Rock slope stability analysis of a North Hungarian open-pit mine

Treball realitzat per:
Isabella Guzmán Rincón

Dirigit per:
Gyula Bögöly
Jose Antonio Gili Ripoll

Grau en:
Enginyeria Civil

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Abstract

Landslides and rock falls have been known as a non-predictable natural phenomenon during many years, causing not only economic consequences but also dangerous situations. In the last 50 years, many studies in mining and civil engineering have carried out new techniques to calculate and predict the likelihood of occurrence of these phenomena. These calculations have become even simpler thanks to the advanced technologies and the creation of computer software able to calculate the probability of failure of a slope with just a few input parameters.

The main goal of this thesis was to carry out a rock slope stability analysis in a real open-pit mine in the North of Hungary (Mány quarry) to determine the maximum safe overall slope angle (probability of failure equal to zero). A safe overall slope angle is the most important parameter in the stability of a quarry to ensure a good operation throughout its life cycle and to maximize profits. To carry out this analysis it was important to first set the main steps which are necessary to reach the objective above.

First, it was essential to undertake a **desk study** to understand **the typical failure mechanisms** and stability problems which can happen in an open-pit mine (chapter 2). The main failure mechanisms for rock slopes which were considered to be the most significant for this case of study were: The Planar failure, the Wedge failure and the Circular mechanism, as they are the most typical ones in case of rock slopes.

Once the possible failure mechanisms were detected, the next step was to **focus on the specific study area** (Mány quarry). **In situ geological data and samples for the stability calculations were obtained** (chapter 3 and 4) and **the necessary laboratory tests were performed** (chapter 4)**.** A **geological background** study was made, a **description of the current state of the quarry** and some **samples were taken to the laboratory.** From the geological data, main joint sets, GSI value, JRC and all the necessary parameters from the field were determined. In the laboratory, characterization of the samples, UCS test and Brazilian test were performed.

The next step was then, **to define the best numerical analysis method for the case of study** taking into account the information about the failure mechanisms and the specific geological data from the previous chapter. The stability analyses were carried out using different Rocscience Incorporation software, these programs work with Kinematic methods and Limit Equilibrium Methods for both planar and wedge failure mechanisms and a global analysis for circular failure mechanism. Dips software for the both kinematic analyses and RocPlane and Swedge software for limit equilibrium analyses for planar and wedge failure mechanisms. Finally, a global analysis was carried out using the Slide 7.0. software.

From the in-situ data and the laboratory test, the input parameters for the numerical modelling were obtained. And consequently, all the **numerical modelling** was carried out (chapter 5). **The results achieved showed a maximum overall slope angle of 57º**.

The steepness defined, ensures a safe operation of the quarry because it reduces the probabilities of failure of the slope to zero percent. To complete the thesis, the slope angle was applied to the walls of the quarry and the final sections can be seen in the annexes.

Acknowledgements

This bachelor's thesis is the last step in completing my bachelor degree in Civil Engineering at the Polytechnic University of Catalonia. It is written in collaboration with the Budapest University of Technology and Economics. My supervisors have been Gyula Bögöly and Péter Görog (professors at BME); and Jose Antonio Gili Ripoll (professor at UPC),.

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Last but not least, thanks a lot to my family, for supporting me during all these years and make me go further and reach my goals, to Joan for being there and growing up with me.

THESIS ASSIGNMENT

Name:	Isabella Guzman Rincon	Neptun code:	VR5G2E

Academic program:	BSc in Civil Engineering	Semester:	2016/17/2

Specialization:	Geotechnics	Registration number.:	BSc-C-xxx-16/17/2

Title:	Rock slope stability analysis of a North Hungarian open-pit mine

Assignment's description:

The thesis describes the characteristic stability problems which can occur in case of open cast excavations. It focuses on the typical failure modes of a dolomitic rock mass. Through investigations of a given quarry in North Hungary the candidate has to carry out own calculations regarding the stability of the rock slopes. The main aim of the thesis is the optimization of the slope steepness using the given instructions of the mining engineers. The input parameters for the calculations should be derived from in situ and laboratory tests. For the numerical modelling the different software of the Rocscience Incorporation will be available.

The planned parts of the thesis:

1. Description of the geological background
2. Desk study about the typical stability problems of an open cast mine
3. In situ and laboratory investigations for determining the parameters of the rock mass
4. Numerical modelling of the different failure types with using probabilistic values
5. Discussion of the results.
6. Final conclusions, recommendations.

Consultant:		
Gyula Bögöly PhD, assistant professor	BME, Department of Engineering Geology and Geotechnics	

Co-consultant(s):		
Péter Görög PhD, assistant professor	BME, Department of Engineering Geology and Geotechnics	
	Universitat Politècnica de Catalunya	

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László Nagy PhD on behalf of Major of Geotechnics	László Dunai PhD Dean

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1. Introduction

1.1. Goals

In the open pit mining design, the stability analysis of the slope is one of the principal requirements to ensure a good operation of the quarry throughout its life cycle. The stability of the slope is very important for economic, safety and social reasons. Economically, the consequences of the failure are not only the loss of production but also a direct cost of removing the failed rock and stabilizing the slope and a wide range of indirect cost, which includes injuries and damage. To reach a good operation, It is essential to **determine the failure mechanisms and the maximum safe slope angle for the specific conditions of each opencast mine**. In opencast mines, the slope angles must be as steep as possible to maximize the profits (Wyllie, Mah and Hoek, 2004).

The design of the angle requires the understanding of the local geology and geotechnical data of the specific emplacement, principally the structural geology, also the rock mass characteristics such as the rock strength and finally, other conditions such as groundwater or seismic conditions.

The aim of this project is to perform the slope stability analysis of a given quarry in order to obtain the optimum slope steepness for all the walls of the quarry. For open pits, an increase in slope angle of one or two degrees will result in a saving of several million cubic meters of rock excavation. The numerical modelling is done using the Rocscience Incorporation software programs. The input parameters are derived from in situ and laboratory tests.

To reach this aim, the main steps of the thesis are:

- To **understand the typical failure mechanisms** and stability problems in open cast mine. In particular, on the dolomitic rock mass.
- To **obtain in situ the geological data** necessary for the calculations and **perform the laboratory tests.**
- To carry out the **numerical modelling** of the slope.
- To **achieve the results, discussions, conclusions and steepness of the slope** (slope angle).

1.2. Thesis structure

The structure of the thesis is divided into six main chapters. These parts will analyse and give an answer to the principal objectives in the same order as they are described above.

The first step corresponds to the second chapter of the thesis. It consists of a desk study and review of literature in order to understand the typical stability problems.

The third chapter focuses on the emplacement and it describes the location and the geological background of the quarry.

A visit to the open pit mine was made to check the current state of the excavations and to obtain all the necessary information for the laboratory tests and the stability calculations. The main information taken from the quarry was:

- Observe the joint orientation and obtain the dip directions and dip angles of the bedding planes and the discontinuities.
- The roughness of the surfaces.
- Take samples for the laboratory tests.

Also in the fourth chapter all necessary the laboratory test are carried out. The main ones are: the Uniaxial compressive strength test according to ISRM (1979) and the Brazilian test according to ISRM (1978). Calculations of the GSI, JRC and JCS values are performed.

The fifth chapter consists in introducing the input parameters and performing the calculations. This part can be subdivided into the following subgroups depending on the Rocscience software used:

- In order to get the sets of joints and be able to see the main directions of the discontinuities, stereographic plots will be made with Dips software.
- Kinematic analyses for both planar and wedge failure methods are also performed with the Dips software from Rocscience.
- To analyze the planar failure the Rocplane software is used, introducing the results of the last point.
- For the wedge failure, the software used is the Swedge also with the results of the obtained from the Dips software.
- A global failure analysis is completed with the Rocscience software Slide 7.0.

Finally, in the last chapter, an overview of the results is made and they are discussed in order to obtain the conclusions. The ultimate steepness of the slope (slope angle) is determined to ensure a 0% probability of failure. Just a single slope angle is determined for the entire quarry.

The application of maximum slope angles for each wall of the quarry means a more exhaustive study in the different sections and it is not comfortable for the design of the road paths for the dumpers and other machinery, neither for the design of its excavation.

2. Desk study

2.1. Open cast excavations

The kind of excavation which is carried out in the near surface, using one or more horizontal benches is known as open-pit mining. When the materials obtained from an open-pit mine are building materials such as aggregates, sand and gravel or dimension stones, the excavation is usually named quarry. (Technomine, 2012)

There are three main elements in the design of an open pit slope:

- The **overall pit slope angle** from crest to toe incorporates all ramps and benches. This slope is flatter if there are weaker, surficial materials and a steeper if there is a stronger intact rock at depth. This is the angle that is calculated in this thesis.
- The **inter-ramp angle** is the slope, or slopes, lying between each ramp that will depend on the number of ramps and their widths.
- The **face angle** of individual benches depends on the vertical spacing between benches and the width of the benches required to contain minor rock falls. This angle is determined by taking into account the orientation of the main joint set if there are joints that dip out of the face at a steep angle. If this does not happen, then the bench angle will be related to the overall slope geometry (Wyllie, Mah and Hoek, 2004).

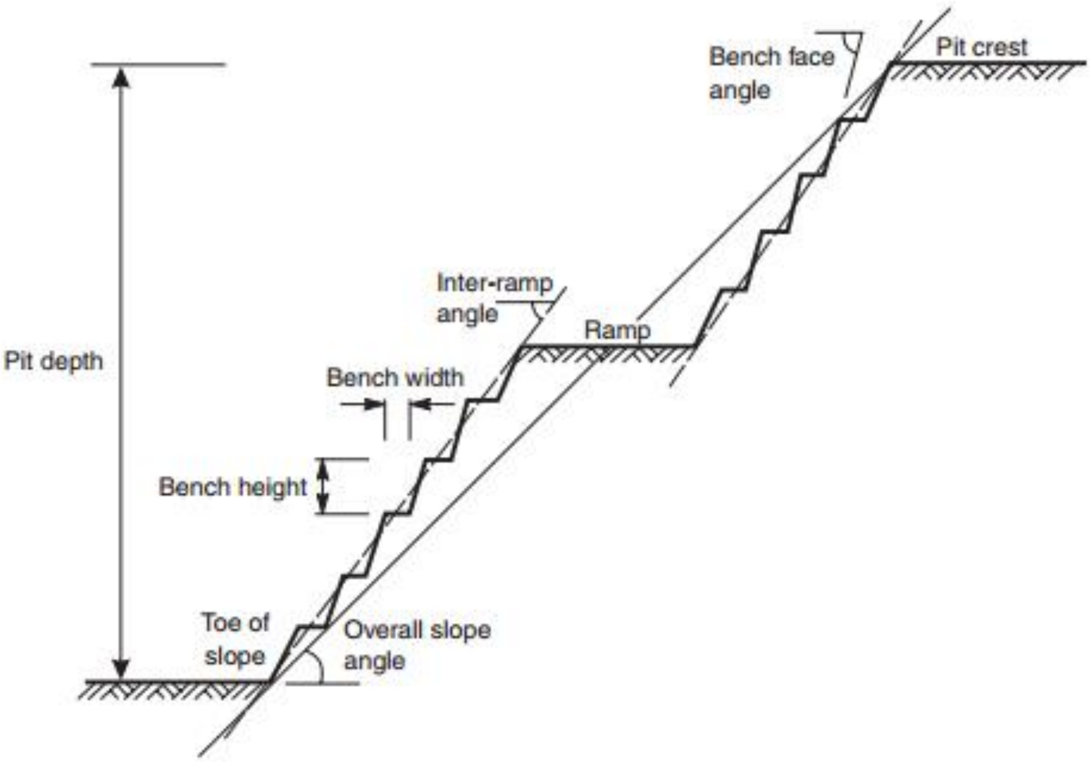


Figure 1. The geometry of an open pit mine, showing its main angles (Wyllie, Mah and Hoek, 2004).

2.2. Main types of discontinuities

Rock masses usually have heterogeneous and anisotropic properties. They contain different types of discontinuities such as bedding planes, faults, joints and folds. These main kind of discontinuities will be described below.

Fault

Faults are fractures where there has been an observable shear displacement. Faults usually occur in parallel sets of discontinuities. They are usually classified by the sense of the displacement and their sizes vary from meters in major cases to millimeters in local faults (Bell, 1993).

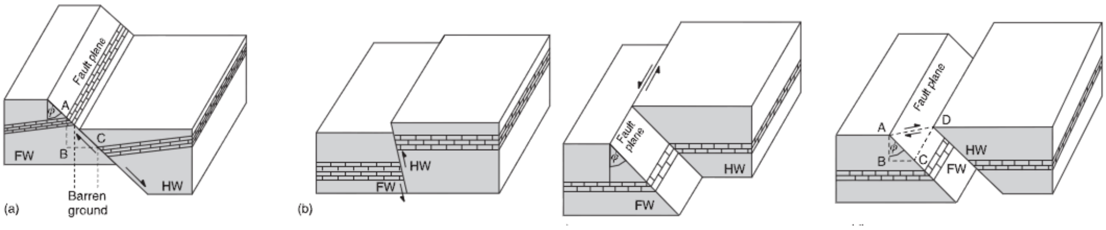


Figure 2. Types of faults: (a) normal fault, (b) reverse fault, (c) wrench or strike-slip fault, (d) oblique-slip fault (Bell, 1993).

Bedding

Bedding planes are typical from sedimentary rocks. It consists in a surface parallel to the surface of deposition, strata. It creates a preferred orientation of particles in the rock creating planes of weakness parallel to the bedding (Bell, 1993).

Joint

Brittle-fracture surfaces where there has been no observable relative movement. Joints extend along different directions. A series of parallel joints is called a joint set and two or more intersecting sets produce a joint system (Bell, 1993).

2.3. Failure Mechanisms

For any soil or rock slopes, there is a wide range of typical failure mechanisms. The failure of a slope is classified depending on the geometrical and mechanical properties of the rock mass and its discontinuities. It is very important to determine for each particular case, which are the most likely failure mechanisms and the best method of analysis for each of them. The following table shows the principal failure mechanisms for soil and rock slopes and their analysis methods:

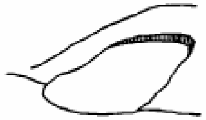
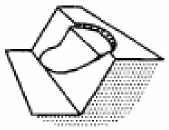
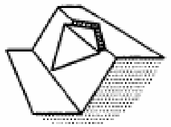
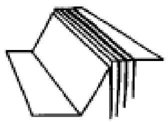
STRUCTURE	TYPICAL PROBLEMS	CRITICAL PARAMETERS	ANALYSIS METHODS	ACCEPTABILITY CRITERIA
 Landslides.	Complex failure along a circular or near circular failure surface involving sliding on faults and other structural features as well as failure of intact materials.	• Presence of regional faults. • Shear strength of materials along failure surface. • Groundwater distribution in slope, particularly in response to rainfall or to submergence of slope toe. • Potential earthquake loading.	Limit equilibrium methods which allow for non-circular failure surfaces can be used to estimate changes in factor of safety as a result of drainage or slope profile changes. Numerical methods such as finite element or discrete element analysis can be used to investigate failure mechanisms and history of slope displacement.	Absolute value of factor of safety has little meaning but rate of change of factor of safety can be used to judge effectiveness of remedial measures. Long term monitoring of surface and subsurface displacements in slope is the only practical means of evaluating slope behaviour and effectiveness of remedial action.
 Soil or heavily jointed rock slopes.	Circular failure along a spoon-shaped surface through soil or heavily jointed rock masses.	• Height and angle of slope face. • Shear strength of materials along failure surface. • Groundwater distribution in slope. • Potential surcharge or earthquake loading.	Two-dimensional limit equilibrium methods which include automatic searching for the critical failure surface are used for parametric studies of factor of safety. Probability analyses, three-dimensional limit equilibrium analyses or numerical stress analyses are occasionally used to investigate unusual slope problems.	Factor of safety > 1.3 for "temporary" slopes with minimal risk of damage. Factor of safety > 1.5 for "permanent" slopes with significant risk of damage. Where displacements are critical, numerical analyses of slope deformation may be required and higher factors of safety will generally apply in these cases.
 Jointed rock slopes.	Planar or wedge sliding on one structural feature or along the line of intersection of two structural features.	• Slope height, angle and orientation. • Dip and strike of structural features. • Groundwater distribution in slope. • Potential earthquake loading. • Sequence of excavation and support installation.	Limit equilibrium analyses which determine three-dimensional sliding modes are used for parametric studies on factor of safety. Failure probability analyses, based upon distribution of structural orientations and shear strengths, are useful for some applications.	Factor of safety > 1.3 for "temporary" slopes with minimal risk of damage. Factor of safety > 1.5 for "permanent" slopes with significant risk of damage. Probability of failure of 10 to 15% may be acceptable for open pit mine slopes where cost of clean up is less than cost of stabilization.
 Vertically jointed rock slopes .	Toppling of columns separated from the rock mass by steeply dipping structural features which are parallel or nearly parallel to the slope face.	• Slope height, angle and orientation. • Dip and strike of structural features. • Groundwater distribution in slope. • Potential earthquake loading.	Crude limit equilibrium analyses of simplified block models are useful for estimating potential for toppling and sliding. Discrete element models of simplified slope geometry can be used for exploring toppling failure mechanisms.	No generally acceptable criterion for toppling failure is available although potential for toppling is usually obvious. Monitoring of slope displacements is the only practical means of determining slope behaviour and effectiveness of remedial measures.
 Loose boulders on rock slopes.	Sliding, rolling, falling and bouncing of loose rocks and boulders on the slope.	• Geometry of slope. • Presence of loose boulders. • Coefficients of restitution of materials forming slope. • Presence of structures to arrest falling and bouncing rocks.	Calculation of trajectories of falling or bouncing rocks based upon velocity changes at each impact is generally adequate. Monte Carlo analyses of many trajectories based upon variation of slope geometry and surface properties give useful information on distribution of fallen rocks.	Location of fallen rock or distribution of a large number of fallen rocks will give an indication of the magnitude of the potential rockfall problem and of the effectiveness of remedial measures such as draped mesh, catch fences and ditches at the toe of the slope.

Figure 3. Typical stability problems for soil and rock slopes. (Hoek, 1991)

From the previous list, the most important failure mechanisms that a quarry can undergo are the following:

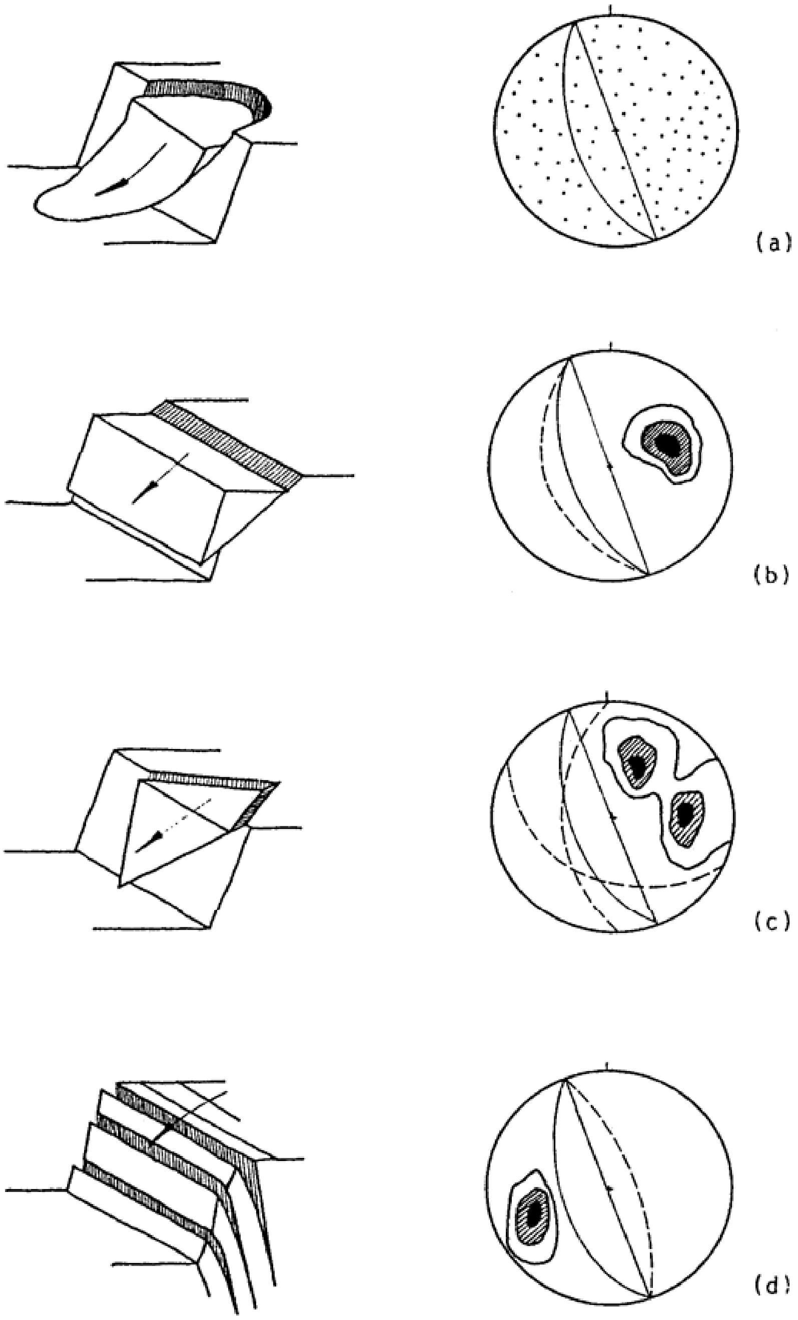
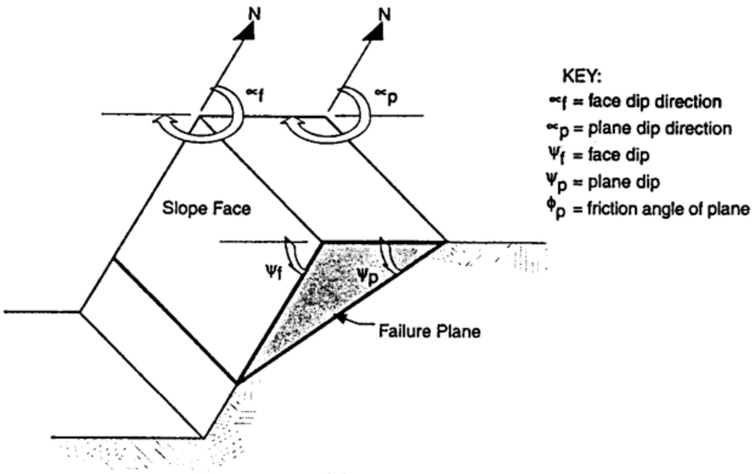


Figure 4. Four failure mechanisms (a) Circular failure; (b)Planar Failure; (c) Wedge failure; (d) Toppling failure; (Hoek and Bray, 1991).

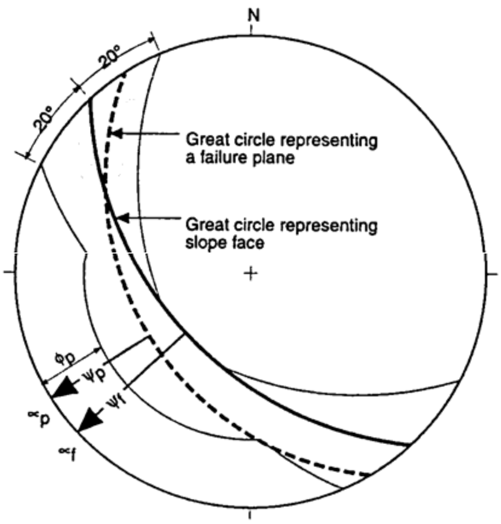
Planar failure

Planar failure is the process where sliding occurs along a plane of weakness which has appeared in the rock mass as a result of changes in shear strength and/or ground water conditions (Hoek & Bray, 1981). The geometric conditions for planar failure to occur will be explained below.

- Planar failure occurs when the strikes of the bedding planes or another discontinuity are nearly parallel to the slope within 20º and it produces the sliding of the plane.
- The dip of the discontinuity plane must be smaller than the dip of the slope and greater than its angle of friction.
- The likelihood of planar failure is not high because of the requirements that it needs and also because in the lateral extent of the mass, it can be other rock masses blocking the sliding.



(a) Plane Failure Model



(b) Great Circle Representation

Figure 5. Kinematic conditions for Planar Failure (Norrish and Wyllie, 1996)

Wedge failure

The wedge failure occurs when two discontinuity planes intersect and slide in the direction of this line of intersection. The wedge failure is, due to its geometric conditions, more frequent than planar sliding. The conditions for wedge failure to occur are explained below.

- The dip direction of the intersection and the dip direction of the slope must be similar.
- The dip of the slope must be higher than the dip of the intersection line and also the dip of the intersection line must be higher than the friction angle.

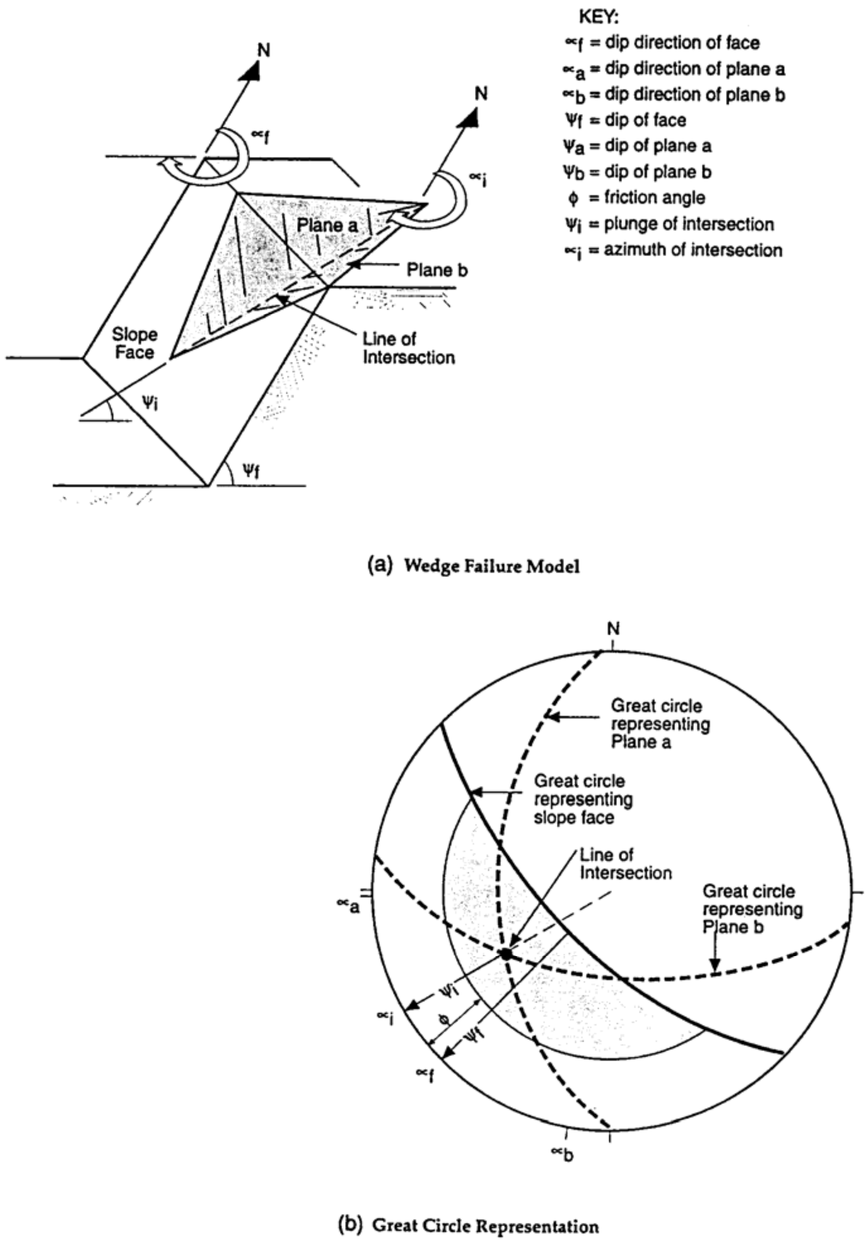


Figure 6. Kinematic conditions for Wedge Failure (Norrish and Wyllie, 1996)

Toppling failure

The toppling failure occurs due to the rotation of blocks of rocks which are distributed as slabs or columns. The basic conditions for this type of failure are:

- The plane of the toppling has the same strike of the slope but opposite direction
- The dip must be very steep, higher than 70º.
- It must satisfy the following equation.

$$[90º - \phi(dip\ of\ plane) \leq \phi(dip\ of\ slope\ face) - \phi(friction\ angle\ along\ plane)]$$

This kind of failure is easy to see in the field and it will not be considered for this investigation.

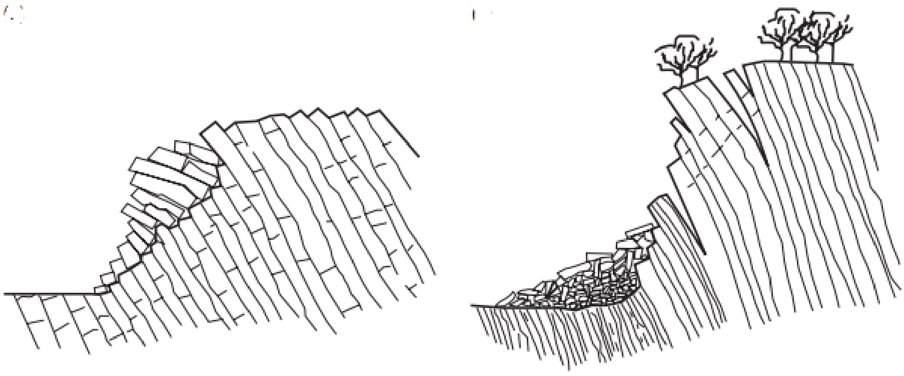


Figure 7. Toppling failure mechanism

Circular failure

The circular failure occurs when the rock mass is highly weathered and fragmented. In this cases the discontinuities do not influence in failure mechanisms, it is a failure without a pattern. Circular failures are highly influenced by the mechanical properties of the particles within the rock slope. Circular failure are likely to occur when the individual particles are very small in comparison to the overall outcrop. Highly weathered rocks tend to behave as a soil. (Wyllie, Mah and Hoek, 2004).

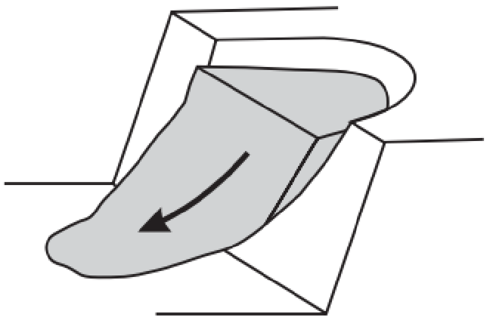


Figure 8. Circular failure mechanism

2.4. Groundwater

The groundwater level is an important parameter for the slope design due to the effects of the water pressure in the rock mass properties. More precisely, it is a relevant factor in zones with high levels of precipitation where it is necessary to develop a system of drainage and to take care of the periods of intense rainfall or snowmelt.

The water pressure reduces the stability of the slope by diminishing the shear strength of the discontinuity surfaces. Freezing can cause wedging in water-filled fissures due to the volume changes and it can block the drainage paths decreasing the stability. It also causes erosion on the surface.

2.5. Main properties which influence the rock slope stability

In any kind of rock excavations, the structural geology highly influences the stability of the rock slope. All the discontinuities occurring in the rock mass create the structural geology: faults, bedding planes, joints, and so on. The discontinuities change the properties of the intact rock, mainly the strength, creating planes of weakness where failure may occur. To observe the types of failure it is essential to know the properties of the discontinuities such as the orientation, roughness, weathering, filling and persistence.

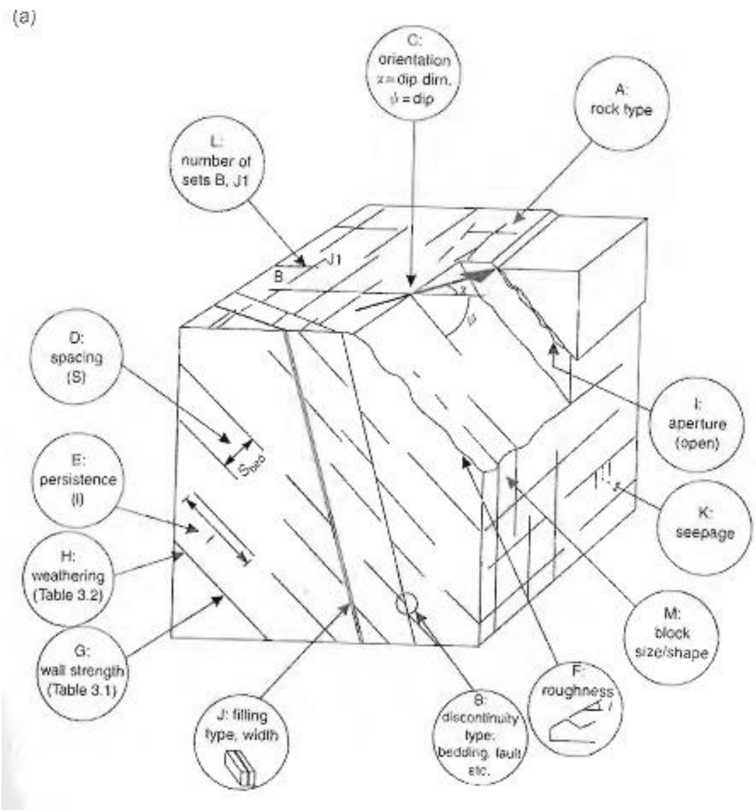


Figure 9. Properties of the discontinuities (Wyllie, Mah and Hoek, 2004)

Orientation

The main geological property which influences the stability is the orientation of the discontinuities. To obtain the information it is necessary to measure it from the surface and it is represented with the dip angle and dip direction are used (Wyllie, Mah and Hoek, 2004).

- Dip angle is the maximum inclination of a discontinuity to the horizontal.
- Dip direction is the direction of the horizontal trace of the line of dip, measured clockwise from north.
- Strike is the imaginary line drawn by the intersection of the slope with a horizontal plane; it is perpendicular to the dip direction.

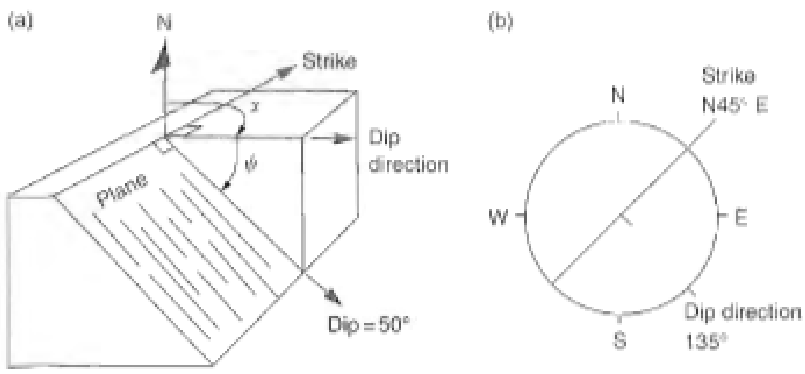


Figure 10.Representation of the orientation. (Hoek, 2000)

Rock strength

From the discontinuities parameters, there are some of them which affect the rock strength and are relevant for the slope design. The main strength parameters determined for rock slope design are the shear strength of the discontinuities and the shear strength of the rock mass, the weathering of the rock mass and the compressive strength of the intact rock.

To calculate these parameters, laboratory test are carried out and the shear strength is usually obtained from drilled core samples (Wyllie, Mah and Hoek, 2004). Also, the compressive strength can be calculated in the laboratory from core samples or from in situ test in the field.

3. Location and geological background description

3.1. Location and description

The project takes place in the quarry Mány I. Strázsahegyi dolomite quarry. It is located 0n the northeast side of Fejér county, 40 km west from Budapest in the periphery of the municipality of Mány. The emplacement is easy to reach by car, south-eastside direction of the public road no. 1104 between Bicske and Piliscsaba.

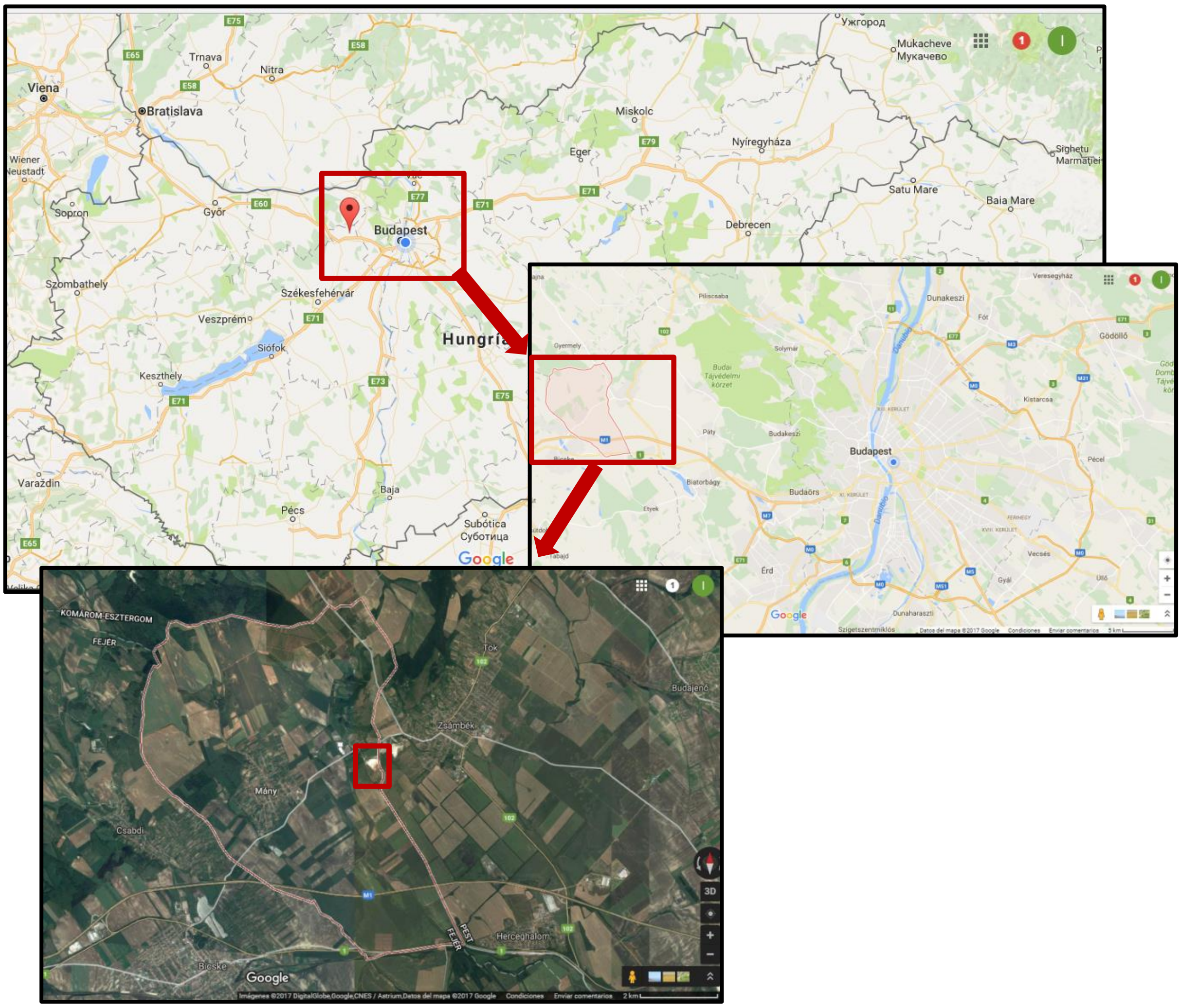


Figure 11. Location of the Mány. Quarry in Hungary. From Budapest and inside the municipality of Mány. Available at: Google Maps

It consists in a dolomitic rock mass formation, more precisely it has the name of Budaörsi Dolomitic formation, the lithology can be described as well-stratified, bedded, sometimes as a cyclic structure of dolomite; Dasycladacea algae residues can be found.

- The area of the quarry is, approximately, 20ha 7500m2, that is to say, 0.21km2.
- The current elevation of the lowest point is at 18m meters above sea level and the present elevation of the highest point at 245 meters above sea level (Survey, 2014).
- The original elevation of the hill was 265m above the sea level and the designed base level is 150m meters above the sea level.

3.2. Geological Background

The quarry situated between the Vértes and Gerecse mountains on the north-west side of the Zsámbék basin. The region of the quarry is covered by upper Pleistocene-Holocene sediments: proluvial and deluvial sediments, clayey aleurolite and upper Pleistocene loess. Several Miocene formations are typical in the region. Near to the quarry the Sarmatian age Tinnye Limestone formation can be found. The Csákvár Clay Marl Formation is also present around the quarry. The above-mentioned limestone is a yellow biogenetic, oolitic limestone, calcareous sandstone which is usually poorly-bedded (Gyalog and Császár, 1996). The Csákvár Clay Marl Formation is grey clay marl, white marl which is typically formed in the basins (Jámbor, 1980). The average dip direction of the layers of the region is north-northeast.

The quarried material of the mine is the middle-upper Triassic age Budaörs Dolomite Formation. It is mostly white, cyclical and well-bedded dolomite. It forms rocky outcrops on the edge of the Mány basin. The thickness of the formation is approximately 300-1200 m. (Gyalog and Császár, 1996)

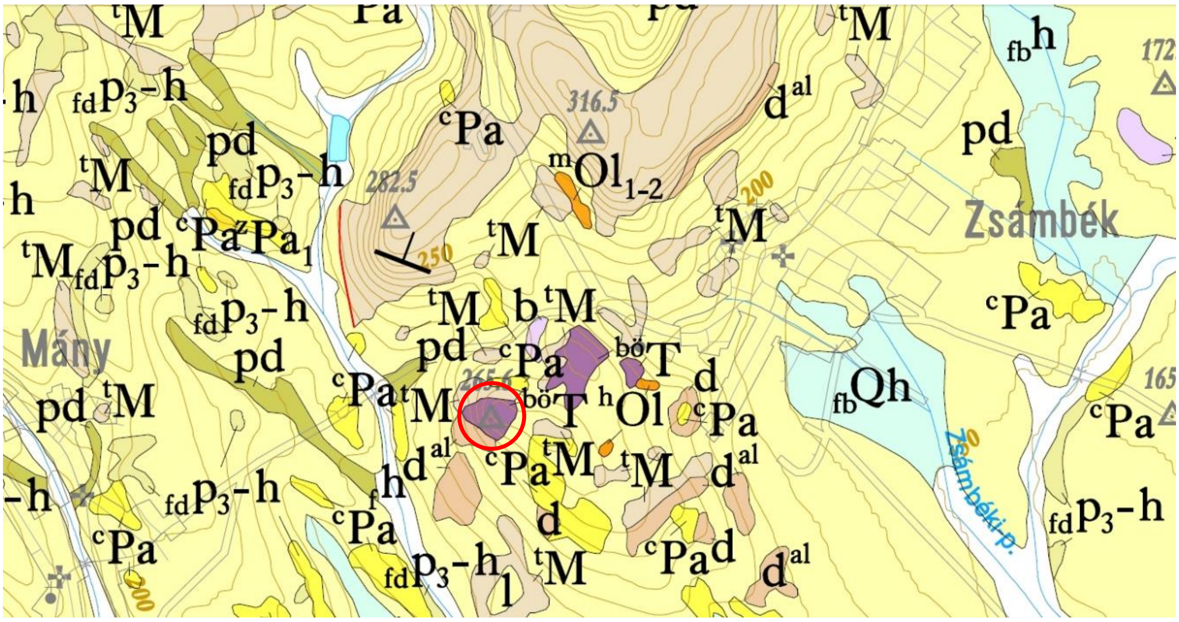


Figure 12. Geologic map of the region of Mány and Zsámbék

Legend: red circle shows the location of the quarry, $^{\text{bö}}$T: Budaörs Dolomite Formation; ^{c}Pa: Csákvár Clay Marl Formation; M^{t}: Tinnye Limestone Formation; l: Loess; d^{al}: aleurite; pd: proluvial and deluvial sediments; ^{h}Ol: Hárshegy Sandstone Formation; (MÁFI, 2005)

Since the quarry is located in a high spot of the region the presence of groundwater is not expectable. Until now during the excavation, there was no sign of groundwater. However, near the base level of the quarry, it is possible that groundwater will appear since the elevation of the brooks approximately the same as the elevation of the base level (150 m above the sea level).To be conscious of accurate information regarding the level of groundwater the stability calculations have to be re-evaluated in the future.

3.3. In situ observations

The in situ observations showed that the works have advanced since the last observation and perforations have been made to carry out new explosions.

The dolomite in the quarry is strongly fractured, more in the part of the quarry where the works are not taking place now. It is easy to observe different bedding planes and joints in all the sections of the quarry.

3.4. Dolomite formations

Dolomite or dolostone is a sedimentary carbonate rock composed mostly of the mineral dolomite, CaMg(CO3)2. It has a small content of calcite not higher than 10% of the calcite-dolomite pair's content.

Dolomite is a very common sedimentary rock (formed before the Mesozoic). It originates from the calcium carbonate mud accumulated in marine environments. It can be found worldwide in sedimentary basins. This process is called dolomitization.

Dolomitization occurs when the calcium carbonate is replaced by calcium magnesium carbonate through the action of magnesium-bearing water percolating the limestone or limy mud.

Uses of Dolomite

Dolomite rocks are important oil reservoir rocks because average dolomite has usually higher porosity than limestone.

Dolomite is **used as construction material** more specifically for their ability to neutralize acids. Dolomite and limestone are crushed and used as an aggregate in construction projects and also in the manufacture of cement. They are also cut into blocks and slabs for use as a dimension stone. Its greater hardness and lower solubility make it a superior construction material and more resistant to the acid content of rain and soil than limestone.

Dolomite is used as a source of magnesia (MgO) in the chemical industry. The steel industry uses dolomite as a sintering agent in processing iron ore. In agriculture, dolomite is used as a soil conditioner and as a feed additive for livestock. Dolomite is used in the production of glass and ceramics.

4. Obtaining data for the stability analyses

The main goal when undertaking a rock slope stability analysis in a quarry is to achieve the safe and functional design of the excavated slopes (Eberhardt (2003)).

For the rock slope stability analysis, as it was explained previously, it is important to into consideration the rock slope stability conditions; to investigate potential failure modes and finally, to determine the optimal excavated slope parameters in terms of safety reliability and economics.

Following these criteria and the steps of the thesis, the stability analysis starts with field investigation study including **geological and discontinuity mapping** to provide the necessary input data for the analysis. And also a rock mass characterization by **collecting rock samples for laboratory testing** is necessary for obtaining valuable input data.

4.1. Numerical modelling for rock slope stability analysis

Nowadays, a wide number of advanced software for numerical modelling methods is available for slope stability analysis. However, it is very important to understand the strengths and limitations of each methodology. Depending on the site conditions and potential failure mode, different analysis techniques are appropriate for different sites (Eberhardt, 2003). For this particular case, conventional methods of rock slope analysis will be used, as the quarry does not present any complex failure mode. The image below shows the different levels of complexity of failure mechanisms and its adequate methods of calculation:

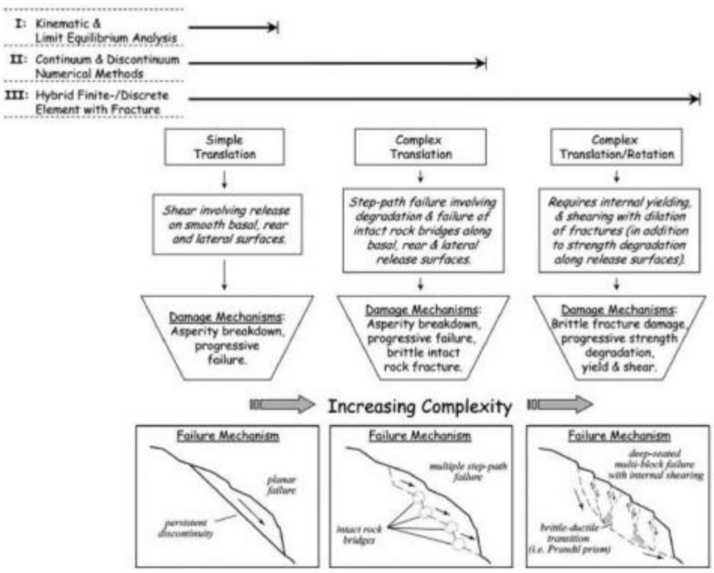


Figure 13. Increasing complexity of failure requires different levels of analysis. After Stead et al. (2006)

Conventional methods of Rock Slope Analysis

Conventional methods have been widely used in rock slope engineering during the last 25 years, for calculating simple failure mechanisms. These methods consist of two main techniques: kinematic and limit equilibrium analysis.

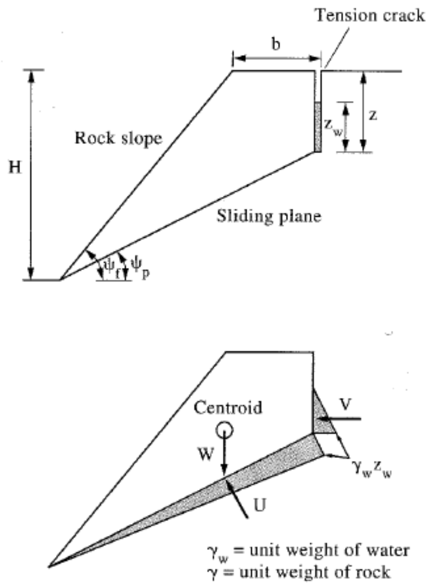
These methods are adequate just for noncomplex mechanisms of failure, such as simple block failure along discontinuities, and they can be useful as a first approach for more complex failure mechanisms (e.g. progressive creep, internal deformation and brittle fracture, liquefaction of weaker soil layers, etc.).

Kinematic Analysis

Kinematic methods analyse the slope stability based on the rock mass structure and the geometry of existing discontinuities. It focuses on transitional failures due to the formation of daylighting wedges or planes. It only recognizes potential sliding failures along a single discontinuity (planar) or along discontinuity intersections. (Eberhardt, 2003). To perform the kinematic analysis a stereonet plot is used to calculate wedge and planar failure. In this particular case, the Dips software from Rocscience is used.

Limit Equilibrium Method (LEM)

The Limit Equilibrium Method (LEM) studies the stability based on static forces and moment equilibrium for both translational and rotational failures. The method is the basis for stability analysis of simple rock slopes and has been broadly used since the 20th century. LEM are highly recommended for calculations of simple block failures along known discontinuities or highly fractured or strongly weathered rock mass which then acts as soil. LEM compares the driving forces/moments against the resisting forces/moments, which act on a predefined sliding plane in order to compute a Factor of Safety (FS) or a probability of failure. Limit equilibrium analysis will be performed to calculate the rock slope stability of the Màny quarry for planar and wedge failure as it is shown in the images below.

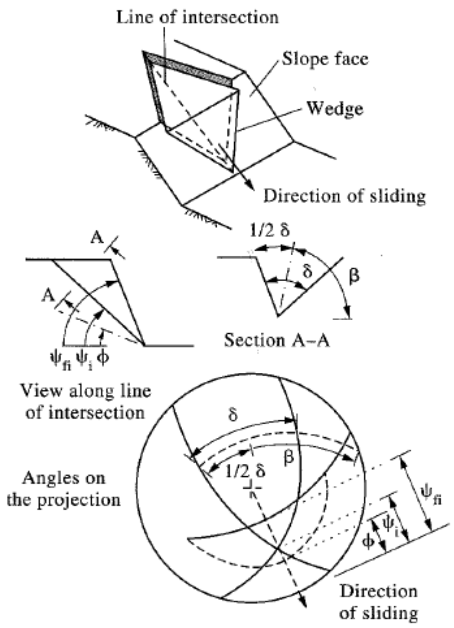


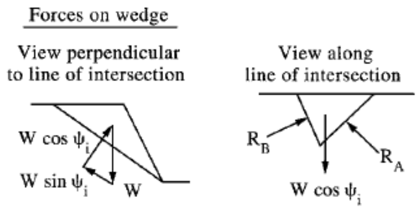
$$SF = \frac{c'(H-z)\csc\psi_p + (W\cos\psi_p - U - V\sin\psi_p)\tan\phi'}{V\cos\psi_p + W\sin\psi_p}$$

Where:

SF = safety factor
c' = effective cohesion
ϕ' = effective friction angle
ψ_p = dip of slide plane
W = weight of block
U = uplift force due to water pressure along slide plane
V = force due to water pressure in tension crack
H = slope height
z = tension crack depth

Figure 14. Limit equilibrium solution for planar failure. (Hudson and Harrison, 1997)





$$SF = \frac{(R_A + R_B)\tan\phi}{W\sin\psi_i}$$

$$\text{and } R_A + R_B = \frac{W\cos\psi_i\sin\beta}{\sin(0.5\,\delta)}$$

Where: SF = safety factor
ϕ = friction angle
ψ_i = dip of the line of intersection
W = weight of block
β, δ = wedge geometry factors

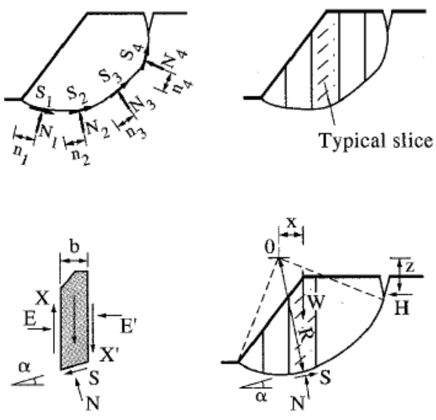
Figure 15. Limit equilibrium solution for wedge failure. (Hudson and Harrison, 1997)

Rotational analysis

The Rotational analysis is the third technique for limit equilibrium analysis. For very fragmented rock masses which are formed by very weak rock acting mostly as a soil, the structural geology above does not work to calculate the stability of a slope. This kind of slope will generally produce circular failures, rotational failures and curvilinear failures.

To calculate these failure mechanisms, iterative procedures are used, considering force/momentum equilibrium of unstable masses. To perform a rotational analysis it is necessary to select the potentially unstable slide masses and dividing them into slices (Method of Slices). The most commonly used methods are the Ordinary, the Bishop and Janbu methods of Slices.

For the particular case of the Mány quarry, the Bishop and Janbu methods of slices are used. A detailed limit equilibrium analysis of the slope has been done as shown in the following image with the Slide program of Rocscience.



$$SF = \frac{\sum SF\,(S\sec\alpha)}{\sum(W\sin\alpha) + Hz/R}$$

Where: SF = safety factor
S = effective shear strength
(i.e. $S/\Delta b = c' + \sigma_n \tan\phi'$)
α = dip of base of slice
W = weight of slice
H = hydrostatic thrust from tension crack
z = depth of tension crack (relative to O)
R = length of moment arm.

Figure 16. Limit equilibrium solution for circular failure. (Hudson and Harrison, 1997)

Advanced Numerical methods of rock slope stability

Some rock slope stability problems often involve a higher degree of complexity related to geometry, material anisotropy, non-linear behaviour, internal deformation, structural fabric, pore pressures, seismic loading and other factors (Eberhardt, 2003). For such instabilities, simple LEM methods are not good enough and it is necessary to work with more advanced numerical modelling techniques. These techniques are more general and offer a better solution for a wide range of problems which cannot be solved using the conventional methods explained above. Advanced numerical methods allow the user to introduce the initial conditions of the slope, such as water pressure, in situ stresses and boundary conditions and it gives back a simulated mechanical response. Therefore, calculations are much slower than LEM.

The advanced methods are based in the division of the geometry of the rock mass into small areas with assigned the intrinsic material properties. The limits of the areas can represent the discontinuities of the rock mass and can be then, modelled as a continuum or a discontinuum.

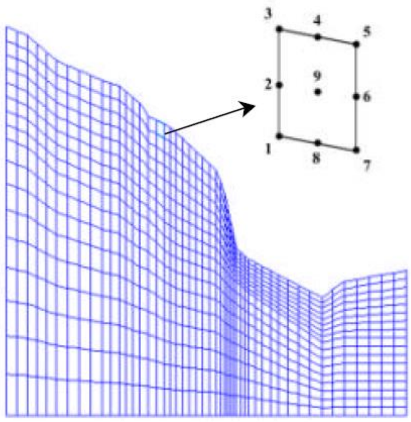


Figure 17. Finite-Element mesh of a natural rock slope (Eberhardt, 2003)

Discontinuum modelling

Discontinuum methods are suitable for slopes which are defined by some joint sets that rule the mechanism of failure. These methods are used if the rock masses have discontinuities. Then, the divisions are formed by separate, interacting blocks which are assembled together as a rock mass. External loads are applied to each block and every discontinuity is assigned with their properties such as orientation and location. This allows the representation of motion of the blocks within time (Eberhardt, 2003).

Continuum modelling

When the material is homogeneouss the rock mass can be treated as a continuum and the discontinuities can be represented implicitly. Therefore, the most suitable method will be the Continuum modelling. A wide range of software is available for continuum modelling, both in 2D and 3D (Wyllie, Mah and Hoek, 2004).

Finite Element Methods

FEM are used in continuum models. The geometry is divided in small discrete, interacting elements, such as triangles.

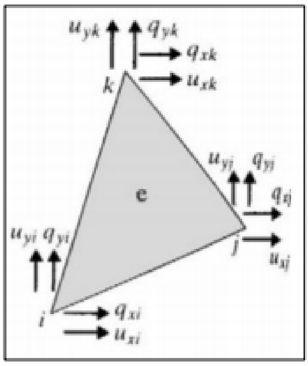


Figure 18. Example of the element (e) used in FEM. The nodes are defined by the letters i, j and k. Displacement and forces at the nodes are given by the vectors u and q (Brady & Brown, 2013).

Displacement components are given by the nodal displacement (u), which guarantees continuity of the displacement. From the displacement, the strain can be found by using strain-displacement relations. The stress can be also determined by the strain and the elastic properties of the material. The transmission of internal forces (q) is represented by interactions of the nodes of the elements (Brady and Brown (2013)).

4.2. Classes of calculation of rock strength

This chapter will show the most suitable rock mass and discontinuity shear strength criteria which will be applied in the stability analysis of the quarry. Reliable input parameters for the rock strength properties in both LEM methods and other numerical methods are essential in order to obtain reliable results.

Sliding surfaces can exist either along pre-existing discontinuities or through the rock mass, depending on the geological conditions. Therefore, in slope stability analyses, classes of rock strength can be either discontinuity shear strength or rock mass shear strength depending on the character of the rock mass as shown in the next figure.

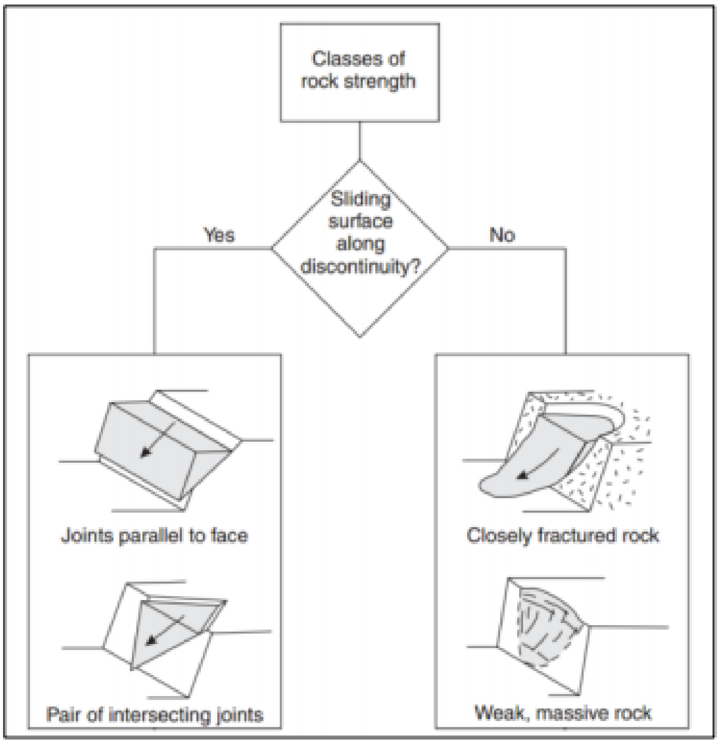


Figure 19. Classes of rock strength. (Wyllie and Mah,2004)

Rock mass shear strength

The rock mass shear strength is calculated when there are fractured rock masses, composed of intact pieces of hard brittle material separated by joints covered by layers of weaker materials (infilling). Then, the strength of the rock masses is dependent on the strength of the intact pieces and on their freedom to move.

To determine the strength the Hoek-Brown failure criterion (Hoek, 1983) is usually chosen. It is based on the number, orientation, spacing and shear strength of discontinuities. Hoek proposed an empirical failure criterion for estimating the strength of jointed rock mass. This relationship has been modified and revised over the years. One of the last modifications included a new classification system "Geological Strength Index (GSI)" (Hoek, 2000) which will be explained below. The GSI value is applied in the numerical modelling of the quarry for the Global Analysis.

The Generalised Hoek-Brown failure criterion for jointed rock masses is:

$$\boldsymbol{\sigma 1}' = \boldsymbol{\sigma 3}' + \boldsymbol{\sigma ci}\,(\boldsymbol{mb}\,(\boldsymbol{\sigma 3}'/\boldsymbol{\sigma ci}) + \boldsymbol{s})^{\boldsymbol{a}}$$

Where:

$\sigma 1'$ and $\sigma 3'$ = maximum and minimum effective principle stresses at failure

σci = uniaxial compressive strength of intact rock mass

mb = value of the Hoek-Brown constant m for the rock mass s and a = constants which depend on the rock mass characteristics

This equation should only be applied for rock masses which are suitable for the criterion, which means, isotropic rock mass behaviour: materials with a large number of similar discontinuities with analogous properties. The criterion can also be applied if the area of the analysis is big enough and composed by small block sizes. The following figure shows how the criterion usage is dependent on sample size.

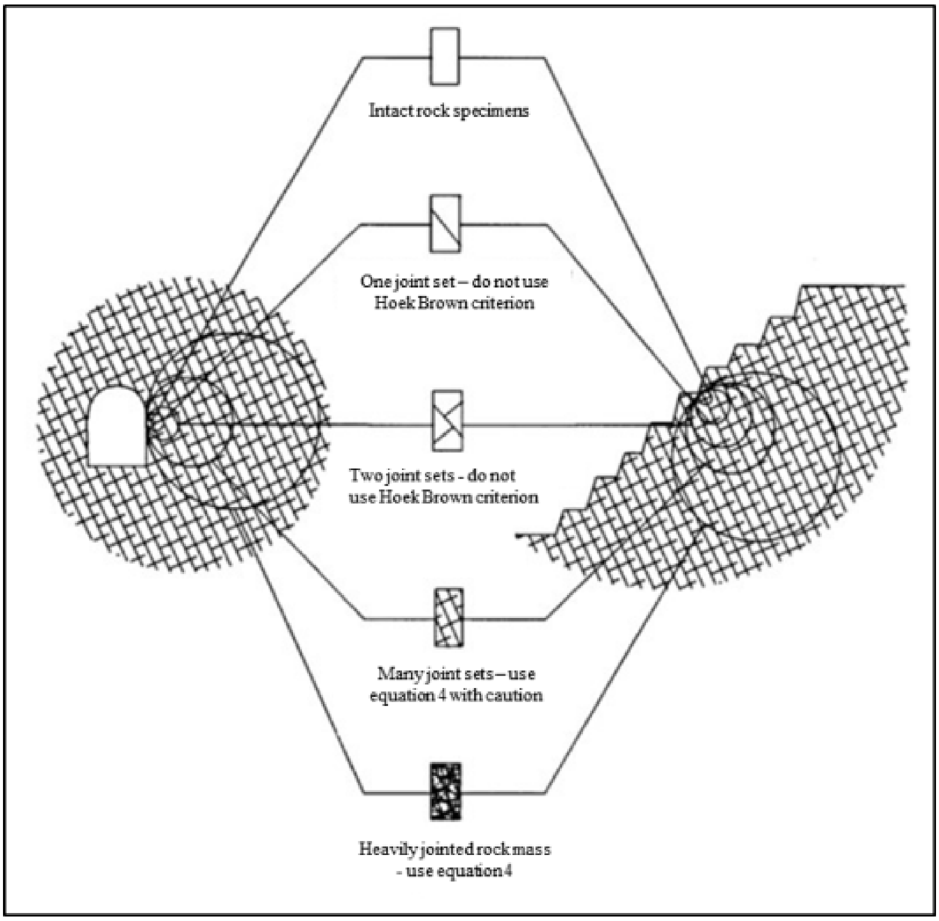


Figure 20. The transition from intact rock to heavily jointed rock mass. Modified from Hoek (2007).

Geological Strength Index (GSI)

The GSI classification system is used to determine the rock mass quality in the field and is composed by two factors; the degree of fracturing and the condition of the fracture surfaces (Figure 21). It was introduced as an upgrade of Hoek-Brown criteria in 2000 (Hoek,2000). The GSI value for the rock mass is included in the Generalized Hoek-Brown criterion as follows:

$$\boldsymbol{mb} = \boldsymbol{mie} \ (\ \boldsymbol{GSI}\mathbf{-100\ 28-14}\boldsymbol{D} \ )$$

$$\boldsymbol{s} = \boldsymbol{e} \ (\ \mathbf{-100\ 9-3}\boldsymbol{D} \ )$$

$$\boldsymbol{a} = \mathbf{1\ 2} + \mathbf{1\ 6} \ (\boldsymbol{e} \ \mathbf{-}\boldsymbol{GSI}/\mathbf{15} - \boldsymbol{e} \ \mathbf{-20}/ \ )$$

D is the disturbance factor which depends on the degree of disturbance due to blast damage and stress relaxation (Hoek, 2007).

Figure 21. Table to determine the GSI value (Hoek, 2007).

Shear strength of discontinuities

When the stability of the rock slope is ruled by discontinuities, it is necessary to introduce parameters related to the strength of these discontinuities. These parameters will be obtained by undertaking shear strength tests.

Mohr-Coulomb method creates a curve which shows the material has an elastic behaviour until the peak strength is reached, then a residual strength remains. (Wyllie & Mah, 2004).

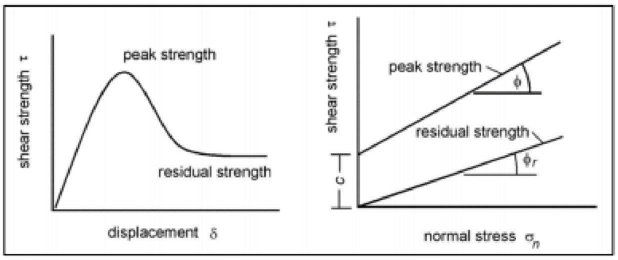


Figure 22. Peak strength and residual strength plotted after shear testing of planar surfaces. (Wyllie and Mah (2004)).

Mohr-Coulomb is one of the the most commonly failure criterion in geotechnical engineering. It links shear stress (τ) and normal stress (σn) acting along the failure plane, cohesion of the material (c), and the friction angle re(φ) (Johnson & DeGraff, 1988);

$$\boldsymbol{\tau = c + \sigma n \tan \varphi}$$

Adjusting the Mohr-Coulomb criteria, some new specific criterion for rock masses have been developed.Through studies of the behaviour of natural rock joints, the first non-linear strength criterion for rock joints was developed by Barton (1973, 1976): the Barton-Bandis criterion for rock joint strength and deformability (Barton & Bandis, 1991):

$$\boldsymbol{\tau = \sigma n \tan (\varphi r + JRC \log 10 (\ \ /\sigma n))}$$

Where:

τ = shear strength

σn = normal stress

φr = residual friction angle

JRC = joint roughness coefficient

JCS = joint wall compressive strength

For natural, unweathered surfaces the residual friction is approximately equal to the basic friction angle, φb (Bandis, 1993). The basic friction angle is valid for smooth, planar surfaces while the residual friction angle refers to joint surfaces after shear displacement. Furthermore, the criterion considerate the relationship between the friction angle and normal stress acting on the plane, as high levels of normal stress will reduce the friction angle due to shearing of asperities (Wyllie, Mah & Hoek, 2004). The Barton-Bandis criterion is applied in the following chapters to calculate the shear strength for planar and wedge failure mechanisms.

4.3. Input parameters from the field

Geotechnical data of joints was collected during fieldwork, and has been applied for the determination of discontinuity shear strength. This chapter will describe the methods used in the field for determination of input parameters used in the stability analyses.

Field observation

During the visit to the quarry the current state of the excavation works and the shape of the quarry were compared with the given mapping. These maps were obtained from the mining engineers who work the excavations. The excavation works were highly advanced but the slope direction of each face of the quarry was fairly similar to the mapping of the quarry. There are six (6) principal slope dip directions corresponding to the seven sides of the quarry as will be shown in the fllowing figure. In annex 2, the drawings of the quarry can be checked.

Figure 23. General view of the quarry. Picture taken during the visit.

Every face was checked to see the state of the rock mass. In all of them, it was observed that the rock mass was highly fragmented and weathered. Any sign of rock fall or landslide was seen during the visit.

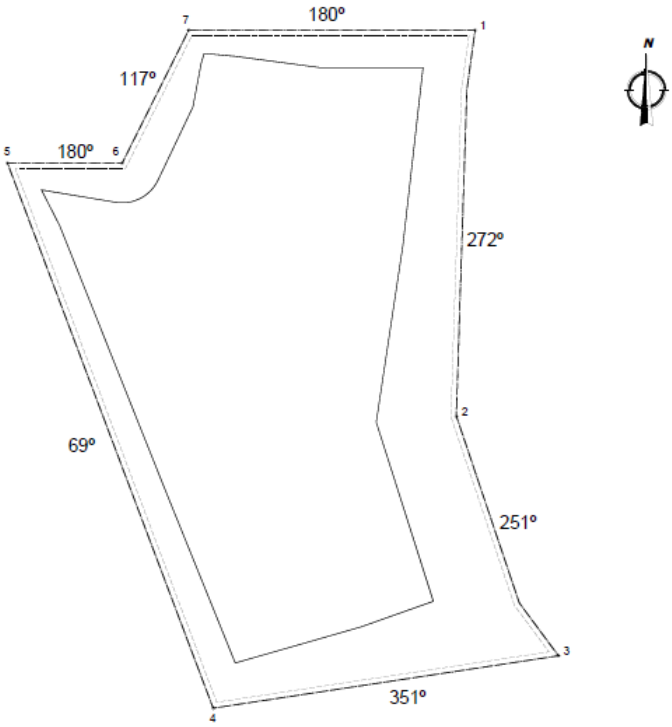


Figure 24. Slope dip directions representation (from the drawings)

Orientation of discontinuities and Joint sets

The calculations of the global stability started with the measurement of the dip angle and dip direction of the discontinuities around all the faces of the quarry. This procedure was made as using a geological compass and a digital compass for smartphones: Rocklogger v1.972.

Figure 25. Geological compass

On the spot, the whole area of the quarry was examined. Dip directions and dip angles of 93 discontinuities were measured and introduced into the Dips software.

Figure 26. Example of the measurements done in the quarry

The stereographic projection allows the representation of the three-dimensional data in two dimensions. The stereographic projection consists of a reference sphere in which its equatorial is horizontal and its orientation is fixed relative to north (Wyllie, Mah and Hoek, 2004). The equatorial projection is the most common one is the equal area or Lambert (Schmidt) net.

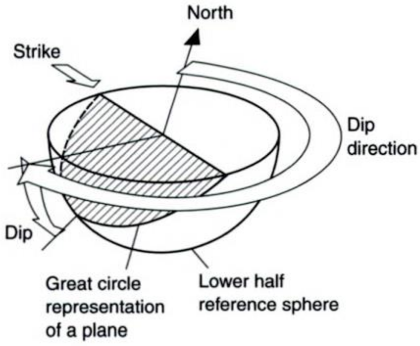


Figure 27. Stereographic representation. (Wyllie, Mah and Hoek, 2004)

This software reproduces a stereographic plot by introducing all the measurements and allows identifying the main join sets and drawing them in order to get the mean values of the dip angle and dip direction of each set.

Once the orientations were in the software, 3 main sets were identified. The characteristics of the different sets are shown in the following figure (in degrees).

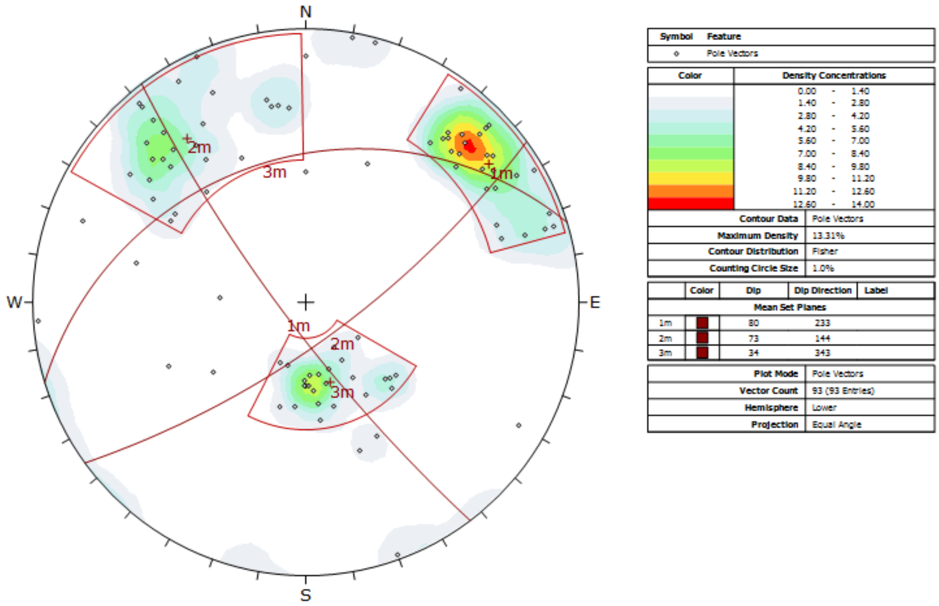


Figure 28. Stereographic projection in Dips software, main joint sets.

Set	Average Dip	Maximum difference	Minimum difference	Average Dip direction	Positive difference	Negative difference
1m	**80**	7	8	**233**	20	18
2m	**73**	16	14	**144**	30	24
3m	**34**	13	9	**343**	40	40

Table 1. Main joint sets

GSI value

From the field observation, the first in-situ parameter determined was the the GSI value (Geological Strength Index). The GSI value depends on the structure of the rock mass and condition of the discontinuity surface. The dolomite rock mass observed in the quarry was highly fragmented and disrupted (Figure 29). The condition of the surfaces was between good and tolerable category. Based on the in-situ observations, the **GSI value for the following calculations will be 35.**

Figure 29. Many quarry observations in situ. Highly fragmented rock mass

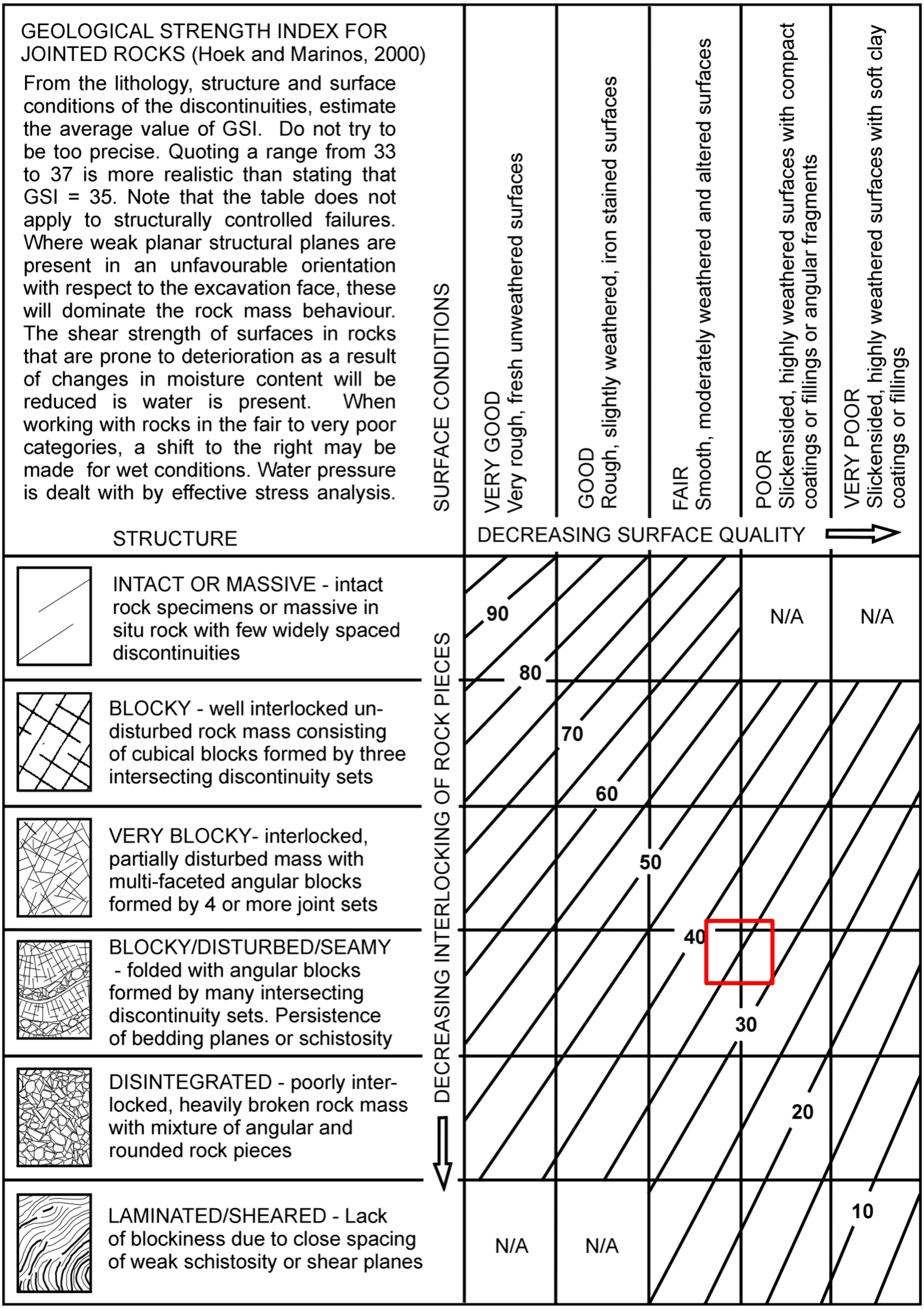


Figure 30. Hoek-Brown failure criterion. Determination of the GSI value.

Joint Roughness Coefficient, JRC

The Joint Roughness Coefficient, JRC, is an empirical index used to characterize the surface roughness following the Barton Bandis criterion explained in the previous chapter. There are several ways to measure this parameter, both in the field and in the laboratory.

In this case, the roughness of the surface was recorded in situ using the Barton comb and the JRC was determined following Barton and Choubey (1977) (Figure 31 and 32) and Barton and Bandis (1982) (Figure 33). The measurements were carried out in four directions; along the dip direction, along the strike and ± 45° relatively to the dip direction. The determined JRC value was 11. And can be seen in table 2 below.

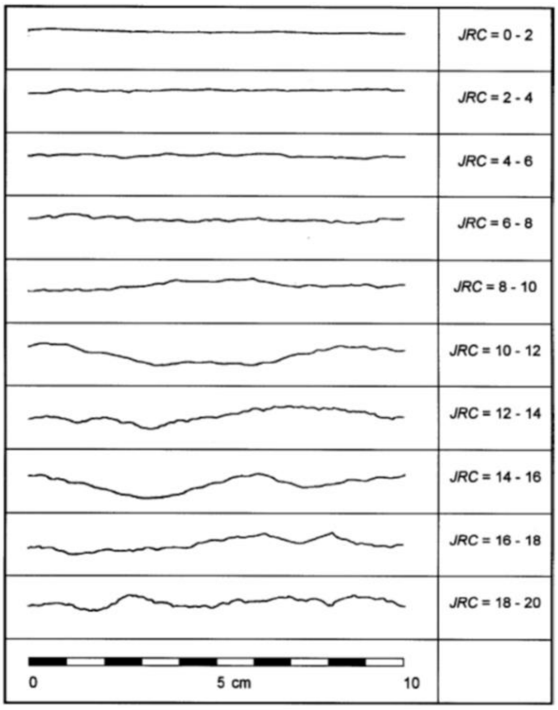


Figure 31. Determination of JRC by Barton and Choubney (1977). (1)

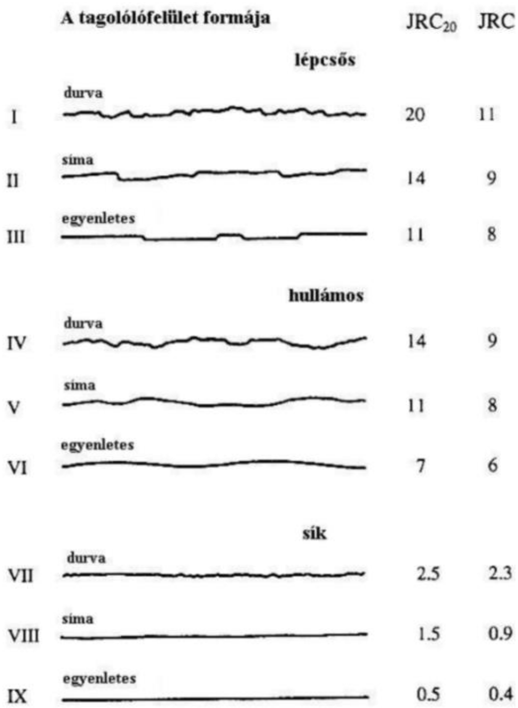
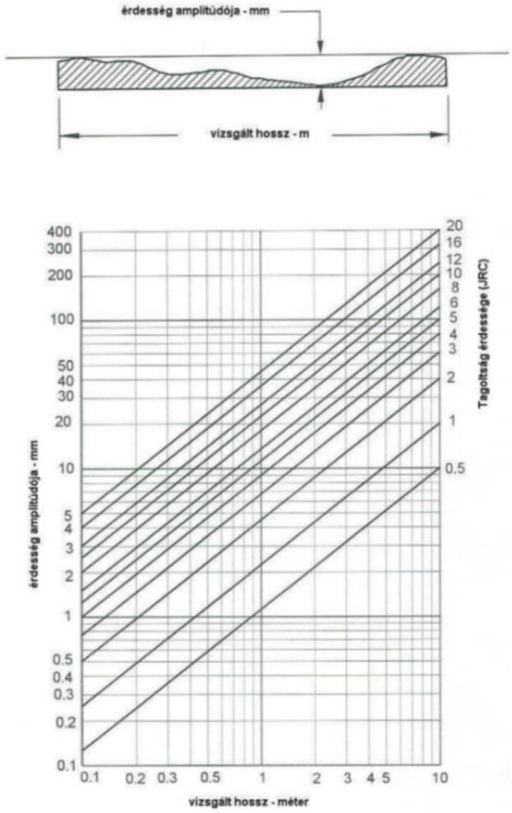


Figure 32. Determination of JRC by Barton and Choubney (1977). (2)



4-9. ábra. Vizsgált tagoltsági hosszon a JRC értékének meghatározása az érdesség amplitúdójának ismeretében (Barton & Bandis, 1982)

Figure 33. Determination of JRC by Barton and Bandis (1982) (3).

Sample	Fig. 4	Fig. 5	Fig.6
1	10	11	20
2	8	8	17
3	16	15	19
4	9	11	9
5	9	11	14
6	11	12	19
7	8	9	18
8	9	9	19
9	9	9	10
10	10	11	20
11	6	9	12
12	6	9	12
13	6	8	8
14	8	11	10
15	8	8	6
Weighted mean		11,04444	
JRC (applied)		11	

Table 2. JRC calculations from the field

Samples acquisition

During the fieldwork in the quarry, three rock samples were collected for laboratory analysis, in order to obtain input parameters related to the strength and deformability of the rock mass.

4.4. Input data from the laboratory testing

Three rock samples were obtained during the field work. Therefore, the laboratory tests were performed to determine the input parameters related to the strength and deformability of the rock mass. These tests were carried out following the recommendations from the Eurocode 7. This chapter will describe the process which was followed to obtain the specimens, perform the laboratory test and finally reach the input parameters for the following stability analysis calculations. **All the results from the laboratory test can be seen in annex 1 of the thesis.**

Drilling of the samples and first measurements

The laboratory testing has been conducted on three rock samples collected from different parts of the quarry. The blocks measure approximately 30 x 20 x 30 cm.

From these blocks ten specimens were drilled and consequently cut with an approximately 1:2 length-diameter ratio for the UCS test and eight more were drilled and cut with a 1:1 ratio for the indirect tensile stress test (Brazilian test) following the Eurocode 7 recommendations.

The diameter of the samples was nominally 37 mm. The Specimens were tested under oven-dry condition and were photographed before and after the tests.

Figure 34. One of the rock samples after drilling

Figure 35. Drilled specimens before cutting.

Figure 36. Cutting of the samples in the laboratory

The specimen volume was determined by measuring the diameter and height (ISRM 1981). The reading accuracy used was 0.01 mm. Test specimens with their dimensions and shape parameters are listed in annex 1.

Ultrasonic velocity measurement

Two orthogonal shear waves are propagated along the longitudinal axis of the sample. The time that takes to the wave to cross the specimen is measured, and the velocity is determined. This test allows the definition of the dynamic elastic properties of the material, including Young's modulus, bulk modulus and Poisson's ratio (MTS).

Uniaxial Compressive Strength Test Results

Uniaxial compression tests are used to determine the complete stress-strain curve for cylindrical intact rock core specimens. The compressive strength refers to the maximum stress that the rock sample can sustain under uniaxial load conditions and it is called uniaxial compressive strength (σucs or σc). It is probably the most widely used rock engineering parameter.

In the uniaxial compression test, the sample, a right-cylindrical specimen of material, is placed in the loading machine between the lower and upper plates. The confining pressure and the pore pressure are zero. Before starting the loading, the upper plate is adjusted to be in contact with the sample and the deformation is set as zero. The test then starts by applying a constant axial strain. The load and deformation values are recorded for obtaining a complete load-deformation curve (Geotechdata, 2017).

Figure 37. Sample number 6 before being tested

Figure 38. The positioning of the sample for the UCS test.

The studies were made based on the suggested methods of ISRM (1979).The stress-strain behaviour of rock samples was studied for a total of 10 uniaxial compression tests. Three different pieces of the rock mass around the quarry were taken to the laboratory and the samples were drilled and cut from the core samples with a 300 mm diameter diamond saw blade.

The uniaxial compression test was carried out at the Laboratory of Geotechnics, Budapest University of Technology and Economics.

The values obtained for the uniaxial compressive strength were in the range of 20.15-76.02 MPa with the mean value of **47.57 MPa**. The large width of the range of the values is due to the fractures of the rock, making some samples very weak for the uniaxial compressive test.

Uniaxial compressive strength tests

	Sample number	Diameter	Height	Area	Volume	Mass	Bulk density	UCS	Elastic modulus
		mm	mm	mm^2	cm^3	g	kg/m^3	MPa	GPa
Oven-dry	1	37,30	72,70	1092,72	79,44	224,06	2820	67,63	30,21
	2	37,03	67,83	1076,95	73,05	205,20	2809	55,83	7,80
	3	37,70	71,84	1116,28	80,19	216,82	2704	52,48	5,81
	4	37,50	70,61	1104,47	77,99	216,76	2779	31,03	7,31
	5	43,30	62,50	1472,54	92,03	258,58	2810	76,02	10,13
	6	43,70	59,90	1499,87	89,84	240,90	2681	20,15	1,83
	7	37,93	72,08	1129,94	81,45	222,56	2733	53,23	11,98
	8	37,58	71,15	1109,18	78,92	209,52	2655	23,02	6,09
	9	37,78	72,38	1121,02	81,14	220,39	2716	33,73	8,22
	10	43,20	48,70	1465,74	71,38	192,13	2692	62,58	7,74
	Mean	39	67	1219	81	221	**2740**	**47,57**	**9,71**
	Std. dev.	3	8	181	6	18	60	19,38	7,69

Table 3. Uniaxial compressive strength results

Unit weight

From the previous calculations for the UCS test, the unit weight is calculated. The following equation is used. Its standard deviation and relatives minimum and maximum were also calculated.

$$\gamma = \rho \times g$$

Sample	Bulk density	Unit weight
1	kg/m^3	MN/m^3
2	2820	0,027669
3	2809	0,027557
4	2704	0,026523
5	2779	0,027267
6	2810	0,027562
7	2681	0,026304
8	2733	0,026807
9	2655	0,026045
10	2716	0,026646
	2692	0,026404
Mean	**2740**	**0,026878**
Std.Dev.	60	**0,000591**

Table 4. Unit weight calculations

Rel. Min	0,000834
Rel. Max	0,000790

Joint Compressive Strength (JCS)

The JCS value will be equal the Unconfined Compressive Strength (UCS) if the surface of the discontinuities is completely unweathered. This is not the case in practice since rock walls generally are weathered to a certain degree. JCS was obtained by probabilistic calculations. The equations used to determine the characteristic UCS value and the JCS value follow the Eurocode 7 and will be shown below.

$$\mathrm{kn} = 1{,}64 * \sqrt{\frac{1}{\mathrm{n}}} \text{ where } \mathrm{n} = \text{number of samples}$$

$$\text{UCS characteristic} = \text{mean} * (1 - \mathrm{kn} * \text{coefficient of variation})$$

$$JCS = 0{,}92 * UCScharacteristic$$

Determination of characteristic value	
Mean:	47,57
Deviation:	19,38
Coefficient of variation	0,4074
kn, stat. par:	0,54666667
characteristic UCS:	36,97
JCS	34,016

Table 5. Determination of the UCS characteristic value

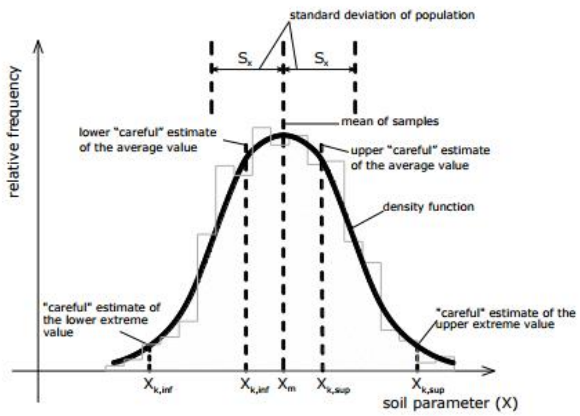


Figure 39. Determination of the UCS characteristic value (Eurocode 7)

Laboratory Brazilian Test

Brazilian Test is a geotechnical laboratory test for indirect measurement of tensile strength of rocks. Indirect methods are used to calculate the uniaxial tensile strength test due to the difficulty to perform it and because the rock does not fail in direct tension in situ conditions.

Due to the simplicity and efficiency of the Brazilian test, it is one of the most commonly used laboratory testing methods in the geotechnical investigation in rocks. The test is sometimes used also for concrete.

In the Brazilian test, calculated following the ISRM (1978), a disc shape specimen of the rock is loaded by two opposing normal strip loads at the disc periphery. The specimen length/diameter ratio is approximately equal to one. The load is constantly increased until failure of the sample occurs. With this test, the loading is obtained and it is used to calculate the tensile strength of the rock as follows.

$$t=2P/(\pi DL)$$

P= applied load

D=diameter of the sample (D=2R)

L =thickness of the sample

The axial compression test was also carried out at the Laboratory of Geotechnics, Budapest University of Technology and Economics with the same machine than the one for the UCS test.

Figure 40. Picture from the laboratory tools for UCS and Brazilian test.

The values for the tensile strength obtained were in the range of 2.77- 6.66 MPa with the mean value of 4.73MPa.

Indirect tensile strength tests

	Sample number	Diameter	Height	Area	Volume	Mass	Bulk density	Tensile strength
		Mm	Mm	mm^2	cm^3	g	kg/m^3	MPa
Oven-dry	11	37,93	34,60	1129,94	39,10	106,39	2721	3,44
	12	38,05	32,50	1137,10	36,96	103,68	2806	4,48
	13	37,47	34,19	1102,70	37,70	103,06	2734	5,25
	14	37,72	32,06	1117,46	35,83	97,91	2733	5,92
	15	37,91	33,09	1128,75	37,35	102,04	2732	4,79
	16	37,75	33,34	1119,24	37,32	98,65	2644	2,77
	17	37,71	32,60	1116,87	36,41	99,37	2729	4,53
	18	37,95	34,56	1131,13	39,09	105,52	2699	6,66
	Mean	38	33	1119	37	101	2712	**4,73**
	std. dev.	0	1	10	1	3	36	1,25

Table 6. Indirect tensile strength test results

Friction angle

The friction angle is a shear strength parameter of rocks and soils. It is defined as an input parameter from the Mohr-Coulomb failure criterion and it is used to describe the friction shear resistance of rock masses or soils. The basic friction angle is the angle of inclination with respect to the horizontal axis of the Mohr-Coulomb shear resistance line.

For determining the friction angle, the Hoek and Brown criteria has been followed. The empirical studies of Hoek and Brown have demonstrated that the value of friction angle is similar depending on the type of rock. They develop a list of the approximate values for basic friction angle for the principal type of rock. According to the next table, the friction angle of the dolomite is between 27º and 31º.

For the following calculations, **a friction angle of 27º will be used.**

Approximate values for the basic friction angle ϕ for different rocks.

Rock	ϕ-degrees
Amphibolite	32
Basalt	31 - 38
Conglomorate	35
Chalk	30
Dolomite	27 - 31
Gneiss (schistose)	23 - 29
Granite (fine grain)	29 - 35
Granite (coarse grain)	31 - 35
Limestone	33 - 40
Porphyry	31
Sandstone	25 - 35
Shale	27
Siltstone	27 - 31
Slate	25 - 30

Lower value is generally given by tests on wet rock surfaces.

After Barton[82].

Figure 41. Approximate values for the friction angle (Wyllie, Mah and Hoek, 2004)

4.5. Chapter overview and determined parameters

The main objectives of this chapter have been: firstly, to set out the calculation methods that will be used to determine the geometry of the slope of the quarry and secondly, to obtain the parameters needed to carry out these calculations.

A summary of all the parameters obtained will be shown below. These results are summarized in tables and they are divided according to the method in which they are needed.

The analysis methods determined are **Kinematic and Limit Equilibrium** analysis: a kinematic and a limit equilibrium for planar failure and the same for wedge failure. Lastly, a global rotational analysis is carried out.

To carry out all the calculations, first of all, the slope dip directions have been determined for each side of the quarry, the following table shows these dip directions. Also, the maximum height was obtained. In Annex 2 a drawing with the slope dip directions and maximum heights of the quarry is attached.

Section of the quarry	Slope dip direction	Maximum Slope height
7-1, 5-6	180º	30 m (both)
6-7	117º	30 m
1-2	272º	60 m
4-5	69º	22,5 m
2-3	251º	65 m
3-4	351º	65 m

Maximum slope height	65 m

Table 7. Parameters obtained from the drawings

Planar and Wedge failure mechanisms. Kinematic analyses:

Main Joint sets						
Set	**Dip**	**Maximum difference**	**Minimum difference**	**Dip direction**	**Maximum difference**	**Minimum difference**
1m	**80**	7	8	**233**	20	18
2m	**73**	16	14	**144**	30	24
3m	**34**	13	9	**343**	40	40
Friction angle			27º			

Table 8. Input date for Kinematic analyses

Planar and Wedge failure mechanisms. Limit Equilibrium analyses:

Barton and Bandis method. Input parameters	
Parameter	**Value**
JRC	11
JCS	34,01603 MPa
Friction Angle	27º
Unit weight	0,026878 MN/m^3

Table 9. Barton and Bandis method input parameters for LEM analyses

Rotational mechanism. Global analysis:

Generalized Hoek and Brown method. Input parameters	
Parameter	**Value**
UCS	34,01603 MPa
GSI	35
Elastic modulus	9,71 GPa
Unit weight	0,026878 MN/m^3

Table 10. Generalized Hoek and Brown method input parameters for the global analysis

5. Stability analysis using the Rocscience software

5.1. Introduction

For determining the geometry of the quarry (slope angle) a kinematic analysis will be first done, as it was explained in the previous chapter, in order to check the rock slope stability on the stereonet plot into the Dips software for:

- Planar sliding
- Wedge sliding

Once the kinematic analysis is done, an equilibrium analysis will be carried out to complete the analysis and finally determine the maximum safe angle for the slope.

The Rocscience Incorporation reports with the analysis information of the multiple calculations is attached in Annex 3.

5.2. Planar Failure Analysis

As it was explained in the chapter 2 (Failure mechanisms) of this document, the planar failure also known as planar sliding is a test for sliding on a single plane. The planar sliding will be first calculated by a kinematic analysis: using the pole vector mode (normal vectors) in the Dips software to get the critical failure zones of each face of the quarry. Once the critical failure planes are obtained, an equilibrium analysis will be developed with the Rocplane software. It will determine the maximum angle with no probability of failure for each critical failure plane. The exact procedure used in both programs will be explained in the following paragraphs.

To carry out the kinematic analysis, which means the determination of the critical failure planes of planar sliding, it is necessary to retake the information from the chapter 4.3 of this document: Joint sets. In that chapter, all the orientations of the discontinuities were introduced to the dips software and the main join sets were determined.

Then, it is also needed to define the daylighting condition for planes and the pole friction cone. Additionally, the lateral limits can be also introduced.

- The **Daylight envelope** is determined by the slope dip direction, all pole vectors that are inside the region of the Daylight Envelope represent planes which can kinematically daylight from the slope.

- The **Friction Cone** represents the friction angle of the rock mass, which was determined theoretically as it was explained in the last chapter. It defines the limits of frictional stability on the stereonet. All poles outside of the pole friction cone represent planes which dip steeper than the friction angle and can slide if kinematically possible.
- The **Lateral Limits** estimate a certain angular range of the slope dip direction in which is possible to occur the planar failure. Typically a value of 20 to 30 degrees is used based on empirical observations (Goodman 1980, Hudson and Harrison 1997). In this case, it was used a **lateral limit of 20º degrees**.

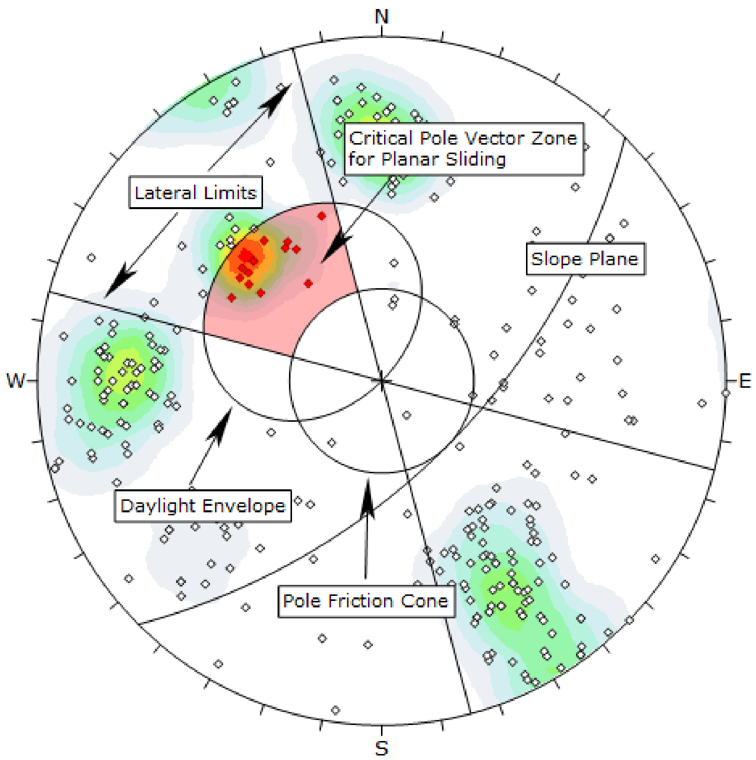


Figure 42. Kinematic Analysis Overview. Planar Sliding main elements to be defined. From Rocscience.com. (2017).

The critical zone of planar failure will be in the red area of the plot which means: inside the daylight envelope and outside the pole friction cone and inside the lateral limits.

To start the analysis a steep angle of 70 degrees was introduced in the program as slope dip angle. The 7O degrees slope angle is not meaningful, it is just used as a very steep angle to start the calculations and the main asset of the project is to decrease this value until a safe value is reached. The procedure which explains the reduction of this value will be explained below.

Taking this approximate slope angle value of 70º, every face of the quarry was calculated using the Dips' planar sliding option to observe the possibility of planar failure on each slope face (by introducing the slope dip direction of each face). The following images show the results of each face of the quarry:

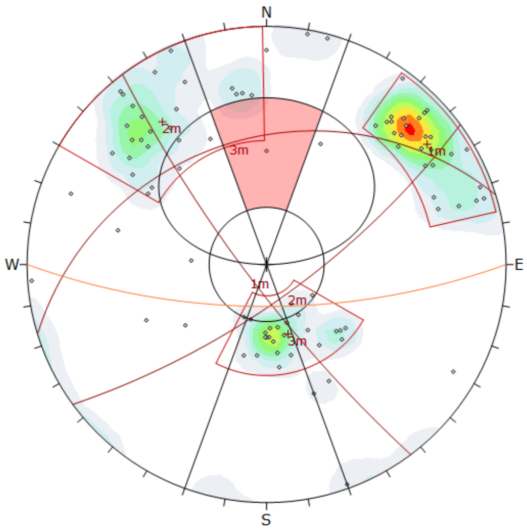


Figure 43. Planar failure 180º

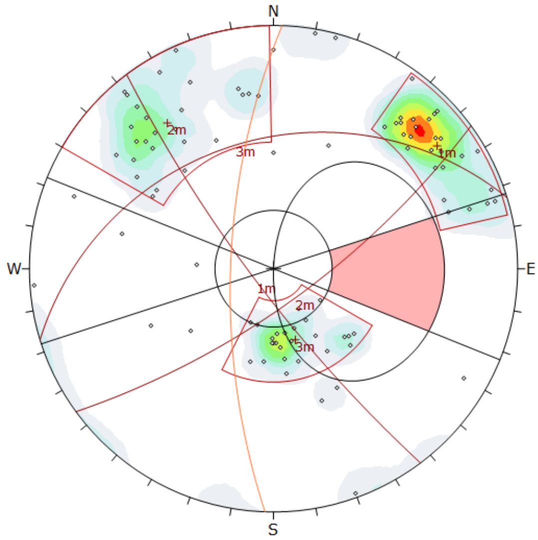


Figure 44. Planar failure 272º

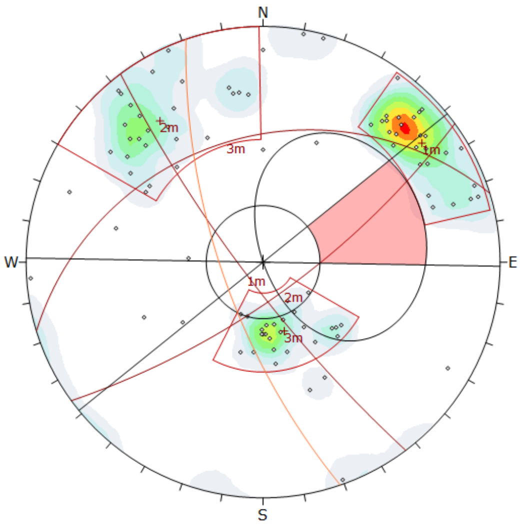


Figure 45. Planar failure 251º

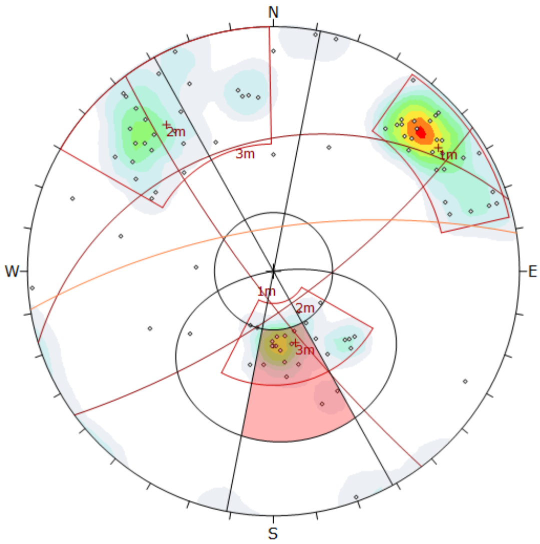


Figure 46. Planar failure 351º

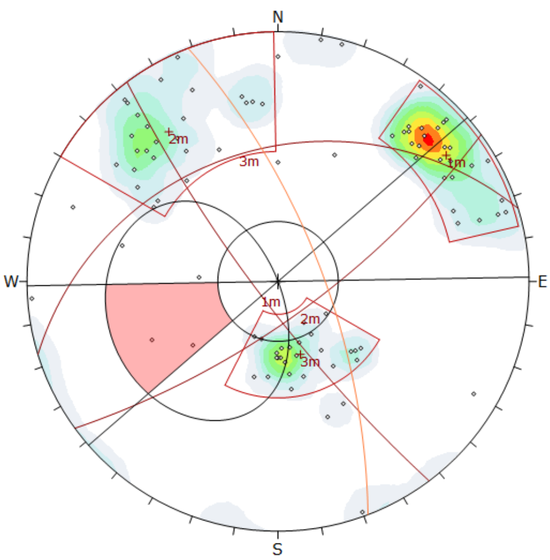


Figure 47. Planar failure 69º

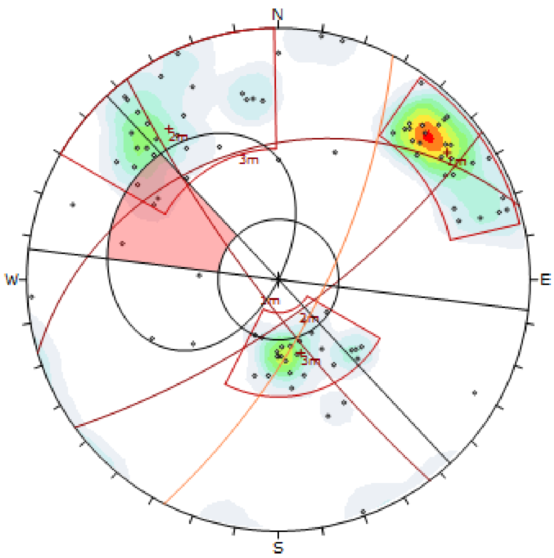


Figure 48. Planar failure 117º

The following results shown on the table were obtained from the Dips software, as it can be seen in the plots.

Section of the quarry	Slope dip direction	Critical failure zone
7-1, 5-6	180º	2m
6-7	117º	2m
1-2	272º	-
4-5	69º	-
2-3	251º	-
3-4	351º	3m

Table 11. Results from Dips software for Planar failure.

Once the critical failure planes of each side of the open pit were determined, the next step was to calculate the probability of failure of each critical failure plane using the equilibrium analysis with the RocPlane software.

The RocPlane analysis is a 2-dimensional analysis, where it is assumed that the strike of the face slope, upper slope, failure plane and the tension crack, are parallel, or nearly parallel – within approximately plus or minus 20 degrees, in order for the analysis to be applicable.

According to the kinematic study, there are 3 critical failure planes on 3 different sides of the quarry where the planar of the rock mass is possible, so the equilibrium test is to be checked for their stability. The calculations were made using the safety factors prescribed by Eurocode 7.

Eurocode 7 is a design document which establishes standards for geotechnical engineering design in Europe. With Eurocode 7, partial factors of safety are applied to different components of the analysis. The partial factors are applied prior to the analysis to give design values that are used in the calculation. The final result is an over-design factor, which must be greater than 1 to ensure the serviceability limit state requirement is satisfied. For more information about using Eurocode 7 in geotechnical design see the references at the end of the thesis.

Permanent Actions (A)		
Unfavourable	γ_G	1
Favourable	γ_G	1
Variable Actions (A)		
Unfavourable	γ_Q	1.3
Favourable	γ_Q	0
Material Parameters (M)		
Effective cohesion	$\gamma_{c'}$	1.35
Coefficient of shearing resistance	γ_{ϕ}	1.35
Weight density	γ_{γ}	1
Shear strength (other models)		1.35
Resistance (R)		
Earth resistance	γ_{Re}	1
Anchorage (R)		
Bolt tensile capacity	γ_a	1.1
Seismic		
Seismic Coefficient		1
Water		
Water Pressure		1

Figure 49. Eurocode 7 factors of safety

During the equilibrium analysis, the main aim is to reduce the slope angle (a first approximation of 70º) to achieve safety conditions for planar sliding, which means that the probabilities of failure are equal to zero.

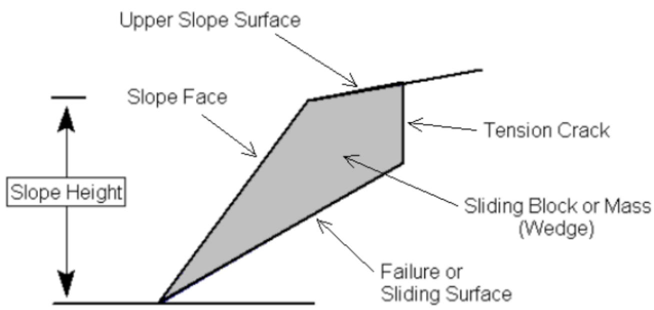


Figure 50. Wedge failure model in RocPlane (Rocscience, 2017)

The input data needed to develop the equilibrium analysis of planar failure is:

- **Slope angle**: refers to the maximum inclination of the walls of the quarry and it is our main asset in this project. As it was mentioned before, an approximate value of 70º has been chosen to start the calculations and it will be decreased until the probability of failure is equal to 0.

- **Unit weight**. The unit weight was obtained from the mean bulk density of samples for the UCS test. Its standard deviation and relatives minimum and maximum were also calculated.

	Bulk density	Unit weight
Mean	**2740**	**0,026878**
Std.Dev.	60	**0,000591**

Rel. Min	0,000834
Rel. Max	0,000790

Table 12. Unit weight

- **Height of the quarry**. From the drawings of the open pit mine, a maximum height of approximately 65 meters for the 3-4 wall and 30 meters for bothe 7-1 and 5-6 walls was obtained from the drawings (annex 2). See the table below.
- **Upper face angle**. That angle refers to the inclination of the top of the slope. It was obtained also from the drawings with an approximate value of 10º.
- **Benches**. No benches were considered because the aim is to calculate the overall angle.
- **Data of from the critical failure planes**. From the previous calculations it is necessary to consider the following tables:

Section of the quarry	Slope dip direction	Slope maximum height	Critical failure zone
7-1, 5-6	180º	30 m (both)	2m
6-7	117º	30 m	2m
1-2	272º	60 m	-
4-5	69º	22,5 m	-
2-3	251º	65 m	-
3-4	351º	65 m	3m

Table 13. Input parameters for planar failure, LEM analysis (1)

Set	Dip	Maximum difference	Minimum difference	Dip direction	Maximum difference	Minimum difference
1m	**80**	7	8	**233**	20	18
2m	**73**	16	14	**144**	30	24
3m	**34**	13	9	**343**	40	40

Table 14. Input parameters for planar failure, LEM analysis (2)

These results show that joint sets 2m and 3m are critical failure planes its date will be introduced in the RocPlane software.

- **Strength.** A critical assumption in planar slope stability analysis involves the shear strength of the sliding surface. There are several models in rock engineering that establish the relationship between the shear strength of a sliding surface and the effective normal stress acting on the plane. In this case, the Barton-Bands model will be used to determine the shear rock strength.
  The Barton-Bandis strength model establishes the shear strength of a failure plane as:

$$\tau = \sigma_n \tan\left[\phi_r + JRC \log_{10}\left(\frac{JCS}{\sigma_n}\right)\right]$$

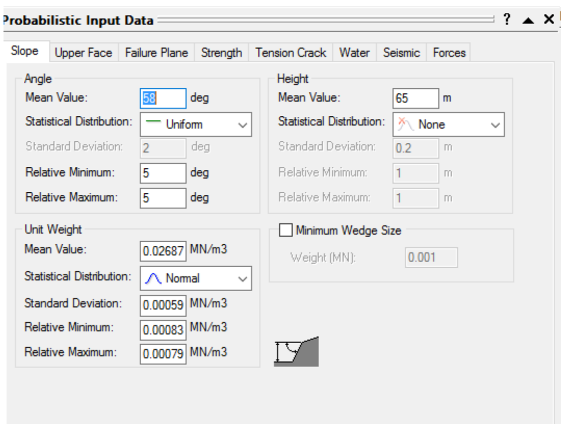
Input parameters for the Barton-Bandis model were obtained in the last chapter from laboratory and field investigations. These parameters are:

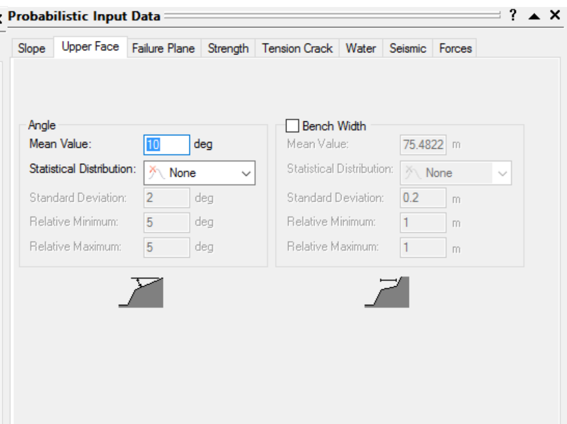
Parameter	Value
JRC	11
JCS	34,01603 MPa
Friction Angle	27º

Table 15. Input parameters for planar failure, LEM analysis (3)

- **Waviness Angle.** Waviness is specified as the average dip of the failure plane, minus the minimum dip of the failure plane. A waviness angle greater than zero will increase the effective shear strength of the failure plane.

The following figures show the windows of RocPlane where it is necessary to introduce all the data explained above.





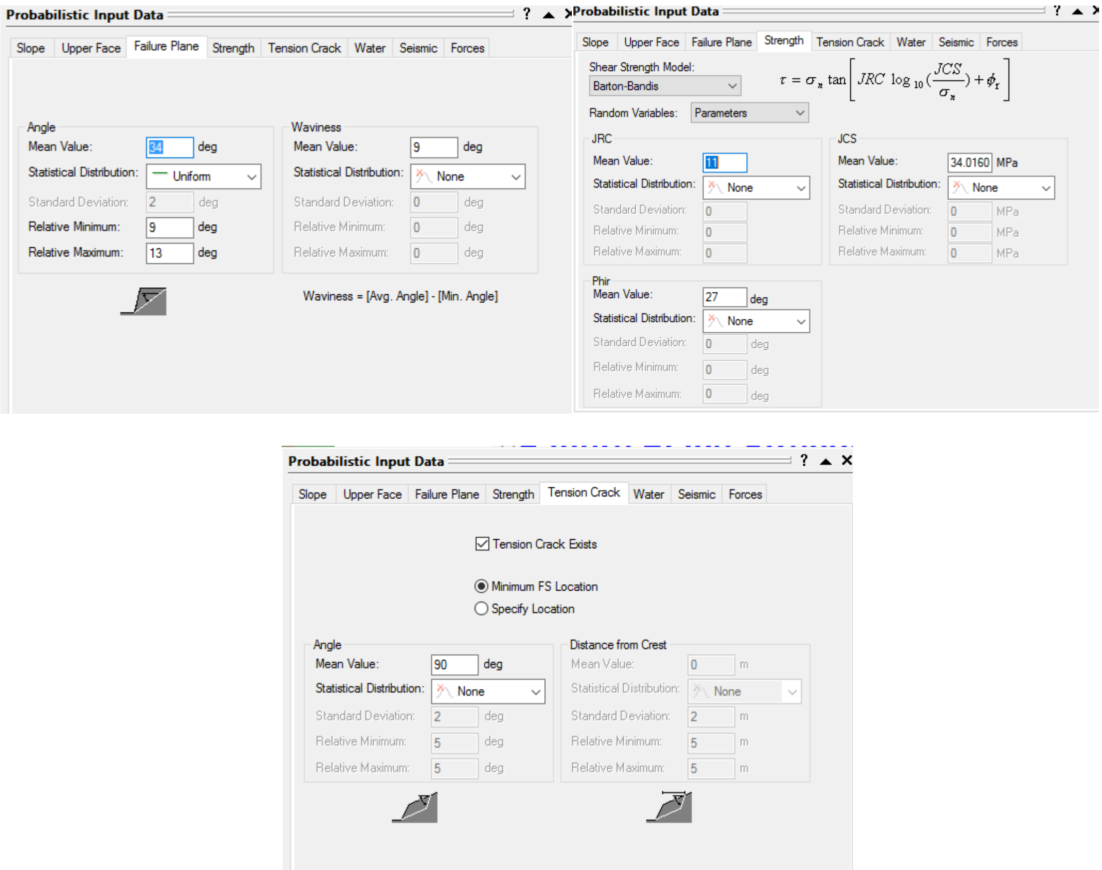


Figure 51. Input parameters for RocPlane

The first calculation was made for the joint set 3m (dip angle=34º), 65 m height and a slope angle of 70º. The results showed a probability of failure of 8,11%, due to this big probability, the slope angle was decreased until it was 0%. The maximum slope angle was 58º.

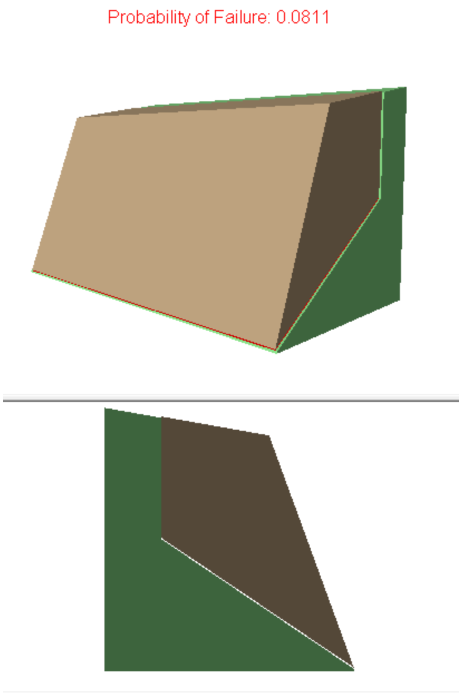


Figure 52. 70º Probability of failure of 8,11%

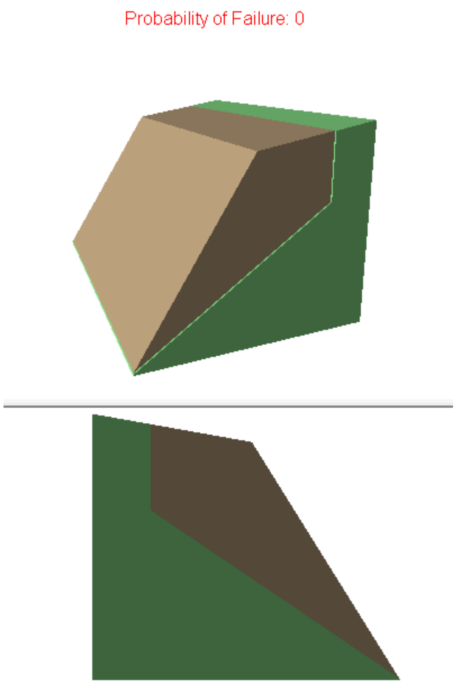


Figure 53. 58º Probability of failure of 0%

The new safest slope angle will be 58º for the following calculations.

The same procedure was done again for the joint set 2m (dip angle=73º), a slope height 30 meters and a slope angle of 58º. The results showed a probability of failure of 0,04% and the slope angle was decreased until it was 0% probability of failure. **<u>The final maximum slope angle was 57º.</u>**

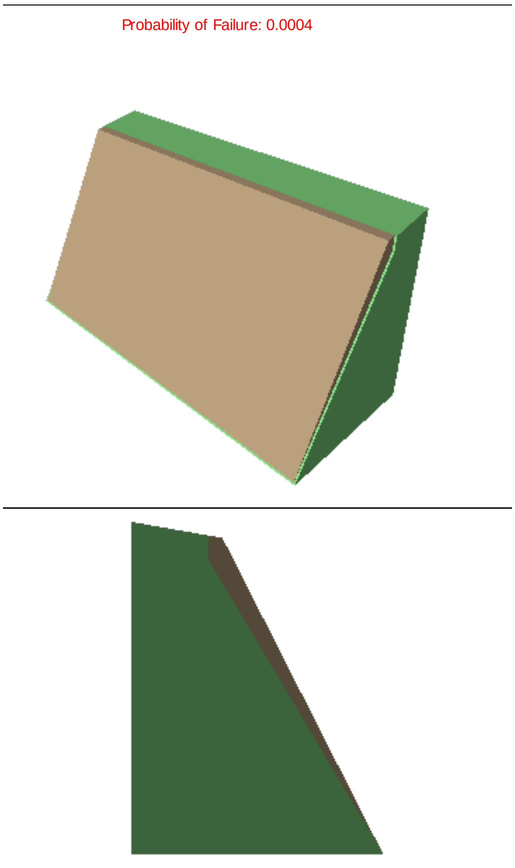


Figure 54. 58º Probability of failure of 0,04%

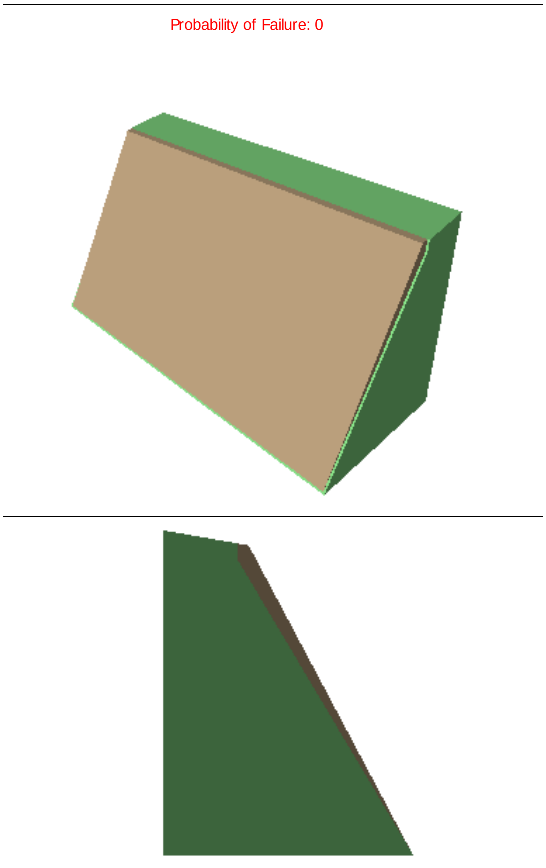


Figure 55. 57º Probability of failure of 0%

5.3. Wedge Failure Analysis

Continuing with the maximum safe slope angle calculation the wedge failure analysis is performed. As it was explained in chapters 2 and 4 of this document, the Wedge Sliding kinematic analysis failure mode is a test for sliding of wedges formed by the intersection of two planes. When two joint planes intersect, they can form a wedge which can slide out of the slope.

Wedge sliding kinematic analysis is based on the analysis of intersections. If an intersection point satisfies the frictional and kinematic conditions for sliding then it represents a risk of wedge sliding. The Wedge sliding analysis will be first computed into the Dips software in order to obtain the critical failure intersections of each side of the quarry. Once the critical failure intersections are obtained, an equilibrium analysis will be developed with the Swedge software. With this program, the maximum safe slope angle with no probability of failure for each critical failure intersection will be determined. The exact procedure used in both programs will be explained in the following paragraphs.

To carry out the kinematic analysis, which means the determination of the critical failure intersections for wedge sliding, the main elements are:

- **Slope angle**. 57º will be used as it was the maximum safe angle obtained in the previous calculations.
- **Intersection plotting**. In this case, the main sets intersections will be taken into account.
- **Friction angle**. As in the last calculations, it will be 27º.
- **Lateral Limits**. Just like in the last calculation, they estimate a certain angular range of the slope dip direction in which is possible to occur the wedge failure. The typical value is 17 degrees (Goodman 1980, Hudson and Harrison 1997).

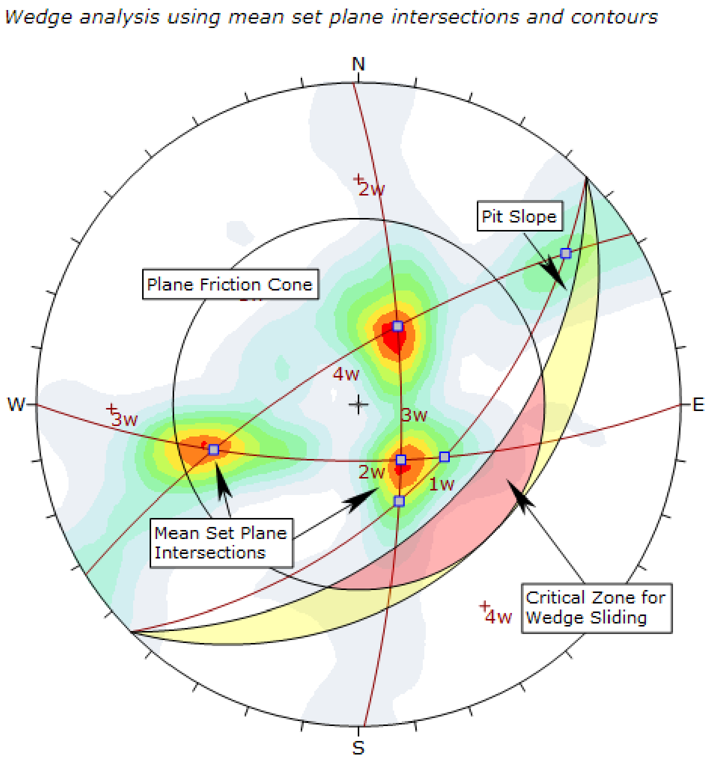


Figure 56. Wedge interpretation in Dips software

The critical zone of wedge failure will be in the red area of the plot which means: inside the friction cone and outside the slope plane.

Taking this approximate slope angle value of 57º, and by introducing the dip directions of every face of the quarry the following images showing the results were obtained:

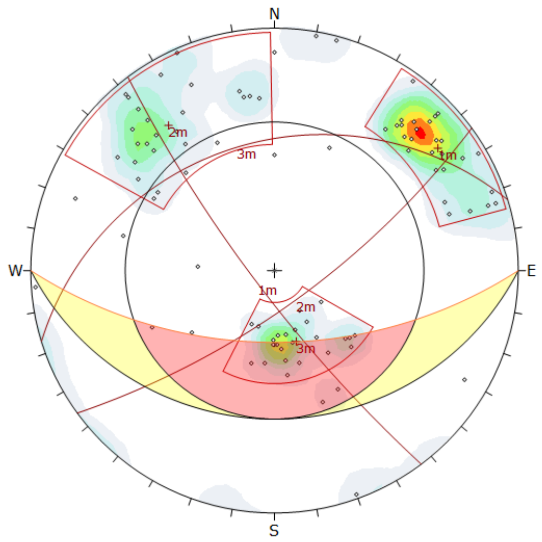


Figure 57. Wedge failure 180º.

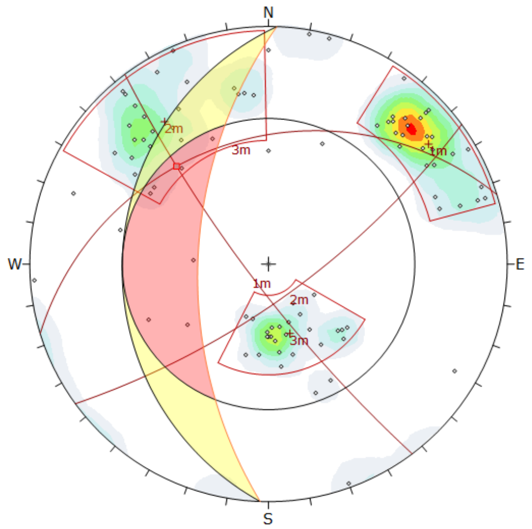


Figure 58. Wedge failure 272º.

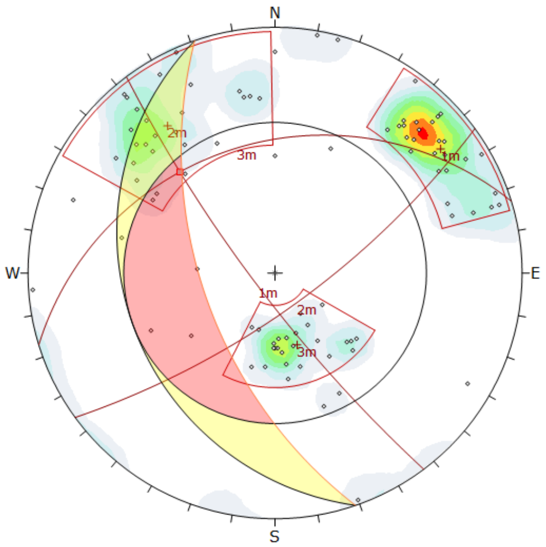


Figure 59. Wedge failure 251º.

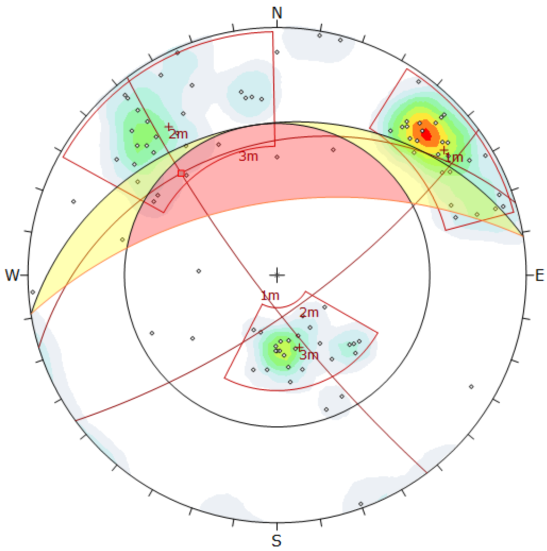


Figure 60. Wedge failure 351º.

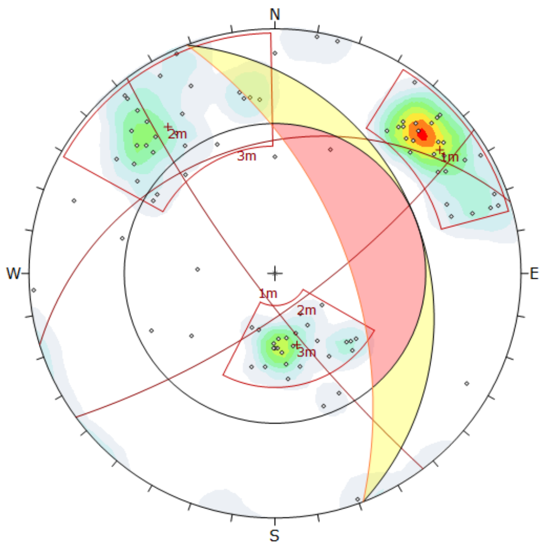


Figure 61. Wedge failure 69º.

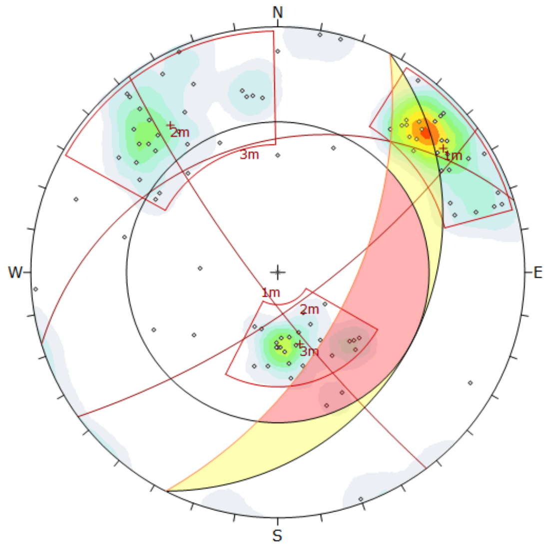


Figure 62. Wedge failure 117º

The following results shown on the table were obtained from the Dips software, as it can be seen in the plots.

Section	Slope dip	Slope height [m]	Combination
7-1, 5-6	180°	30 m (both)	-
6-7	272°	30 m	1m-3m
1-2	251°	60 m	1m-3m
4-5	351°	22,5 m	1m-3m
2-3	69°	65 m	-
3-4	117°	65 m	-

Table 16. Results from Dips software for Wedge failure

Set	Dip	Maximum difference	Minimum difference	Dip direction	Maximum difference	Minimum difference
1m	**80**	7	8	**233**	20	18
2m	**73**	16	14	**144**	30	24
3m	**34**	13	9	**343**	40	40

Table 17. Critical failure planes for Wedge failure

Once the critical failure intersections of each side of the open pit were determined, the next step was to calculate the probability of failure of each critical failure intersection using the equilibrium analysis with the Swedge software.

Swedge computes the probability of failure of a tetrahedral wedge formed in a rock slope by two intersecting discontinuities, the slope face, the upper ground surface, and a tension crack (if any).

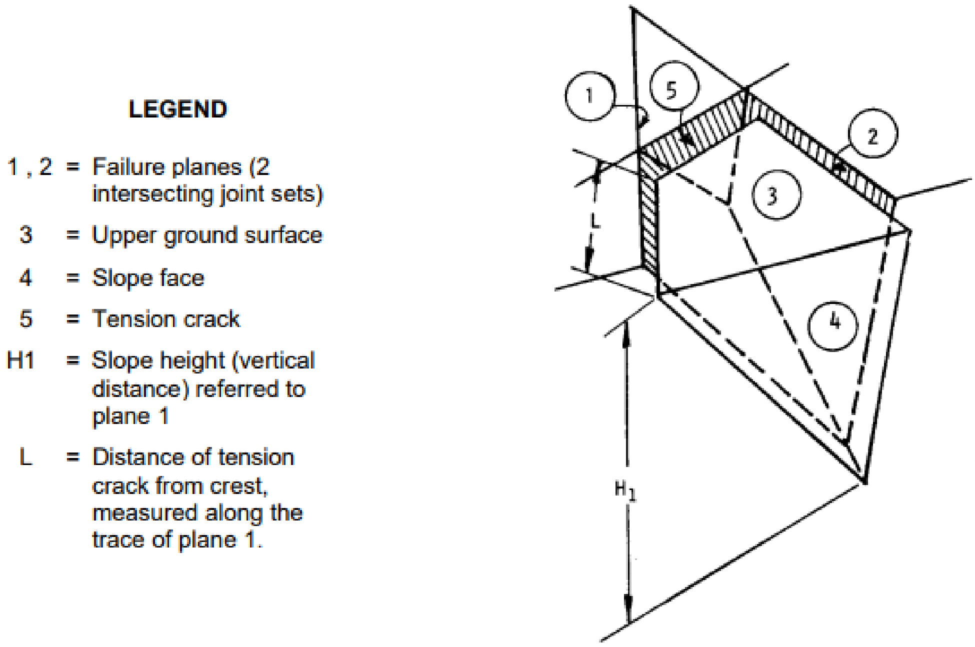


Figure 63. Wedge geometry for Swedge analysis. (Rocscience, 2017)

According to the kinematic study, there are 1 critical failure intersections on 3 different sides of the quarry where the wedge failure is possible, so the equilibrium test is to be checked for their stability. The calculations were made, as the previous ones, using the partial safety factors prescribed by Eurocode 7.

The input parameters for the Wedge failure equilibrium analysis are:

- **Slope parameters**. It refers to the parameters of each side (B, D) critical for wedge failure, the input parameters are the following:
  - **Dip angle**: Maximum dip angle obtained from the previous parameters (57º).
  - **Dip direction**: dip direction of the particular wall (272º, 251º and 351º).
  - **Slope height**: 30 meters, 60 meters 22.5 meters respectively.
- **Upper face parameters.** Means the dip (inclination) of the upper face of the slope.
- **Joints data.** For each critical joint set which forms the intersection, dip angle/dip direction and its strength parameters (Barton and Bandis Model) are required.

The input data for the 272º slope direction face of the quarry is shown below. The calculations for the 251º and 351º face will be equal.

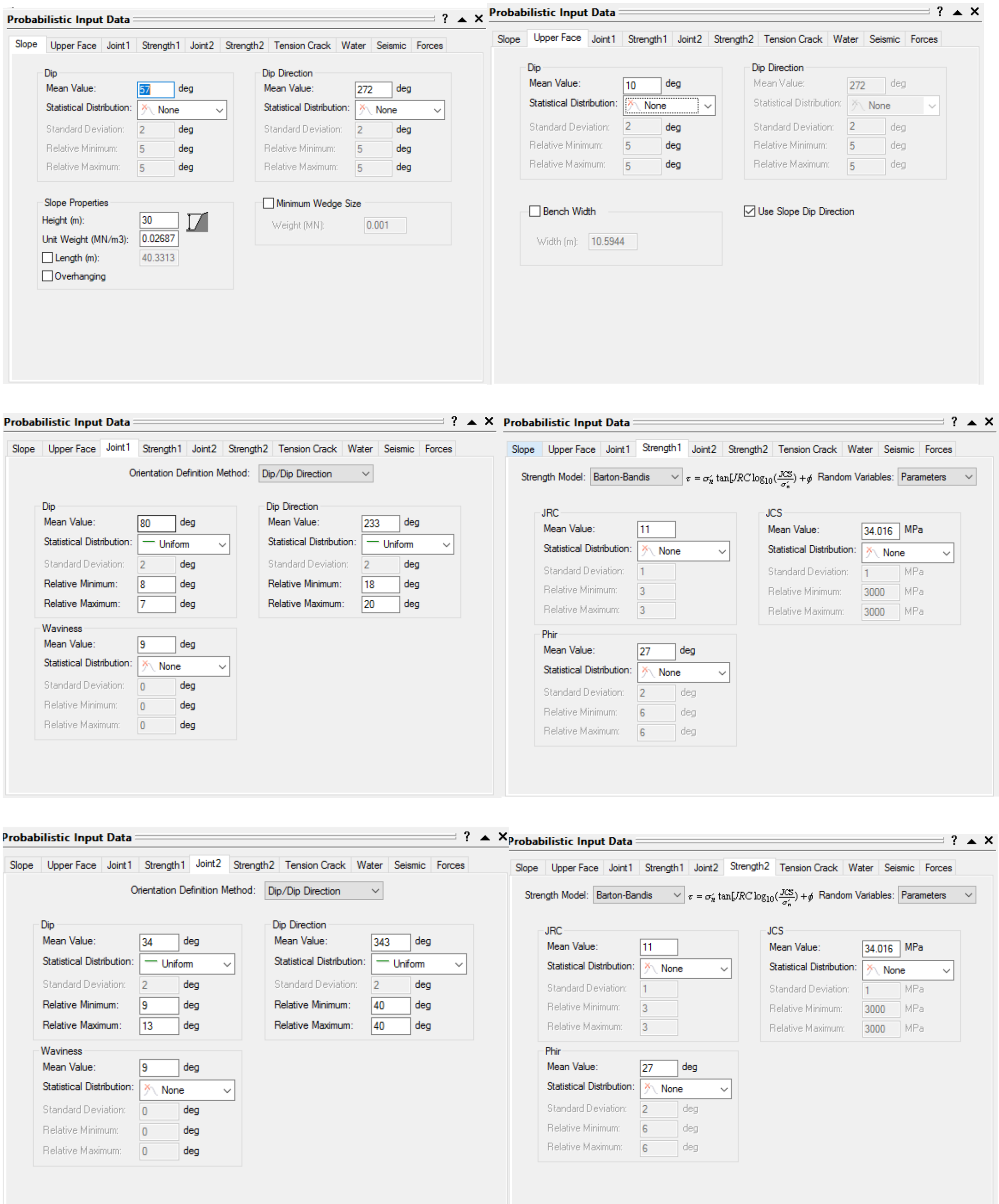


Figure 64. Input parameters for Swedge

Finally, the probability of failure of each slope is obtained. As it is shown in the following figures, the probability of failure for 251º, 272º and 351º slope directions are zero. This result **means that the maximum safe slope dip angle does not vary and it is 57º.**

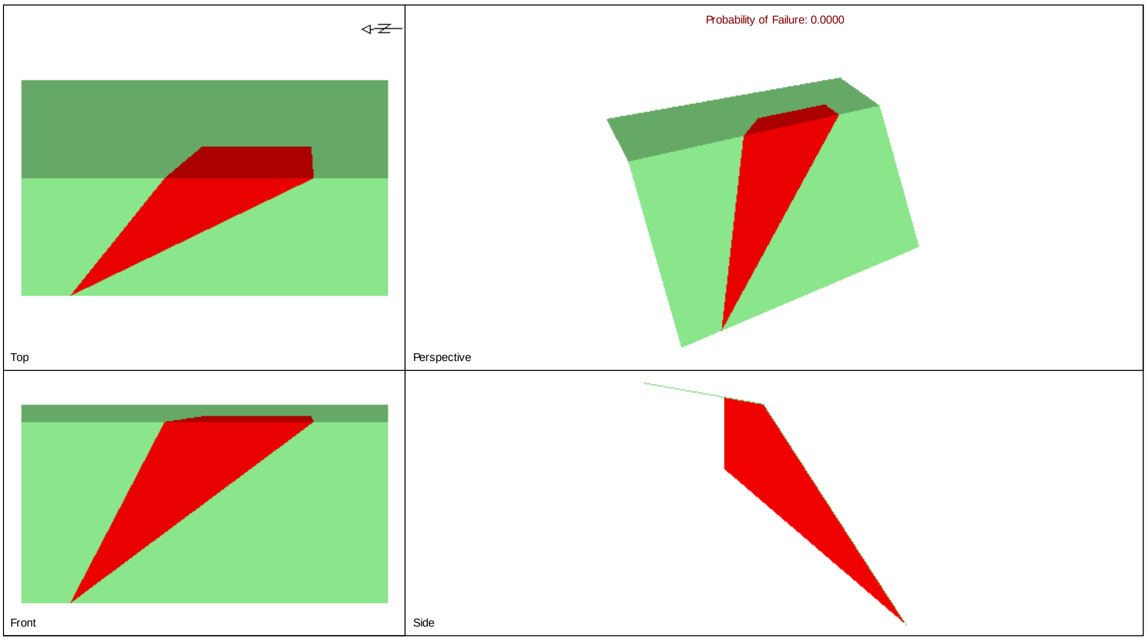


Figure 65. Wedge failure results from 272º and 30 m height. Probability of failure 0%

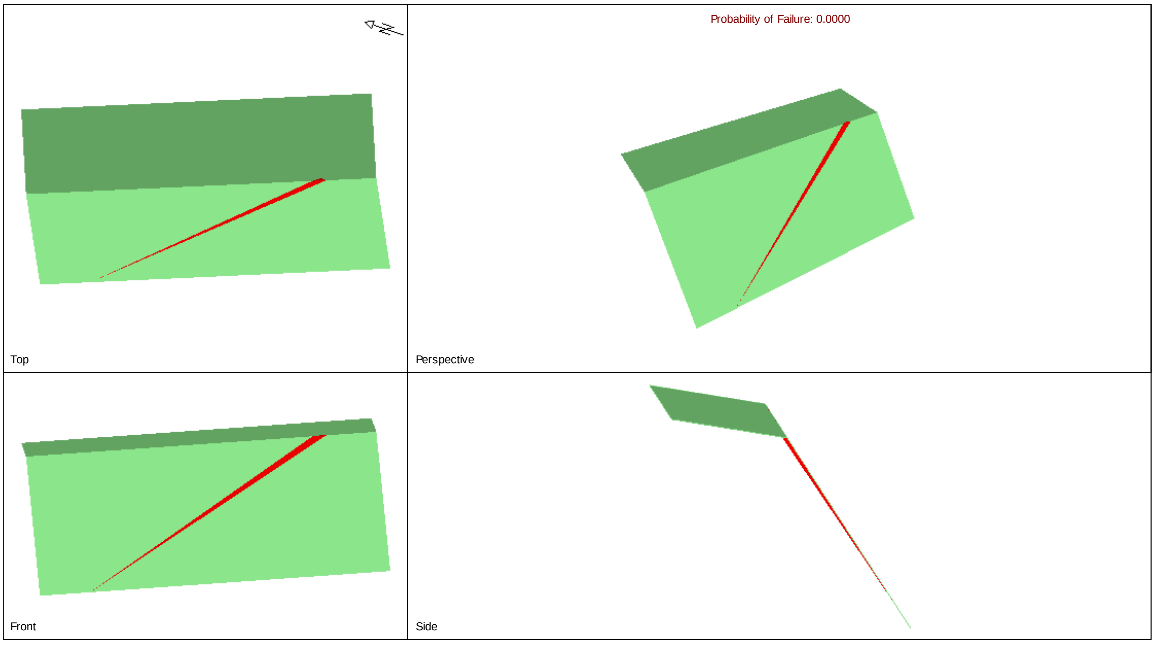


Figure 66. Wedge failure results from 251º and 60 m height. Probability of failure 0%

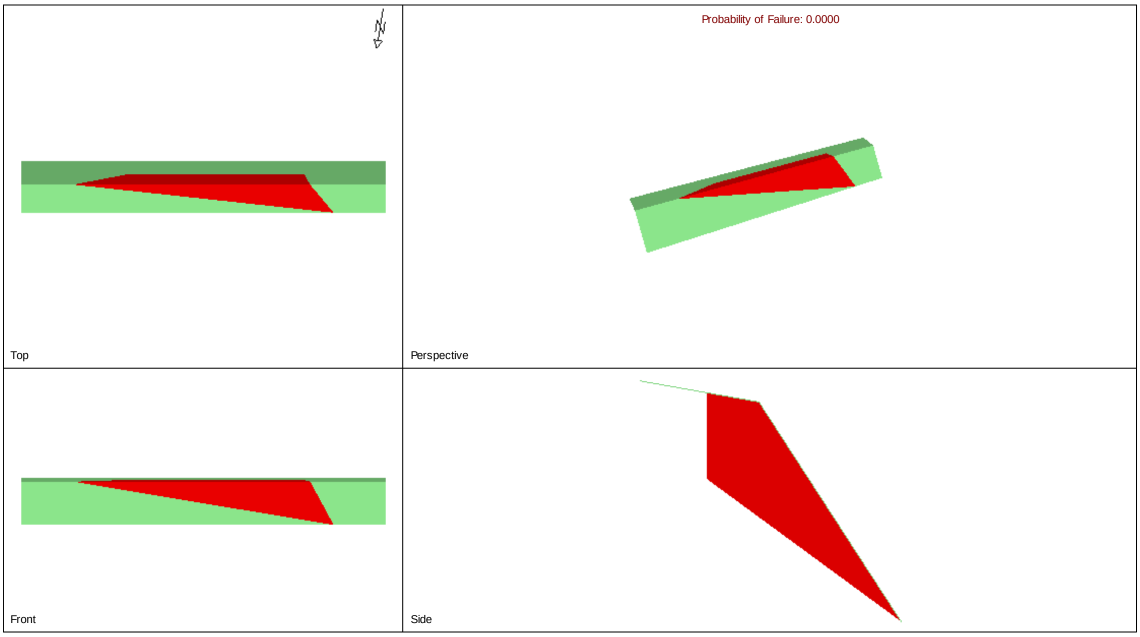


Figure 67. Wedge failure results from 351º and 22.5 m height. Probability of failure 0%

5.4. Global analysis

Once the kinematic and equilibrium analyses have been made for both planar and wedge failure mechanisms, it is necessary to ensure the safety of the slope angle by performing a global analysis of the final geometry.

In calculating global stability, the previous discontinuities were not taken into account because they were already checked in the previous chapters. Since the structure of the rock mass is fragmented, we can consider it homogenise, so in this case, the stability of all the slope geometry as a homogeneous mass was performed by taking into account the shear strength characteristics of the rock. The calculations also include the partial safety factors prescribed by Eurocode 7.

The global analysis will be calculated in the Slide software program of Rocscience. Slide is a 2D limit equilibrium slope stability program for evaluating the probability of failure, of circular or non-circular failure surfaces in soil or rock slopes. All types of soil and rock slopes, embankments, earth dams, and retaining walls can be analysed.

For the test, the strength parameters of the rock mass and the input data of the mode shall be determined on the basis of the Hoek-Brown failure criteria, which require the GSI of the rock mass, and the strength characteristics and bulk density of the rock.

The missing data in order to perform the global stability analysis was obtained using the RocData software of Rocscience. This program gives all the Hoek – Brown parameters when introducing the following input parameters:

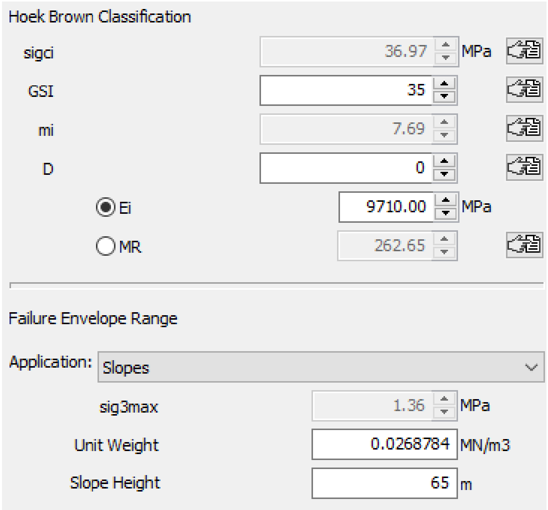


Figure 68. RocData input parameters

- **Compressive strength**. This value was obtained in the laboratory as can be seen in the previous chapter.
- **GSI value.** This value was also explained in the previous chapters an refers to the degree of fragmentation of the rock mass.
- **mi.** The mi value for each rock type comes from normalizing multiple good quality datasets by the division of sigci, and then curve fitting through the normalized datasets. For dolomites, this value is 9.
- **Disturbance factor D**. For this calculation, this factor was taken to D = 0, because the impact of the blasting once the rock mass test has been taken into consideration so that the cracks generated during the blasting were included in the GSI value.
- **Unit weight**. As calculated from the laboratory samples in the previous chapters.
- **Slope height**. The maximum slope height is considered (65 m), because it represents the worst situation possible.
- **Elastic Modulus**. Also obtained from the UCS laboratory results graphics.

When performing the calculation, all the Hoek- Brown criterion parameters will be computed (see the red rectangle).

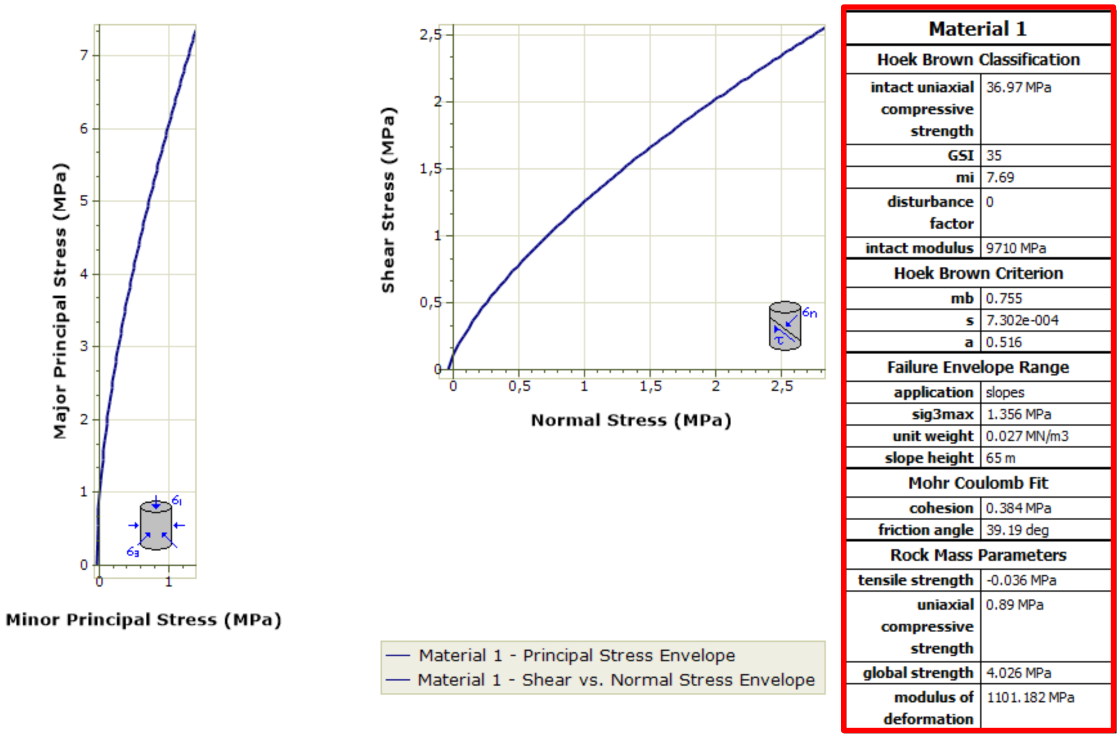


Figure 69. Obtaining input parameters from RocData

To sum up, the following table shows the data needed from the RocData program for the next calculations:

	Basic Data				New calculated data		
	UCS characteristic(MPa)	ρ_0 (kg/m^3)	GSI	m_i	mb	s	a
Dolomite rock mass	36,97	27,40	35	9	0,755	0,00073	0,516

Table 18. Input parameters for the Global analysis

In order to perform the global analysis of the slope, it is necessary to introduce the geometry and the last new calculated parameters on the Slide software as shown below.

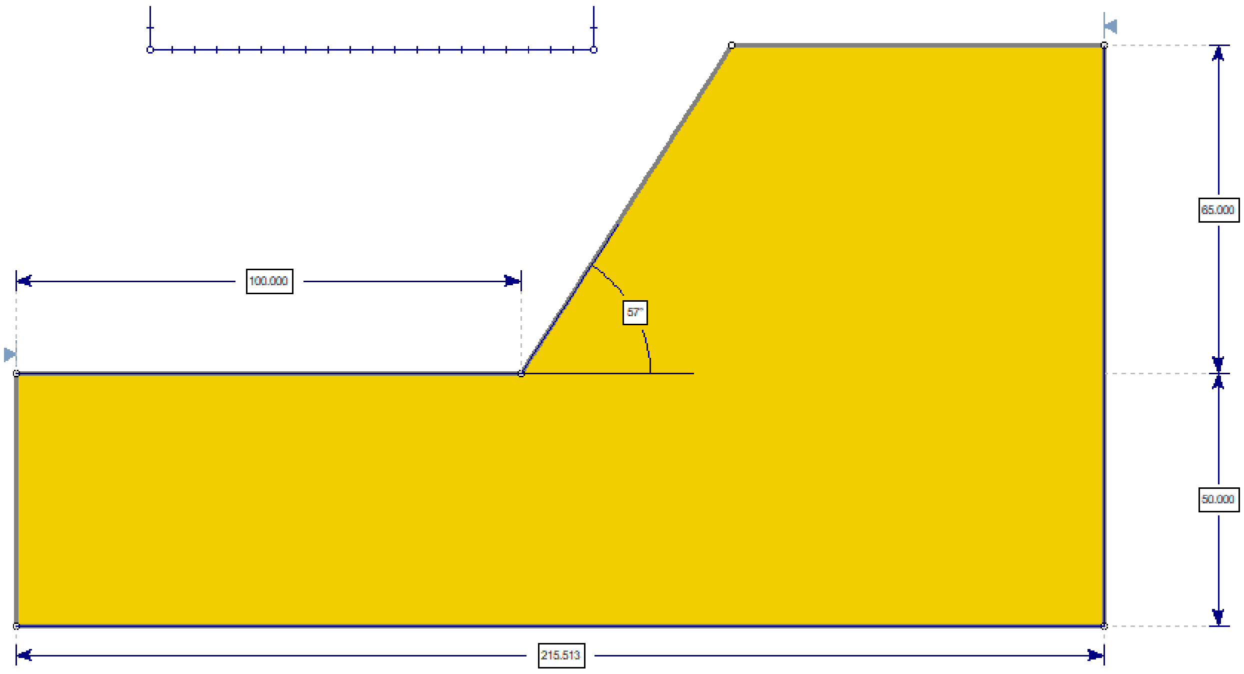


Figure 70. Geometry introduced into Slide software

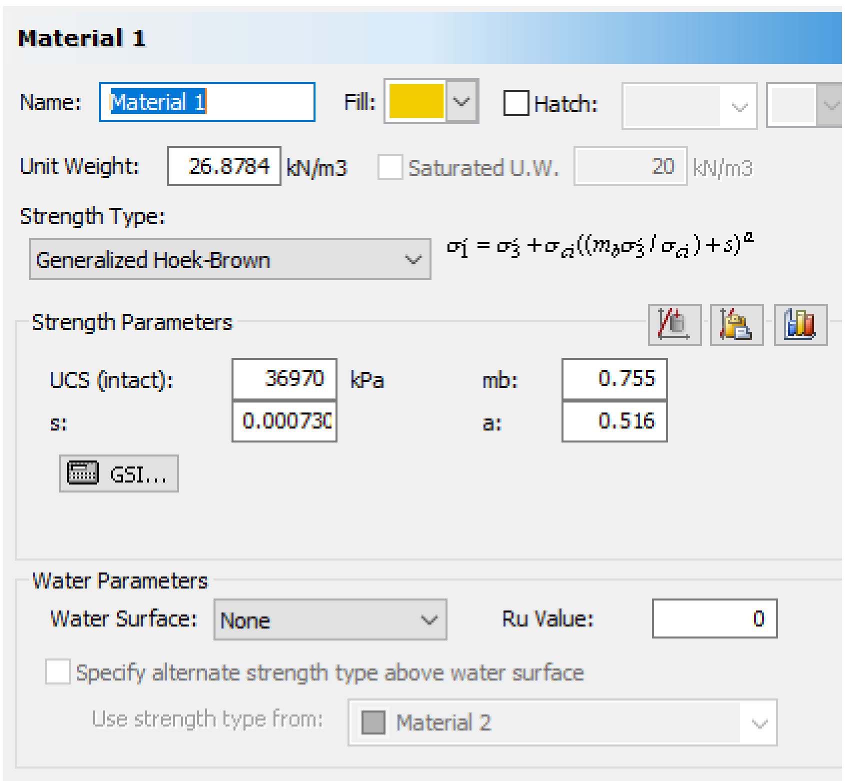
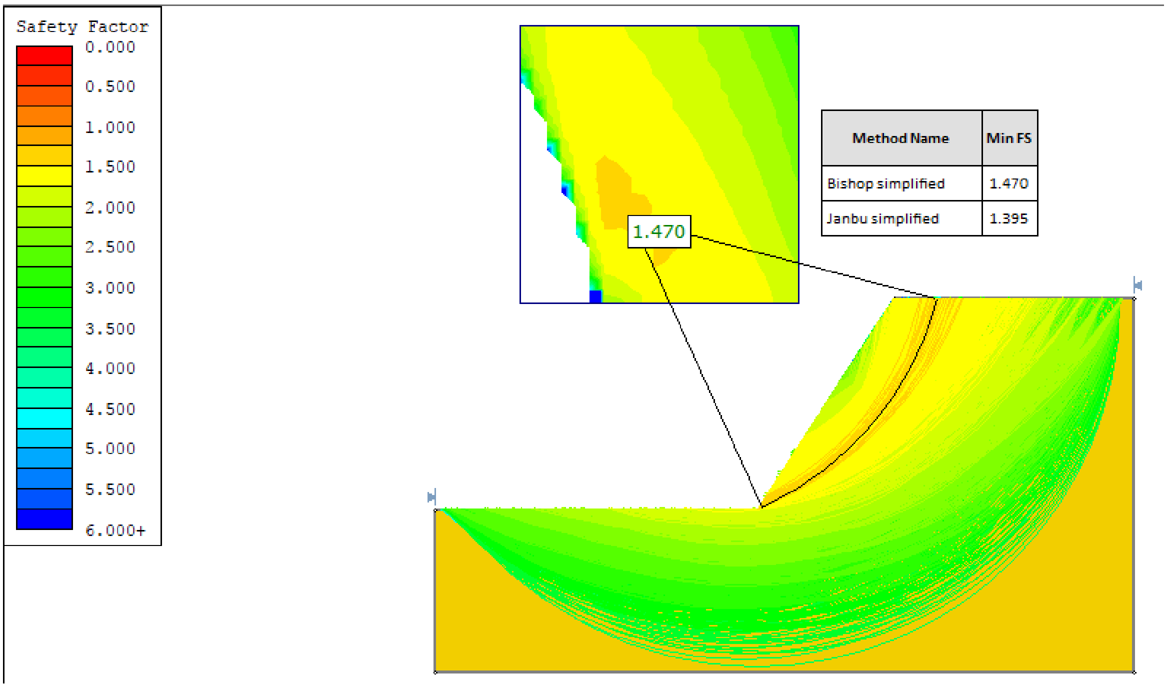


Figure 71. Input parameters for Slide



Material Name	Color	Unit Weight (kN/m3)	Strength Type	UCS (kPa)	m	s	a	Water Surface	Ru
Material 1		26.8784	Generalized Hoek-Brown	36970	0.755	0.000730178	0.516	None	0

Figure 72. Minimum Factor of Safety of 1.47 for Bishop method and 1.395 for Janbu method.

Stability calculations were carried out using the Janbu and Bishop methods for both circles and polygon slides. Based on these, the calculation resulted in a minimum safety factor of FSmin = 1.47 for Bishop method (Figure 72). According to the Hoek criteria, and as it can be seen in Figure 3, to accept the slope angle, the factor of safety must be bigger than 1.3 for "temporary" slopes with minimal risk of damage. **Therefore, the 1.47 factor of safety fulfills the requirement and the 57º slope angle is accepted as a safe slope angle**.

Global stability analysis, assuming the 57° slope for the walls of the mine showed to be safe as a homogeneous rock mass. The stability is related to the height of the wall, with the above-calculated determination of the highest planned rock wall, if lower rock walls there are more stable.

6. Final results, conclusions and recommendations

Final results

The rock slope stability analysis of the Mány quarry has been successfully carried out: the overall slope angle has been calculated and it ensures a safe operation of the quarry. In this chapter, the author will analyse the main parts of the thesis in order to give the reader a general overview to later reach the final conclusions.

The first chapter described the goals of the thesis and its structure to create a scheme easy to follow by the reader and also, the main steps which are essential to carry out the rock slope stability analysis.

From the second chapter, the author could understand for any general case, the main failure mechanisms which can occur in an open-pit mine the main properties of the rock mass which determine the possibilities of failure. The failure mechanisms which were considered to the stability analysis are:

- **Planar failure mechanism**
- **Wedge failure mechanism**
- **Rotational mechanism**

Once a review of the literature of general cases was done, the author focused on Mány quarry from the third chapter. In this chapter, the quarry was fully examined, and some important observations, essential for the rock slope stability were made:

- **The formation in the quarry is dolomitic.**
- **The groundwater level of the quarry is below the lowest elevation designed for the quarry, so it wasn't necessary to take it into account.**
- **Slope directions of each wall of the quarry were determined (see annex 2)**

Continuing the observations from the field in chapter 4, dip angles and dip directions from the orientations were measured to define the main joint sets, the JRC was measured and the GSI value was determined. Moreover, some samples of the rock mass were taken to the laboratory to make the suitable test for obtaining the input parameters (see annex 1). The results obtained were:

Main Joint sets						
Set	**Dip**	**Maximum difference**	**Minimum difference**	**Dip direction**	**Maximum difference**	**Minimum difference**
1m	**80º**	7º	8º	**233º**	20º	18º
2m	**73º**	16º	14º	**144º**	30º	24º
3m	**34º**	13º	9º	**343º**	40º	40º

Table 19. Main joint sets (final results)

Determined parameters	
Parameter	**Value**
JRC	11
JCS	34,01603 MPa
Friction Angle	27º
Unit weight	0,026878 MN/m^3
UCS	34,01603 MPa
GSI	35
Elastic modulus	9,71 GPa
Friction angle	27º

Table 20. Determined parameters form the field and the laboratory (final results)

Once the quarry was examined, different numerical modelling methods were described and the most suitable ones were chosen to carry out the stability analysis:

- **The kinematic methods.**
- **The limit equilibrium methods.**

The first kinematic analysis for planar sliding, based on the orientation of the discontinuities, showed four sides of the quarry with critical failure zones. These critical failure planes were then analyzed with the RocPlane software from Rocscience (limit equilibrium analysis) with an approximate slope angle of 70º. The main objective was to reduce this angle to a safe value. During the limit equilibrium analysis in RocPlane, the slope angle was reduce until **a safe slope angle of 57º was obtained.**

The next step was a kinematic wedge analysis, from this analysis three walls of the quarry had the critical intersection 1m-3m which was analyzed by the Swedge software. The limit equilibrium analysis in Swedge was carried out by introducing the slope angle value obtained above: 57º. **No probability of failure was detected so the maximum safe slope angle remained 57º.**

The last step was a Global stability analysis of the quarry, a rotational analysis. The whole material of the quarry was considered homogeneous because the discontinuities were already taken into account in the previous calculations. A factor **minimum factor of safety of 1.47 was obtained, which means that the quarry is safe enough with a slope angle of 57º.**

In Annex 3 all the calculations from the stability analyses are attached.

Conclusions

Open-pit mines are frequently exposed to slope failures which can cause economic and human costs. This project examines the potential failure mechanisms and to calculated the maximum steepness for the slopes of the Màny quarry.

- The planar kinematic analysis showed three critical failure planes in four different walls of the quarry.
- From the previous critical failure zones and using LEM analysis for planar failure, the slope angles was reduced to 57º.
- The wedge failure kinematic analysis showed one critical combination in three different walls of the quarry.
- From the LEM analysis, the probability of failure obtained with the previous angle (57º) remained zero. Therefore, the slope angle stayed 57º.
- The global analysis showed a FS=1.47 which is higher than the 1.3 FS recommended by Hoek-Brown criteri. Therefore, the slope angles of 57º is considered safe.

From the previous analysis an overall slope angle of 57º was determined as the maximum angle which has no probability of failure. To understand better the results, three drawings are attached in annex 2 and the calculations in annex 3. The drawings show the walls of the quarry which were detected as critical zones for planar and for wedge failure and another drawing with two sections once the final slope angle determined is implemented.

Recommendations

Since the quarry is located in a high spot of the region the presence of groundwater is not expectable. Until now during the excavation, there was no sign of groundwater. However, near the base level of the quarry, it is possible that groundwater will appear since the elevation of the brooks approximately the same as the elevation of the base level (150 m above the sea level).To be conscious of accurate information regarding the level of groundwater the stability calculations have to be re-evaluated in the future.

Furthermore, stereographic kinematic analysis takes into account only the orientation data and not the internal and external that can be acting in the quarry an may affect the stability of the slopes such as the water pressure and external loads. Therefore, if any external load is applied or the groundwater level is reached, it is recommended to use an advanced numerical modelling method por the stability analysis of the quarry.

As it was explained in chapter 4, there are many advanced numerical modelling methods which are being used in order to calculate the stability of slope which have a more complex behaviour. These methods can simulate the mechanical response of a rock mass subjected to a bigger number of initial and boundary conditions. To carry out an advanced analysis, this model requires more complex input parameters related to the geometry, the material anisotropy, pore pressures, non-linear behaviour, etc.

For the case of study, the Mány quarry, a noncomplex stability analysis was enough because there was not groundwater level conditions, the geometry was simple and the behaviour of the rock mass was quite homogeneous. However, in case of bigger excavation plans or any other complex situation, the author recommends an advanced numerical method modelling, such as finite element methods to obtain more accurate result.

In case of excavating deeper, a new stability analysis will be required because the groundwater level will be reached and it can affect the stability of the quarry.

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