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Determination of Rock Mass Behaviour Types - a Case Study

Markus Pötsch & Wulf Schubert
Institute for Rock Mechanics and Tunnelling, Graz University of Technology

Andreas Goricki & Arnold Steidl
3G Gruppe Geotechnik Graz ZT GmbH

ABSTRACT: In the engineering practice geotechnical engineers and geologists are faced with data which due to their nature have a wide spread. Predictions derived from these data also contain uncertainties. This paper proposes a procedure to facilitate the processing of these data, and to consistently consider uncertain parameters in the rock mass characterisation process. For the application of the procedure a computational model has been developed. It allows modelling the geological architecture, assigning the rock mass parameters and evaluating the rock mass behaviour with different analytical calculation models. A case study from an alpine base tunnel demonstrates the procedure.

1 INTRODUCTION

The determination of Behaviour Types is an integral part in the rock mass characterisation process stipulated by the Austrian Guideline for the design of underground structures with conventional excavation (ÖGG 2001). The design process consists of four general steps including the determination of Rock Mass Types (RMT), Behaviour Types (BT), excavation and support, and the resulting System Behaviour (Schubert 2004). Rock mass characterisation includes the determination of Rock Mass Types and Behaviour Types. The purpose of this paper is to highlight the steps of rock mass characterisation applying a computational model and probabilistic methods. The configuration and application of the computational model is described on the basis of the rock mass characterisation for the Paierdorf exploratory tunnel which was performed during the tender process. The tunnel is part of the investigation campaign for the Koralm base tunnel project (Vavrovsky, Schneider & Harer 2001). Its length is about 5.5 km and the maximum overburden is about 700 m. The tunnel starts in a depth of about 160 m. The access is provided by a shaft with a diameter of about 9 m. It is a challenging project since it explores for a long stretch the conditions of approximated 2 km Lavanttal fault zone (Mussger, Steidl & Harer 2004). The case study is based on the corresponding ground expertise from 3G & BGG (2004).

1.1 *Principles of the determination of Behaviour Types*

The first step in rock mass characterisation starts with the description of the geological model and proceeds by defining geomechanically relevant key parameters for each ground type. Rock Mass Types are then distinguished according to the selected key parameters. The number of Rock Mass Types elaborated depends on the project-specific geological conditions and on the stage of the design process.

The second step involves evaluating the potential rock mass behaviours considering each Rock Mass Type and local Influencing Factors, including the relative orientation of relevant main discontinuity sets to the excavation, ground water conditions, primary stress situation, and the size, shape and location of the underground structure. This process results in the definition of project specific Behaviour Types. The Behaviour Types form the basis for determining the excavation and support methods as well as assist in evaluating monitoring data during the excavation. The distribution of the expected Behaviour Types along the alignment of the underground structure provides the basis for establishing the bill of quantities and the bid price during tender (Goricki 2003).

2 COMPUTATIONAL MODEL FOR ROCK MASS CHARACTERISATION

2.1 *Basic structure of the computational model*

The three-dimensional geological model is cut into slices along the considered tunnel alignment. These slices are called "calculation segments". Key parameters and boundary conditions are assigned to the calculation segments. Key parameters can be the rock type, discontinuity properties, and/or mechanical and hydraulic properties, etc. The Rock Mass Type of the calculation segment results from the current combination of key parameters and their corresponding values.

In the next step the Influencing Factors are assigned to the calculation segments. They represent the relevant boundary conditions of the calculation segment for the determination of the rock mass behaviour. Their assessment is based on the three-dimensional geological and hydrogeological model and, additionally, it can be supported by further investigations such as in situ stress measurements, borehole test, etc.

The calculation segments are analysed with respect to the response of the current Rock Mass Type to tunnel excavation under the corresponding Influencing Factors. For the analysis different analytical calculation models are simultaneously applied. Every model provides a physical value which describes an aspect of the rock mass response. Comparing these results to previously defined delimiting criteria various Behaviour Types can be distinguished. The most critical Behaviour Type is then assigned to the calculation segment. After assigning the Behaviour Type to the calculation segments, the distribution of the Behaviour Types along the considered tunnel stretch is obtained by adding up the length of calculation segments with the same Behaviour Type.

2.2 *Probabilistic data processing*

In probabilistic simulations the deterministic values of the input parameters (key parameters of the RMT and parameters of the geological model) are replaced by statistical distributions. The results are also obtained in terms of statistical distributions. In the proposed method the integration of the output functions is performed with a Monte Carlo simulation. In this simulation the computational model is consecutively calculated various times while in every calculation step (iteration) the input parameter values are varied according to their statistical distribution. The number of iterations must be sufficient to obtain an invariable result. The output values are sampled and approximated by another statistical distribution using a best-fit procedure.

3 MODELLING THE GEOLOGICAL ARCHITECTURE

Every rock mass characterisation has to be based on a three-dimensional geological model from which the relevant parameters are derived. This fact has to be taken into account already during the geological site investigation. Realistic modelling of the geometry of a rock mass depends on the understanding of geological processes and their complex interactions such as tectonic deformation, weathering, morphogenesis, etc. (Riedmüller 1998, Riedmüller & Schubert 2001).

3.1 *Geological overview of the project area*

The Koralm mountain range is part of the Koriden unit within the Middle-Austroalpine nappe complex of the Eastern Alps. The metamorphic bedrock in the project area of the Paierdorf exploratory tunnel is characterised by frequent lithological variations and smooth transitions from one rock type to the other at scales from a few decimetres to some tens of metres. Larger lithological sequences could be distinguished. These are the gneiss, mica schist, marble, amphibolite and cataclastic rock sequences. The geological conditions for the Paierdorf exploratory tunnel can be very roughly subdivided into three sections.

- The dominating geological structure is the Lavanttal fault system at the lower western hill slope of the Koralm. The entire section is characterised by a heterogeneous composition (tectonic melange) of weak fault rocks and sound parent rocks (Riedmüller, Brosch, Klima & Medley 2001). The entire section is characterised by a complex groundwater situation of aquifers and aquicludes.
- The rocks of the upper western hill slope of the Koralm generally are thickly bedded to massive and slightly jointed. Weathering phenomena including discontinuity surfaces, alteration of rock material resulting in rock strength reduction, open joints, etc. were generally encountered up to a depth of about 200 metres.
- The Lavanttal fault system generated the Lavanttal tertiary basin which includes fine-grained sedimentary rocks with very low strength. These rocks are sensitive to water with a high potential of slaking.

3.2 *Computational modelling of the geological architecture*

The parameters for modelling the geological architecture include the location of the lithological sequences, the corresponding rock types, and the fracturing and weathering of the rock mass. The boundaries of lithological sequences have been considered to be fixed according to the geological model. The distribution of the rock types within the sequences has been probabilistically modelled. The length of each zone of a rock type within a lithological sequence has been derived from evaluations of drill cores and related to the tunnel location and orientation.

Another important input parameter is the fracturing of the rock mass as the basis for the determination of geotechnical rock mass properties. Correlations between the fracturing and rock types, and fracturing and weathering, respectively, have been taken into account (Figure 1).

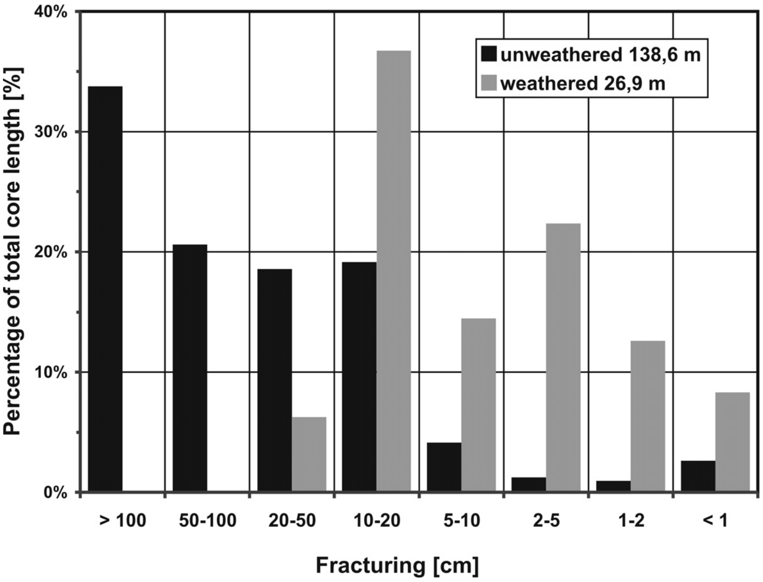


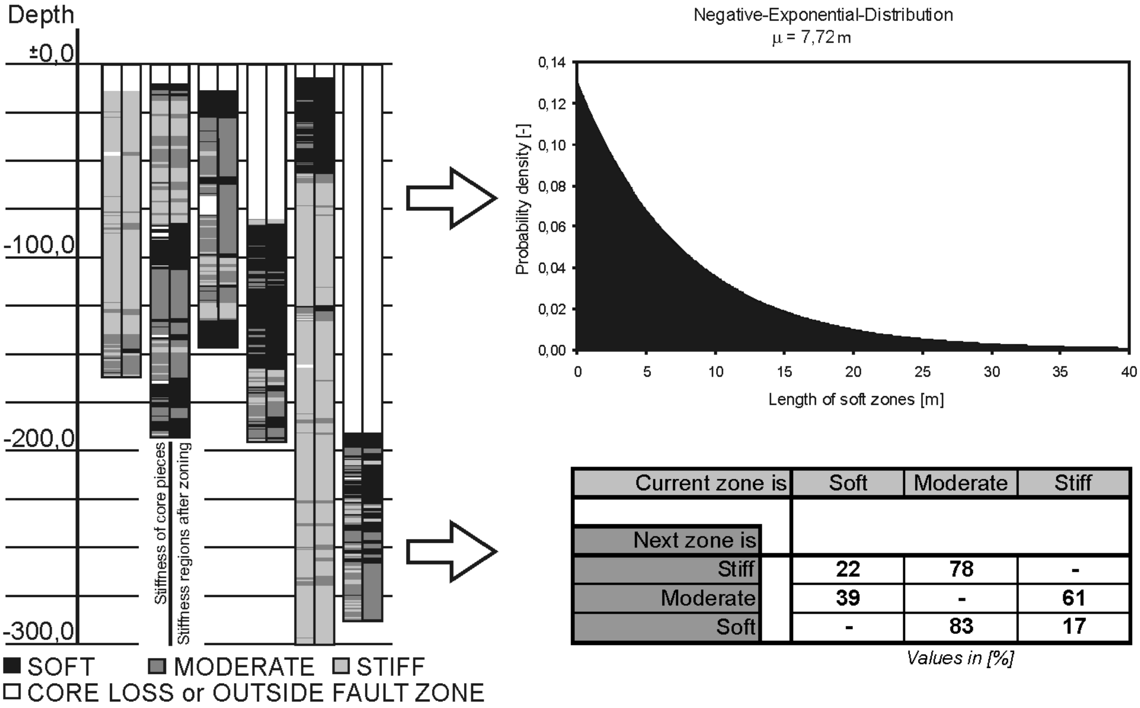
Figure 1. Fracturing of a rock type, distinguished between unweathered and weathered condition

3.3 *Consideration of faults and fault systems*

Faults and fault systems have a major influence on the rock mass behaviour and the required excavation and support (Schubert & Riedmüller 2000). The application of probabilistic methods allows assessing the influence of faults on tunnelling under consideration of uncertain fault locations and properties. One approach to model faults in a computational model is to acquire their thickness, spacing or frequency, and orientation relative to the tunnel axis. The fault pattern in the geological map and the core logs serve as the data basis. Based on these parameters faults can be probabilistically generated. This method is reasonable when the faults are thin compared to the unfaulted rock mass, i.e. when the fault frequency is relatively low. The previously generated basic geological architecture is then adjusted to the newly generated fault pattern (Goricki 2003).

The Lavanttal fault system is a rock mass with a high fault frequency, where the interdependence between faults dominates the rock mass behaviour. It is controlled by stress redistributions due to stiffness contrasts in the rock mass. To account for this effect another approach was applied. The drill cores in the fault zone were evaluated with respect to the stiffness of the core pieces (Figure 2, left). The left side of the drill core bars represent the stiffness of the drill cores, whereas the right side of the drill core bars represent derived stiffness zones which are relevant for tunnelling. Stiff, moderately stiff and soft zones were distinguished. The stiffness zones were analysed with respect to their lengths and sequence. Therefore, density functions for the length of different stiffness zones were introduced. Figure 2, top right shows the distribution of the length of soft zones evaluated from the drill cores. This distribution was considered to be a significant distribution for the Lavanttal fault system.

The different stiffness zones have to be arranged in a sequence which is significant for the considered fault zone. To account for this sequence generation rules (Markov chains) were used. Markov chains determine the stiffness of the subsequent zone based on the stiffness of the current zone. For instance, if the current zone is stiff, the subsequent zone is soft with a probability of 22% or moderately stiff with a probability of 78% (Figure 2, bottom right). The values of the Markov chain are derived form the evaluated drill cores. The application of the described input parameters allows generating fault zones which have properties similar to the faults in the drill cores.



Current zone is	Soft	Moderate	Stiff
Next zone is			
Stiff	22	78	-
Moderate	39	-	61
Soft	-	83	17

Values in [%]

Figure 2. Derivation of input parameter distributions (top right) and generation rules (bottom right) for fault zones from evaluated drill cores (left).

4 DETERMINATION OF ROCK MASS BEHAVIOUR TYPES

4.1 *Assignment of Rock Mass Types and Influencing Factors*

After generation of the geological architecture the mechanical parameters were assigned to each calculation segment according to the current rock type (refer to 3.1). The parameter values and their statistical distributions were determined on an extensive number of laboratory tests. Together with the parameters from the geological model Rock Mass Types of the calculation segments were distinguished. The relevant properties to distinguish Rock Mass Types were the unconfined compressive strength, the degree of fracturing, discontinuity properties and the degree of weathering or carstification. Based on the relevant parameters rock mass properties were derived for each RMT using the GSI (Hoek 1999), results from in situ tests and, additionally, geotechnical experience.

The Influencing Factors were derived from the geological model. The primary stress condition was related to the overburden at the location of the calculation segment and additionally supported by in situ stress measurements and the evaluation of the tectonic history. The results of field mapping and drill core analysis were sampled in orientation plots and served for the derivation of the orientation of the main discontinuities relative to the tunnel, whereby the spread of the orientations was taken into account. The ground water situation was derived from a hydrogeological model and related to highly fractured and/or carstified rock masses.

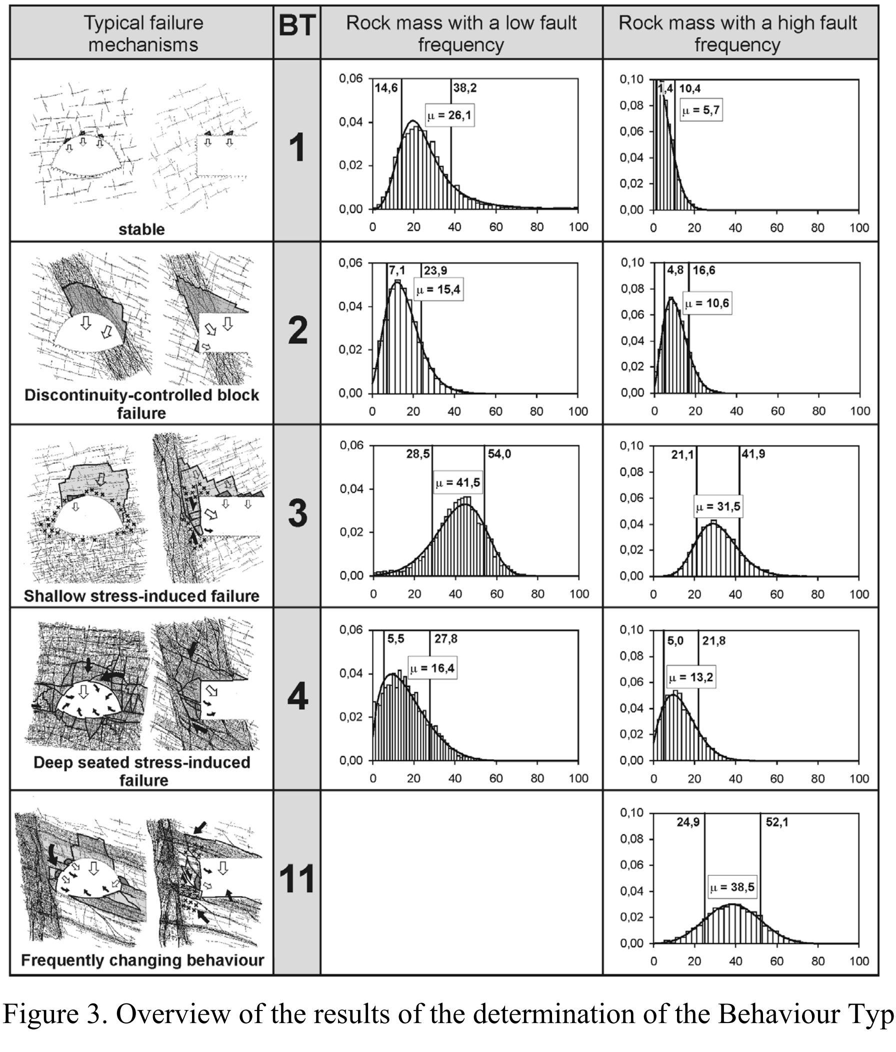


Figure 3. Overview of the results of the determination of the Behaviour Types

4.2 *Evaluation of the rock mass behaviour*

The calculation segments were evaluated with respect to the behaviour of the RMT to excavation under the corresponding Influencing Factors. The rock mass behaviour was determined using analytical models for displacements, depth of stress-induced failure, discontinuity-controlled failures, rock burst, stress redistributions in a heterogeneous rock mass, and flowing and swelling ground. Applying delimiting criteria for the results of the analytical models the rock mass response was classified into Behaviour Types. Delimiting criteria considering the depth of failure, displacements, depth and volume of overbreak, severeness of rock burst, etc. were applied (Goricki 2003).

In a Monte Carlo simulation 5000 iterations of the computational model were performed. Figure 3 gives an overview of the results for regions with a high and a low fault frequency. It includes sketches of typical failure mechanisms for the identified BT. Figure 3 also shows the obtained probability density functions for the occurrence of each BT within the considered section in percent. It includes also the mean values and the 15% and 85% quantiles. The variance of the density functions highlights the uncertainty of the geological-geotechnical prediction and allows assessing the influence of the different BT on tunnel excavation. Applying only singular deterministic values neither the different BT, nor their quantity, spread and interdependence could have been reasonably determined.

5 CONCLUSION

A procedure for the determination of Behaviour Types according to the Guideline for the Geomechanical Design (ÖGG 2001) has been presented. The application of computational models and probabilistic methods to process geological and geotechnical data and determine the rock mass behaviour has been shown. One of the major benefits of this approach is the ability to quantify the uncertainty in the geological-geotechnical prediction and the identification of failure mechanisms which result from complex geological situations, e.g. in a heterogeneous rock mass.

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