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A PROBABILISTIC APPROACH FOR CHARACTERIZING THE COMPLEX GEOLOGIC ENVIRONMENT FOR DESIGN OF THE NEW METRO DO PORTO

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ABSTRACT: The more significant design problems of the new light metro of Oporto city, currently under construction with a special TBM, were related to the complex and highly variable geotechnical conditions. A specific procedure, based on statistical and probabilistic analyses of data was followed to improve the confidence in geotechnical characterization of the ground for input to design. Probabilistic analyses combined with empirical approach were used to cross check the results of FDM numerical analyses conducted for lining design. The threshold values defined by FDM for the rock load are compatible with the values defined by the empirical approach.

1. INTRODUCTION

The new light metro of the city of Oporto (Portugal), currently under construction, consists of two main lines and eleven stations. The two lines have a total length of 20km, of which 7km located under the densely populated city. The tunnels have an 8m internal diameter, and are excavated by a special TBM that operates in both open and closed mode (EPBS), under an overburden varying between 20 and 35m.

The complex and highly variable geotechnical conditions and the short time-frame for the realization of the project, resulted in a particularly challenging design. This paper presents the methodology followed for defining (a) ground properties, and (b) the design loads for the segmental lining, using principles of probability theory as applied to design of underground structures. Definition of the geotechnical properties was also the basis for machine selection (face stability analyses) and for settlement-risk analyses. The approach for geotechnical characterization and design is illustrated in Figure 1.

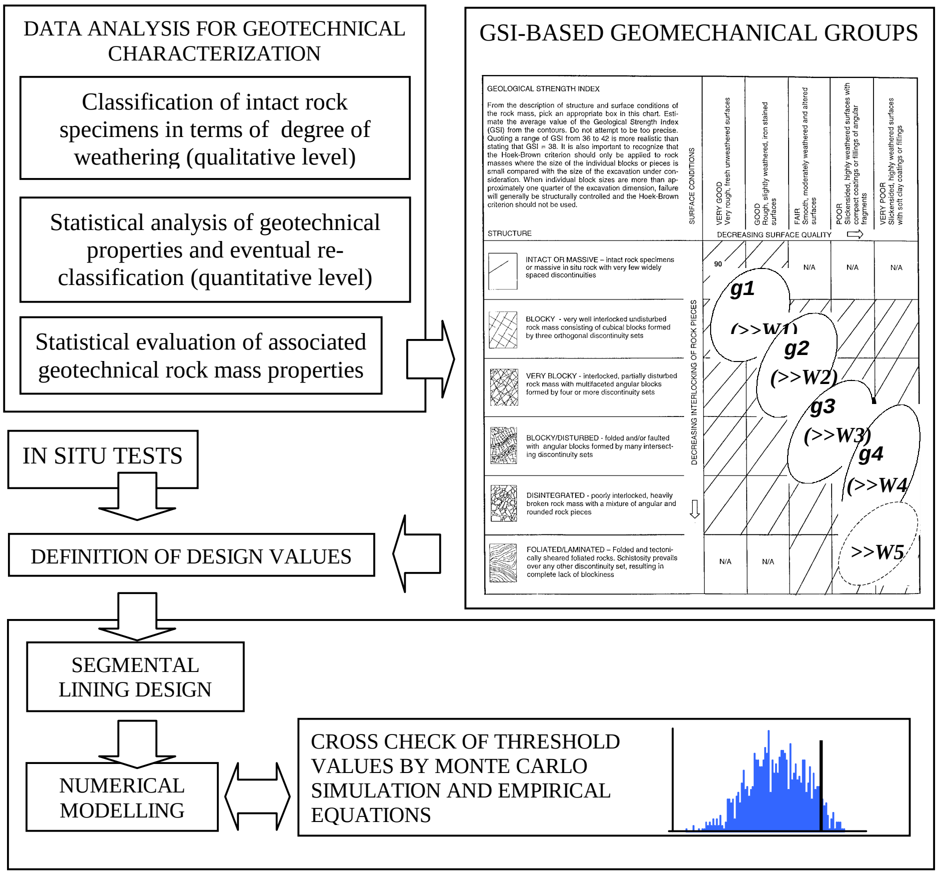

Figure 1. Conceptual procedure for geotechnical characterization and design

2. GEOTECHNICAL CHARACTERISTICS

In geologic terms, Oporto is located at the central Hyberic zone, adjacent to the Porto-Tomar fault. The tunnels are excavated in the so-called "Granite of Porto" which in the project area is associated with weathering of various stages and it is typically presented in masses of sound rock, “bolas”, of various dimensions, embedded in layers of completely decomposed granite. In situ and laboratory tests were carried outto define the geotechnical ground properties. The results of 156 uniaxial compressive tests conducted are illustrated as an example in Table 1 and Figure 2.

2.1 Weathering and intact rock properties

Weathering proved to be the key-factor in governing the geomechanical properties of the granitic rock mass. The degree of weathering was classified according to the scheme proposed by the Geological Society of London (1995), which qualitatively distinguishes six classes ranging from sound rock (W1) to residual soil (W6). Through analyzing the associated geomechanical properties from laboratory tests, an appropriate process of re-classification of the degree of weathering was developed aiming at defining better the characteristic values of each class and reducing the overlap between classes. Figure 3 shows the original and final intervals as defined after the reclassification procedure. It is observed that new ranges are in a good agreement with the indications of Martin and Hencher (1986).

Table 1: Basic statistics of the uniaxial compressive strength, Co, of granite

Co [MPa]	Average	37.4
	Median	24.1
	St. deviation	34.4
	minimum	1.1
	maximum	146.4

Note: The statistical distribution of the 156 values can be described by a probability density function of negative exponential type

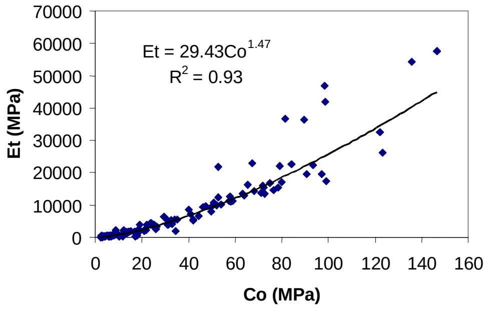

Figure 2. Correlation between Co and elasticity modulus, Et, as derived from uniaxial compressive tests

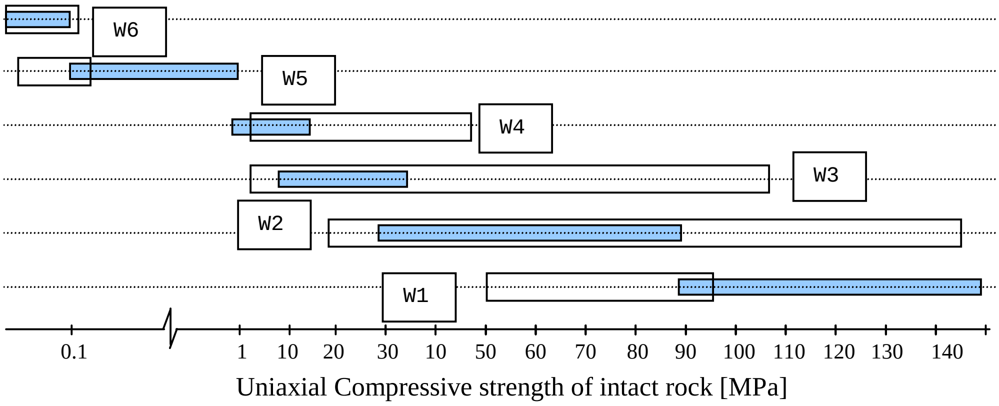

Figure 3. Weathering classes over the uniaxial compressive strength range (clear bars indicate classification based only on qualitative evaluation, shaded bars indicate re-classification after statistical analysis and consequent “cut-off”).

2.2 Rock Mass Characterization

Definition of the design geotechnical parameters consisted, in short, of the following phases:

- definition of alignment segments of homogeneous conditions in terms of weathering;
- statistical analysis of the geomechanical conditions (especially discontinuity density and condition) as encountered in the segments identified along the alignment of the same weathering class;
- definition of geomechanical groups of homogeneous conditions;
- determination of the Geological Strength Index (GSI, Hoek et al.1995, Hoek and Brown 1997); and
- derivation of the design parameters for each geomechanical group using reliable empirical correlations, results of the in situ tests and Monte Carlo simulation.

Statistical analysis of the borehole data brought to the definition of 7 geomechanical groups characterized by the conditions given in Table 2. These conditions are representative of the groups identified and are to be re-evaluated aiming for a narrower definition of geotechnical parameters for the design of a specific structure when in the vicinity of boreholes.

Table 2: Geomechanical groups and associated conditions

Geomechanical groups	Weathering degree (W) of intact rock[1]	Fracturing degree (f)[2]	Correlation [%] W-f	Discontinuity Condition[3]	GSI
g1	W1	f1-f2	80-85	d1-d2	65-85
g2	W2	f2-f3	80-85	d2-d3	45-65
g3	W3	f3-f4	70-75	d3-d4	30-45
g4	W4	f4-f5	65-70	d4-d5	15-30
g5	W5	(f5)	90-95	(d5)	(<20)
g6	W6	n.a.	-	n.a.	n.a.
g7[4]	n.a.[5]	n.a.	-	n.a.	n.a.

Note: [1] of intact rock strength given in Figure 3; [2] based on ISRM (1981) to which classes correspond the following (in cm) discontinuity spacing ranges: f1: >200, f2: 60-200, f3: 20-60, f4: 6-20, f5: <6; [3] classes of Surface conditions for "GSI-Based geomechanical Groups" (fig.1; Hoek, 1998) [4] the index is not applicable; [5] g7 engroups "man-made" material and alluvial soils.

Definition of the geotechnical properties of groups g1, g2, g3, and g4 was based on corrective factors, that permit the scaling down of intact rock properties to those of the rock mass. The GSI system was chosen to derive the rock mass properties, since it is a system based solely on intrinsic rock mass conditions and the result of a long-term research dating back to the early' 80s.

Groups g5, g6 and g7 refer to material with a soil-like behavior. Thus it is generally possibile to apply principles of soil mechanics in defining the geotechical parameters. As of this, the design values of the soil mass were based on sample properties, taking into account the results of the available in situ tests (SPT, etc.).

Deformation modulus was derived from empirical correlations and the results of the 136 Menard tests conducted in the boreholes. It is worth noting that the values of the pressiometric modulus showed significant variability when only associated with the weathering class. On the other hand, when the structure of the mass was considered, variability and discrepancies were significantly reduced.

2.3 Definition of the design parameters

Monte Carlo simulation, employing the latin hypercube sampling scheme, was used to define the following distributions of the geomechanical parameters of rock masses:

- Co: uniform distributions as defined from the values given in Figure 3;
- m_i: triangular distributions: g1(16,18,20); g2(14,16,18); g3 and g4(8,10,12);
- GSI: symmetric triangular distributions limited by the values given in Table 2.

The derived values are presented in Table 3a, while, for completeness, the values corresponding to the groups with a soil-like behavior (g5, g6, and g7) are given in Table 3b. The resulting shear strength envelopes are shown in Figure 4.

Table 3a: Design values of geotechnical parameters for rock mass groups

Geomechanical Group	Unit weight, γ (kN/m^3)	Hoek-Brown criterion parameters m_b	s	E_d (GPa)[(2)]
g1	25-27	7.45 (*1.15*)[(1)]	6.9E-2 (*3.2E-2*)	35.0 (*10*)
g2	25-27	3.2 (*0.5*)	7.5E-3 (*3.4E-3*)	10.7 (*3.0*)
g3	23-25	0.98 (*0.07*)	7.5E-4 (*1.7E-4*)	1.0 (*0.5*)
g4	22-24	0.67 (*0.12*)	0	0.4 (*0.2*)

Note: [(1)] The numbers presented are average values, while in brackets are given the standard deviations of the distributions as defined by the Monte Carlo method; [(2)] for groups g2-g4 the values are derived from Menard tests.

Table 3b: Geotechnical parameters design values for the groups with a soil-like behavior

Geomechanical Group	Rock mass unit weight, γ (kN/m^3)	Mohr Coulomb criterion parameters c'	ϕ'	E_d (GPa)
g5	19-21	0.01-0.05	32-36	0.05-0.20
g6	18-20	0-0.02	30-34	0.02-0.07
g7	18-20	0	27-29	<0.05

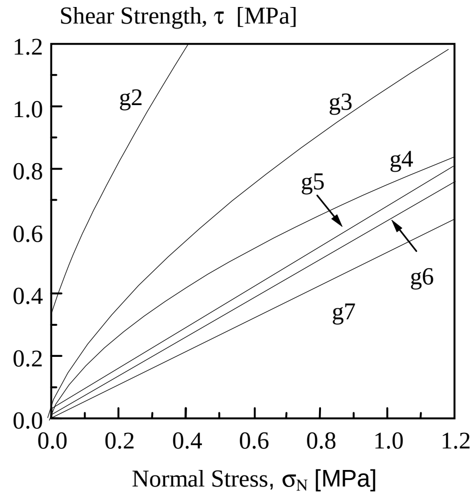

Figure 4. Shear strength envelopes for the different geotechnical groups

3. DESIGN ANALYSIS

Design analysis resulted in two types of pre-fabricated lining segments, both of which 30cm-thick, but with different steel reinforcement for sustaining the loads transmitted from the TBM advancement, which differ between open and closed modes of excavation.

This design was essentially based on numerical modeling of the various loading conditions including manufacturing, transportion, positioning of the segments as well as long-term loading conditions, which include complete mobilization of the lithostatic loads.

With reference to this last load type i.e., lithostatic loads, the procedure followed consisted of:

- Numerical simulation of the interaction between ground and structure using the Finite Difference Method (FDM) as implemented with FLAC software code (version 3.4

ITASCA). Parametric analyses were conducted simulating different geotechnical conditions (see example in Figure 5) that permitted to define:

- deformation of the opening function of the front distance;
- plastic zone development;
- loads evaluation;
- forces and moments in the lining;
- an indication of the surface settlement.

- Detailed structural analysis of the lining using the Hyperstatic Reaction Method by simulating all phases of segmental lining manufacturing, transportation, and positioning as well as long-term conditions that were based on the results of the FDM analysis.
- Cross check of the results of the previous phase by employing probabilistic methods, which allow to incorpate in the analysis the ground condition variability and residual uncertainties. In the following section this approach is synthetically described.

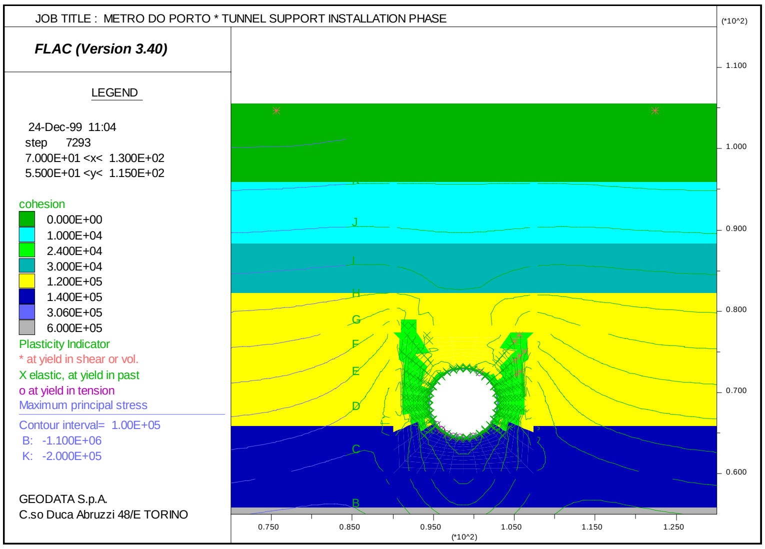

Figure 5. Example of the development of the plastic zone for a typical section as resulted from FLAC analysis

3.1 Verification of lithostatic loads by use of a probabilistic method

The parametric analyses conducted with FLAC demonstrated, as expected (Wong and Kaiser, 1991, in Barla, 1994) for k (horizontal-to-vertical in situ stress ratio) in the range of 0.35-0.50, that the shear strength of the ground is exceeded by the shear stresses in the two tunnel sidewall areas (Figure 5). This failure develops towards the surface as ground conditions deteriorate. It can be reasonably assumed that the ground enclosed by these failure surfaces directly loads the structure. This load is defined by the so-called rock load height (Ht) which is indicated by the analyses for the geomechanical groups:

- Rock masses (g1, g2, g3, g4): Ht ≤ 1D
- Soils (g5, g6, g7): Ht ≤ 1.5D

where D is the external diameter of the lining equal to 8.4m.

In order to verify the correctness of these threshold values, Ht was calculated with empirical equations by applying the probabilistic method (section 2.3). The relevant equations proposed by Unal and Terzaghi were used (Bieniawski, 1984 and AFTES, 1993, respectively) for geomechanical groups:

- rock masses: $Ht=[(100-RMR)/100]D$ (1)

- soils: $Ht=\{B(\gamma-2c/B)[1-\exp(-2H\tan\varphi/B)]\}/(2\gamma\tan\varphi)$ (2)

where:

RMR = Rock Mass Rating (Bieniawski, 1984);

B = base width of the loading area equal to $D\tan(3\pi/8-\varphi/4)$ in the case of circular section;

H= overburden height.

It is worth noting that equation (1), has also been verified by the results of analysis conducted using the discrete element method (Barla, 1994), where the medium is modelled as a discontinuum, for H>2.5D.

The RMR values were derived from GSI using the following equation (Hoek et al., 1995):

$$RMR = GSI + r_w + r_o - 10 \quad (3)$$

where r_w and r_o are, respectively, the ratings assigned to the groundwater parameter and the adjustment for the discontinuity orientation conditions as defined by the RMR classification scheme. Triangular probability density functions were used to describe the ratings of these parameters defined by (0,4,15) and (-12,-5,0) for r_w and r_o, respectively. The other variables were modeled using uniform distributions defined by the values given in Tables 3a and 3b. The maximum overburden considered in the analysis, for all groups (except g7), was 35m (for g7, the corresponding value was 25m).

Table 4 illustrates characteristic values of the Ht statistical distributions as resulted from Monte Carlo simulation, while Figure 6 shows, as an example, the histogram of Ht for g4.

Table 4: Characteristic values of the Ht distributions as resulted from Monte Carlo simulation for the different geomechanical groups

Geomechanical group	min [m]	5% percentile [m]	average [m]	95% percentile [m]	max [m]
g1	1.2	1.9	2.9	3.8	4.4
g2	3.0	3.7	4.5	5.4	6.0
g3	4.5	5.3	6.1	6.9	7.5
g4	5.9	6.5	7.3	8.1	8.7
g5	5.8	6.1	7.8	8.8	9.6
g6	8.5	8.4	10.1	10.3	11.5
g7	11.2	11.3	11.6	12.0	12.1

It can be observed that the results of the statistical simulation confirm the adequacy of design values defined for Ht as derived from FLAC analyses. The Ht threshold values considered (for rock masses: 1D=8.4m; and for soils: 1.5D=12.6m) are, in fact, for practical purposes equal to the worst possible conditions. Only, in the case of g4 the maximum value is slightly higher than the corresponding limit of 8.4m. The 8.7m value corresponds to the worst geomechanical conditions under the maximum overburden which are conditions associated with a probability of occurence of less than 0.005. But even in this case, the load increment corresponding to 0.3m, is negligible for the lining structure.

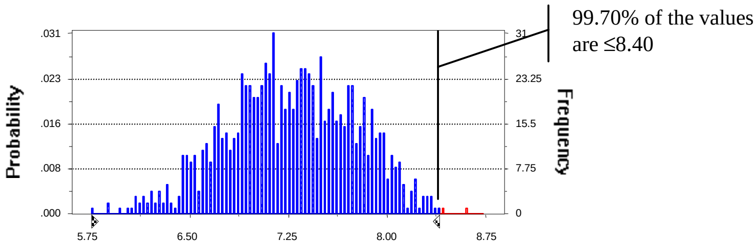

Figure 6: Distribution of the rock load height, Ht [m], for g4

4. CONCLUSIONS

The design of the underground works of the Metro of Porto, under construction in a densely populated city, presented considerable design difficulties due to adverse and highly variable geotechnical conditions. These conditions required the selection of equipment, structures, and design procedures adequate for the demanding geotechnical conditions under the constraints imposed by the environment.

In this context, the role of geomechanical characterization is crucial since it forms the basis of all design and construction decisions. The characterization procedure used was based on re-classification and statistical analyses of the in situ and laboratory test results and brought to the definition of 7 geomechanical groups ranging from massive to very weathered granite with a soil-like behavior.

Probabilistic analyses combined with the empirical approach to design were used to cross-check the results of the numerical analyses conducted for the design of the lining. It was shown that the threshold values defined for the rock load height by the FDM are compatible with the values derived by the empirical approach.

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