DISCRETE ELEMENT MODELLING OF THE MÓRÁGY GRANITE FORMATION IN SOUTHERN HUNGARY

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INTRODUCTION

The first underground radioactive waste repository in Hungary is being built in the southern part of the country, situated in the outskirts of the village of Bátaapáti. Construction of the repository complex was started in 2005 and excavation of the first two emplacement chambers was completed at the end of 2011. Underground facilities of the National Radioactive Waste Repository (NRWR) are embedded in the Mórágy Granite Formation (Balla, 2004).

The detailed design of the rock support of tunnels was based on a combination of empirical and analytical design methods and state-of-the-art numerical modelling, and was performed by Mott MacDonald Magyarország Kft. To assess the overall stability of the excavation at critical cross-sections and junctions, sophisticated numerical models were built employing the continuum modelling approach. Based on the discontinuous nature of the granite formation, however, it could be considered an assembly of blocks rather than a continuous body, therefore it is reasonable to assume that the discontinuum modelling approach would provide a more representative picture of its overall behaviour. This paper discusses the applicability of discrete element modelling approach to predict the performance of the fractured granite bedrock.

CONTINUUM AND DISCONTINUUM MODELLING

In contrast to empirical and analytical design methods, numerical analysis offers the ability to explicitly model complex conditions, including adjacent structures, varying geological conditions, complex constitutive behaviour, dynamic loading and construction sequences. This unparalleled capability is essential for studying the anticipated and actual performance of structures in rock, therefore supporting rock engineering design. The most important decision to be made prior to building a numerical model is the choice between the continuum and discontinuum approach. This choice determines the applicable numerical methods since different techniques have been developed for continuous and discrete systems.

Continuum versus discontinuum modelling

A basic difference between the continuum and discrete approach is the way they handle rigid body motion. For a discrete system, this is often the dominant mode of deformation. This is contrary to the continuum based methods in which the rigid body motion mode of displacement is generally eliminated because it does not produce strains in the elements. In a discrete system, the individual units (blocks) are independent to move according to the equations of motion, therefore their motion can be 'liberated' from other units. In continuum methods, the individual elements are not free to move, but are kept within the same neighbourhood of other units by displacement compatibility conditions. Therefore a continuous system reflects more the 'material deformation' of the system, while a discrete system reflects mainly the 'member (unit, element) movements' of the system (Jing and Stephansson, 2007).

In case of a particular problem the choice of modelling approach depends on many problem-specific factors, but predominantly on the relation of the problem scale and fracture system geometry, as illustrated in Figure 1.

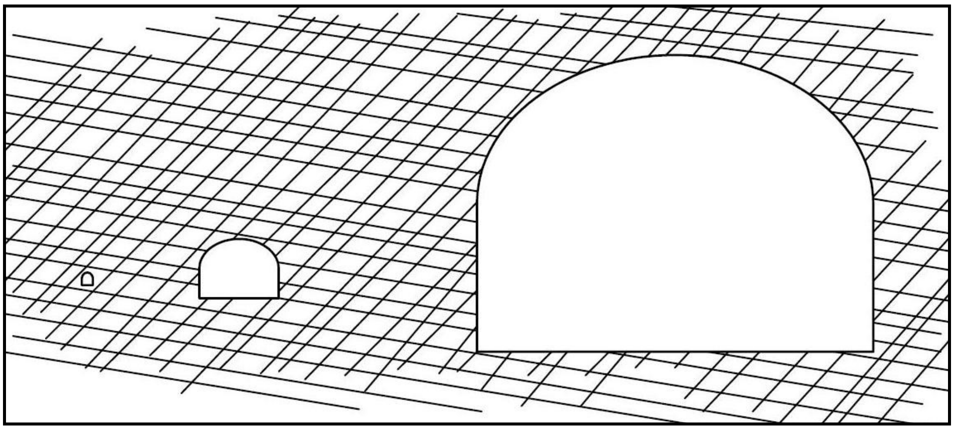

Figure 1: The relation of problem scale and fracture system geometry (Edelbro, 2004)

Discontinuum modelling

The discontinuum approach is most suitable for moderately fractured rock masses where the number of fractures is too large for the continuum-with-fracture-elements approach, or where large-scale displacement of individual blocks is anticipated. The suggested range of Tunnelling Quality Index (Q) values for which discontinuum modelling will most likely be more appropriate than continuum modelling (Q~0.1-10) is provided in Barton *et al.* (2001). Considering the blocky nature of the rock mass, the discontinuum approach devotes most of the attention to the characterization of rock blocks and rock joints. Currently the most adequate numerical technique for realistic modelling of discontinua is the discrete element method (DEM). The numerical modelling software *UDEC* (Itasca, 2006), which was used for the numerical modelling study presented in this paper, has been the most commonly used application of the DEM approach for rock mechanics problems.

CROSS-SECTION KON-8

In the underground facilities of the NRWR, convergence measurement arrays (see Figure 2) were installed in a number of sections to monitor rock mass response to tunnel excavation. This monitoring system has provided *in situ* deformation data. Consequently, modelling one of these sections offered the ability to check the validity of analysis results and to calibrate models against *in situ* measurements, which is essential to ensure that predictions are realistic. For this reason, one of the convergence measurement sections, section Kon-8 was chosen for discrete element modelling. Based on face mapping during tunnel excavation, rock support of Class III was installed in section Kon-8, which consisted of 150mm of steel fibre-reinforced shotcrete (SFRS) and 3m-long rockbolts with spacing of 1.5m in-plane and 1.0m out-of-plane, as shown in Figure 3.

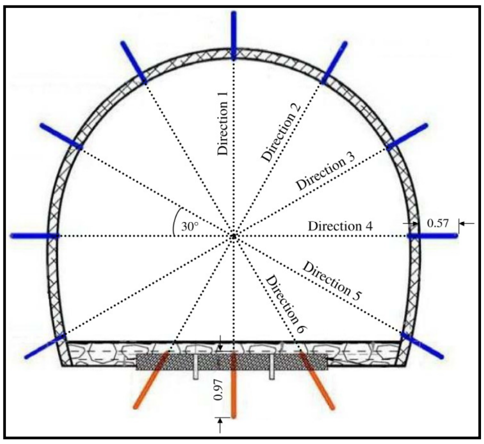

Figure 2: General arrangement of convergence measurement pins

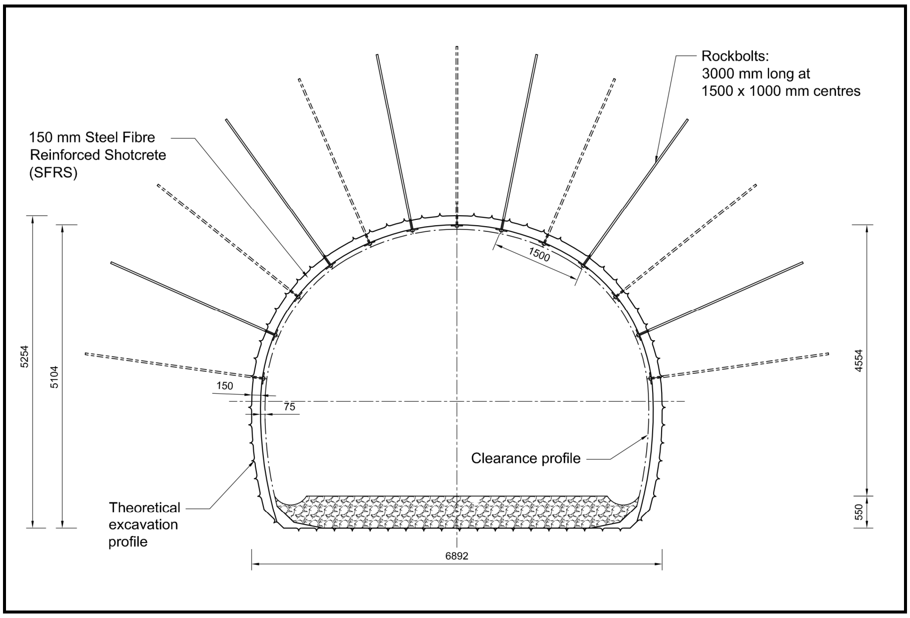

Figure 3: Cross-section type B with rock support of Class III

MODELLING IN UDEC

The numerical modelling study presented in this paper was a sensitivity study which investigated the effect of the variation of two key joint material properties on the performance of the rock mass and the support system. The sensitivity study included the creation of a total of 14 model variations.

Joint geometry

In a multiple-jointed rock mass it is not feasible to incorporate all identified joints in the numerical model. In order to create models of reasonable size and execution time, only the most characteristic joint sets should be incorporated. Based on statistical analysis of discontinuities identified during excavation of the underground facilities of the NRWR, the GIR suggested 7 dominant joint sets to be considered in the design of rock support, as listed in Table 2. For simplicity, joint sets with small variations (<15°) in their orientation relative to each other were assumed to belong to a common joint set. Accordingly, the incorporation of only 3 joint sets in the *UDEC* models was deemed representative. Additional parameters required to define joint geometry included joint spacing, trace length and gap spacing. Based on the face log corresponding to the modelled cross-section, an average joint spacing of 1.40m was defined in the models. As no sufficient information was available on the trace length of joints, in the models joints were treated as continuous.

Table 2: Dominant joint sets

Joint set ID	Dip direction [deg]	Dip angle [deg]	Apparent dip [deg]
JS1	36	80	75.76
JS2	21	27	13.88
JS3	66	65	64.12
JS4	19	89	87.80
JS4	199	89	87.80
JS5	248	89	88.97
JS6	66	89	88.96
JS7	36	80	75.76

Material properties

When available, material properties were taken directly from the Geotechnical Interpretive Report (GIR) for the project (Kandi *et al.*, 2010) which incorporates the evaluation of all qualitative and quantitative information available on the host rock mass. In contrast to continuum modelling, one of the main drawbacks of discontinuum modelling is its demand for a vast amount of additional input parameters related to rock joint properties which predominantly govern rock mass response. Required joint properties not discussed in the GIR were determined based on tests and methods in literature.

In the *UDEC* models the Mohr-Coulomb constitutive model was assigned to rock blocks assuming that the intact rock is an elastic-perfectly plastic material. Equivalent Mohr-Coulomb parameters for the rock mass (i.e. effective cohesion and effective angle of friction) were estimated based on the Generalized Hoek-Brown criterion, as proposed in Hoek *et al.* (2002).

Based on shear machine tests on samples from the Bátaapáti site, joint cohesion varies between 500 and 600kPa, while joint friction angle ranges from 16° to 39° due to the variation of joint roughness and joint coating (clay, calcite) (Buocz *et al.*, 2010). From the specified range of values, cohesion of 500kPa and friction angle of 30° were chosen to be the median values in the study. For both parameters a maximum deviation of 50% was considered (see Table 1). Joint dilation angle and tensile strength were assumed to be zero in the models.

Table 1: Joint parameters in the sensitivity study

Joint cohesion [kPa]	**Percent deviation**	**Joint friction angle [deg]**	**Percent deviation**
750	+50%	**41**	+50%
600	+20%	**38**	+35%
550	+10%	**34**	+17%
500	*Median*	**30**	*Median*
460	-8%	**24**	-23%
420	-16%	**20**	-37%
375	-25%	**16**	-50%
250	-50%		

Rock support

Rock support measures were incorporated in the models explicitly. Rockbolts were simulated with cable elements, whereas the sprayed concrete lining was represented by a series of beam elements attached to the periphery of the excavation. The age-dependent nature of SCL properties (i.e. strength and stiffness) was accounted for when defining the material properties of the SFRS.

Excavation sequence

To obtain realistic outputs from 2D numerical models, it is essential to consider the 3D effects of tunnel excavation, namely the longitudinal redistribution of stresses around the advancing tunnel face. As tunnel excavation progresses, *in situ* stresses are relaxed. The proportion of stress relief that occurs in a particular section before any support is installed is expressed by the so-called relaxation ratio. The choice of this factor is mainly influenced by ground conditions and the method and sequence of excavation. Calibration modelling performed by Mott MacDonald Magyarország Kft. suggested the value of 85% be used in numerical models for the design of underground structures in the Mórágy Granite Formation (Nyíregyházi and Kandi, 2009). To account for the effects of stress-relaxation, the following excavation stages were defined in the models (see also Figure 4):

(i) Establishing initial equilibrium simulating *in situ* conditions;
(ii) First stage of relaxation - excavation of the tunnel and gradual reduction of the total radial stress along the periphery of the excavation from 100% to 15% in 5% steps;
(iii) Second stage of relaxation - installation of support measures and gradual reduction of remaining stresses to zero in 5% steps.

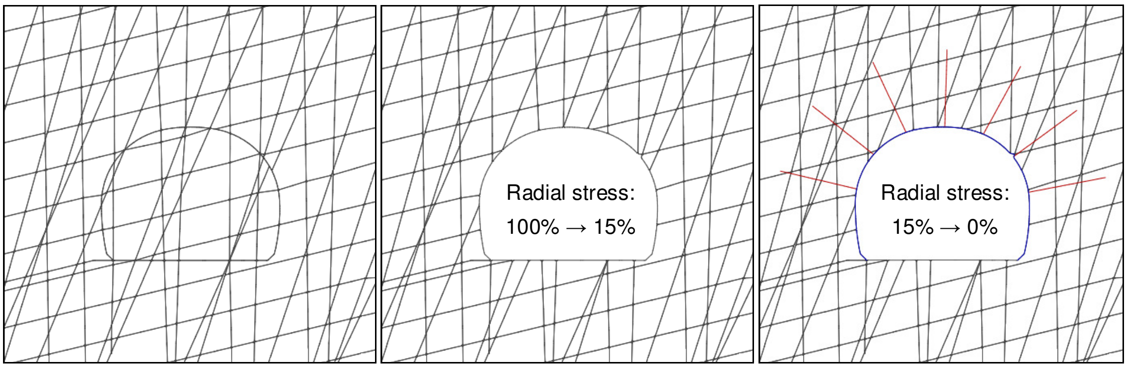

Figure 4: Implementation of excavation stages in UDEC

INTERPRETATION OF RESULTS

Displacements

As a first estimate of the predictive capability of *UDEC*, displacement results were compared to *in situ* data provided by convergence measurements. Characteristic figures are presented in Figure 5 and Figure 6. It should be noted that convergence solely indicates the relative displacement of two opposite points of the excavation boundary along each measured direction (see Figure 2), and not the actual deformed shape of the excavation. In this study convergence was assumed to be shared equally between the relevant pins.

Figures show that the pattern of deformation changes noticeably due to the variation of joint friction angle, whereas it remains the same regardless the variation of joint cohesion. Further differences can be captured in terms of displacement magnitude, namely that results vary considerably as a consequence of the variation of the joint friction angle, while they only vary moderately due to the variation of joint cohesion. These findings suggested that under the actual stress conditions it was the joint friction angle that had the more remarkable influence on joint shear strength. These phenomena also exemplify the effect of joint shear strength on block displacement and how it can directly govern the stability of the excavation.

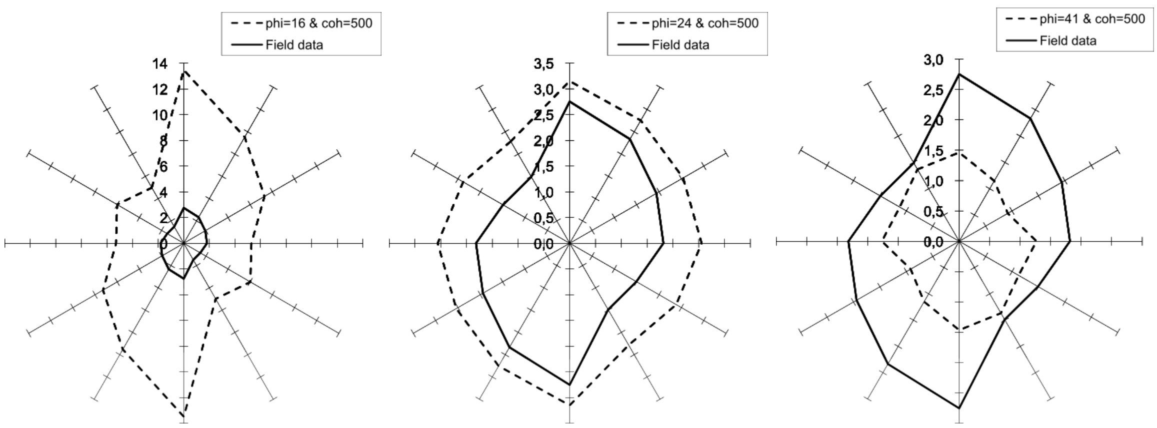

Figure 5: Predicted and measured convergence according to the variation of joint friction angle [mm] (Horváth, 2011)

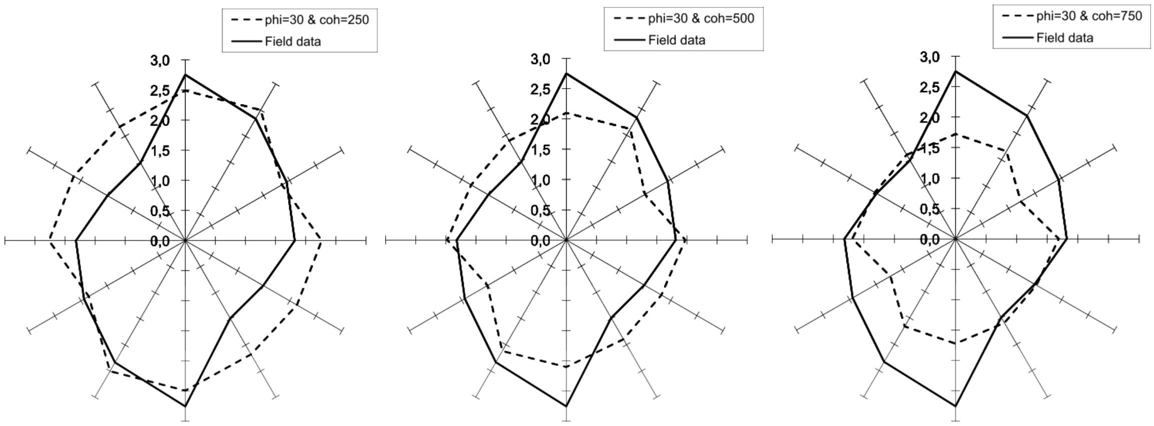

Figure 6: Predicted and measured convergence according to the variation of joint cohesion [mm] (Horváth, 2011)

Based on the displacement results, the model with joint friction angle of 30° and joint cohesion of 250kPa was found to agree best with monitoring data (see Figure 6). In comparison to test results discussed above, the chosen joint cohesion value might be considered relatively low, however it is deemed a reasonable average value given that a portion of joints is filled with clay and consequently their cohesive strength is negligible.

Rockbolt results

Extraction of results of cable elements offered the ability to gain better understanding of stress development in rockbolts. As Figure 7 demonstrates, rockbolts show very diverse loading depending on their location. Stress development in rockbolts is governed by the movement of blocks they support. Where blocks are stable, relatively small axial forces develop, while rockbolts may be subjected to significant tension in zones of local instability (e.g. unstable wedges). Consequently, rockbolt loading is closely connected to the shear strength of joints which is a governing factor in block displacement as discussed in the previous section. The variation of axial force in rockbolts according to the variation of joint shear strength parameters is illustrated in Figure 8. Rockbolt results further emphasize the sensitivity of analysis results to the variation of the joint friction angle.

Lining forces

The variation of axial and shear forces developing in the lining according to the variation of joint shear strength parameters is shown in Figure 9 and Figure 10, respectively. In terms of axial forces, the variation of joint friction angle has a more remarkable impact on results than the variation of joint cohesion. Shear force results indicate that peak values are all localized. These local peak values are the direct indicators of local instability problems (i.e. unstable wedges along the periphery of the excavation) that result in local peaks in movements leading to the development of high shear stresses. In some cases these anomalies can be handled by manually removing the evidently unstable blocks from the model similarly to real life excavation where smaller unstable wedges are removed by scaling.

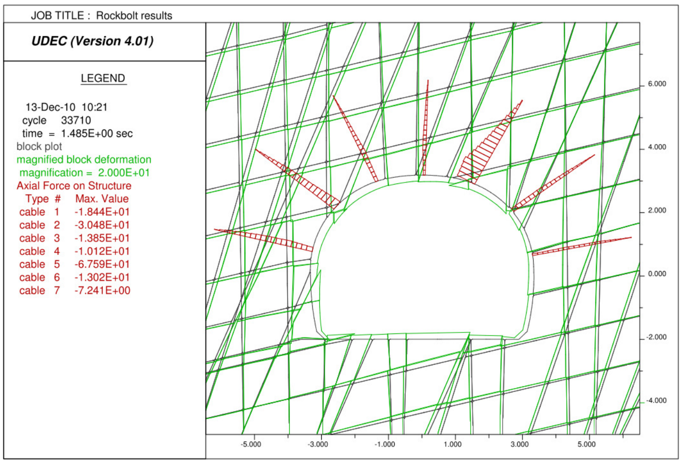

Figure 7: Rockbolt forces [kN] (Horváth, 2011)

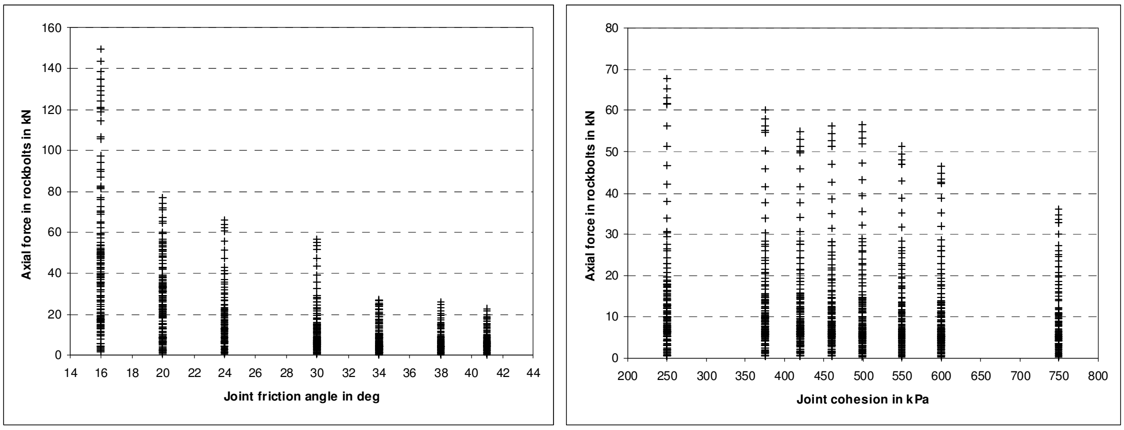

Figure 8: Rockbolt tension according to the variation of joint shear strength parameters (Horváth, 2011)

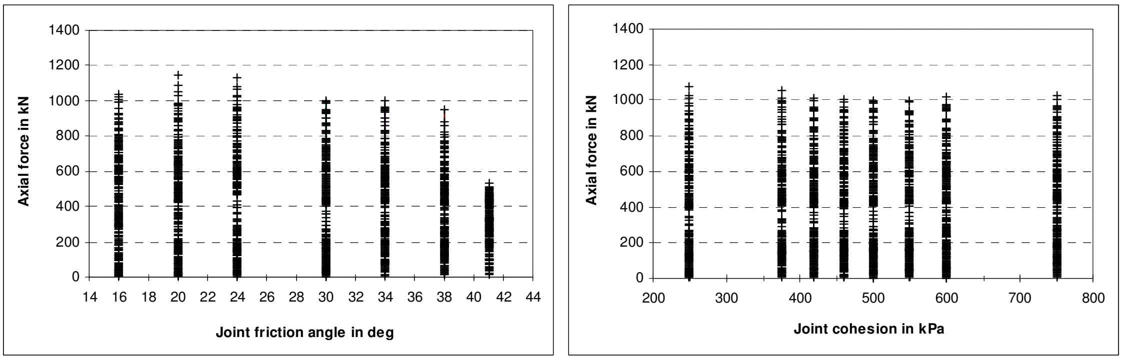

Figure 9: Axial forces according to the variation of joint shear strength parameters (Horváth, 2011)

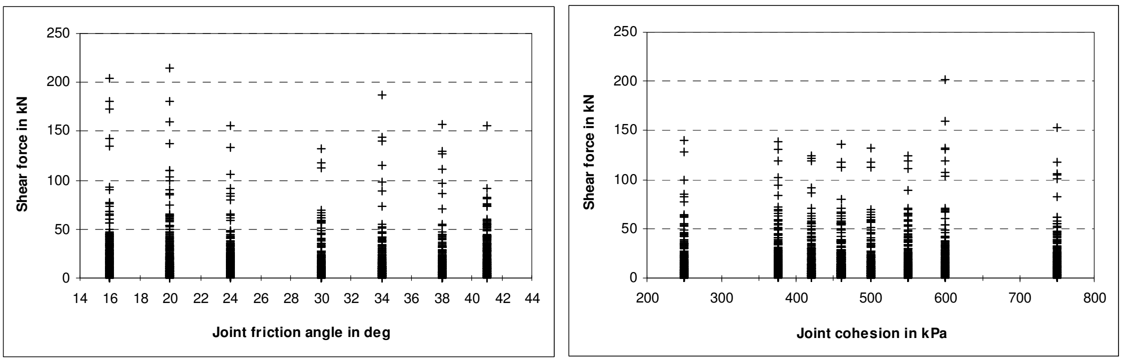

Figure 10: Shear forces according to the variation of joint shear strength parameters (Horváth, 2011)

CONCLUSIONS

Based on the numerous analyses the numerical modelling study presented in this paper comprised, it can be concluded that if provided with reliable input parameters and calibrated against field data, discrete element modelling with *UDEC* is evidently capable of providing realistic predictions about the induced stresses in the support measures and the deformation of the excavation boundary. The main strength of discrete element modelling is that it facilitates the analysis of global stability of the excavation and can also shed light on local instabilities, thus enabling designers to optimize rockbolt design.

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