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A practical technique to estimate shear stress state in 2D DEM analyses

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1 INTRODUCTION

In fractured rock mass, shear behavior of rock joints is particularly important because it dominantly controls its deformability and strength, hence the overall stability or instability of the rock mass. The above statement should lead analysts to the use of discrete elements methods for numerical analysis in order to simulate as best as possible the desired problem. *UDEC* (Itasca 2018) is one of the most complete commercially available codes for discrete element analysis, not only accepted by the scientific community but also much widespread due to its multi field functionality.

The simplest and the most used model for simulating discontinuity strength is the linear Mohr-Coulomb (M-C) friction model. This constitutive model is sufficient for planar and smooth discontinuities such as faults at residual strength, which are non-dilatant. However, the Barton-Bandis (B-B) model (Barton & Choubey 1977, Bandis 1980, Bandis et al. 1983) may be more appropriate to describe the non-linear behavior, often encountered mainly in rough rock joints. The latter takes into account several features of discontinuity strength and deformation behavior than the M-C model and simulates better the overall response of the fractured rock mass.

Despite the obvious weaknesses of the M-C model (Prassetyo et al. 2017), it is still the most frequent option for analysts and some of the main reasons for this are the following:

- small number of parameters that come under the criterion
- lack of experimental or other data in order to estimate the parameters of the B – B model
- lack of necessary experience of many users
- the non-applicability of B-B model on a 3D analysis, considering the currently commercially available numerical analysis software

Focusing in the most common 2D analyses using the M-C model for simulating discontinuities, *UDEC* allows for a wealth of information by printing or plotting magnitudes of shear displacements, slip state, separation, current ratio of shear/normal force, etc. All of these data may be helpful to the analyst in order to interpret the response of a model, but they cannot answer one common question regarding the "current shear strength state" of joints - contacts once shearing has occurred with significant deviations in unknown stress paths and the spatial distribution of critical joints – contacts of a model.

Scope of the present study is to present a simple and practical technique to exploit extracted information from the code, in order to evaluate the "current shear state" and classify either all or contacts in specific areas of interest into two categories.

2 DESIGN AND ANALYSIS

In order to demonstrate the above technique a model simulating fractured rock mass was analyzed in *UDEC*. The model analyzed measured ca. 83 × 61 meters (W × H) and compromised a fifteen meter span tunnel in thick bedded limestone. Also, two types of subvertical structures were included in the model; interbed joints without any infill with a typical spacing of 1 to 2.5 times the bed spacing (SV1) and persistent throughgoing joints extending through the rock mass with variable spacing (SV2). The respective geotechnical profile constitutes, an often encountered thin bedded fractured sedimentary rock mass. The overall parameters used for both materials and discontinuities / joints for the area of interest are given in Table 1.

Table 1. Mechanical joint and intact rock properties.

Limestone Joint Properties					Intact Rock Properties		
Property	Dimensions	Bedding	SV1	SV2	Property	Dimensions	Limestone
Knn	MPa /m	12000	8000	5700	K	MPa /m	1919
Kss	MPa /m	120	100	95	G	MPa /m	1624
Friction angle	Degrees	33	32	31	Cohesion	MPa	9.3
Residual friction angle	Degrees	29	28	27	Friction angle	Degrees	40

Once the model was consolidated and came to an equilibrium two analyses were executed using as start point for both the consolidated and balanced state.

First an intrinsic (un-supported) analysis was executed in order to examine the response and stability of the model. The tunnel in question was excavated in three stages in the following order; upper left part (left haunch), upper right part (right haunch) and finally the lower part (bench). For each part that was excavated model was cycled to equilibrium using solve command. Equilibrium state of model was double checked through history displacements curves that were placed in critical positions above the crown and the haunches of the tunnel.

The second analysis compromised the temporary support of the tunnel. Support of the tunnel was simulated with use of cables and shotcrete. The excavation sequence was the exact same as in the intrinsic analysis. The differences lie to the fact that once excavation of a part was completed model was cycled through a few hundred cycles, in order to simulate relaxation phenomena of rock mass. Afterwards, cables were installed and model was cycled again for a few hundred cycles. Finally, the structure (simulating shotcrete) was applied to the excavated part and model was cycled to equilibrium. The described sequence was followed for all three excavation stages and gradually each part of the structure was connected to the previous. For simplification reasons shotcrete was applied with its final properties, young properties of shotcrete were not considered in this analysis.

Once analyses completed, for the respective structure of rockmass and material properties, the following were established for the intrinsic (unsupported) model:

- Detachment of some blocks due to development of a secondary failure mechanism that functioned as a wedge at the upper right haunch of the tunnel.
- The overall model was on a limit equilibrium state despite the large shear displacements developed.
- Development of zero or almost zero vertical stress area above the crown of the tunnel and subsequently zero normal forces mainly on the contacts of bedding joints.

All the above verified the need of support since the crown region was on critical limit state. For example, a slight change to the selected parameters could lead to an overall collapse of the crown.

3 RESULTS AND DISCUSSION

In order to check the current shear status of the fractured rockmass the contact data of a selected area around the tunnel were extracted. Contacts are depicted in Figure 1 as pairs of absolute shear displacement - shear stress values. In this figure three major areas are detected:

- The first area depicts contacts that are following the trend line of the respective Kss(i) and these according to the simulation model have sheared elastically and have not exceed their maximum shear strength at any point.
- The second area depicts contacts that are on the horizontal axis of the chart and have an absolute shear displacement value that ranges from very small numbers up to ca. 4.5 millimeters. Normal and respective shear stresses of these contacts are zero, therefore their available shear strength is zero.
- Finally, the third area depicts all the contacts that are amongst the previously described areas. These contacts have exceeded their maximum shear strength at some point (i.e. in past) and now are functioning with their residual friction angle, given that this has been set from the user. These contacts could either be in a stable state, like in the case examined, where a limit state was referred, either they could be in a state of cursive movement, as would happen in a case of collapse.

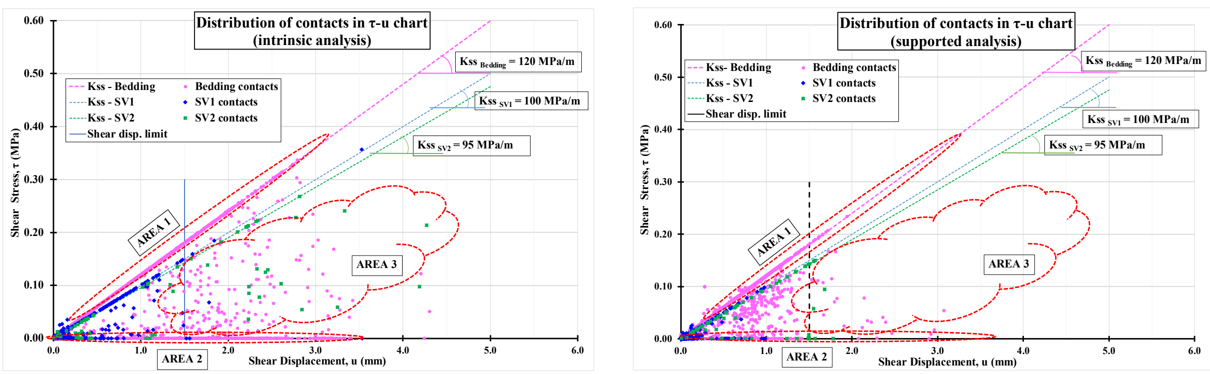

Figure 1. Distribution of contacts in τ-u chart for intrinsic (left) and supported analysis (right).

Subsequently each contact, depending on the area that appears in, is classified in one of the two following categories.

The first category relates to contacts that functioned elastically (1st area), and to contacts of the 3rd area that have exceed their maximum shear strength but have not sheared more than the shear limit value, which in our case is 1.5mm. This limit should be set by the user based always on the available geotechnical data and engineering judgement. However, given that only B-B model has an available equation to estimate the peak shear displacement of a joint (which is independent to the normal stresses), it is suggested that this should be a percentage of the maximum shear displacement (e.g. 50-70%). The reasoning behind this is that above this limit value the geometrical features and some of the mechanical characteristics of the joints start to gradually alter.

The second category includes all the contacts that have no shear strength available (2nd area) as also all the contacts of the 3rd area that have sheared significantly, which in our case examined are those that developed a shear displacement above 1.5mm.

In Figure 2 is given an indicative *UDEC* plot of y-displacements in order to illustrate better the magnitude of deformation occurred for intrinsic and supported analyses.

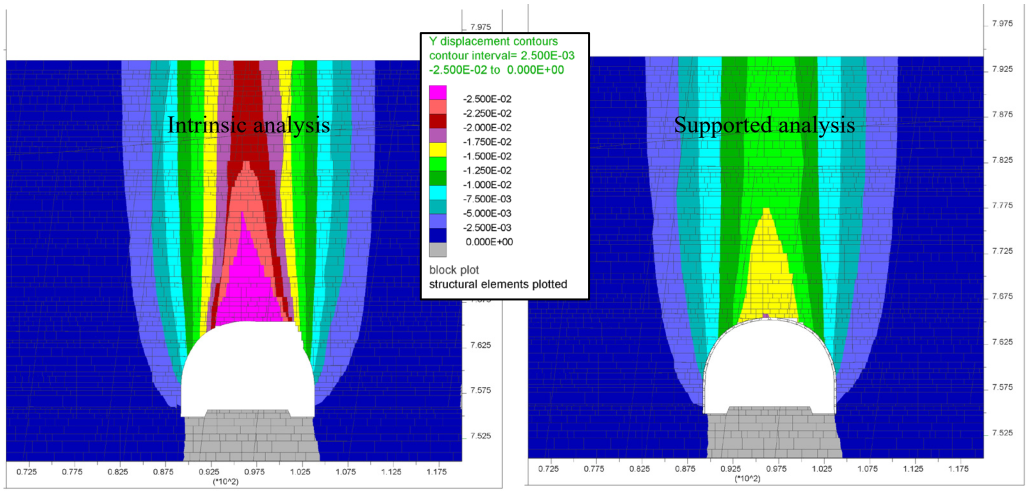

Figure 2. Y-displacement of blocks for intrinsic (left) and supported (right) analysis.

4 CONCLUSIONS

In Figure 3 are depicted all the contacts in space, using their relative coordinates from the *UDEC* output data, (left for the intrinsic – right for supported) classified in the above two described categories.

In blue are the contacts from the first category and in red those of the second. As a general conclusion it could be stated that whenever an area of the model shows that the majority of contacts are classified in the second category (i.e. "problematic contacts"), then this area could be described as potential failure zone. Respectively this technique could depict the efficacy of the applied support measures, as shown on the left-hand side chart of Figure 2, whereas previous potential failure zones are now shown in a stable state.

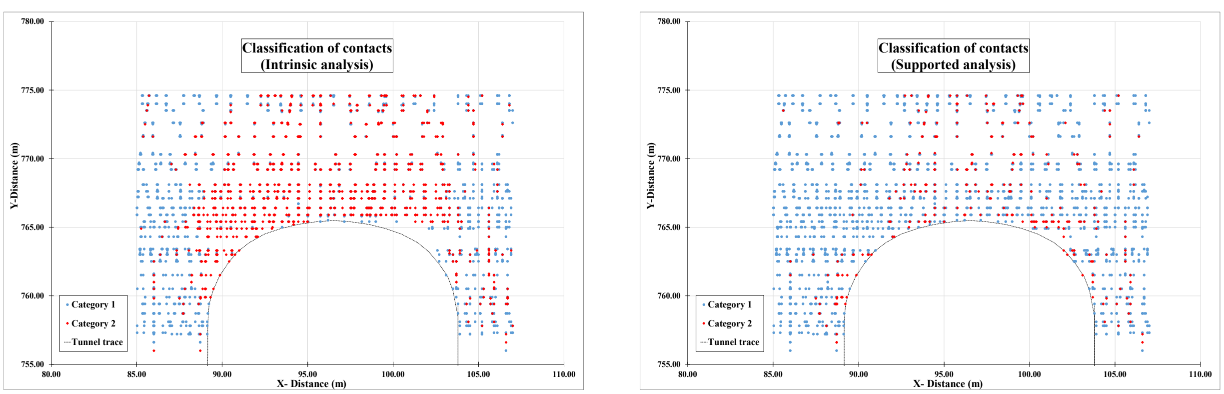

Figure 3. Classification of contacts based upon the suggest categories for intrinsic (left) and supported (right) analysis.

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