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PROBABILISTIC ASSESSMENT OF ROCK MASS BEHAVIOUR AS BASIS FOR STABILITY ANALYSES OF TUNNELS

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SUMMARY

The uncertainties inherent in each geological and ground model naturally lead to uncertainties in stability analyses of geotechnical structures. Only when a consistent procedure is used for the quantification of the geotechnical variability, the probability of instability of a tunnel can be assessed.

A consistent procedure for the determination of the rock mass behaviour as a result of the tunnel excavation has been developed and will be outlined in the presentation. A distinct geological model, the selection of relevant parameters and a systematic and quantitative rock mass characterization form the basis for all further probabilistic evaluations. After the definition of key parameters and factors of influence it is important to determine the spread of parameters and to quantify the possible variations of the geological model. In a first step rock type specific key parameters are selected. The evaluation of the rock mass properties then leads to the identification of rock mass types. The next step includes the assignment of rock mass types to the different sections along the tunnel. Then the rock mass behaviour in each section as a result of the excavation is evaluated considering the rock mass properties and influencing factors, like stresses, discontinuity orientation, ground water, etc.

The probabilistic processing of the input data results in a quantification of the probability of occurrence of the single rock mass behaviour types. By assigning a support concept to each behaviour type the system behaviour then is evaluated and compared to the requirements of stability and serviceability. As also the behaviour within a behaviour type has a certain spread, for example magnitude of displacements, size of instable blocks etc., this spread has to be considered in the stability analysis. The models used for the analysis in terms of sophistication depend on the phase of the project, and on the environmental constraints. Relatively simple closed form solutions may satisfy the requirements in early project phases or in less sensitive areas, while numerical models will have to be used in highly sensitive situations. Examples of analyses will be shown.

1. INTRODUCTION

Currently, there are no standardized procedures to determine excavation and support for underground openings. This lack of consistency makes it difficult to technically review or audit designs, collect, evaluate, and compare data from different sites and designs.

A sound and economical tunnel design depends on a realistic geological model [1], a quality rock mass characterization, and the assessment of influencing factors such as primary stresses, groundwater, and kinematics. Despite this requirement it is still current practice to base the tunnel design primarily on experience, basic empirical calculations, and standardized rock mass classification systems. Additionally, the on site decisions on excavation and support modifications are frequently based more on intuition than on analyses. This is especially true for tunnels with high overburden in complex geological conditions where limited information is available in the pre-construction phase.

On the other hand, the quantitative rock mass classification systems presently in use [2, 3, 4, 5] have severe shortcomings. One of the main deficiencies is that the classification parameters are universally applied to all Rock Mass Types. Especially in heterogeneous and poor ground conditions these classification methods may provide misleading results, while other shortcomings include the lack of consideration for different rock mass failure modes and ground-support interaction [6]. These schematic procedures have the potential to make tunnel design appear rather simple. Frequently, a few specific parameters are determined and simple classification formulas are applied to achieve a rating. Then with a design chart a support method is determined. No reference is made to project specific requirements or to boundary conditions.

For this reason, it was decided to develop a consistent method for tunnel design, from the pre-construction phase through the tunnel construction, applicable to all rock mass conditions. In general, the final design process continues into the construction phase. The procedure developed, allowing an objective and unbiased decision making process was published in the form of a guideline [7]. The process outlined in this paper clearly distinguishes between rock and rock mass description, behaviour of the rock mass as a result of the excavation, and the system behaviour resulting from excavation and support. Examples show the application of the procedure for the design of tunnels.

2. GENERAL PROCEDURE

The geotechnical design, as part of the excavation design, serves as a basis for approval procedures, the determination of excavation classes and their distribution, and the determination of the excavation and support methods used on site [8]. The flow chart in Figure 1 shows a basic procedure for the design of underground structures, consisting of four general steps to develop the geotechnical design, beginning with the determination of the Rock Mass Types and ending with the definition of excavation classes.

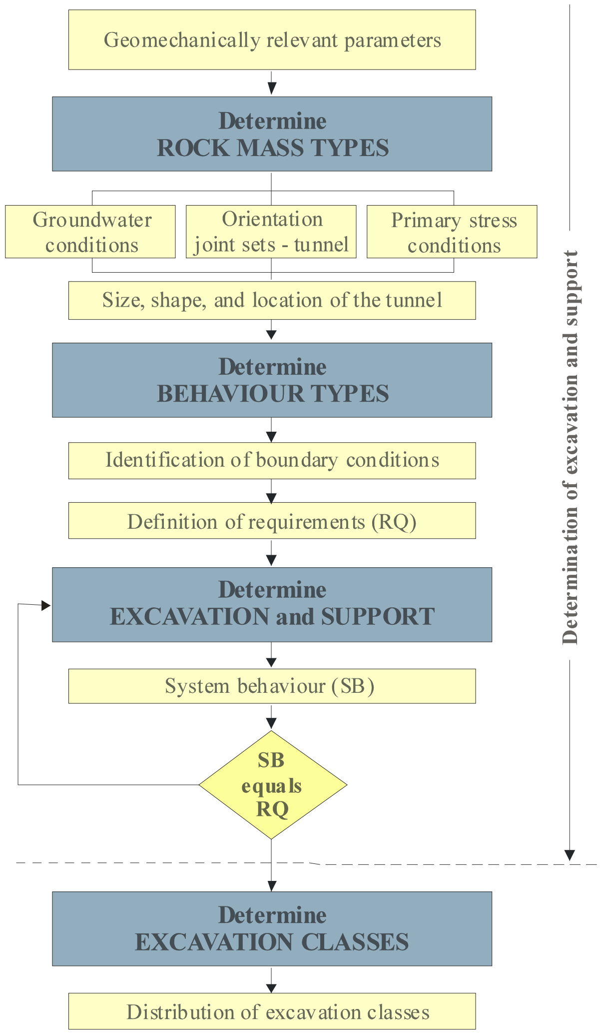


Figure 1 Flow chart of the basic procedure of excavation and support design for underground structures

2.1 DETERMINATION OF ROCK MASS TYPES (RMT)

The first step starts with a description of the geologic architecture and proceeds by defining geomechanically relevant key parameters for each ground type. The key parameter values and distributions are determined from available information and/or estimated with engineering and geological judgment. Values are constantly updated as pertinent information is obtained. Rock Mass Types are then defined according to their key parameters. The number of Rock Mass Types elaborated depends on the project specific geological conditions and on the stage of the design process.

2.2 DETERMINATION OF ROCK MASS BEHAVIOUR TYPES (BT)

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, stress situation, etc. The rock mass behaviour is defined in the sense of this procedure as the rock mass response to the full cross sectional area without considering any modifications including the excavation method or sequence, support, or other auxiliary measures.

The expected rock mass behaviour resulting from the analyses is then categorized into Behaviour Types which have to be assigned to one of the eleven general categories listed in Table 1. In case more than one Behaviour Type is identified in one of the general categories, sub types have to be assigned.

Behaviour Type (BT)		**Description of potential failure modes/mechanisms during excavation of the**
1	Stable	Stable rock mass with the potential of small local gravity induced falling or sliding of blocks
2	Stable with the potential of discontinuity controlled block fall	Deep reaching, discontinuity controlled, gravity induced falling and sliding of blocks, occasional local shear failure
3	Shallow shear failure	Shallow stress induced shear failures in combination with discontinuity and gravity controlled failure of the rock mass.
4	Deep seated shear failure	Deep seated stress induced shear failures and large deformation
5	Rock burst	Sudden and violent failure of the rock mass, caused by highly stressed brittle rocks and the rapid release of accumulated strain energy
6	Buckling failure	Buckling of rocks with a narrowly spaced discontinuity set, frequently associated with shear failure
7	Shear failure under low confining pressure	Potential for excessive overbreak and progressive shear failure with the development chimney type failure, caused mainly by a deficiency of side pressure
8	Ravelling ground	Flow of cohesionless dry or moist, intensely fractured rocks or soil
9	Flowing ground	Flow of intensely fractured rocks or soil with high water content
10	Swelling	Time dependent volume increase of the rock mass caused by physical-chemical reaction of rock and water in combination with stress relief, leading to inward movement of the tunnel perimeter
11	Frequently changing behaviour	Rapid variations of stresses and deformations, caused by heterogeneous rock mass conditions or block-in-matrix rock situation of a tectonic melange (brittle fault zone)

Table 1 General categories of Rock Mass Behaviour Types; based on [7]

The Rock Mass Behaviour Types form the basis for determining the excavation and support methods as well as assist in evaluating monitoring data during the excavation.

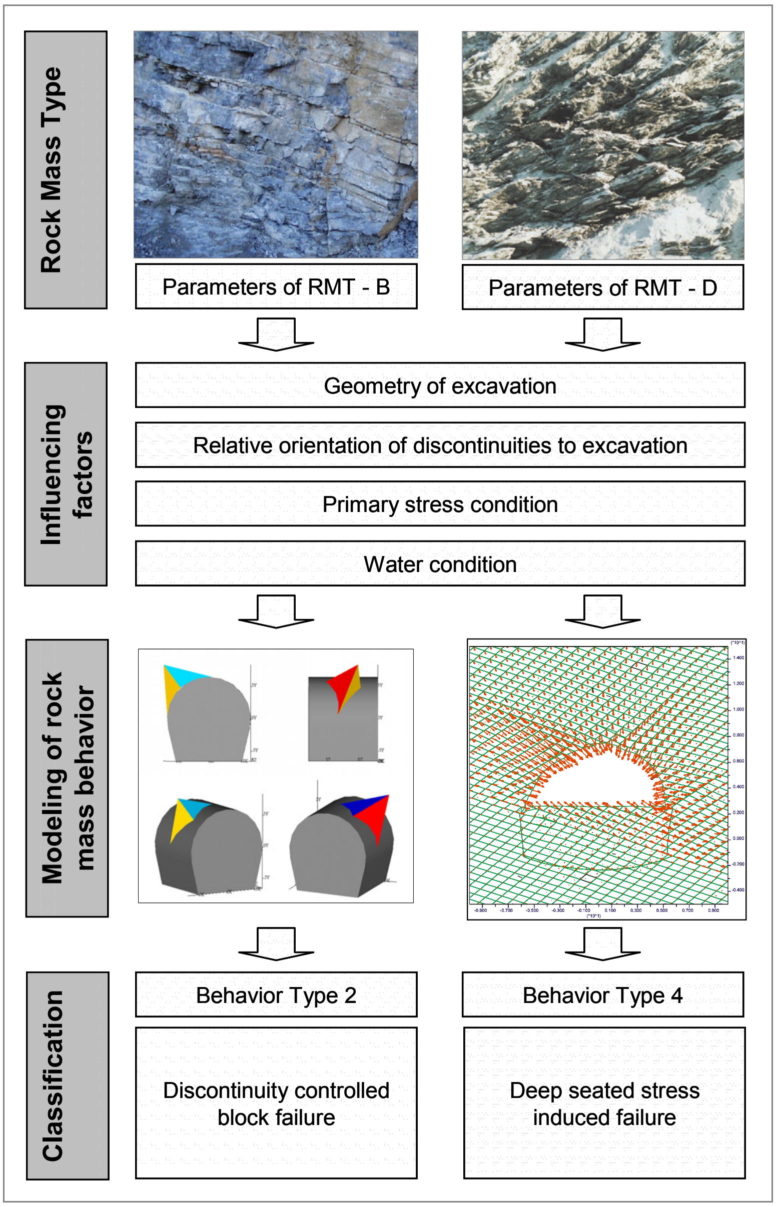


Figure 2 Example for determination of Behaviour Types

In Figure 2 the exemplary presentation of the procedure for the classification of two different rock types is continued. Based on the parameters, data of the specific Rock Mass Types, and influencing factors the rock mass behaviour of the actual rock mass condition is determined. In this example different modelling methods are illustrated for the different Rock Mass Types. For the carbonatic rock mass by assuming the in situ stresses below the rock mass strength a key block analysis [9] is a proper way to determine one of the possible failure modes. A weaker phyllite could fail due to stresses exceeding the strength. In this case a closed form solution or numerical model would be appropriate [10]. After systematically investigating all possible failure modes the determined rock mass behaviours are classified into Behaviour Types.

2.3 DETERMINATION OF EXCAVATION AND SUPPORT

Based on the defined project specific Behaviour Types, in the third step, different excavation and support measures are evaluated and acceptable methods are determined. The System Behaviour (SB) is a result of the interaction between the rock mass behaviour and the selected excavation and support schemes. The evaluated System Behaviour has to be compared to the defined requirements. If the System Behaviour does not comply with the requirements, the excavation and/or support scheme has to be modified until compliance is obtained.

The methods for the analysis of the System Behaviour depend on the boundary conditions of the underground structure (such as limited surface settlements or blast vibrations), the variability of the influencing factors and the environmental impacts. Similar to the Behaviour Types basically the following methods for analysis of the System Behaviour are applicable:

- Comparative studies, based on experience from previous comparable projects
- Analytical methods
- Numerical methods

The analyzed System Behaviour is then compared to the previously defined design requirements and proves the stability of all construction stages, the compliance with environmental requirements (surface settlements, vibrations, ground water disturbance, etc.), and that displacements are within defined limits (critical strain, serviceability, compatibility). It is emphasized, that different boundary conditions or different requirements may lead to different support and excavation methods for the same Behaviour Type even within one project.

2.4 DETERMINATION OF EXCAVATION CLASSES

In the final step of the design process the geotechnical design must be transformed into a cost and time estimate for the tender process. Excavation classes are defined based on the evaluation of the excavation and support measures, which is regulated for example in Austria with the standard ÖNORM B2203-1 [] by classifying round length and support measures. These excavation classes form a basis for compensation clauses in the tender documents.

The distribution of the expected Behaviour Types and the excavation classes along the alignment of the underground structure provides the basis for establishing the bill of quantities and the bid price during tender.

3. PROBABILISTIC PROCESSING

The goal was to develop a analytical model which processes the input data from the rock mass characterization to rock mass and project specific Rock Mass Types, Behaviour Types, and by assigning the excavation and support measures, to excavation and support classes [11]. Then time and costs can be assigned to these classes which results in an analytical correlation between the geomechanical input parameters and the costs of the underground excavation [12].

Throughout the project development the quantity and quality of the available data change, as well as the influence of different parameters. Additionally, various kinds of data such as observed, calculated, or estimated data have to be processed. To quantify the result parameters statistically, such as the spread of the percentage of Behaviour Types or distributions of time and costs, the singular deterministic values are replaced by distributions and the entire analytical system is calculated probabilistically by using Monte Carlo simulations. The continuous collection and probabilistic processing of geological and geotechnical data allows a reasonable determination of distributions of Behaviour Types, excavation and support measures, or time and costs in the various design stages. The advantage of a quantitative and continuously modelled design procedure is the possibility to trace the influence of the input data to the result. By applying this analytical design procedure the geotechnical risk - as the range of possible values of tunnel costs and their likelihood of occurrence - can be evaluated based on the variation and the probability of the geological and geotechnical data.

3.1 DEVELOPMENT OF A GEOLOGICAL MODEL

A sound geological model together with carefully investigated and reasonably selected rock mass parameters is the basis for all further investigations. Based on the evaluation of the results from geological site investigations, including core drillings, laboratory and in-situ tests, detailed geological mapping, outcrop studies, geophysical surveys, and so on, a three-dimensional rock mass model is developed and geotechnically homogenous volumes are defined. Geological singularities of a rock mass model such as faults, lithological boundaries, and aquifers have a significant influence on the rock mass behaviour. The necessary data to describe these singularities can be either measured in boreholes, outcrops, aerial and satellite images or estimated. In any case, these results have to be reconsidered after each new investigation campaign.

3.2 GEOTECHNICAL LONGITUDINAL SECTION

Based on the results of the site investigations, a spatial rock mass model can be developed and the expected geological architecture (lithological arrangement, fault zones, and ground water situation) can be characterized. Along the tunnel alignment the spatial model is cut and the borders of the lithological units and the fault zones as well as zones of similar influencing factors, such as tunnel shape or primary stress conditions, divide the tunnel alignment into homogenous sections, the so-called calculation segments. These calculation segments can differ strongly in length due to the variability of the rock mass conditions and project parameters like depth of the structure or project stage. If the calculation segments are very short the results are more detailed but the calculation effort considerably increases with the number of calculation segments.

The values of the key parameters are assigned to these calculation segments and provide the basis for the further classification into Rock Mass Types. Additionally, the influencing factors, which are the default tunnel geometry, the estimated primary stress conditions, groundwater conditions, and the measured relative orientation of the main discontinuity sets to the tunnel axis are assigned to the calculation segments. The result is a longitudinal section divided into homogenous sections – the calculation segments - with all relevant geomechanical data plotted along the tunnel alignment.

Figure 3 shows an example of a longitudinal section. It includes the information from site investigation and shows a simple method to model faults. The distribution of the thickness, spacing, and orientation of the fault zones for a geotechnically homogenous project area is determined. This can be done with different methods for example from the results of core logging and the evaluation of optical scanner measurements. Additionally, the distance of major fault zones with a geomechanically relevant thickness can be derived from three-dimensional models using scan line mapping techniques. These parameters are used as input to generate a probabilistical longitudinal section of the tunnel. The longitudinal section is individually computed in each simulation based on the probabilistic input parameters.

3.3 EVALUATION OF ROCK MASS TYPES

In addition to the lithological discrimination between rock types, significant differences of parameters within a certain rock type are used to define Rock Mass Types. The determination is the result from different combinations of key parameter values and has to be pre-defined for each kind of rock and rock mass. The pre-defined matrix is project and rock mass specific and can be derived from the results of investigations from an earlier design stage or can be developed from experiences form projects under similar conditions.

Figure 3 shows an example for the parameter flow of the probabilistic analytical determination of the Rock Mass Types. The input parameters are distributed values which leads to varying parameter combinations in each iteration of the Monte Carlo simulation. For each iteration the actual Rock Mass Types are assigned to the calculation segments. By summing up the results of the iterations the percentages of the Rock Mass Types within the defined Geotechnical Units or along the entire tunnel are obtained probabilistically.

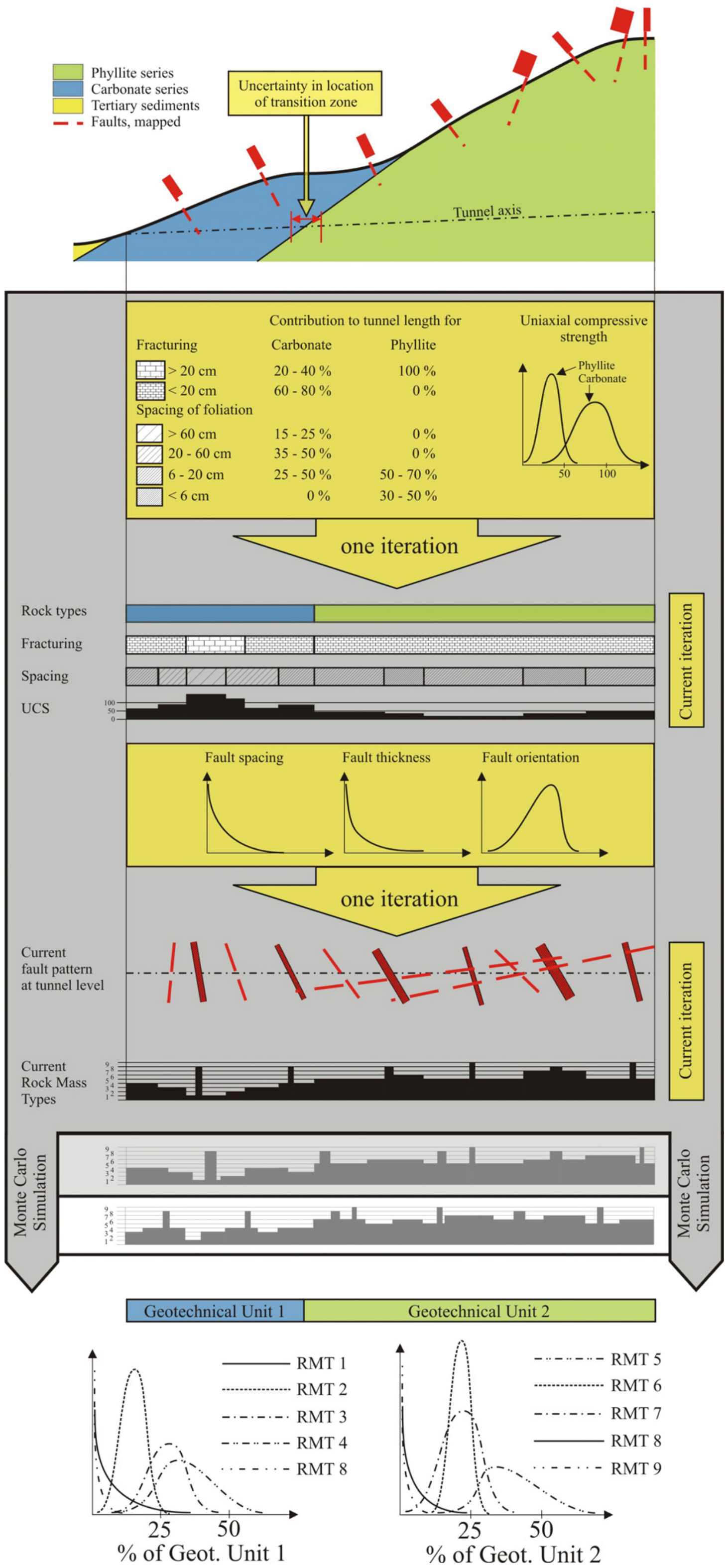


Figure 3 Example for the analytical determination of Rock Mass Types

3.4 EXCAVATION AND SUPPORT DETERMINATION

Based on the geotechnical longitudinal section an analytical model is developed to determine the rock mass behaviour and to classify the behaviours into Behaviour Types. The procedure is described for one singular calculation segment. First different independent analytical calculations are performed for the determination of the different possible rock mass behaviours. Then the Behaviour Types are determined from the calculated rock mass behaviour, and finally the obtained Behaviour Types are ordered. The result is a listing of possible Behaviour Types of the rock mass within a calculation segment.

The excavation and support measures are determined for the predicted Behaviour Type. This can be done with any method applicable for the particular rock mass behaviour. The input parameters for the excavation and support design are the key parameters of the Rock Mass Type and the influencing factors, which all together represent the parameters of the rock mass characterization. Finally, the resulting excavation and support measures are assigned to the Behaviour Type of the calculation segment of the analytical system.

After the rock mass classification and excavation and support determination within each calculation segment of the tunnel alignment the percentage of the Behaviour Types for the entire tunnel can be obtained. Additionally the percentage of excavation and support measures and, if assigned, the time and costs for the tunnel are the final result.

Different analytical models are used to calculate the possible behaviours of the rock mass. Different analytical system can be used to to determine the behaviours of the unsupported rock mass like the volume and depth of overbreak, the depth of failure zone and displacements, or the stress induced failure caused by heterogeneous rock mass conditions.

In each calculation segment calculations with different analytical models are performed. The results are checked against the criteria defined for the single Behaviour Types. It is possible that different behaviours are identified in one section. On the other hand, it is possible that only one type or no type of failure is identified. Stable conditions for the unsupported excavation can be assumed in the case that no failure is identified or any of the criteria defining the Behaviour Types 2 to 11 fulfilled. The results of the calculations are numerical values for the different investigated mechanisms within the calculation segment.

Delimiting criteria are defined to classify the calculated rock mass behaviour into one of the eleven basic Behaviour Types as defined in Table 1. Additional criteria can be set to classify the project specific defined sub Behaviour Types. The values of the delimiting criteria are based on the rock mass and its specific failure and correlate to the Behaviour Types.

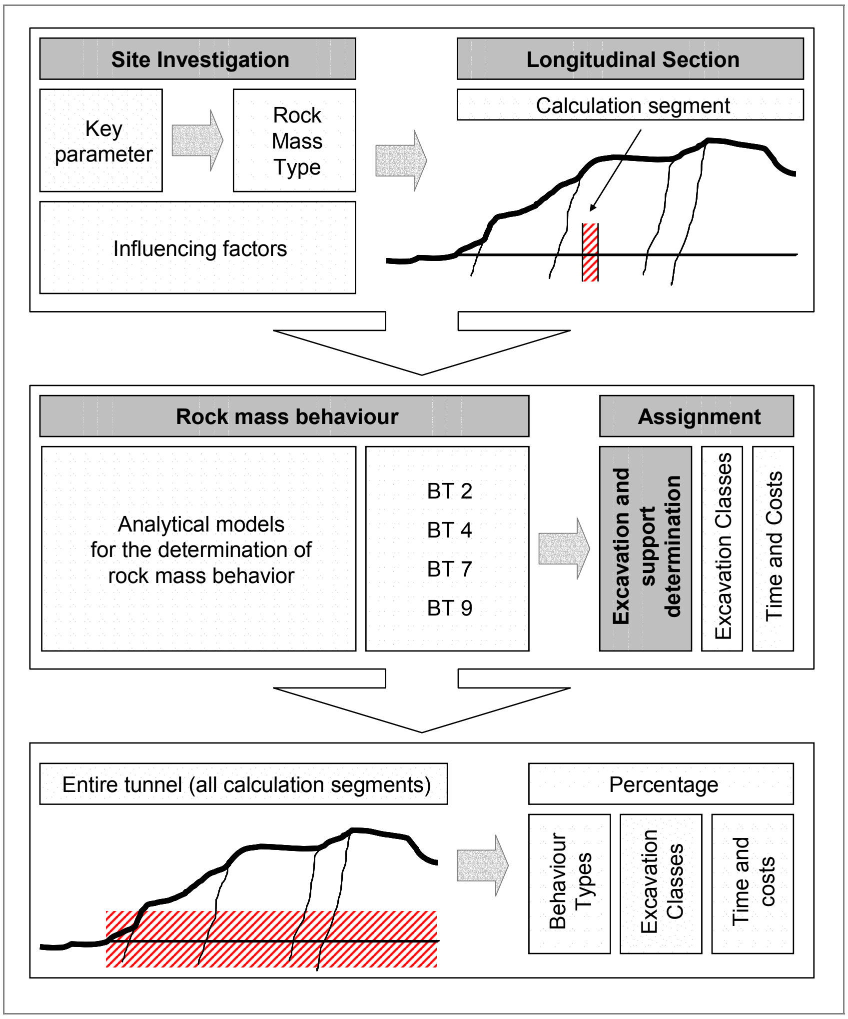


Figure 4 Process of the deterministic analytical method for excavation and support determination

The result of the quantitative rock mass characterization and classification is a sequence of Behaviour Types for each calculation segment. With this information the rock mass along the entire tunnel alignment is described in terms of its predicted behaviour around the unsupported underground excavation.

Due to the probabilistic input data of the geotechnical properties the rock mass model along the tunnel alignment changes geometrically and/or mechanically with each iteration of the Monte Carlo simulation. This can result in the change of the Behaviour Type in a calculation segment in the different iterations. Due to the large number of calculation cycles it is possible to obtain the probability of occurrence of each Rock Mass Type in a calculation segment. By considering the length of the calculation segments, the distribution of the percentage of one Rock Mass Type along the entire tunnel alignment can be calculated [13].

Figure 5 shows a detailed flow chart of the described process. The chart on the right hand side shows the probability distribution of the calculated Behaviour Types due to the variation of the input parameters within a homogeneous section of a tunnel.

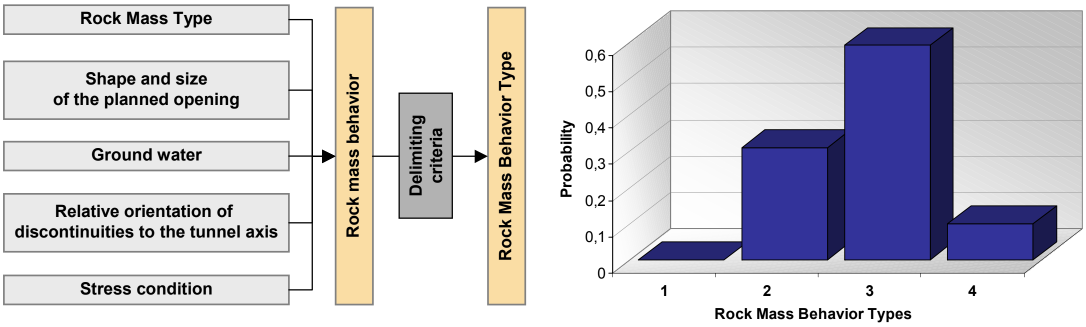


Figure 5 Probability of occurrence of determined Behaviour Types for one calculation segment

3.5 DETERMINATION AND ASSIGNMENT OF EXCAVATION AND SUPPORT

The determination of the excavation method and the support measures is based on all information gained from the entire process of rock mass characterization and classification. This means that qualitative descriptions of the geological and geomechanical conditions as well as quantitative rock mass parameters, including the evaluated possible failure modes are used as input parameters for the excavation and support determination. The selection of an appropriate method for the determination highly depends on project specific parameters like project stage or task of actual investigations.

The design of the excavation and support has then to be based on the range of possible behaviours within the Behaviour Type. If, for example, the deterministic calculation of the Behaviour Type results in a rock mass deformation of 17 cm, the probabilistic determination could lead to a range from 8 to 25 cm. The spread of the rock mass deformation requires a support system applicable for the entire parameter range or the design of two different support systems. By using numerical methods the design for spread rock mass conditions can be done by different approaches. One example is the use of integrated Monte Carlo simulations. Due to the generally high calculation effort this method can only be applied for very simple models. Another method is the determination of input parameters which are sensitive to the result and the variation of only these few parameters [14].

The finally identified excavation and support measures are then assigned to the actual Behaviour Type of the calculation segments within each iteration of the Monte Carlo simulation. This finally results in a distribution of the support measures and the excavation design in every calculation segment. Based on local standards or contractual needs the excavation and support measures can be classified into excavation classes. Additionally, time and costs can be determined for the defined excavation and support concepts and assigned to the correlating Behaviour or Excavation Type. Finally it is possible to present percentages of Rock Mass Types, Behaviour Types, Excavation (and Support) Classes, and time and costs for the entire tunnel alignment or individually defined tunnel sections. These results are directly correlated to the input parameters of the rock mass and the influencing factors based on the analytical determination of different failure modes and rock mass behaviours.

4. APPLICATIONS SEMMERING BASE TUNNEL

The described procedure of quantitative rock mass characterization has been applied in different design stages like route selection [15] or tender design [16]. The comparison of different tunnel systems and construction methods in the tender design stage for the Semmering base tunnel project is shown below.

4.1 INTRODUCTION

The traffic route across the Semmering mountain range has a long tradition. The Semmering railway was build in the mid nineteenth century and was the first European railroad crossing a major mountain range. It is one of the main trading routes between Vienna and the Adriatic Sea. Despite some improvements the Semmering railway does not meet the requirement of a modern railway and should be substituted by a base tunnel.

The Semmering base tunnel, being part of the Austrian high speed railway project, has been designed with a total length of approx. 22 km [17]. In a late design stage two different tunnel alternatives were investigated in terms of technical feasibility and risk:

- A double track tube with an escape tunnel
- Two single track tubes

For both alternatives different excavation methods (NATM and TBM) were investigated [18].

4.2 GEOLOGICAL – GEOTECHNICAL INVESTIGATIONS

The geological site investigations consisted of detailed geological mapping (scale 1:5000) of the whole alignment corridor and subsurface investigations by geophysical survey, trenches, core drilling, borehole tests and the excavation of a pilot tunnel on a part of the route.

It is pointed out that the maximum depth of drill holes was approx. 200 m, even in the alignment sections with high overburden. The drill hole locations were arranged depending on the geological structure and allowed, in accordance with detailed geological mapping, the extrapolation of drilling results down to the level of the tunnel and the establishment of a realistic three dimensional geological model. To investigate the complicated geological conditions of the southern part of the alignment a 4300 m long pilot tunnel was excavated.

Based on the evaluation of the results of the investigations, a three dimensional model was developed and geotechnical homogeneous areas were defined. This model includes the expected lithology, geological architecture, intact rock, discontinuity, and hydraulic properties, fault zones and groundwater situation. Figure 6 shows two examples of three dimensional models of parts of the project area with different information included in the models. The upper left model shows the geological conditions at the surface. The model in the middle has the lithological units and boundaries included. The model on the right hand side visualizes the faults in this area.

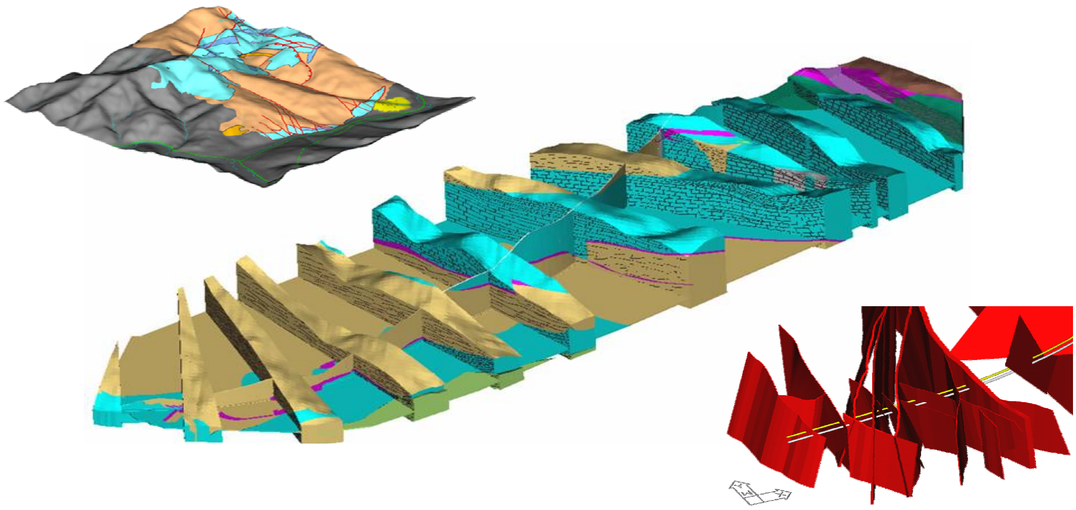


Figure 6 Examples of the 3D-model of the project area

4.3 ROCK MASS TYPES

By summarizing the results of the field investigations and the laboratory tests, Rock Mass Types – rock masses with similar properties - were determined by selecting and quantifying relevant geotechnical key parameters for tunnelling. These key parameters were: lithology; mechanical intact rock properties (UCS, C, ν, E, CAI); discontinuity parameters such as persistence, surface properties and aperture; foliation/anisotropy; block size; and rock properties.

The strength characteristics of Rock Mass Types were estimated on the basis of the Geological Strength Index [19]. The GSI values have been estimated from the rock mass conditions and have additionally been back calculated from the monitoring results of the pilot tunnel excavation.

In addition to the lithology, the significant differences within the key parameters were used to define 23 project specific Rock Mass Types. The specified Rock Mass Types cover all geomechanical relevant engineering geological characteristics investigated at the stage of the design phase. The key parameters were described by using singular deterministic values and frequency distributions. Additionally the results of the laboratory tests were presented statistically. When a sufficiently large number of data could not be obtained, the parameter distributions were estimated. Table 2 shows an example of two Rock Mass Types.

	Rock Mass Type 1	**Rock Mass Type 11**
lithology	marble	phyllite
foliation / anisotropy	massive	flaky to platy, highly anisotropic
block size	> 20 cm	< 20 cm
joint properties	mainly rough	undulating, smooth
persistence	low	dominating low
aperture	closed	dominating closed
intact rock		
parameter	average / standard deviation / number of samples	average / standard deviation / number of samples
UCS [MPa]	102,6 29,0 / 26	28,2 / 13,6 / 19
c [MPa]	24,2 / 8,2 / 20	10,8 / 3,1 / 3
φ [°]	40,7 / 4,9 / 20	31,7 / 1,5 / 3
E [GPa]	68,3 / 17,6 / 23	26,7 / 19,1 / 18
CERCHAR Abrasivity Index	1,4 / 0,4 / 18	no value
ν [ ]	0,19 / 0,4 / 18	0,43 / 0,18 / 2
Hoek constant m_i [ ]	13,4 / 6,2 / 20	14,5 / 6,0 / 3
rock mass		
parameter	average / standard deviation	average / standard deviation
Geological Strength Index	70 / 10	40 / 5
UCS [MPa]	33,2 / 12,1	3,9 / 2,0
c [MPa]	8,0 / 2,8	1,1 / 0,5
φ [°]	37,7 / 4,7	31,3 / 3,6
E [GPa]	35,0 / 19,4	3,0 / 1,0
joint properties		
parameter		average / standard deviation / number of samples
friction angle [°]	35 - 45	33,7 / 6,3 / 15
residual friction angle [°]	30 - 40	28,5 / 5,6 / 23

.... estimated values

Table 2 Geological parameters, laboratory test results, and calculated rock mass parameters for the Rock Mass Type 1 and 11

4.4 ROCK MASS BEHAVIOUR TYPES

Based on the parameters of the Rock Mass Types and the influencing factors the Behaviour Types were defined for both, the NATM and the TBM construction method. According to the appearance of the main influencing factors in nature these input parameters were computed as probabilistic parameters.

The described probabilistical procedure was used to determine the rock mass behaviours of the tunnels with different excavation geometries, sizes and excavation methods. To improve the accuracy of the results for specific rock masses, the analytical models were adjusted to the results of numerical calculations and physical models. The experience gained from the already excavated pilot tunnel and from previous projects under similar conditions was also used to evaluate the results of the models. If the analytical models are calibrated a parameter can be changed and the probabilistical analytical model immediately calculates the influence to the behaviour for all calculation segments. This means that the change of the percentage of Behaviour Types along the entire tunnel alignment for example due to the change of the tunnel diameter can be seen and evaluated directly.

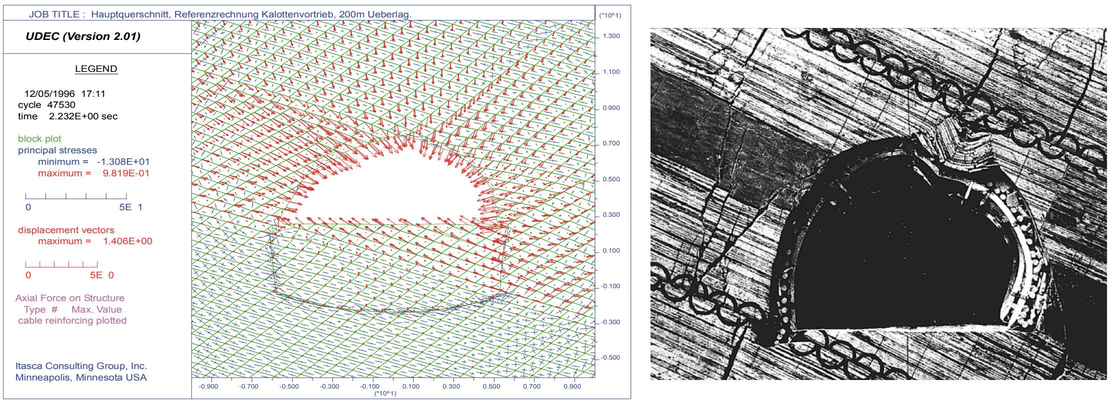


Figure 7 Failure mechanisms for phyllite [10, 20]

With the use of delimiting criteria the Behaviour Types were obtained for the different shapes and sizes of opening and for the different construction methods (NATM, TBM). For the entire tunnel 16 different Behaviour Types have been identified. In Table 3 a description of the sub-Behaviour Types 3/2 and 4/1 is given. All Behaviour Types are described by their input parameters – the key parameter and the influencing factors – and a qualitative description of the failure. A sketch is thought to illustrate the specific failure mechanisms. In addition to the qualitative description the Behaviour Types are defined by quantitative values of the delimiting criteria.

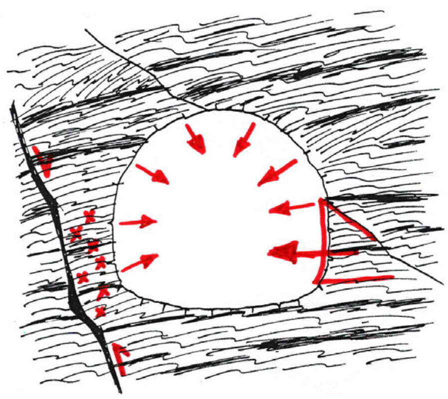
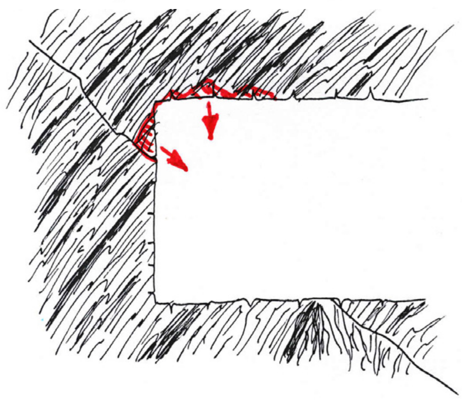
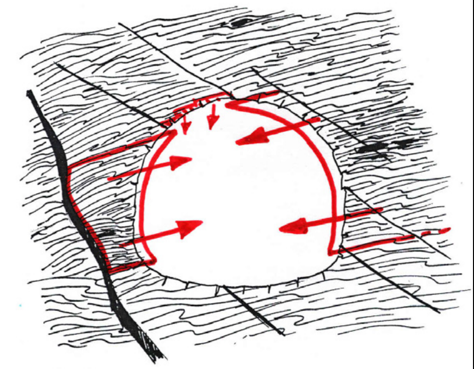
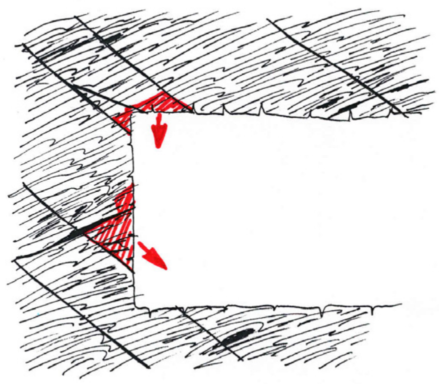
	Behaviour Type 3/2	**Behaviour Type 4/1**
Rock Mass Type	RMT 11 and 12 (phyllite)	RMT 11 and 12 (phyllite)
Main discontinuity orientation	The foliation strikes perpendicular to the tunnel, dip angle 45° - 75°	The foliation strikes perpendicular to the tunnel, dip angle 45° - 75°
Stress conditions	The stress level is close to the rock mass strength	Stresses are higher than the rock mass strength
Groundwater conditions	Little to no groundwater inflow	Little to no groundwater inflow
Rock mass behaviour (excavatability, failure mechanisms)	Highly anisotropic rock mass with relatively uniform deformations. Small block failures controlled by foliation. Shear along foliation planes. Favourable face stability for foliation dipping into the face, potential block slides for foliation dipping into the excavation	Block failures controlled by the foliation. Low shear strength along discontinuities results in structurally controlled anisotropic behaviour. Slickensides close to the tunnel excavations increases the secondary stresses and cause local deep seated failures. Structurally controlled face failure.
Recommended excavation method	Drill and blast, roadheader	Drill and blast, roadheader
Deformation characteristics	Initial deformation rates are relatively high. Radial deformations are expected to stabilize in the range of 10 cm	Deformations are highly controlled by the rock mass structure and vary in a rang of 10 to 25 cm
Symbolic diagram for phyllite (excavation towards NE)	 	 

Table 3 Description of the Behaviour Types 3/2 and 4/1

4.5 ASSIGNMENT OF EXCAVATION CLASSES, SUPPORT, TIME AND COSTS

Based on the Behaviour Types and the project specific requirements different excavation methods and support measures were determined and assigned. The design of the support measures for NATM was primarily based on the distinct failure modes related to the Behaviour Types. For example, various support classes were designed for rock mass with large deformations. Based on the predicted deformation from the probabilistic analytical model the applicable support classes were assigned to the calculation segments with the corresponding Behaviour Types. The decisions concerning TBM system and support were also primarily based on the geomechanical conditions of the rock mass. The parameters depth of the failure zone and rock mass strength for example were used to assign the advance system which directly influences the advance rate of the TBM.

The System Behaviour was determined and if the requirements were fulfilled the excavation and support was assigned to the corresponding Behaviour Type. This resulted in excavation classes and support measures for each calculation segment. According to the excavation classes and support measures, time and costs were determined and assigned to the particular segments.

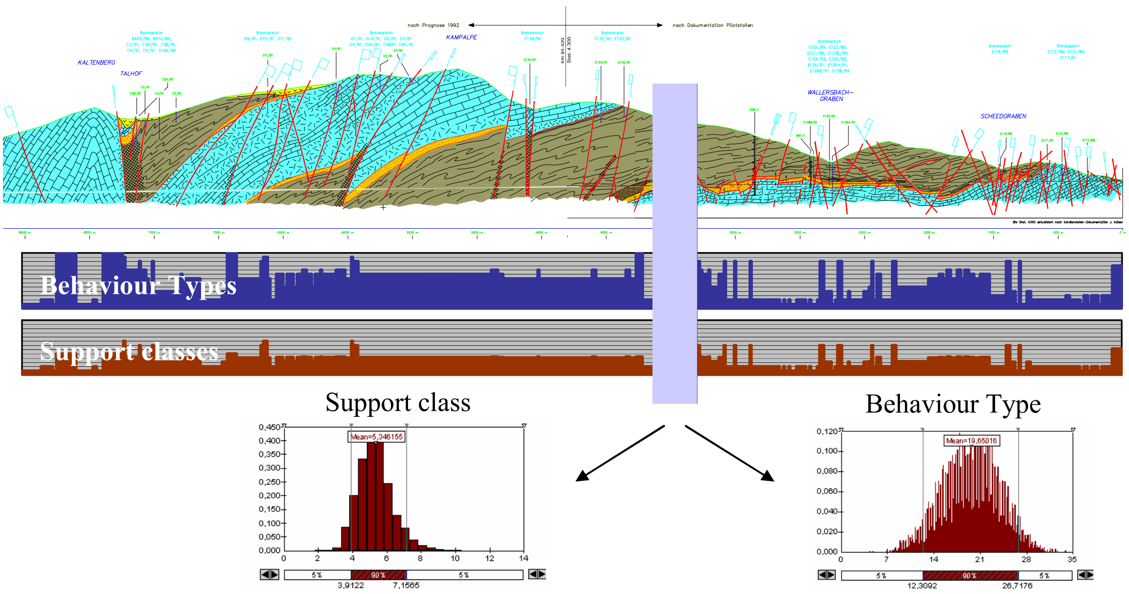


Figure 8 Determination of the distributions of Behaviour Types and Support classes

The determination of time and costs especially depends on the excavation method. For the conventional method the advance rate is dominated by the time needed for excavation and support installation. For TBM excavation the advance rate is influenced by the penetration rate depending on the Rock Mass Types, different utilization factors and factors for difficulties, such as mixed face – conditions.

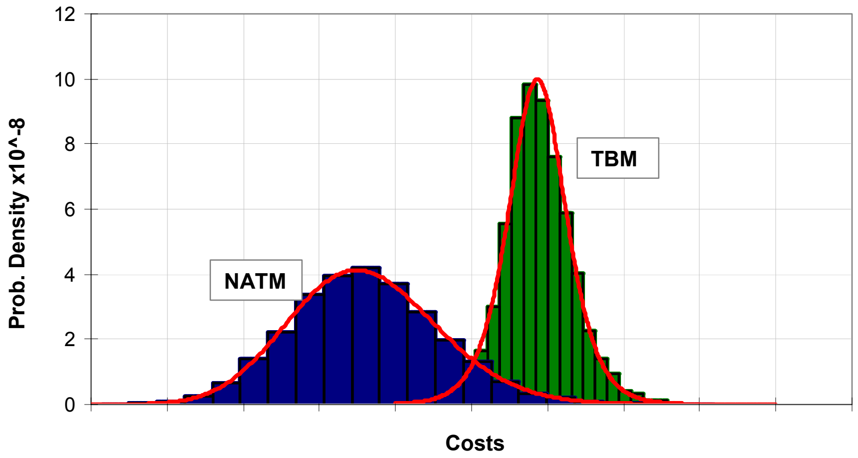


Figure 9 Typical distributions of costs for different excavation methods

Figure 9 shows the consequence of the specific risk-relevant behaviour. It shows the comparison in the geotechnical risk (determined by cost) for two excavation methods (D&B and TBM) for the entire tunnel in very heterogeneous ground with high overburden The conventional method shows a wide deviation and lower basic costs, whereas the mechanical method shows a narrow deviation at higher basic costs. The higher flexibility of the conventional method allows a variation of the measures to be applied depending on the rock mass behaviour encountered on site. With the mechanical excavation method, higher basic investments into the technology are required to cope with the expected rock mass behaviour over the major part of the tunnel with small variations in excavation and support. The costs of the TBM excavation are further increased in this project, as in some sections the high risk identified would require a conventional excavation.

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