DETERMINATION OF GEOMECHANICAL PARAMETERS USING A KBS SYSTEM AND APPLICATION TO AN UNDERGROUND STATION

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ABSTRACT: In this paper a KBS computational system, designated by GEOPAT, for the calculation of geomechanical parameters for modelling underground structures in rock and soil formations is presented. To congregate the specialists' knowledge in rock and soil domains, and in particular in the tunnel engineering, causal nets have been established for decision support. GEOPAT determines the parameters for rock and soil formations, as well as heterogeneous rock formations. The developed GEOPAT system was applied to a large underground station in urban environment, excavated in granite formations, in order to obtain the deformability and strength parameters of the surrounding rock masses. Numerical models were developed considering either the obtained geomechanical parameters using artificial intelligence techniques or the parameters used in design. The numerical results obtained with the two sets of parameters are compared, as well as with the monitoring results. Some conclusions are drawn.

1. INTRODUCTION

Geotechnical design is often a subjective exercise where experience and empirical knowledge are of vital importance. Therefore there are indubitable advantages of congregating the experience and knowledge of one or several experts. Artificial Intelligence (AI) techniques play an important role generating calculation means that make possible to reach this goal. The systems that are approached in this work are the knowledge based systems (KBS).

A KBS system named GEOPAT (GEOmechanical PArameters for Tunnelling) was developed in order to obtain the main geomechanical parameters in both rock or soil masses and also heterogeneous formations. The system uses interactive interfaces and was implemented in three programming platforms – Visual Basic, Excel and KAPPA-PC [1], (Figure 1).

In the case of rock masses, RMR, Q and GSI empiric systems are applied using data collected in the field and some developed correlations. With this information the KBS system obtains the geomechanical parameters of the rock mass supported by causal networks. For soil formations results from *in situ* and laboratory tests are used for the calculation of deformability and strength parameters taking into account the strain levels interesting the serviceability of underground structures. For highly heterogeneous formations a methodology using the RMR system was adopted in order to obtain a probabilistic distribution of the GSI parameter [2]. This methodology allows the calculation of mean and characteristic values for strength and deformability parameters.

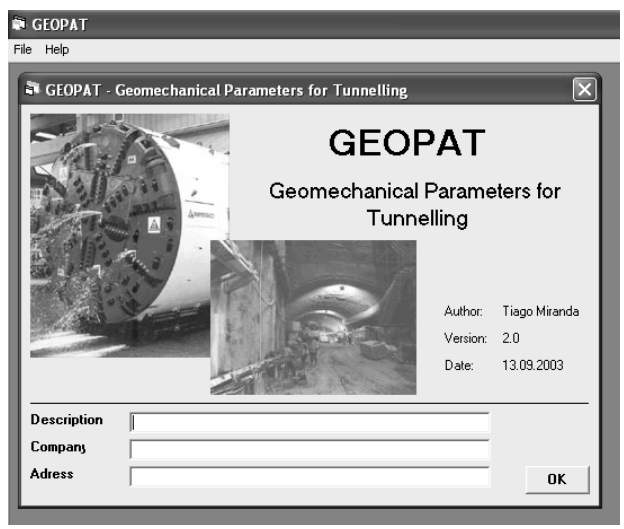

Fig. 1. Initial window of GEOPAT.

The system was applied to an underground station of Metro of Porto built in granite formations [3, 4] in order to obtain the corresponding strength and deformability geomechanical parameters. These parameters are compared with equivalent ones used in design.

Numerical models using a finite element software were developed considering either the obtained geomechanical parameters using the KBS system or the considered in design. Both numerical results obtained with the two sets of parameters are compared. Finally some important conclusions are drawn.

2. GEOPAT KBS SYSTEM

2.1. General

The first and most important step in the development of a KBS system is the acquisition of knowledge. This phase was carried out by an extensive bibliographic research, interviews with specialists and detailed studies of the several expressions and hypotheses to use. In the next sections the establishment of the knowledge base and the architecture for each considered formation will be presented.

2.2. Rock formations

In the case of the deformability modulus of the rock formations a comparative study of several expressions found in literature was carried out. After comparing the results and based on the experience of some specialists, a group of expressions was selected and, to some of them, limitations were imposed [2]. Table 1 summarizes the considered expressions.

Table 1. Expressions for the calculation of the deformability modulus in rock masses

E_M		Limitations	Ref
$E_M = 10^{(RMR-10)/40}$	(1)	RMR ≤ 80	[5]
$E_M = 2RMR - 100$	(2)	RMR >50 and σ_c>100MPa	[6]
$\frac{E_M}{E_R} = \left(0{,}0028RMR^2 + 0{,}9e^{(RMR/22{,}82)}\right)$	(3)	-	[7]
$E_M = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_c}{100}}\,10^{(GSI-10)/40}$	(4)	σ_c ≤ 100MPa	[8]
$E_M = \left(1 - \frac{D}{2}\right)10^{(GSI-10)/40}$	(5)	σ_c > 100MPa	[8]
$E_M = 10Q^{1/3}$	(6)	-	[9]
$E_M = 1{,}5Q^{0{,}6}E_R^{0{,}14}$	(7)	E_M ≤ E_R and Q ≤ 500	[10]

E_M – deformability modulus of the rock mass; σ_c – uniaxial compressive strength of the intact rock; E_R – deformability modulus of the intact rock; D – disturbance factor to account with stress relaxation and blast damage.

Hoek-Brown failure criterion [8] is used to calculate the strength parameters of the rock masses.

Data can be inserted in two different ways. In the first, RMR and Q systems are applied and eventually the values of the interaction matrix, as formulated by Hudson [11] are inputed. In the second, data is inserted in a more expedite way considering the direct introduction of GSI. In Figure 2 the window related to the RMR system is presented.

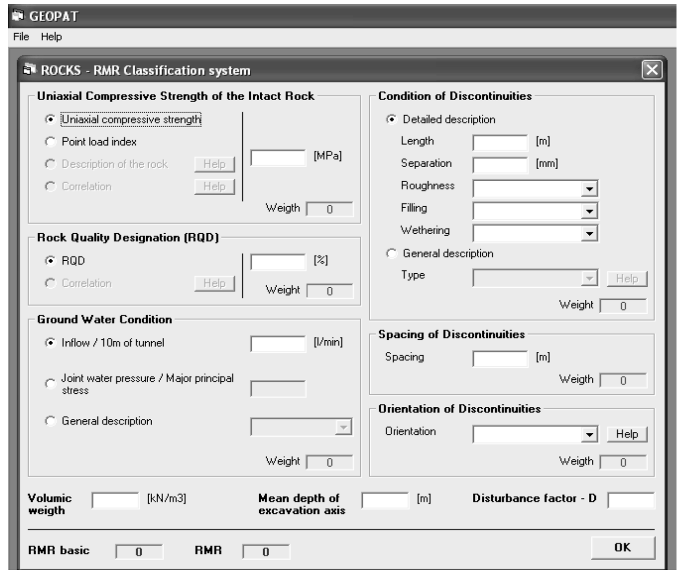

Fig. 2. Window related to the RMR system.

As the information is being inserted the values of the several weights, RMR_{basic} (RMR without the orientation of discontinuities correction) and the value of RMR are calculated and presented to the user. This interactive form of data inserting allows the user to analyze the sensibility of the values of RMR to any changes of the initial data.

For the application of the Q system a similar methodology as applied for the RMR system was followed. After inserting this information the value of GSI is calculated through correlations with the RMR (corrected value) or Q' which is a modified form of the Q parameter [12].

The values of the deformability and strength parameters are then calculated using the described expressions. As they are several expressions to calculate the deformability modulus, a methodology was defined for obtaining one final value taking into consideration the mean and variance values [2] In Figure 3 is presented the causal net that summarizes this process. Other causal net was also developed for the strength parameters. In Figure 4 is presented an example of results given by GEOPAT using the RMR and Q systems.

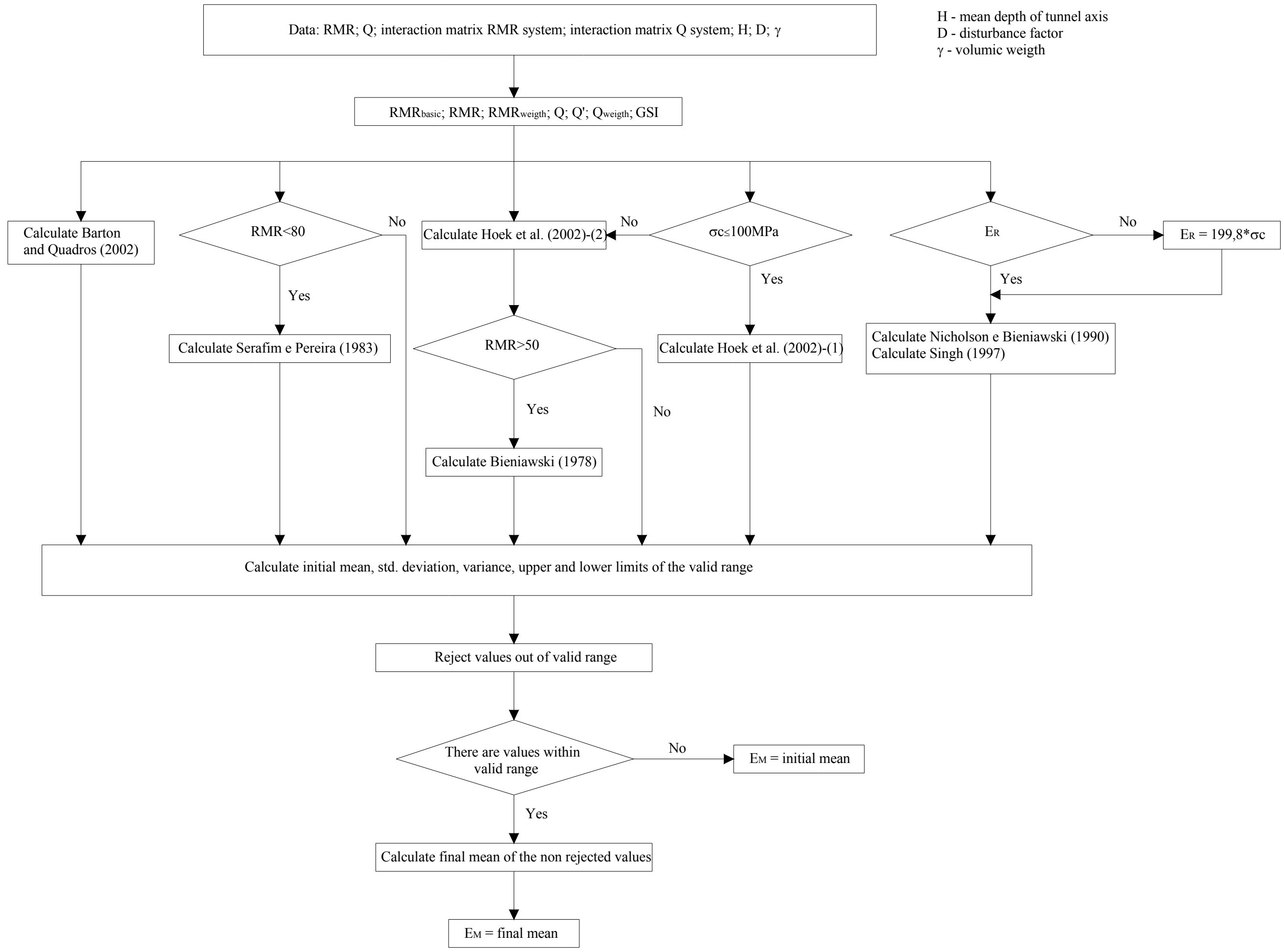

Fig. 3. Causal net for the determination of the deformability modulus in rock masses.

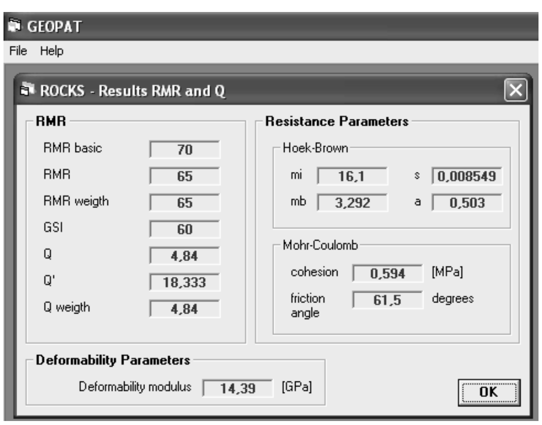

Fig. 4. Example of results using the RMR and Q systems.

2.3. *Soil formations*

In the case of soil formations GEOPAT calculates shearing resistance parameters and deformability parameters from a wide range of laboratory and *in situ* tests distinguishing the cases of residual and transported soils. The expressions used in the system where found in the work of several authors and a summary can be found in [13]. In this paper only the expressions used for transported soils will be presented.

In the system, the peak angle of shearing resistance (φ'_P) of the soil can be calculated from the results of the cone penetration test (CPT) and Marchetti dilatometer test (DMT). The expressions used for the calculation of this parameter are presented in Table 2.

Table 2. Expressions for the calculation of the friction angle in transported soils

ϕ'_P		Ref
$\phi'_p = \arctan\left[0{,}1 + 0{,}38 \cdot \log\left(q_c / \sigma'_{v0}\right)\right]$	(8)	[14]
$\phi'_P = 20^o + \dfrac{1}{0{,}04 + 0{,}06 / k_D}$	(9)	[15]

φ'_P - peak angle of shearing resistance; q_c - tip resistance of the CPT test; σ'_{v0} - initial vertical effective stress; k_D - horizontal stress index obtained with the results of the DMT test.

If there is enough information, dilatance angle (α) is calculated using the empirical strength-dilatancy relationship proposed by [16]:

$$\alpha = \phi'_p - \phi'_{cv} = m\left\{D_R \left| Q - \ln\left(\sigma'_{mf}\right)\right.\right\} - R \qquad \phi'_P \geq \phi'_{cv} \qquad (10)$$

where: *m* is a coefficient respectively equal to 3 and 5 for axisymetric and plane strain conditions; D_r is the relative density index; R≈1 for sands; Q is a logarithmic function of grains compressive strength (quartz sands≈10 and calcareous sands≈8); σ'_{mf} the mean effective stress at failure (in the system this value is considered equal to σ'_{v0}). The value of D_r can be obtained from correlations with the standard penetration test (SPT) and CPT tests [17].

The calculation of the secant modulus for deformation levels which interest the underground works is done based in the very small strain Young's modulus (E_0). Some tests can provide this value directly while others are better related with G_0, based on which it is easy to obtain E_0. For the calculation of G_0 the considered tests are the following: SPT, CPT and CH. The expressions which relate the parameters obtained by these tests and G_0 are presented in Table 3.

Table 3. Expressions for the calculation of G_0 in transported soils

G_0		Ref
$\frac{G_0}{q_c} = 290.57\left[\frac{q_c}{(\sigma'_{vo}p_a)^{0.5}}\right]^{-0.75}$	(11)	[18]
$\frac{G_0}{q_c} = 144.04\left[\frac{q_c}{(\sigma'_{vo}p_a)^{0.5}}\right]^{-0.631}$	(12)	[19]
$V_S = 69 \cdot N_{60}^{0,17} \cdot Z^{0,2} \cdot F_A \cdot F_G$	(13)	[20]
$G_0 = \rho \cdot V_s^2$	(14)	
$\frac{G_0(\text{MPa})}{F(e)} = [3.16 \text{ to } 5.72]\cdot[p'_0(\text{MPa})\cdot 10^3]^{0.4}$	(15)	[21]
where: $F(e) = \frac{(2.17-e)^2}{1+e}$	(16)	

p_a – reference stress (100kPa); V_s - wave velocity (m/s); N_{60} - number of blow/feet for a energy ratio of 60%; Z - depth (m); F_G - geological factor (clays=1; sands=1.086); F_A - age factor (Holocene=1; Pleistocene=1.303); ρ - is the total mass density; e – void ratio; p'_0 – mean effective stress.

The value of E_0 can be directly from the modulus determined by the DMT test (M_{DMT}) using the approximate relation:

$$E_0 \approx 0.8 M_{DMT} \qquad (17)$$

As can be observed from Tables 2 and 3 there are several expressions to obtain the geomechanical parameters. Therefore, the calculation of final values of the parameters is done through the same methodology already described for the rock masses.

It is well known that the deformability modulus of geotechnical materials is highly dependent on the strain levels. In the case of soil masses, the deformability modulus to use in design should be adapted for the expected level of strains according the serviceability limit state of the structure. For this purpose the system proceeds to a correction of E_0 (mean value obtained by the described methodology) multiplying it by a corrective factor (F). Considering several proposals which can be found in literature is assumed reasonable a value of 0.05% for strain levels, in the case of bored tunnels, and 0.3% in the case of SEM/NATM tunnels [3].

Studies done in sands and clays using resonant column tests [22], show that for a depth of tunnel axis of, approximately, 15 m and for typical values of plasticity index of granite residual soils (≈15%), one can conclude that F values are similar for materials with and without plasticity. Considering this analysis and specialists' judgment, F values of 5 in the case of bored tunnels and 3.5 in the case of SEM/NATM tunnels were adopted. In Figure 5 is presented the window related to the data input for soil formations.

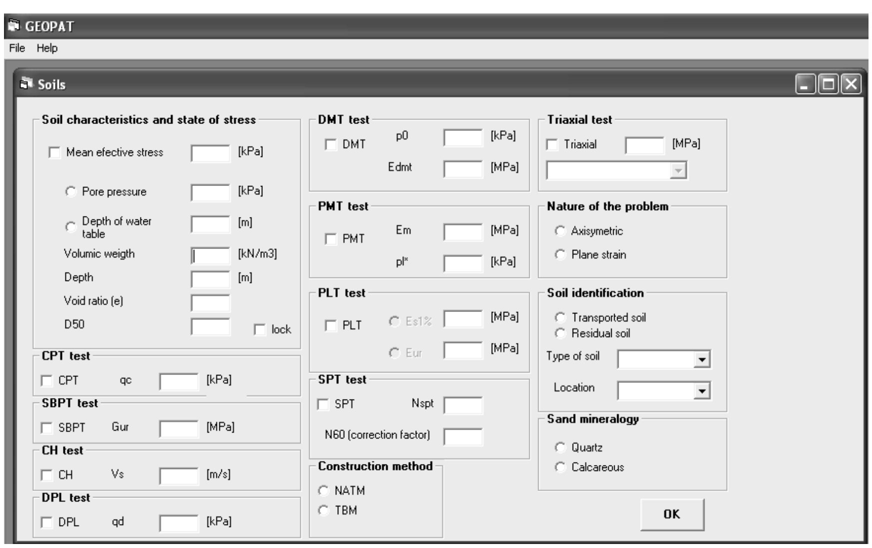

Fig. 5. Window for data input of the tests in soil formations.

2.4. Heterogeneous formations

Relatively to the heterogeneous rock formations and due to the great uncertainty in their geomechanical behavior, a probabilistic approach was implemented. A statistical distribution of these geotechnical structures is obtained using the RMR system. The mean and standard deviation of the weights of this classification must be inserted. Then, assuming a normal distribution, the system generates a thousand random values for each of the weights using the Monte Carlo method. These values are added being obtained the correspondent values of the RMR which are transformed in the GSI parameter. By this way a probabilistic distribution of this parameter is obtained, which can be visualized through one histogram (Figure 6). Mean and characteristic values of GSI which cover, practically, all possible scenarios, are presented and can be later used for the determination of the strength and deformability parameters.

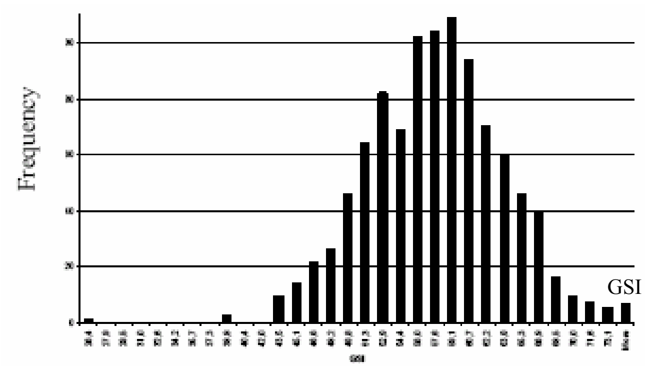

Fig. 6. Histogram of the GSI parameter given by GEOPAT.

3. APPLICATION TO AN UNDERGROUND STATION

3.1. Description

The Bolhão underground station from the "Metro do Porto" (Porto Light Train system) network, is situated in one of the main commercial areas of the city [4]. This station was built at a depth of 12 m, under buildings dating from the beginning of the 20th century and "Capela das Almas" which is town patrimonial heritage.

Basically the station layout consists in two perpendicular caverns with 70 and 62 m of length and diameters of excavation of 18 and 16 m, respectively.

The caverns were excavated in a granite formation commonly known as "Granito do Porto". This formation is characterized by the occurrence of highly heterogeneous weathering profiles, which hinders the establishment of a standard geomechanical behaviour. Figure 7 presents a plant of the layout of the cavern as well as the spatial distribution of the geomechanical groups. Figure 8 shows a longitudinal cut of the main cavern along its axis and in Figure 9 a photo of the cavern during construction stage is presented.

In design stage, the geomechanical parameters were obtained based in a reference report and complementar information obtained by *in situ* and laboratory tests. For the geomechanical groups G3 to G5, Tables 4 and 5 present the parameters used in design.

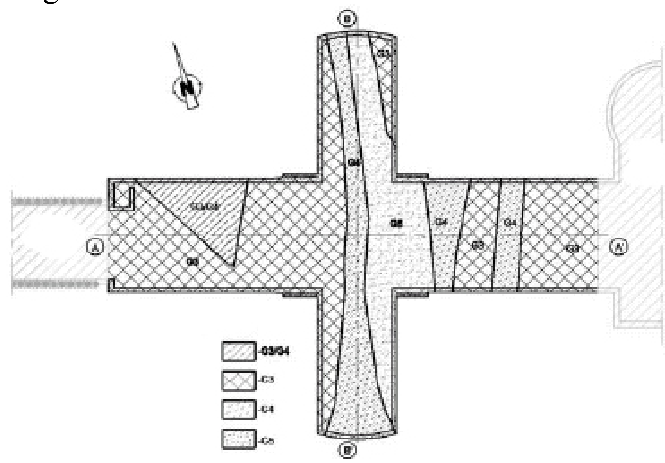

Fig. 7. Plant of the cavern layout with spatial distribution of the geomechanical groups.

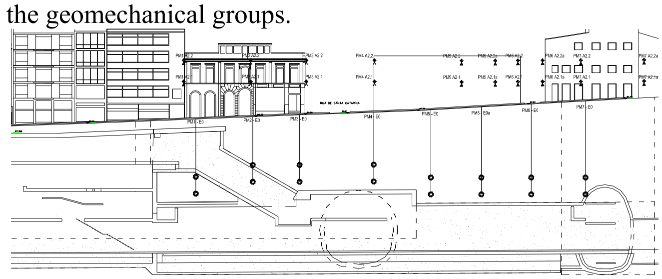

Fig. 8. Longitudinal cut along the axis of the main cavern.

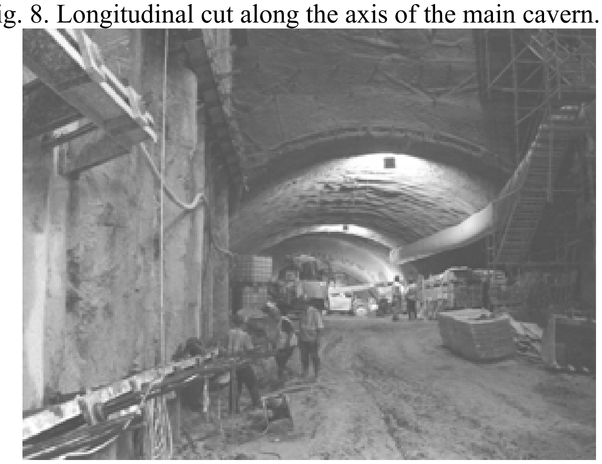

Fig. 9. Photo of the cavern in construction stage.

Table 4 – Some geomechanical parameters used in design stage for Bolhão underground station

Geomechani-cal groups	weathering and fracture	γ (kN/m^3)	E (GPa)	σ_c (MPa)
G3	W_3; F_4 a F_3	24	1.5	25
G4	W_4; $F_{4\text{-}5}$	23	0.6	10
G5	W_5; F_5	20	0.15	-

Table 5 – Strength parameters used in design stage for Bolhão underground station

Geomechani-cal groups	Mohr-Coulomb		Hoek-Brown		
	ϕ´ (°)	c' (kPa)	m_b	s	a
G3	40	150	0.98	7.5E-4	0.5
G4	35	75	0.67	0	0.5
G5	35	40	-	-	0.5

3.2. *Application of the system*

The described system was applied to this station in order to obtain the geomechanical parameters and perform some comparisons with the design parameters. The entrance values and the results given by the system are presented in Tables 6 and 7.

Table 6 - Entrance values for the application of the GEOPAT system

Geo. groups	γ (kN/m^3)	σ_c (MPa)	GSI	D
G3	24	22.5	37.5	0.2
G4	23	10	25.5	0.2
G5	20	2	15	0.2

Table 7 – Geomechanical parameters given by GEOPAT

Geo. groups	E (GPa)	Mohr-Coulomb		Hoek-Brown		
		ϕ´ (°)	c' (kPa)	m_b	s	a
G3	3.4	54	126	1.35	5.87E-4	0.513
G4	1.6	44	66	0.84	1.41E-4	0.530
G5	0.84	28	24	0.55	4.03E-5	0.561

Comparing the two sets of parameters (used in design and given by GEOPAT) it is possible to conclude that they are very different. Concerning the deformability modulus the values given by the system are much higher than the ones used in design. For the strength parameters there is no pattern of which set gives the highest values.

3.3. *Numerical modeling*

Numerical models using a finite element software (Phases^2) were developed considering the two sets of parameters (Figure 10). The considered section for the model was far from the intersection between the caverns to avoid disturbance in the results due to the three-dimensional effect of the geometry.

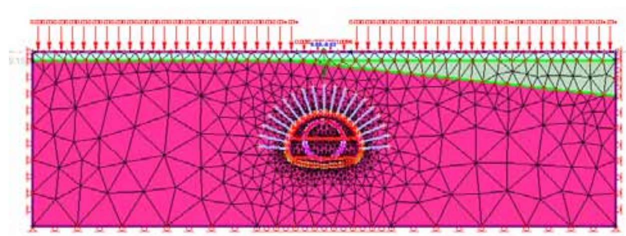

Fig. 10. Finite element model for the underground structure.

The construction stages which were considered are presented in Table 8. The calculated stresses in the support systems are similar with the two sets of parameters but the displacements are very different. Using design parameters, the surface settlement is about 4.9 mm while using GEOPAT parameters is 1.8 mm. This fact is due to the very different values of deformability modulus considered in the two sets of parameters. In fact, the ratio between the two calculated values of settlement is 2.7 which is similar to the ratio between the deformability modulus (≈2.3) for the most predominant geomechanical group (G3).

Table 8 – Stages of the construction sequence

Stage	Description
1	Opening of the gallery with TBM and application of concrete rings.
2	Excavation of the upper part of the cavern and demolition of the concrete rings. Placement of a temporary embankment.
3	Application of a 30 cm layer of shotcrete and execution of Super Swellex 200kN rockbolts, 6 m length and 1.5 m spacing.
4	Excavation of the inferior part of the cavern, demolition of the remaining parts of the concrete rings and removal of the temporary embankment.
5	Application of a support scheme similar to the defined in Phase 3.
6	Application of the definitive support in concrete.

In Figure 11 the calculated curves of the surface settlements along the construction stages is presented. The curves are very similar to the theoretical ones.

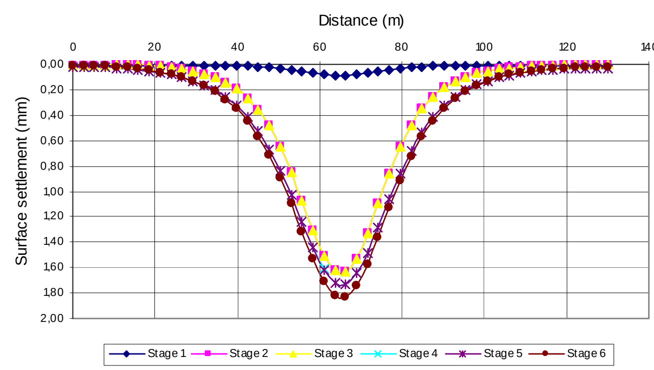

Fig. 11. Calculated curves of surface settlements during the construction stages.

The maximum monitored surface settlements in section were about 2 mm which agree very well with the computed values using GEOPAT.

The influence of surface loads in the behavior of the rock mass and structure was investigated. The conclusion was that, in this case, surface loads have little influence in the global behavior since a substantial increase of the loads led to insignificant changes in every computed parameter. In terms of deformation, the behavior of the rock mass is controlled, fundamentally, by the intrinsic characteristics of the materials (geomechanical parameters), the construction method and excavated volume.

4. CONCLUSION

In this paper a KBS system called GEOPAT was presented. This system can be applied for the calculation of geomechanical parameters for underground structures modelling. Different methodologies were defined for rock, soils and heterogeneous rock masses. The knowledge base was developed based on intensive bibliographic research, interviews with specialists and detailed studies. The gathered knowledge was then organized in causal nets and a methodology was developed to calculate the final values of the parameters.

In the case of the rock masses, strength parameters are calculated using the Hoek-Brown criterion supported by a causal net. For the deformability the calculation is executed through several expressions, selected after a study, also supported in a causal net.

For the soil masses the values of the geomechanical parameters are calculated based on the results of a great variety of tests. Distinction is made when dealing with transported or residual soils. Corrective factors for E_0 were proposed for the calculation of deformability modulus for deformation levels which interest the underground works. It was considered reasonable to assume a value of 0.05% for strain levels, in the case of bored tunnels, and 0.3% in the case of SEM/NATM tunnels. Based on studies done in sands and clays and specialists' judgment F values of 5 in the case of bored tunnels and 3.5 in the case of SEM/NATM tunnels were adopted.

The calculation of the parameters in heterogeneous rock masses is executed through a probabilistic analysis of the value of RMR and GSI.

The developed GEOPAT system was applied to a large underground station in urban environment, excavated in granite formations, in order to obtain the deformability and strength parameters of the surrounding rock masses. Design parameters were compared with GEOPAT parameters and it was possible to conclude that they were very different. Concerning the deformability modulus the values given by GEOPAT are much higher than the ones used in design. For the strength parameters there is no pattern of which set gives the highest values.

Numerical models were developed using a finite element software and the two sets of parameters. The calculated stresses in the support systems are similar but the displacements were very different. The surface settlement is about 4.9 mm and 1.8 mm using design and GEOPAT parameters, respectively. This fact is due to the very different values of deformability modulus considered in the two sets of parameters for the most predominant geomechanical group (G3).

The calculated curves of the surface settlements along the construction stages are very similar to the theoretical ones. The maximum monitored surface settlements, about 2 mm, agreed well with the one calculated using GEOPAT parameters.

It was also concluded that, in this case, surface loads have little influence in the global behavior of the rock mass and structure. The behavior of the rock mass is controlled, fundamentally, by the geomechanical parameters of the rock mass, the construction method and excavated volume.

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