# UNDERGROUND STRUCTURES ANALYSIS IN JOINTED ROCK ENVIRONMENT

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## Introduction

Deformation process and strength of rock mass is significantly influenced by presence of planes of discontinuity, such as are e.g. layering, fault planes or cracks. Therefore, when predicting behavior of underground structures, the effort is to take the influence of these planes of discontinuity into consideration.

In the article, application of two approaches is presented on examples – numerical calculation of a tunnel and research using a physical model.

## 2D numerical calculation of a single tunnel

In numerical analyses of tunnels (and more generally of any building structures) in jointed rock environment these models are used: *discrete* (discontinuities modeled separately by means of contact elements), *equivalent* (the real system is replaced by a continuum with constitutional relation including the influence of intact material and discontinuities) and *hybrid* (models which are a combination of discrete and equivalent models).

Here a discrete model is applied on the model task (Fig. 1). Contact elements with zero thickness are used. Apart from a reference case without occurrence of a plane of discontinuity in the rock mass, cases of continuity of the rock mass being disturbed by planes of discontinuity were solved [1]. Varying location of this plane of discontinuity was considered (discontinuity running above the tunnel, under the tunnel and through the tunnel), as well as its direction given by the angle $\alpha$, contained by a joint and the horizontal plane. Elastic normal ($k_n$) and shear ($k_s$) stiffness of the discontinuity ranged between 11.4 to 11.8 GPa/m and 1.03 to 1.07 GPa/m. Strength parameters of the discontinuities were considered as follows: cohesion was considered very low $c_i$ = 1 kPa, the friction angle $\Phi_i$ was assumed variantly to be $n$ – multiple of the angle of inner friction of the intact rock (45°), when $n$ gradually assumed the value of 1 to 0.2 with the pace being 0.1. The dilatancy angle on the plane of discontinuity $\psi_i = 0$.

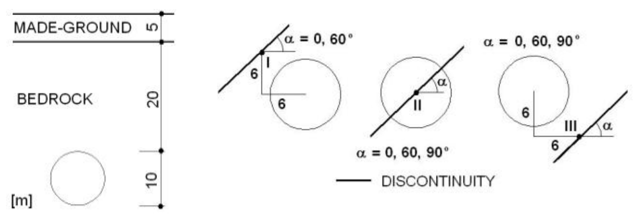


Fig. 1 Localization of the tunnel (left) and weakness of the rock environment (right)

The tunnel was driven in a full profile, the tunnel lining was not considered in the calculations.

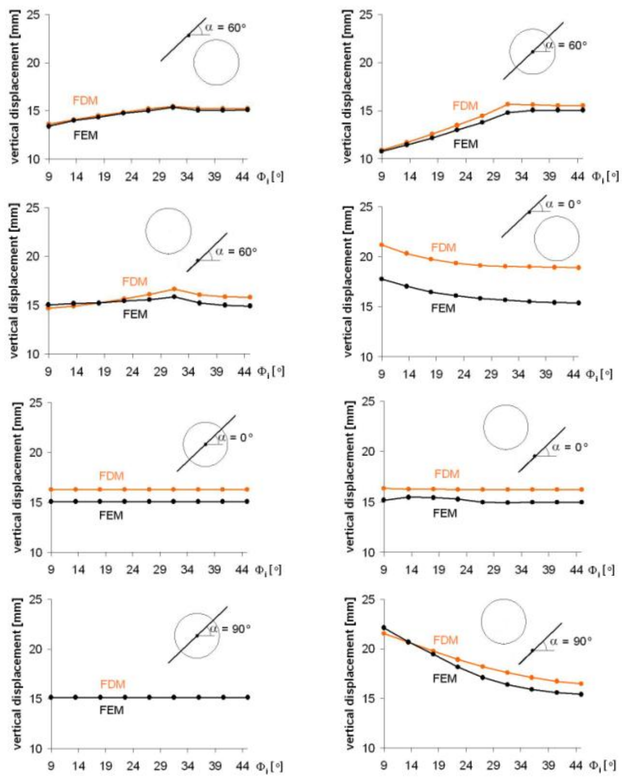


Fig. 2 Comparative calculation of FEM and FDM – prediction of vertical displacement of the tunnel top

The numerical analysis was made by two methods – by the finite element method – FEM (in the program Plaxis) and by the finite difference method - FDM (in the program Flac). The comparison is then performed for vertical displacement of the tunnel top. Its development depending on intensity of the friction on the plane of discontinuity $\Phi_i$ is shown in the Fig. 2 and it can be stated that there is no great qualitative difference. Also from the quantitative point of view there is no difference between the FEM and FDM solutions. The maximum difference in the compared results is 19 %, having been confirmed in a single solved variant. In the other variants of the model task the value of the difference is not greater than 8%, the displacement calculated by the FDM being higher in most cases.

**Model GEO – BRNO – III - 2, 2008, Scale 1:10**

The research using a physical model was directed on the third limit state, crack limit state, visible damage of an underground structure occurring in conditions of the most difficult orientation of planes of discontinuity, which are the determining factor that essentially influences the interaction of the underground structure and its surrounding [2].

This model solved three-dimensional tasks of excavation of underground structure with variable advance with horizontal orientation of planes of discontinuity. The model was built in modelling stands sized 200 x 200 x 50 cm.

Based on extensive collection of laboratory experiments, granular equivalent materials were chosen. The equivalent materials compose basic and bonding compounds. The following materials were used for building the model: siliceous sand (granularity 0.2 - 1.5 mm), ground mica (granularity 0.1 – 0.5 mm) for creating layer planes and cracks, steatite (granularity up to 1 mm), chalk (granularity up to 0.8 mm), ballotini: glass beads (granularity 0.08 – 0.2 mm), ferrosilicon: metal beads, polystyrene beads (alternative filling compound of jointing – water etc.), ecostyrene beads, gelatin (high optical sensitivity in photoelasticimetry), viscidities (vaselines and motor oils).

A real segment with low overburden was selected for physical modelling. In the base there are two circular collectors with 200 cm profile (a cable collector CC, a circular sewer CS) placed in the depth of 200 cm. For the underground structure US, circular armature with a diameter of 400 cm was assumed (Fig. 3 - left).

Fig. 3 The model Geo-2008 III Scale 1 : 10: Situation after finishing building of the model (left), Formation of cracks after finishing the tunnel excavation – detail (right)

After placing and calibrating of 7 pressure cushions, the building of the model was performed layer by layer according to time schedule. There were altogether 34 layers of equivalent material, total weight of the building material was 1 665 kg.

Monitoring of the stress state and transformation during building and subsequent excavation was realized by means of pressure cushions, tensometric measurement, electromechanical resonance (string) tensometers, mechanical scanners, electromechanical thermometers (core-type and contact-type), electrical tensometers, geodetic and photogrammetric measurement. After 10-week monitoring of the model with spatial orientation of planes of discontinuity, the procedure of excavation technology with variable advance was started and performed in stages (Fig. 3 – right, Fig. 4).


Fig. 4 Model Geo-2008 III Scale 1 : 10: Formation of cracks after finishing the excavation

During the excavation (with two-compound skin of the shield), behavior of values needed for the *KZP* coefficient (ratio of the advance to the tunnel diameter) was monitored. The optimum *KZP* margin of safety for our model was, based of the experiment, stated as follows: *KZP* = 0.32 – 0.37. The second significant result of the experiment is the value of the *KSt* coefficient – i.e. the ratio of the settlement trough volume (which equals to the volume of the equivalent material to the volume of the tunnel surface area): *KSt* = 0.25 – 0.31.

It can be stated in conclusion that in case of an underground structure in extreme conditions it is advisable for efficient and economical handling of possible accident to investigate not only all the documentation of the engineering and geological survey, results of in-situ measurement, but also to confront mathematical modelling with physical modelling.

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**References**

1. Boštík, J. Contribution to analysis of driven underground structures. Brno, Akademické nakladatelství CERM, s.r.o., 2009, 114p. (in Czech)
2. Weiglová, K.; Procházka, P. Increase of Stability of Underground works, 1st International Conference on Underground Spaces – Design, Engineering and Environmentas Aspects – Underground Spaces, Wessex, UK, WITpress, 2008, p. 139 – 147.