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Modeling large underground structures in rock formations

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Abstract. A methodology for jointed rock mass characterization starts with a research based on geological data and tests in order to define the geotechnical models used to support the decision about location, orientation and shape of cavities. Afterwards a more detailed characterization of the rock mass is performed allowing the update of the geomechanical parameters defined in the previous stage. The observed results can be also used to re-evaluate the geotechnical model using inverse methodologies. Cases of large underground structures modeling are presented. The first case concerns the modeling of cavities in volcanic formations. Then, an application to a large station from the Metro do Porto project developed in heterogeneous granite formations is also presented. Finally, the last case concerns the modeling of large cavities for a hydroelectric powerhouse complex. The finite element method and finite difference method software used is acquired from Rocscience and ITASCA, respectively.

Keywords: underground structure; numerical modelling; rock formation.

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1. Introduction

The determination of geomechanical parameters of rock masses for underground structures is still subject to high uncertainties which are related to geotechnical conditions and types of construction. An accurate determination of the geomechanical parameters is a key factor for an efficient and economic design of the underground excavation support and for the excavation itself. The methodologies used to obtain the parameters are based on laboratory and in situ tests, as well as on the application of empirical methodologies. They can provide an overall description of the rock mass, while enabling us to determine key parameters relevant to the strength, deformability and permeability of the ground.

In this paper a general methodology for jointed rock mass characterization used in large underground structures is presented, along with a reference to the need of developing new models in order to improve predictions for the parameters values. Conceptual numerical models for modeling large underground structures are also discussed. Three important cases of numerical modeling for underground structures are presented respectively, for cavities in volcanic formations for an UPHS hydroelectric project, for the Marquês Station of the Metro do Porto in heterogeneous formations, and for the hydroelectric

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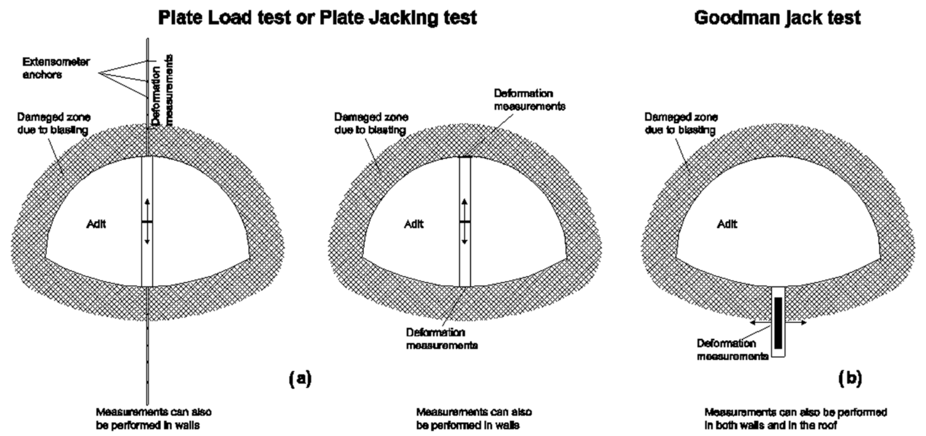


Fig. 1 Schematic of two methods for the in situ deformability evaluation: (a) Plate Load or Jacking test (with two types of possible measurements layout) and (b) Goodman Jack test

powerhouse complex of Venda Nova II. All the structures presented are located in Portugal.

2. Rock mass characterization

Due to the natural variability of the rock formations, the geotechnical properties evaluation is one of the issues with large degree of uncertainty. This is a consequence of the complex geological processes involved, due to the inherent difficulties of geomechanical characterisation. As the first step in the construction of underground works, geomechanical parameters are determined and included in establishing the engineering models. Based on these initial results, decisions are made with some degree of uncertainty. After new information is gathered, the knowledge about the analyzed problem is updated and included in the models to generate improved results and perform decisions based on the less uncertain data.

The calculation of the parameters is mainly carried out through in situ and laboratory tests and also by the application of empirical methodologies. The in situ tests for deformability characterization are normally carried out by applying a load in a certain way and measuring the correspondent deformations in the rock mass. In Fig. 1 two methods for the in situ deformability evaluation inside a gallery are presented, in particular the Plate Load or Jacking test and the Goodman Jack test.

To quantify the deformability of the rock masses, the number of in situ tests should be rationalised. Typically, a methodology that combines a small number of large scale tests with a larger number of small scale tests is adopted. The methodology can be carried out by the following four main tasks.

- Zoning of the rock mass considering the available geological information, the type of rock formations and their weathering degree, the main discontinuities and the use of empirical classification systems.
- For each zone, execution of small scale tests, in boreholes and eventually in galleries. They should be enough in number to assure a good characterisation of the rock mass. Their locations can be chosen randomly in order to obtain a mean value of the deformability modulus or in zones where this parameter is expected to be of low values.

- For each zone, execution of a small number of large scale tests to reduce the cost involved. The results should be calibrated with the values obtained from the small scale tests.
- Individual analysis of the most important faults. Carry out representative tests on the fault filling material.

In the deformability characterisation tests, the scale effect is mainly translated by the highest variability in the results of the small scale tests. In order to account for this effect, the number of tests should be enough to compensate this variability.

The tests for strength characterization are not fully satisfactory. Normally shear or sliding tests in low strength surfaces of both in the rock mass and in rock samples are performed. One of the main difficulties in performing large scale in situ tests for the strength parameters evaluation is to apply a load to a large volume of rock mass until it reaches ultimate failure. Normally, these tests are carried out until a certain stress is generated on the rock mass allowing us to obtain the deformability modulus of the rock mass, but without reaching failure. The above concerns raises important cost and time issues and therefore the strength evaluation is usually carried out using the Hoek-Brown criterion associated to the GSI system (Miranda 2003, Sousa *et al.* 2009).

In the cases where the rock masses present high anisotropy levels, the tests should be carried out in order to define the parameters that characterise that anisotropy. This is normally done by computing indexes which relate rock properties (for instance the uniaxial compressive strength, point load strength and longitudinal wave velocity) perpendicular and parallel to the planes of anisotropy (Saroglou and Tsiambaos 2007).

For some types of rock masses, the time-dependent behaviour is an important parameter for the prediction of the long-term stability in rock engineering. Creep, relaxation and loading tests at different stress or strain rates can be carried out for rheological experiments (Li and Xia 2000). Due to the difficulty involved in field tests, to obtain creep and relaxation laws, laboratory tests on intact rock samples are often conducted using simple mechanical or servo-controlled testing machines.

The evaluation of geomechanical parameters has been improved due to several developments, such as the advent of new instruments and equipment for testing with higher accuracy; development of more powerful numerical tools particularly in performing back analysis in identification problems; development of innovative tools based on Artificial Intelligence techniques for the establishment of new models; and new probabilistic methodologies for rock mass characterization based on the Bayes theory (Miranda 2007, Sousa 2010).

In the initial stage, the information available for the rock masses is limited. However, the construction of geotechnical models is a dynamic process and, as the project advances, it can be updated as new data are gathered. The geomechanical parameters are used in the numerical models for design purposes. During the construction process, new information is obtained from several sources with various levels of accuracy, for instance, one can use the data related to the mapping of the tunnel front and field measurements in back analysis calculations. This information can be used to update the values of the geomechanical parameters in a dynamic process so as to improve the accuracy of predictions as the quantity of data increases (Miranda *et al.* 2008).

Also, Data Mining techniques can be applied in order to discover new geotechnical models that are consistent with the existing knowledge. The models developed using these techniques allow us to analyse large databases of complex data, which are expected to have higher accuracy than existing ones. The application of the Data Mining techniques in rock mechanics is a very recent one. It is worth to mention that several Artificial Intelligence algorithms have been used in the study undertaken at the University of Minho on the development of new models for predicting the deformability modulus for rock masses. The results obtained so far show that these models have a much higher accuracy in the prediction of this parameter than the current expressions based mostly on the empirical classification systems application.

In order to improve the predictions and the developed models, a research project submitted to DUSEL laboratory suggested the use of the Bayesian Networks combined with the Data Mining techniques which allow the inclusion of uncertainties related to the geotechnical and construction aspects, risk management and decision making during underground construction (Sousa *et al*. 2008).

3. Modelling large underground structures

Structural design of underground works is a global process that takes into account various aspects depending on the specific nature of such works. It covers the conception stage and the calculation stage. Conception of an underground structure is basically related with the choice of the site, location and orientation, and shape and geometry of the cavities, while the calculation is related with determining structural solutions for achieving a certain performance.

The calculation methods for underground structures comprise the setting up and application of numerical models, an idealization of the reality, with simplifications made for the situations met in a structural design. Numerical models have provided an important contribution to engineering practice, in spite of the vast uncertainties on the characterization of the rock masses and even of the design. They are based either on continuous mechanics (essentially using the differential and integral methods such as the cases with the finite element method and the boundary element method), using appropriate homogenization methodologies for the rock masses (Chalhoub 2006, Yufin *et al*. 2007), or on the discontinuous mechanics, namely, the discrete element method (DEM) (Lemos 1987). In general, the continuous models are used in rock masses of good geomechanical quality. As an example, a 3D continuous model using the FLAC3D software provided by ITASCA is presented in Fig. 2 for the underground powerhouse complex and the foundation of Cahora-Bassa hydroelectric project in Mozambique.

The discontinuous medium approach calls for a mechanical characterization of the rock material and geometrical and hydromechanical characterization of the system composed of discontinuous components. In the finite element method (FEM) models, the relevant discontinuities can be represented by joint finite elements. In the discrete element method (DEM) models, the rock mass is represented by a system of blocks or particles (Lemos 2010). A typical example of application of these models is the case of the Gjovik cavern with a span of 62 m, in Norway (Barton *et al*. 1994). The discontinuous medium approach is often applied using the limit equilibrium methods in order to quantify the forces that act on the supports due to the fall of blocks (Goodman and Shi 1985).

4. Large cavities in volcanic formations

Madeira Electricity Company decided to repower the Socorridos hydroelectric project that is integrated in a multiple purpose project with the same name. The hydroelectric complex is equipped with reversible units with a differential elevation of about 450 m between the Covão upper tunnel and the lower storage tunnels. The rock mass involved is predominantly basaltic.

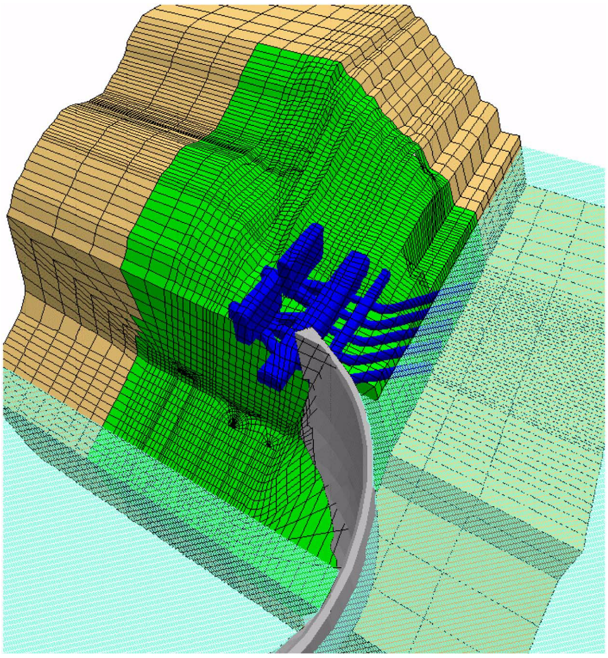


Fig. 2 3D model for Cahora-Bassa project (Lemos 2010)

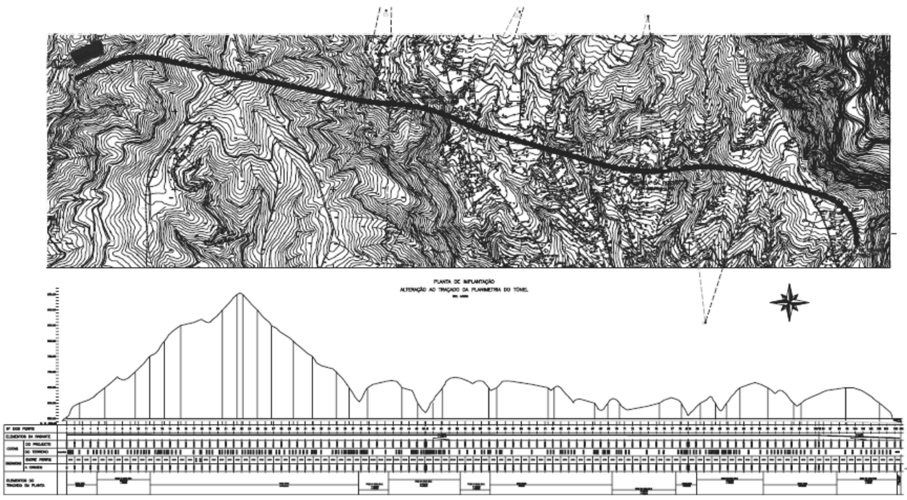


Fig. 3 Plan and longitudinal cross section of Covão tunnel (Cafofo and Sousa 2007)

The repowering included the following sequence of underground works: a 5.2 km tunnel located at the upper level (Covão tunnel); a gallery for storage of water with a total capacity of 40,000 m^3; and a cavern pumpage station, where the pumpage equipment is located. The tunnel has an extension of 5244 m and the gallery a storage capacity of about 40,000 m^3 (Fig. 3). The tunnel was designed in a complex topography region, in the volcanic complex β_2 identified in the geological

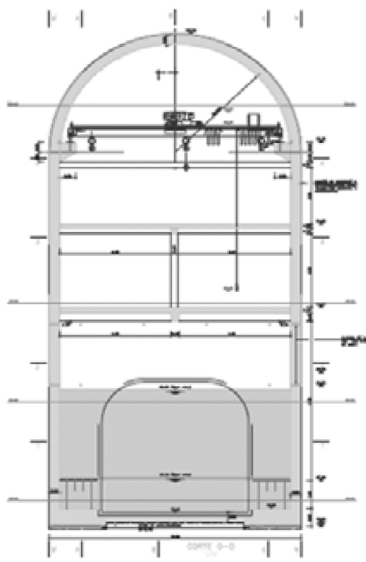


Fig. 4 Cross section of the pumping station

map of the Island (Menezes *et al*. 2007). The ground crossed by the tunnel consists of basalts, breccias and tuffs.

An empiric system was developed specifically for volcanic rock formations, derived from the RMR system and from a classification developed at IPT, Brazil, for the design of several tunnels in basaltic formations (Menezes *et al*. 2007, Moura and Sousa 2007). The parameters considered were: uniaxial compressive strength; rock characteristics; intensity of jointing; discontinuity conditions; presence of water; and disposition of blocks (Menezes *et al*. 2007). The addition of all weights gives an index called VR. The rock mass is classified in 6 classes: class I – $100 \geq VR \geq 91$ (Excellent); II – $90 \geq VR \geq 76$ (Good); III – $75 \geq VR \geq 61$ (Reasonable); IV – $60 \geq VR \geq 41$ (Regular); V – $40 \geq VR \geq 21$ (Poor); VI – $20 \geq VR \geq 0$ (Very Poor).

The hydroelectric project of Socorridos is reversible, thus the lower underground reservoir, with the extension of 1,200 m, and pumping stations are located underground. A cross section of the pumping station is illustrated at Fig. 4. More details about the characteristics of these underground works are presented in Cafofo and Sousa (2007). Numerical models were created with the FEM software in order to assess the different underground works. Regarding the pumping station, 2D and 3D model were analyzed with the software Plaxis and Phase2 (Cenorgeo 2005, Cafofo and Sousa 2007). Fig. 5 shows the scheme of the 3D model. Analyses of the reservoir tunnel were conducted for different types of formations, basalts, breccias and tuffs. The meshes used consist of triangular elements with 6 nodal points. A value of 0.8 was considered for the bulk modulus K_0.

5. Metro underground station

This section presents studies about the Marquês Station from Porto Metro. The station was built in a central elliptical 27 m deep shaft, 48 m and 40 m wide along its axis, from which two opposite

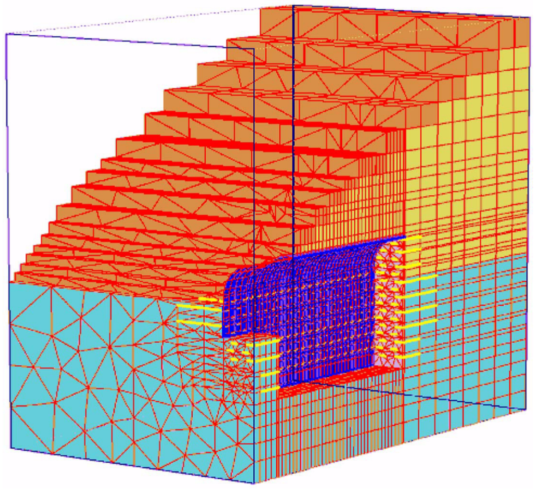


Fig. 5 3D model developed for the pumping station (Cenorgeo 2005)

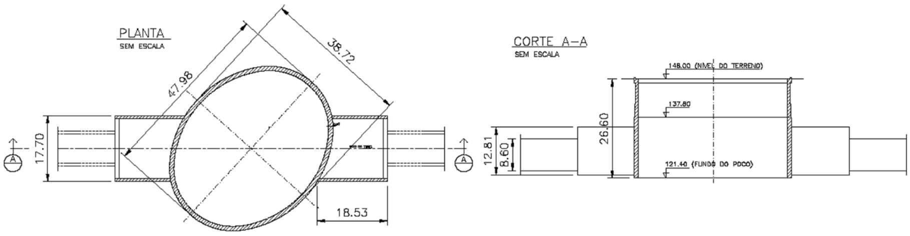


Fig. 6 Marquês station from Porto Metro

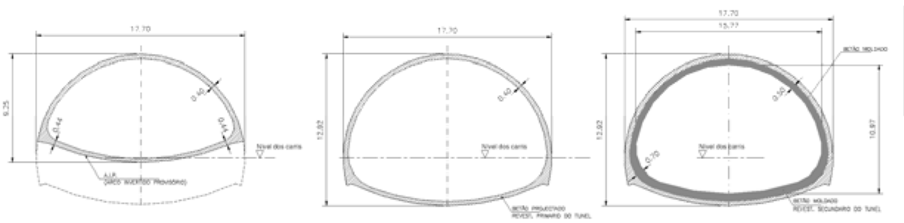


Fig. 7 Galleries building process

galleries 18 m long and about 18m wide form a section with an area of 180 m^2 (Figs. 6 and 7, Ferreira *et al.* 2005). The shaft was constructed in order to minimize the occupied space and social impact, either on car traffic and on existing arboreal species.

The station was excavated in heterogeneous granite formations, in which a sub-vertical fault was found slightly oblique to the main diameter of the shaft. The station was located in a place crossing medium-grained two mica granite. Some weathering grades were found ranging from fresh granite (W1) to residual soil (W6). The spacial development of the weathered rock is completely irregular

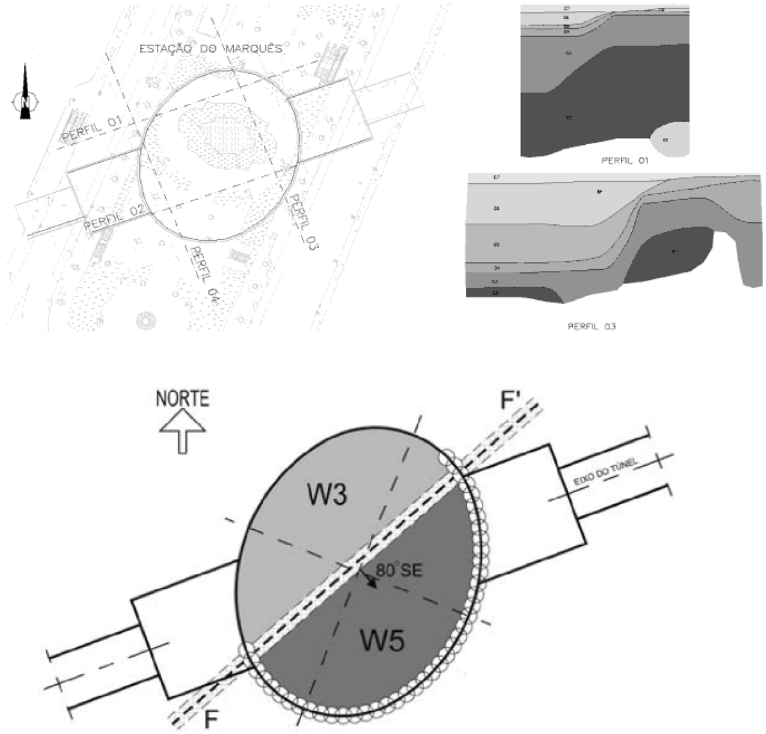


Fig. 8 Geology of the galleries and location of the fault

and erratic in the Douro valley. The change from weathered zone to good granite is abrupt, as demonstrated by a G5/G3 fault defined with a N40º orientation with a 80º SE dip and with a 0.6 m to 0.7 m thickness slightly oblique to the main shaft diameter (Fig. 8).

A geotechnical survey to collect information about the ground formations surrounding the station was carried out. Measurements were done by geophysical prospectors and also laboratory and in situ tests were accomplished. SPT, Lugeon permeability tests and dilatometer tests made it possible to characterize the interested formation. Before the construction started, the water level was 6 m deep. During the excavation of the shaft and with the help of a drainage system, the water level was lowered to the bottom. Average values were adopted to characterize the geotechnical parameters for each group considered (G2, G3, G4, G5, G6 and G7 at surface level) (Fig. 8). The rock mass strength was modelled using the Hoek-Brown and equivalent Mohr-Coulomb criteria. The collected geomechanical information was input into the GEOPAT system (Miranda 2003). To obtain the geomechanical parameters, some considerations were made, such as an average depth of 27 m and $K_0 = 0.5$, usually taken for Porto granite formations. The Mohr-Coulomb strength parameters were derived from the parameters of the Hoek-Brown criteria. Table 1 presents the geomechanical parameters obtained along with the converted parameters.

The construction process is described in detail in Ferreira *et al.* (2005). To model the structural behavior of the station, 3D numerical models were developed for the shaft zone considering the existing geology fault based on the software FLAC3D. A mesh with 8 parallelepiped nodes was elaborated, with 64,728 elements, 66,990 nodal points and 1,872 shell elements with 1976 nodal

Table 1 Geomechanical parameters obtained and converted using GEOPAT system

Gr.	γ kN/m^3	σ_c MPa	GSI	E GPa	Hoek-Brown			Mohr-Coulomb	
					m_b	s	a	c' MPa	Φ' °
G1	26	90.0	75	36.0	5.98	5.1e-2	0.50	2.33	63
G2	26	52.5	55	8.7	2.70	4.7e-3	0.50	0.44	61
G3	24	22.5	37.5	2.1	1.35	5.9e-4	0.51	0.15	52
G4	23	10.0	25.5	0.7	0.84	1.4e-4	0.53	0.08	41
G5	20	2.0	15	0.17	0.55	4.0e-5	0.56	0.03	26

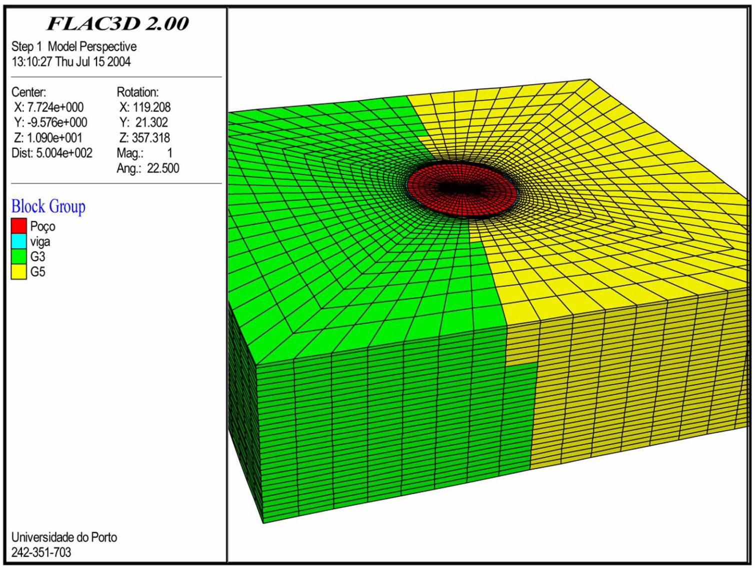


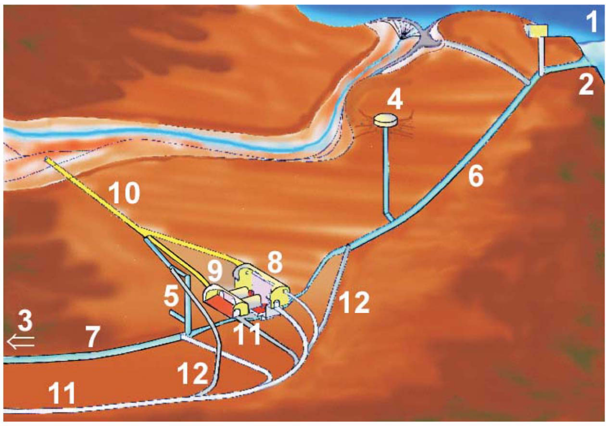
Fig. 9 3D model for the Porto Metro station

points (Fig. 9). The contact between two different Geomechanical groups G3 and G5 was taken into account. For the galleries, 2D models were developed using the software Phase2 from Rocscience. The results obtained are presented in detail in the publication of Ferreira *et al*. (2005).

6. Underground hydroelectric complex

This section presents studies about the underground powerhouse complex of Venda Nova II located in the North of Portugal. The project is almost fully composed of underground facilities, including caverns and several tunnels and shafts with total lengths of about 7.5 km and 750 m, respectively (Fig. 10), (Miranda 2007, Sousa *et al*. 2009).

The project was built in a granite rock mass with good overall quality in spite of the presence of some less favourable geological features, like the Botica fault. Before and after construction, an extensive site assessment program was conducted (Miranda 2007). Reference should be made to the LFJ tests carried out near the powerhouse in a relatively undisturbed rock mass, which was fairly



1 - Venda Nova reservoir
2 - Upper intake
3 - Lower intake
4 - Upper surge chamber
5 - Lower surge chamber
6 - Headrace tunnel
7 - Tailrace tunnel
8 - Powerhouse cavern
9 - Transformer cavern
10 - Ventilation galleries
11 - Access tunnel
12 - Auxiliary tunnel

Fig. 10 Venda Nova II underground hydroelectric complex (Miranda 2007)

Table 2. Prior and posterior estimates for the mean value of E for normal and lognormal distributions (GPa)

Parameter	Mean	Std. dev	95% CI for the mean
E prior – normal dist.	38.5	2.0	35.2-41.8
E prior – lognormal dist.	32.8	2.5	28.9-37.1
E posterior – normal dist.	37.4	0.7	36.2-38.6
E posterior – lognormal dist.	35.2	0.9	33.6-36.7

CI – Confidence interval

representative of the expected overall behaviour of the granite formation. The number of LFJ tests were 160, with a mean value of E (deformability modulus of the rock mass) equal to 36.9 GPa and a standard deviation of 6.5 GPa.

With the initial geomechanical information obtained through the application of empirical formulas (RMR, Q and GSI) and Data Mining techniques via the SAS Enterprise Miner software (Miranda 2007), an initial distribution for the elastic modulus E was obtained with a mean value of 38.5 GPa and a standard deviation of 17.6 GPa (prior distribution of E). Using the additional data provided by the results of the LFJ tests, which was less uncertain due to the lower value of the standard deviation, a Bayesian methodology was developed and applied, which allows us to obtain the correspondent updated value of E considering the two sets of data with reduced uncertainty. With this methodology, two types of distributions were considered for E, namely the normal and lognormal distributions. Table 2 shows the results for the distribution of the mean value of the parameter E and the corresponding standard deviation for the prior distribution and for the updated one (Miranda 2008). The uncertainty reduction from the prior to the posterior can be clearly observed by a high decrease in the standard deviation of the mean. To illustrate this fact, Fig. 11 shows the prior and posterior probability density functions of the mean of E considering the mean

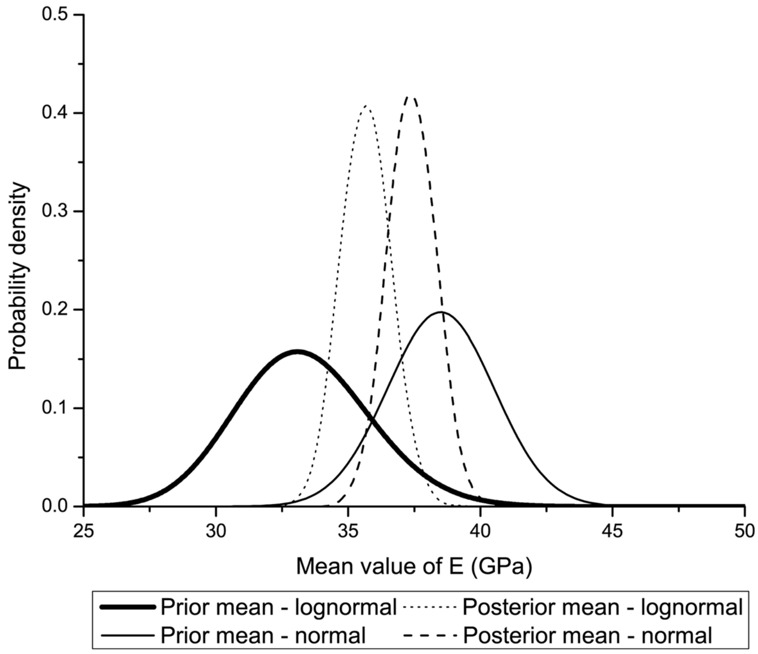


Fig. 11 Prior and posterior probability density functions for the mean value of *E*

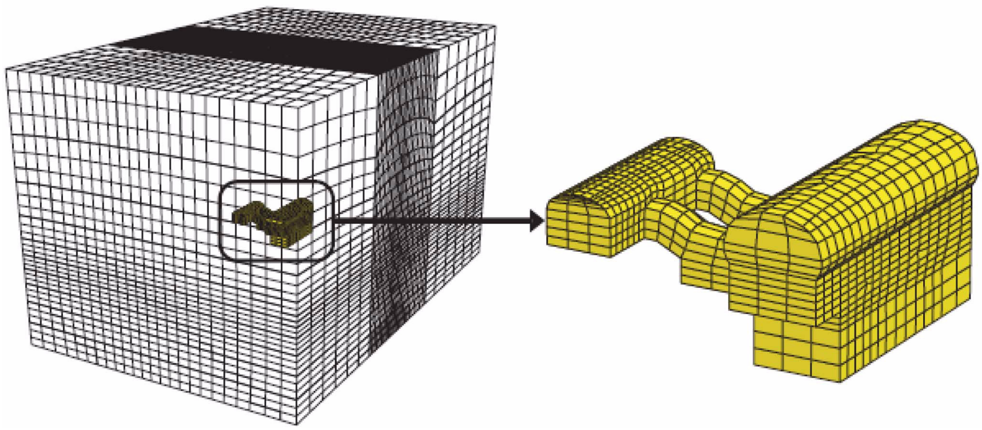


Fig 12 3D developed model

value of its standard deviation.

In situ state of stress tests were also performed during excavation, including the overcoring and SFJ tests. The vertical stresses correspond approximately to the overburden gravity load. A K_0 value of 1 was found for a direction perpendicular to the cavern axis and of 2.3 for the parallel direction. These values were obtained using the results acquired from the in situ tests together with the application of inverse methodologies

A 3D model was carried out using the software FLAC3D focusing only on the powerhouse complex (Fig. 12), (Miranda 2007). The construction sequence was simplified relatively to the one defined in design. The application of inverse methodologies with different optimization techniques was analyzed for obtaining the parameters E and K_0 (Eclaircy-Caudron *et al*. 2007). The initial geomechanical parameters of the granite formation for the numerical models were obtained using the software GEOPAT which is based on the artificial intelligence techniques (Miranda *et al*. 2004).

For inverse geomechanical analyses, two different techniques were used, namely a optimization software called SiDolo which is based on a hybrid technique combining two traditional optimization algorithms and an evolution strategy (ES) algorithm from the field of evolutionary computing like the genetic algorithms (Eclaircy-Caudron *et al*. 2007, Miranda 2007). The computed displacements were compared with the ones measured by extensometers placed in two sections along the caverns as part of the identification process.

The first preliminary calculations were performed using a 2D model as an approach to the identification problem, highlighting possible problems on the process and establishing a possible variation range for the parameters. The values obtained were not far from the initial guess using the software GEOPAT. The values of E range from 40 GPa to 45 GPa, while those of K_0 from 1.90 to 2.45. Some problems related to the convergence process were identified.

Concerning the use of the 3D model, as referred, two methodologies were used. The software SiDolo based on traditional algorithms and the ES algorithm were coupled with the model for performing the identification process. In the calculations using the SiDolo, the initial approximation for the parameters were $E = 45$ GPa and $K_0 = 2.0$. The optimised set of parameters is not significantly different from the initial guesses especially when K_0 is concerned. Values of 56.7 GPa and 1.90 were found for E and K_0, respectively. The error function undergoes a 29% decrease with the optimized set of parameters.

Two back analysis processes were performed using the ES algorithm considering different values for the termination criterion in order to evaluate the influence of these parameters on the identification process. The calculation results are referred as ES (10^{-5}) and ES (10^{-7}) respectively for the higher and lower values of the termination criterion embedded in the algorithm. Table 3 shows the results of the identification process for these calculations.

In the first calculation, with a higher value for the termination criterion, the optimised set of parameters vary by the same relative magnitude in relation to the initial values. In particular, the optimised value of E is 14% higher and K_0 14% lower. The adoption of a stricter termination criterion allows us to improve the results in relation to the observed measurements. Fig. 13 shows a comparison between the observed displacements and the ones computed with the initial and optimised sets of parameters, which indicates a more smooth distribution of errors for the optimised set of parameters.

The results obtained by the set of parameters identified by the ES algorithm slightly outperform the ones obtained by SiDolo. The reason can be attributed to the following: i) the traditional algorithm based on the software SiDolo was kept in a local minimum; ii) the termination criterion

Table 3 Results of the identification process using ES algorithm

Case	E (GPa)	K_0	Error value × 10^{-6}
Initial values	45.0	2.0	1.90
ES (10^{-5})	52.1	1.72	1.37
ES (10^{-7})	58.0	1.98	1.34

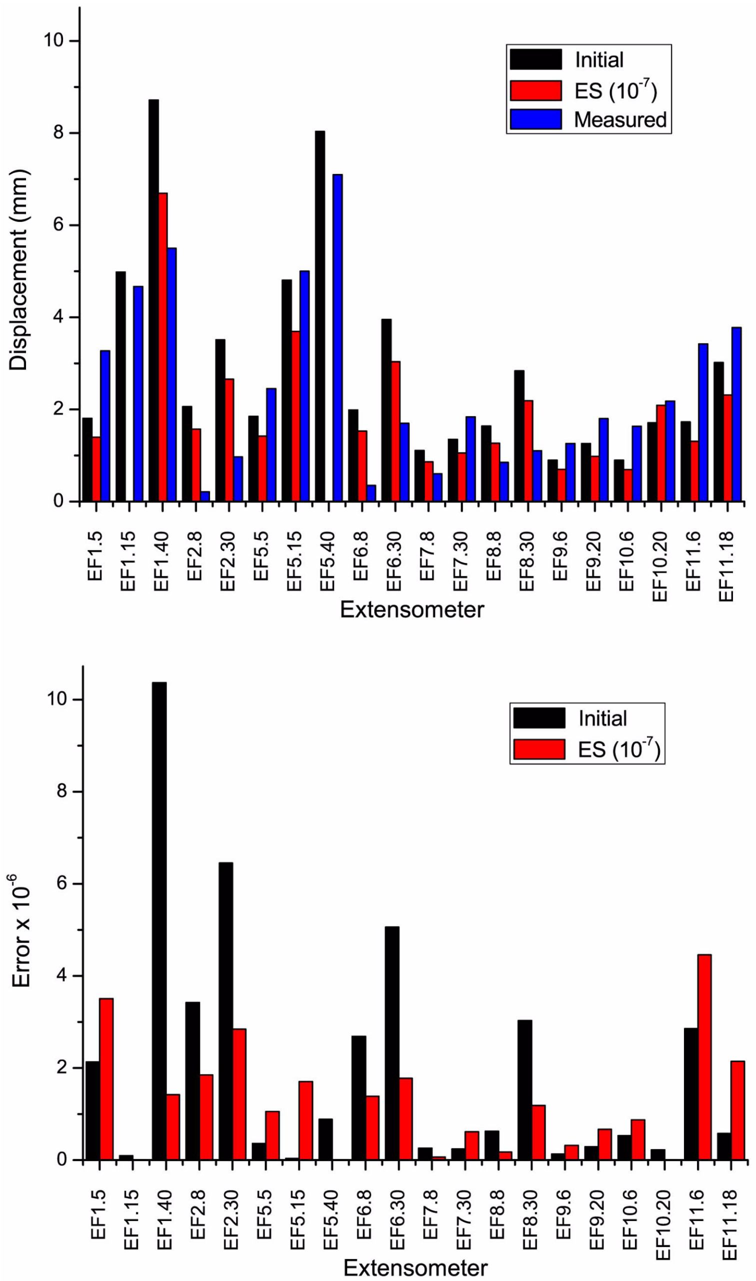


Fig. 13 Comparison between measured displacements and the ones computed with the initial set of parameters and the optimized ones

of the ES was stricter. Fig. 14 shows the topology of the error function for this case and the location of the identified values by both methodologies. In the plan view, one observes that the solution given by SiDolo and the ES using the stricter termination criterion lie near the same isoline. The remaining solution corresponds to a higher value of the error function caused not by the local

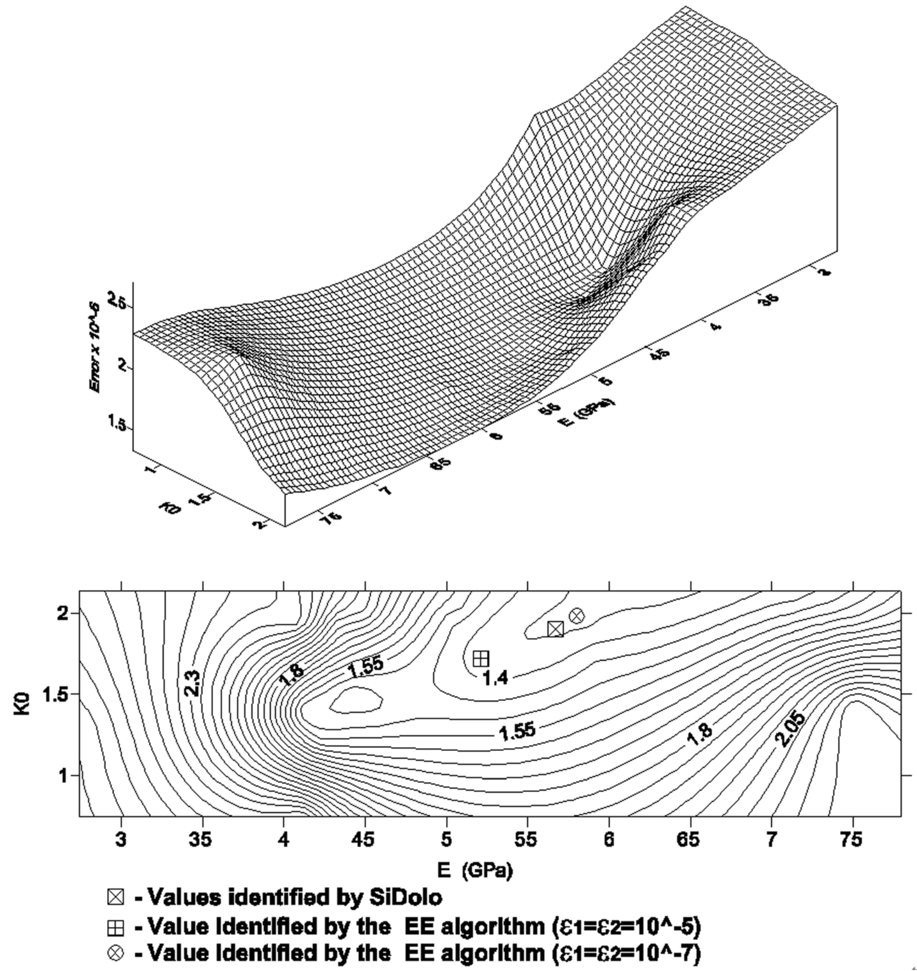


Fig. 14 Topology of the error function: (a) 3D view, (b) Plan view

minimum, but by the higher allowable value to terminate the process. Therefore, it is concluded that the slight differences in the solutions provided by the two methodologies is related with the termination criterion.

Besides, one observes a local minimum in the error function near the region corresponding to *E* and K_0 values of 45 GPa and 1.5, respectively. Both methodologies were able to avoid this local minimum and converge to the global solution.

7. Conclusions

As a remark, the spectra of available numerical models for large underground structures is immense. For each case it is necessary to select the appropriate model that can adequately represent the characteristics of the underground structure and meet the purpose of study. Advanced 3D numerical models are nowadays easier to develop and faster to compute, which can provide a better insight about the complex interactions between the rock mass and the structure, as well as guidelines on design, construction process and safety concerns. New tools from the field of Artificial Intelligence and Data Mining can be used for the decision making process, for instance in the geomechanical parameters calculation. Meanwhile, new optimization algorithms can be used in back analysis calculations that show increased robustness and efficiency in comparison to the classical ones. These studies allow us to obtain the geomechanical parameters based on the real measured behavior of the structure and surrounding rock mass. In this study, applications were presented for three large underground structures in Portugal. For the underground hydroelectric complex, special emphasis was given to the deformability modulus updating with a Bayesian methodology that shows interesting results especially in uncertainty reduction. Also, reference is made to the different back analysis techniques developed concerning the convergence characteristics of the solutions.

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