


Factor of Safety (FoS):

- Morgenstern-Price - 0.877/1.186
- Ordinary - 0.759/1.109
- Bishop - 0.886/1.169
- Janbu - 0.883/1.117

Section: N1 Drained Condition

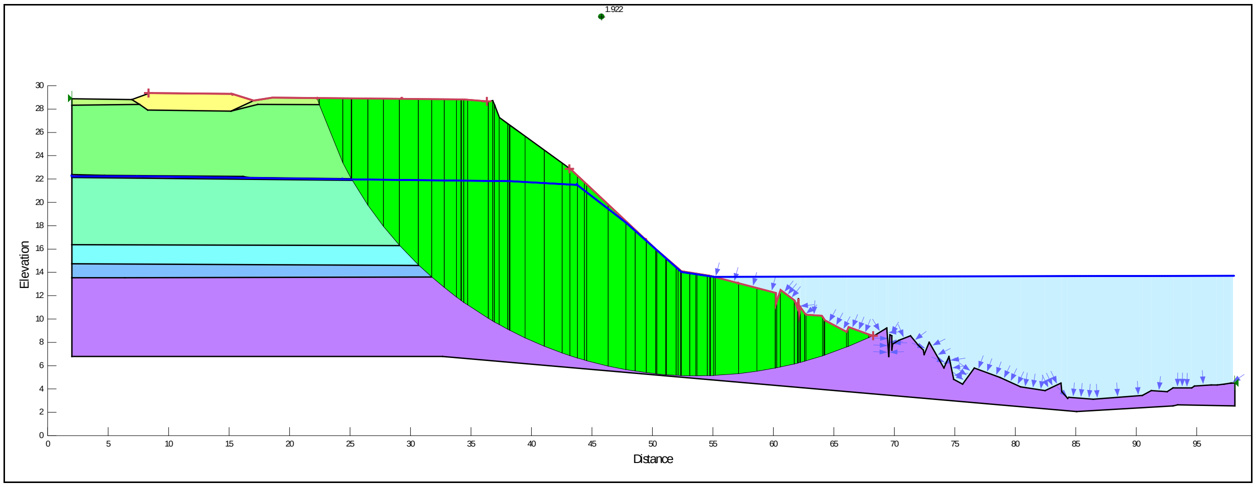


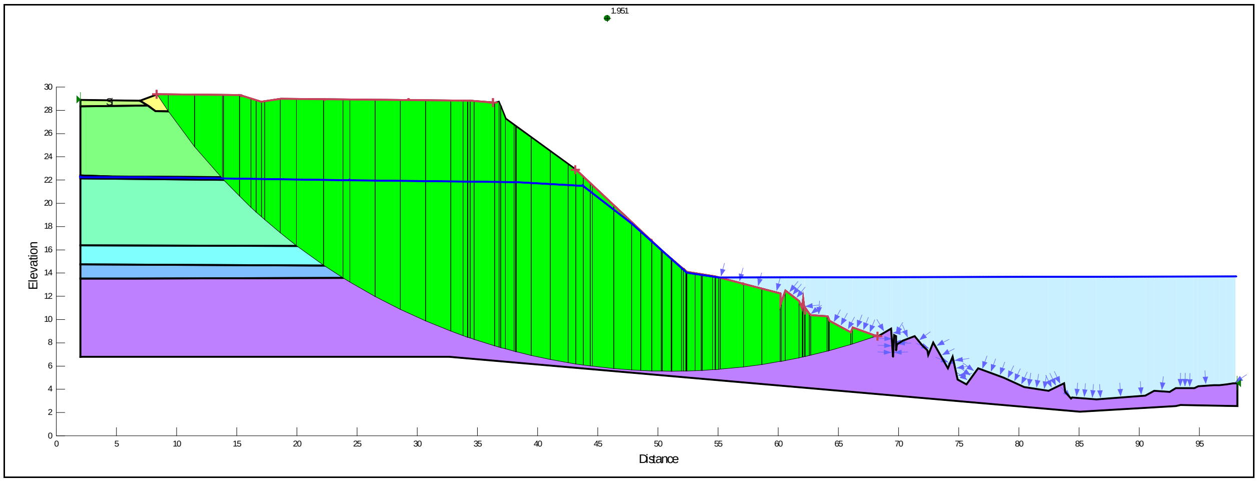


Factor of Safety (FoS):

- Morgenstern-Price - 0.663/1.166
- Ordinary - 0.597/1.128
- Bishop - 0.652/1.163
- Janbu - 0.617/1.141

Section: N1 Undrained Condition

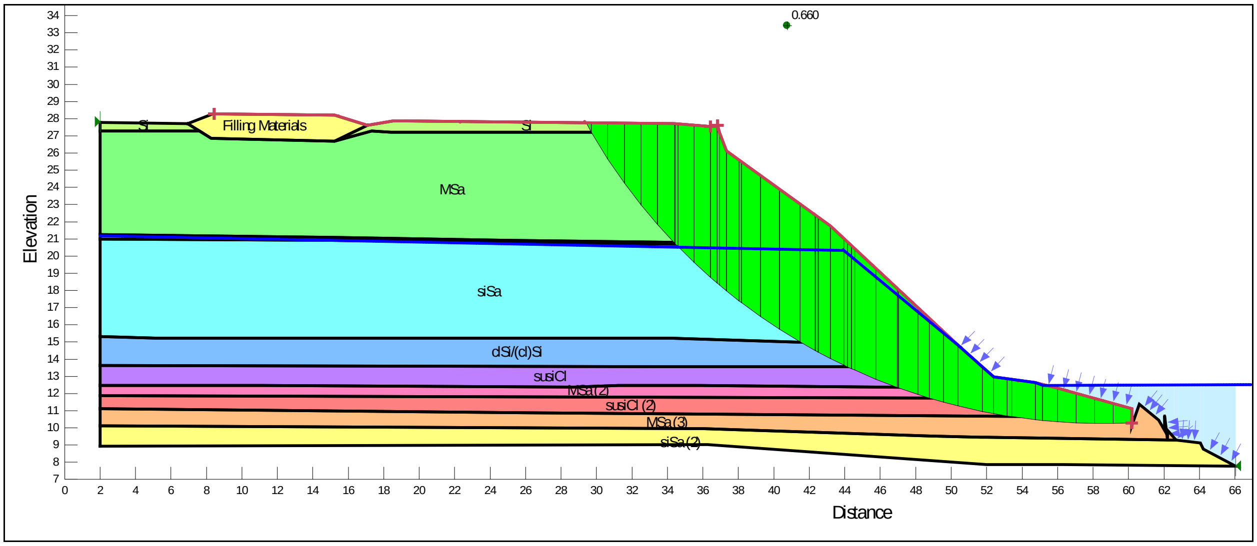


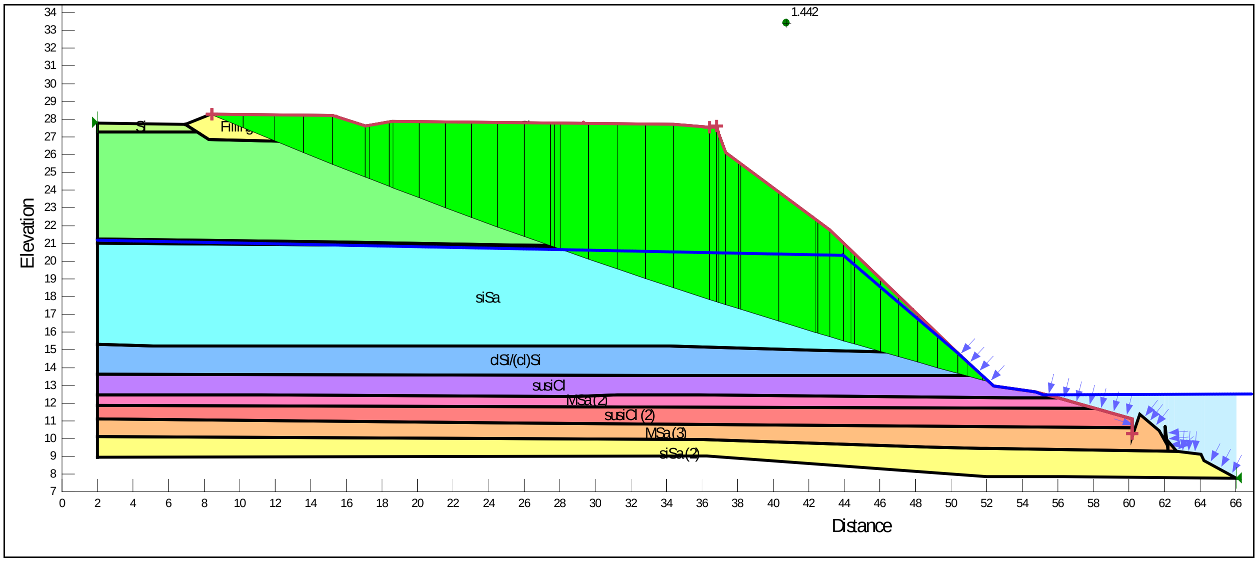


Factor of Safety (FoS):

- Morgenstern-Price - 1.922/1.951
- Ordinary - 1.922/1.951
- Bishop - 1.922/1.951
- Janbu - 1.912/1.921

Section: N1b Drained Condition

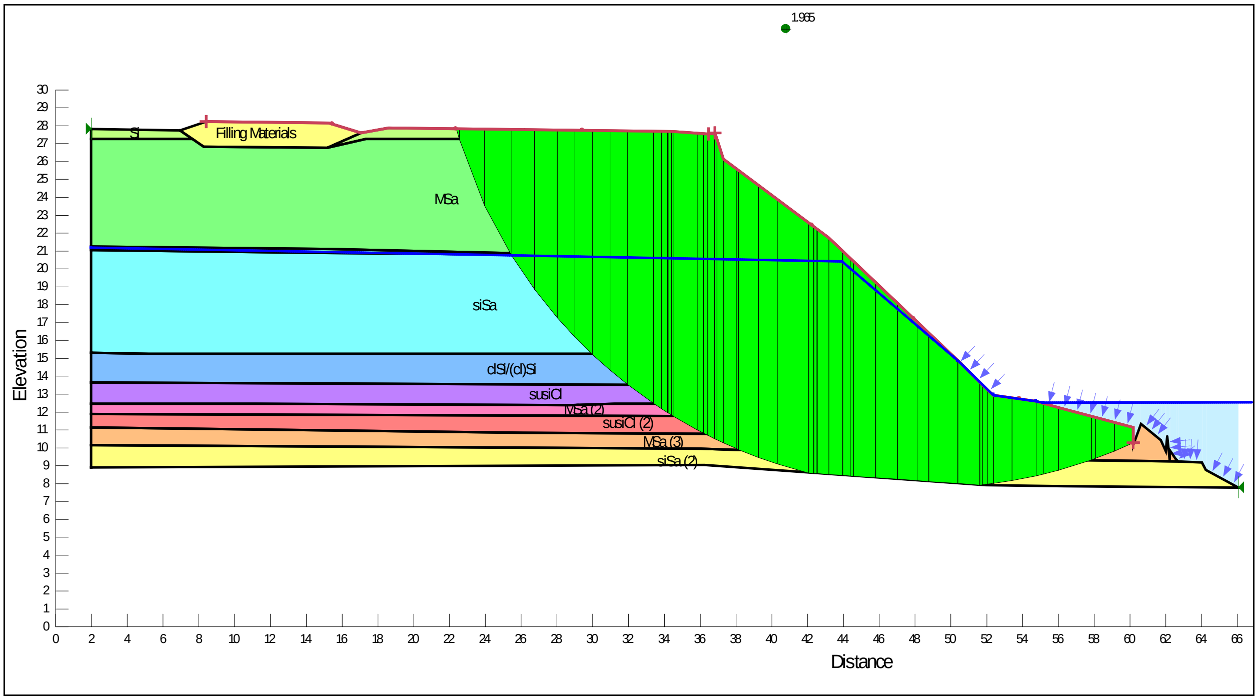


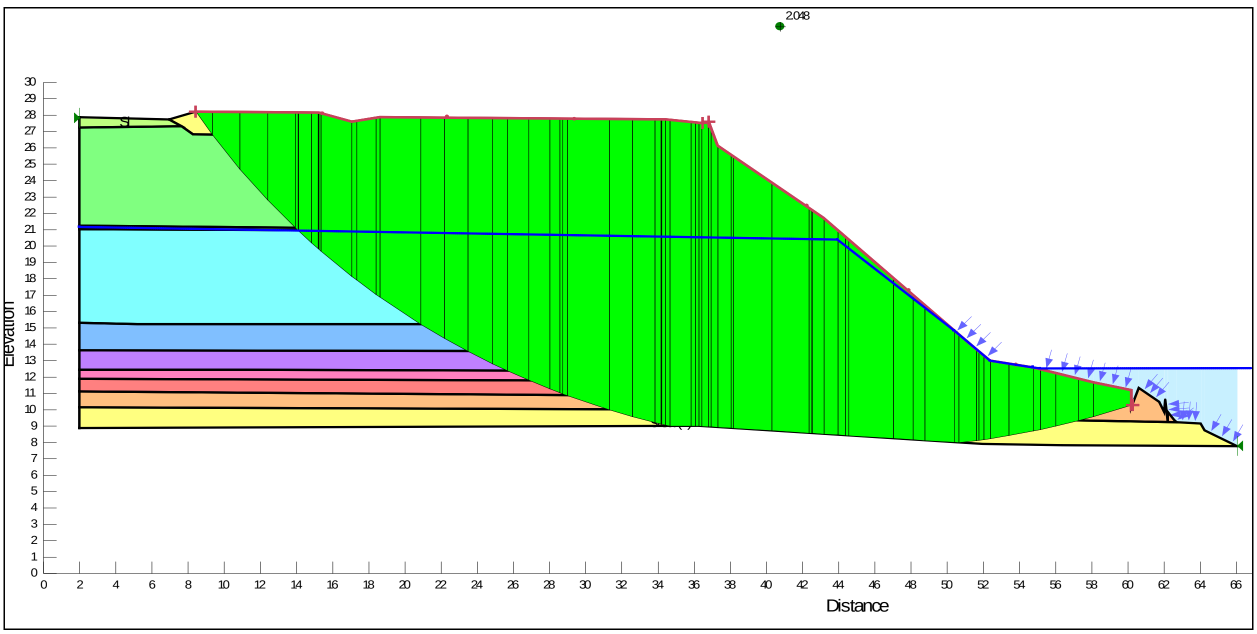


Factor of Safety (FoS):

- Morgenstern-Price - 0.660/1.442
- Ordinary - 0.597/1.402
- Bishop - 0.652/1.442
- Janbu - 0.616/1.409

Section: N1b Undrained Condition

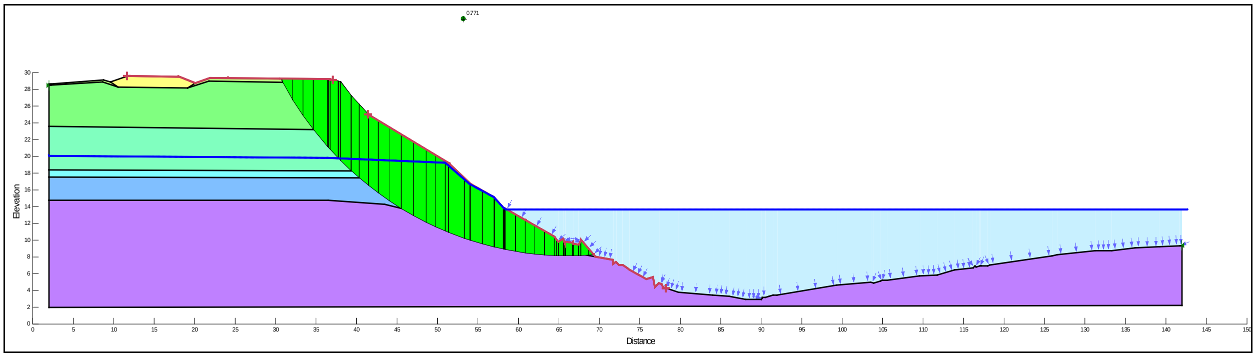


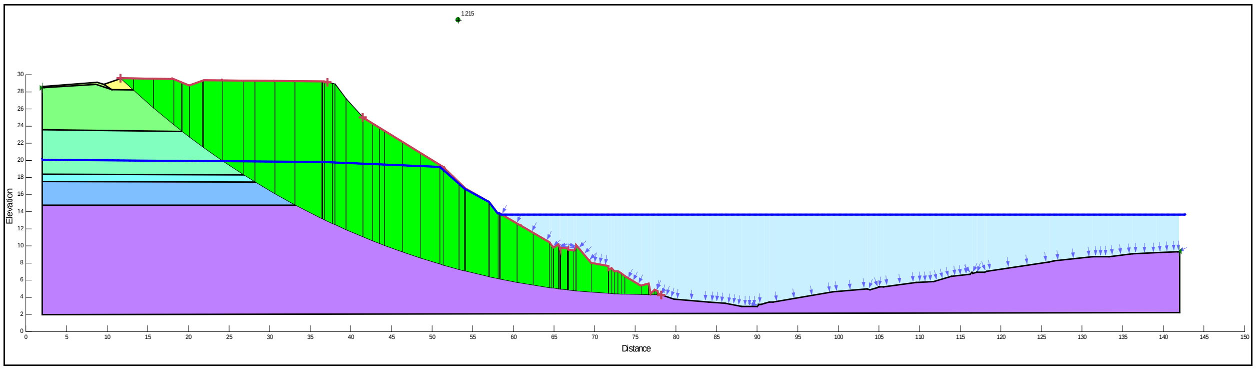


Factor of Safety (FoS):

- Morgenstern-Price - 1.965/2.048
- Ordinary - 1.963/2.047
- Bishop - 1.963/2.048
- Janbu - 2.016/2.000

Section: N2 Drained Condition

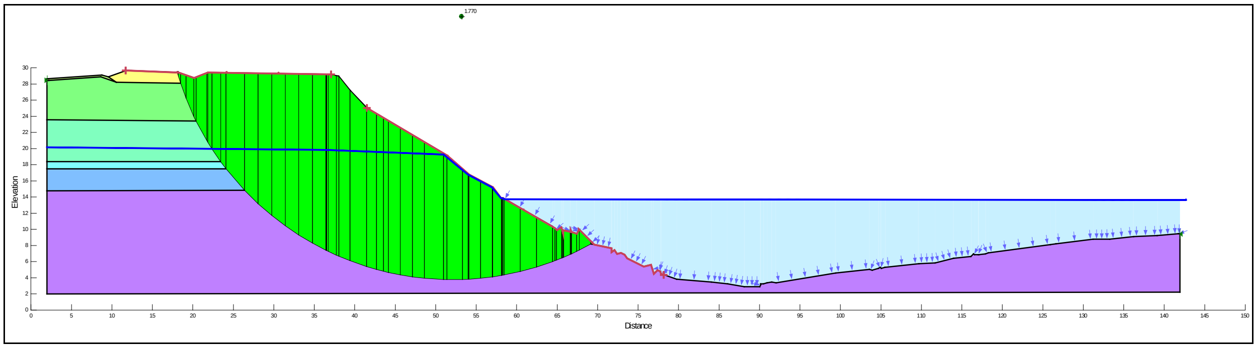


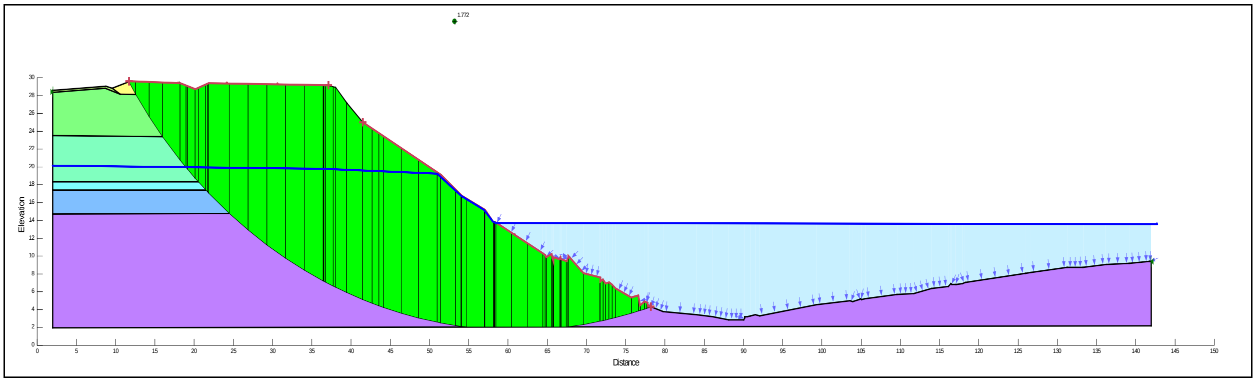


Factor of Safety (FoS):

- Morgenstern-Price - 0.771/1.215
- Ordinary - 0.671/1.162
- Bishop - 0.747/1.209
- Janbu - 0.702/1.182

Section: N2 Undrained Condition





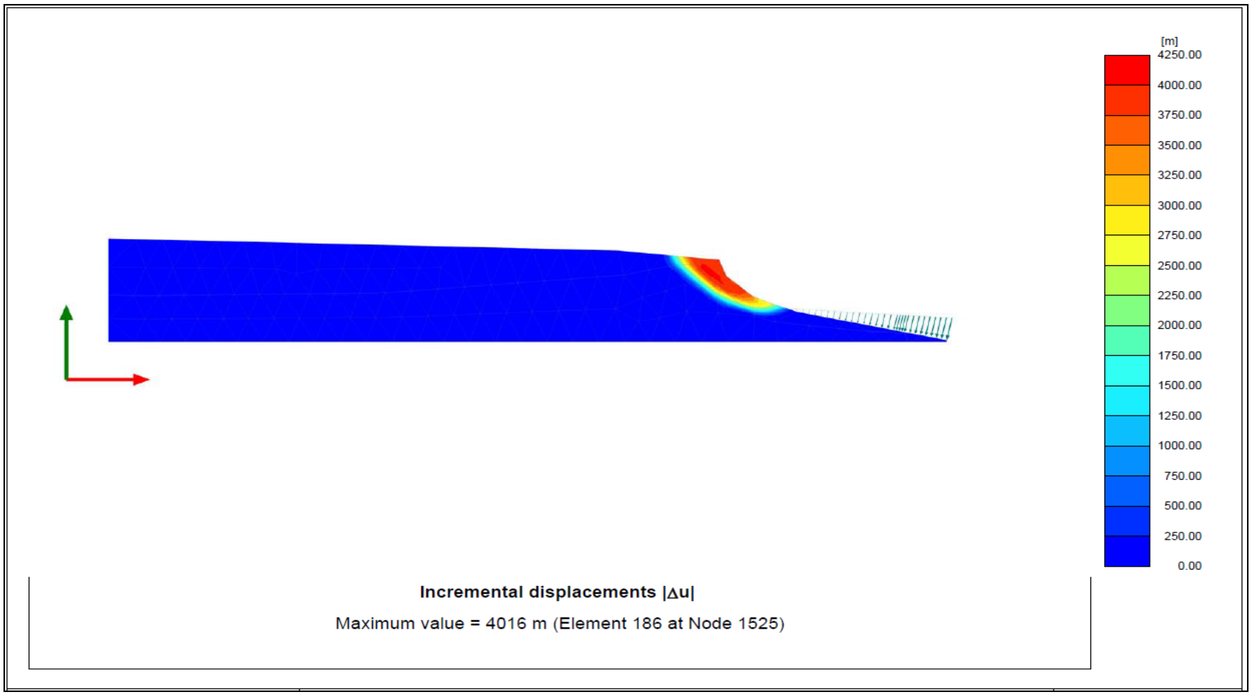
Factor of Safety (FoS):

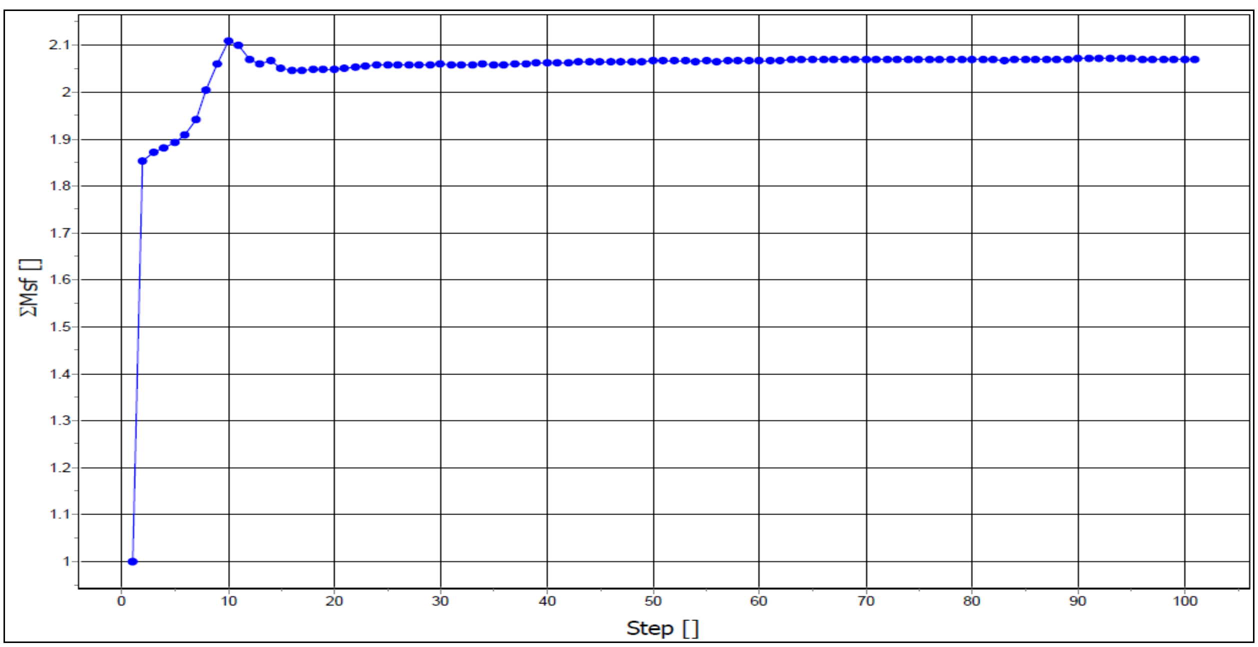
- Morgenstern-Price - 1.770/1.772
- Ordinary - 1.770/1.772
- Bishop - 1.770/1.772
- Janbu - 1.774/1.720

Appendix B

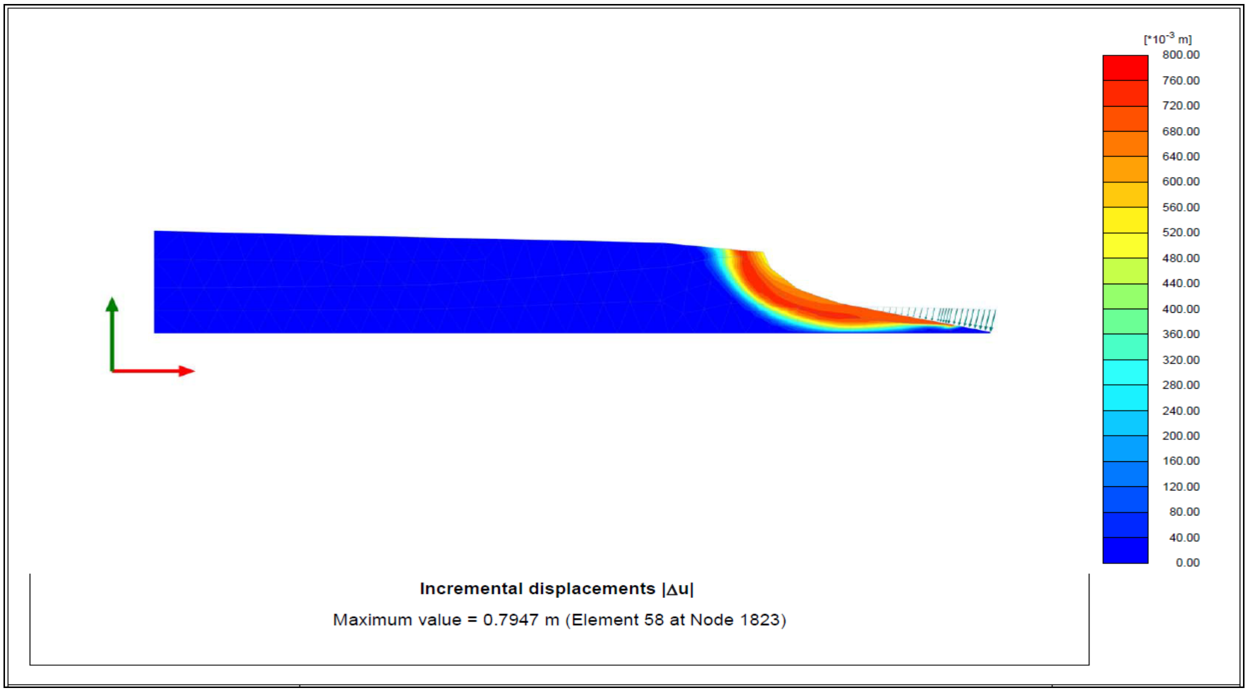
Result from analyses with from Plaxis 2D are presented in Appendix B for drained and undrained conditions. The total incremental displacements that illustrate the position of the slip surface and the associated safety factor are given separately.

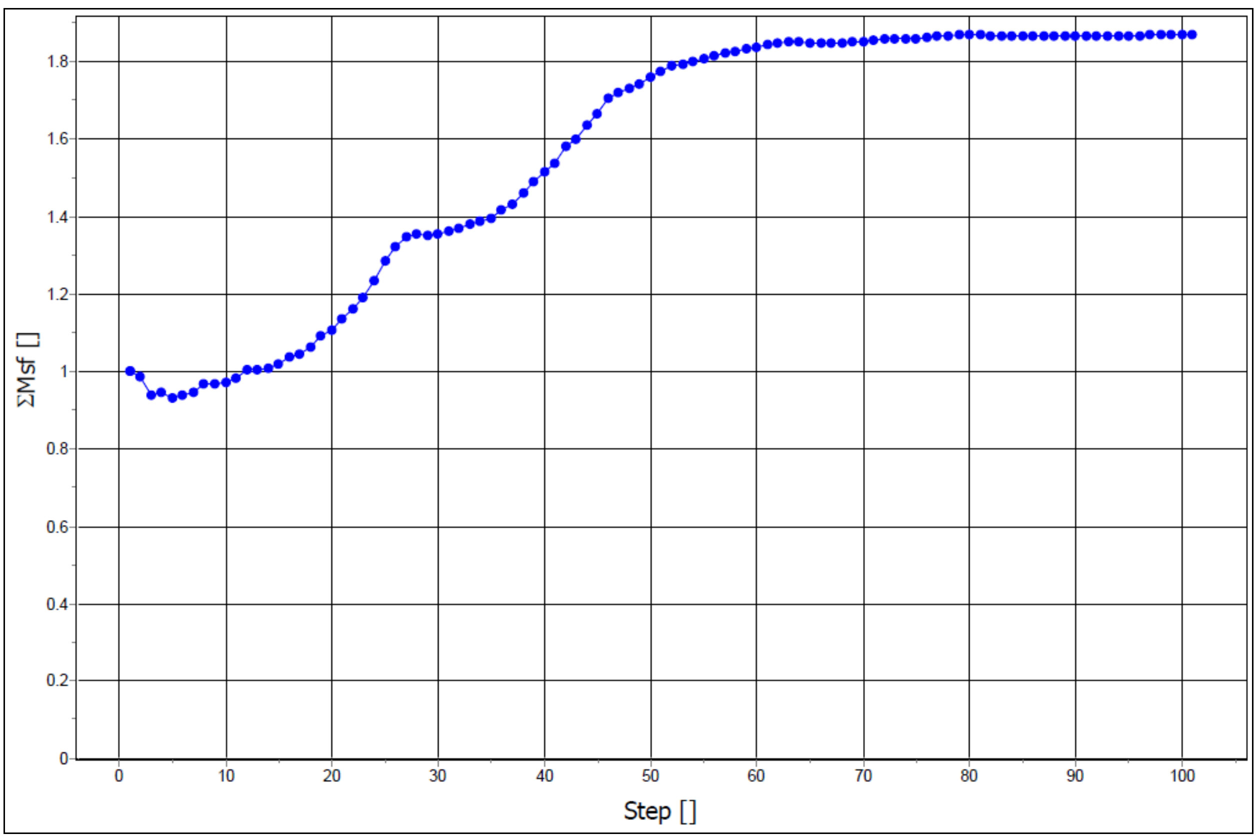
Section B1 Drained



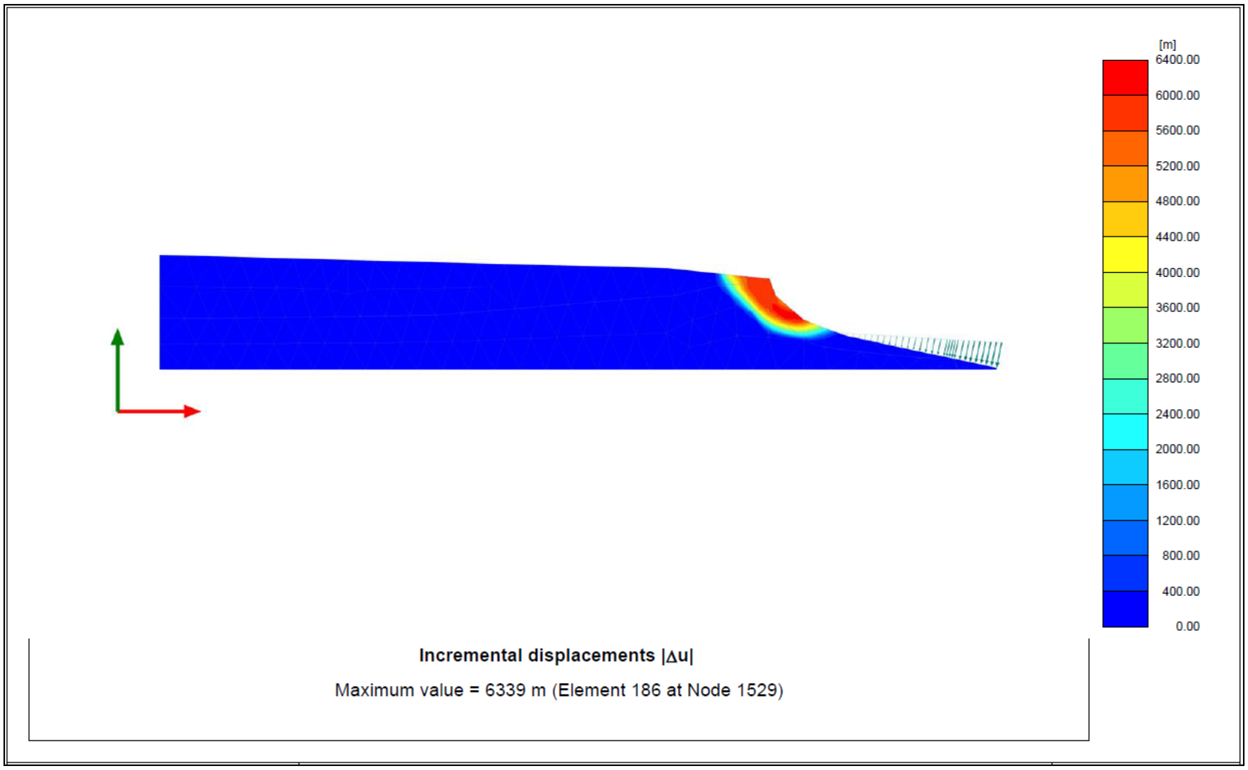


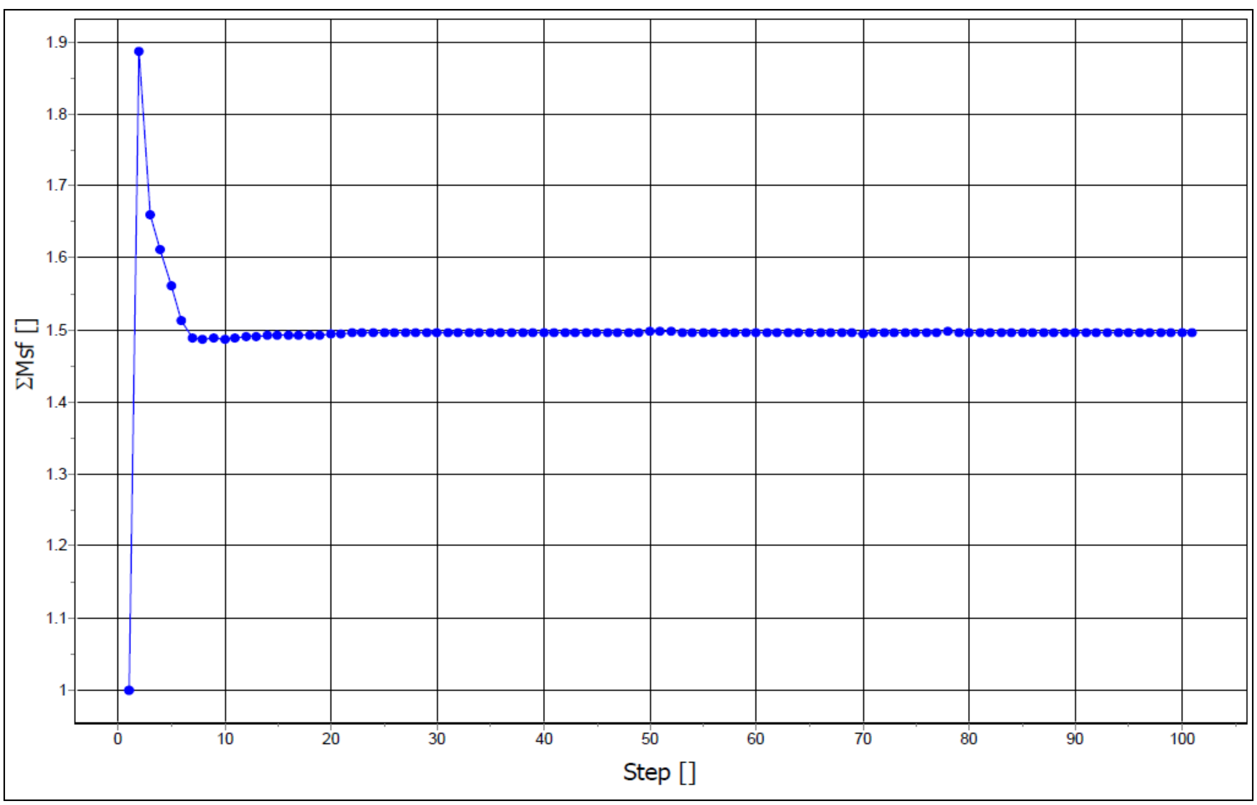
Section B1 Undrained



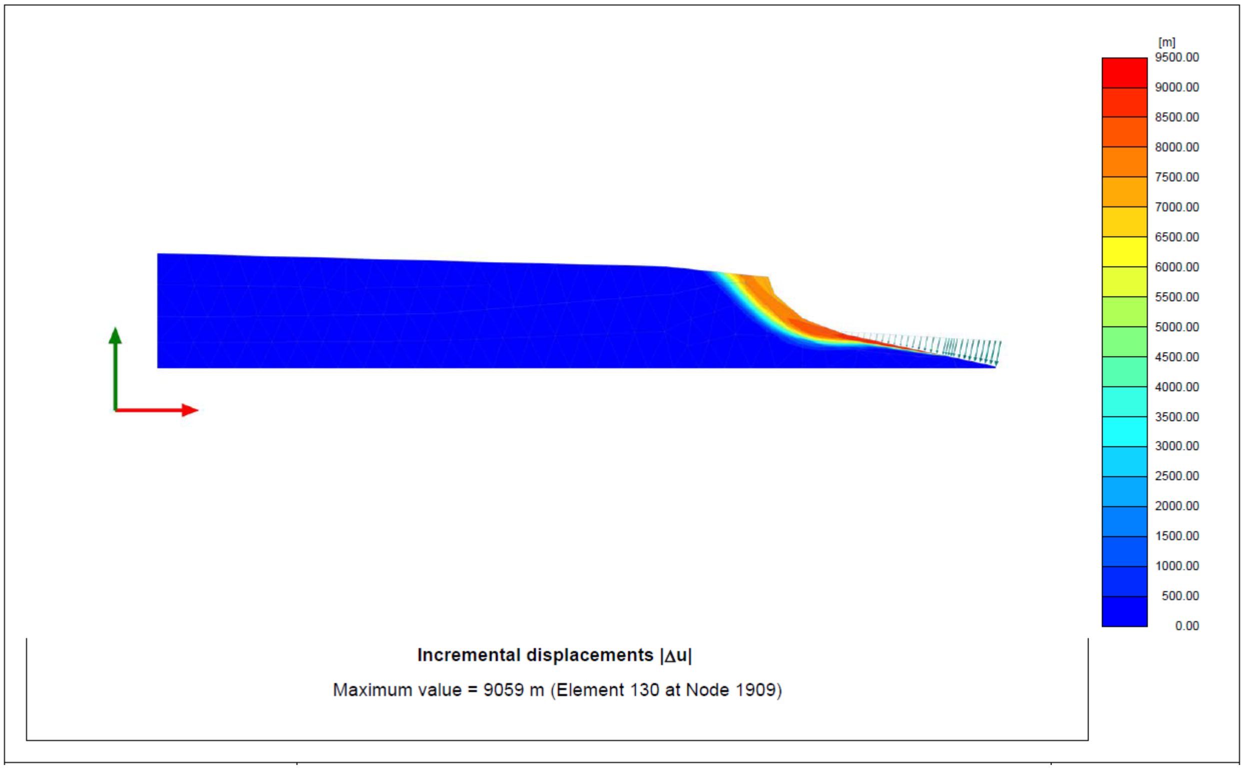


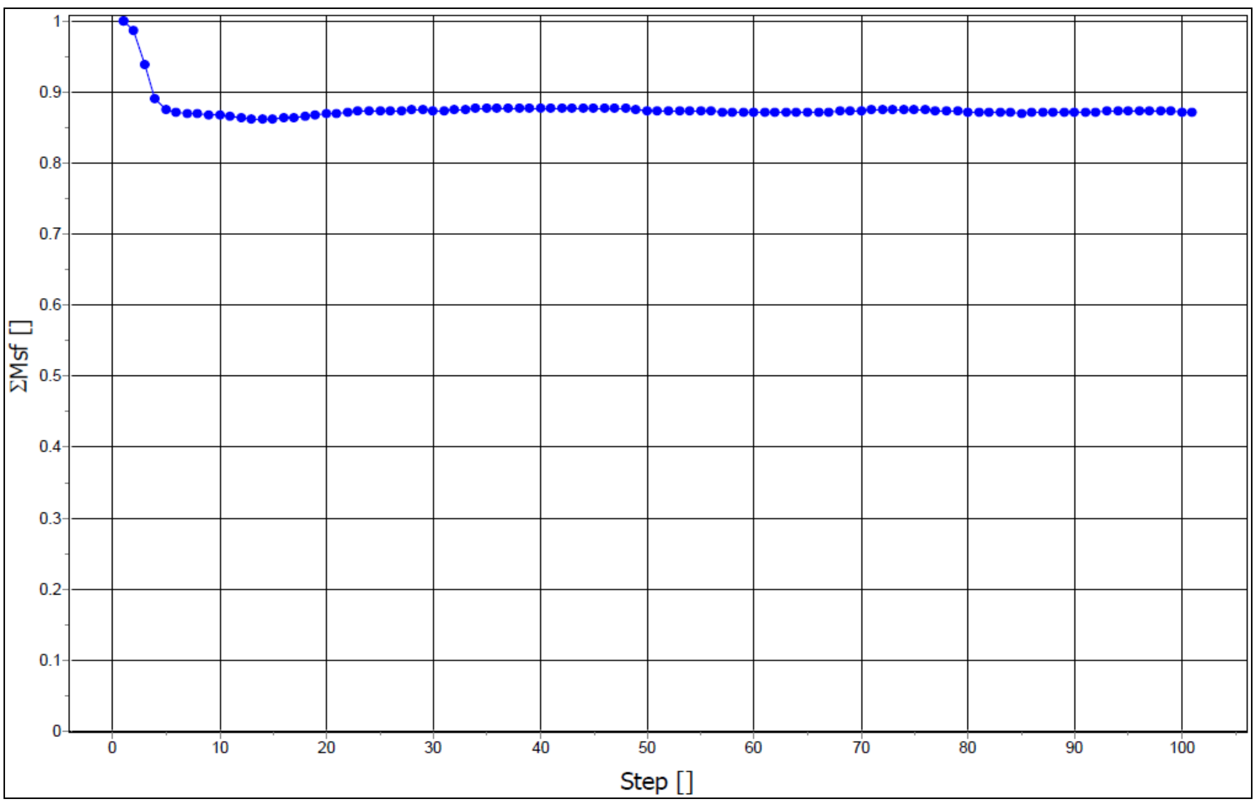
Section B2 Drained



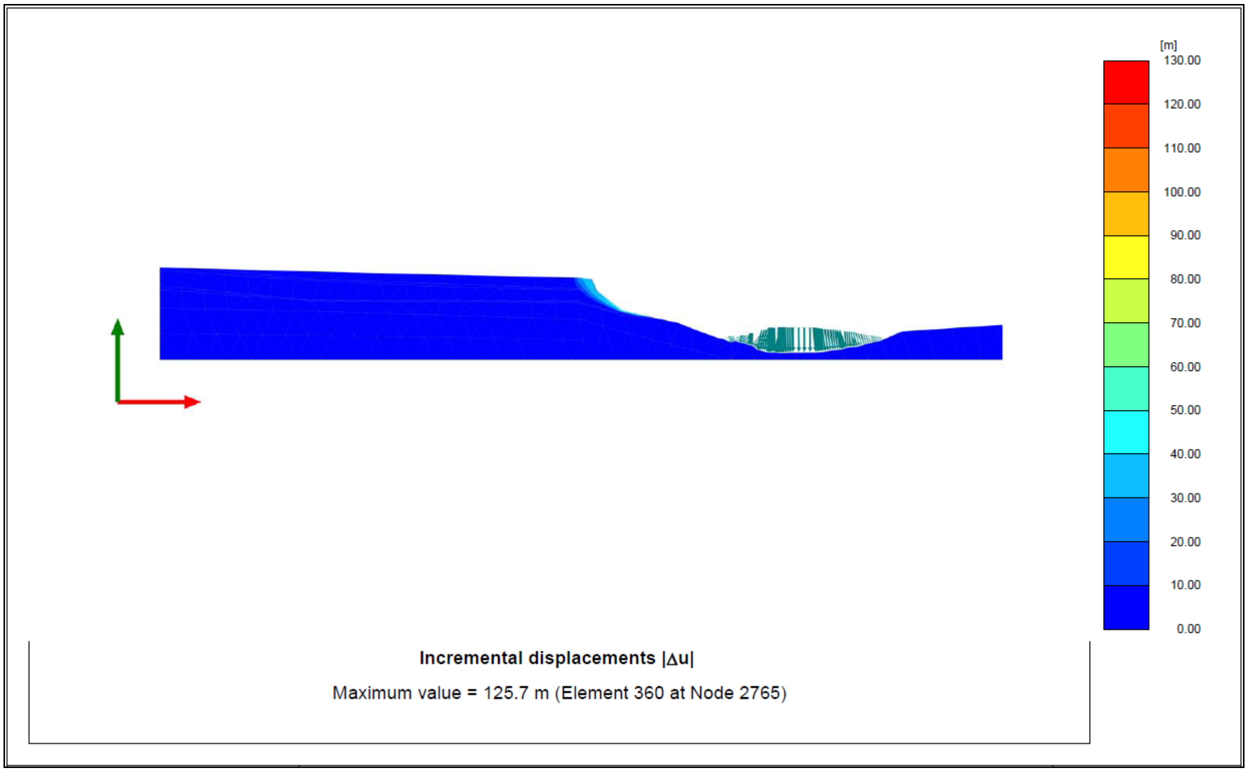


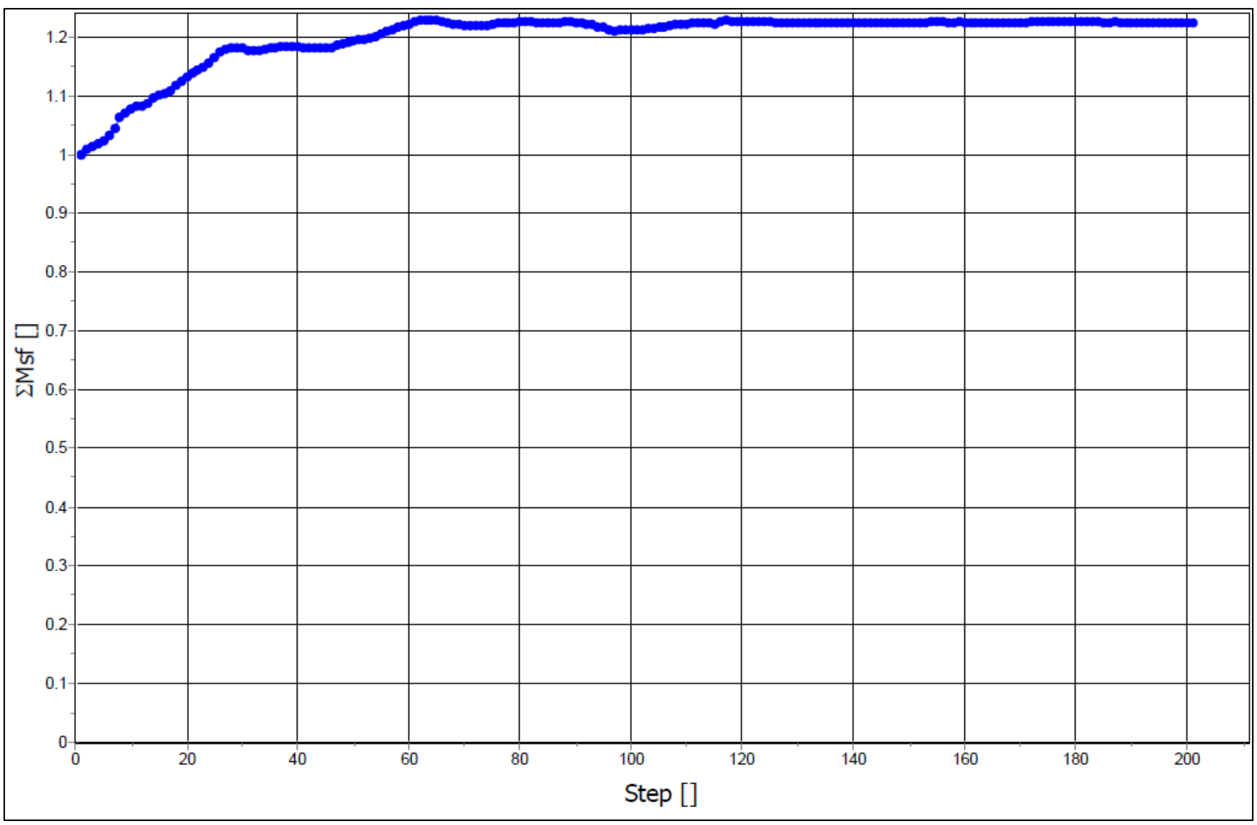
Section B2 Undrained



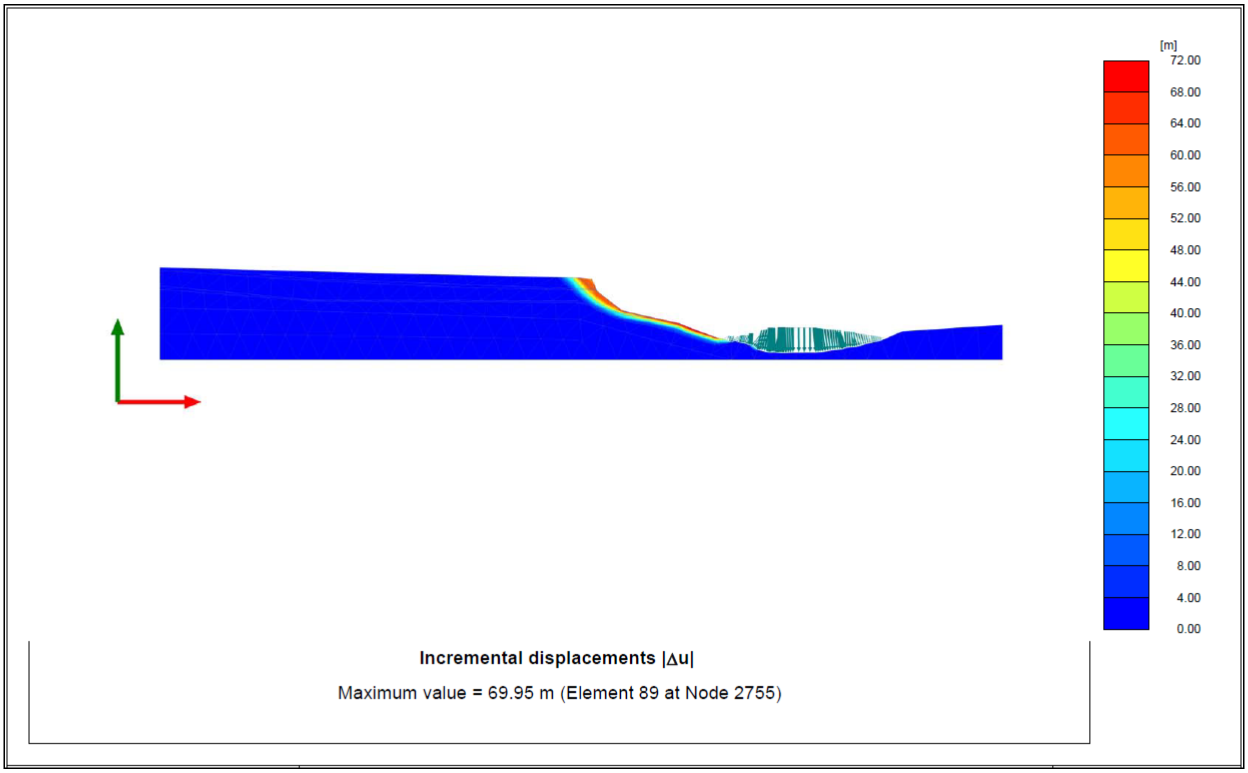


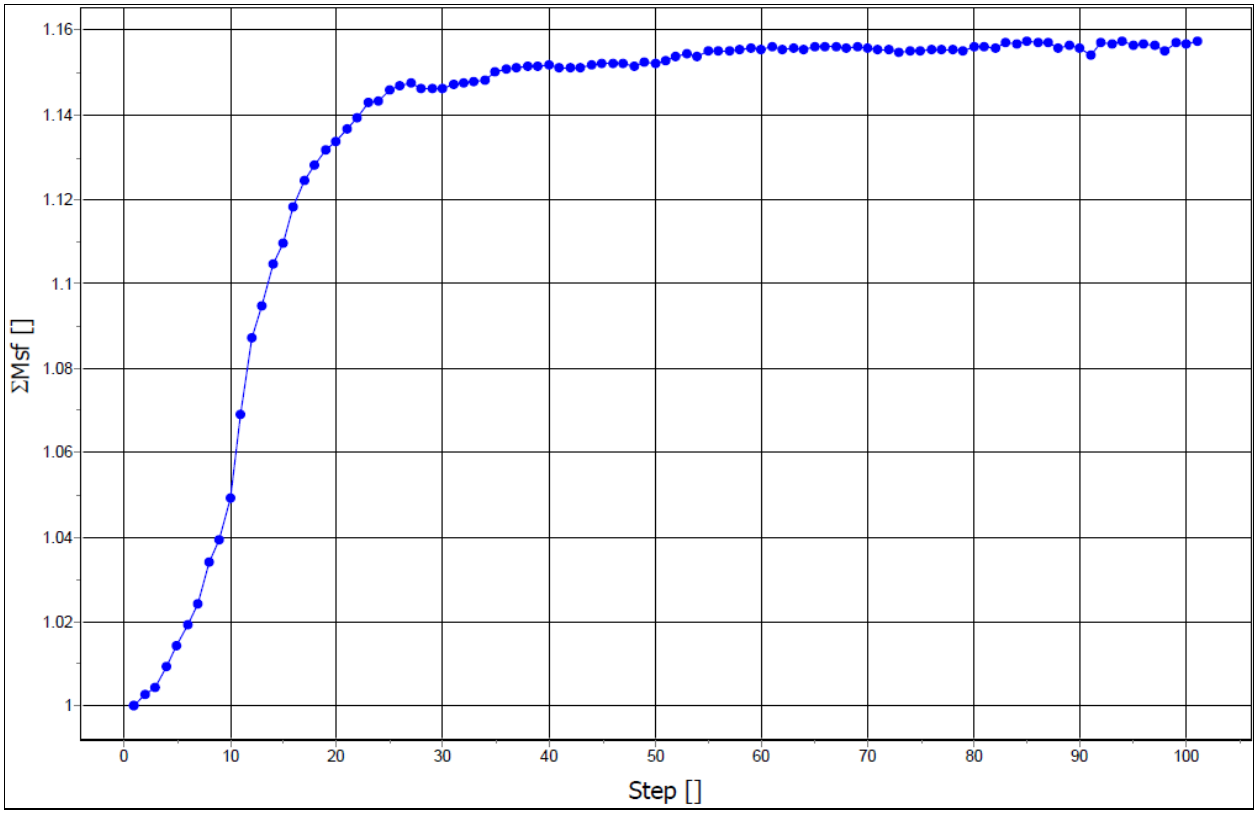
Section C1Drained



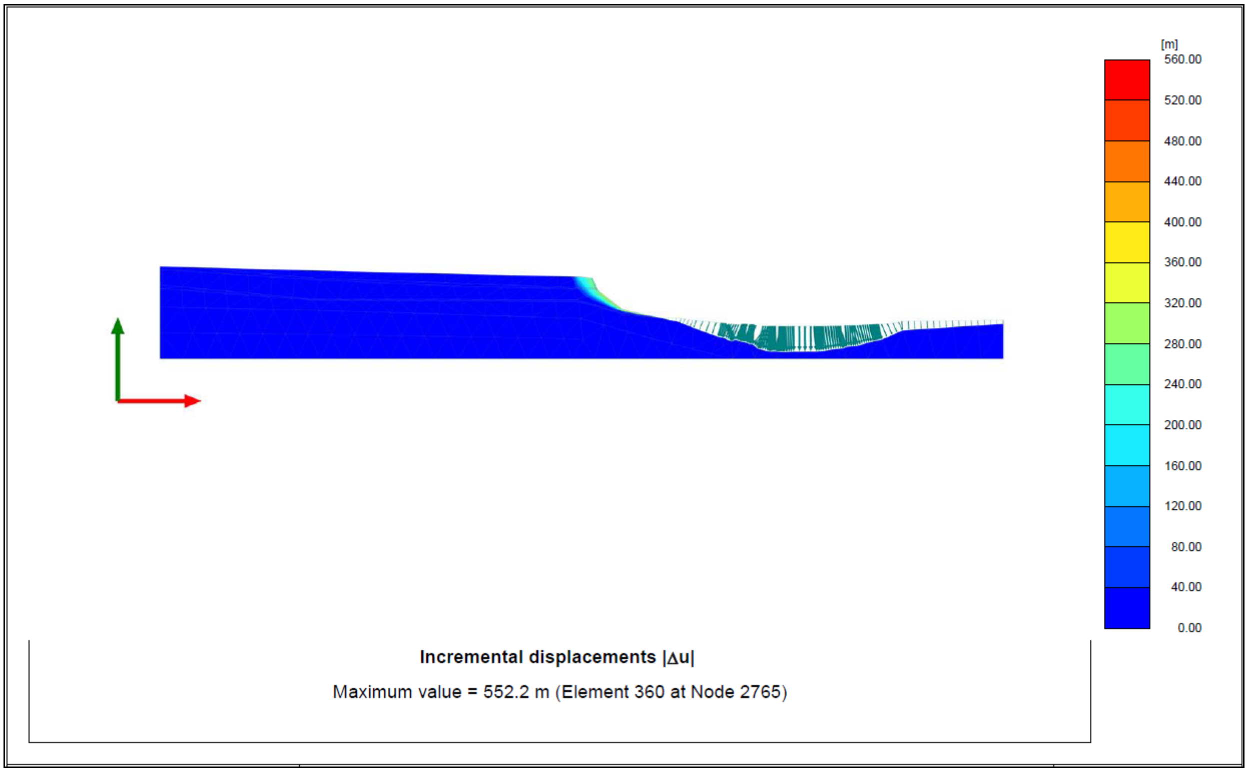


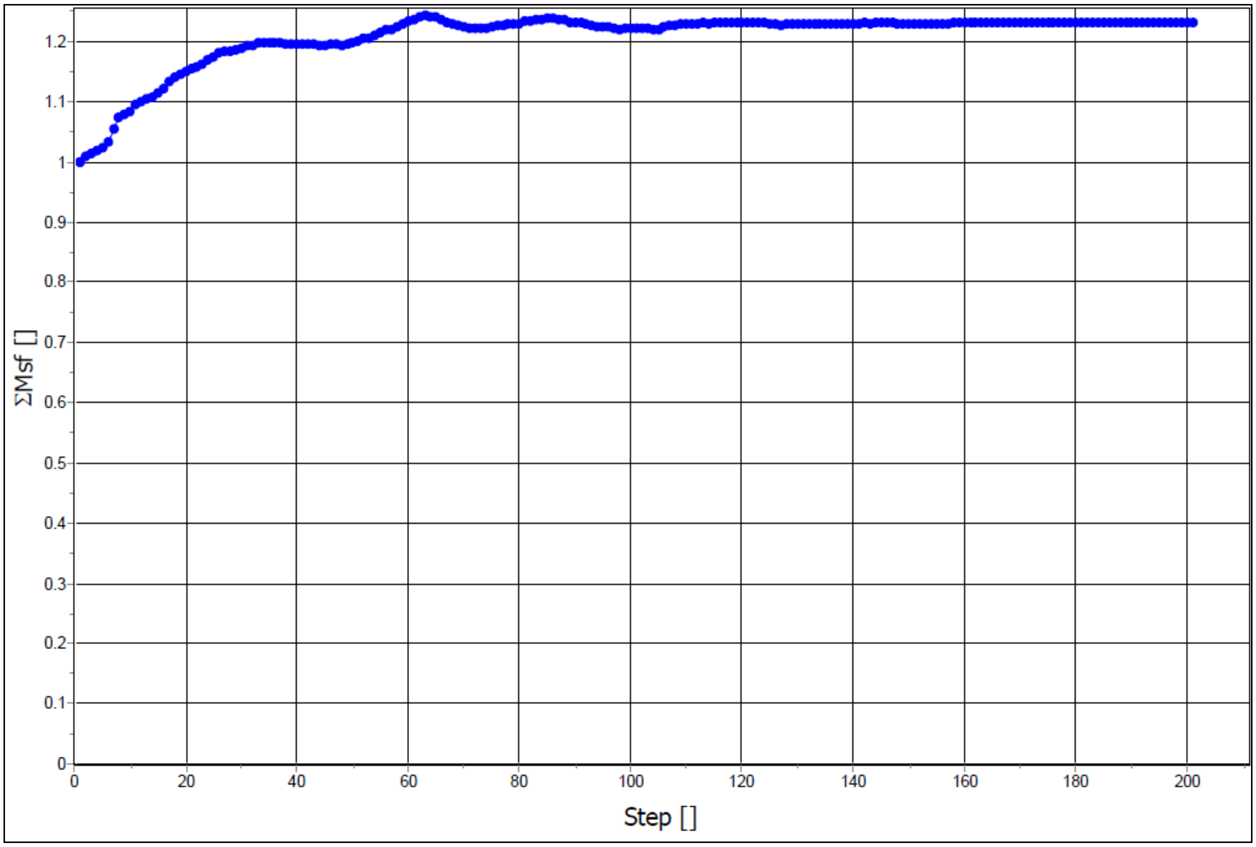
Section C1Undrained



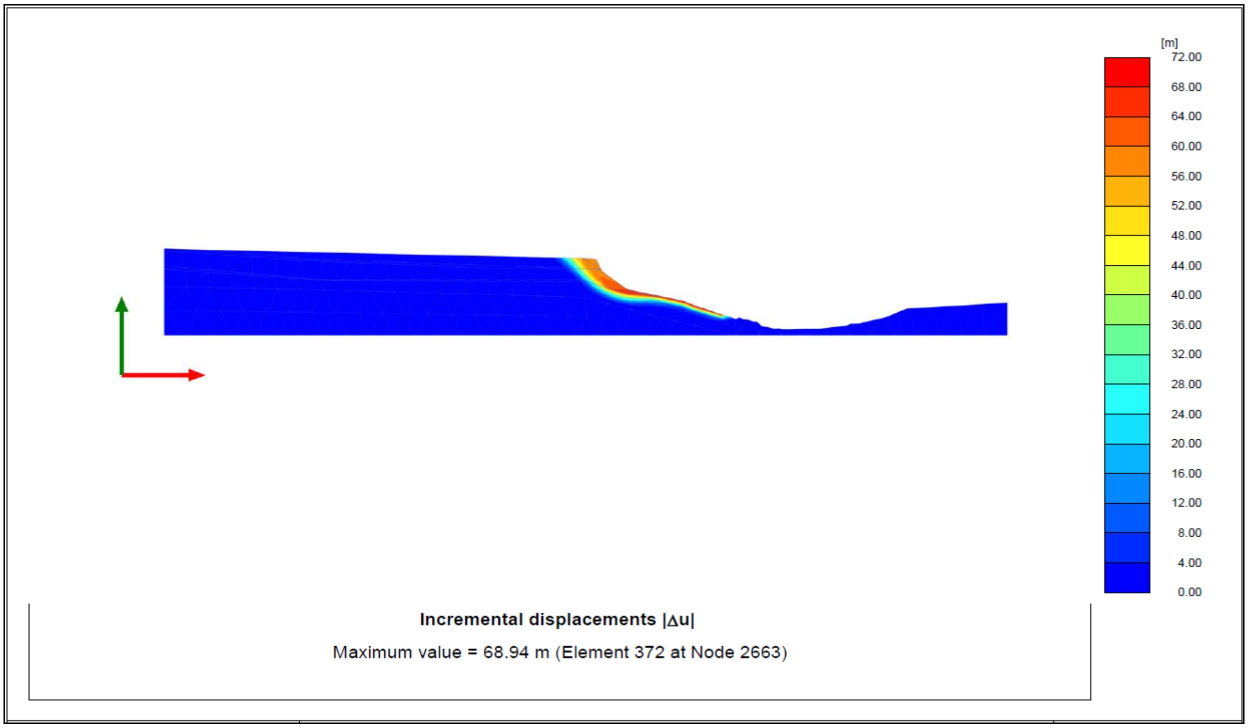


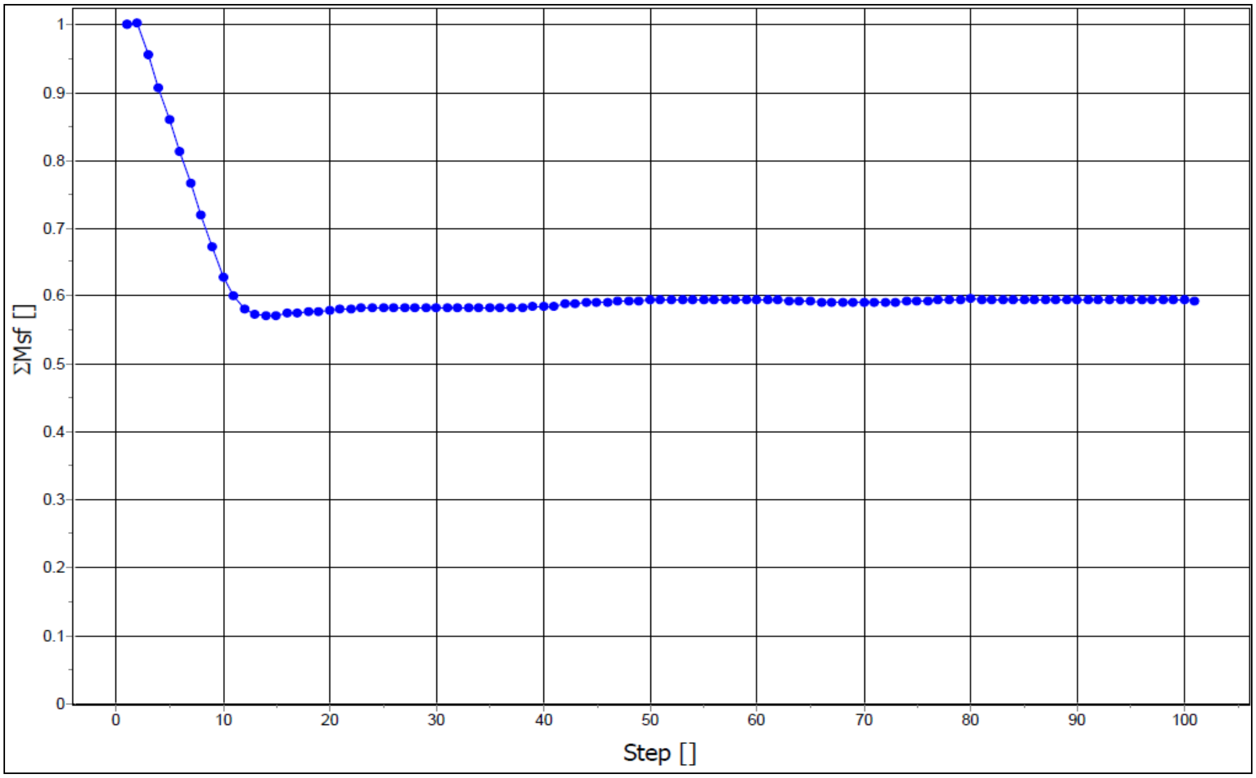
Section C2Drained



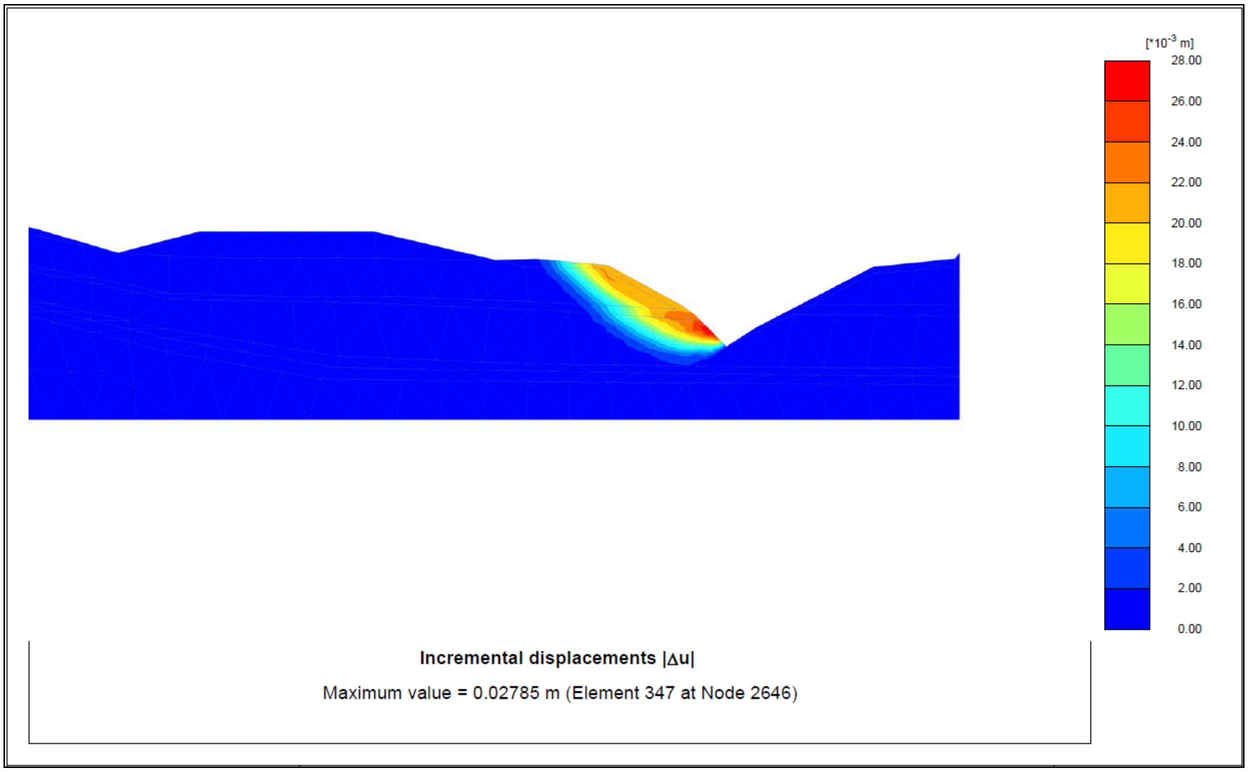


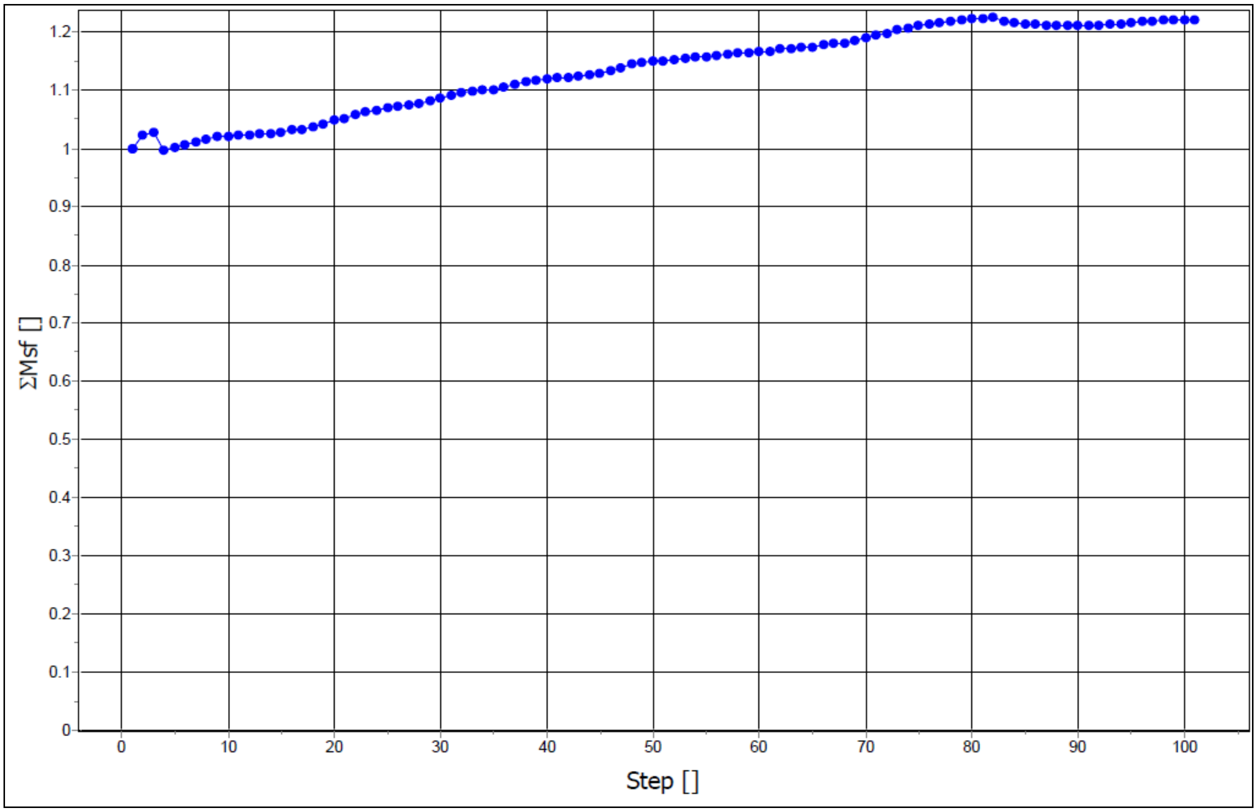
Section C2 Undrained



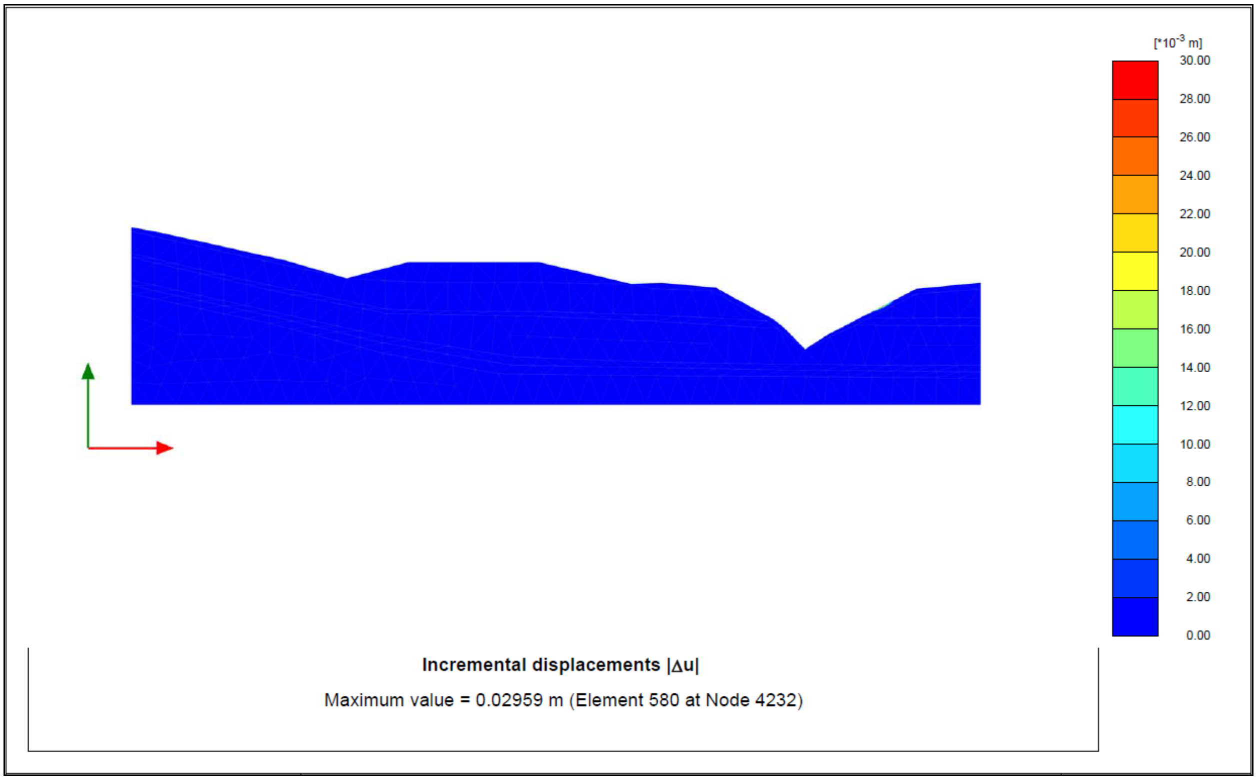


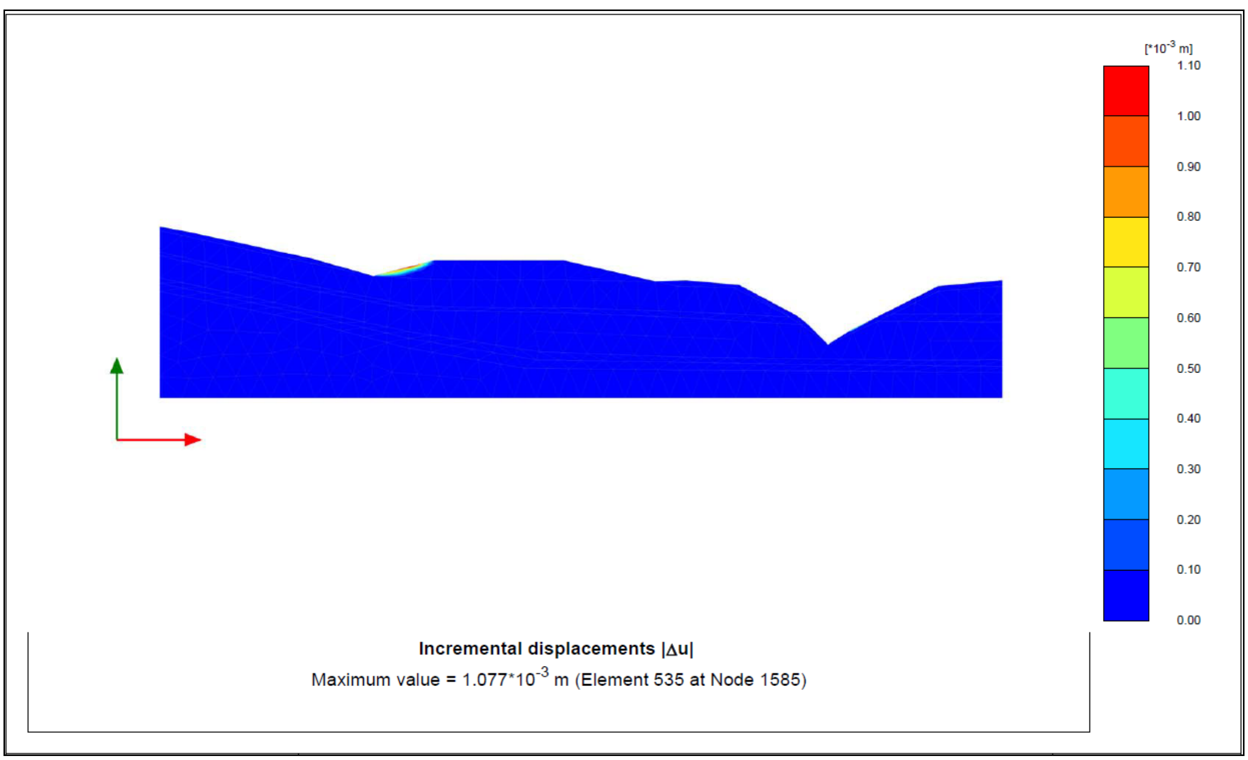
Section D1 Drained

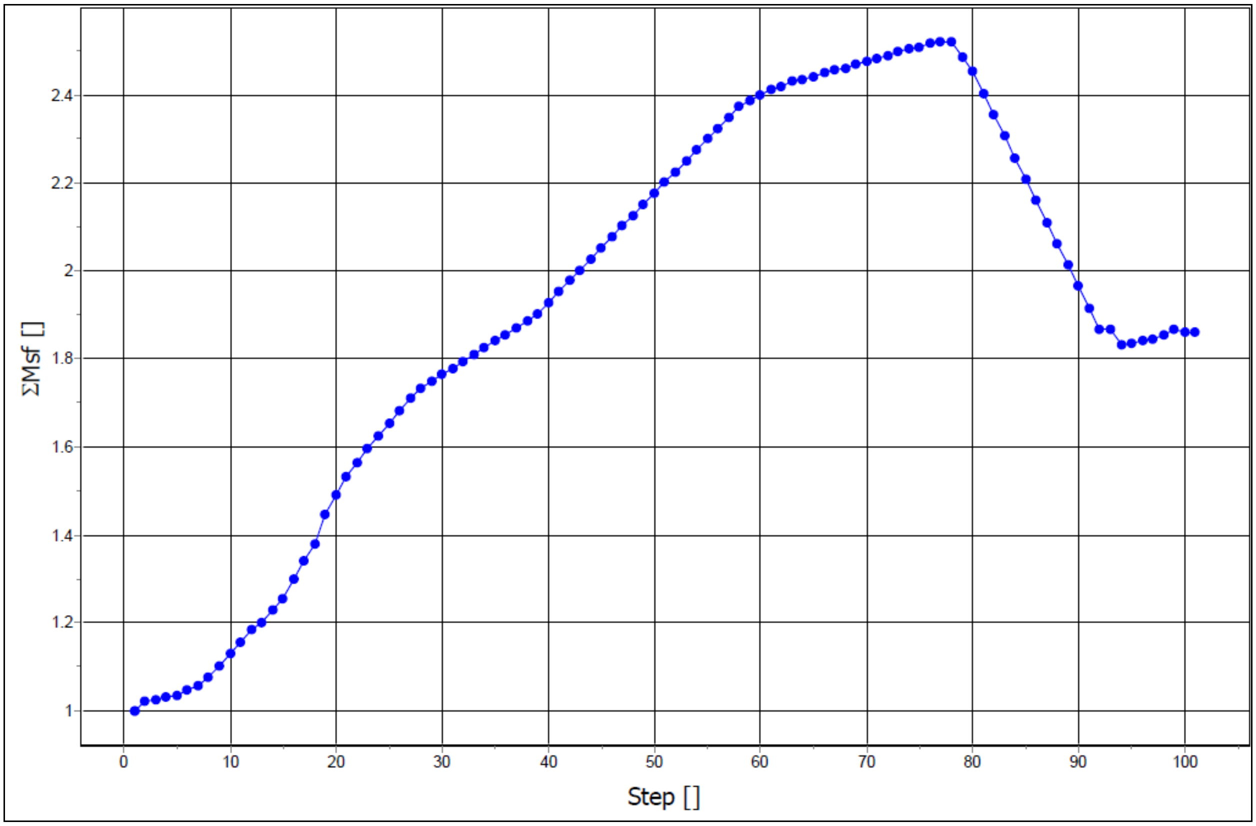




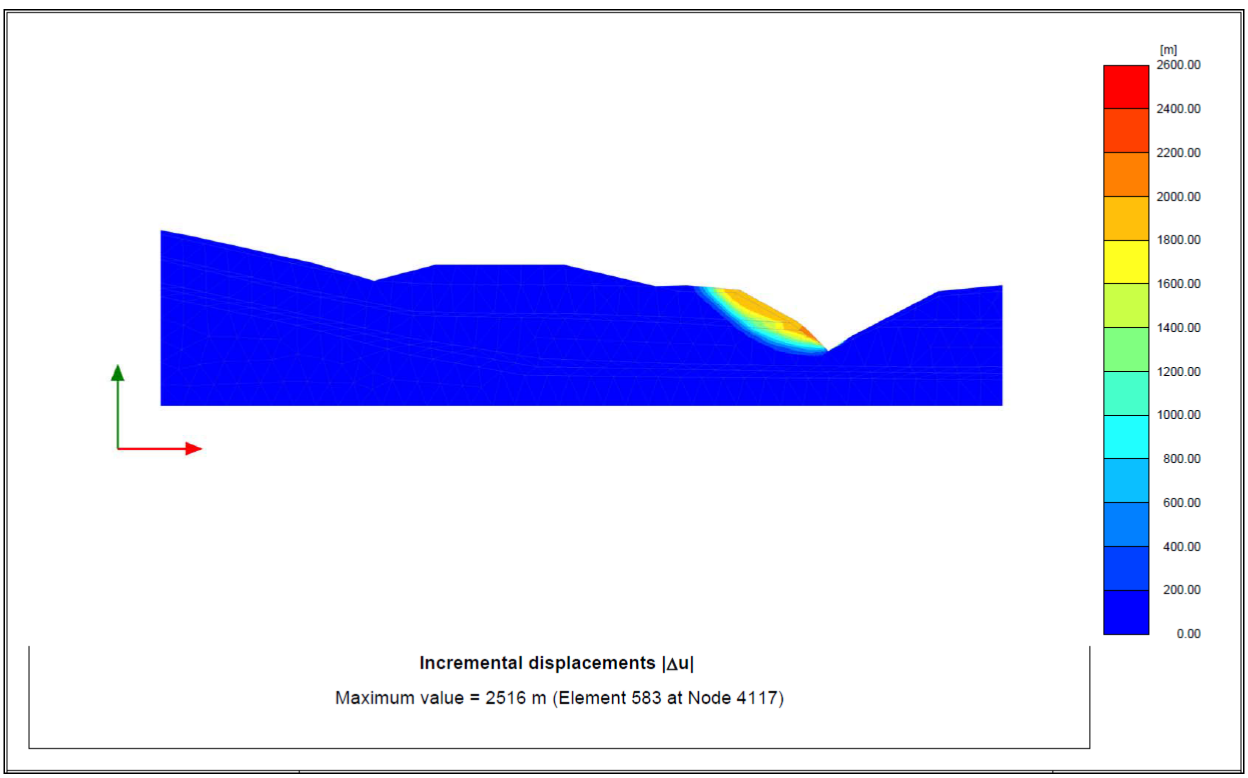
Section D1 Undrained

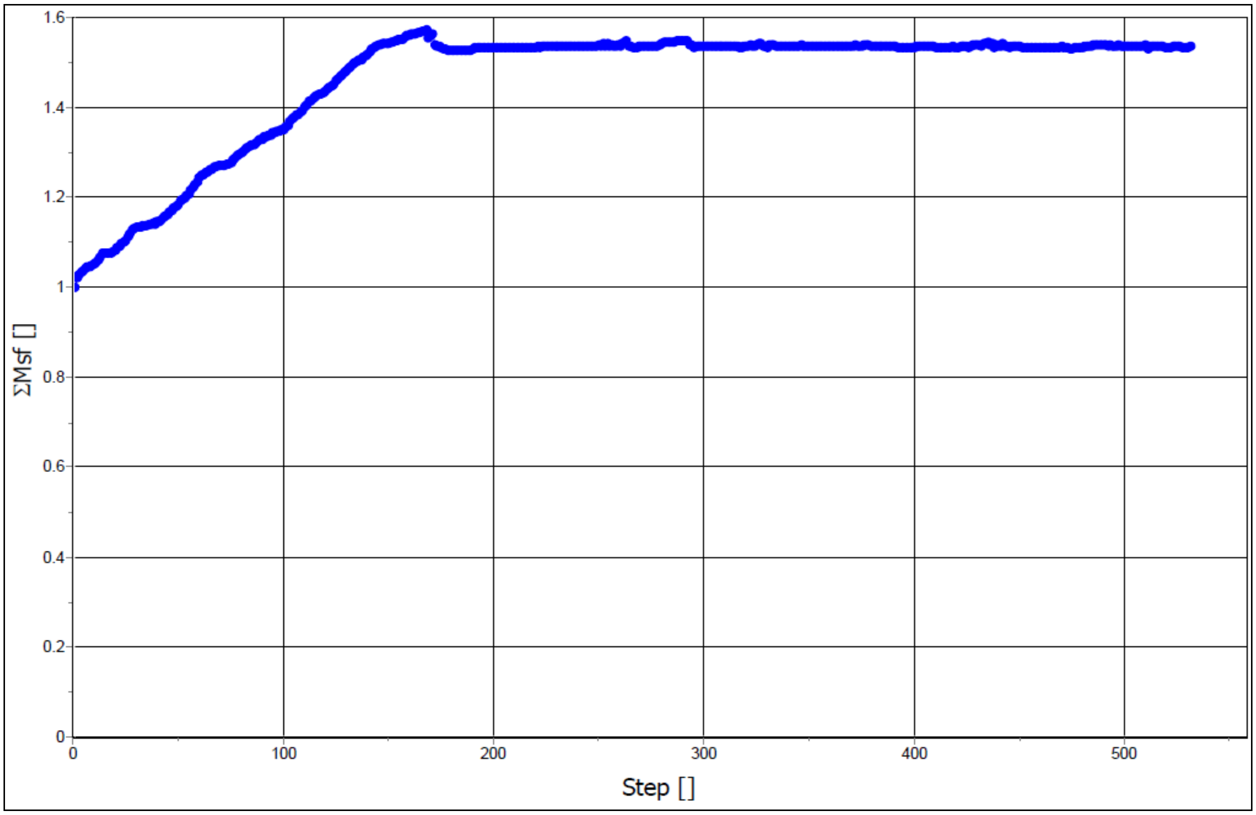




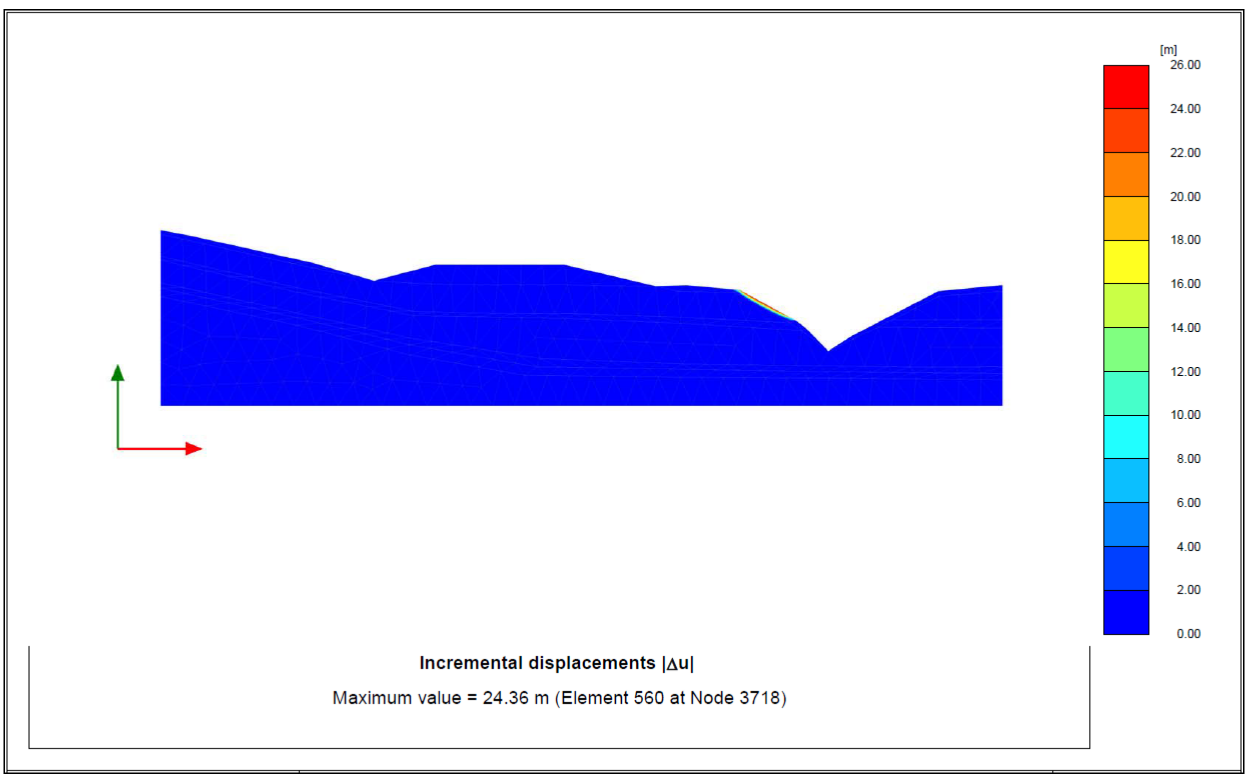


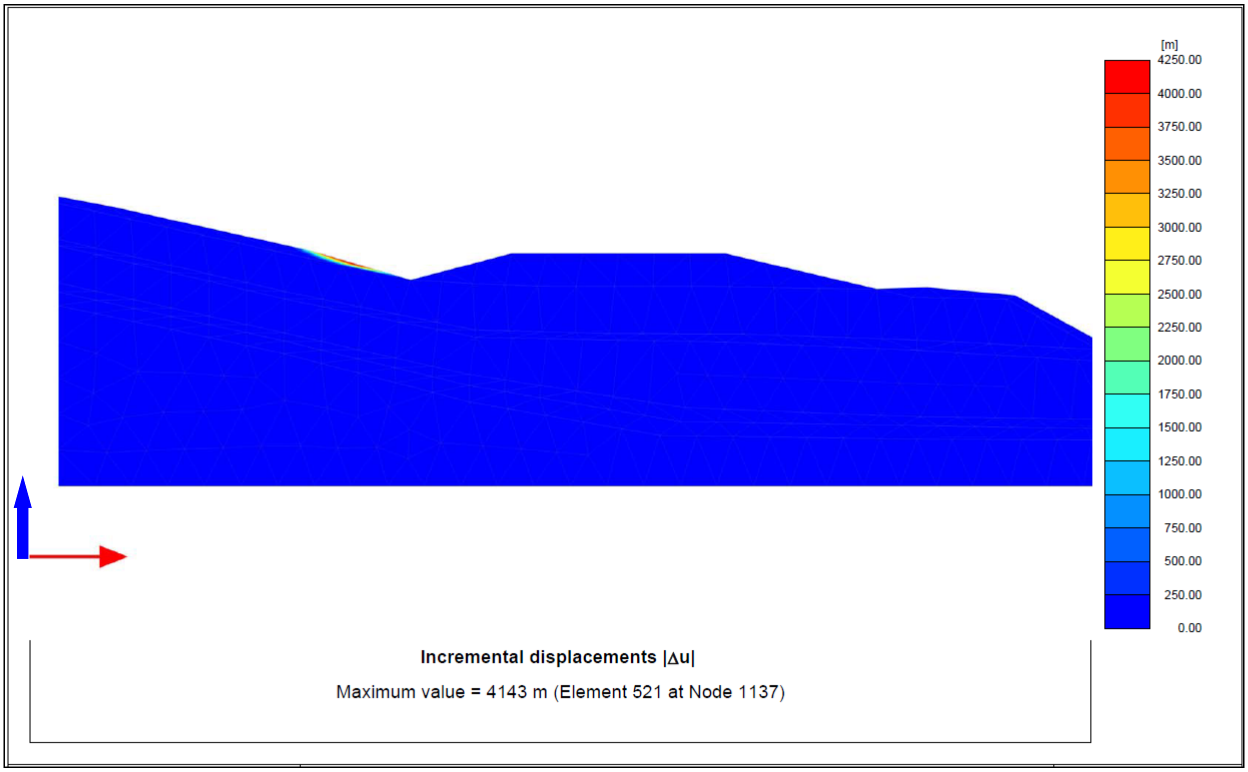
Section D2 Drained

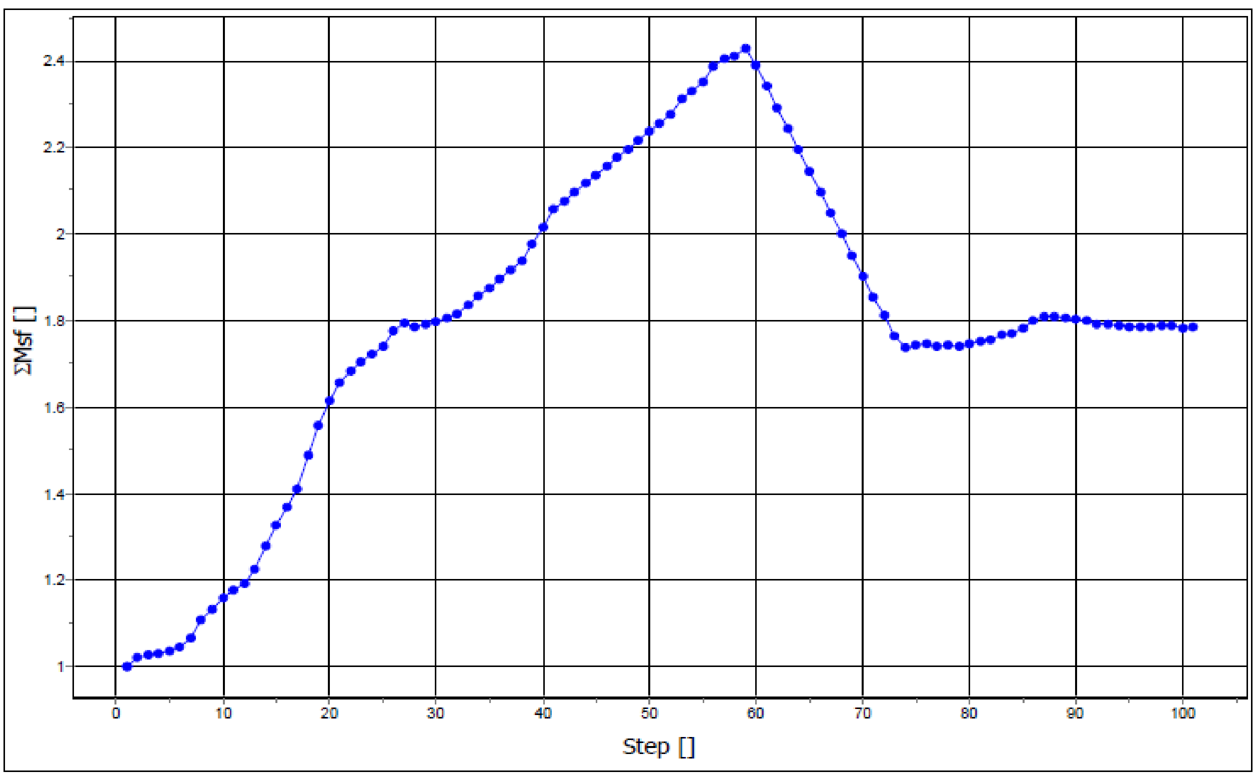




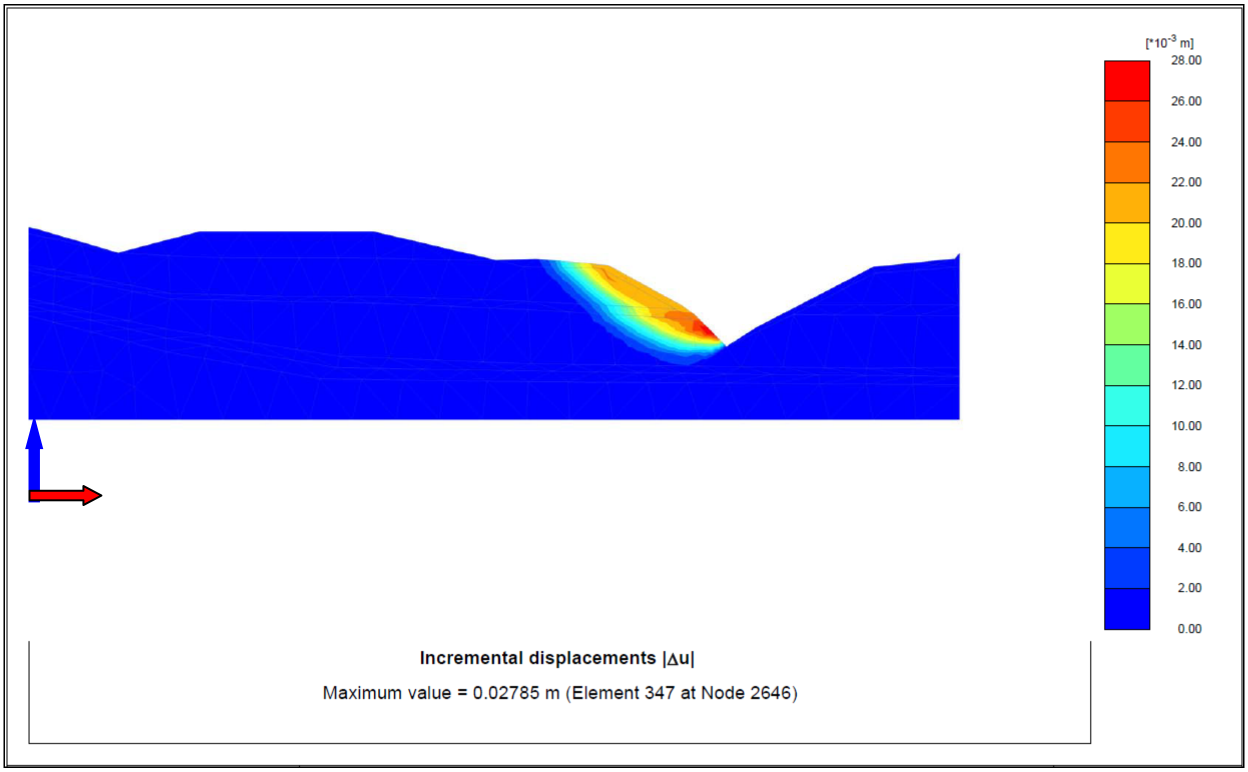
Section D2 Undrained

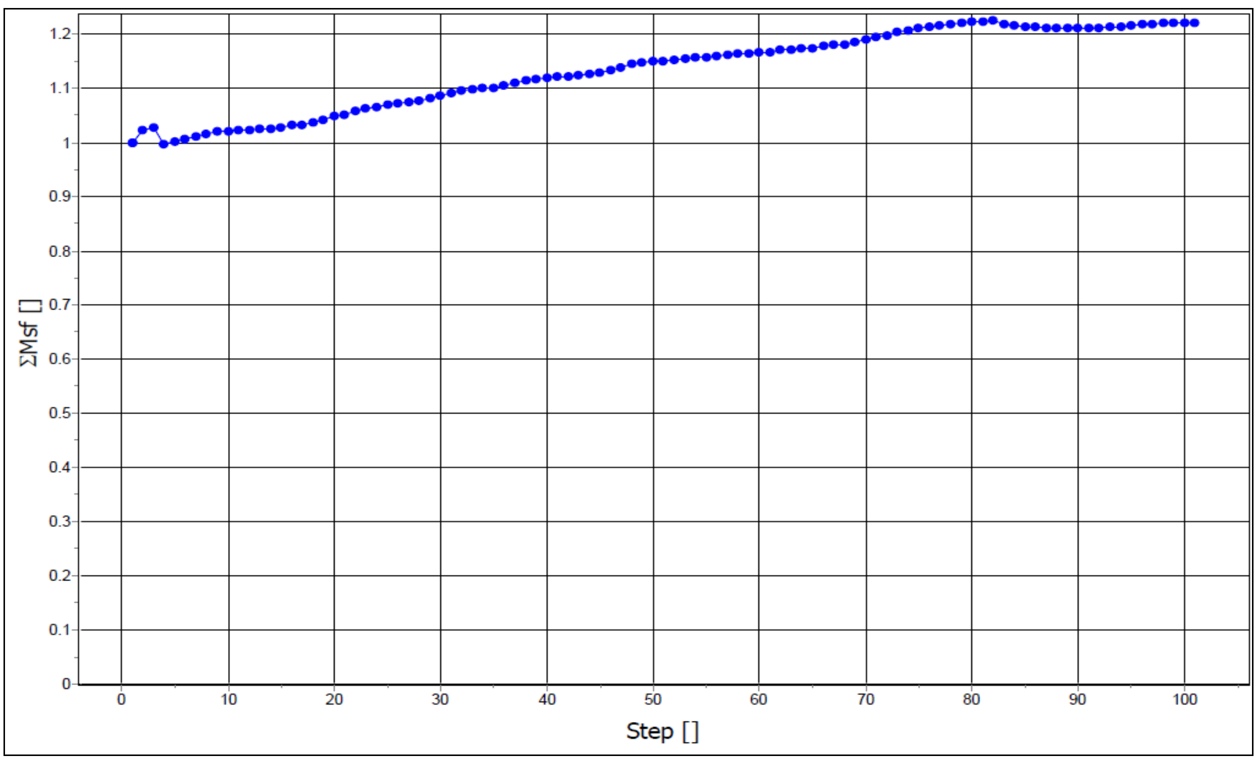



[m]
4250.00
4000.00
3750.00
3500.00
3250.00
3000.00
2750.00
2500.00
2250.00
2000.00
1750.00
1500.00
1250.00
1000.00
750.00
500.00
250.00
0.00
Incremental displacements |Δu|
Maximum value = 4143 m (Element 521 at Node 1137)

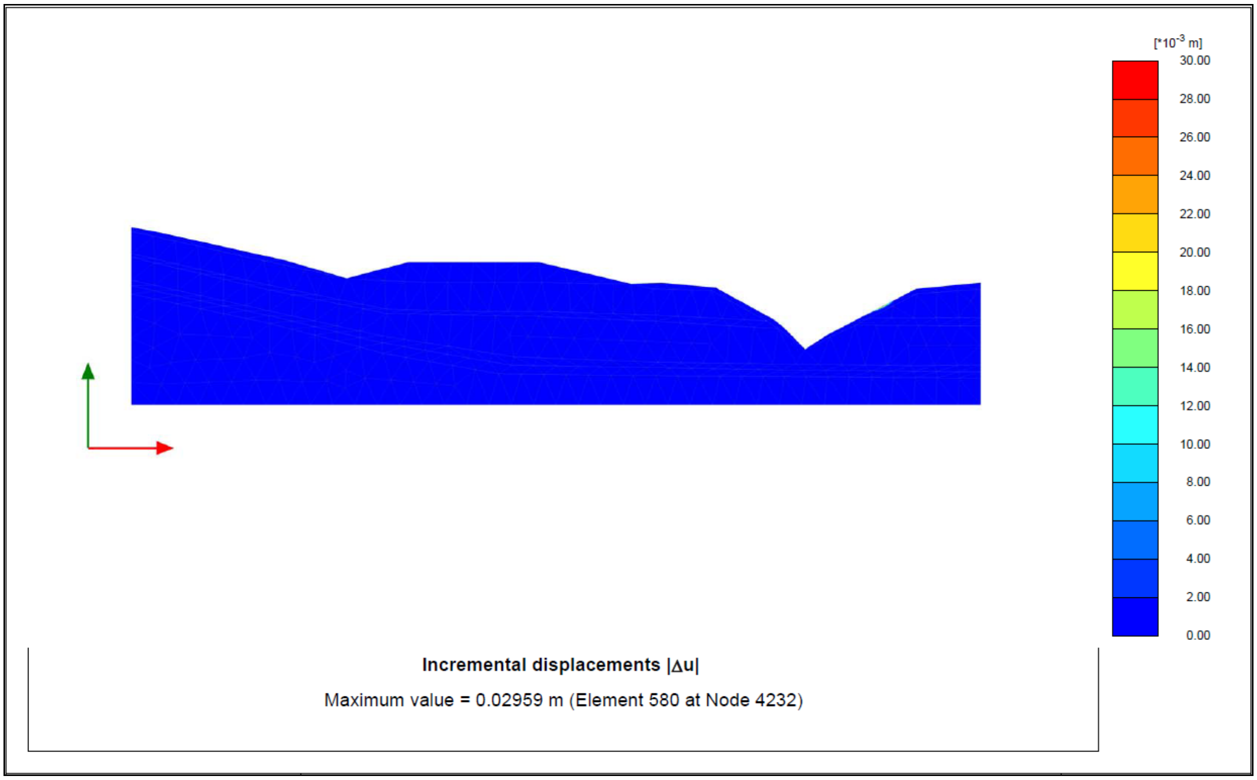

ΣMsf []
Step []
1
1.2
1.4
1.6
1.8
2
2.2
2.4
0
10
20
30
40
50
60
70
80
90
100

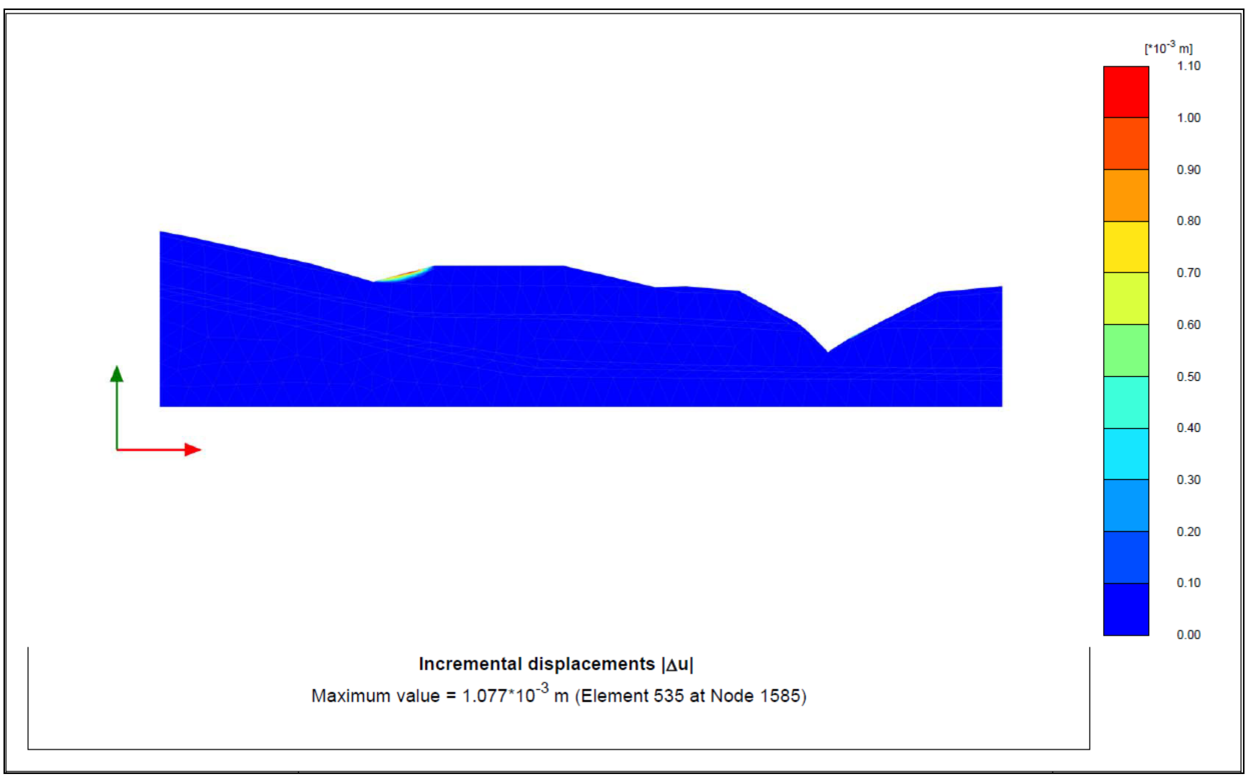
Section D.1B Drained

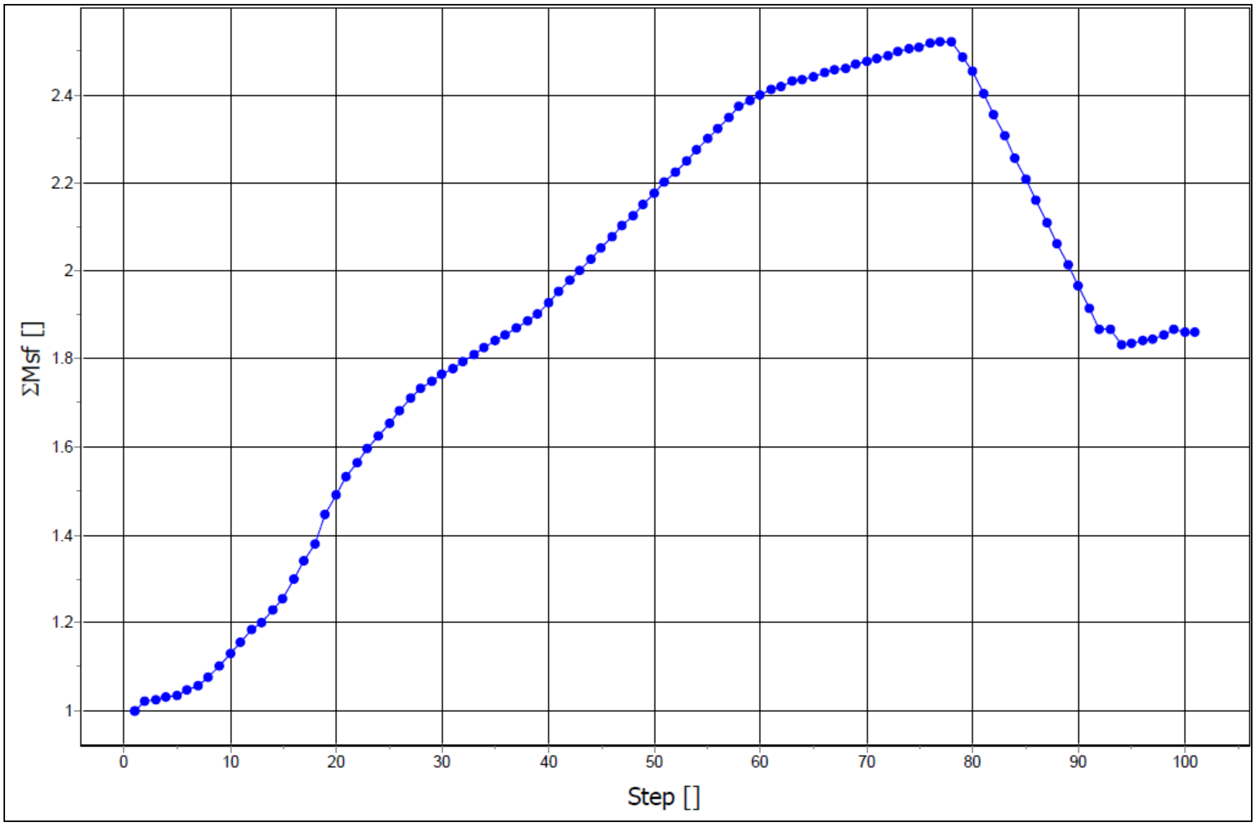




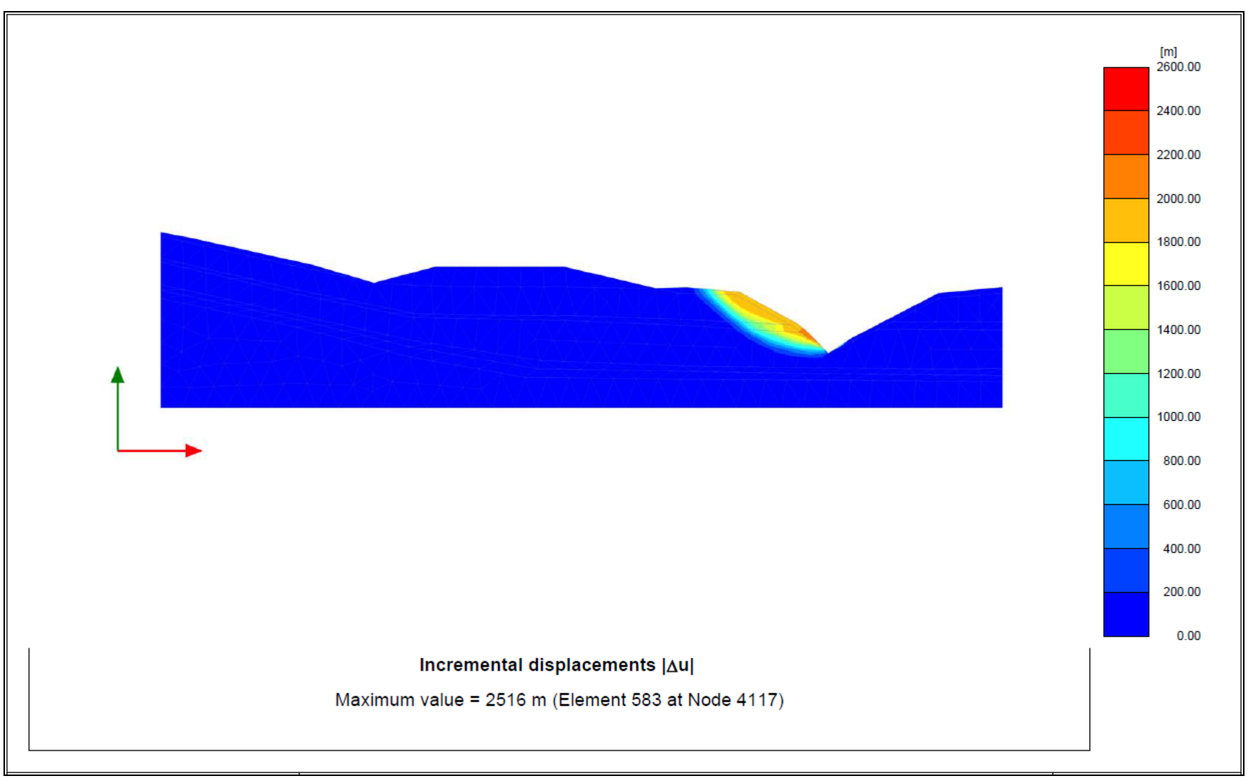
Section D1.B Undrained

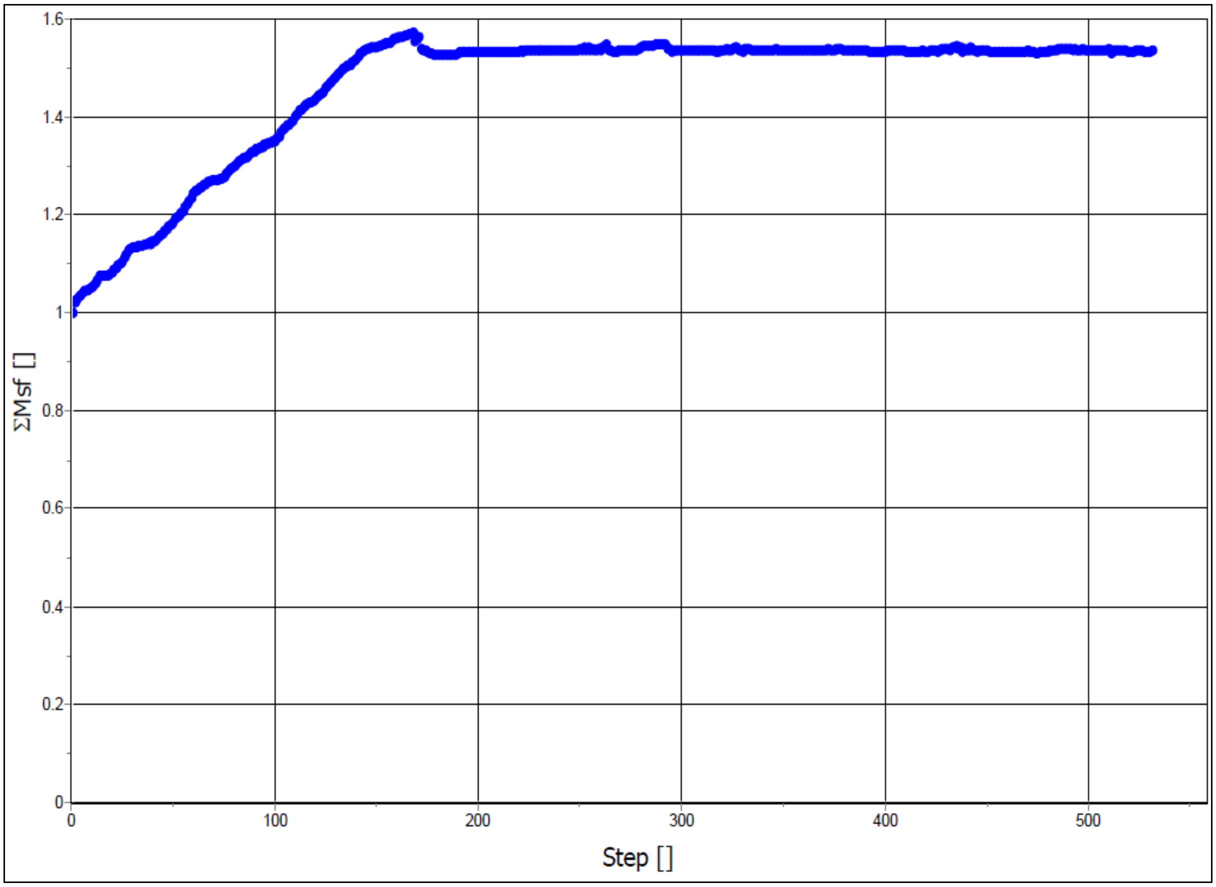




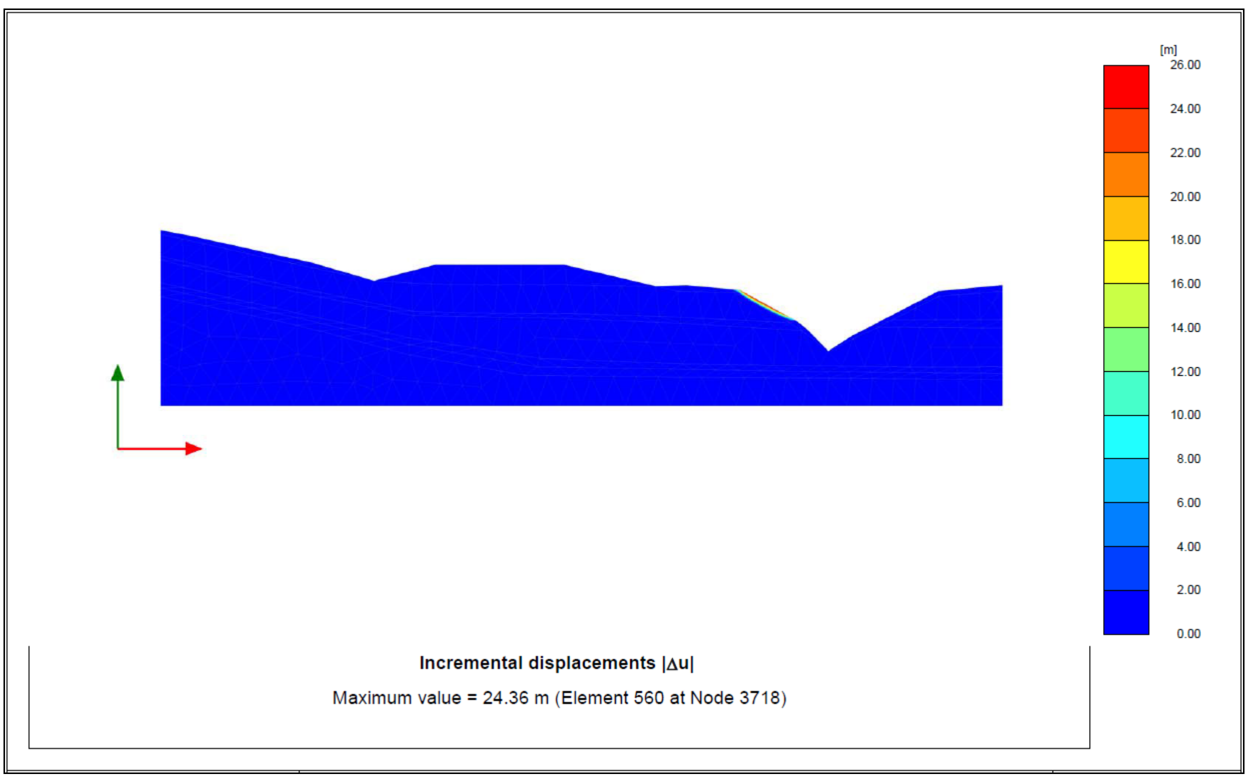


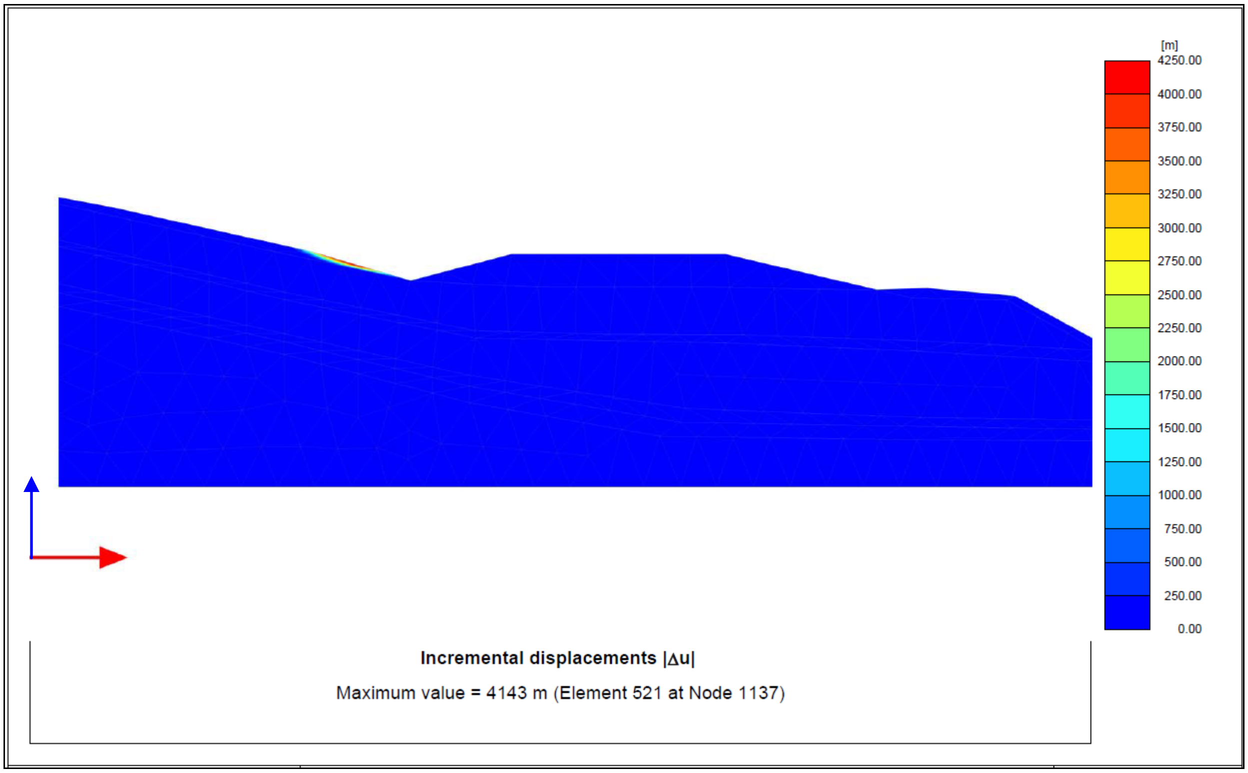
Section D.2B Drained

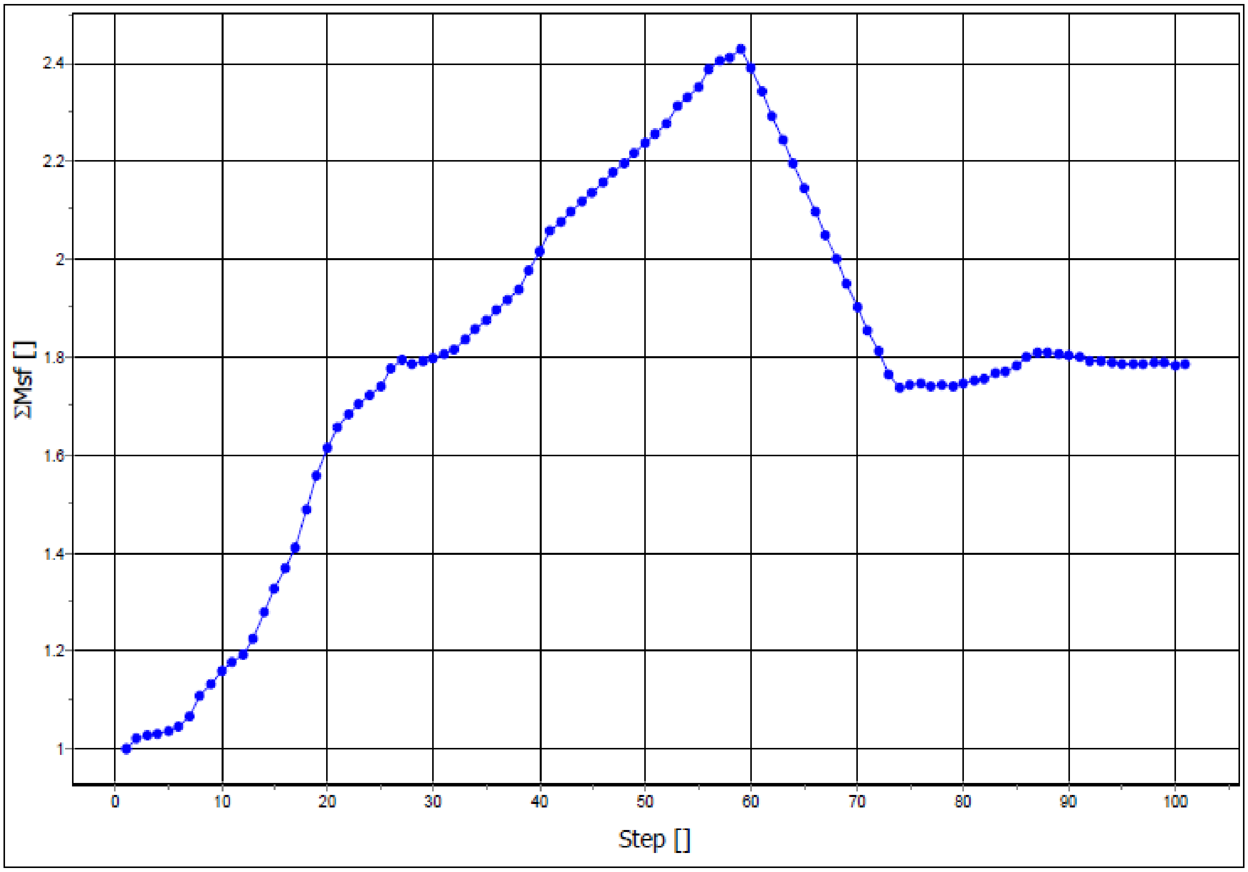




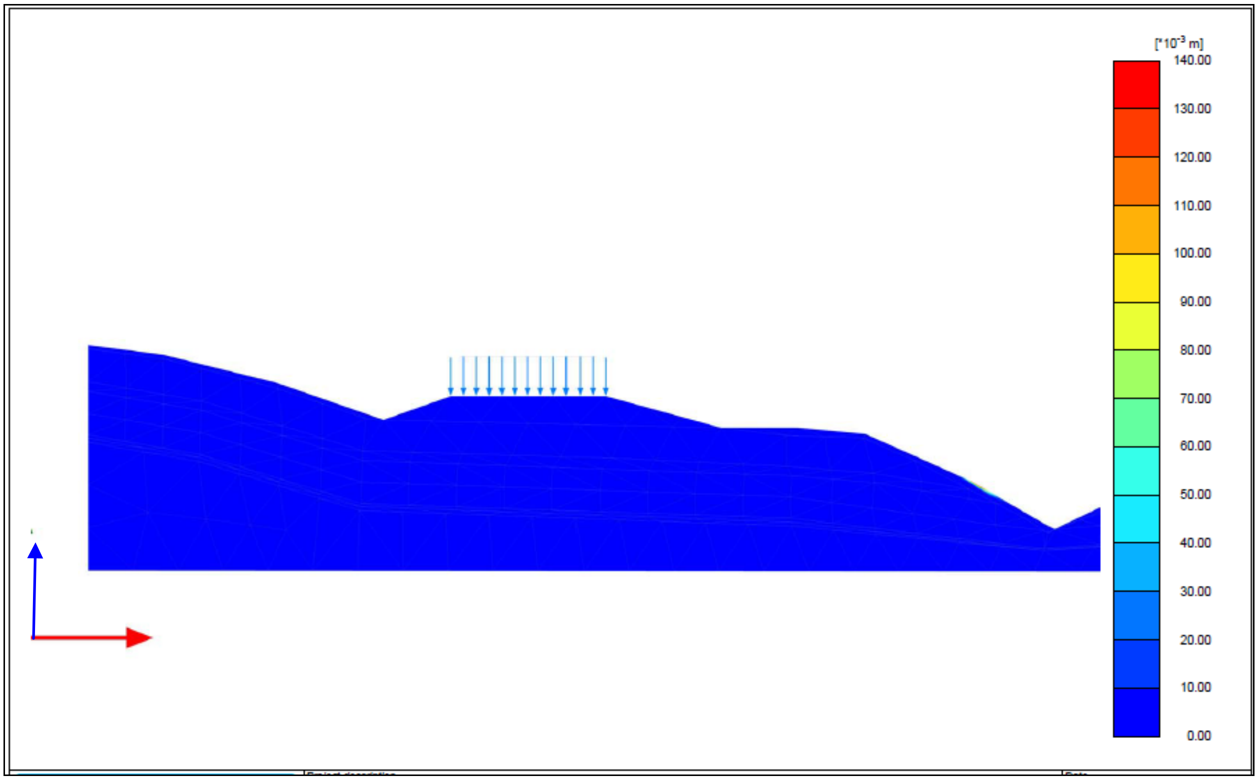
Section D.2B Undrained

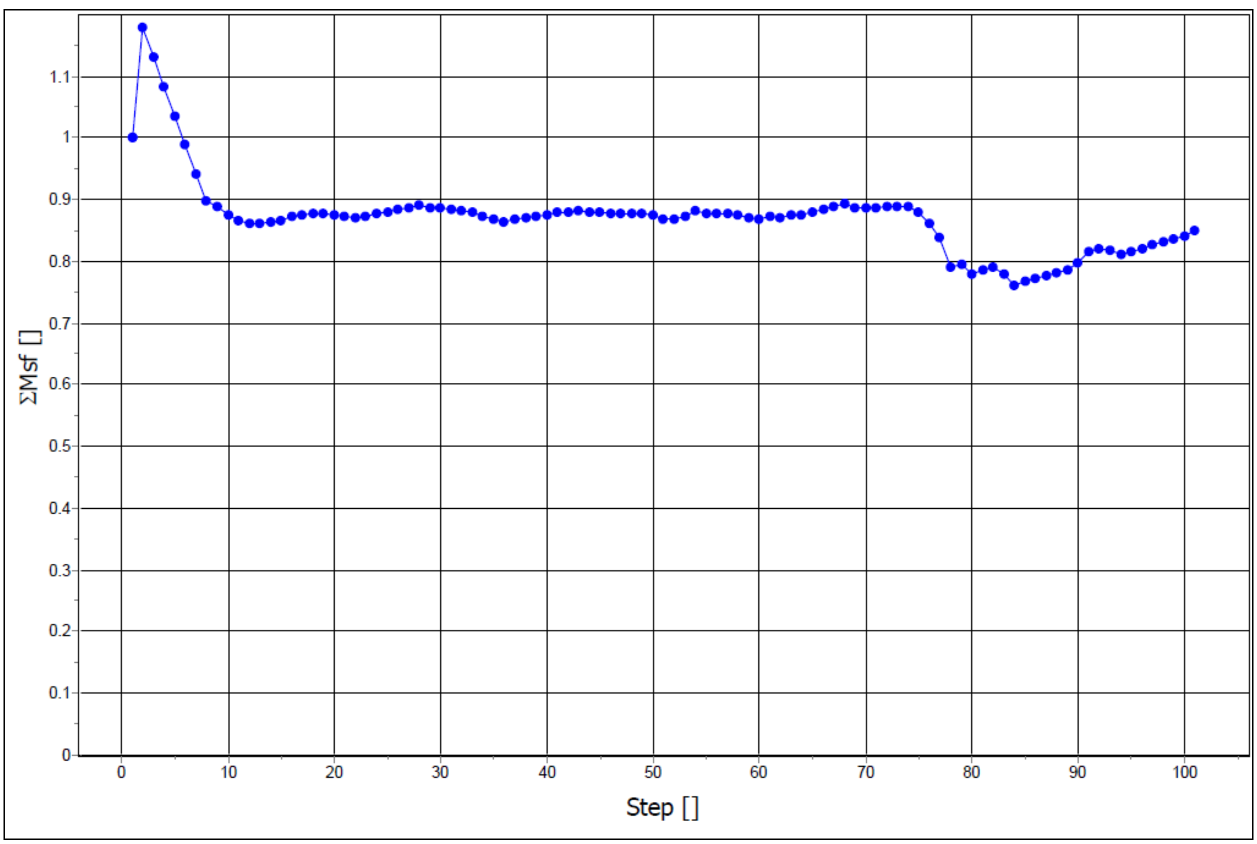


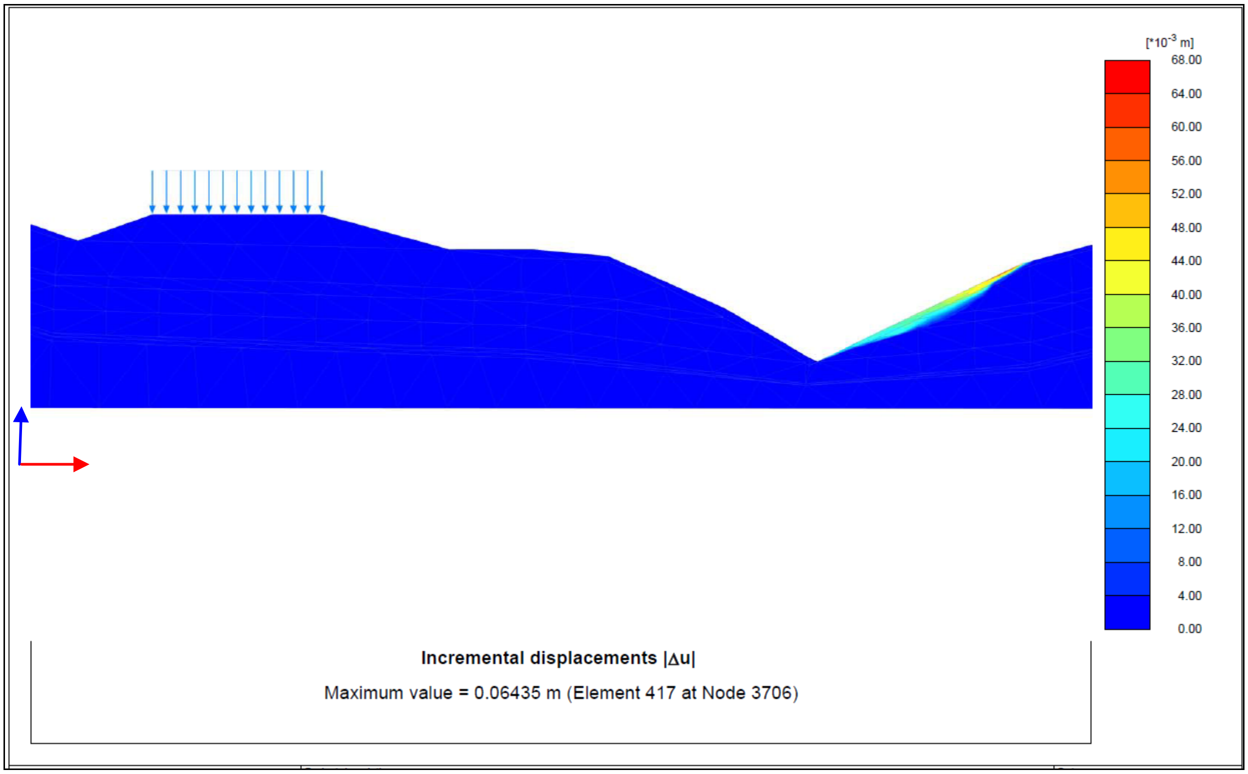

[m]
4250.00
4000.00
3750.00
3500.00
3250.00
3000.00
2750.00
2500.00
2250.00
2000.00
1750.00
1500.00
1250.00
1000.00
750.00
500.00
250.00
0.00
Incremental displacements |Δu|
Maximum value = 4143 m (Element 521 at Node 1137)

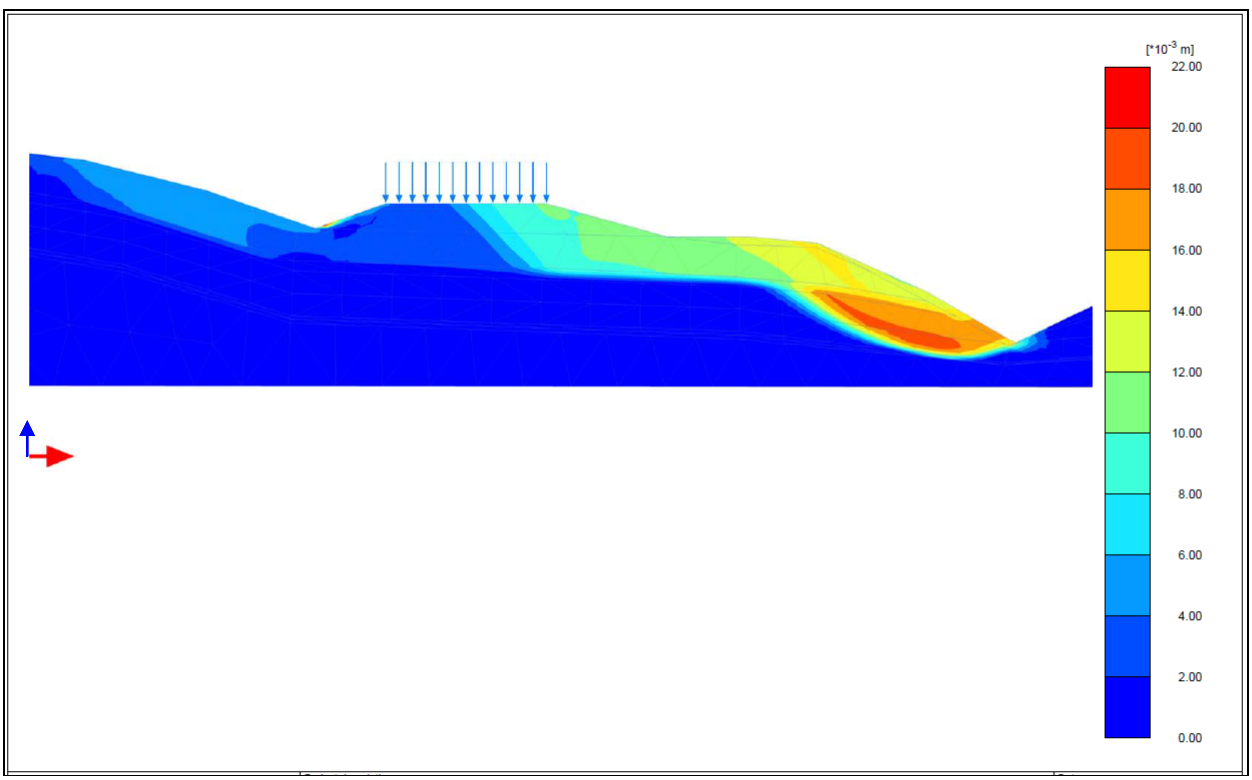

ΣMsf []
Step []

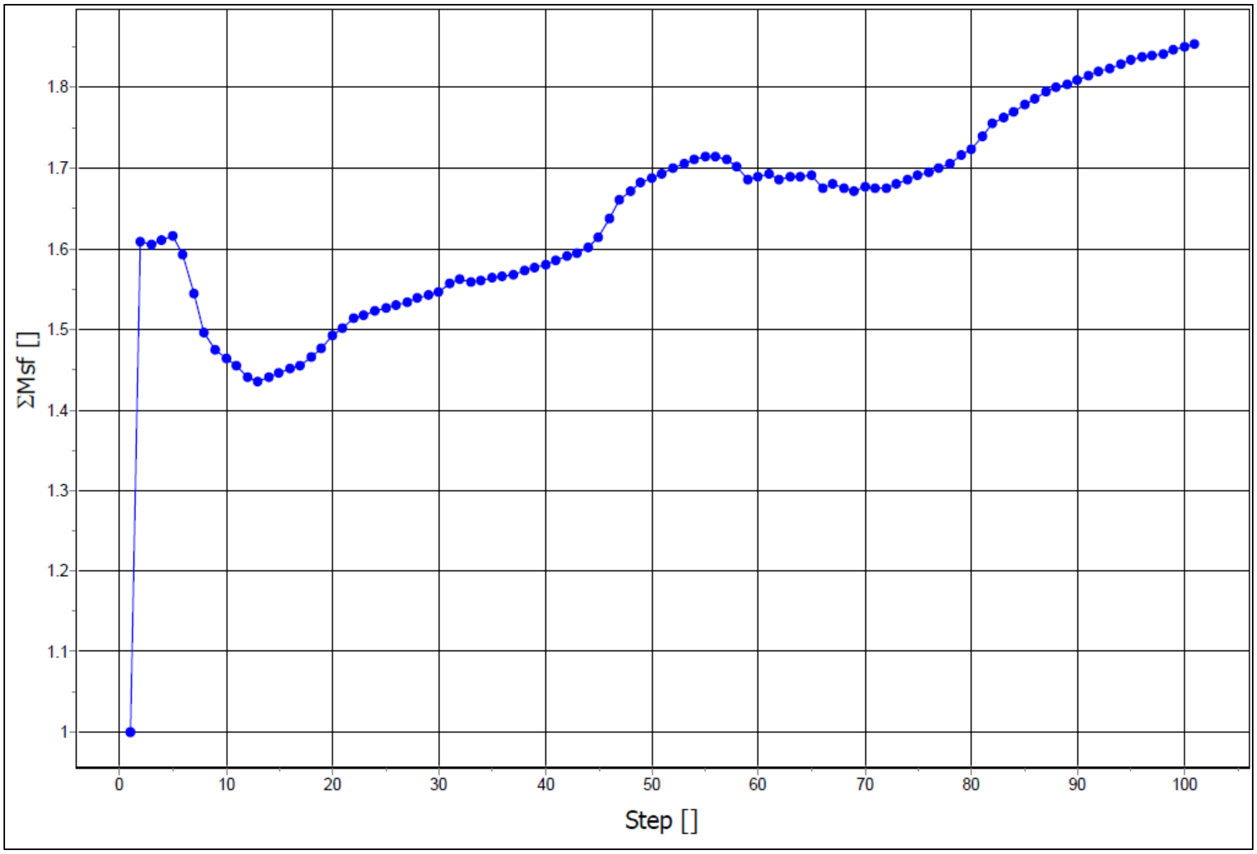
E1B Drained


[*10^-3 m]
140.00
130.00
120.00
110.00
100.00
90.00
80.00
70.00
60.00
50.00
40.00
30.00
20.00
10.00
0.00

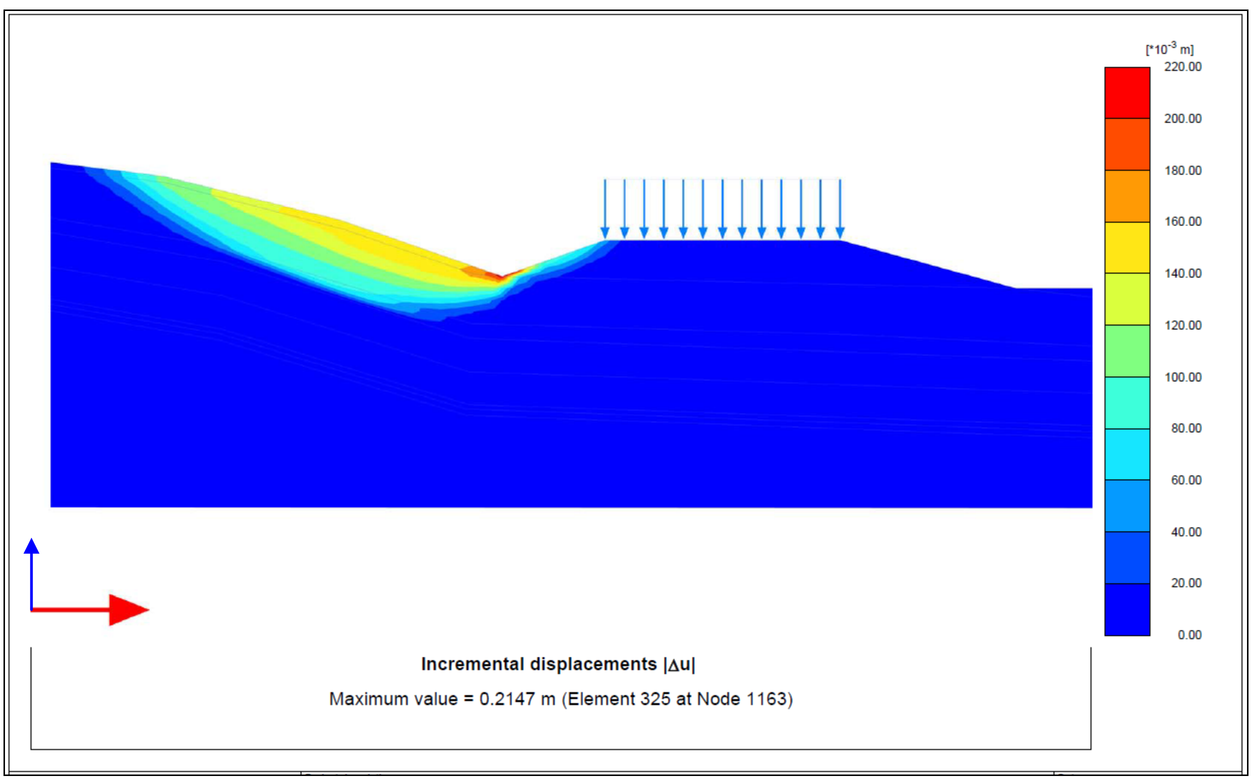

ΣMsf []
Step []
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
0
10
20
30
40
50
60
70
80
90
100

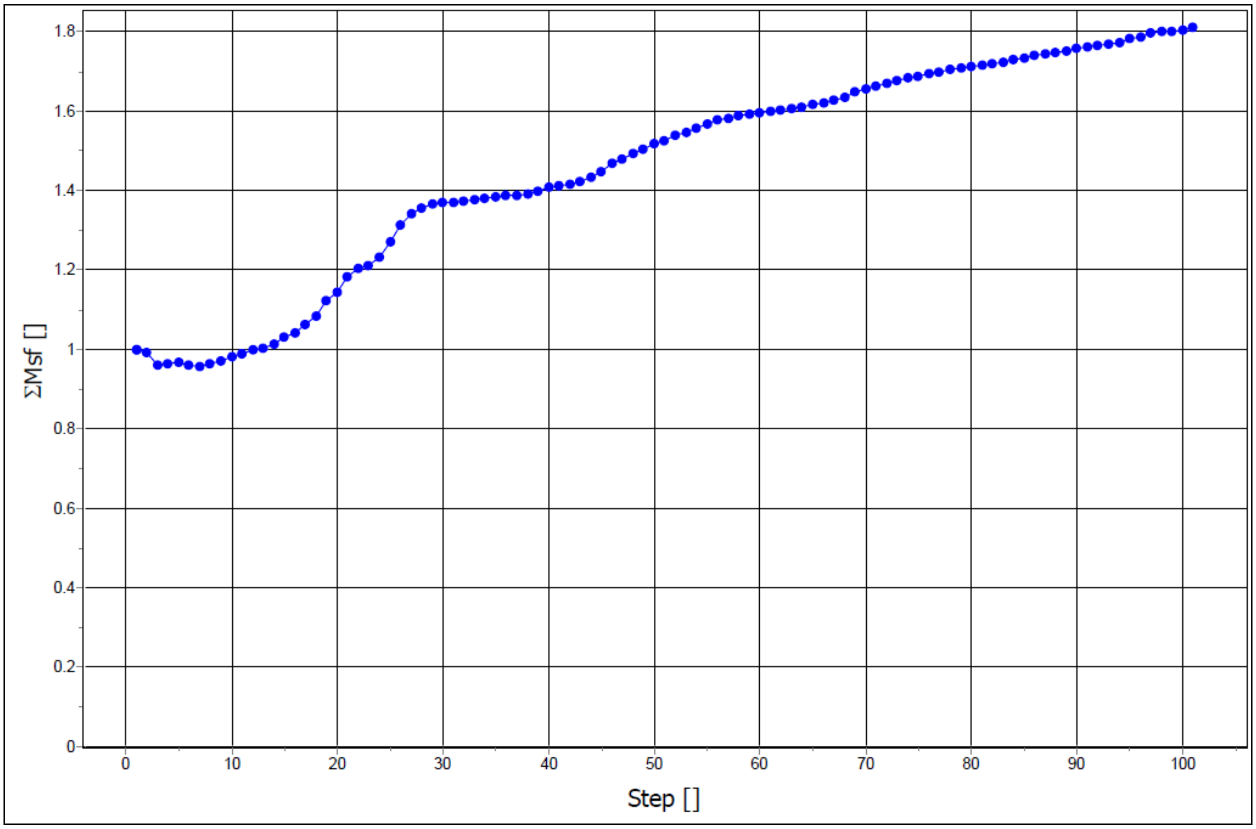

[*10^-3 m]
68.00
64.00
60.00
56.00
52.00
48.00
44.00
40.00
36.00
32.00
28.00
24.00
20.00
16.00
12.00
8.00
4.00
0.00
Incremental displacements |Δu|
Maximum value = 0.06435 m (Element 417 at Node 3706)


[*10^-3 m]
22.00
20.00
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00



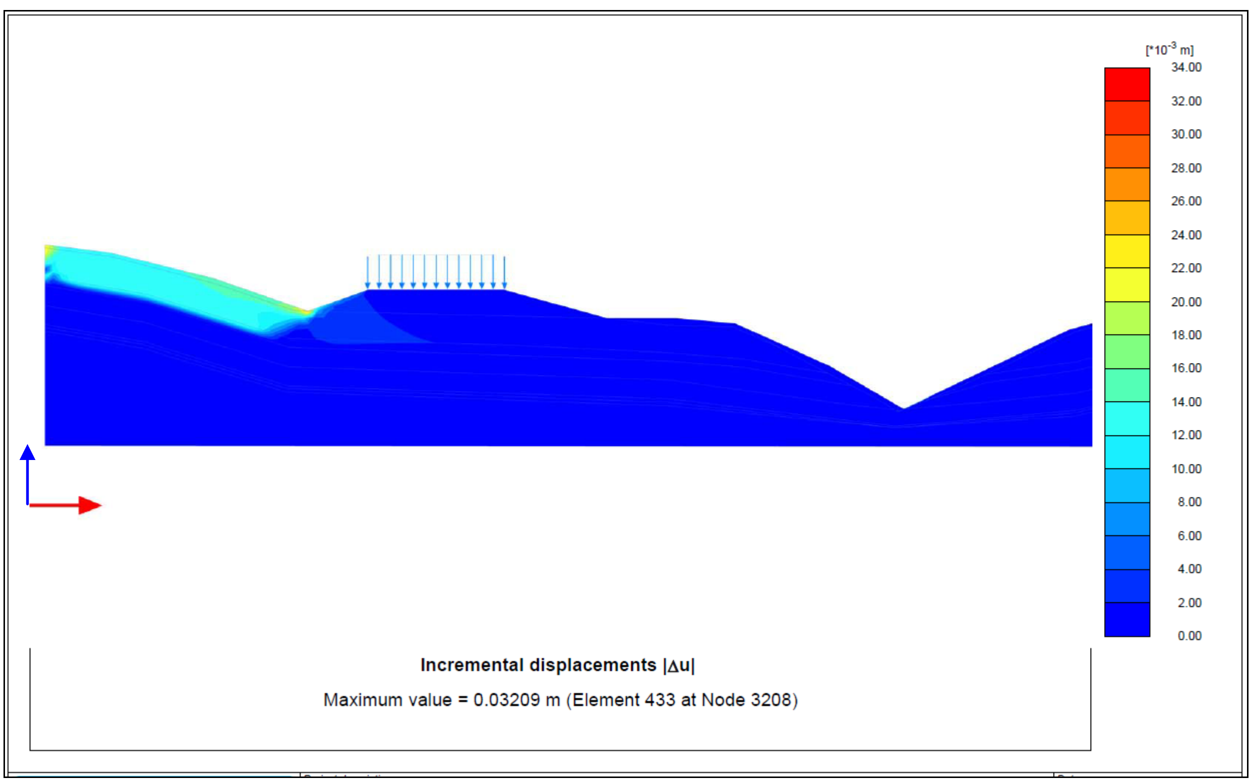
E1B Undrained



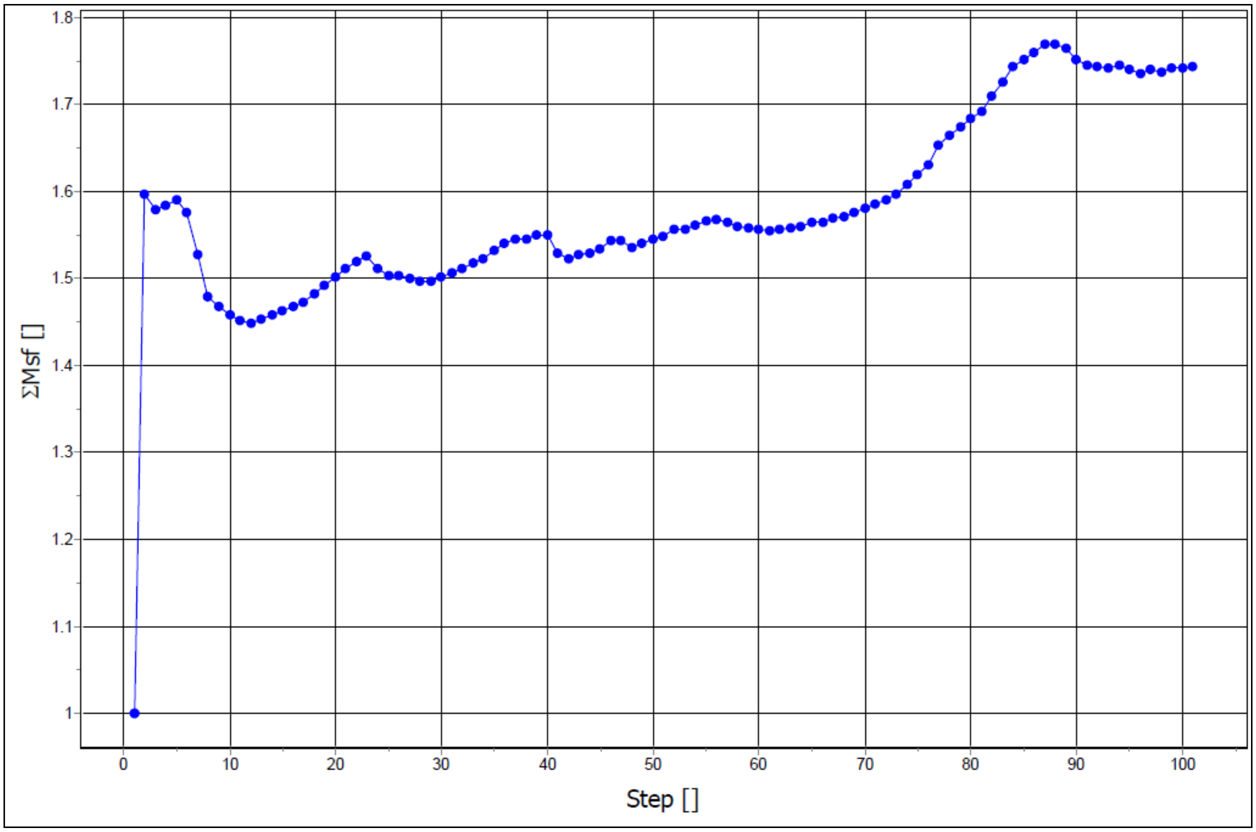


ΣMsf []
Step []

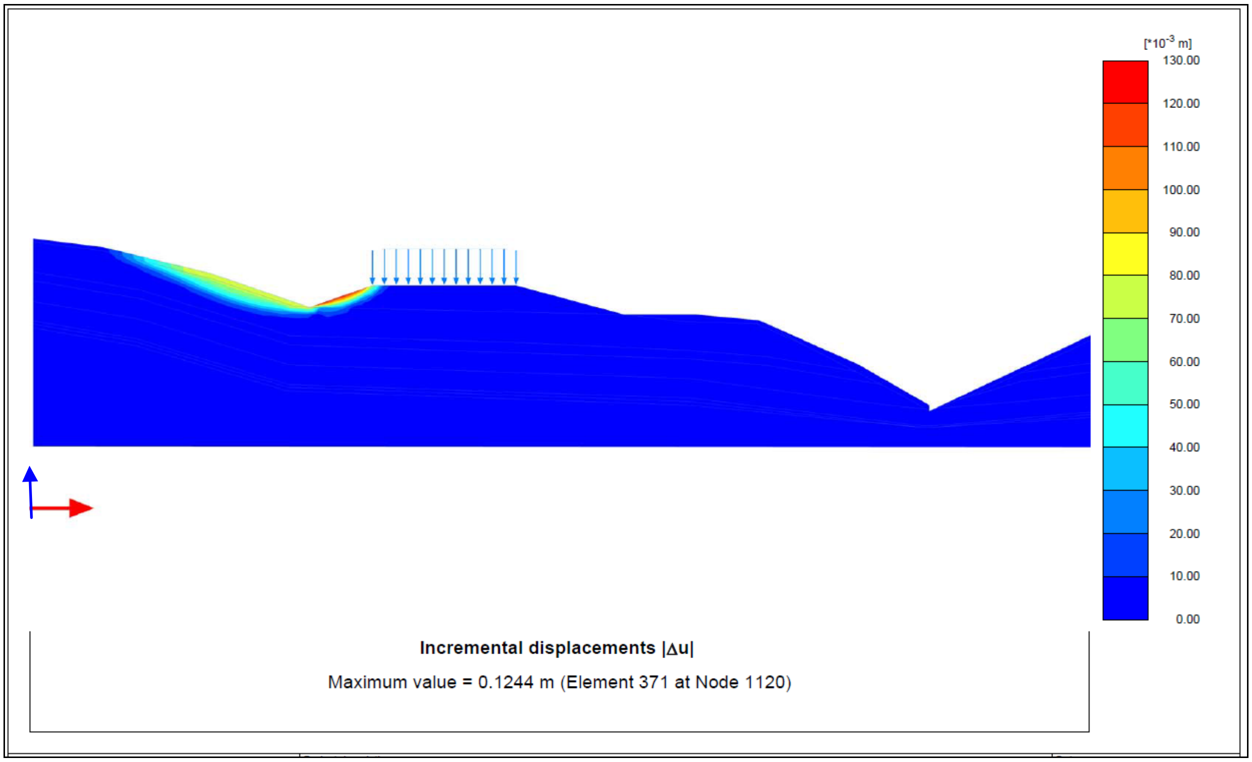
E2B Drained

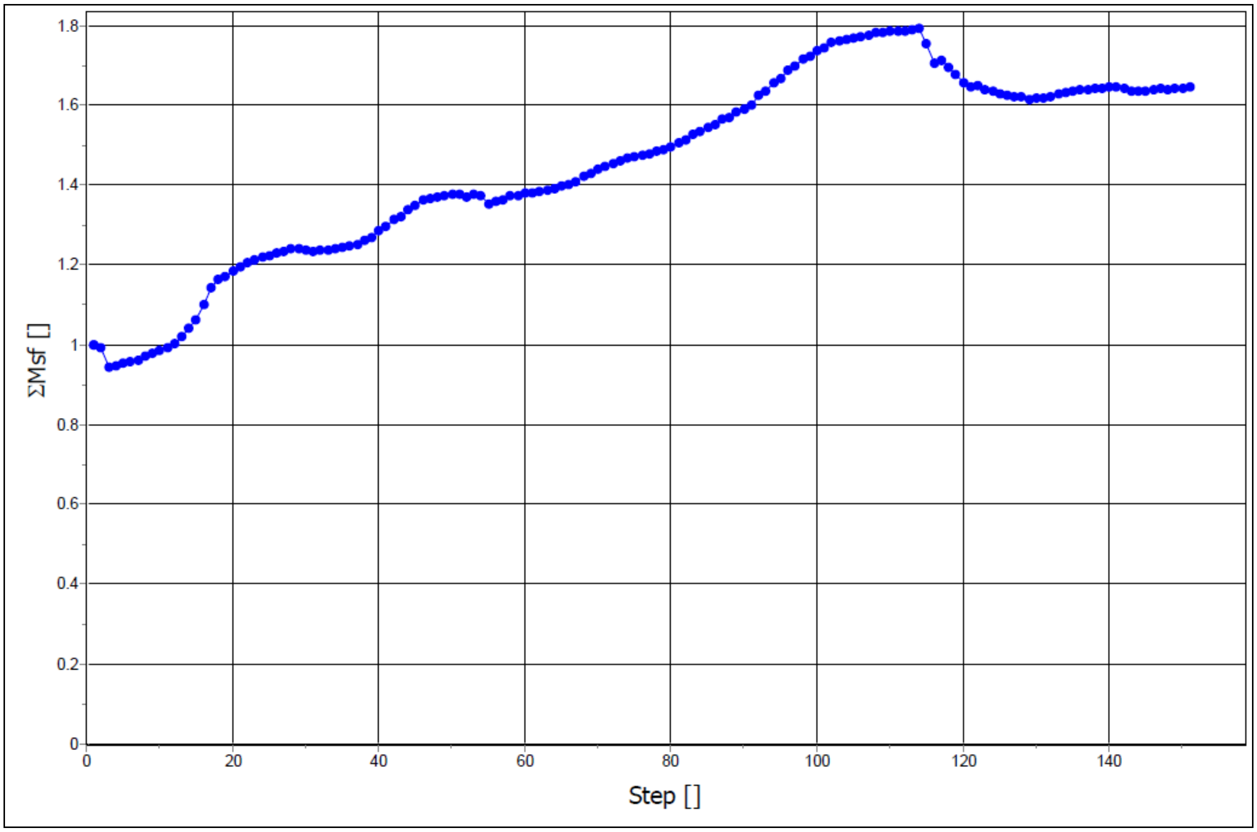


[*10^-3 m]
34.00
32.00
30.00
28.00
26.00
24.00
22.00
20.00
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
Incremental displacements |Δu|
Maximum value = 0.03209 m (Element 433 at Node 3208)

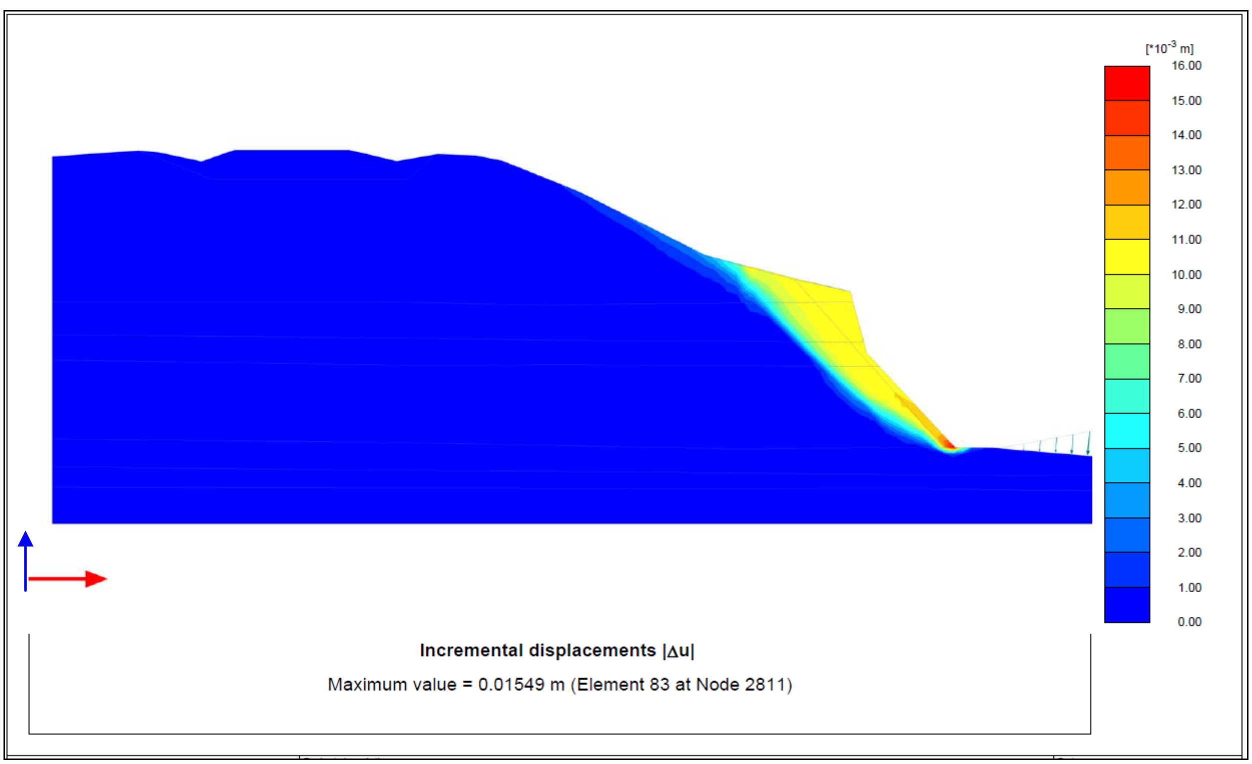


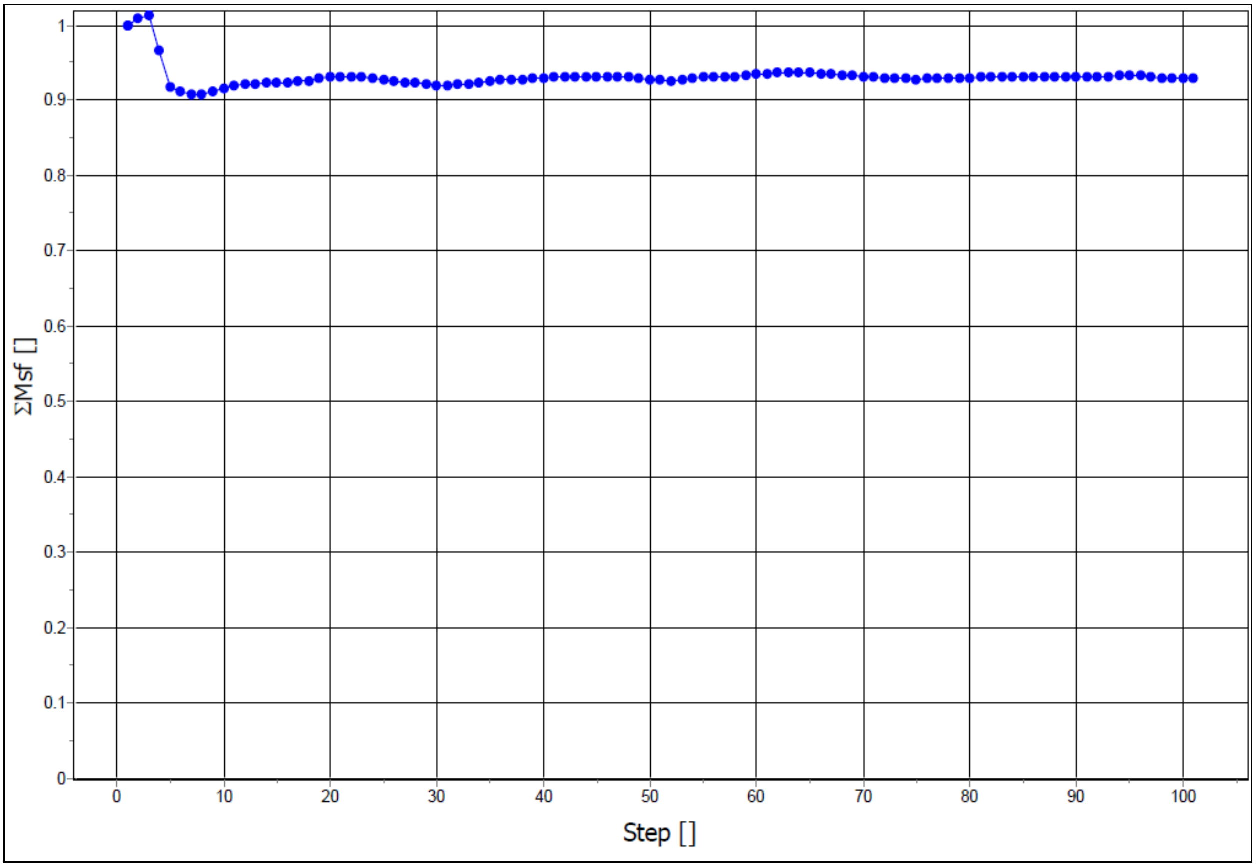
E2B Undrained

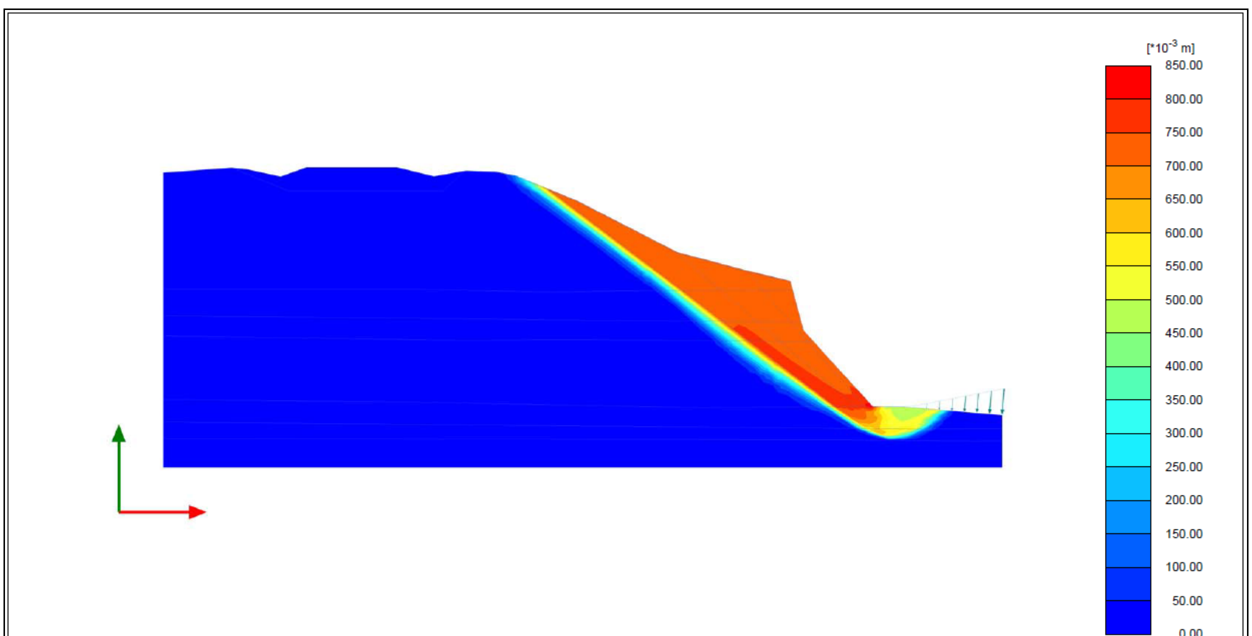



ΣMsf []
Step []
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
0
20
40
60
80
100
120
140

F1 Drained

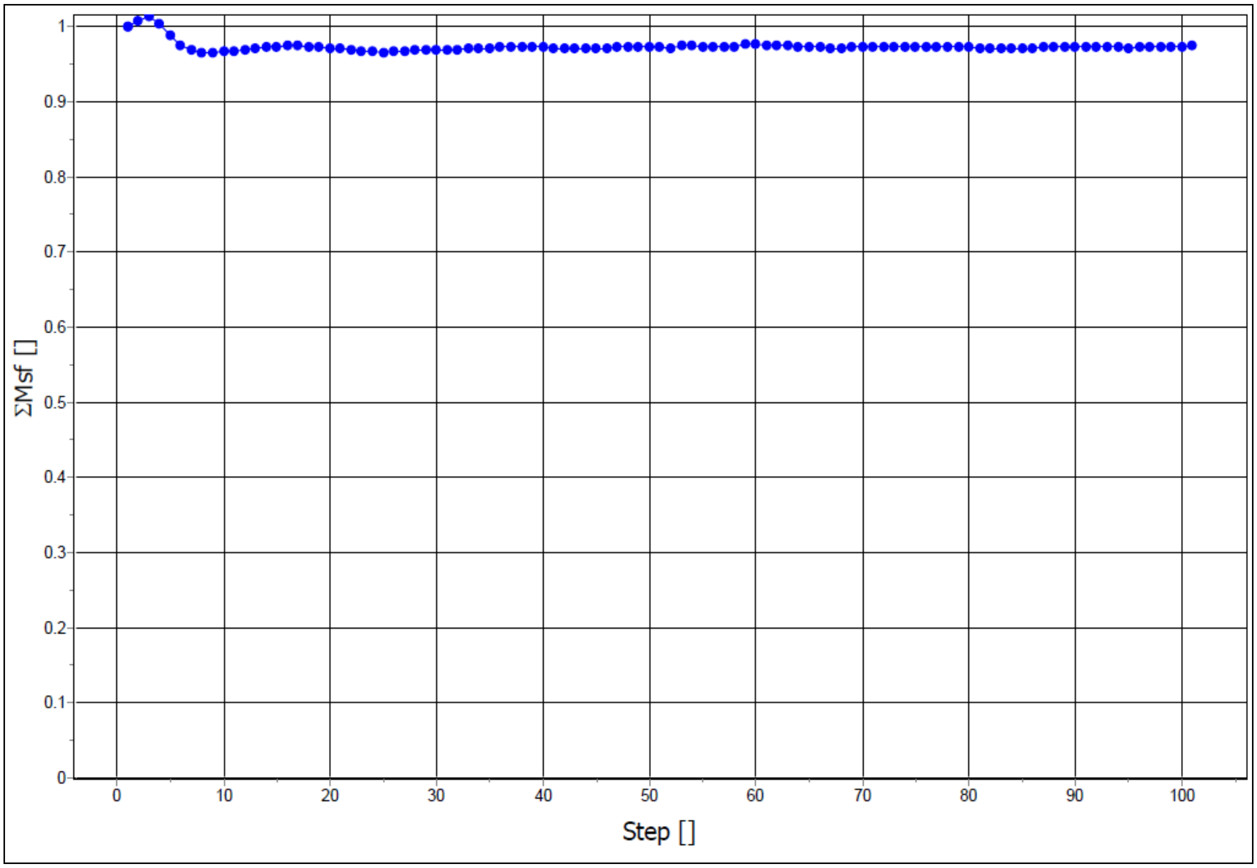

[*10^-3 m]
16.00
15.00
14.00
13.00
12.00
11.00
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
Incremental displacements |Δu|
Maximum value = 0.01549 m (Element 83 at Node 2811)



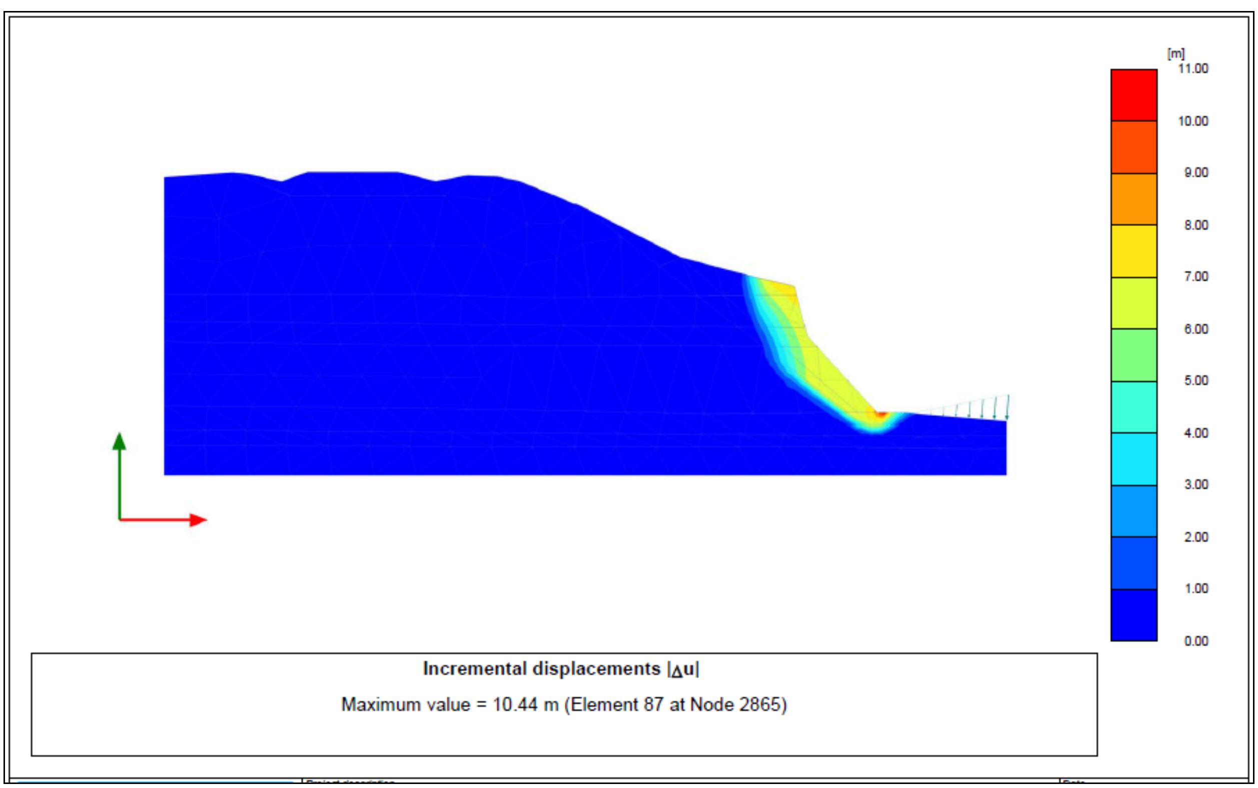


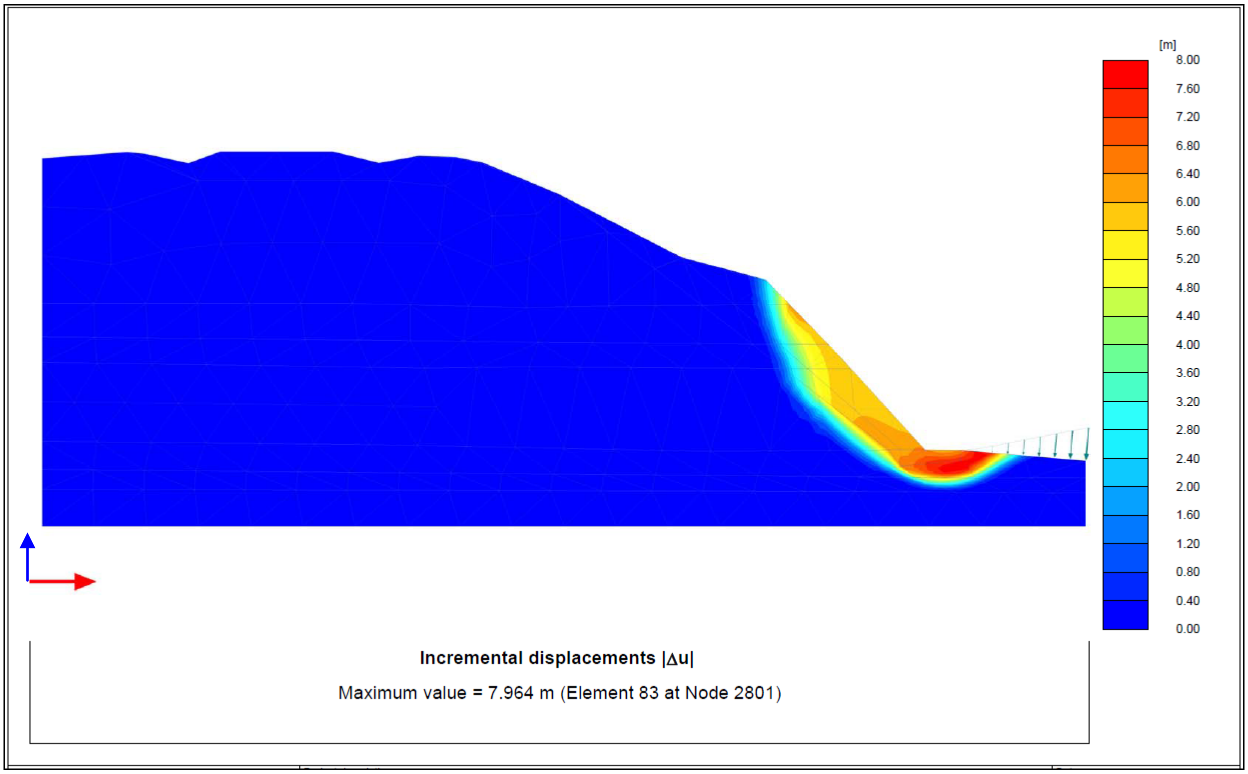
Incremental displacements |Δu|

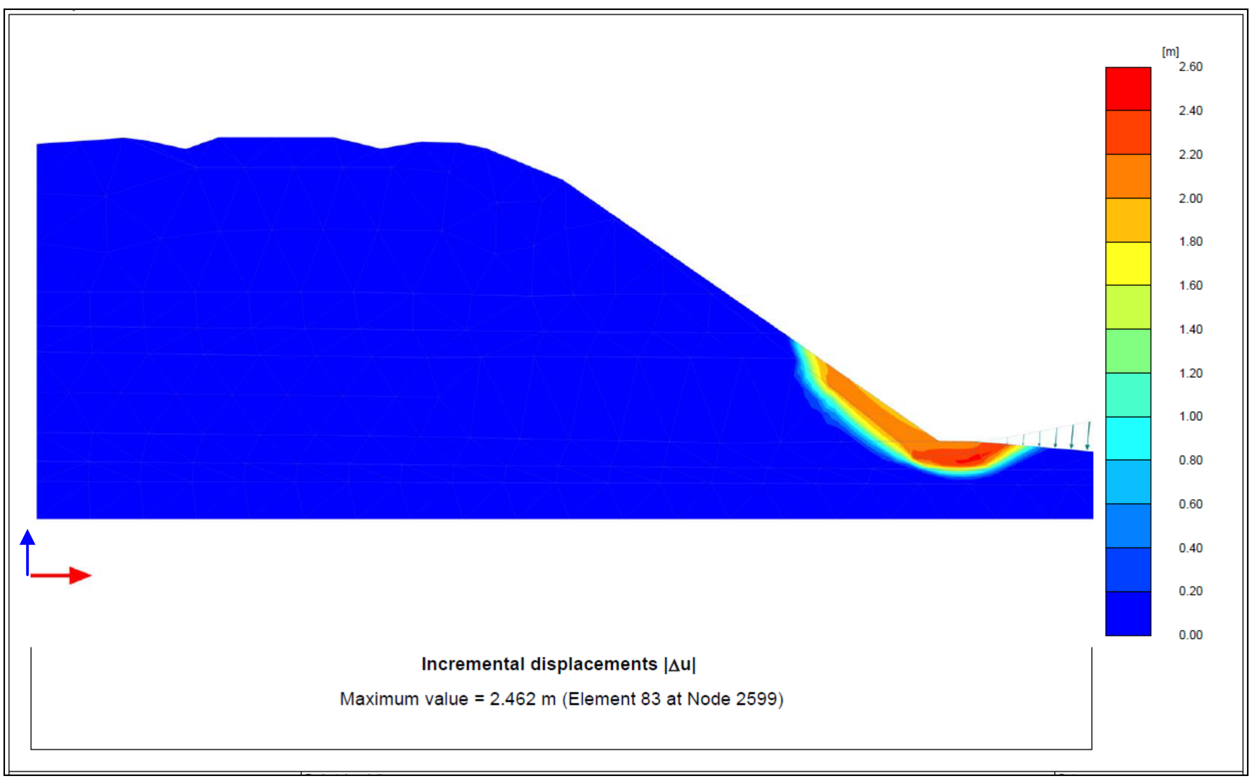
Maximum value = 0.8289 m (Element 83 at Node 2954)

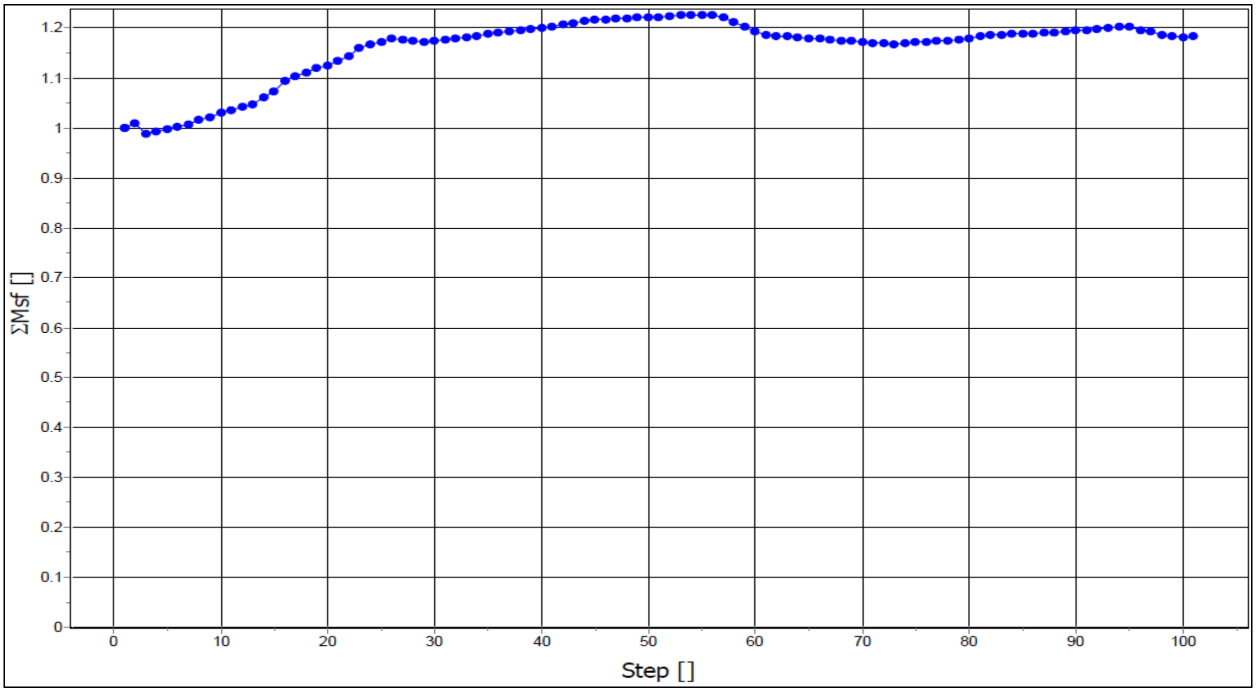


F1 Undrained



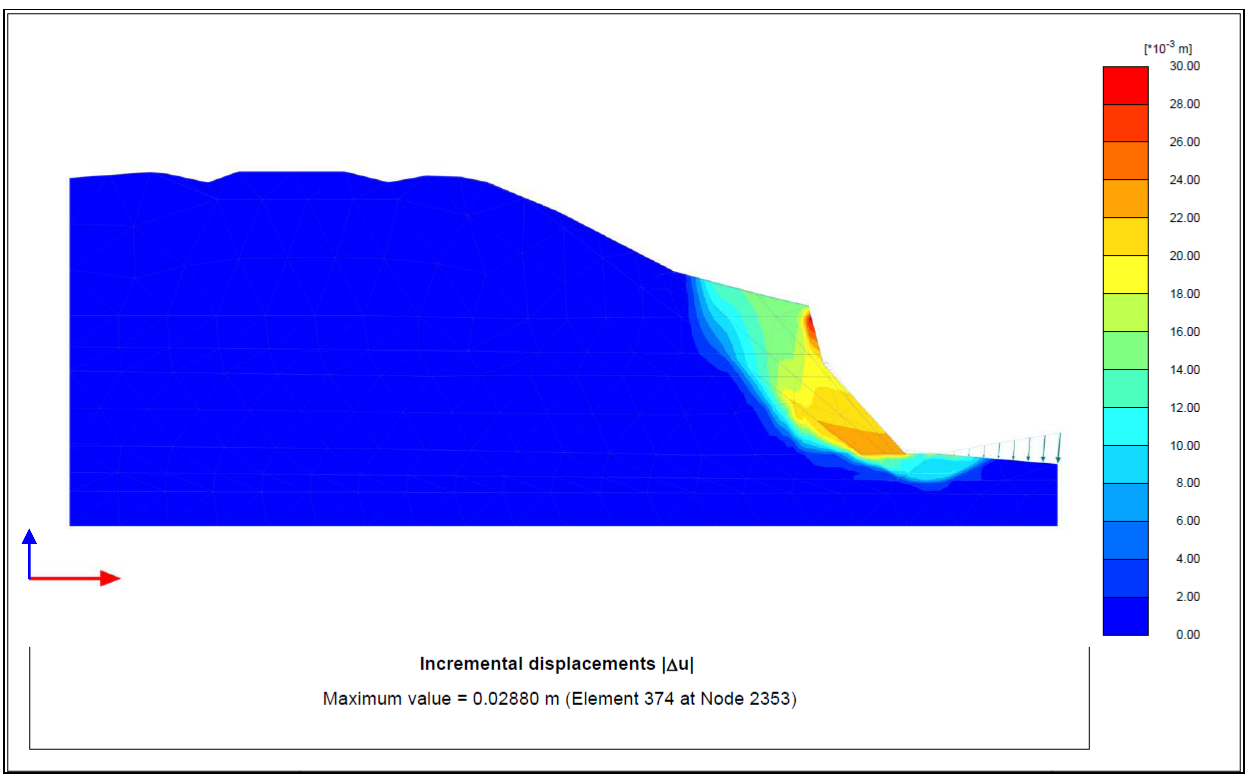

[m]
8.00
7.60
7.20
6.80
6.40
6.00
5.60
5.20
4.80
4.40
4.00
3.60
3.20
2.80
2.40
2.00
1.60
1.20
0.80
0.40
0.00
Incremental displacements |Δu|
Maximum value = 7.964 m (Element 83 at Node 2801)


[m]
2.60
2.40
2.20
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
Incremental displacements |Δu|
Maximum value = 2.462 m (Element 83 at Node 2599)

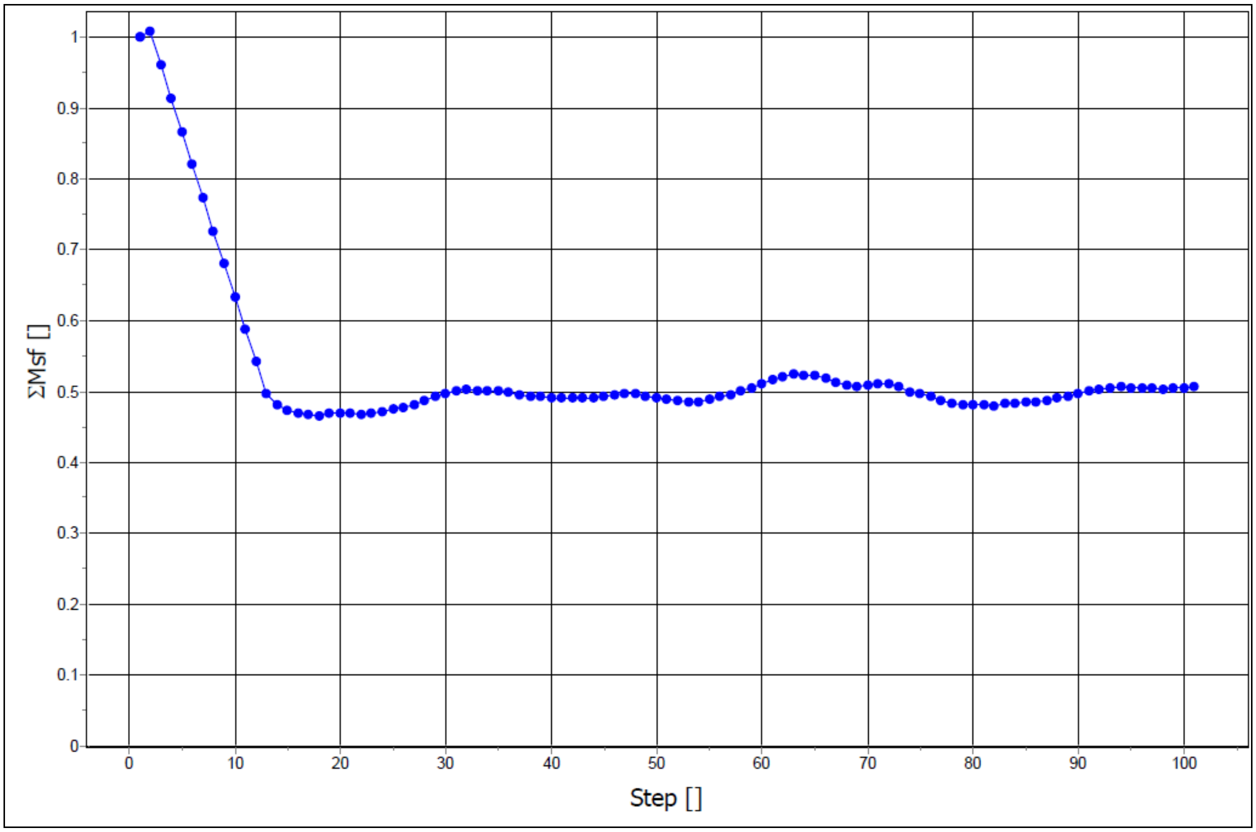


ΣMsf []
Step []

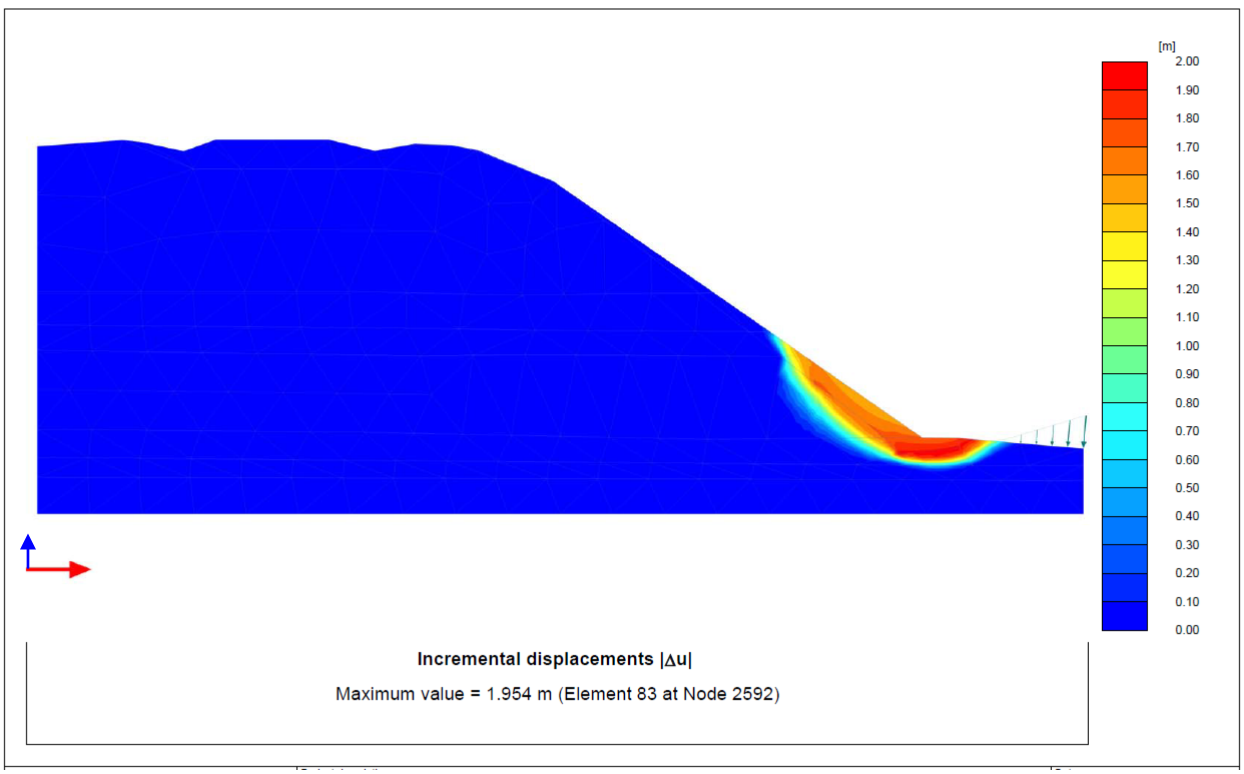
F2 Drained

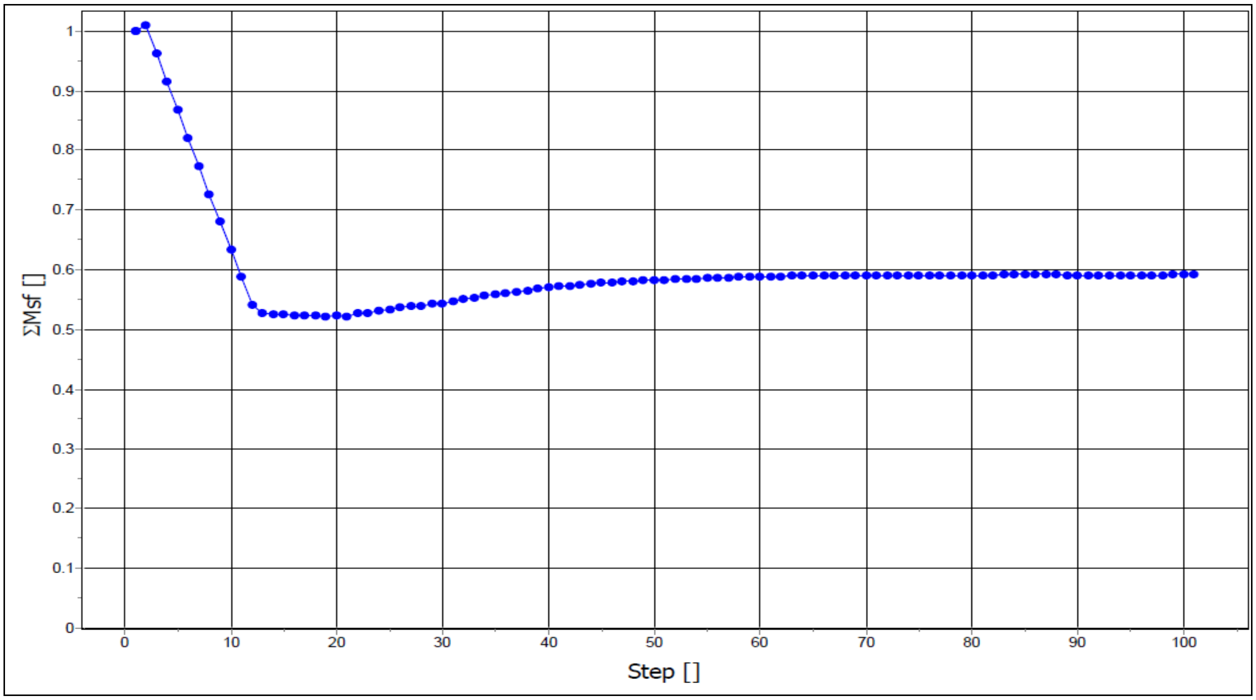


[*10^-3 m]
30.00
28.00
26.00
24.00
22.00
20.00
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
Incremental displacements |Δu|
Maximum value = 0.02880 m (Element 374 at Node 2353)

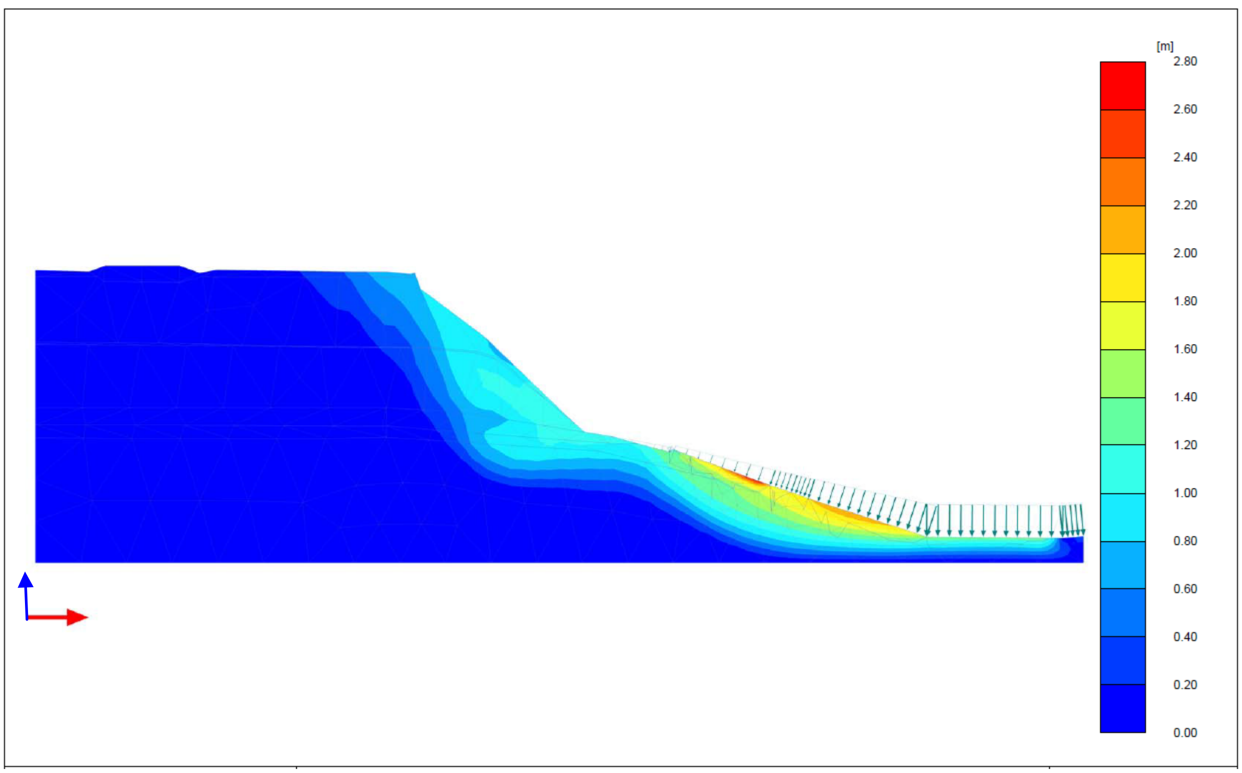


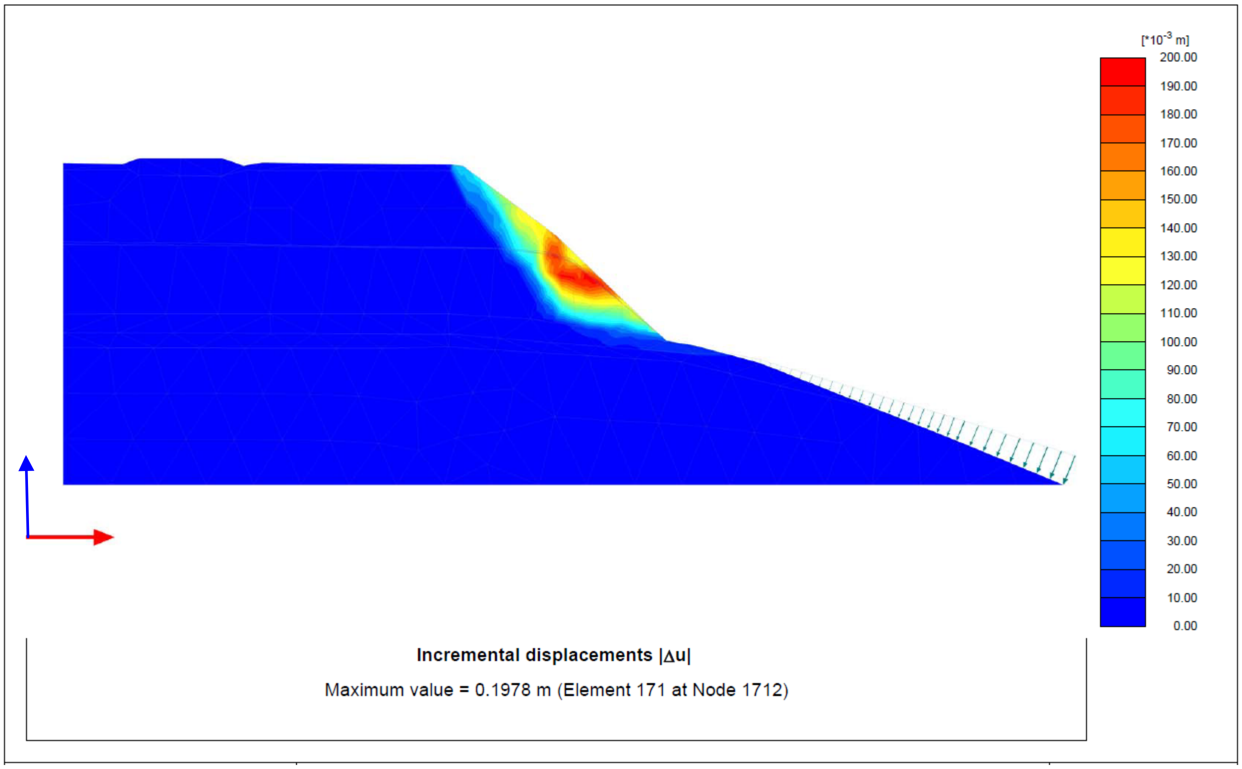
F2 Undrained



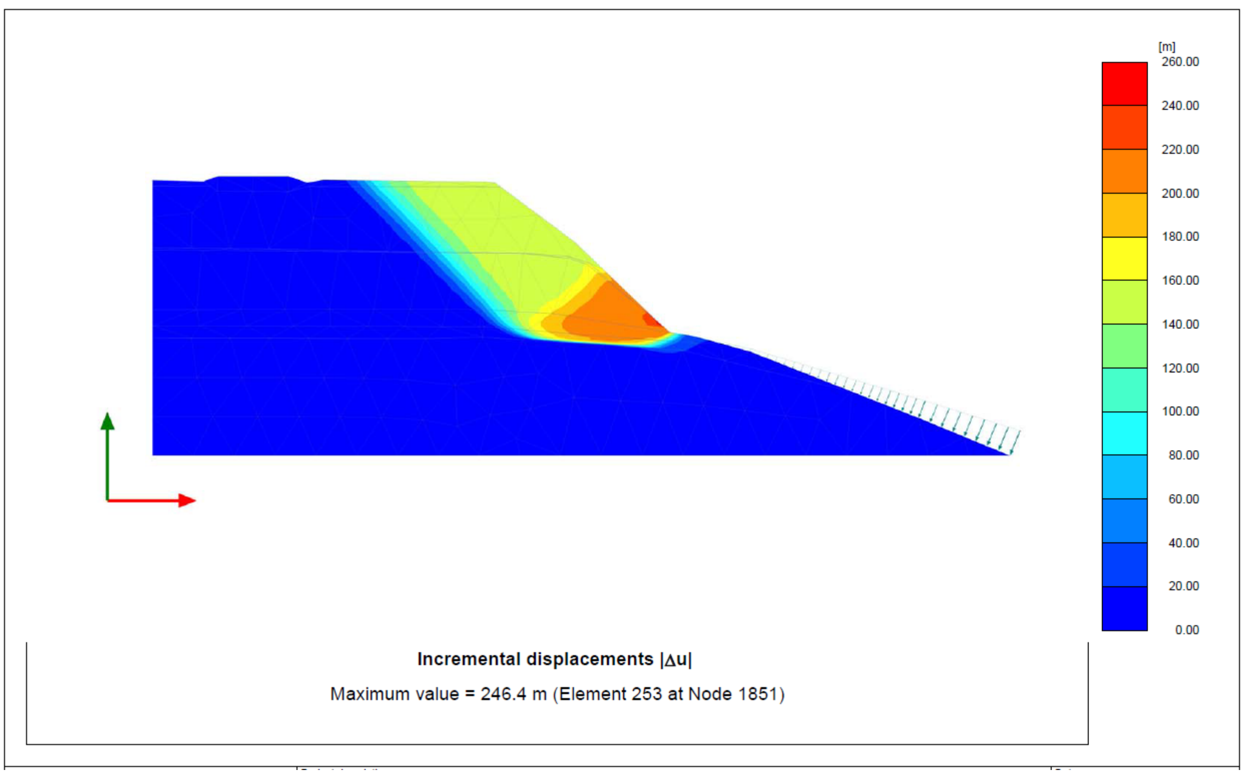

ΣMsf []
Step []
0
10
20
30
40
50
60
70
80
90
100
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1

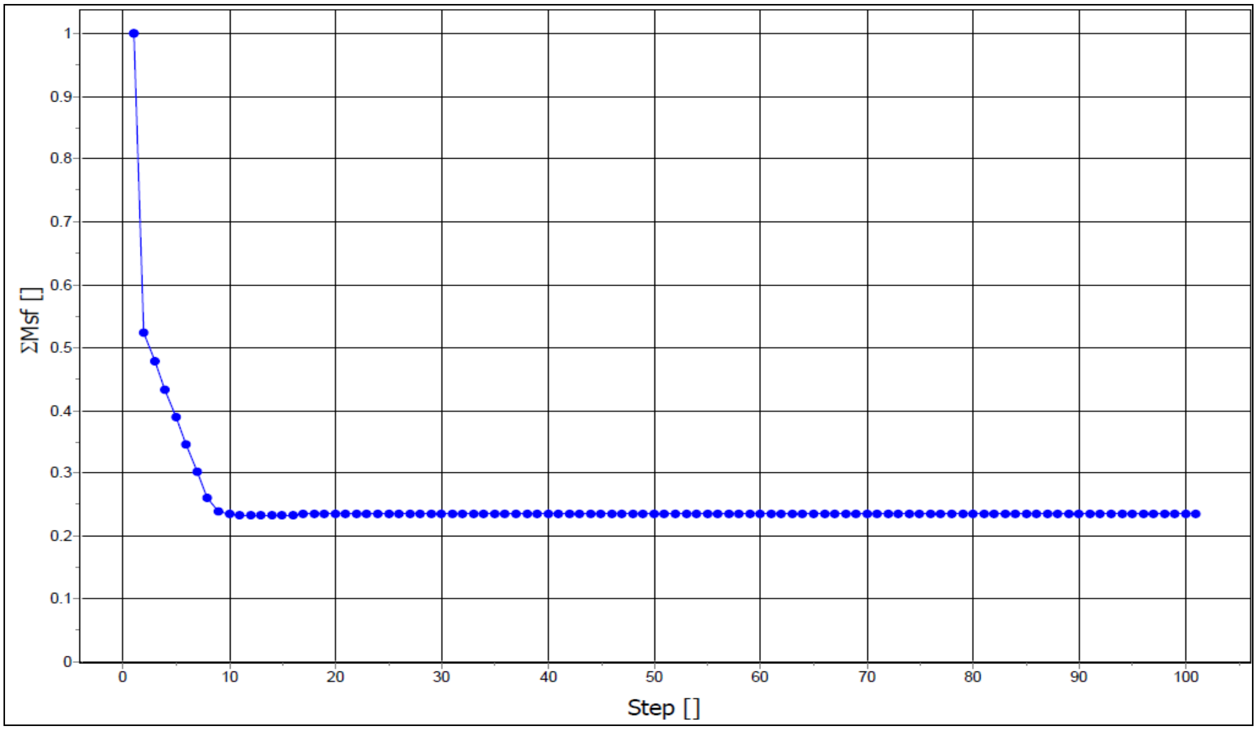
N1 Drained


[m]
2.80
2.60
2.40
2.20
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00



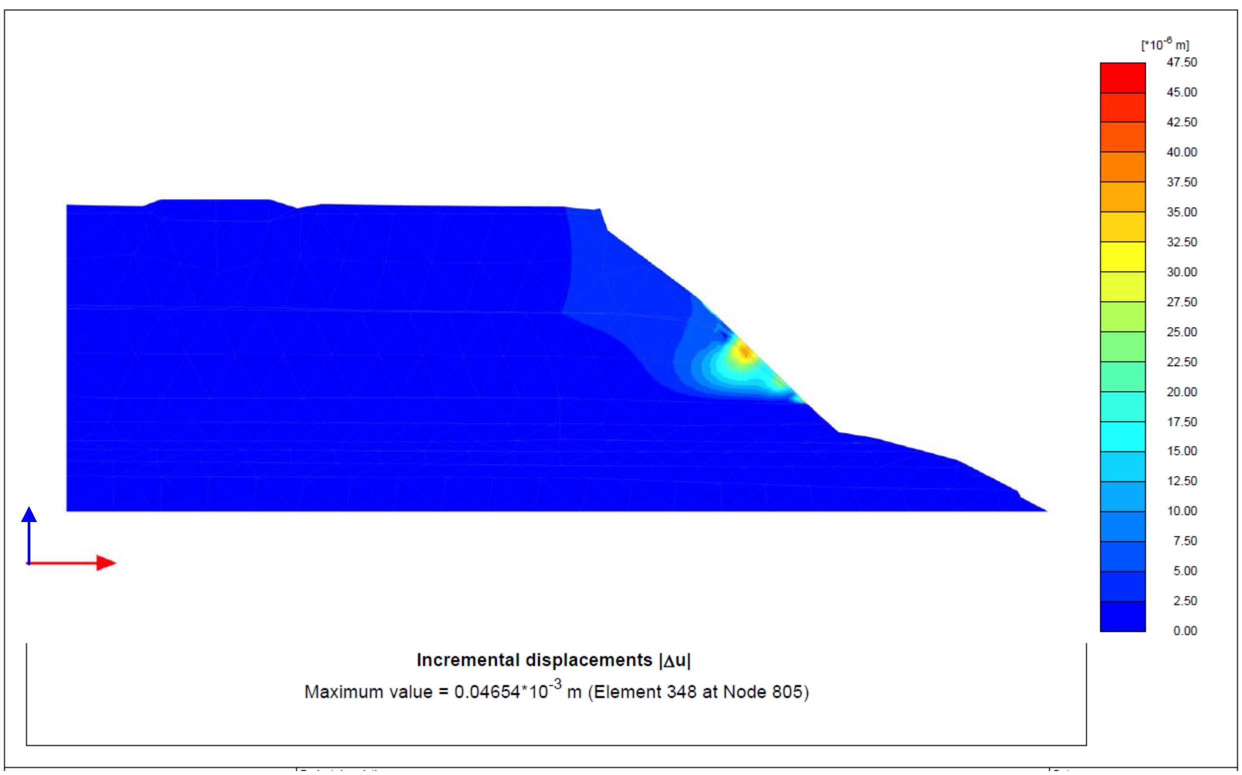
N1 Undrained



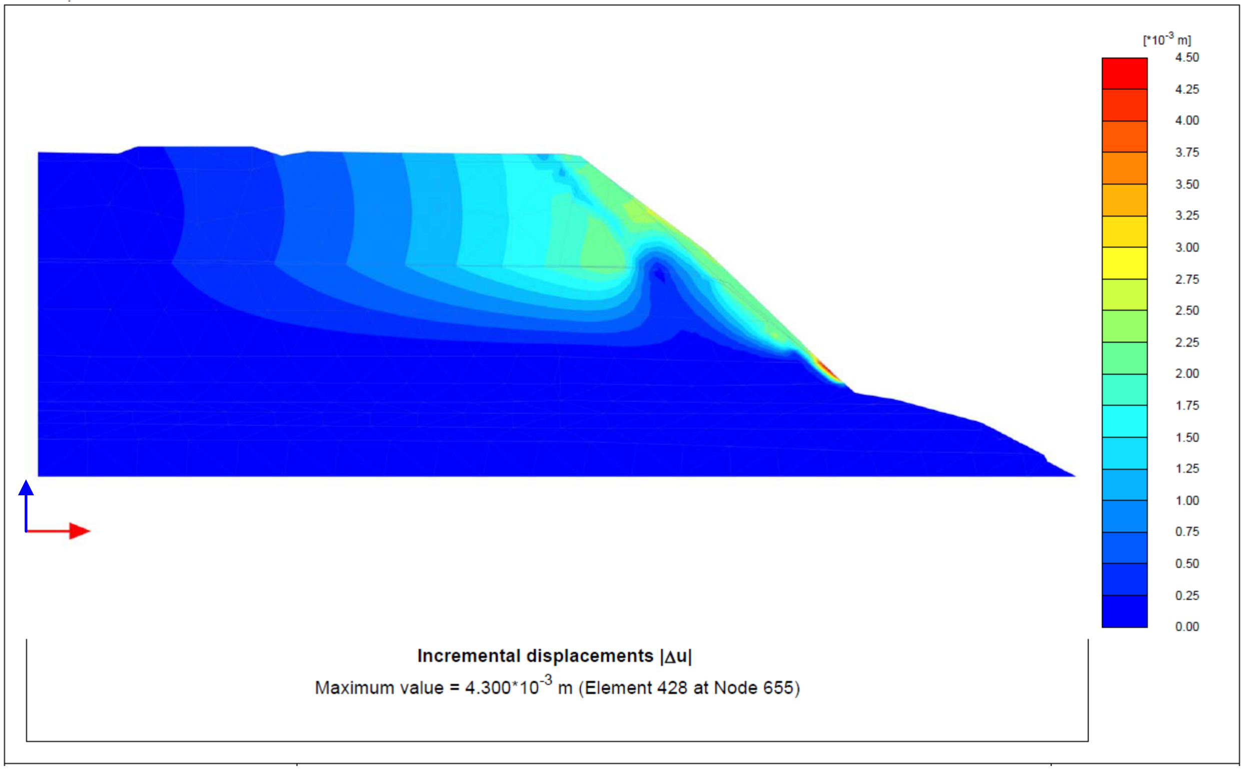


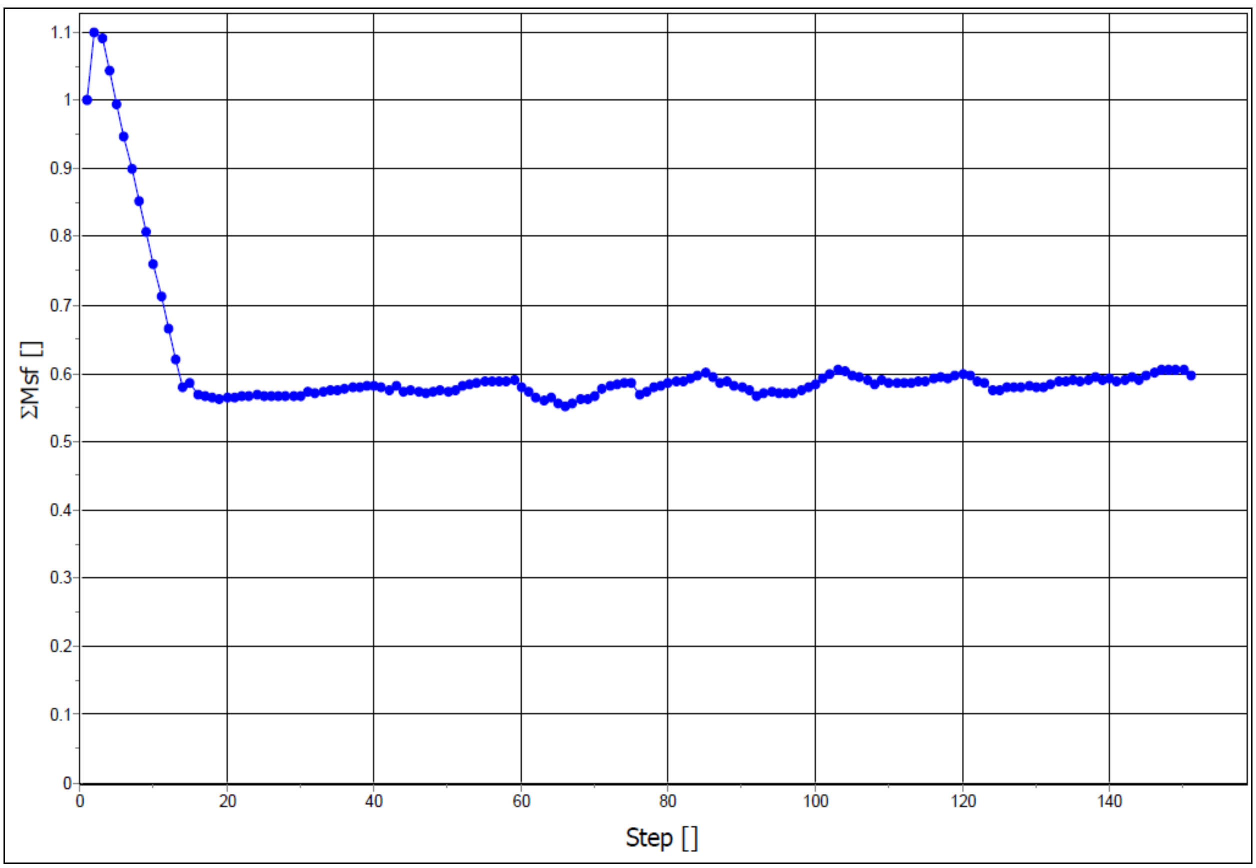
ΣMsf []
Step []

N1b Drained

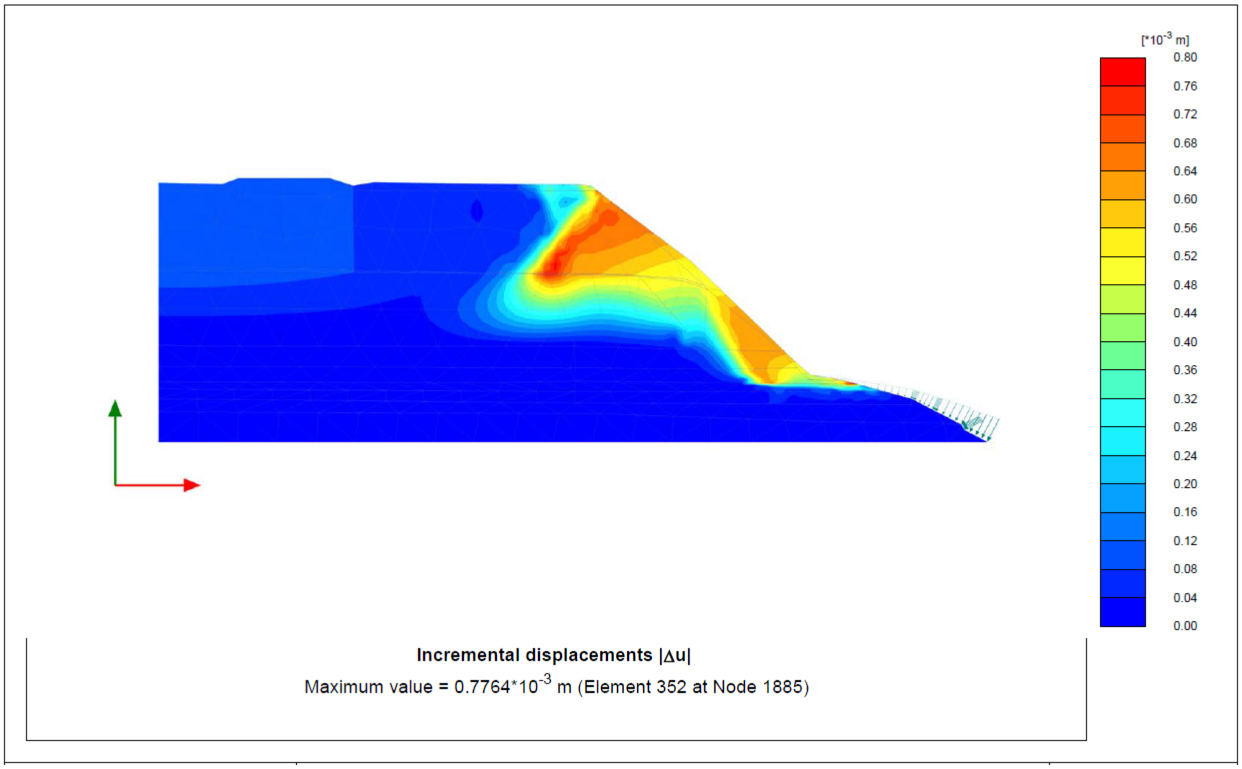


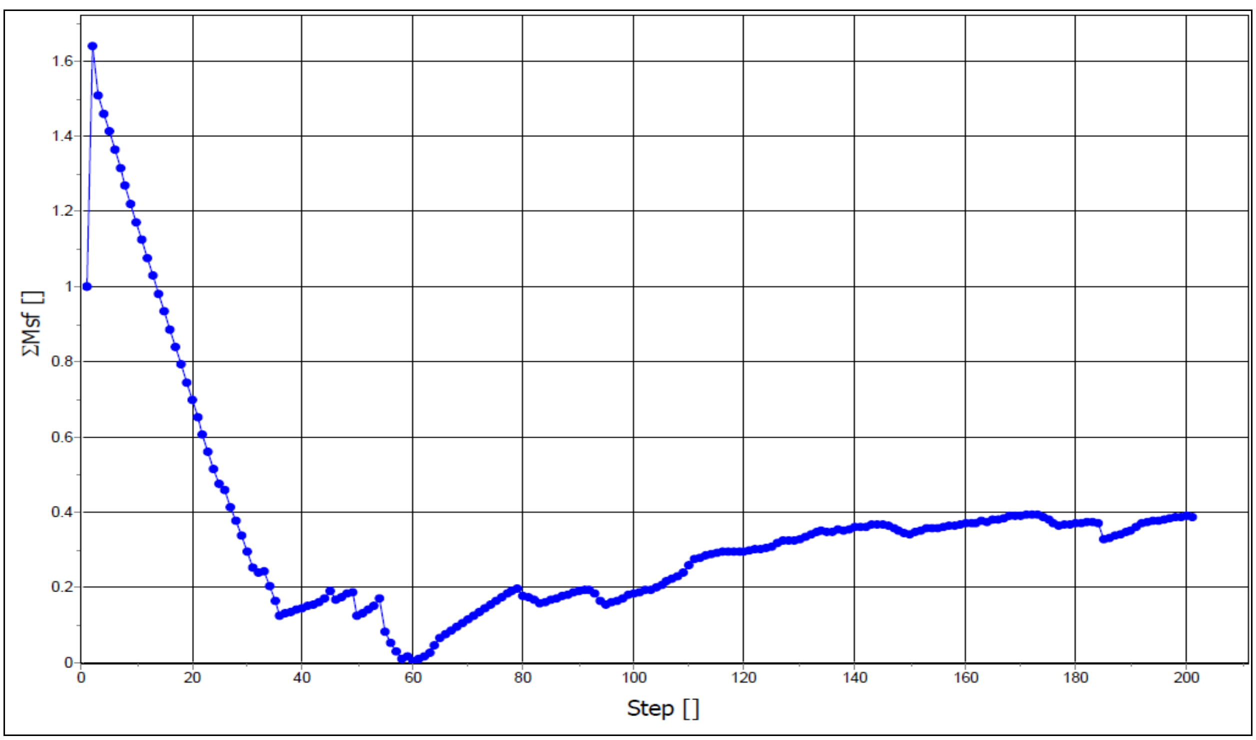
[*10^-6 m]
47.50
45.00
42.50
40.00
37.50
35.00
32.50
30.00
27.50
25.00
22.50
20.00
17.50
15.00
12.50
10.00
7.50
5.00
2.50
0.00
Incremental displacements |Δu|
Maximum value = 0.04654*10^-3 m (Element 348 at Node 805)


[*10^-3 m]
4.50
4.25
4.00
3.75
3.50
3.25
3.00
2.75
2.50
2.25
2.00
1.75
1.50
1.25
1.00
0.75
0.50
0.25
0.00
Incremental displacements |Δu|
Maximum value = 4.300*10^-3 m (Element 428 at Node 655)

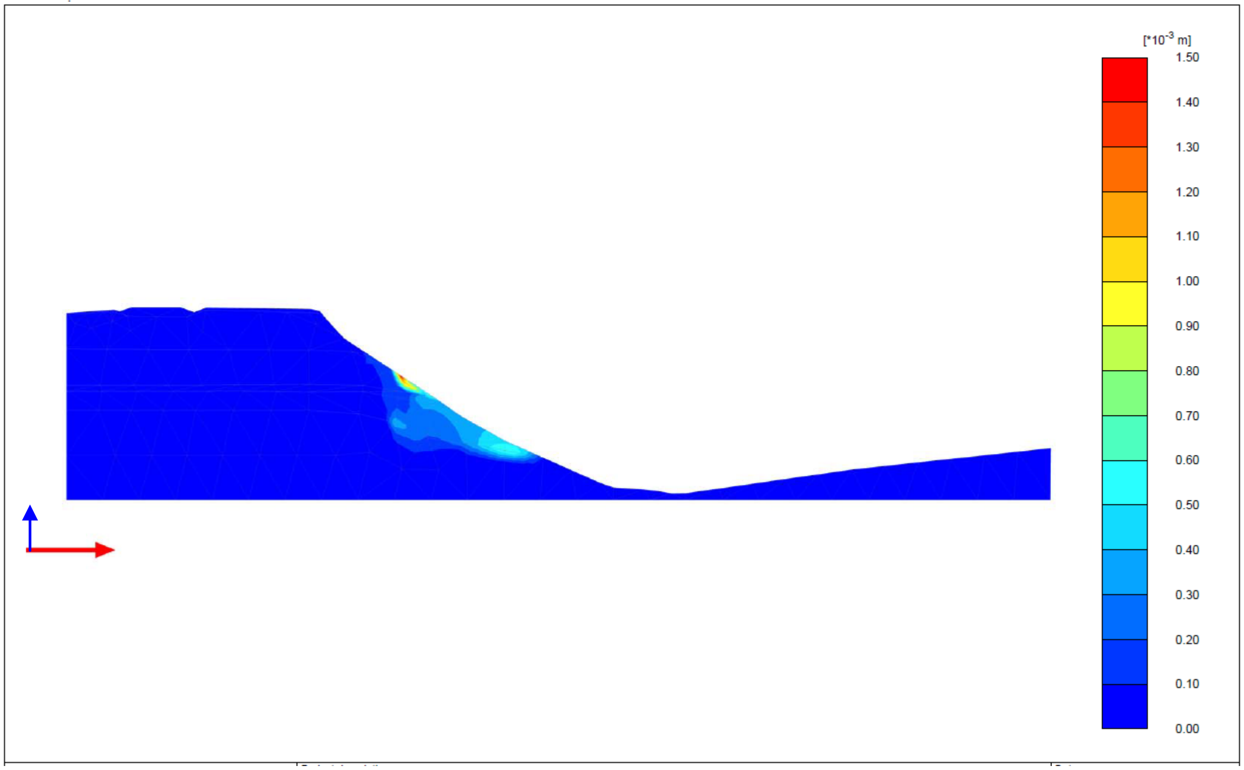

ΣMsf []
Step []

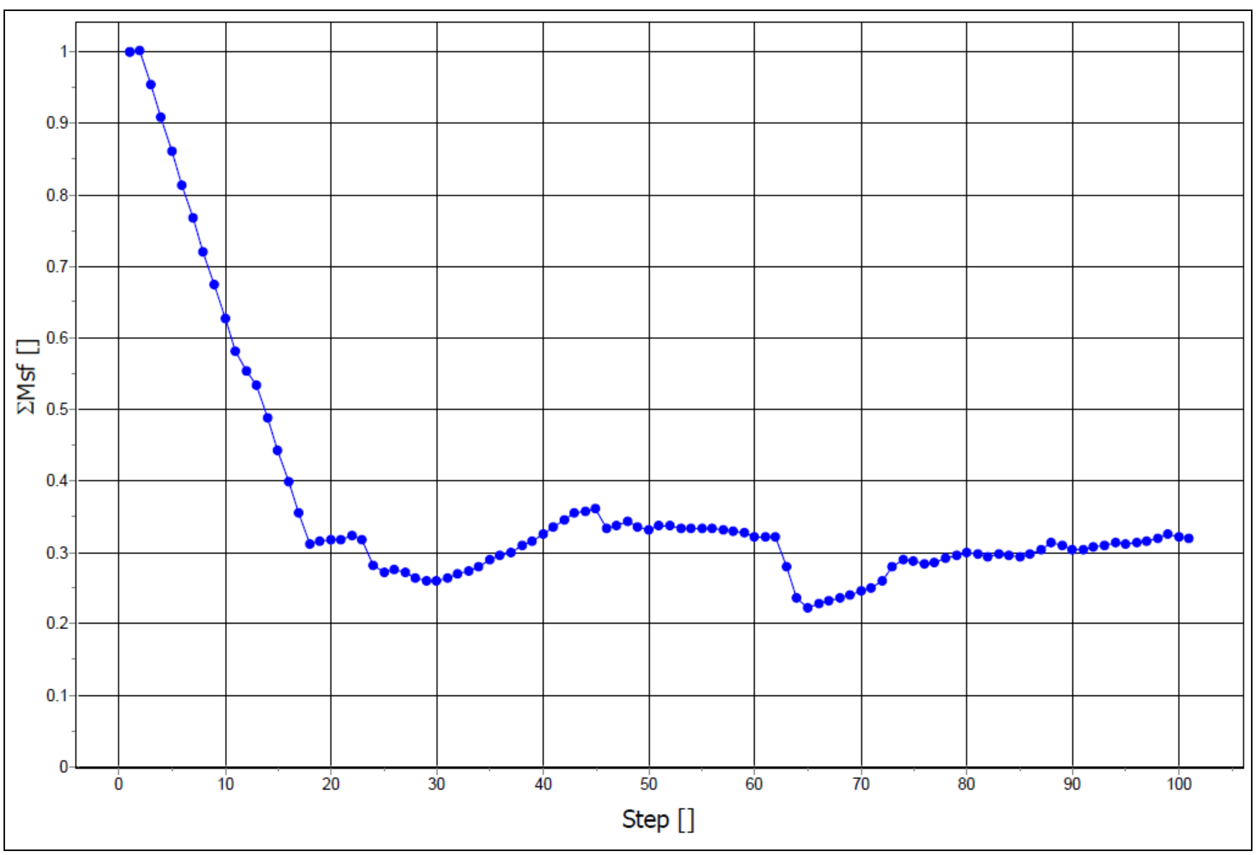
N1b Undrained

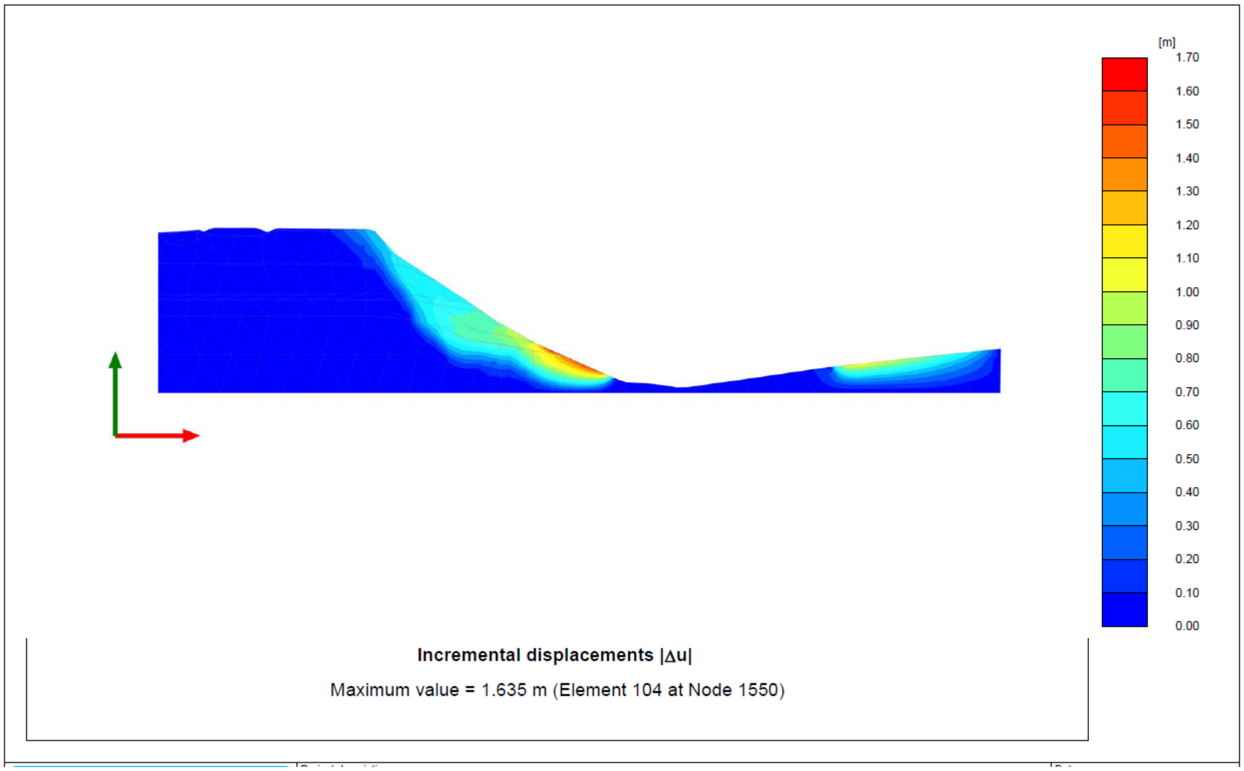




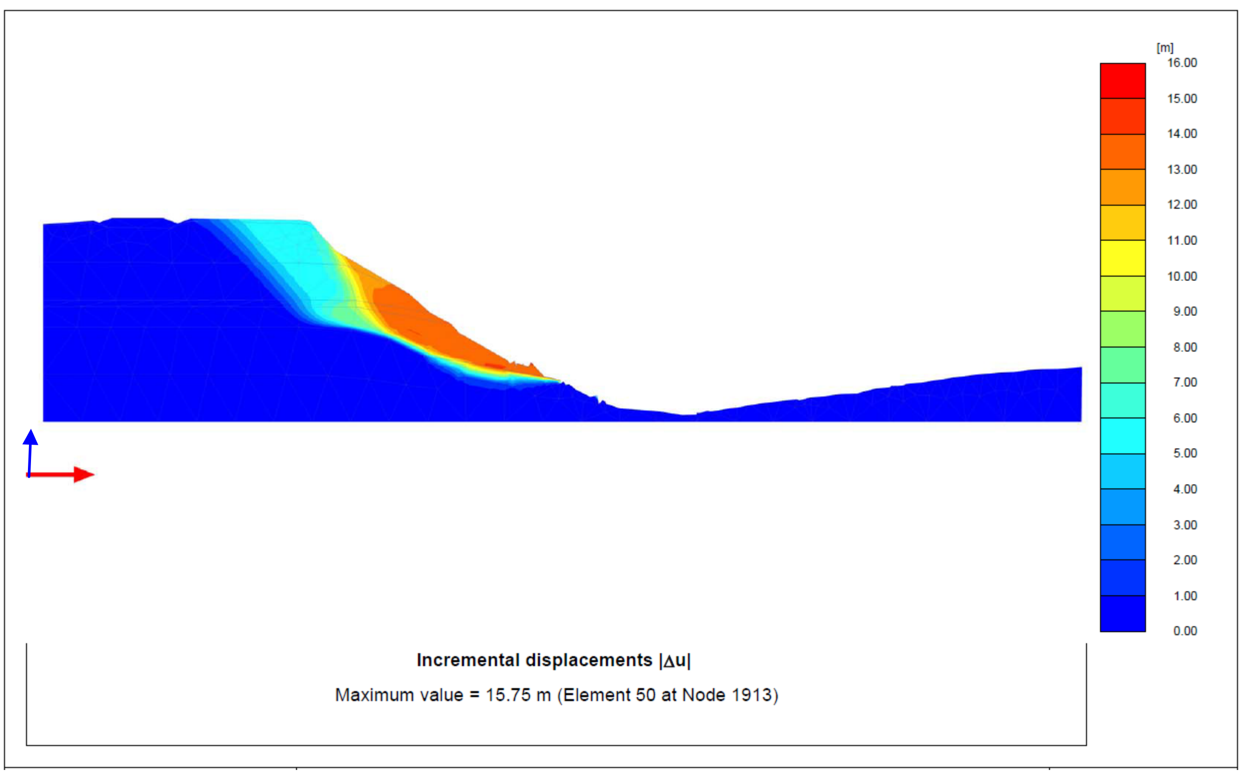
N2 Drained

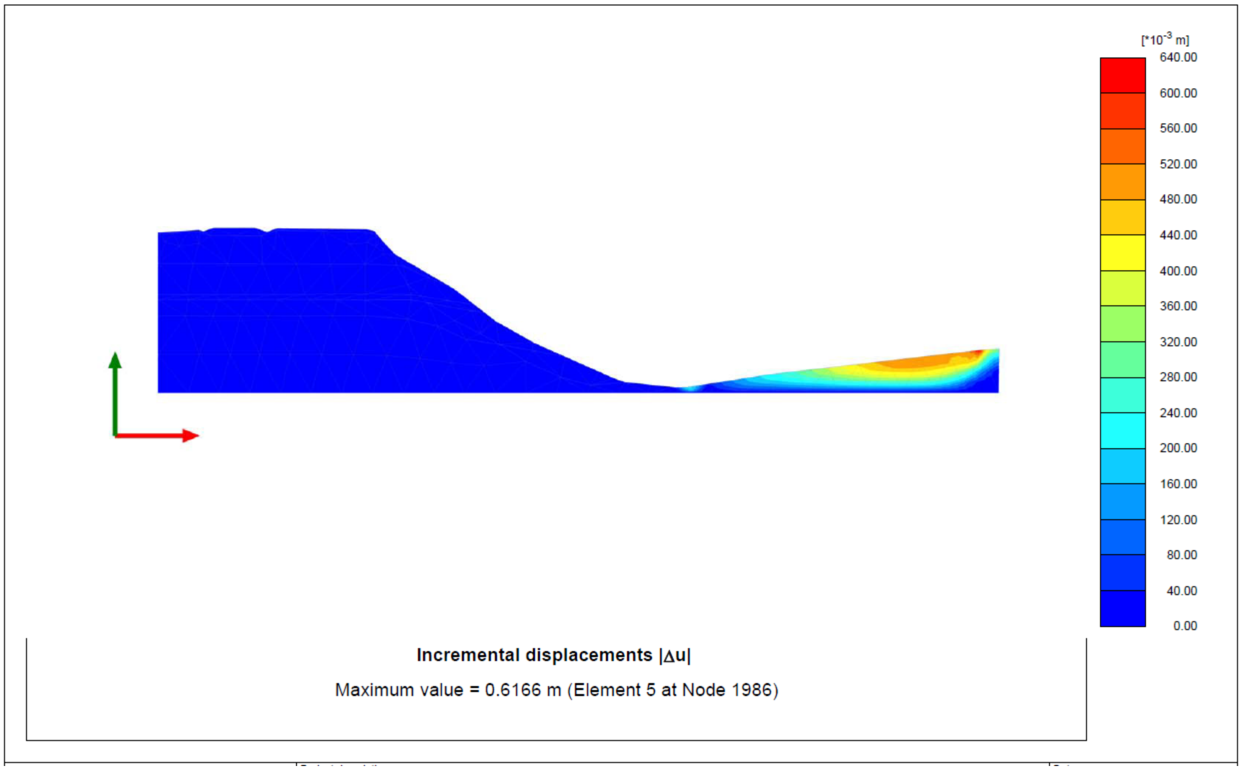

[*10^-3 m]
1.50
1.40
1.30
1.20
1.10
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00

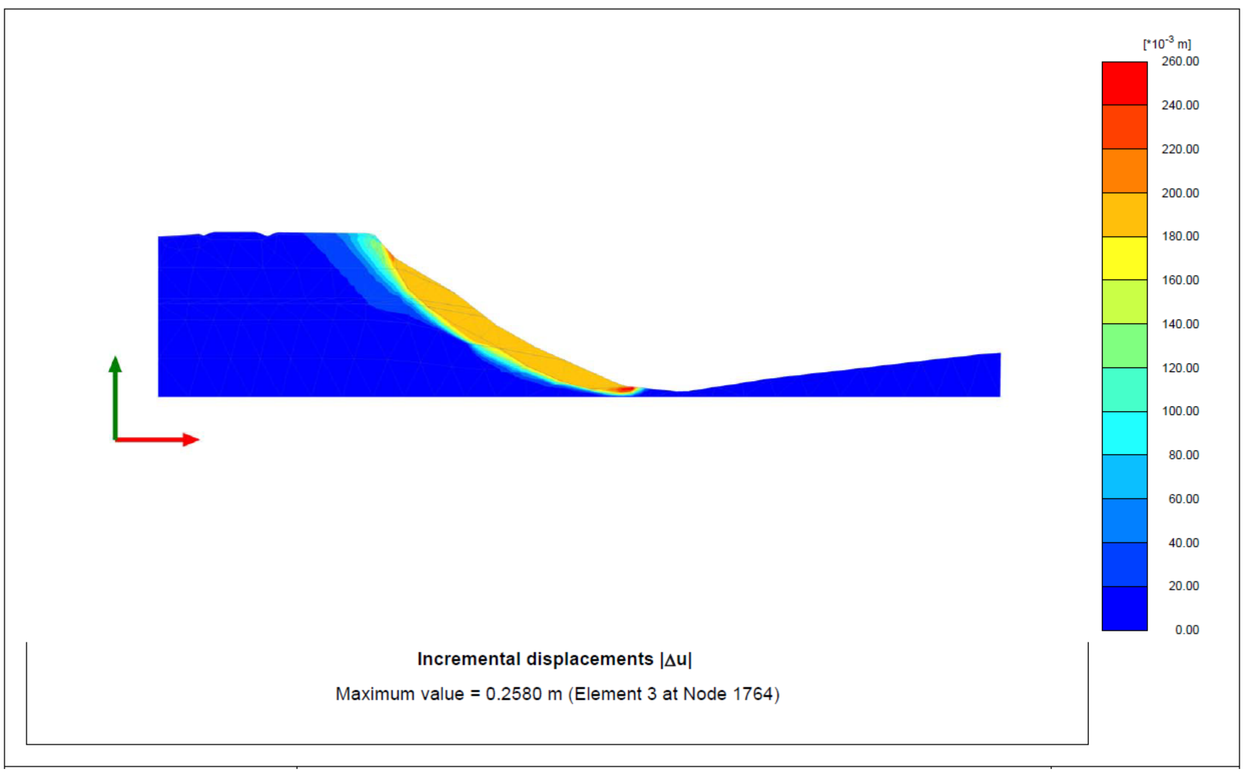

ΣMsf []
Step []
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
0
10
20
30
40
50
60
70
80
90
100

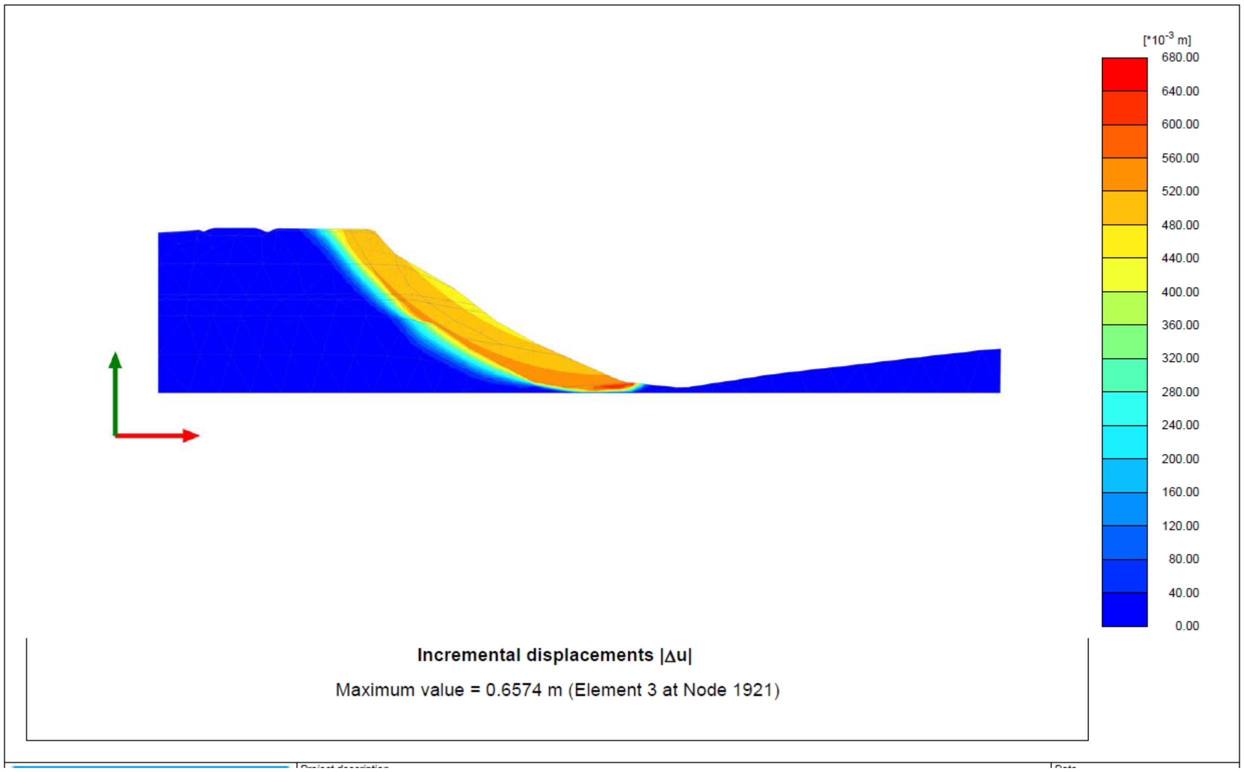


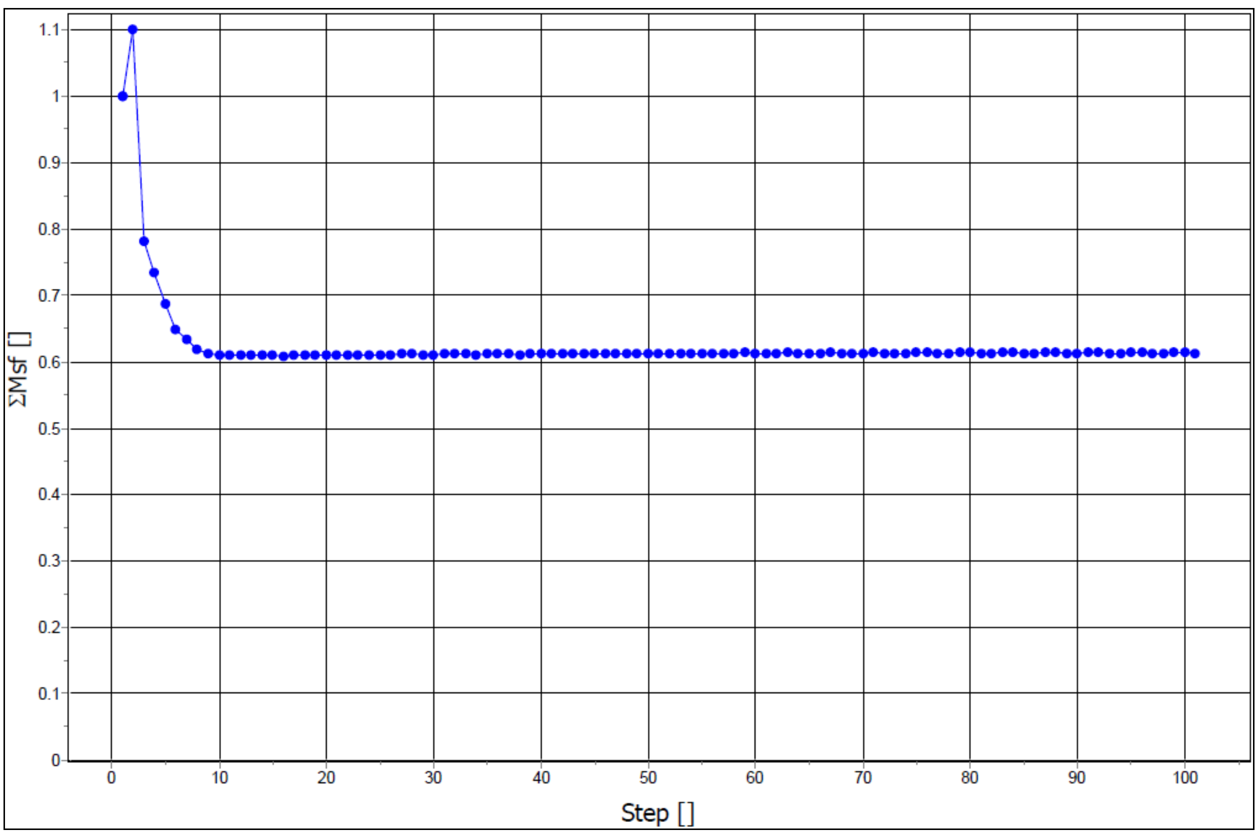
N2 Undrained




[*10^-3 m]
640.00
600.00
560.00
520.00
480.00
440.00
400.00
360.00
320.00
280.00
240.00
200.00
160.00
120.00
80.00
40.00
0.00
Incremental displacements |Δu|
Maximum value = 0.6166 m (Element 5 at Node 1986)


[*10^-3 m]
260.00
240.00
220.00
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
Incremental displacements |Δu|
Maximum value = 0.2580 m (Element 3 at Node 1764)


[*10^-3 m]
680.00
640.00
600.00
560.00
520.00
480.00
440.00
400.00
360.00
320.00
280.00
240.00
200.00
160.00
120.00
80.00
40.00
0.00
Incremental displacements |Δu|
Maximum value = 0.6574 m (Element 3 at Node 1921)


ΣMsf []
Step []
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
0
10
20
30
40
50
60
70
80
90
100

Appendix C

Symbol for different soil type

Rock and soil

Main term		Modifier		Layer	
B	rock				
Bl	boulders	bl	boulder-bearing		
Br	fragmented rock				
Cs	suspected contamina-tion	cs	local contamina-tion(routine field evaluation)	cs	contaminated layer
Dy	dy	dy	dy-bearing	dy	dy layer
F	fill, refuse, man-made soil				
Gr	gravel	gr	gravelly	gr	gravel layer
Gy	gyttja	gy	gyttja-bearing	gy	gyttja layer
Gy/Le	contact gyttja and clay (gyttja above, clay below)	( ) (sa)	somewhat, e.g. somewhat sandy	( )	thin layer
J	soil				
Le	clay	le	clayey	le	clay layer
Mn	till				
BlMn	boulder and cobble till				
StMn	cobble till				
GrMn	gravel till				
SaMn	sand till				
SiMn	silt till				
LeMn	clay till				
Mu	humus, topsoil	mu	humus-bearing	mu	humus layer
Sa	sand	sa	sandy	sa	sand layer
Si	silt	si	silty	si	silt layer
Sk	shells	sk	shell-bearing	sk	shell layer
Skgr	shell gravel				
Sksa	shell sand				
St	cobbles	st	cobble-bearing	st	cobble layer
Su	sulphide soil	su	sulphide-bearing	su	sulphide layer
T	peat			t	peat layer
Tl	fibrous peat				
Tm	pseudo-fibrous peat				
Th	amorphous peat				
Vx	plant (wood) remains	vx	containing plant remains	vx	layer of plant remains

T after main term, e.g. Let and Sit = dry crust of clay and silt

v varved, e.g. vLe = varved clay (the term should be reserved for glacial deposits)

The modifiers are placed before the main term. If there are several modifiers the name of the fraction which gives the soil its most characteristic properties is placed closest to the main term. The further the modifier is placed from the main term, the less the importance of the fraction in question. Layer designations are placed behind the main term. Example: sisaLesi = silty, sandy clay with silt layers. Mineral soils can be divided into the fractions fine, medium and coarse, respectively f, m and g, e.g. Saf = fine sand.

Berg och Jord

Nedanstående förslag baseras på Vägverkets och Banverkets (från 2010-04-01 Trafikverkets) översättningsnyckel från SGF:s beteckningssystem till beteckningar enligt SS-EN 14688-1.

Huvudord			*Tilläggsord*			*Skikt/lager*		
Ro	B	berg						
Bo	Bl	blockjord	bo	bl	blockig			
FrRo	Br	rösberg						
Dy	Dy	dy	dy	dy	dyig	dy	dy	dyskikt
Cs	Cs	Misstänkt förorenad jord enligt rutinbedömning i fält	cs	cs	lokalt förekommande föroreningar	cs	cs	föroreningar finns som tunnare skikt
Mg	F	fyllning						
Gy	Gy	gyttja	gy	gy	gyttjig	gy	gy	gyttjeskikt
Gy/Cl	Gy/Le	kontakt gyttja överst, lera underst	( )	()	något, t ex (sa) = något sandig	(_)	(_)	tunnare skikt
Gr	Gr	grus	gr	gr	grusig	gr	gr	grusskikt
So	J	jord						
Cl	Le	lera	cl	le	lerig	cl	le	lerskikt
Ti	Mn	morän						
BoTi	BlMn	block- och stenmorän						
CoTi	StMn	stenmorän						
GrTi	GrMn	grusmorän						
SaTi	SaMn	sandmorän						
SiTi	SiMn	siltmorän						
ClTi	LeMn	lermorän (moränlera)						
Hu	Mu	mulljord (mylla, matjord)	hu	mu	mullhaltig	hu	mu	mullskikt
Sa	Sa	sand	sa	sa	sandig	sa	sa	sandskikt
Si	Si	silt	si	si	siltig	si	si	siltskikt
Sh	Sk	skaljord	sh	sk	med skal	sh	sk	skalskikt
Shgr	Skgr	skalgrus						
Shsa	Sksa	skalsand						
Co	St	stenjord	co	st	stenig	co	st	stenskikt
Su	Su	sulfidjord	su	su	sulfidjordshaltig	su	su	sulfidjordssikt
Sucl	SuLe	sulfidlera						
Susi	SuSi	sulfidsilt						
Pt	T	torv				pt	t	torvskikt
Ptf	Tl	lågförmultnad torv (tidigare benmänd filttorv) (eng. fibrous)						
Ptp	Tm	mellantorv (eng. pseudo-fibrous)						
Pta	Th	benämnd dytorv) (eng. amorphous)						
Pr	Vx	växtdelar (trärester) (eng. remains)	pr	vx	med växtdelar	pr	vx	växtdelsskikt
dc	t	(efter huvudord) torrskorpa, t ex Let och Sit = torskorpa av lera resp. silt. Exempel Cldc, Sidc	v	v	varvig, t ex vLe = varvig lera (beteckningen varvig bör förbehållas glaciala avlagringar)			

Tilläggsord som beskriver ingående underfraktioner (t.ex. sandigt grus saGr, grusig lera grCl) skrivs med gemener. Underfraktioner skall placeras som adjektiv i den ordning intill huvudordet som visar deras respektive betydelse.

Skiktad jord skrivs med understrukna tilläggsord med gemener efter huvudordet, (t.ex. grusig lera med sandskikt grCl sa). Huvudfraktionen ska för klarhetens skull anges med versal begynnelsebokstav.
Fyllningens innehåll skrivs ut i klartext på engelska t.ex. Mg/asphalt, brick,

Mineraljordarter delas in i grupperna fin, mellan och grov med nedanstående beteckningar:

Grovgrus	CGr	Mellansand	MSa
Mellangrus	MGr	Finsand	FSa
Fingrus	FGr	Grovsilt	CSi
Grovsand	CSa	Mellansilt	MSi
		Finsilt	FSi