Workshop "Failure Prediction in Geotechnics"

EXTENDED ABSTRACTS

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Failure Prediction in Geotechnics

CONTENT

Failure Prediction in Geotechnics - 50 Years after Vajont

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Since the Vajont catastrophe in 1963 rock mechanics has made considerable progress in understanding the behaviour of rock masses building up slopes and dam abutments as well as surrounding tunnels. Today discontinuum mechanics make simulations of the behaviour of jointed rock masses possible. The development of new monitoring techniques helps to observe rock mass displacements much more exactly than in the years when the Vajont dam was constructed. However, do we have enough information about the rock, the rock structure, the rock mass as well as ground water conditions? What has remained since the Vajont catastrophe are the uncertainties regarding the properties and the behaviour of the rock and of the rock mass, the rock structure, the properties of joints, ground water conditions etc. These uncertainties result in uncertainties assessing rock and rock mass behaviour. We can never be sure that the behaviour of the construction we build, of the rock structure we investigate will falsify our theories, our knowledge.

On October 9th, 1963, 250 million m³ of rock slid downwards from Monte Toc into the narrow Vajont valley, where water was dammed up by the Vajont dam, and 140 m upwards on the opposite slope, pushing about 100 million m³ of water 235 m above the crest of the dam. Falling from there more than 400 m deep into the Vajont gorge the water poured into the Piave valley as if out of a jet, destroying the villages of Longarone, Pirago, Villanova, Rivalta and Fae and taking about 1900 lives [6].

Figure 1: Arial oblique view (upstream) of the Vajont slide; in the foreground the dam, Monte Toc and sliding plane to the right, rest of the reservoir in the background; From [12].

This catastrophe was preceded by continuous attempts to bring down the rock mass slowly, since the slide into the reservoir had been detected in 1960 soon after the start of damming up the water. Slow movements had already been observed, but the extent of the moving mass did not become evident until after the upper edge of the sliding plane appeared in October 1960. Several series of raising and lowering the reservoir level followed by a downward movement of the sliding mass were performed. Leopold Müller often reported that the engineers had been very proud that the slide reacted like a circus horse. These attempts to bring down the rock mass slowly were successful until October 9th, 1963, when creeping passed into sliding of the rock mass with a velocity of a high speed train.

Though in those days monitoring systems like laser scanning, numerical methods being able to simulate even very fast movements of landslides etc. did not exist, the final position of the moving mass was calculated exactly and a bypass gallery was constructed in order to be able to operate the rest of the reservoir after sliding down of the rock mass. However, the assessment of the velocity of the moving mass under different circumstances was wrong.

First of all, the Vajont catastrophe was a lesson to the whole world how urgent investigations of the slopes of a reservoir are already during the design of a dam. Figure 2 shows that the East flank of Monte Toc had moved already before water was dammed up in the Vajont valley. This perception, having become generally known only after the catastrophe, would have prevented the construction of the dam.

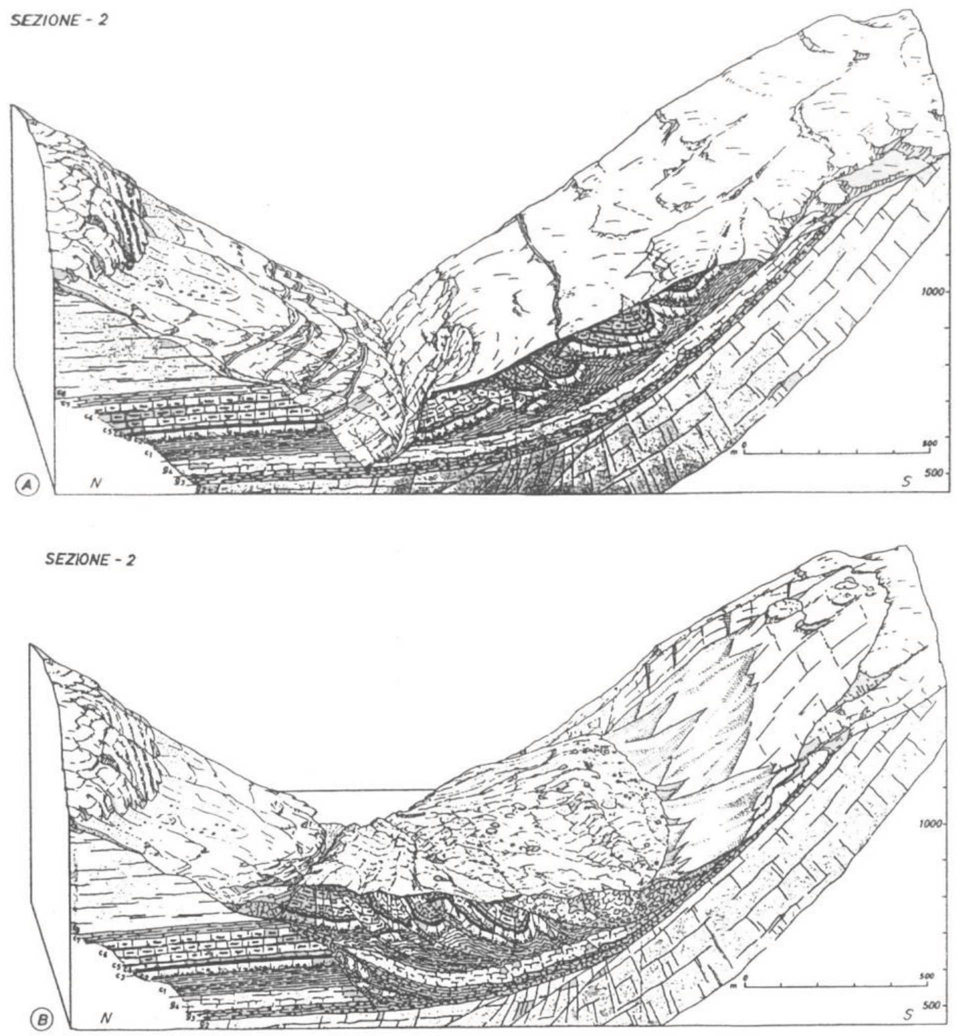

Figure 2: Cross sections of the left Vajont slope before (A) and after the slide (B); From: [1].

Secondly, the mechanics of progressive failures had not been studied as far as today. However, though the term "progressive failure" is used very often (e.g. in 71 papers at the 11th International Symposium on Landslides in Banff 2012), we still know very little about progressive failure.

The main problem, however, was that in spite of all investigations, the properties and the structure as well as ground water conditions could not be explored and possibly would not be explored exactly enough today. Rock is a complex, heterogeneous material, which will never be known in every detail. Though we cannot take into account every detail in assessments, in simulations etc., we have to know as many details as possible in order to abstract conditions in reality in the right way and make possible simulations.

The Malpasset and the Vajont catastrophes generated a boom of rock mechanics. We know much more about rock slope failure mechanisms today. In 1976 Goodman and Bray published "Toppling of rock slopes" [3]. Until then only sliding of rock masses was accepted as a rock slope failure mechanism. Since then investigations of rock slope failure mechanisms prospered [9]. Systematic investigations e.g. of rock fall started in the early 1970s.

Fukuzono [2] introduced the method of inverse velocities for predicting the point of time when a rock mass detaches from a slope in 1985. Applying this method to the monitoring results in Vajont shows that using this method it would still have been a demanding task to predict the failure. Moreover, the method of Fukuzono can only help predicting a failure, if there is a velocity tendency. If there is no tendency in displacement observation results (e.g. increasing velocity), the probability of failure e.g. for calculating the risk of a landslide can only be assessed by taking the probability of an earthquake triggering the landslide. The probability of an earthquake triggering the landslide is in any case the minimum of the probability of failure. Thus the risk as the product of the damage and of probability is the minimum risk, which is an important value in cost (for mitigation measures) – benefit (risk reduction) analyses.

Rock mechanics engineers had to accept that rock is not a continuous mass, but that it is discontinuous and fractionized by joints and faults, when the Malpasset dam broke in 1959 due to the failure of a rock tetrahedron separated from the bedrock by faults and joints in the left abutment. Discontinuum mechanics has made enormous progress since then. However, do we know the rock structure always exactly enough in order to install the correct input into discontinuum mechanics analyses?

In 1965 Müller & Pacher [7] made model tests in order to find out, how jointed rock fails. Among a lot of findings, which are the basis for modern rock mechanics, they found that the elongation transverse to the direction of the maximum compression stress is much bigger than the shortening in the direction of the maximum compression stress, when jointed rock fails. Today´s design of measuring programs monitoring dam abutments is based on this result of the model tests.

Since the 1980s the borehole probe TRIVEC makes possible measurements of displacement vectors along measuring lines (boreholes) and thus provides information on the deformation behaviour of the rock as well as on ground-structure interaction. This monitoring device gives a real insight into discontinuous, heterogeneous rock and was a big progress in models for predicting a failure.

In 1944 Rabcewicz [10] proposed a method for monitoring deformations in pilot galleries in order to explore tunnel behaviour and to have a better basis for dimensioning the tunnel. In most cases we know the rock mass through which we are excavating a tunnel only to a very limited extent. Therefore monitoring of tunnel deformations helps to know a little bit more about the rock mass surrounding the tunnel. In 1969 Szechy [13] proposed measurements only for the control of tunnels during operation, but not for dimensioning the support during excavation. In 1980 Hoek & Brown wrote: "As the subject has matured, the approach to the use of instrumentation in underground construction projects has become more responsible and there is now a tendency to use instrumentation as part of an overall design and construction control package." [4]. Although tunnel monitoring as well as the interpretation of tunnel monitoring results were developed extensively [5] on the basis of Vavrovsky´s work [14], we need to have a model of the rock mass and of its deformation and failure mechanism for interpreting e.g. displacements in the right way.

What remains since the Vajont catastrophe are the uncertainties regarding the quality, the properties and the behaviour of the rock and of the rock mass, the rock structure, the properties of joints as well as ground water conditions etc. Already in the late 1950s large in situ tests on a rock volume of several m³ were performed in order to investigate the deformation behaviour of the foundation rock of the Kurobe IV arch dam [8]. Have we really made much progress in in situ testing since then? Analyses of mass movements show that in most cases water plays an important role. However, do we really know where the water is, influencing stability [11]? The uncertainties regarding the quality, the properties and the behaviour of the rock and of the rock mass, the rock structure, the properties of joints as well as ground water conditions etc. result in uncertainties assessing rock and rock mass behaviour. We can never be sure that the behaviour of the construction we build, of the rock structure we investigate will falsify our theories, our knowledge. Hopefully a catastrophe like Vajont will not happen again; however, we cannot be sure.

References

[1] Carloni, G.C. & Mazzanti, R. 1964. Rilevamento geologico della frana del Vaiont. *Giornale di Geologia* 32, pp. 105-138.

[2] Fukuzono, T. 1985. A new method for predicting the failure time of a slope. *Proc. of the IVth International Conference and Field Workshop on Landslides*, Tokyo, Japan.

[3] Goodman, R.E., Bray, J.W., 1976. Toppling of rock slopes. *Proc. "Rock Engineering for foundations and slopes"*, Boulder, Colorado, USA, pp. 201-234.

[4] Hoek, E. & Brown, E.T. 1980. *Underground excavations in rock*. The institution of mining and metallurgy: London.

[5] Moritz, B. & Schubert, W. 2009. Application of the observational method in geotechnical safety management. *Geomechanics and Tunnelling* 2 (3), pp. 269-281, Ernst & Sohn: Berlin.

[6] Müller, L. 1964. The Rock Slide in the Vajont valley. *Felsmechanik und Ingenieurgeologie* 2 (3-4), pp. 148-212, Wien: Springer.

[7] Müller, L. & Pacher, F. 1965. Modellversuche zur Klärung der Bruchgefahr geklüfteter Medien. *Felsmechanik und Ingenieurgeologie*, Suppl. 2, pp. 7-24. Wien: Springer.

[8] Nose, M. 1964. Rock test in situ. Conventional tests on rock properties and design of Kurobegawa No. 4 dam based thereon. *Proc. 8th Congr. ICOLD,* Q 28, R 12, pp. 219-252, Edinburgh.

[9] Poisel, R. & Preh, A. 2004. Rock slope initial failure mechanisms and their mechanical models. *Felsbau* 22 (2), pp. 40-45, Essen: VGE.

[10] Rabcewicz, L. 1944. *Gebirgsdruck und Tunnelbau.* Wien: Springer.

[11] Rat, M. 1992. Conclusions. *Bulletin of the Int. Association of Engineering Geology* 45, pp. 75-76.

[12] Stüwe, K., Homberger, R. 2011. *High above the Alps. A Bird's Eye View of Geology.* Weishaupt Publishing (http://www.alpengeologie.org/).

[13] Szechy, K. 1969. *Tunnelbau.* Wien: Springer.

[14] Vavrovsky, G.M. & Schubert, P. 1995. Advanced analysis of monitored displacements opens a new field to continuously understand and control the geotechnical behaviour of tunnels. *Proc. 8 thInt. Congress on Rock Mechanics, Tokyo,* Vol. III, pp. 1415-1419. Rotterdam: Balkema.

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Mid-Magnitude Rockfalls: Too Big for Protection Measures - Too Small for Acceleration Anticipation?

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Recent rockfall events have demonstrated the vulnerability of infrastructure like highways, federal roads or railway traces. Examples include the Gotthard Railway Trace which was affected by a 1000 m^3 rockfall in June 5, 2012 and the federal road B 311 in the Saalach-Valley which was hit by a rockfall in December 23, 2012. To investigate natural hazards above infrastructure in a deterministic way, it is necessary to perform a detailed process analysis, using a combination of accurate field reconnaissance of the geomechanical setting and run out modelling of single rockfall events. If we assume that hundreds of possible objects for mid-magnitude failures exist above federal roads, it is not possible to select those for detailed analysis by their hazard potential. It then rather makes sense to go the other way round and to select possible hazardous spots above vulnerable infrastructure and analyse possible consequences of events. We have to ask what occurrence probability certain events have and what extent of loss can be caused to allocate our resources to high risk objects. The selected mid-magnitude rockfalls imposing high-risk should receive a comprehensive kinematic and mechanical analysis since their failure timing is very difficult to anticipate.

With our contribution we aim to give input to the topic of rockfall process analysis and risk evaluation by answering the following key questions:

- How can we carry out a hazard evaluation of mid-magnitude rockfall for traffic routes?
- How can we achieve an entire process understanding by considering the run out analysis and rock block fragmentation engaged with the failure process?
- How can we characterize the limitations of hazard analysis?
- How can we select relevant high-risk objects in terms of the vulnerability of the traffic roads?

We address these questions at suggested objects with a certain hazard potential in terms of magnitude, where the probability of occurrence is very difficult to determine.

At the region Berchtesgadener Land in the Bavarian Alps, we focused on slopes above the federal roads B 21 and B 305 between Bad Reichenhall, Unterjettenberg and Schwarzbachwacht.

Along the federal road B 305 we performed a detailed mapping of the source areas, including the analysis of discontinuities as well as a kinematic consideration referring to Markland (1972) and Talobre (1957). For one block subjected to planar failure, we accomplished an analysis of the discontinuities and shear parameters referring to Barton & Choubey (1977) and ISRM (1978). The information obtained from accurate field work was used as input parameters for the run out analysis using the code Zinggeler & GEOTEST (Krummenacher et al. 2005) and Rockyfor 3D (Dorren 2010). To provide quantification for the underground parameters and possible block sizes the Mean Obstacle Height (MOH) as well as the block axes were counted in squares of 20 x 20 m.

In this contribution we aim to demonstrate how to provide quantitative parameterization of mid-magnitude rockfall events to enhance risk evaluation above vulnerable infrastructure.

1. HAZARD EVALUATION USING CURRENT AVAILABLE DATA

By means of this study we aim to process current available data in a way so that responsible practitioners have a first overview of possibly hazardous roads. The results should provide sufficient evidence for the responsible institutions to attain a decision support for the spatial and temporal implementation of mitigation measures.

Further this proceeding offers the possibility to integrate ongoing knowledge, like latest constructed protective measures, into the system, and so to update and increase the reliability of risk assessments into the future step by step.

The following five key-features provide the basis for the subsequent methodology of risk analysis and risk assessment in the study area:

1. Hazard index maps (including relevant 3-D simulations)
2. Easy-to-handle, systematic event detection provided by the Road Construction Administration.
3. A systematized experience or incident protocol with a simplified geological assessment of the situation in the hazard zone.
4. The documented, existing, natural, operational and physical mitigation measures.
5. The possible extent of loss - primarily determined by the average daily traffic (ADT) of the roads.

The instantaneous available data basis for performing a first risk assessment in the complete study area (and also in the largest part of the Bavarian Alps), is the hazard index map, which was recently created by the Bavarian State Office for Environment and based on accurate 3-D simulations performed by the Geotest AG. In these simulations the source areas were determined by using two models:

- Disposition Model 1: Determination of potential source areas in terms of rockfalls from the Georisk data.
- Disposition Model 2: Determining possible source areas for falling rock using the digital terrain model and critical inclination angle.

Based on current field surveys, an average block size for each geological unit was determined. These block sizes were assigned to one of four parameter classes, which were afterwards used as input parameters for the simulations.

The intersection of the simulation results and traffic data (e.g. DTV) of the affected roads is the first basis for a risk evaluation. Through the results, the hazard areas for "normal events" (= events with an average probability of occurrence < 100 years and block sizes < 5 m^3) can be located and identified, and a first risk classification is possible.

2. REQUESTED DATA AND TOOLS FOR AN ENHANCED RISK/HAZARD ANALYSIS

After having analysed the results in other regions with a higher investigation, it was decided to improve the first general approach, by introducing two instruments:

At first, a GPS supported event detector device was installed. Using this device rockfall hazards, based on hazard categories (capture program with drop-down menus), can be quickly and accurately recorded by street guards and sent to the responsible authorities. In addition, the program enables the recording of historical events retrospectively, which leads to an extended collection period. Especially for a first assessment, a higher reliability can be achieved.

Secondly, a systematized experience fact sheet for a first hazard assessment in the field was created. In this protocol, information about abnormalities in operational service, for example rock slope site clearance, the presence of silent witnesses, as well as basic information on source, transit and run out areas for each risk area, are recorded.

In terms of an accurate analysis of the data obtained from these two instruments, the scheduled probability over 100 years can be modified and the risk assessment can be improved significantly.

To further assess which rockfall fragments run out to infrastructure, it was necessary to examine the existing protection measures. On this purpose, all structures, which are available with the most relevant data and positional accuracy were integrated in a Geo-Information System (GIS) and must be taken into account directly for the risk analysis (Figure 1).

3. LIMITATIONS OF A RISK ANALYSIS WITH A REDUCED LEVEL OF ACCURACY - HOW DO WE DEAL WITH THESE LIMITATIONS?

The presented risk analysis, which has been identified and described in terms of this study, is up to now not comparable in quality with rating systems like the risk values from the concept Risk Natural Hazards, neither National Roads (ASTRA 2012) nor the program EconoMe, which is applied in Switzerland. The results in their current stage cannot yet be termed a decision support, which assigns specified mitigation measures in order to minimize hazard or risk in total. The data basis is not provided to instruct authorities, which funding might be necessary to acquire prevention measures or to determine which measures are convenient for funding.

The methodology connected with the compiled data identifies and filters the adequate construction areas in an objective way, so that the responsible authorities may use the available resources in an optimal way to achieve risk minimization.

We must also be aware that hazardous spots with block sizes > 5 m³ which are usually associated with a very uncertain probability of occurrence are not considered. Such dangerous spots must be specifically recorded, investigated and assessed. A reasonable approach for these cases can be presented in the case of the large block above the federal road B 305.

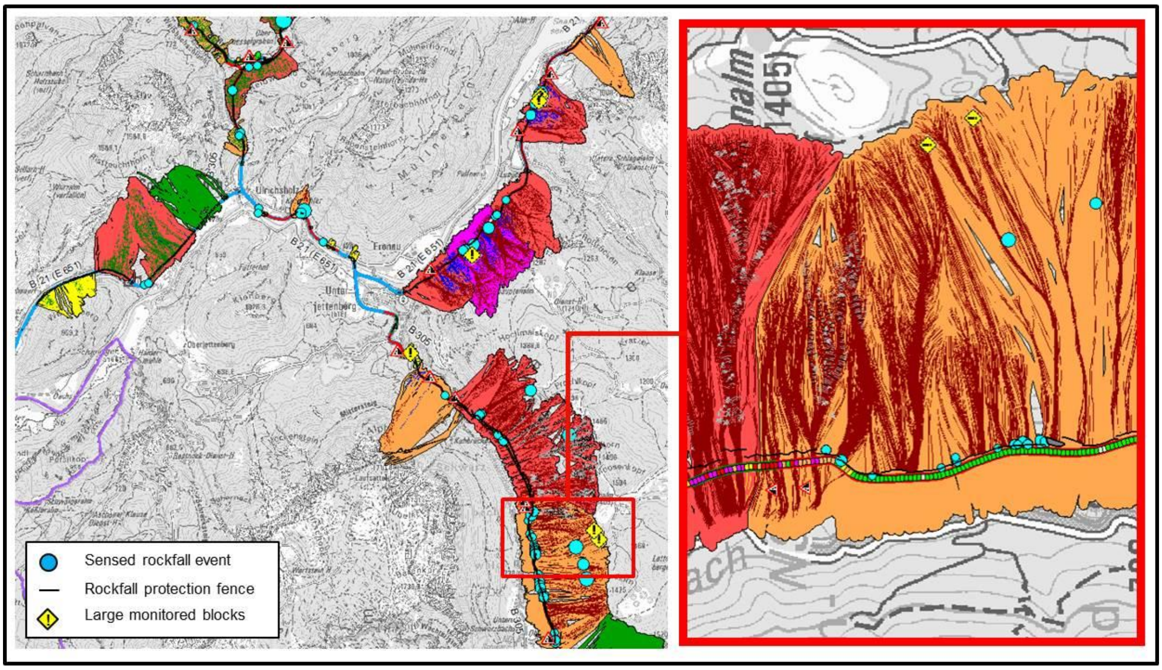

Figure 1: Left: Overview of the risk analysis in the investigation area. Right: Detail section of a danger area on the B 305 with two large blocks to be monitored.

4. HOW CAN WE ACHIEVE AN ENTIRE PROCESS UNDERSTANDING BY CONSIDERING THE RUN OUT ANALYSIS ENGAGED WITH THE FAILURE PROCESS?

To obtain knowledge about possible rockfall trajectories it is common to perform rockfall modelling in 3D referring to codes like Rockyfor 3D (Dorren 2010) or Stone 3D (Guzetti et al. 2002). Up to now the current rockfall codes provide the possibility to perform run out analysis but with no regard to the detachment or failure process. Therefore we suggest having a closer look at the failure process to obtain information about the entire rockfall process.

4.1. Characterization of the failure process

We performed an accurate mapping of the rockfall source area at the project site above the federal road B 305, which consists of carbonates belonging to the Dachstein-Formation. This includes an analysis of the discontinuities orientation as well as a kinematic analysis referring to Markland (1972) and Talobre (1957), taking wedge and planar failure into account (Figure 2, right). Considering the friction angle we assumed a range between 30° and 35° for the Dachstein-Formation referring to Cruden & Hu (1988), Heckmann et al. (2012). Considering the cases of planar failure, we recorded an approximately 200 m^3 single block, where the failure surface underneath the block is directly accessible (Figure 2, left). For this “key-block” we performed the following methods to characterize the failure process: an accurate roughness-mapping of the failure surface in a high level of detail (1: 50) according to Barton & Choubey (1977) and ISRM (1978). We determined the shear parameters referring to Barton & Choubey (1977) to take roughness (JRC) as well as the joint compressive strength (JCS) into account, when analysing the limit equilibrium state of the endangered block. For the determination of the JCS we used a Schmidt Hammer (Schmidt 1957) and evaluated our results according to Woszidlo (1989). By taking the total friction as well as the driving and resisting forces into account, we conclude to the influence of cohesion forces referring to Barton & Choubey (1977).

To consider the degree of fragmentation as well as the connection of the block to the rock mass, we analysed the persistence of joints through the block and set the accessible cavities in relation to the contact area of the block.

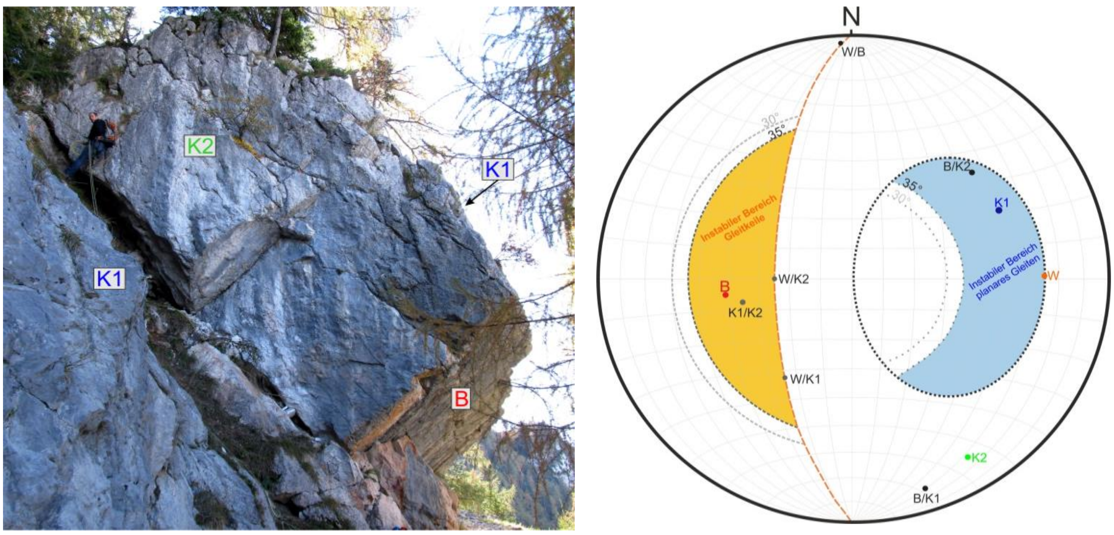

Figure 2: Left: 200 m^3 Block subjected to planar failure above the federal road B 305. Right: Kinematic analysis of the joint sets at the source area taking planar and wedge failure into account.

4.2. Falling process – Run-out analysis

To analyse the run-out for different failure scenarios we performed rockfall modelling in 3D using the Code Rockyfor 3D (Dorren 2010). To characterize the underground roughness as well as the block diameters in a quantitative way, we performed a statistical counting of the mean obstacle heights (MOH) in quadrate areas of 20 x 20 m. The counting sites were such distributed over the slope that we obtained detailed information about the talus slope from the source area to the valley bottom. Further parameters like damping and forest stand characteristics were also determined by accurate field work.

Probable magnitudes were assessed by composing the information about the rockfall deposits, like the counting of block axes at the talus slope, and the field work at the source area. For our key object of the study (Figure 2, left), we analysed the persistence of joints through the block in order to consider possible failure scenarios in terms of fragmentation. Since it is up to now not common to consider fragmentation during the falling process in Rockyfor 3D, we performed parameter studies considering different block diameters/axes (d1, d2 and d3) at the release area.

References

[1] ASTRA 89001: Risk Natural Hazards on National Roads. Bern, Ausgabe 2012 | V2.00.

[2] Barton, N. & Choubey, V. (1977): The Shear Strength of Rock Joints in Theory and Practice.-Rock Mechanics, 10, pp. 1-54.

[3] Cruden, D.M. & Hu, X.Q. (1988): Basic friction angles of carbonate rocks from Kananaskis country, Canada. - Bulletin of the International Association of Engineering Geology, 38, pp. 55-59.

[4] Dorren, L. (2010): Rocky for 3D revealed - Description of the complete 3D rockfall model. Association ecorisQ.

[5] Guzetti, F., Crosta, G., Riccardo, D. & Agliardi, F. (2002): STONE a computer program for the three-dimensional simulation of rock-falls.- Computers & Geosciences, 28, pp. 1079-1093.

[6] Heckmann, T., Bimöse, M., Krautblatter, M., Haas, F., Becht, M. & Morche, D. (2012): From Geotechnical Analysis to quantification and modelling using LiDAR Data: a study on rockfall in the Reintal catchment, Bavarian Alps, Germany.- Earth Surf. Process. Landforms, 37, pp. 119-133.

[7] ISRM - International Society for Rock Mechanics (1978c): Suggested methods for the quantitative description of discontinuities in rock masses. - Commission on Standardization of Laboratory and Field Tests. - Int. J. Rock Mech. Min. Sci. & Geomech. Abstr., 15, No. 4, pp. 319-368.

[8] Krummenacher, B., Pfeifer, R., Tobler, D., Keusen, H. R., Liniger, M. & Zinggeler, A. (2005): Modellierung von Stein- und Blockschlag; Berechnung der Trajektorien auf Profilen und im 3-D Raum unter Berücksichtigung von Waldbestand und Hindernissen. – Fan-Forum ETH Zürich am 18.02.2005, 9 p., Zollikofen.

[9] Markland, J.T. (1972): A Useful Technique for Estimating the Stability of Rockslopes when the Rigid Wedge Sliding Type of Failure is expected. Imperial College, Rock Mech. Research Report, No. 19: 10 p.

[10] Schmidt, E. (1957): Betonprüfhammer: Gebrauchsanweisung. – PROCEQ S.A., Zürich.

[11] Talobre, J. (1957): La Mécanique des roches. pp. 39-44, Paris (Dunop).

[12] Woszidlo, H. (1989): Untersuchungen an Festgesteinen mit dem Prallhammer nach Schmidt. -. Nat. Tag. Ing-Geol., Bensheim, Ber. 7, pp. 287-294.

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Safety Definition and Estimation for Slopes with Creep Behavior – a Geotechnical Challenge

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Estimating the stability of creeping landslides by classical definitions of "safety factor" is a real challenge. Static limit equilibrium methods are generally insufficient to estimate the behavior of creeping landslides because the important factors are displacements, deformation rates and time dependent behavior. The paper is intended to provide an overview of the main problems by dealing with safety definitions in case of creeping landslides. Therefore, the classical definitions for slope stability will be presented and subsequently their disadvantages, in connection with creeping landslides, will be shown. Finally, already existing analytical models for determining necessary stabilization measures are presented and the problems with safety definition in case of these models will be explained.

1. DEFINITION OF SLOPES WITH CREEP BEHAVIOUR

Creeping landslides are usually close to the ultimate limit state in their natural state, depending on the current state and the current environmental conditions (e.g. precipitation). Furthermore, these landslides show time dependent rates of movement. It is not certain whether these movements could be taken as creep behaviour in the proper sense (deformation under the influence of constant stresses). The different deformations and rates of movement could be also considered as a result of the continuous changing boundary conditions and only the average deformation seems to be a constant creeping. In many cases it is difficult to determine whether or not the movements are creeping. The average rate of movement can range from mm/year to m/year. The present report only covers creeping landslide with a potential volume bigger than about 10,000 m³.

2. GENERAL DEFINITONS FOR SLOPE STABILITY

The most common definition for the factor of safety of slopes is the comparison of maximum shear strength of the material and mobilised shear strength, which is required for equilibrium. This definition is based on Fellenius. The following equation describes the Fellenius method.

$$\eta = \frac{\text{shear strength}}{\text{mobilised shear strength}} = \frac{\tan \varphi'}{\tan \varphi'_{mob}} = \frac{c'}{c'_{mob}} \geq 1.00$$

The Fellenius method is based on the assumption that cohesion and tangent of friction angle are only mobilised in a certain extent (for reaching equilibrium) for the determination of the slope stability. According to Fellenius failure is defined as $\eta > 1.00$. In case of creeping landslides it is questionable whether failure could be also defined as a maximum rate of movement or maximum deformations.

In addition to the Fellenius method, probabilistic methods and risk analysis are also used to define the stability of slopes, especially of landslides. Probabilistic methods often combine the before mentioned analytical methods with probability distributions for the various input parameters. In case of risk analysis, factors like probability of landsliding, runout behavior, vulnerability of property and people are combined to estimate the landslide risk [3].

3. SAFETY DEFINITONS FOR SLOPES WITH CREEP BEHAVIOR

3.1. Detection of creeping landslides and determination of required parameters

To detect a potential creeping landslide in its natural state is often a challenging task. With the help of geotechnical measurements and laboratory tests it may be possible to detect and to investigate a creeping landslide but in most cases these measures do not allow the determination of the current factor of safety in natural state.

The most important parameters for determining a factor of safety for creeping landslides are geometry, soil parameters (especially shear strength) and external influences / loads (like precipitation, fluctuating water levels in reservoirs adjacent to the creeping landslide). Although a large number of geotechnical measurements (inclinometer, extensometer, geodetic measurements) are developed to detect a creeping landslide it is hardly possible do define an exact geometric boundary. The determination of accurate soil parameters in the laboratory also presents a major challenge due to the highly heterogeneous subsoil. Therefore, an accurate determination of a factor of safety seems to be almost impossible. Nevertheless, it is, despite all the difficulties, achievable to define roughly suitable parameters, these values often result in a factor of safety equal to or slightly higher than 1.00. Due to these circumstances it is hardly possible to use the common safety definitions because in this case the factors of safety, prescribed by law, cannot be met.

3.2. Problems with classical definitions for slope stability

The general methods for estimating slope stability assume a continuous slip surface along which soil behaves as a rigid plastic body satisfying the Mohr-Coulomb failure criterion. A combination of these assumptions with a static limit equilibrium method leads to slope stability, expressed in a factor of safety. The factor of safety is assumed to be constant along the slip surface. These assumptions may be realistic for small slopes, but in case of large landslides they lose its justification. For creeping landslides expected displacements, velocities and time dependent behavior are the main factors for estimating the risk of failure, i.e. the slope stability.

3.2.1. Constant factor of safety along slip surface

The common limit equilibrium methods assume a constant factor of safety along the slip surface. This circumstance is not justifiable for landslides with large dimensions because of the following two points. On the one hand material parameters can change significantly along the slip surface due to the heterogeneous conditions and on the other hand the material of creeping landslides often shows a brittle behavior. Therefore, it is possible that critical state strength is mobilized at some locations while ultimate shear strength is considered for other parts of the sliding surface (progressive failure). In case of such brittle material behavior it is necessary to know the strains in the material but strains cannot be calculated with static limit equilibrium methods.

3.2.2. Selection of accurate shear strength

For the stability analysis it is necessary to define the maximum shear strength. Figure 1 shows a typical stress-strain curve of soil in a drained shear test. The strength is the value of maximal mobilized shear stress under certain strains. The value for the shear strength could be the peak value at relative small strains, the critical state strength, where the sample shows constant stress and constant volume and in case of clayey soils it could be the residual strength at very large strains.

Skempton [8] recognized a difference between critical state strength and residual strength only in soils containing more than 20% of clay minerals. For other soils the critical state strength is equal to the residual strength. The difference between these two states in case of clayey soils is due to the different soil structures at the two states. Wood [10] describes the two states in the following way: When soil is at a critical state, it is being continuously remoulded and churned up, and its structure remains random. When soil is shared to a residual state on a failure surface, the deformations have been so large that the clay particles on both sides of the failure surface have become oriented parallel to the failure surface.

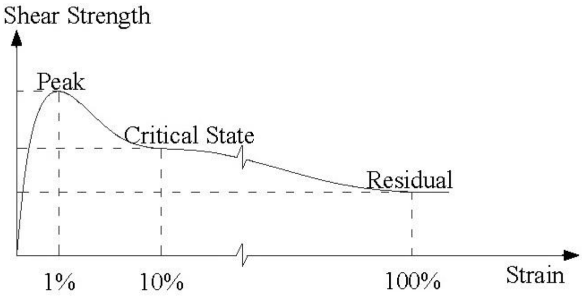

Figure 1: Behavior of soils during shearing.

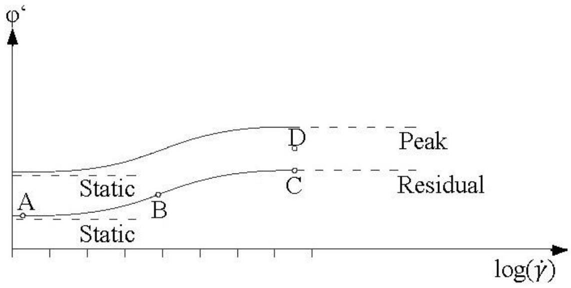

Figure 2: Effect of shearing strain rate [2].

For calculating a factor of safety for a reactivated landslide one has to determine whether or not a difference between critical state strength and residual strength exists and in case of different stresses at the two states one has to decide which strength is used for the calculation.

3.2.3. Rate dependency of shear strength

Another important factor for stability analysis of creeping landslides is the rate dependency of the shear strength. The question rises whether or not the shear strength (friction angle and cohesion) on the slip surface is rate dependent. In the literature different approaches exist. Puzrin & Schmid [7] and Lippomann [6] considered for their stability analysis of creeping landslides rate dependent shear strength. Haefeli [5] and others do not consider any rate dependency. In case of rate independent shear strength it is debatable why creeping landslides seem to be stable under large rates of movement (> 100 mm/y) as well as under small rates of movement (< 10 mm/y). Furthermore, it is questionable how the factor of safety is changing in case of different rates of movement. Alonso [2] assumes that the factor of safety is always equal to 1.00 for a landslide with constant rate of movement, based on the assumption of rate dependent shear strength. The creeping landslide reaches equilibrium if the mobilized friction angle is in A (see Figure 2). If the landslide accelerates (external change of stresses) the available friction angle increases to B or C. If it is able to maintain a condition of dynamic equilibrium, the landslide will maintain a constant rate of movement. If not the landslide accelerates (point D). Therefore, the factor of safety never changes according to the classical definition because any increase of shear stress / rates of movement leads to an increase of maximum shear strength up to a certain extent.

There are many studies on the rate dependency of shear strength. Most of them investigate the undrained behavior under triaxial conditions. Unfortunately these circumstances do not reflect the conditions in a slip surface of a creeping landslide. Tika [9] investigated the rate dependency of residual strength in a ring shear apparatus. The results show a rate dependency for cohesive soils but not for granular soils. A significant influence could only be noticed at rates of shearing significantly higher than they occur in case of creeping landslides.

Up until now it has not been possible to clarify the questions about the rate dependency of the shear strength and further investigations on this topic are required.

3.3. Selected analytical models – problems with safety definition

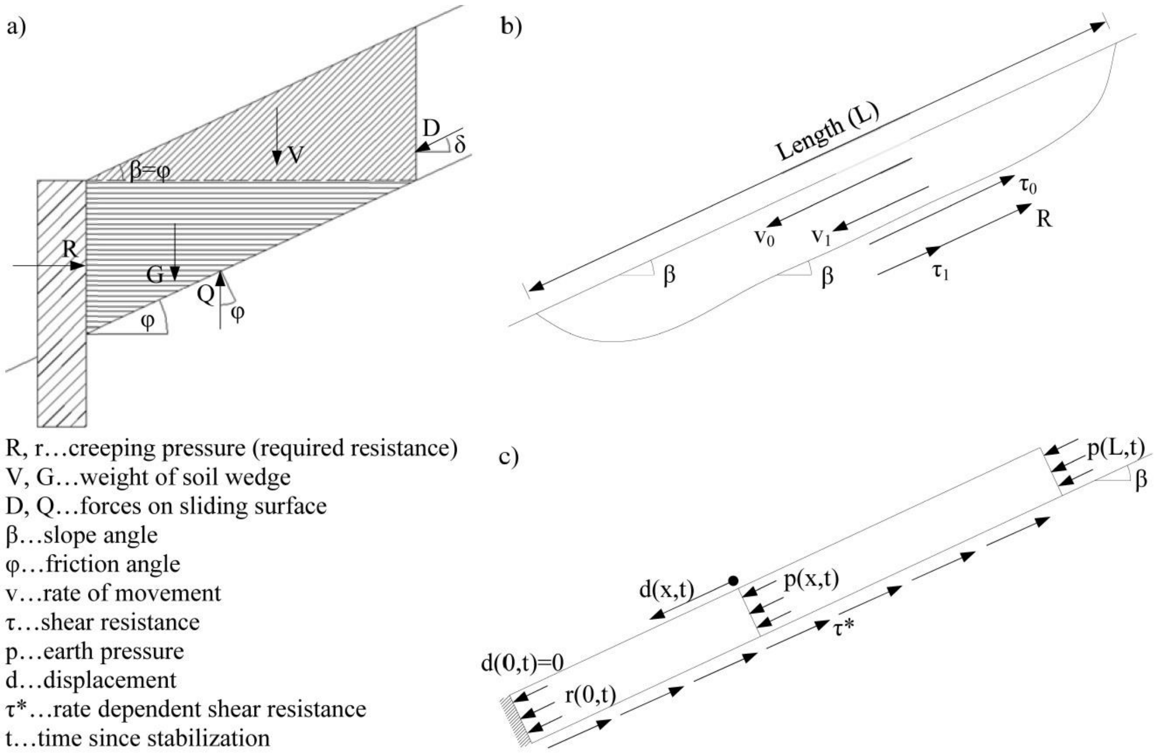

Figure 3: Analytical models: a) Haefeli [5], b) Lippomann [6], c) Puzrin & Schmid [7].

Figure 3 shows three selected analytical models for calculating creeping pressure in a creeping landslide.

Haefeli [5] based the calculation of the creeping pressure on the classical earth pressure theory. He assumed a secondary sliding surface, which leads to an overcoming of the stabilization measure. So, the sliding mass and the geometry of the landslide are neglected. A comparison of available resistance and creeping pressure only reflects a safety factor against overcoming but not a factor of safety for the entire landslide.

Lippomann [6] assumes rate dependent shear strength. The difference in the shear strength due to different rates of movement has to be compensated by the resistance (R) of a stabilization measure. An increase of the factor of safety after the installation of a stabilization measure is not directly quantifiable because the decrease of shear strength is equal to the resistance of the stabilization measure.

Puzrin & Schmid [7] considered rate dependent residual shear strength on the sliding surface. For the determination of the safety factor Puzrin & Schmid [7] considered the passive earth pressure as resistance against failure at the lower end of the landslide. Therefore, the assumed failure is similar to Haefeli [5] and the calculated factor of safety only describes the behavior of the soil behind the stabilization measure but not of the entire landslide.

In summary, these three models show that the safety definition for creeping landslides is a complex task and therefore further investigation is needed.

References

[1] Atkinson, J. 2007. *The Mechanics of Soils and Foundations*. Taylor & Francis, New York, USA.

[2] Alonso, E.E. 2012. Deformation analysis of landslides: Progressive failure, rate effects and thermal interactions. In: Eberhardt, E. (Ed.), Froese, C. (Ed.), Turner, A.K. (Ed.) & Leroueil, s. (Ed.), *Landslides and Engineered Slopes: Protecting Society through Improved Understanding*, Canada, 03.-08. June, pp. 175-214. Taylor & Francis: London, UK.

[3] Dai, F.C.; Lee, C.F. & Ngai, Y.Y. 2002. Landslide risk assessment and management: an overview. *Engineering Geology 64*, pp. 65-87, Elsevier: München, Germany.

[4] Fredlund, D.G. & Krahn, J.. 1977. Comparison of slope stability methods of analysis. *Canadian Geotechnical Journal 14 (3)*, pp. 429-439, NRC Research Press: Ottawa, Canada.

[5] Haefeli, R. 1945. Zur Erd- und Kriechdrucktheorie. In: Meyer-Peter, E. (Ed.), *Mitteilungen aus der Versuchsanstalt für Wasserbau an der Eidgen. Techn. Hochschule*, pp. 1-11. Verlag AG Gebr. Leemann & Co: Zürich, Switzerland.

[6] Lippomann, R. 1988. *Ingenieurgeologische Kriechhangsicherung durch Dübel*. Doctoral Thesis, Institute of Soil Mechanics and Rock Mechanics, Karlsruhe Institute of Technology, Germany.

[7] Puzrin, A.M. & Schmid, A. 2012. Evolution of stabilized creeping landslide. *Géotechnique 62, No. 6*, pp. 491-501, ICE Publishing: London, UK.

[8] Skempton, A.W. 1985. Residual strength of clays in landslides, folded strata and the laboratory. *Géotechnique 35, No. 1*, pp. 3-18, ICE Publishing: London, UK.

[9] Tika, T.E.; Vaughan, P.R. & Lemos, L.J. 1996. Fast shearing of pre-existing shear zones in soil. *Géotechnique 46, No. 2*, pp. 197-233, ICE Publishing: London, UK.

[10] Wood, D.M. 2007. *Soil Behaviour and Critical State Soil Mechanics*. Cambridge University Press, New York, USA.

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Key Aspects in 2D and 3D Modeling for the Stability Assessment of a High Rock Slope

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This presentation is intended to describe the rock mechanics and rock engineering studies carried out for the stability assessment of a 120 m high rock slope in a limestone quarry in the Piedmont Region (Italy). The rock slope is characterised by the presence of a 340,000 m^3 estimated rock volume (Figure 1), standing in limit equilibrium conditions, which impairs quarrying activities below the berm elevation reached. Following an outline of the case study, the in situ investigations carried out, including detailed geological mapping, 3D imaging with a laser scanning equipment and infrared thermo-graphic methods will be described. Then, the results of real-time monitoring of the rock face by using a Ground-Based Synthetic Aperture Radar (GBInSAR) will be presented. Three dimensional continuum and discontinuum modeling involving a back analysis of a plane sliding instability at the toe of the slope and detailed slope stability studies of the rock volume, aimed at the definition of the likely instability scenarios, will be described. Finally, the actions envisaged in order to continue with the quarrying activities at the site will be discussed.

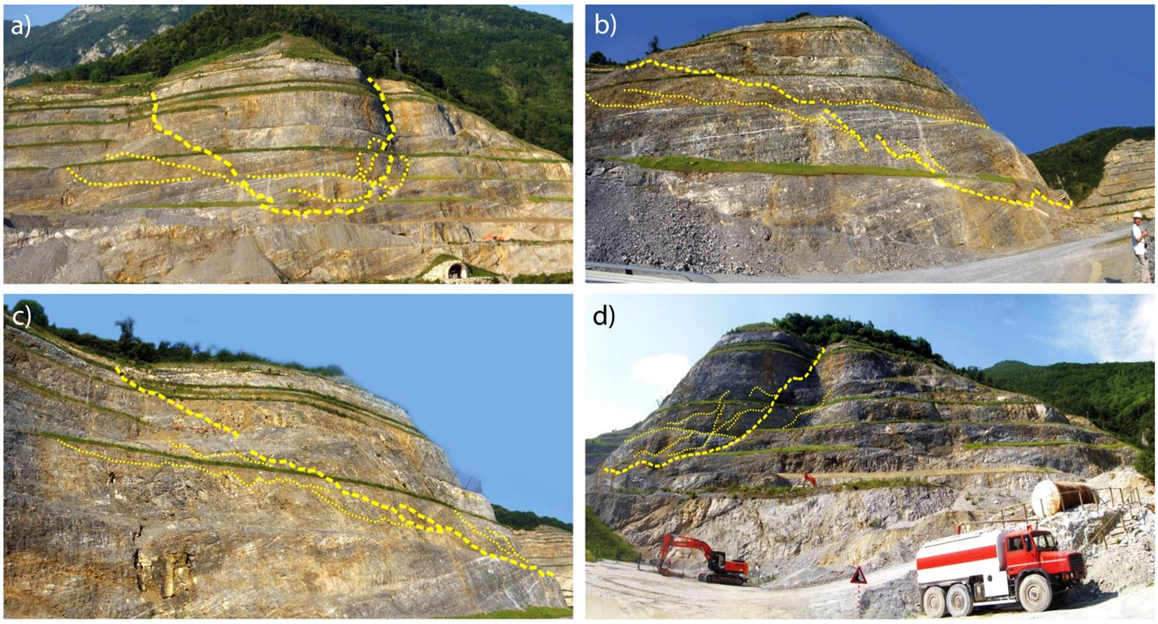
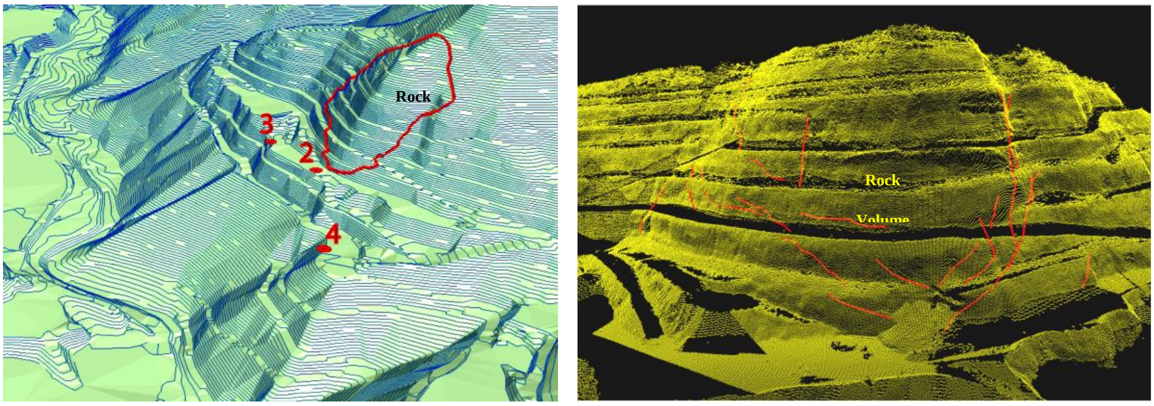


Figure 1: A view of the rock slope with the rock volume in limit equilibrium conditions as observed in the front a) and laterally b), c), d).

1. SITE DESCRIPTION

As shown in Figure 2, the rock slope of interest is part of a limestone quarry face being developed in a bench sequence from the top, at elevation 1000 m a.s.l. approximately, down to elevation 880 m a.s.l. approximately, to a total height of 120 m. During the benching down activities a fault zone, characterized by the same dip direction of the quarry face and 30° inclination approximately, was progressively identified and shown to isolate a rock buttress having an estimated volume of 340,000 m^3 (also see Figure 1) and posing a significant risk for the planned activities below it. This prompted a thorough study to be undertaken in order to analyse its stability conditions, prior to continue excavation down to the toe of the slope at elevation 740 m a.s.l..



(a) (b)

Figure 2: The quarry face with the rock volume in limit equilibrium conditions: (a) 3D visualization, (b) Laser scanning 3D imaging with indication of the fault plane traces.

2. ROCK MASS CONDITIONS, IN SITU OBSERVATIONS AND MONITORING

A SW-NE cross section taken nearly orthogonal to the slope face is shown in Figure 3. The limestone rock mass is of fair quality with the Geological Strength Index (GSI) estimated to be in the range 50-60. The intact rock uniaxial compressive strength is equal to 50-60 MPa for limestone and 15-20 MPa for brecciated limestone which is present along the main fault F4, which nearly isolates the rock volume of interest. The bedding (BG) and three joint sets (K1, K2, K3) characterise the limestone rock mass.

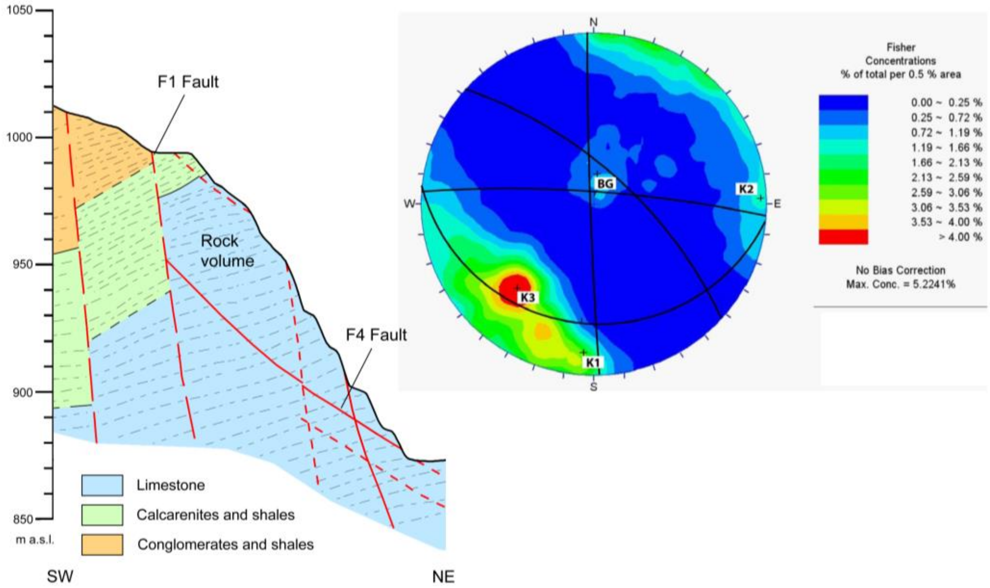


Figure 3: Schematic SW-NE cross section of the slope with a stereographic plot showing the bedding (BG) and joint sets (K1, K2, K3) in limestone.

Detailed studies by using 3D imaging with a laser scanning equipment and infrared thermo-graphic methods have been carried out in order to define the main geometrical features of the slope including an improved description of the major discontinuities (Faults and Shear zones) by determining orientation, roughness, undulation and persistence.

In addition, slope monitoring has been undertaken for a 3.5 month time interval, by using a Ground-Based Synthetic Aperture Radar. The acquisition interval of the radar images was set at first equal to 30 minutes with possible increase of the acquisition frequency up to 6 minutes in case of unexpected slope behavior. Figure 4 illustrates a typical displacement map showing zones A, B, C, D, E of the quarry face undergoing small movement (B, C, D, E) or being in stable conditions (A). In particular the B zone at the toe of the rock volume has been affected by a local sliding instability.

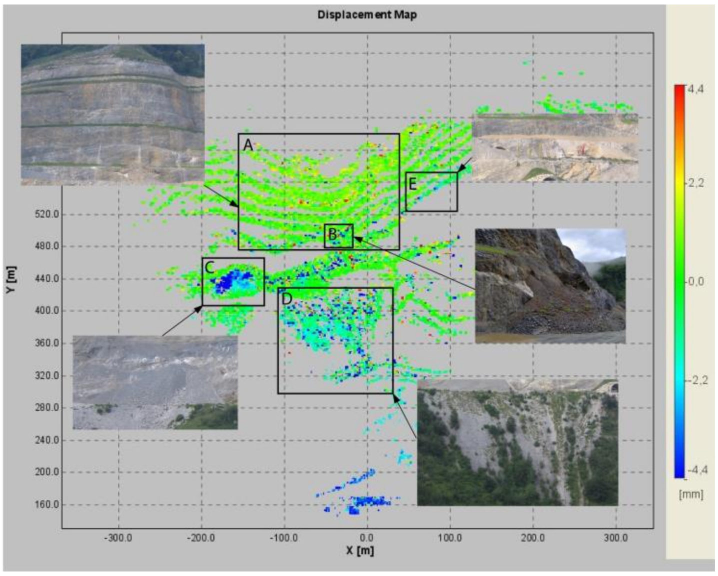
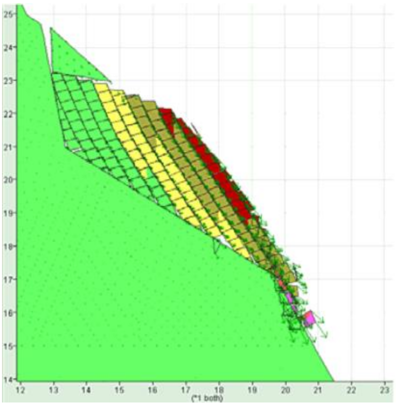


Figure 4: Ground-Based Synthetic Aperture Radar displacement map of the quarry face.

3. 2D AND 3D CONTINUUM AND DISCONTINUUM MODELING

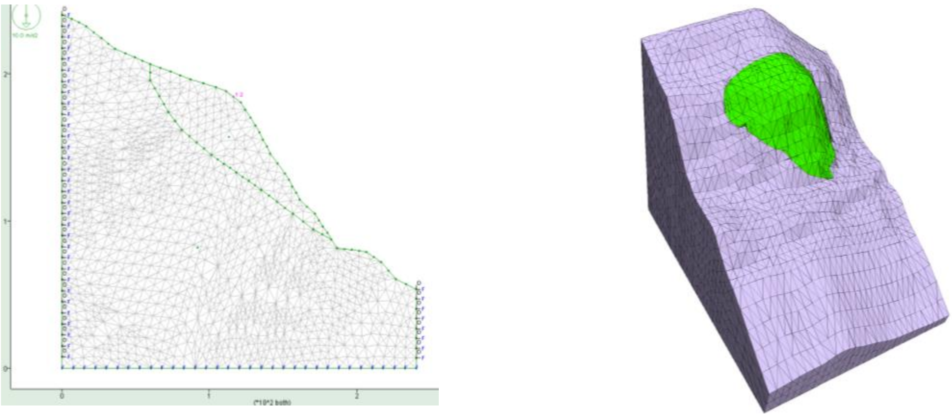
With the above in mind, 2D and 3D continuum and discontinuum modeling were carried out with the dual purpose to assess the stability conditions of the rock volume of interest and to develop possible instability scenarios in view of the future quarrying activities. A special attention was devoted first to the assessment of the strength and deformability characteristics along the sliding surface (typically fault F4), including its continuity and possible existence of rock bridges along it. This was possible through a back analysis of the plane sliding instability occurred at the toe of the slope (Zone B in Figure 4). A 2D discrete element simulation with a friction angle along the joints equal to 35.7° associated to 80% persistence is illustrated in Figure 5. The combinations of friction angle and persistence values along the F4 fault which may lead to a limit equilibrium condition (instability) are also shown.



F4 fault without water pressure $F_s = 1$		F4 fault with water pressure $F_s = 1$	
Persistence [%]	φ_r [°]	Persistence [%]	φ_r [°]
100	45.8	100	53.2
80	36.7	80	43.8
70	34.1	70	41.6
0	30.8	0	40.4

Figure 5: 2D discontinuum modelling of the toe instability (Zone B in Figure 4) and combination of friction angle and persistence values along the fault leading to a limit equilibrium condition.

2D and 3D simulations finalised to assess the stability conditions of the entire slope were then carried out with the 2D and 3D models shown in Figure 6. Also performed were simulations with the combined finite element-discrete element method in order to anticipate possible run-out trajectories along the slope, should instability occurs. In brief, it was found that for an assumed persistence along the sliding surface (i.e. the F4 fault) equal to 80% (i.e. rock bridges are present) and for friction angles in the range 37-44°, the rock buttress between elevation 1000 m and 880 m approximately would reach limit equilibrium conditions (i.e. safety factor equal to 1), in particular if a water pressure distribution was assumed to be present along the sliding surface.



(a) (b)

Figure 6: Continuum and Discontinuum modeling of the quarry face. (a) 2D model, (b) 3 D model.

4. CONCLUDING REMARKS

Based on the results reached, different scenarios were considered regarding the future activities at the quarry as follows: (a) continue excavation and re-profiling below elevation 880 m as initially planned, under closely controlled real-time monitoring; (b) abandon the quarry face with the rock volume in limit equilibrium conditions and continue the mining activities along the neighbouring faces; (c) remove the rock volume of interest, "entirely or partially", by means of slope re-profiling, in conjunction with stabilization measures. At present, option (c) is envisaged.

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Rockfall: Risk-Analysis based on a Magnitude-Frequency-Analysis – a Case Study with an Endangered Family Home

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1. GENERAL

On April 7th 2011 a rock-fall with a block-size of two cubic meters reached a residential building and caused severe damage to a family home. Fortunately the residents were not present in the house during the event – so nobody was injured. A similar rock-fall event took place in the 1980ies, which damaged the garage of the old farmhouse.

he first geological expertise after the event in 2011 declared a high remaining rock-fall hazard for the building and recommended a permanent evacuation. In this first stage rock-fall protection measures were declared as not being capable to protect the house. In order to determine the economic and technical feasibility of mitigation measures it was inevitable to provide a reproducible basis for the decision, whether to evacuate the building permanently or to construct protection measures which would lead to an acceptable remaining risk.

Therefore a risk analysis was carried out applying various methods including engineering geology and forestry including an approach which dealt with the frequency/magnitude relation of past and future rock-fall events providing clues for a failure-prognosis of protection measures potentially to be constructed.

Figure 1: Familiy home on the foot of the slope damaged by the rock-fall 2011, situation sketch.

2. METHODS APPLIED

In order to be able to give reproducible evidence how often such failures on the rock-faces above the settlement took place in the past and would happen in the future, an engineering geological assessment of the rock masses had to be executed, the findings being evaluated und finally supported by the results of a historical research involving past rock-fall events and a frequency-analysis of rock-fall impacts on trees on the slope.

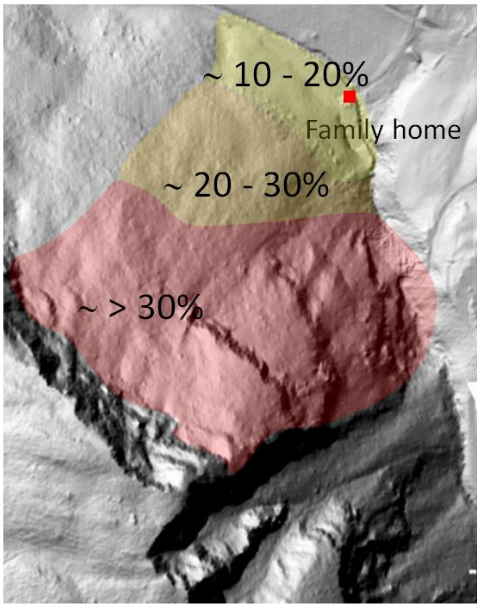
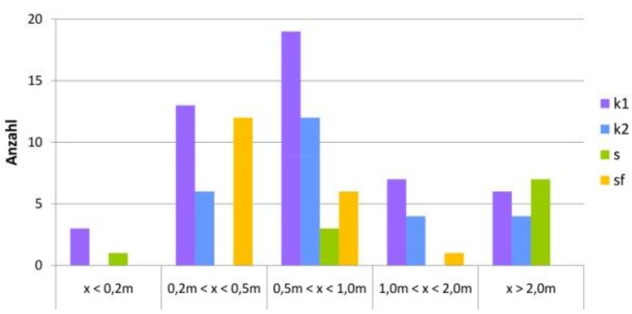


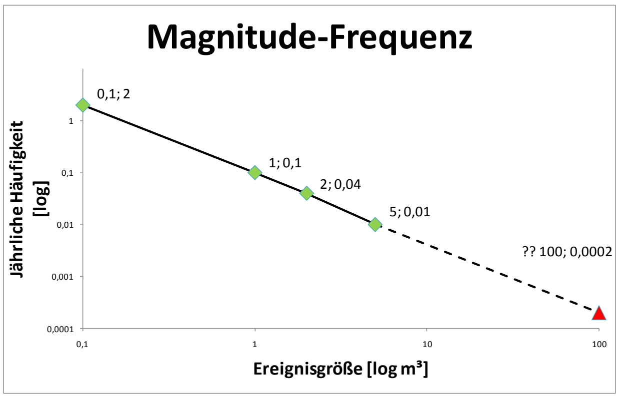
Figure 2: Probability of impact-marks on trees on the slope depending on position (result of field assessment).

Based on the geotechnical analysis of the rock faces and the deposits of historical events present on the slope combined with the research of historical events reaching the foot of the slope, a frequency/magnitude relation was established. Based on this distribution a rock-fall simulation was executed, evaluating the scenarios of the various block-sizes. Given the results of the rock-fall simulation, the failure probability of a potential protection measure (rock-fall protection net fence) could be established, potential failures being overload (energy) and/or blocks jumping over the barrier.



Type	very common to common			rare		
	KK1	KK2	KK3	KK4	KK5	KK6
K1 [m]	0,2	1,00	0,50	1,00	2,00	2,00
K2 [m]	0,2	1,00	0,50	1,00	2,00	2,00
Foliation [m]	0,2	0,50	1,00	1,00	0,50	1,00
Volume [m³]	0,01	0,50	0,25	1,00	2,00	4,00
shape	kubic	slice	slice	kubic	slice	slice

Figure 3: Left: Statistically analyzed spacing of joint sets (104 data) following Heitfeld 1966; Right: Examples of the most frequent blocks.



Block-size	Reconstructed frequency
< 0,1 m³	2 events/y
2 m³	2 events each 25 to 30 y
5 m³	1 event appr. each 50 y
Rock-slide	No proof of endangering run-out

Figure 4: Diagram and table showing magnitude/frequency relation of rock-fall events on the slope.

To calculate the probability of fatalities, the presence of persons in the building, their vulnerability for the different block-size scenarios together with the occurrence frequency of the different block-sizes were used.

Table 1: Probability of fatality in family home on foot of the slope for different scenarios.

Scenario (block size)	No protection	With protection
0,5 m³	1,4E-06	1,4E-09
1 m³ (lower release area)	2,1E-05	2,1E-08
1 m³ (upper release area)	3,5E-05	1,8E-06
2 m³	4,2E-04	4,2E-07
5 m³	2,5E-04	2,5E-06

3. RESULTS

The resulting probability (summarized collective risk for all scenarios) to encounter a fatality in the residential building due to rock-fall without the construction of a protection measure turned out to be in the range of Σ P(DG) = 7,2E-04. With the presence of mitigation measures of a certain capacity (energy and height) the probability could be reduced to an amount of Σ P(DG) = 4,8E-06.

In order to decide, whether the resulting risk without protection measures would be in an inacceptable range and therefore the realization of protection works would be reasonable, the Swiss recommendations “Schutzzielmodell” ([2] Eckhardt 2009) were consulted due to the fact, that at the time being such limits were not published in Austria.

With the results of the risk analysis it could be demonstrated, that the existing risk was too high and mitigations measures should be installed to reduce the risk to an acceptable value. It could also be shown by the analysis, that with protection measures with certain capabilities the risk to persons could be reduced to an acceptable value.

References

[1] Sausgruber, J.T., Mölk, M. & Schreiner, I. (2012). Aussiedlung oder Schutz? Die Risikoanalyse als Entscheidungsgrundlage im Fall eines durch Steinschlag bedrohten Wohnhauses. (Evacuation or protection? Risk analysis as a tool for decision making in case of a residential building endangered by rock-fall). Geomechanics and Tunneling 4, Volume 5, August 2012, pp. 1865-7362.

[2] Eckhardt, A. (ed.) 2009. Schutzzielmodell. Nationale Plattform für Naturgefahren PLANAT, Bern. p. 17.

[3] Hess, J.T. (2008). Schutzziele im Umgang mit Naturrisiken in der Schweiz. – Diss. ETH Zürich Nr. 17956.

[4] Wyllie, D.C. (2006).: Risk management of rock fall hazards. – Sea to Sky Geotechnique, Conference Proceedings, Vancouver 2006.

[5] Hungr, O., Evans, S.G. & Hazzard, J.(1999). Magnitude and frequency of rock falls and rock slides along the main transportation corridors of south-western British Columbia. Canadian Geotechnical Journal 36, pp. 224-238.

[6] Bründl, M., Romag, H., Holthausen, N., Merz, H. & Bischof, N. (2009). Risikokonzept für Naturgefahren – Leitfaden. Teil A: Allgemeine Darstellung des Risikokonzepts. Nationale Plattform Naturgefahren PLANAT.

[7] Dussauge-Peisser, C., Helmstetter, A., Grasso, J.R., Hantz, D., Desvarreux, P., Jeannin, M. & Giraud, A. (2002). Probabilistic approach to rock fall hazard assessment: potential of historical data analysis. Natural Hazards and Earth System Sciences 2, pp. 15-26.

[8] Fell, R, Ho, K.K.S., Lacasse, S. & Leroi, E. (2005). A framework for landslide risk assessment and management. In. Hungr, O., Fell, R., Couture, R. & Eberhardt, E. (eds), Landslide Risk Management, pp. 3-25. London: Taylor & Francis.

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2D Dynamic Sliding Analysis of a Gravity Dam with Fluid-Foundation-Structure Interaction

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The seismic sliding stability of concrete gravity dams is the most important factor when making a safety assessment of such structures. Sliding modes can occur in the dam-foundation contact or in sliding planes formed due to geological conditions.

This work is focused on the behavior of the dam-foundation contact. Several variations of the friction angle in the contact plane are done until the structures displacement is getting progressive. For this simulation, the structure and the foundation are fully discretized with finite elements. Time histories of accelerations are applied in two directions (x/y) to the model. The reservoir is modeled in one case with added masses and in another case with acoustic elements. The pore pressure distribution due to a grout curtain in the contact plane is also considered.

This investigation of the sliding stability shows the qualitative possible and critical displacements of a concrete gravity dam.

1. INTRODUCTION

Investigating the sliding stability of gravity dams at seismic loading shows, that many additional factors are influencing the dynamic behavior. Treating the gravity dam as rigid block may lead to wrong results regarding stresses and displacements, due to the self-oscillations of the structure. Additionally to the dynamic load from the excited structure and water, there are also static loads acting on the structure like the hydrostatic water load and the pore water pressure in the contact plane, which is also influenced by a grout curtain and drainage systems. Due to the self-weight we get a shear force and cohesion in the contact plane according to the Mohr-Coulomb failure criterion, which are the only two parameters against sliding in our model.

Contact modelling is one of the most complicated numerical procedures. It gets even more complicated if the problem is dynamic. Many parameters, e.g. time-integration schemes and time integration factors, may be influencing the results significantly. Dynamic investigation of structures with contact modelling should be examined critically.

2. PROBLEM DESCRIPTION

The structure of interest is a concrete gravity dam. The focus of this work is the sliding safety of such a structure on a horizontal rock foundation due to seismic loading.

2.1. Structural Model

The structural model contains three parts, the gravity dam, the foundation and the reservoir, which are assembled together by specific interaction conditions.

The geometry of the gravity dam is based on the dimensions of the Birecik dam and has therefore a height of 62.5 meters. The finite element dam model is discretized with linear quadrilateral and triangular elements. The linear triangular elements are only used near the contact surface between dam and foundation, because of the mesh refinement, due to the use of linear elements.

The foundation has a total length of 300 meters and a height of 100 meters. The boundaries are fixed normal to their surface for static loading conditions. The grout curtain is situated approximately in middle of the foundation model, which is 7 meters in distance from the upstream surface of the dam and he reaches 30 meters into it. The finite element foundation model is fully discretized with linear quadrilateral elements.

The reservoir is modeled once with acoustic elements and on the other hand with an added mass approach. It has a length of 150 meters and the same height as the gravity dam model. The same elements are used as for the foundation. The boundary condition on the end of the reservoir is defined as viscous boundary, so no reflection can occur. On the upper surface, the pressure is zero and no interaction with the foundation is accounted for.

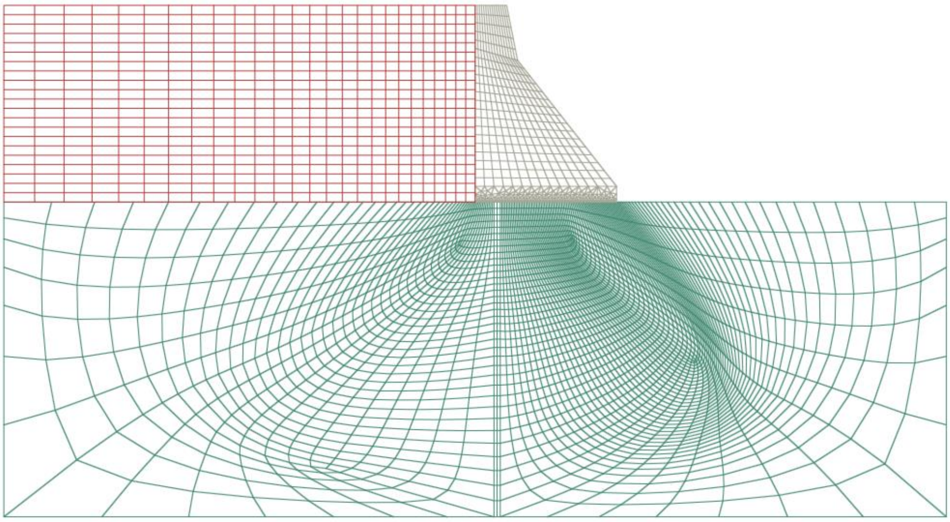

Figure 1: Gravity Dam Dimensions and FE-Model.

Table 1: Material Properties.

	Density [kg/m^3]	Permeability [m/s]	Poisson – Ratio [-]	Youngs/Bulk – Modulus [MPa]
Gravity Dam	2500	0	0.17	25000
Foundation	0	10^{-4}	0.2	30000
Grout Curtain	0	10^{-8}	0.2	27000
Reservoir	1000	-	-	2200

2.2. Contact Modeling

Besides the structural modeling, the contacts between the different parts have to be defined. For the interaction of the gravity dam and the reservoir, the coupling is defined as “tie constraint”, so no relative movement is possible.

The interaction modeling between the dam and the foundation is much more complicated. Therefore, the contact modeling technique has been defined as simple as possible, to get proper and converging results, which could not be that easy to achieve in a transient dynamic simulation. The contact in Abaqus/Cae is defined as finite sliding with a “surface to surface” discretization. For the tangential behavior, the penalty formulation is used, which means that the friction angle and a maximum elastic slip have to be specified. The friction angle is changed in each simulation separately and for the elastic slip the default value of 0.005 is used. The cohesion is neglected in these simulations, so the friction angles used, can be understood as the residual friction angle.

The normal contact formulation is set to “hard contact”. Additionally to these parameters any separation of the contact surfaces is neglected, to reach convergence more easily. The contact modeling and the accompanying convergence problems are also the main reason for using linear elements instead of quadratic ones.

2.3. Dynamic Modeling

In the last step of the simulation, the seismic loading is applied. The accelerations are acting in both directions X and Y on the foundation boundaries in normal direction. The two orthogonal independent acceleration-time-history records have been generated according to the Austrian guidelines and are based on spectra. The maximum acceleration in each direction is set to 0.1g. For solving the equation of motion, implicit direct time integration according to Hilber-Hughes-Taylor (α-Method) is used. Because of the contact modeling the time integration parameter α is set to -0.333. This value accounts for maximum numerical damping, which means that the high frequency responses of the structure are neglected and convergence is reached more easily.

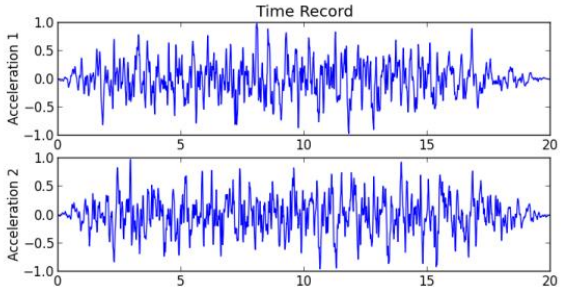

Figure 2: Acceleration-Time-History Records.

For all simulations Rayleigh-Damping is applied to the model. The stiffness- and mass-proportional damping factors are calculated for the first and third eigenfrequency of the dam-reservoir system, for a critical damping factor of 5 percent.

3. RESULTS

All simulations have been performed for different friction angles and zero inclination of the contact plane. The friction coefficient starts at 1.0 (45 degrees) and is reduced until the displacement is getting progressive.

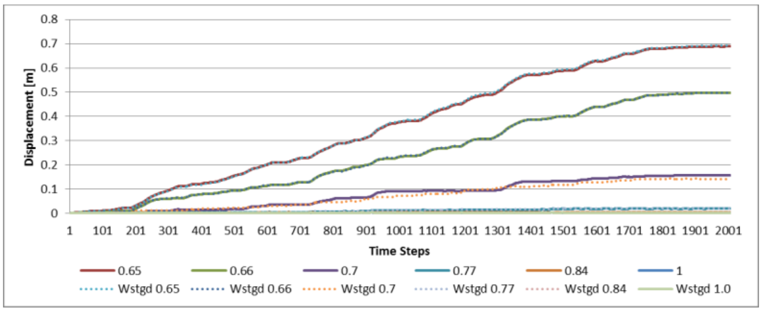

Figure 3: Gravity Dam Displacement for different Friction Coefficients between Acoustic Elements and Added Mass.

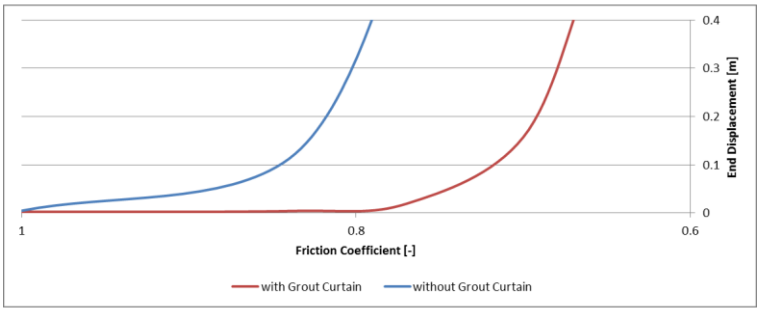

Figure 4: End-Displacement comparison for different Friction Coefficients between the Model with and without Grout Curtain.

Using two different modeling techniques of the reservoir, the Acoustic Elements and the Added Mass approach according to Westergaard, showed that the results of displacement are not differing very much. The increase of the displacement due to seismic loading and lowering of the friction coefficient behaves more or less logarithmic for both kinds of the reservoir discretization. The end-displacement of the gravity dam, after 20 seconds of transient earthquake, starts to get progressive after reaching a friction coefficient of approx. 0.8 (38.7 degrees). By reducing the friction coefficient even more, the simulation stopped at a value of 0.65 (33 degrees).

Investigations of the displacement behavior for conditions where the grout curtain isn't working anymore, which means that the pore water pressure in the contact plane behaves linear from the upstream to the downstream side of the dam, the simulation didn't converge anymore already at a friction coefficient of 0.7 (35 degrees).

4. CONCLUSION

The investigations showed that different modeling techniques of the reservoir don't have a big impact in the results of the displacement. Nevertheless, one should be aware of the fact that this just holds for the relative displacements between two parts, but can influence stresses, velocities, accelerations, deformations, etc., significantly. Therefore, further investigations regarding this topic should be done. Looking at the end displacements of the structure for the different friction coefficients shows, that by reaching a specific value, in this case 0.8 with and 0.84 without grout curtain, the displacement gets progressive.

It can be concluded that the failure of such a structure will happen suddenly after reaching a specific value of resistance.

References

[1] Bathe, K.J. (2001). *Finite Elemente Methoden.* Berlin Heidelberg: Springer.

[2] Chopra, A.K. and Hall, J.F. (1982). Hydrodynamic effects in the dynamic response of concrete gravity dams. *Earthquake Engineering and Structural Dynamics*, pp. 333-395.

[3] Dassault Systemes. (2011). *Abaqus 6.11-EF2 Documentation.* Dassault Systemes.

[4] Westergaard, H.M. (1933). Water pressure on dams during earthquakes. *Transactions, ASCE, Vol. 98*, pp. 418-472.

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Dynamic Stability Analysis of a Gravity Dam with Newmark-Method

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One of the important factors in the safety assessment of gravity dams is the seismic sliding stability of such structures. Sliding can occur in the horizontal joints and one of the joints which are more likely to experience relative motion is the interface between dam and foundation. The sliding displacement of the dam monolith relative to the ground can be estimated by Newmark Rigid Block analysis. This study is investigating the sliding displacement of the dam monolith for various friction coefficients by Newmark method (Rigorous and Simplified). The dam is subjected to a horizontal ground motion records and hydrodynamic force on dam's face is modeled by added mass method.

The investigation shows the importance of two factors for performing Newmark Analysis. The first parameter is choosing an appropriate friction coefficient and the Second element is applying appropriate time-history acceleration. Furthermore, the results show that the simplified Newmark method is estimating the total displacement fairly close to those from Rigorous Newmark Analysis.

1. INTRODUCTION

Failures of dams are infrequent, but it can be extremely high consequence events. Therefore, the assessment of the dam safety is treated with great care. Concrete gravity dams traditionally have been designed and analyzed by simple procedures. The earthquake forces were treated as static forces without considering ground-motion characteristic and the dynamic response of the dam-reservoir-foundation system. For many dams built at the beginning of twenty's century, the design did not correspond to today's requirements. It is not surprising if many existing dams were considered safe are now unsafe based on recent specifications.

The design of concrete gravity dams is generally performed by assuming complete bonding in the horizontal joints. However, it is necessary to evaluate the possibility of relative motions such as sliding which influence the stability of the structure. One of the important horizontal joint is the interface between the dam and the foundation. therefore, possibility of sliding in this interface has to be evaluated during safety analysis of a gravity dam.

2. PROBLEM DESCRIPTION

The structure of interest is Birecik concrete gravity dam and it has a height of 62.5 meters from the foundation and 45 meters wide at its base. The dam is subjected to a horizontal acceleration which has been generated according to the Austrian guidelines and it is based on spectra with the maximum acceleration was set to 0.1g (Figure 1).The focus of this work is the sliding safety of concrete gravity dam on a horizontal rock foundation due to seismic loading and estimating sliding displacement by Newmark method.

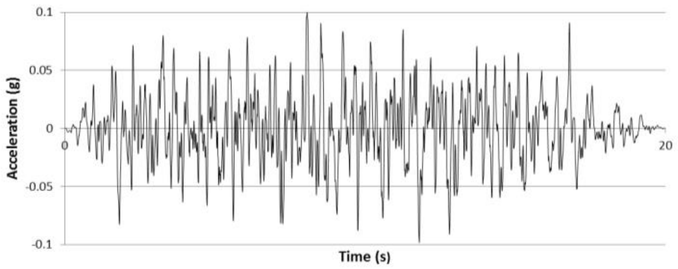

Figure 1: Horizontal ground acceleration in g unit.

2.1. Newmark Method

The pseudostatic method of analysis provides the factor of safety but no information on deformations associated with the failure. Since earthquake-induced accelerations vary with time, the pseudostatic factor of safety will vary throughout an ground-motions. Newmark [1] proposed a method of analysis that estimates the permanent displacement of a slope subjected to ground-motions by assuming a slop as a rigid block resting on an inclined plane. When a block is subjected to a pulse of acceleration that exceeds the yield acceleration, the block will move relative to the ground. The relative acceleration is given by:

$$\ddot{u}_{rel}(t) = a(t) - a_y$$

Where $\ddot{U}_{rel}$ is the relative acceleration of the block, $a(t)$ is the ground acceleration at time t and a_y is the yield acceleration. By integrating the relative acceleration twice and assuming linear variation of acceleration the relative velocity and displacement at each time increment can be obtained (Figure 2).

Sliding is initiated in the downstream direction when the upstream ground acceleration $a(t)$ exceeds the yield acceleration a_y. Downstream sliding ends when the sliding velocity ($\dot{u}_{rel}$) is zero and the ground acceleration drops below the yield acceleration.

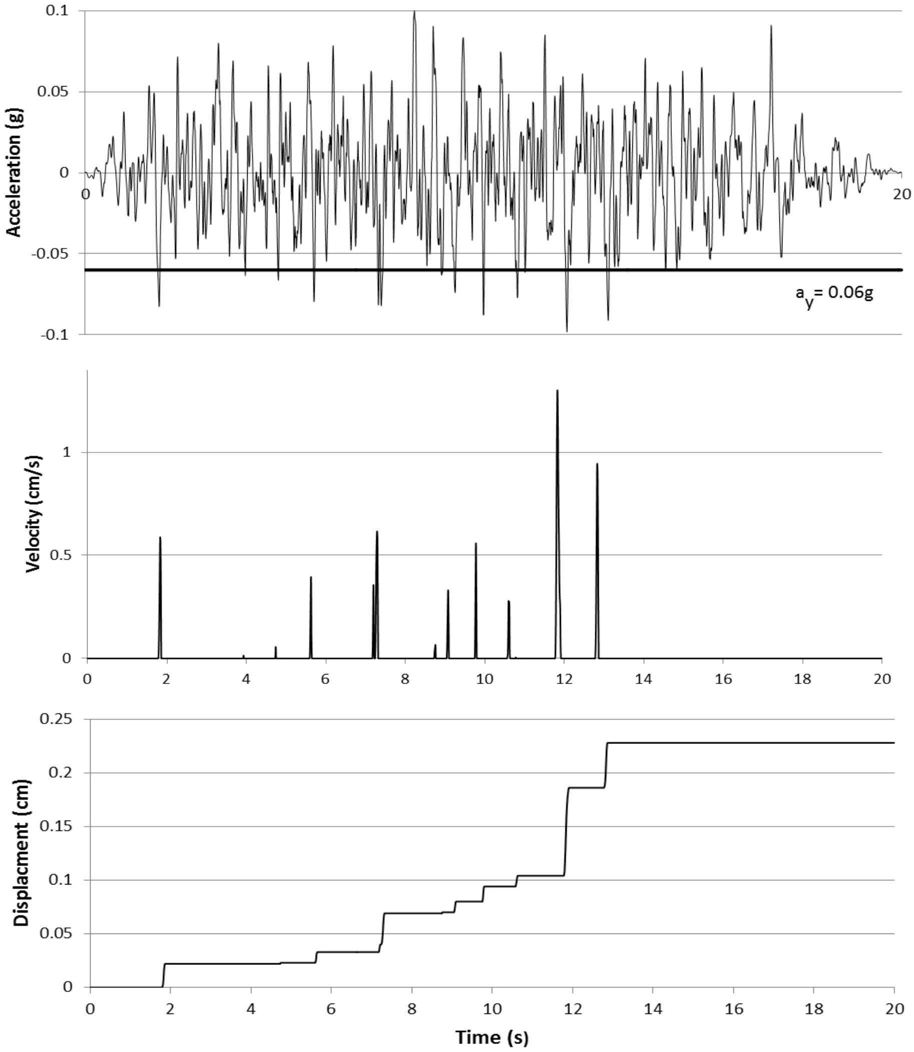

Figure 2: Illustration of the new mark method for D/S sliding of the gravity dam. $a_y = 0.06g$.

2.2. Yield Accelerations

Conducting a Newmark analysis requires characterization of two key elements. The first element is the dynamic stability of the rigid block and it can be quantified as the yield or critical acceleration $(\boldsymbol{a_y})$. This parameter is the threshold ground acceleration necessary to overcome sliding resistance force and initiate permanent block movement. The second parameter is the ground motion records to which the block will be subjected.

To perform a Newmark analysis the gravity dam assumed to be a rigid body of mass M and weight W supported on horizontal ground that is subjected to acceleration $\boldsymbol{a(t)}$. In reality, the dam is bonded to the foundation, however, in this study the dam is assumed to rest on horizontal ground without any mutual bond and the only force against sliding of the dam is the friction force between the base of the dam and the ground surface. Selecting an appropriate friction coefficient ($\boldsymbol{\mu_s}$) is complicated because after earthquake forces overcome the bond between dam and foundation rock, the cracked surface will be rough and the friction coefficient for such a surface is significantly higher than for a planar dam-foundation interface.

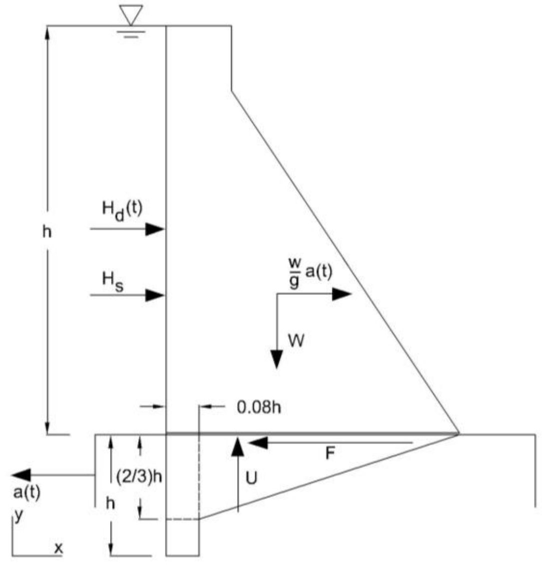

Figure 3: Forces acting on the dam before downstream sliding.

The hydro-static force $\boldsymbol{H_s}$ acting on the face of the dam is always pushing the dam in the downstream direction. The inertia force associated with the mass of the dam is $-(\boldsymbol{W/g})\boldsymbol{a(t)}$ and it is acting opposite to the acceleration direction. The hydro dynamic force can be determined as below:

$$H_d(t) = -a(t)\int P_{hd}(z)\,dz = -M_{hd}a(t)$$

Where $P_{hd}(z)$ is the hydrodynamic pressure on the upstream face of the dam due to unit acceleration in the upstream direction and M_{hd} is the added mass which moving with dam and produces inertia force. The added mass M_{hd} can be determined by Westergaard [2] equation as below:

$$M_{hd} = \int_0^h \frac{7}{8}\rho\sqrt{h(h-z)}dz = 0.583\rho h^2$$

Where ρ is the density of water.

Consider the equilibrium of forces shown in Fig. 3, where the friction force F before the dam starts to slide is:

$$F = \mu_s(M - U)$$

Where U is the uplift force at the base of the dam with grout curtain and it is shown in Figure 3 according to Austrian approach. The dam is in a state of incipient sliding in the downstream direction when the upstream acceleration $a(t)$ reaches the yield acceleration $\boldsymbol{a_y}$. The yield acceleration can be calculated by[3]:

$$\frac{a_y}{g} = \frac{1}{M + M_{hd}} [\mu_s (M - U) - H_s]$$

Because the hydrostatic force always acts in the upstream direction, the yield acceleration necessary to slide the dam downstream in significantly smaller than that for upstream sliding, therefore the upstream sliding is negligible even for a very strong earthquake.

2.3. Empirical estimation of the Newmark displacement

The Newmark Method is depend on the acceleration records and determining a proper acceleration time history for a specific site is complicated and time consuming. The empirical formulas were developed to estimate the Newmark displacement based on past strong-motion records. Ambraseys and Menu [4] proposed various regression equations to estimate the newmark displacement as a function of yield and maximum acceleration based on 50 strong-motion records from 11 earthquakes. They concluded that the following equation with 0.3 standard deviation is best characterizes the results of their study:

$$log\, D_N = 0.90 + log\left(\left(1 - \frac{a_y}{a_{max}}\right)^{2.53} \left(\frac{a_y}{a_{max}}\right)^{-1.09}\right) \qquad \pm 0.30$$

Where a_y is the yield acceleration, a_{max} is the maximum acceleration and D_N is the newmark displacement in centimeters. Different forms of equations have been proposed in other studies with additional parameters to estimate newmark displacement. Jibson [5], proposed the following regression equation which is known as jibson93 and it based on 11 acceleration records which suitable for a_y values of 0.02, 0.05, 0.10, 0.20, 0.30 and 0.40g with 0.409 standard deviation:

$$log\, D_N = 1.460 log I_A - 6.641 a_y + 1.546 \qquad \pm 0.409$$

Where a_y is the yield acceleration (in g's), I_A is the Arias intensity (in meters per second) and D_N is the newmark displacement in centimeters. The Arias intensity (Arias,[6]) is a measure of the strength of a ground motion and can be determined by the equation below:

$$I_A = \frac{\pi}{2g} \int_0^{T_d} a_{(t)}^2 dt$$

Where g is the gravity, $a_{(t)}$ is the ground motion acceleration and T_d is the duration of the ground motion. The Arias intensity measures the total acceleration content of the records and it provides a better parameter for describing the content of the strong-motion record than does the peak acceleration. In the Jibson93 equation, a_y is a linear term and it makes the model overly sensitive to small changes of yield acceleration. Jibson et al. [7] modified the equation to make all terms logarithmic and then performed rigorous analysis of 555 strong-motion records from 13 earthquakes for the same a_y values as indicated for Jibson93 to generate the following regression equation:

$$log\, D_N = 1.521 log I_A - 1.993 A_C - 1.546 \qquad \pm 0.375$$

3. RESULTS

The first part of the study was to perfume a rigorous rigid-block analysis for the gravity dam for various friction coefficients. The integration procedure has been programed by MATLAB for friction coefficients of 0.84, 0.77, 0.7, 0.66, and 0.65.

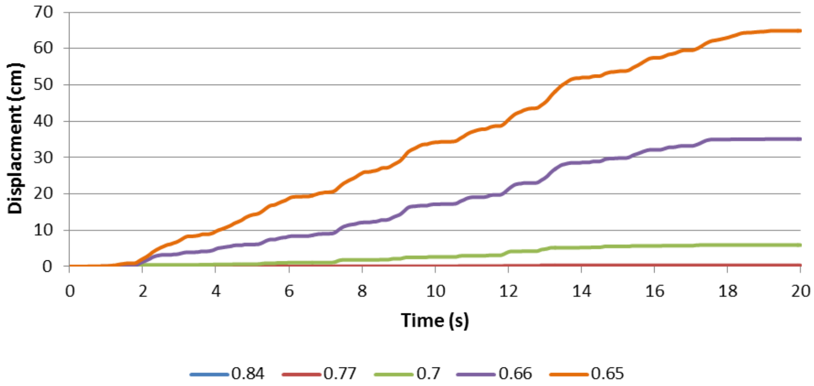

Figure 4: Gravity Dam Displacements for different Friction Coefficients.

The yield acceleration correspondent to friction coefficient 1.0 is well above the peak acceleration 0.1g and no displacement was occurred. On the other hand, the total displacements of lower friction coefficients changed dramatically from 0.7 to 0.65.

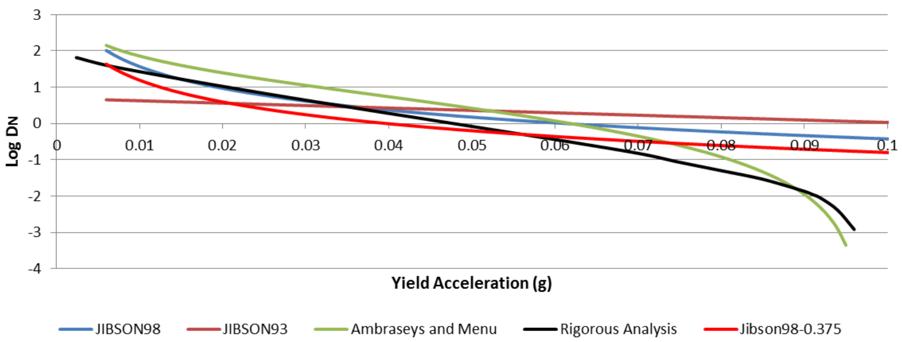

Figure 5: Comparison of Empirical Equations and Rigorous Sliding-block Analysis.

In the second part of the study, empirical relations have been investigated for different yield accelerations and the results compared with rigorous Newmark analysis. It can be seen from Figure 5, that from yield acceleration 0.06g to 0.01g where we have significant displacements, the Jibson98 regression equation estimated the total displacements very close to those from rigorous analysis. Although we have a negligible displacements for yield acceleration larger than 0.06, the results from the rigorous sliding block analysis are close to the results from Ambraseys and Menu equation.

4. CONCLUSION

The investigations showed that although the Newmark method is easy to apply to the gravity dam, determination of appropriate friction coefficient is complicated for dam and foundation interface. Because a very small changes in friction coefficient can leads to a very large difference in displacements. The other key element in this analysis is choosing appropriate ground-motion records. Choosing a ground-motion record with low peak value can lead to underestimation of the displacements. Further investigation must be done for choosing appropriate friction coefficient and earthquake records.

Comparison of empirical regression equations and Rigorous Newmark analysis has shown that in this study, the Jibson98 equation can estimate the sliding displacements for low friction coefficients ($\mu \leq 0.77$) fairly close to those from the rigorous sliding-block.

References

[1] Newmark, N. (1965). “Effects of earthquakes on dams and embankments,” Geotechnique, Vol. 15, No. 2, pp. 139-160.

[2] Westergaard, H. M. (1933). Water pressure on dams during earthquakes. Transactions, ASCE, Vol. 98, pp. 418-472.

[3] Chopra, A. K. and Hall, J. F. (1982). Hydrodynamic effects in the dynamic response of concrete gravity dams. Earthquake Engineering and Structural Dynamics, pp. 333-395.

[4] Ambraseys, N.N., Menu, J.M., 1988. Earthquake-induced ground displacments. Earthquake Engineering and Structural Dynamics 16, pp. 985-1006.

[5] Jibson, R.W., 1993. Predicting earthquake-induced landslide displacements using Newmark's sliding block analysis. Transportation Research Record 1411, pp. 9-17.

[6] Arias, A., 1970. A measure oof earthquake intensity. In: Hansen, R.J. (Ed.), Seismic Design for Nuclear Power Plants. Massachusetts Institute of Technology Press, Cambridge, MA, pp. 438-483.

[7] Jibson, R.W., Harp, E.L., Michael, J.M., 1998. A method for producing digital probabilistic seismic landslide hazard maps: an example from the Los Angeles, California area. US Geological Survey Open-File Report, pp. 98-113.

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Geotechnical Observations from the Niagara Tunnel Project: Numerical Back Analysis for Application to shaft Damage Dimension Prediction

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During the excavation of the Niagara Tunnel Project challenges in Queenston Formation, a mudstone, were encountered which slowed the excavation advance. As the tunnel reached 140 m below ground, large overbreak in the order of 3-4 m began to occur due to stress induced spalling, which created a notch shaped geometry. The failure mechanism is the result of increased lateral strain around the excavation due to the anisotropic nature of the mudstone and the high stress concentration at the excavation boundary. Back analysis was conducted and determined at a K_o ratio of 4 a plastic yield zone of roughly 4 m resulted using the Damage Initiation and Spalling Limit approach. Forward numerical prediction of damage in a shaft that will pass through the Queenston Formation at a different site was conducted to estimate the maximum depth of plastic yielding, which was found to be 1.9 m.

1. INTRODUCTION

The newly constructed Niagara Tunnel, for water diversion, went into service in March 2013 after an extended construction period. Difficult tunnelling conditions were encountered in the Queenston Formation, which originally was to be almost 80% of the tunnel length [1]. The 14.4 m diameter Tunnel Boring Machine (TBM) began excavating the 10.2 km long tunnel in September 2006 and excavation was completed in May 2011. Final lining and grouting was completed in early February 2013.

The tunnel was constructed to divert water from above Niagara Falls to an existing power station, the Sir Adam Beck Generating Station (SAB-GS). The major benefit of the project is that it reduces the percentage of time, from 60% to 15%, for which the allowable water diversion exceeds the capacity of the SAB-GS. This paper is intended to discuss the geotechnical challenges faced during the tunnel excavation and explore through numerical back analysis the conditions leading to maximum observed overbreak.

1.1. Geotechnical Properties for Numerical Analysis

The tunnel passes through eleven formations of the Appalachian sedimentary basin in North America. The formations within the basin lie relatively flat, dipping 6 m/km [2]. Southern Ontario is relatively flat, with the exception of several topographic features including the Niagara Escarpment, the Niagara River Gorge and the buried St Davids Gorge. Perras et al. [2] determined that there is a large increase in the horizontal stress magnitude, from 10 to 24 MPa (σ_H in the Queenston) at the nominal elevation of the bottom of the Niagara River Gorge, roughly 40masl.. This corresponds to a stress ratio change from approximately 2.5 to 6 at a similar elevation, as shown in Figure 1a, and is in agreement with previous studies by Yuen et al. [2].

The sedimentary formations which the tunnel was excavated through ranged from limestones, shales, sandstones, mixed sandstone and shale, and mudstone (Queenston) formations. There is a wide spectrum of Unconfined Compressive Strengths (UCS) as indicated in Figure 1b, particularly for the formations above the Queenston. The Queenston strength lies between 20 and 50 MPa within the elevations which were tunnelled through. The UCS of the Queenston is anisotropic, but crack initiation (CI) is isotopic. The average UCS (39 MPa) and CI (15 MPa) values [2] were used as a starting point for the analysis.

2. OBSERVATIONS FROM THE NTP

Observations of the overbreak indicated four behaviour zones, Figure 1c, three within the Queenston [2]. Zone 1 is all the formations above the Queenston. Zone 2 lies at the contact between the Whirlpool and Queenston formations, which is a disconformity. The reduction in stress due to a stress shadow and jointing created large blocks failing from the crown. The overbreak was observed to break back to the overlying

Whirlpool Formation to a maximum depth of 1.4 m, at which time forward spiling was used to advance the tunnel. When the tunnel reached maximum depths, 140 m deep, stress induced failure was observed. However; the behavior was influenced by the buried St Davids Gorge which the tunnel had to pass under.

On reaching the structural influence of the buried gorge, Zone 3, overbreak was in the order of 2.0 m. It should be noted that through most of this zone, forward spiling was used. Vertical jointing, spaced 2-3m, and horizontal and inclined shear surfaces were observed. Jointing remained clamped due to the stress concentration and had minor influence on the overbreak geometry. The shear surfaces likely affected the overbreak, although was not observed. The overbreak geometry remained asymmetric throughout this zone, however; it was generally inconsistent in size and shape, due to the influence of the buried gorge.

Stress induced fracturing became more prominent, as the tunnel passed away from the influence of the buried gorge, marking the transition to stress induced overbreak, zone 4. The crown overbreak formed an arch 7-8 m wide with a consistent notch shape, skewed to the left , likely indicating a high stress ratio with the major principal stress orientation slightly inclined from horizontal (Figure 2a). Overbreak reach maximum depths up to 6 m. Failure in the invert continued with induced spall planes, which were marked with plumose and conchoidal surfaces. Minor sidewall spalling occurred in the sidewall area (Figure 2b).

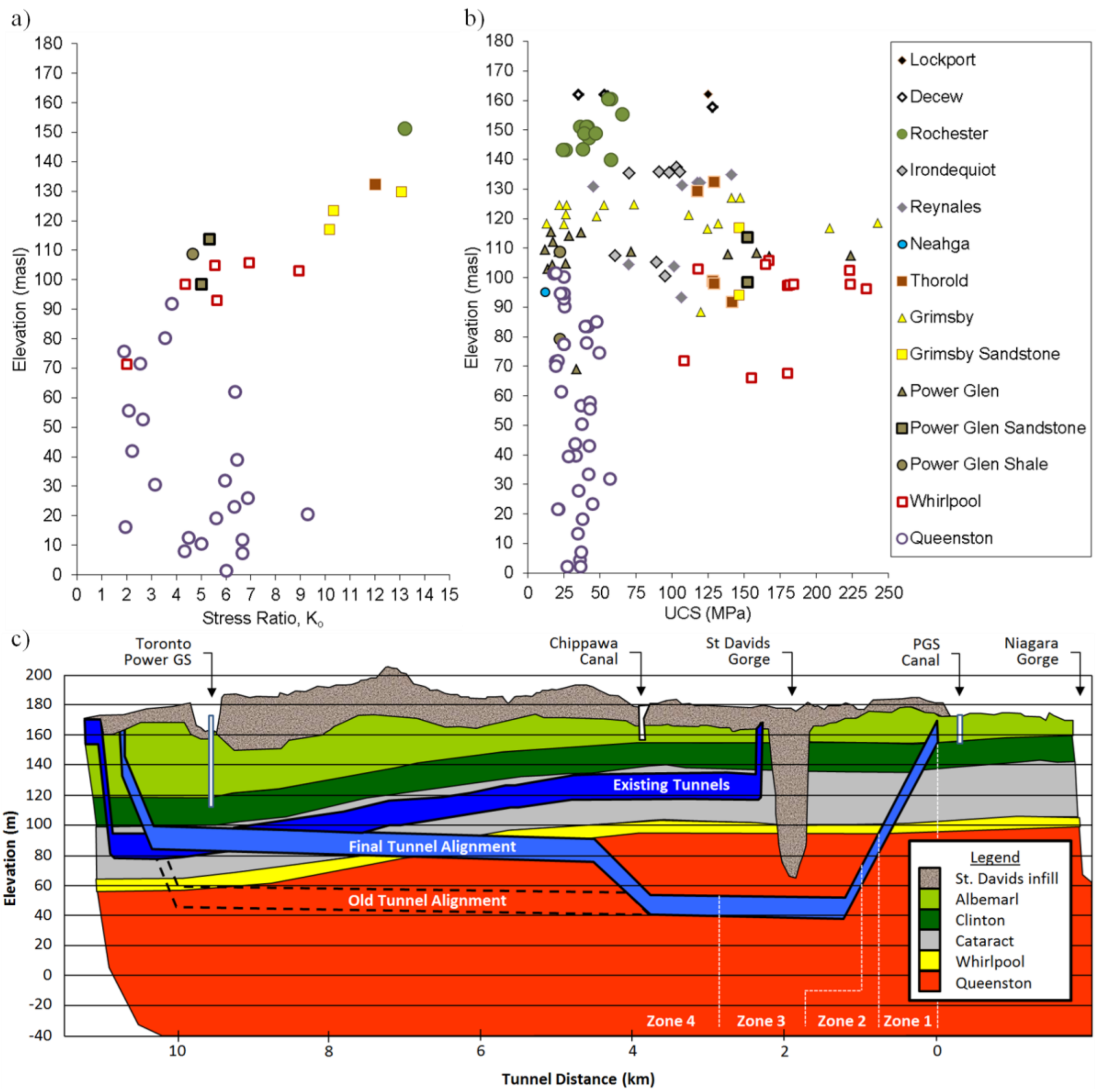

Figure 1: The measured a) stress ratios ($K_o = \sigma_H/\sigma_v$) and b) unconfined compressive strengths (UCS) for the formations and groups c) encountered in the Niagara Tunnel Project.

Figure 2: Observed failure in the Queenston in Zone 4, showing a) typical large notch formed in the high stress regime and b) minor sidewall spalling not associated with structural features.

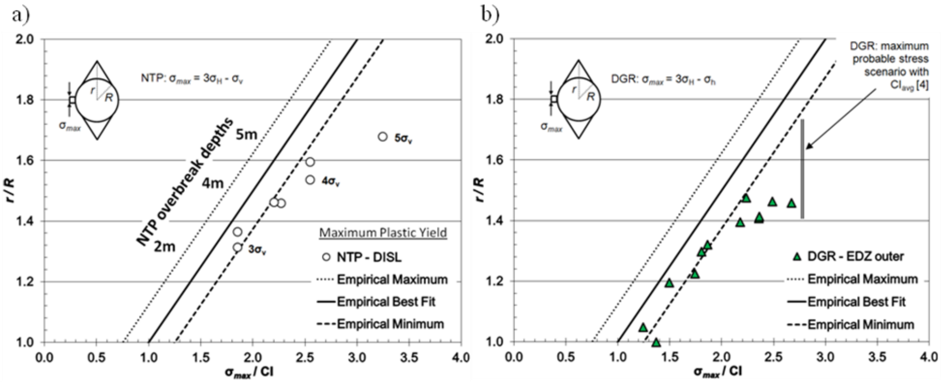

Figure 3: Numerical back analysis results using the DISL approach implemented with the UBJ DY MC model in FLAC 3D for a) the NTP and b) the DGR.

3. NUMERICAL BACK ANALYSIS

The ubiquitous joint double yield (UBJDY) model in FLAC3D, by Itasca, allows for two Mohr Coulomb segments to be used to define the failure envelop, as well as a tension cutoff. This model was chosen due to the simplicity in the input parameters, which only require cohesion, friction and tension values. The model also considers ubiquitous joints to capture the anisotropic strength. The UBJDY envelopes were selected to approximate the Damage Initiation and Spalling Limit [3] peak and residual envelopes. This method can capture the curvature of the DISL peak yield surface for both the NTP and the DGR [4]. The UBJDY model allows for peak and residual properties to be captured with a strain soften/hardening approach, utilizing plastic shear strain as an indicator to reduce/increase the properties.

Since the notch was fully formed prior to installation of rock support, when spiles were not installed, numerical simulation of the rock support has been neglected. Thus the numerical results should yield maximum notch geometries. The observed depth of overbreak (Figure 2a) was used as a target to determine the likely stress conditions. For the stress ratio ranges discussed previously the modeled maximum depth of plastic yielding ranged between 2.0 and 5.0 m (Figure 3a). At a depth of ~4m the stress ratio (K_o) was found to be 4, which is in agreement with the measured values (Figure 1a).

4. IMPLICATIONS FOR CANADA'S DGR

Canada is in the final stages of licensing for a Deep Geological Repository (DGR) to store low and intermediate level nuclear waste (L&ILW). The project will include an access shaft with a radius of approximately 4 m and a slightly smaller ventilation shaft. The shaft will pass through a 200 m thick shale sequence, including the Queenston, overlying the Cobourg host Formation. For a detailed review of the project and the geological setting the reader is referred to the Descriptive Geosphere Site Model [5]. Understanding the damage potential is key to designing cutoffs to restrict flow along the shaft damage zone. Utilizing the understanding from the back analysis of the NTP, modelling was conducted to determine the potential range of the Excavation Damage Zone (EDZ) dimensions at the DGR.

Using the average strength and stiffness values for the Queenston at the DGR site [4] and varying the stress field the range of potential EDZ dimensions were determined using the same UBJDY model approach utilized for the NTP back analysis. The results of the DGR modeling are presented in Figure 3b. The maximum depth of damage for the 4 m shaft model is 1.9 m, with an average depth of damage of 1.3 m. At a maximum stress to CI value between 1.2 and 1.4 there is little to no damage in the numerical models. It should be noted that at the upper end of the possible stress scenarios, for maximum stress to average CI values greater than 2.2, the empirical limits would over predict the depth of damage.

5. CONCLUSIONS

In conclusion the NTP brittle models capture the range of observed notch geometry using the average strength and stiffness properties. Shear based failure criteria was unable to capture the notch shaped geomtry. The target notch depth of 3.8 m (Figure 2a) closely corresponds to the model with K_o = 4.0 and K_{Hh} = 1.4, with a modelled notch of 3.9 m. A similar methodology was implemented to predict the depth of damage around the DGR shaft in the Queenston Formation. The models predict maximum depths of 1.9 m. These models represent a preliminary back analysis of the tunnel and forward prediction for the DGR shaft.

References

[1] Gschnitzer, E. & Goliasch, R. 2009. TBM modification for challenging rock conditions – a progress report of the Niagara Tunnel Project (NTP). TBM-Umbau unter schwierigen Gebirgsbedingungen – ein Baubericht über das Niagara Tunnel Projekt (NTP). *Geomechanics and Tunnelling*, 2, pp. 168–178.

[2] Perras, M.A., Besaw, D. & Diederichs, M.S. 2013. Geological and geotechnical observations from the Niagara Tunnel Project. In Press with the *Bulletin of Eng Geol and the Environ,* BOEG-S-12-00198-3.

[3] Diederichs, M.S. 2007. The 2003 Canadian Geotechnical Colloquium: Mechanistic interpretation and practical application of damage and spalling prediction criteria for deep tunneling. *Canadian Geotechnical Journal*, 44, pp. 1082-1116.

[4] Perras, M.A., Ghazvinian, E., Diederichs, M.S., & Lam, T. 2013. Queenston Formation: Tunnel back analysis and forward shaft prediction. In the Proceedings of the 47th US Rock Mechanics / Geomechanics Symposium, San Francisco, CA, USA.

[5] Intera Engineering 2011. Descriptive Geosphere Site Model. *NWMO Report,* DGR-TR-2011-24.

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Examples for Determination of Tunnel Failure Probability

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1. INTRODUCTION

The design of underground structures is always faced with the challenge of combining overly simplifying analysis methods with both high level of uncertainty and parameter scatter of the ground conditions. While the uncertainties can be reduced to a certain degree by an appropriate investigation and geomechanical characterisation of the ground, the intrinsic scatter of the ground properties always remains present.

Political and economic constraints are increasingly demanding for risk-oriented design of underground structures, featuring both the assessment of most probable costs as well as the quantification of residual risks. If using such an approach, the probability of tunnel failure represents one of the major issues, due to its possibly severe consequences for all sides involved.

2. RISK EVALUATION

Risk is generally defined as the product of unfavourable consequences of a certain event and the respective probability of its occurrence. In engineering it is understood that a risk assessment process has to start with the definition of the system (stating causal and mechanical dependencies) followed by hazard identification, evaluation of the probability of occurrence and a consequence analysis. Based on the probability of occurrence and the consequences of an event, the risk can be determined and checked whether it is acceptable or not. In case of an unacceptably high risk, mitigation measures are determined and applied, and the respective risk is re-evaluated. The process is repeated until an acceptable risk level is obtained.

In tunnel design and construction, the consequence analysis represents a relatively simple task, since costs associated with additional time, material- and personnel efforts, as well as possible third-party damage can be easily assessed. On the other hand, the probability of occurrence can only be assessed with a considerable analysis effort, or is a priori entirely undeterminable (for instance: change of construction and design codes, logistical problems, global economic influences, change of political system and subsequent effects on a big infrastructure project et cetera). This publication concentrates of presenting a sound approach for determining the probability of occurrence, exemplified on the hazard of tunnel failure and discusses the problems associated with the task at hand.

2.1. Risk oriented design approach

The Austrian Guideline for Geotechnical Design [1] already represents a well-structured risk-oriented design approach. The “quantity” and the sort of reaction to the excavation depend on the geological and geotechnical conditions, size of excavation, ground structure and influencing factors, making the act of collecting and interpreting these data clearly a “system definition”. The act of determining the possible failure modes and their associated magnitudes allows identifying hazards, and starting a transparent reasoning along the lines of a sound risk oriented design. Assuming that the allowable risk levels have been set (dictated by the issues of tunnel safety, operability and environmental impact), appropriate risk mitigation measures (support measures and construction methods) can be chosen (Figure 1). Finally, the outcome of their interaction with the ground can be analysed, allowing identification of the residual risks. Once the residual risk reaches an acceptable level and/or the cost of the mitigation measures would be higher than the residual risk, the design can be regarded as satisfying the requirements.

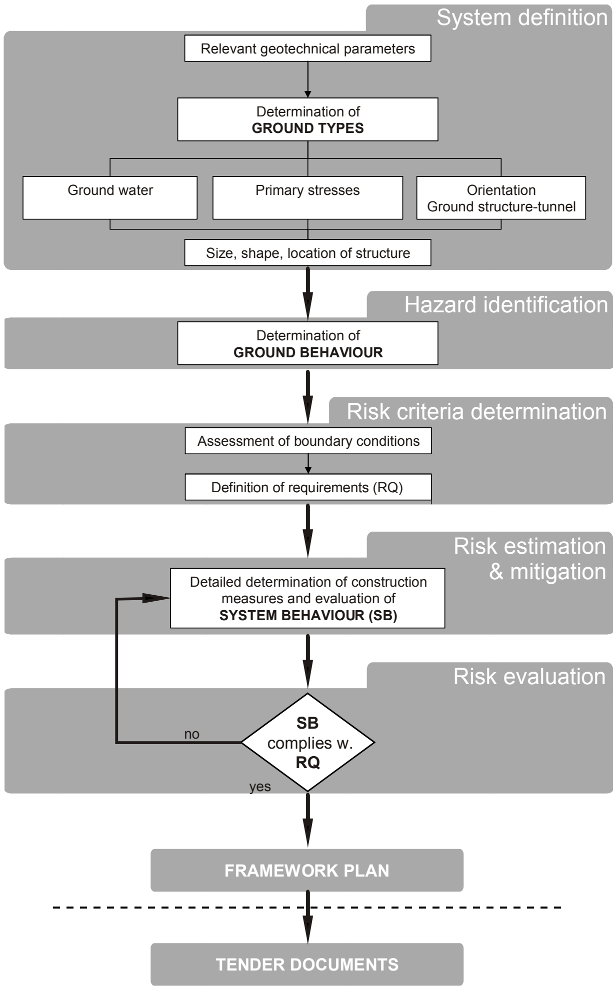

Figure 1. The design methodology as presented in the Guideline of the Austrian Geomechanical Society [1], and the interpretation of its different design steps in the light of a standard risk analysis.

3. FAILURE MECHANISMS AND THEIR CONSEQUENCES

Due to the complex mechanical behaviour of the ground, caused by its inhomogeneity, discontinuities, discontinuity orientation, ground water, anisotropy and not completely known primary stress state (to name a few), there is a plethora of possible failure mechanisms. Based on this information, the identification of the failure mechanism, herein defined as the reaction of the ground to excavating the underground opening without consideration of any support or additional measures, should be conducted. The determination of failure mechanism represents the key concept both for a sound geotechnical design and for risk analysis, since the knowledge of it allows the determination of appropriate support concepts, the estimation of the appropriate analysis method and the proper consequence of support failure. Simply put:

- the application of a very thick and stiff lining in deeply overstressed ground or the application of a ductile support in ground conditions prone to daylighting failure can both be expected to deliver sub-optimal or even catastrophic performance;
- Continuum-based approaches can hardly be applied in jointed rock with block fall as primary mode of failure (this being not the only possible example);
- The consequence of support failure is determined by knowing the ground failure mode and its interaction with the support.

3.1. Defining "failure" and consequence estimation

It is common practice of civil engineers associated with structural analysis to base the ultimate limit state considerations on the imperative that the plastic reserves of the *entire structure* should be ignored. Unfortunately, this practice has been transferred to tunnelling as well, where this concept is either completely or at least partially false – depending on the failure mechanism of the ground.

In case of deep tunnels and weak ground, where high displacements and clearance profile violation represent the main hazard, even the entire lining can exceed its load-bearing capacity and basically disintegrate, and yet no collapse occurs (Figure 2). In order to estimate the bearing reserves, the entire system has to be considered, composed of the deforming (and thus relaxing) ground, shotcrete and rock bolts (in case of conventional tunnelling). If pre-cast concrete segments are used, then the capacity of the system consisting of deforming ground and the support forming a kinematically unstable system should be considered. Compared with the usual structural design approaches (especially as proposed by EuroCode [3]), the deformation capacity of the entire system is much higher, and in contrast to the usual structural analysis, the loads actually *decrease* with lining failure and subsequently occurring additional deformations. The system cannot be divided into separate entities of ground (loading) and support (resistance), but forms an integral structure. Due to the fact that the state-of-the-art analysis methods are not able to reliably estimate the point of failure of such a system, in most cases only the failure of the shotcrete or pre-cast lining can be considered in a reliable manner.

Figure 2. Reprofiling works in the Yacambu adit. Please note the basically completely destroyed initial support ahead of the current advance.

As an example for entirely different failure mode and hazard scenario, the construction of a shallow tunnel in weak ground and low overburden can lead to a sudden and violent collapse. In this case, the bearing capacity of the entire system is defined by the capacity of the lining (up to the point of forming a kinematically unstable structure) and the rock bolts' capacity. The loads induced by the ground *do not decrease* with additional deformations, but are either constant or increase up to the level of the dead load above the tunnel, while the support resistance generally decreases after the initial yielding. Additional complications are given by the ability to detect an abnormal behaviour: while deeply overstressed ground with yielding support allows good insight into the system behaviour at all moments, the time for detecting abnormal behaviour and starting the implementation of mitigation measures is much lower in shallow tunnels in weak ground or tunnels with exceedingly stiff support.

4. EXAMPLES

4.1. Shallow tunnel in rock

For the sake of simplicity, we will assume a shallow tunnel through jointed and weathered granite. Three perpendicular joint sets are intersecting the rock mass, with the vertical joint sets having strongly weathered and open joints with a silt / clay filling. The horizontal joint set is assumed to be less weathered and closed due to vertical loading. Three meters thick, neogenous layer of gravel is assumed above the crystalline rock mass. The sketch of the geological conditions is shown in Figure 3.

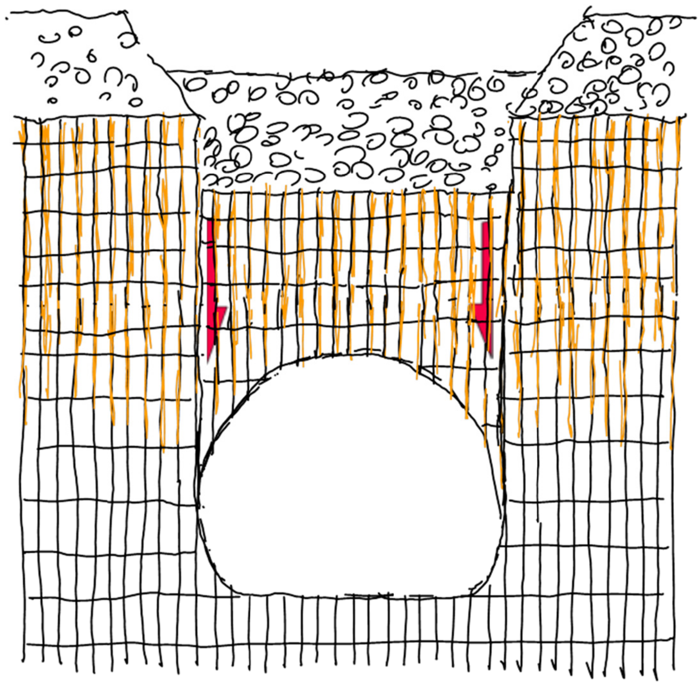

Figure 3. The assumed geological set-up and probable failure mode.

In the first step, the safety factor in case of full-face excavation and no support installation is calculated by a probabilistic analysis of the limit equilibrium for blocks sliding along the vertical discontinuity set.

In order to demonstrate the importance of proper characterization, the limit equilibrium is also determined for “smeared joints” and rock mass properties determined by applying the relationships proposed by Hoek et al. [4] and Cai et al. [2] The assumed range of the parameters has been chosen as shown in Table 1, according to the geological description of the problem.

The results clearly demonstrate the effect of the different characterization methods on the safety factor (Figure 4). The discontinuous approach predicts a definitive vertical shear failure and downward sliding of blocks along joints if no support is installed, while the homogenous ground model suggests stable ground with a slight potential for day lighting failure in case of low confinement stresses. Hence, not only the estimation of the hazard occurrence probability is wrong, but also wrong risk mitigation measures could be the result.

Table 1. Intact rock and discontinuity parameters used in the Monte-Carlo analysis.

		Min.	Max.
Joint set 1 (vertical)	Joint alteration factor J_a [-]	4.00	8.00
	Joint roughness (small scale) J_s [-]	0.75	2.00
	Joint roughness (large scale) J_w [-]	1.00	3.00
	Friction angle φ [°]	15	30
	Normal joint spacing [cm]	20	100
Joint set 2 (horiz.)	Joint alteration factor J_a [-]	2.00	4.00
	Joint roughness (small scale) J_s [-]	0.75	2.00
	Joint roughness (large scale) J_w [-]	1.00	3.00
	Friction angle φ [°]	25	35
	Normal spacing [cm]	20	100
Intact rock	UCS [MPa]	15	50
	Hoek-Brown Constant m_i [-]	10	20
	Unit weight γ [MN/m³]	0.028	
Hom. Granite	Eq. friction angle φ [°]	40	60
	Eq. cohesion c [MPa]	0.10	0.25
	Unit weight γ [MN/m³]	0.028	
Gravel	Friction angle φ [°]	30	40
	Cohesion c [MPa]	0.00	0.05
	Unit weight γ [MN/m³]	0.018	

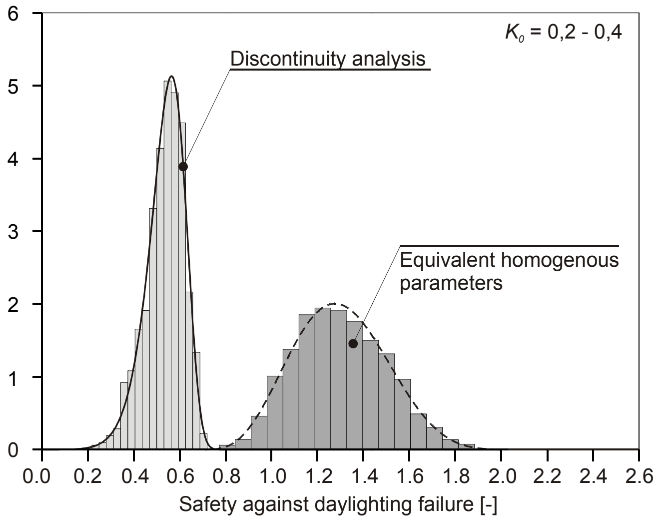

Figure 4. Calculated safety factors against day lighting failure, low confinement pressure.

As the probable failure mode without support is the shearing along the vertical joints, rock bolting in the sidewalls and shoulder areas, in order to intersect and "dowel" the sliding surfaces would be a logical measure (Figure 5). A regular shotcrete lining (thickness 10 – 15 cm) would be sufficient as sealing, preventing smaller blocks from detaching and falling down.

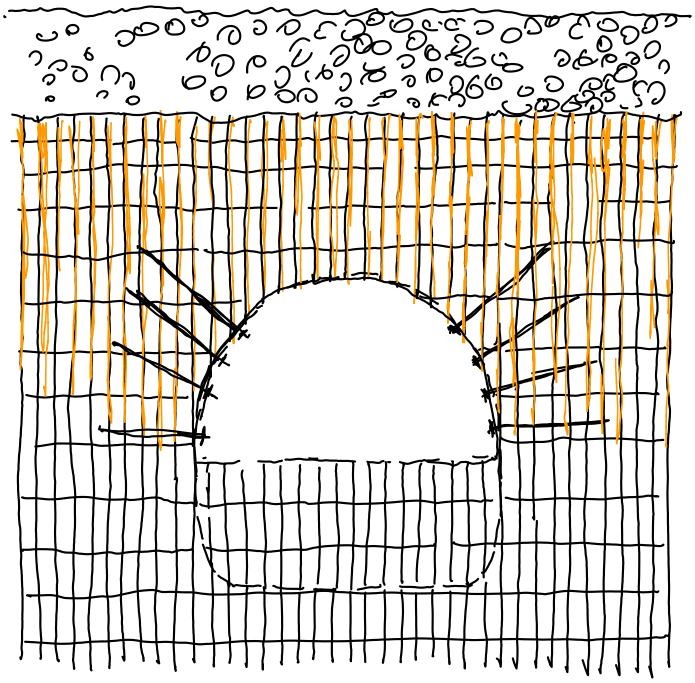

Figure 5. Applicable support concept (top heading with shotcrete lining and systematic rock bolting in the sidewalls).

The interaction between the installed rock bolts and the ground is analyzed by including the simple analytical model provided by Pellet & Egger [5] into the relationships used for determining the ground behavior. It captures the rock bolt contribution to the sliding resistance both by axial elongation and shear (dowel) action. Figure 6 shows the results of the probabilistic analysis of the model with rock bolt installation in sidewalls and shoulders.

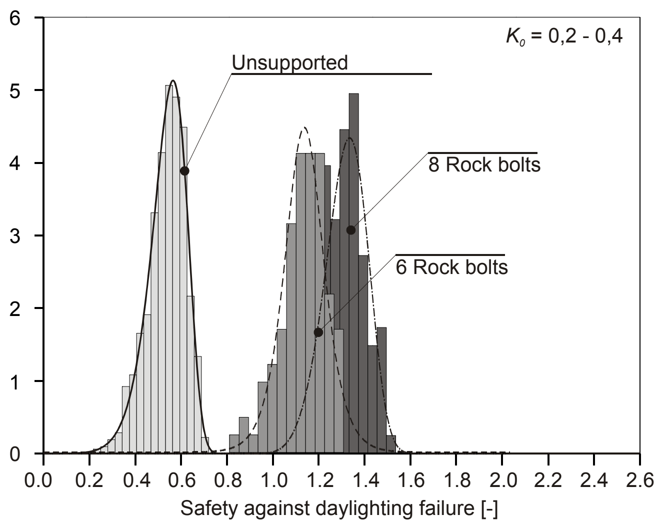

Figure 6. The effect of support measures on the safety factor, as obtained from probabilistic limit equilibrium analysis.

4.2. Deep tunnel in weak rock

In an analogy to the example before, the system behaviour of a deep tunnel with weak ground is predicted by conducting Monte-Carlo simulations with simple calculation methods. Figure 7 shows the result of a probabilistic analysis for the top heading excavation, with a support concept featuring deformation gaps and yielding elements. The likely (and thus, assumed) range of ground properties has been determined by an extensive in-situ test program and used as a basis for a Monte-Carlo simulation coupled with a calculation based on the convergence confinement method. As it can be seen, the frequency plots of the respective longitudinal displacement profiles are in excellent agreement with the monitoring data (Figure 7).

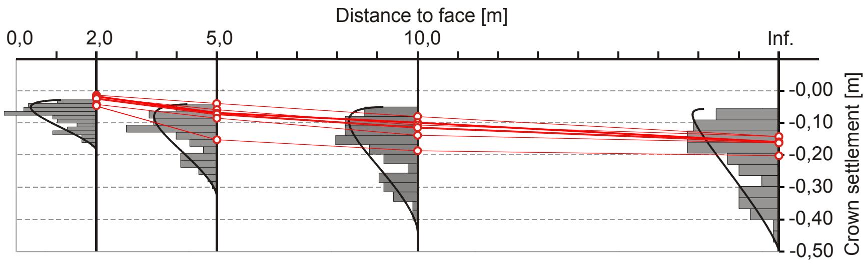

Figure 7. Probability density distributions of the displacement development compared to the monitoring data (lines).

A parameter study depicts the associated frequency distribution of the shotcrete utilisation, demonstrating the effect of the advance rate (and hence, time available for the shotcrete to develop strength before it is loaded) on the lining failure probability (Figure 8).

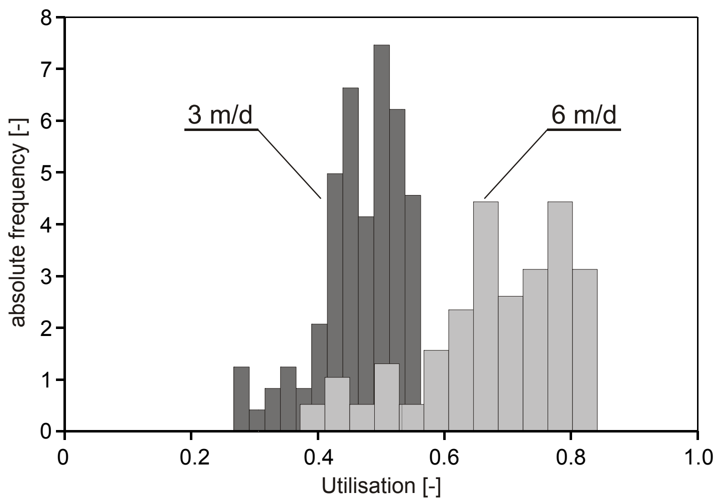

Figure 8. Effect of the advance rate on the shotcrete utilization.

No attempt is made to quantify the overall stability of the system, since the mechanical basics for such a calculation are still limited – as already stated, the state-of-the-art allows the probabilistic estimation of the overall stability of a deep tunnel only with an extreme computational and time effort.

5. CONCLUSIONS AND RECOMMENDATIONS

The analysis methods allowing a sound assessment of the failure probability in tunnelling are currently available. However, the requirement of knowing the correct failure mechanism of the ground cannot be circumvented – otherwise all the efforts render utterly meaningless results.

In case of certain failure modes associated with high confining stresses, the overall stability of the system cannot be soundly predicted by the state-of-the-art characterisation and calculation methods. On the other hand, the failure probability of systems featuring conservative loading (e.g. dead-load) can be determined in a straightforward and reliable manner.

The issue of tunnel failure probability and the associated parent task of risk analysis represent the base for modern, safe and economical tunnel design. Due to the unavoidable uncertainties, only such an approach can yield a plausible and meaningful basis for decision making, and incorporate the investigation efforts, project specific requirements and political constraints in a quantitative manner: the entire design becomes associated with a "price tag" in the end. Risk-oriented design approaches, combined with the observational method, should in the end replace EuroCode 7 [3], which is inconclusive / ignorant for the majority of problems shown above.

References

[1] Austrian Society for Geomechanics. 2008. *Guideline for the geotechnical design of underground structures with cyclic excavation. 2nd revised edition*. Salzburg.

[2] Cai, M. Kaiser, P.K., Uno, H., Tasaka, Y. & Minami, M. 2004. Estimation of rock mass deformation modulus and strength of jointed hard rock masses using the GSI system. *International Journal of Rock Mechanics and Mining Sciences* 41 (1), pp. 3-19, Amsterdam: Elsevier.

[3] Eurocode 7. 2004. *Geotechnical design – Part 1: General rules*. European Committee for Standardisation.

[4] Hoek, E., Carranza-Torres, C. & Corkum, B. 2002. Hoek-Brown failure criterion – 2002 edition. In: Hammah, R. (eds.), Bawden, W. (eds.), Curran, J. (eds.) & Telesnicki, M. (eds.), *Mining and tunnelling innovation and opportunity: Proceedings of the 5th North American Rock Mechanics Symposium*, Toronto, Canada, 07-10 July, pp. 267-273, Toronto: University of Toronto Press.

[5] Pellet, F. & Egger, P. 1996. Analytical model for the mechanical behaviour of bolted rock joints. *Rock Mechanics and Rock Engineering* 29 (2), pp. 73-97.

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