Site characterization and rock testing for the evaluation of design parameters

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ABSTRACT: This paper deals with issues related to the evaluation of design parameters from some types of in situ and laboratory tests performed, during the geotechnical investigations, to characterize the most relevant geotechnical properties of the rock mass and also the actions. Some considerations regarding safety evaluation and limit state design are presented. Several types of Rock Mechanics tests are introduced in some detail, and a few application examples covering different Rock Engineering projects are presented to show how test results can be analysed to determine their values to be used in the design. The following examples are shown: triaxial tests to determine the strength of the intact rock, a joint network study for a dam foundation, laboratory shear tests to assess the shear strength of rock joints, flat jack and overcoring tests to estimate the in situ state of stress for an underground cavern, and borehole dilatometer and large flat jack tests to evaluate the deformability of a concrete dam foundation.

1 INTRODUCTION

Large projects involving a significant amount of Rock Engineering works, such as underground caverns, tunnels, high slopes or concrete dam foundations, require particular design approaches to evaluate their safety conditions. In order to assist in the design of these types of works, flow charts with the main steps to be taken have been put forward by several authors (Hoek & Bray 1977, Hoek & Brown 1980a, Bieniawski 1992, Kaiser 1993, Hudson & Feng 2006, Palmstron & Stille 2007). Traditionally, design aims at identifying the instability sources or the failure scenarios and showing that the available resistance forces are larger than the acting forces by a given amount quantified by an empirical safety factor. In some cases, excessive displacements are also taken into consideration.

An important issue concerns the approaches used to define safety. Traditionally, in Rock Engineering, the most used approach considers the definition of the geometry and of the parameters, leading to the evaluation of a global safety factor. For each type of structure, based on the value of this factor, and according to the designer's background and experience, a statement about safety can be issued.

Eurocode 7 "Geotechnical design – Part 1: General rules" (CEN/TC 250 2004) introduced in Geotechnical Engineering a limit state approach to safety analysis, in the same way as performed in the other structural Eurocodes. Eurocode 7 presents a unified framework for the design of geotechnical structures that allows assessing the safety conditions of the ground and of the structural elements in an integrated and coherent manner. A semi-probabilistic approach is used, which uses rules to introduce safety in different ways: by using representative values of the actions and of the strength parameters, by using partial safety factors that affect them, and by including safety margins in the calculation models.

Though Eurocode 7 is intended to be applied just to the geotechnical aspects of the design of common buildings and civil engineering works, it defines Principles and Application Rules that can be brought into play to establish a comprehensive background to define the basis of geotechnical design of any kind of structure, including the specific works included in Rock Engineering projects, such as underground caverns, tunnels, slopes, and dam or large bridge foundations.

Using the Eurocode framework and logic may allow a uniform approach to the design of geotechnical structures, particularly in the cases when the ground is responsible for the actions on the concrete or steel support components or when the structures and the ground have to interact, as, for instance, in the case of the concrete lining of a tunnel.

Figure 1 shows a flow chart of the Rock Engineering design and construction process (Lamas & Muralha 2007), which is consistent with the limit states' perspective introduced by Eurocode 7.

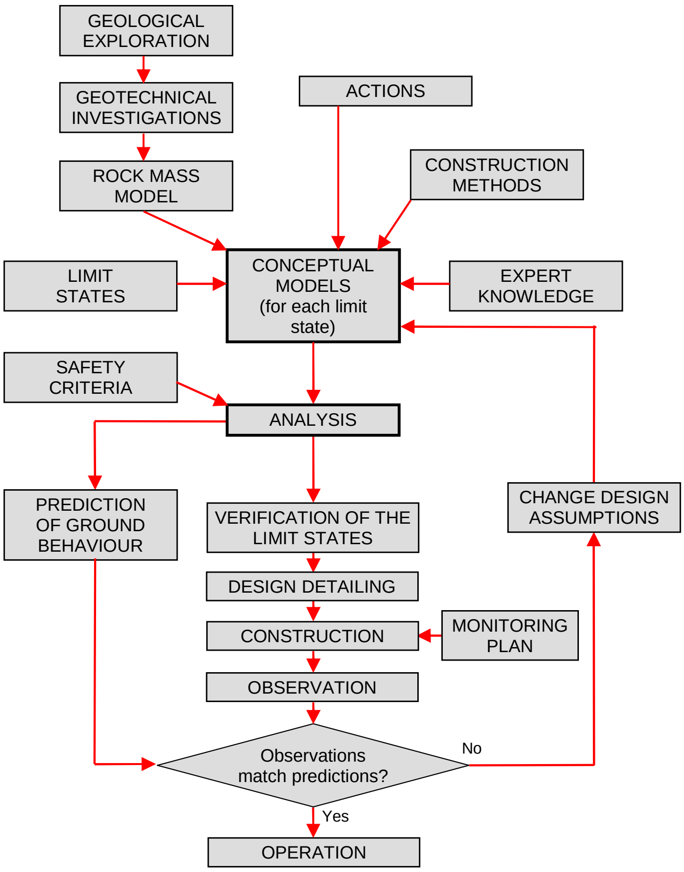

Figure 1. Flow chart of the Rock Engineering design and construction process.

To establish geotechnical design requirements, Eurocode 7 introduces three Geotechnical Categories, to be defined prior to the geotechnical investigations. Geotechnical Category 1, including just relatively simple structures, is not relevant for large Rock Engineering projects. Geotechnical Category 2 takes account of conventional types of structures and foundations with no exceptional risk or difficult soil or loading conditions. Some examples of structures or parts of structures complying with Geotechnical Category 2, such as walls and other structures retaining or supporting soil or water, excavations, ground anchors and other tie-back systems, or tunnels in hard, non-fractured rock and not subjected to special water tightness or other requirements are more likely to be encountered in large projects. Generally, important Rock Engineering projects fall into Geotechnical Category 3, which includes very large or unusual structures and structures involving abnormal risks, or unusual or exceptionally difficult ground or loading conditions.

Eurocode 7 starts by establishing that in all geotechnical design situations it should be verified that no relevant limit state is exceeded. Furthermore, it specifies that limit states should be verified by use of calculations, adoption of prescriptive measures, experimental models and load tests, or observational methods.

The properties of soils, rocks and other materials, along with the actions and geometrical data, are considered if geotechnical design by calculation is performed. Concerning the ground properties of rock masses, Eurocode 7 states that the values of the geotechnical parameters used for design calculations should be obtained from test results, either directly or through correlation, theory or empiricism, and from other relevant data. It also refers that those values should be interpreted according to the limit state considered, and to the differences between test settings and to the aspects governing the behaviour of the geotechnical structure.

Eurocode 7 determines that the selection of characteristic values for geotechnical parameters should be based on results and derived values from laboratory and field tests, complemented by well-established experience. The use of statistical methods to evaluate the characteristic values is not imposed.

Eurocode 7 requires characteristic values to be selected as cautious estimates of the parameters affecting the occurrence of the limit state. Therefore, the characteristic value depends on the extents of the zone of ground that affects the behaviour of the geotechnical structure. If the limit state involves a failure along a large ground volume, a cautious estimate may be a selection of the mean value at a confidence level of 95%. On the other hand, when local failure is concerned, a cautious estimate may be a 5% fractile (value with a 95% probability of being exceeded).

Finally, design values are determined from the characteristic values using partial safety factors, or may be assessed directly. For most of the parameters required in large Rock Engineering projects, partial safety factors have not yet been defined.

In the last couple of decades, the improvements in computer capacities and commercial software have allowed nearly everybody to perform the most complex safety analyses, literally at the tip of a finger. Constitutive geomaterial models have also evolved and they sometimes require several parameters that are not easy to evaluate. Defining the relevant limit states of a given project and establishing the respective adequate conceptual models is a crucial task. Contribution of experienced designers for this task is essential.

Site characterization and rock testing, especially in situ tests, have not experienced similar progresses. The common types of tests and their governing principles are the same as 50 years ago. This situation is particularly felt by in situ tests. Usual field conditions, involving dust, rain or high humidity, and often no electrical power, can explain why no considerable developments have been seen in the topic of in situ characterization of rock masses.

The fields, where evolution in site characterization techniques is more relevant, are electronics and miniaturization, image scanning and processing, data acquisition and graphical presentation, and, very importantly, test standardization and implementation of quality systems in testing laboratories.

Particular attention should therefore be paid to the path that starts with the results of the site characterization and tests performed during the geological exploration and the geotechnical investigations, and ends at the correct estimation of the parameters required to perform safety evaluation analyses.

This paper deals with the issues relating with the evaluation of design parameters from some common in situ and laboratory tests performed during geotechnical investigations to characterize the most relevant geotechnical properties of the rock mass and also the actions.

Several types of Rock Mechanics tests will be presented in some detail, and a few application examples covering different Rock Engineering projects are used to show how test results are analysed to determine parameter values for the design. The following examples are shown: triaxial tests to determine the strength of the intact rock, a joint network study for a dam foundation, laboratory shear tests to assess the shear strength of rock joints, flat jack and overcoring tests to estimate the in situ state of stress for an underground cavern, and borehole dilatometer and large flat jack tests to evaluate the deformability of a concrete dam foundation.

2 INTACT ROCK STRENGTH

To describe the strength of intact rock under triaxial conditions, the most frequently used models are the well-known Mohr-Coulomb and Hoek-Brown criteria. The Mohr-Coulomb criterion can be appropriately used to model the relation between the principal stresses at failure using a linear relation, as long as small ranges of the confining stresses are involved. Experimental results show that the failure envelopes of several different rock types were not linear. Based on both theoretical and experimental aspects of rock behaviour, Hoek & Brown (1980a, 1980b) developed a well-known non-linear relationship between the principal stresses at failure.

For a given rock type, this relationship is characterized by the Hoek-Brown parameters m_i and σ_{ci} (uniaxial compression strength), where the index i stands for intact rock. Along with GSI, they play a relevant role in the definition of the generalized Hoek-Brown criterion for the strength and deformability of jointed rock masses. The Geological Strength Index classification GSI (Hoek 1994, Hoek et al 1995, Hoek & Brown 1997) is a user friendly widespread methodology to assess both strength and deformability characteristics of rock masses relevant to perform many types of safety analyses with very different complexity levels. This fact leads to its wide acceptance and to the increasing relevance of laboratory triaxial tests.

The essential components of a triaxial test comprise a triaxial cell connected to a confining pressure and an axial load system. Due to the high pressures required, some triaxial cells tend to be heavy and difficult to handle. Hoek & Franklin (1968) developed a simple cell (commonly referred to as Hoek triaxial cell) that only applies the confining pressure, and can be used with a conventional compression testing machine to apply axial force to the specimen. The main advantage of the Hoek cell is that it does not require complex and time consuming preparation for assembling and dismantling the cell between tests.

Based on the same principle, other triaxial cells enabling axial and diametral displacement measurements were designed. Figure 2 shows a cell developed by Robertson Geolloging and used at LNEC for testing 54 mm diameter and 130 mm long rock specimens, with the axial displacement transducers (left) and the diametral displacement transducers connected to the rubber sleeve (right).

Figure 2. Triaxial cell and internal sleeve with diametral displacements transducers.

Figure 3 displays an example of a triaxial test with measurement of axial and diametral displacements, converted into axial and transversal strains (Gobbi 2009). It refers to a triaxial test of a volcanic breccia that started with a series of loading-unloading cycles under different confining stresses, namely 2, 5 and 10 MPa, followed by another cycle under 2 MPa, until peak strength. The test continued further beyond peak strength and a final loading-unloading cycle was still carried out.

Values for m_i and σ_{ci}, which can be used as initial estimates, are available for almost all types of rocks (Hoek 2007). However, important projects require specific triaxial tests to be performed to determine the real values.

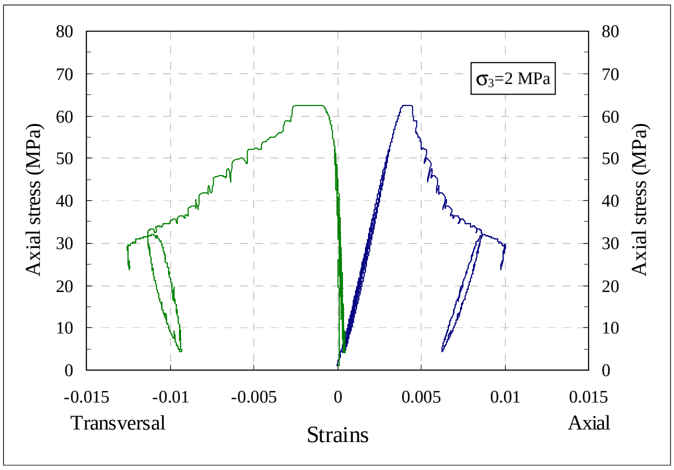

Figure 3. Results of a triaxial test with measurement of axial and transversal strains.

For this purpose, a statistically significant set of triaxial tests should be performed under confining stresses that cover the expected range of stresses. Tests should preferably be performed under a limited set of confining pressures following approximately a geometric sequence, for instance, 1, 2, 5 and 10 MPa. Tests with no confining stress (uniaxial compression) should also be carried out. This allows to define a mean value for each confining pressure, and consequently to calculate mean values for the parameters.

In order to assess parameter variability, groups of rock samples to be tested under all the confining stresses, including σ_3=0, should be prepared from a single long enough homogeneous rock core. This practice aims at reducing the number of tests required to perform simple statistical analyses, and allows determining the $m_{i95\%}$ and $\sigma_{ci95\%}$ values that define envelopes with 95% probability of being exceeded. Figure 4 presents an application example of this principle to a set of triaxial tests on gneissic granite (Muralha 2008). The severe reduction of the 95% envelope is a consequence of the high dispersion of the test results that display a coefficient of variation of 27%.

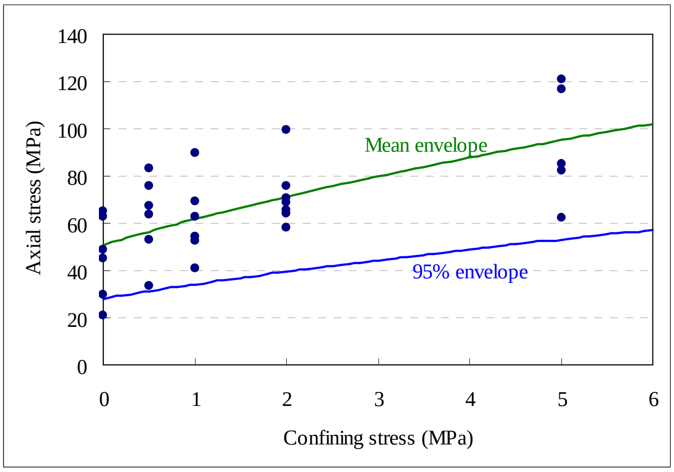

Figure 4. Results from a set of triaxial tests of gneissic granite.

The results presented by Franklin & Hoek (1970) of tests performed at Imperial College were used to assess the dispersion of the results and its consequences, since they considered sets of tests with a large amount of samples N. The values of σ_{ci} and m_i for the mean envelope were calculated by non-linear regression of the Hoek-Brown equation. Then, values of the coefficients of variation C_v of the ratios between the experimental and calculated values were determined. These values, ranging from around 6 to 15%, enabled to evaluate the 95% characteristic parameters of the Hoek-Brown envelopes. The main results of this analysis are presented in Table 1.

Table 1. Analysis of the results by Franklin & Hoek (1970).

Rock name	N	σ_{ci} (MPa)	m_i	C_v (%)	$\sigma_{ci95\%}$ (MPa)	$m_{i95\%}$
Granite (Devon)	39	216.6	19.9	7.10	192.1	17.0
Limestone (Block 1)	29	92.5	7.36	10.27	77.7	5.37
Limestone (Block 2)	33	51.4	7.04	14.66	40.7	4.03
Sandstone (Derbysh.)	21	61.8	15.9	8.10	54.7	12.6
Dolerite (Northumb.)	24	288.9	13.8	5.83	261.5	12.2
Sandstone(DarleyDale)	23	90.0	14.0	12.94	72.6	9.60
Marble (Carrara)	12	90.6	8.46	5.62	82.6	7.21
Sandstone (Pennant)	29	206.4	12.3	3.66	194.2	11.28

Some sets of tests presented several results for uniaxial compression tests (σ_3=0). This investigation considered just the average of those values, and some outlier values were discarded.

The previous examples show that significant reductions are to be expected for the 95% strength envelopes for coefficients of variation in excess of 15%, which are values that can be frequently found in sets of tests. However, as mentioned above, characteristic values have to be assigned in accordance with the way the limit state develops. In the case of intact rock strength, it may lead to consider a cautious estimate of the mean value, since failure surfaces in rock masses can be very large

3 JOINT NETWORK STUDIES

Joint network studies are inevitable in geotechnical projects involving rock masses, whether it is a tunnel, an underground cavern or a dam. The study of the joint network of a dam foundation rock mass will be presented. The general methodology to evaluate the relevant parameters will be pointed out, using the results of a study performed in a schistous rock mass for a 95 m high concrete dam as an example (Muralha & Grossmann 1994).

As for all other rock mass characteristics, the determination of the jointing parameters also requires some kind of sampling. The planning of a joint network study starts with the definition of the locations where joint parameters are to be measured. This sampling should try to be statistically uniform. Sampling locations should cover the whole rock mass in a uniform manner; in other words, each point of the rock mass should have the same probability of being considered as a sampling location and sampling should also cover all attitudes in a uniform way in order to minimize the effects of sampling bias.

Physical limitations to the choice of the sampling locations almost always make it impossible to obtain a uniform sampling of the jointing. These difficulties increase when rock exposures are scarce and there is just limited access to the interior of the rock mass, in particular to the zones where the works will occur.

To correct the sampling bias related to the lack of uniformity in the covering of all attitudes, several statistical techniques have been developed (Priest & Hudson 1981, Grossmann 1984, Priest 1993). Basically, the probability the considered sampling would detect a joint with a given attitude, if the attitudes of all joints in the rock mass followed a uniform distribution is used.

The most common observation surfaces for a joint survey correspond to geometrically well defined domains: boreholes, scanlines and plane surfaces (circles or rectangles). Since the determination of the attitude of joints along boreholes is only possible using TV cameras or if the core is oriented, in most cases scanlines or plane observation surfaces are used.

For a dam foundation, the joint survey should try to sweep the whole valley cross-section, i.e., the upper, middle and lower parts of both banks. In each of these zones, sampling should look not only at the dam foundation area, but also at the adjacent downstream and upstream areas, covering the rock mass volumes around the dam pressure bulb and the grouting and drainage curtains. Furthermore, if adits and/or boreholes are available, the sampling should also cover the evolution of the jointing from the surface to the interior of the rock mass. However, as explosives are used to execute the adits, care should be taken with blast induced joints.

As a result of the application of these general principles, the jointing study for an ordinary dam foundation can easily comprise up to 50 sampling locations. Due to the variability of the jointing parameters, around 30 joints should be sampled in every sampling surface. Consequently, joint network studies often sample in excess of 1000 joints. Therefore, thousands of pieces of information have to be collected during the field survey, usually under difficult conditions. Therefore, joint sampling is a demanding and time consuming task.

At each location, measurements should include all joints intersecting the chosen observation surface. Joints with small intersections (trace lengths) should not be disregard. It is also important to record the characteristics of each joint and not to consider that a certain joint is like the previous one, which is common practice for schistosity or foliation planes.

The characteristics to be registered for each joint are the geologic type, the attitude (strike and dip), the length of the intersection with the observation surface, which may not be the fully visible, as sometimes the discontinuity surface extends beyond the limits of the observation surface, the mean aperture and the type of its filling. Other features, as the depth of occurrence (if the observation surface is a borehole wall or core) and the relation with other joints that are cut by or end at may also be recorded.

In the case of the dam foundation example, not all of the 12 existing adits (6 in each bank) were selected for the joint network study. On the left bank, adits GE1.1 in the upper part of the slope and GE6.1 in the lower were chosen, whereas in the middle section adits GE3.1 and GE4.1 were selected (top of Figure 5). This option was taken because the geophysical survey revealed the occurrence of a weaker zone around mid-height of the margin and, furthermore, those 2 adits were not very long. On the right bank, 3 adits were picked: GD6.1 in the lower part, GD4.1 in the middle and GD2.1 in the upper part of the bank (bottom of Figure 5). The same figure reveals the location of the 40 (20+20) observation zones on the walls of the adits and shows that they are placed along a wide range of directions rendering a uniform spatial covering.

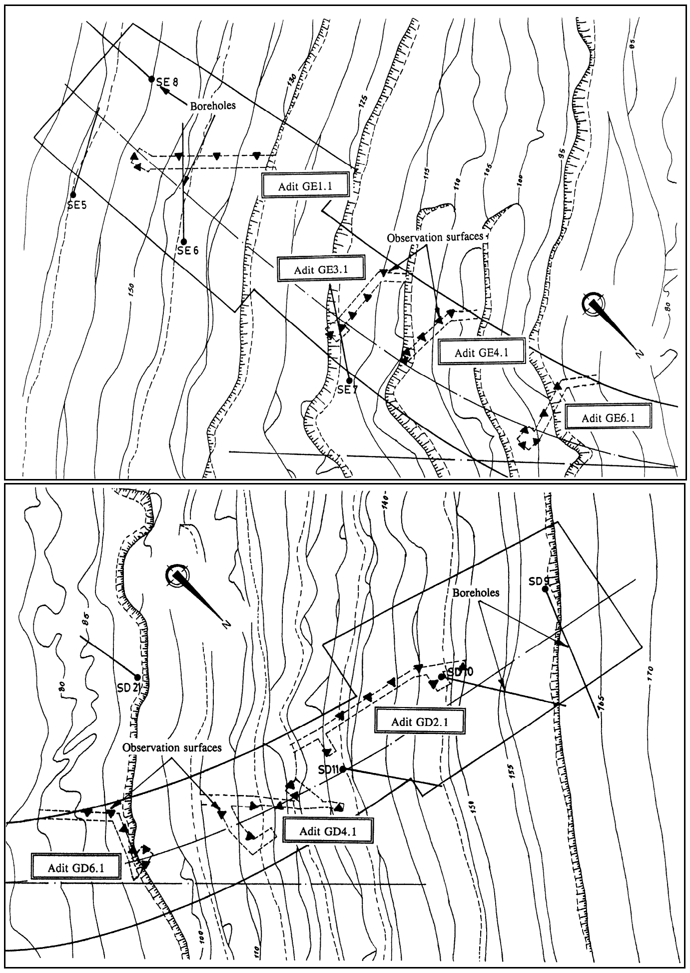

Figure 5. Location of the observation surfaces in the adits of both margins.

At this dam location, all observation surfaces were rectangles. The size of the majority of them was 3.75 m^2 (2.5m×1.5m). The joint survey covered a total area of 144.25 m^2 (70.5 m^2 on the left bank and 73.75 m^2 on the right). In comparison with the whole volume of the dam foundation, the rock mass volume sampled for the jointing study was extremely small (less than 0,1%). Even so, this particular study provided the description of more than 1800 joints.

The principal joint sets of the rock mass were determined using the following methodology proposed by Grossmann (1977).

It starts with the study of each one of the observation surfaces, for which it may be assumed that they are homogeneous as regards the jointing; at this stage, the joint subsets of each zone are defined, and no joint is discarded, even if it does not fall into the most important subsets; the study establishes the mean attitudes of the joint subsets based on statistical techniques and equal area projections. Afterwards, larger zones in the rock mass for which it still may be assumed that they are homogeneous are defined; to study those larger rock mass zones, the corresponding discontinuity sets are obtained by grouping the discontinuity subsets of the small zones, and not directly from the data collected at the sampling locations. The previous step may be repeated using now larger zones gathering still homogeneous rock mass volumes. The homogeneity assumption has to be checked each time the sampling volume is enlarged. The advantage of this procedure is that no conditions of equal representativeness, such as a similar sample size or sampling quality, have to be imposed to the different sampling locations; still, at this stage, no joint is discarded, even if it does not belong to the more significant joint sets. Finally, after defining the joint sets of the whole rock mass, or of its homogeneous parts, the parameters (attitude, intensity, area, aperture, etc.) of all occurring joint sets are calculated.

For the current case, the first step of the joint system study was the definition of the joint subsets for all 40 observation surfaces starting with the usual equal density stereographic plots (Figure 6). Then, in a second stage, the resulting subsets were grouped for each adit; in a third step, the resulting joint sets were grouped for both banks; and finally, the sets obtained for each bank were grouped for the whole rock mass.

The different geometrical parameters of the joint sets have to be described in a statistical manner, as they usually show some dispersion.

The distribution of the attitudes of a joint set can be modelled by a bivariate normal distribution on the tangent plane at the mean attitude (Grossmann 1985). This distribution requires five different parameters for its definition, namely, the strike and the dip of the mean attitude, the maximum and the minimum standard deviations, and the angle that identifies the orientation of the maximum dispersion, since it is not likely to be oriented along the strike or the dip.

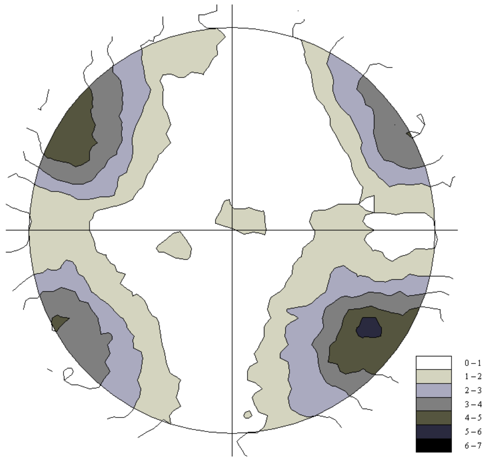

Figure 6. Example of an equal density stereographic plot for a joint study.

The intensity of a joint set describes the degree of jointing that it induces in the rock mass, independently of the individual extent of each discontinuity surface. So, the intensity is quantified by the sum of the areas of the discontinuity surfaces of the set which occur in a unit volume of the rock mass, and its units should be m^2/m^3. Under certain particular conditions, the intensity is the inverse of the spacing. The intensity or the spacing of a joint set, along with the area, are very important parameters, as they determine the persistence and the extent of the joints. Ordinary computer programmes do not take in consideration these parameters, and the persistence of joint sets is only qualitatively described, for instance, stating that the trace length ranges between 2 and 4 m.

The results of the joint network study revealed that the most important joint set presented a mean attitude of N66°W; 74°NE. It corresponds to the schistosity planes and will be referred to as X. This joint set is responsible for around 50% of the total jointing in the whole rock mass. With a 25% contribution to the jointing, discontinuity set V with a mean attitude N7°E; 83°NE is the second most relevant. Only these 2 joint sets occur in the whole rock mass. With a lower degree of relevance, several sub-horizontal discontinuity sets were also detected on each bank. On the left bank, a sub-horizontal joint set (mean attitude N76°E; 34°NW) accounts for about 25% of the jointing in that slope. On the right bank, the horizontal joints are even less important and the joint set with the mean attitude N16°E; 22°SE is responsible for only 7% of the jointing in that slope. The sub-horizontal sets will be globally referred to as H.

The parameters of the statistical distributions of the attitude, intensity and area of joint sets X, V and H are presented in Table 2.

Table 2. Parameters of the most relevant joint sets.

Joint set	Strike (°)	Dip (°)	Intensity (m^2/m^3)	Area (m^2)
X	294	74 NE	5.4	1.4
V	17	83 SE	2.8	1.0
H (left bank)	256	34 NW	2.6	2.2
H (right bank)	16	22 SE	0.9	1.5

Geometrical data referred to in Eurocode 7 are the level and slope of the ground surface and of the interfaces between strata, the water levels, the excavation levels, and the dimensions of geotechnical structures. The geometrical parameters of the joint sets are not mentioned, though they are very important to a number of limit states that occur frequently in Rock Engineering projects. In the case of rock blocks falling from the roof of underground excavations, attitudes of the joint sets are crucial parameters to be considered in any safety analysis. In the case of blocks sliding from excavation walls or along natural slopes, joint set attitude is more important than the shear strength of the joints (Muralha & Trunk 1993).

Geometrical parameters of joint sets demand, not just the appraisal of the mean attitudes, but also the same judgement about the dispersion of the attitudes of the joint sets. In the case of sliding blocks, pole frequency plots (Figure 6) frequently show that the joint set dispersion is higher than 10º, which can make the whole difference in stability conditions, for instance, if schistosity surfaces dip in average 30º on a slope.

Regarding the occurrence of falling blocks, design is usually performed considering that the joints are so persistent that they extend from side to side of the excavation roof, and thus are able to form the largest block possible. Since the probability of occurrence of joints with large areas is low, the joint probability required for a large block to materialize is certainly very low, which renders the assumption of the largest block quite conservative. For these situations, if intensity and persistence properties of the joint sets are not considered, the design will tend to define heavy supports.

4 JOINT SHEAR STRENGTH

Certain projects require the design of geotechnical works in rock masses where stresses are relatively low when compared with the intact rock strength. In these cases, stability is structurally controlled by sliding of individual blocks on their limiting discontinuities (joints, bedding planes, shear zones, faults, and cleavage or foliation planes). The analysis of this type of limit state requires the estimation of the shear strength of the rock joints, which is usually performed by means of shear tests (Goodman 1976; Hoek & Brown 1977, Muralha 2007).

Preferably, in situ direct shear tests should be performed. However, they are time-consuming and expensive and therefore only very few can be performed, which makes it impossible to estimate the shear strength parameters with any statistical significance. To overcome this impracticality, it is preferable to execute a series of laboratory shear tests on joints from the same discontinuity set.

The basic principle of a joint shear tests is to subject a joint sample to various normal stresses and to determine the shear stresses required to cause a certain shear displacement (ISRM 1974; ASTM 1995). Since the purpose of these tests is to evaluate the relations between the shear and the normal stresses at failure, several different normal stresses have to be applied. If each joint could only be tested under a single normal stress, a large amount of joint specimens would have to be tested. So, particular multi-stage test procedures can be followed to evaluate the shear envelope from a single joint (Wittke 1990).

The shear test of a joint sample consists of a series slidings (usually 4 or 5) at different normal stresses, which are kept constant during each sliding. The first sliding, or shearing, takes place under the lowest normal stress and the following slidings are performed under the remaining normal stresses in an ascending order. So, each sliding is carried out under a normal stress larger than the previous. All slidings start with the two joint halves being reset in their mated original position. This practice minimizes the influence of successive repetitions that wear the joint surface breaking joint wall asperities and reduce roughness. To further decrease this inconvenient, all debris is carefully removed from the joint surface prior to each sliding.

Moreover, normal stresses are chosen as a geometric sequence over the range of stresses that are expected to be found in the project, for instance, as in the example of a shear test presented in Figure 7, where the results of a joint shear test are presented. Barton´s well known shear model (Barton 1973), with its log relation between JCS and the normal stress, seems to support this practice.

It is common practice to sample joints for shear tests from borehole cores. These type of samples present several drawbacks. First of all, frequently they display small areas (around 50 cm^2); secondly, they present oval shapes, in which the sample length is difficult to evaluate, and the border regions in the middle zone of the samples do not play any role in the mobilization of shear strength; finally, oval samples with lengths greater than 10 cm are not easy to obtain and so joint roughness may not be adequately represented

It is advisable to sample a statistically significant number of joint samples that enables the evaluation of characteristic values for the shear strength. This implies the determination of the mean shear strength and the respective standard deviation. The analysis of several groups of joint shear tests from different types of rocks showed that coefficients of variation higher than 30% are very common, and smaller values around 10% are only found for particular joint sets with very low roughness, such as schistosity or foliation planes (Muralha 1995).

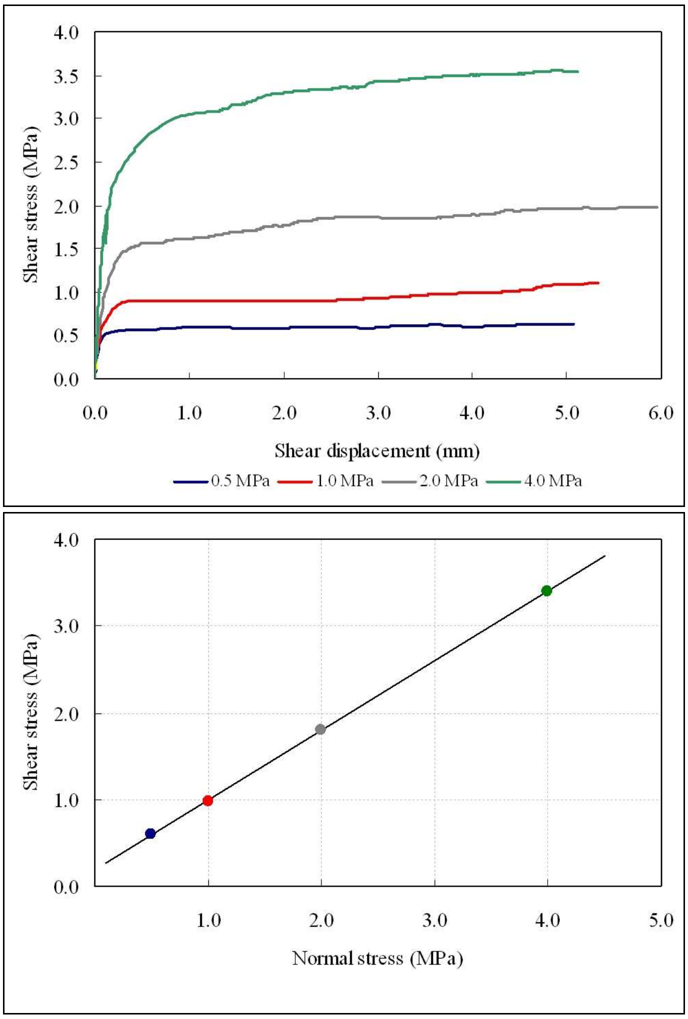

Figure 7. Results of a joint shear test.

If possible, laboratory tests should be performed to estimate the shear parameters for a given joint set, defined during the joint network study. This means that a number around 16-20 joint samples from each joint set should be collected in situ specifically for this purpose.

This strategy was followed in the site characterization studies for the dam presented as example for the joint network survey. In this case, a portable drilling rig was used inside the exploratory adits to extract 150 mm cores containing purposely chosen joint samples pertaining to the 3 major joint sets (X, V and H). This sampling procedure allowed to perform 54 laboratory shear tests (18 on discontinuities from the sub-vertical set X, 20 from the sub-vertical set V and 16 from the sub-horizontal sets referred to as H). The areas of the joints were around 200 cm^2, and the normal stresses applied during slidings were 0.5, 1.0, 1.5 and 2.0 MPa.

It is very important to carry out all tests under the same normal stresses σ_n, in order to perform simple statistical evaluations to determine the average shear strength τ and standard deviation s_τ for each group of slidings at the same normal stress. These values are presented in Table 3.

Table 3. Values of the average shear strength τ and respective standard deviation s_τ for each normal stress.

σ_n	Joint set X		Joint set V		Joint set H	
	τ	s_τ	τ	s_τ	τ	s_τ
0.5	0.489	0.094	0.405	0.073	0.537	0.119
1.0	0.875	0.129	0.748	0.145	0.952	0.194
1.5	1.241	0.168	1.083	0.197	1.346	0.264
2.0	1.598	0.216	1.405	0.262	1.730	0.347

Assuming that, for each normal stress, the shear strength of the joints from a given set follows a normal distribution, values with 50 and 95% of probability of being exceed are easily computed, enabling to define average and characteristic linear envelopes. Figure 8 presents this evaluation for joint set V, and Table 4 displays the values of the linear envelope parameters (apparent cohesion c and friction angle ϕ) for the average and 95% characteristic shear strength values of each joint set.

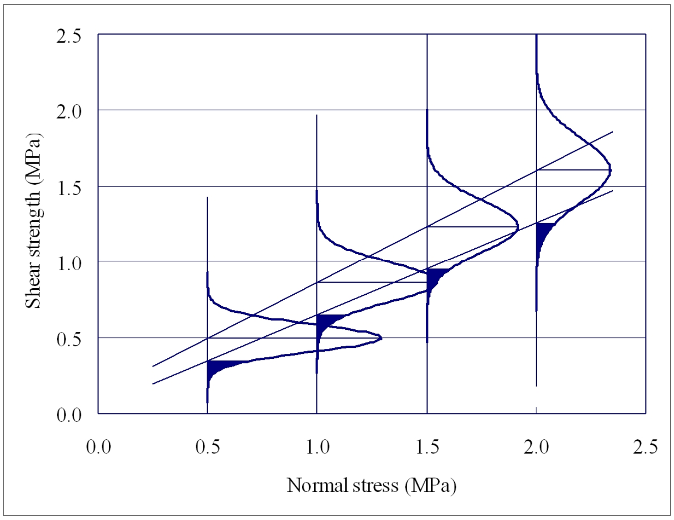

Figure 8. Average and 95% characteristic linear envelopes for the shear tests of joint set V.

Table 4. Values of the average and characteristic parameters for linear envelopes.

	Average parameters			95% characteristic values		
	X	V	H	X	V	H
c	0.128	0.077	0.148	0.044	0.053	0.078
tg ϕ	0.739	0.667	0.795	0.605	0.463	0.547
ϕ	36.4	33.7	38.5	31.2	24.9	28.7

Values in Table 4 show how the higher standard deviations of joint set H, which presents the largest average values of c and ϕ, leads to 95% characteristic values lower than those of joint set X.

Once again, defining characteristic values has to take into account the site specific features that caused the limit state to be analysed. If the failure mechanism involves very large joints, they can be defined as cautious estimates of the mean shear strength. As an example, if characteristic values were chosen for joint set V considering the lower bounds of the 95% confidence interval for the mean shear strength, the apparent cohesion and the friction angle would be 0.070 MPa and 31.3º, respectively.

5 IN SITU STRESSES

Release of the in situ stresses is often the most relevant action in underground projects. It can be stated that in situ stresses are the most elusive parameter to be determined, as all experimental techniques are not reproducible, and stresses are highly influenced by rock mass heterogeneities such as differing deformability zones and faults.

Several field methods are available to determine the in situ state of stress (Cornet 1993). As a case history, an example is presented, where overcoring and flat jack tests were both performed, and a specific methodology was used for global analysis of the test results, in order to obtain the most likely stress field (Sousa et al 1986, Muralha et al 2009).

The project is the repowering of a hydraulic scheme with a new powerhouse in an 80 m high and 30 m diameter shaft. The tests were carried out taking advantage of the existence of adits used during the excavation of the existing powerhouse (Figure 9). In one of these adits, two locations were selected; location 1 was situated at about the same height as the river bed, at a depth of 95 m and around 120 m from the river axis; location 2 was situated around 225 m from the river axis, at a depth of about 130 m and at a level 20 m higher than location 1. At location 1, three small flat jack tests (SFJ) were performed on an adit wall displaying an attitude approximately perpendicular to the river; at the same location, three overcoring tests with STT cells were performed inside a short borehole STT1, perpendicular to the adit wall and dipping 45°. At location 2, three flat jack and two overcoring tests were carried out. Borehole STT2 for the overcoring tests was also perpendicular to the wall and dipped also 45°. The orientations of the adit in locations 1 and 2 are approximately perpendicular.

Small flat jack tests, though very reliable, only allow determining the value of a single stress component. In both locations, the flat jack tests rendered vertical, horizontal and inclined (45°) stress components. On the other hand, the results of the overcoring tests provide the complete stress tensor (three normal and three shear stresses).

At the end, the test results were 6 (3 in each location) normal stress values and 5 stress tensors (3 in location 1 and 2 in location 2). The global analysis of these results requires a global interpretation model able to relate values obtained at different locations by different testing techniques. The results of the SFJ tests are presented in Table 5, and the results of the STT tests are presented in Table 6.

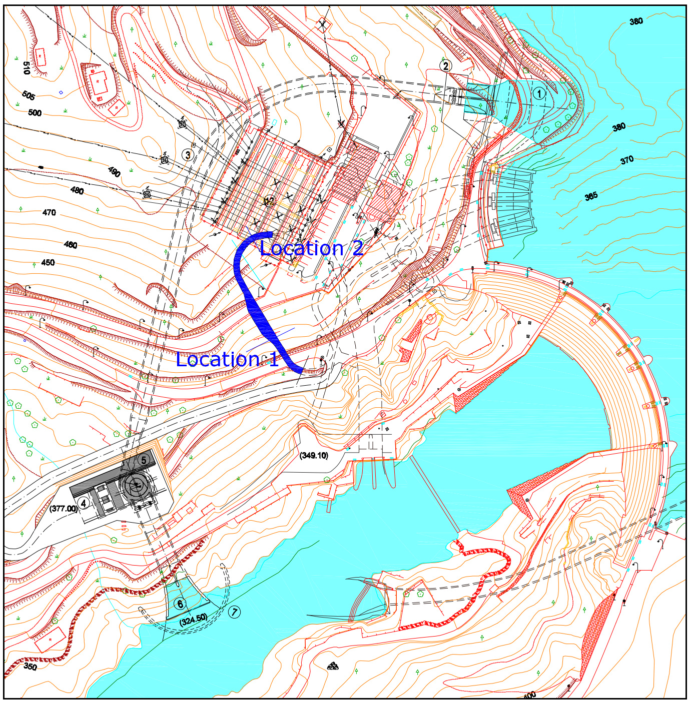

Figure 9. Location of the adits.

Table 5. Results of the small flat jack tests.

Location 1	Vertical stress	σ_v=3.46
	Horizontal stress	σ_h=3.61
	Inclined stress	$\sigma_{45º}$=1.85
Location 2	Vertical stress	σ_v=1.91
	Horizontal stress	σ_h=3.31
	Inclined stress	$\sigma_{45º}$=4.32

Table 6. Results of the overcoring tests.

Borehole	Depth (m)	Principal stresses (MPa)	Principal directions
STT1	12,55 m	1,97	143/18
		2,01	45/21
		6,21	269/62
STT1	14,85 m	0,34	6/26
		0,49	115/33
		4,58	246/45
STT1	17,15 m	2,38	97/42
		2,90	352/16
		7,51	246/44
STT2	5,35 m	2,21	252/7
		2,57	157/32
		3,43	352/57
STT2	9,60 m	1,38	175/40
		1,81	82/4
		7,40	348/49

To consider the influence of the geometric conditions in the overall state of stress, and to jointly interpret all the results from these tests, a global model is required. It has to reproduce the actual terrain topography, which resulted from the eroding action of the river throughout geological times. Furthermore, the tests results do not replicate directly the natural state of stress, since they are influenced by the proximity of the adit.

To begin with, a FLAC (Itasca 2005) 2D numerical model was used to represent the eroding action of the river. Figure 10 displays the model grid with the terrain topography before and after the river eroding effect. This figure also shows that both river banks were modelled, because terrain topography is not symmetric. This geometric feature did not allow reducing the size of the grid, taking advantage of symmetry conditions, and thus forced the grid to be more refined close to the river bank in the zone where the tests were performed.

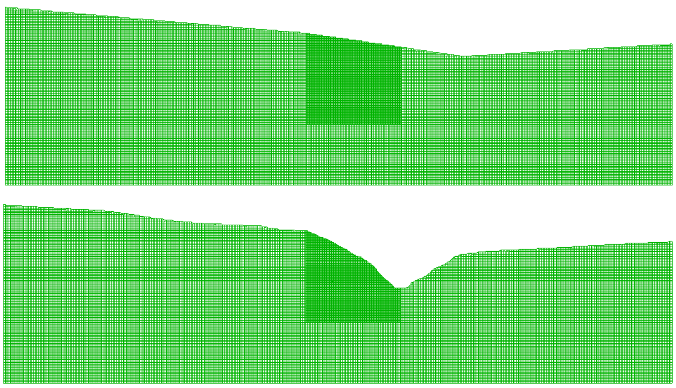

Figure 10. Numerical model (2D) with the terrain topography before and after the river eroding effect.

The following assumptions were considered in this model:

– plane strain conditions;

– the terrain prior and after erosion by the river is represented by the profiles of Figure 10;

– the initial vertical stress, σ_{ver}, prior to valley excavation, varies linearly with depth;

– the horizontal stresses σ_{hor} and σ_{nor}, respectively in the plane of the model and normal to it, prior to valley excavation, also vary linearly with depth;

– σ_{ver}, σ_{hor} and σ_{nor} are principal stresses.

Taking the unit weight of the rock mass γ equal to 27 kN/m^3 and h as the depth measured from the surface of the ground prior to river erosion, σ_{ver}, σ_{hor} and σ_{nor} are given by:

$$\sigma_{hor} = k_1\,\gamma\,h, \qquad \sigma_{ver} = k_2\,\gamma\,h, \qquad \sigma_{nor} = k_3\,\gamma\,h \qquad (1)$$

If each of these stress components is considered as acting separately, the following 3 loading cases E_i (i=1,2,3) are applied:

$$\begin{array}{llll} E_1 \rightarrow \sigma_{hor} = k_1\,\gamma\,h & \sigma_{ver} = 0 & \sigma_{nor} = 0 & \\ E_2 \rightarrow \sigma_{hor} = 0 & \sigma_{ver} = k_2\,\gamma\,h & \sigma_{nor} = 0 & (2) \\ E_3 \rightarrow \sigma_{hor} = 0 & \sigma_{ver} = 0 & \sigma_{nor} = k_3\,\gamma\,h & \end{array}$$

The components of the state of stress from the overcoring tests and the results of the stresses from the small flat jack tests in location 2 may be calculated for each individual loading case using the principle of superposition of effects. Still, in location 1 the state of stress is influenced by the adit opening, and so a second numerical model was developed.

It is a 100×100 m^2 3D model using FLAC^{3D} with a unit width, centred at the adit in location 1 (Itasca 2006). Grid blocks are 0.5×0.5×1 m^3 and the approximate shape of the adit is also modelled (Figure 11). The stresses applied to the boundary of this model were the stresses resulting from each one of the actions referred to in equation (3) at the same places. The outcome of the 3D model enables the calculation of the remaining stress components determined by the tests at location 1.

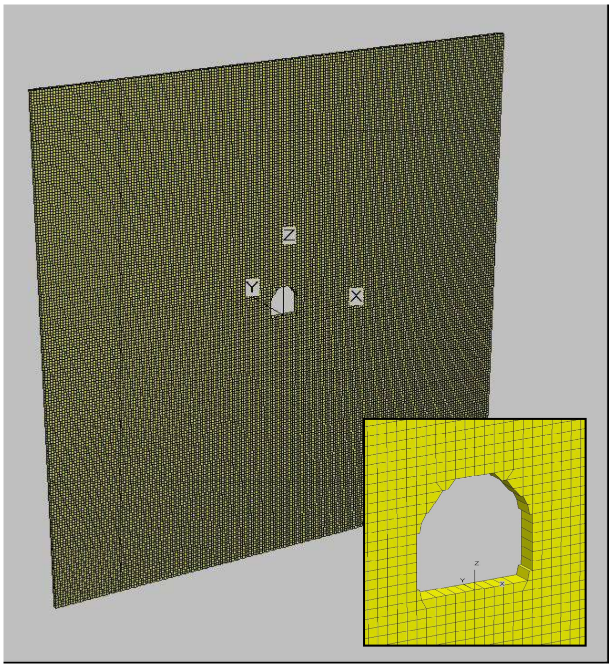

Figure 11. Numerical model (3D) with the adit near location 1.

The computation of the parameters k_1, k_2 and k_3 by the least squares method rendered the following values, respectively: 0.60, 0.91 and 0.75. With them, it is then possible, to estimate the state of stress at any location in the rock mass, namely around the shaft of the new powerhouse. This variation is presented in Figure 12, which displays the end result of the global interpretation model.

This example demonstrates the importance of the global interpretation model in the averaging of the results of any set of in situ stress tests. Without an interpretation model it would be hard to appraise the state of stress considering separately the results of each test. It should be stressed that planning the in situ stress tests compels the interpretation model to be already foreseen at that stage.

Sometimes, long and expensive boreholes are able to reach the rock mass around an underground excavation, but in other cases depth makes it unfeasible. These difficulties may be overcome by performing additional tests as soon as exploratory or access adits reach the excavation zone, namely flat jack tests, and in this way update the values of the stress field.

As in the example presented, the number of in situ tests performed during the site characterization stage to support the design is generally very scarce.

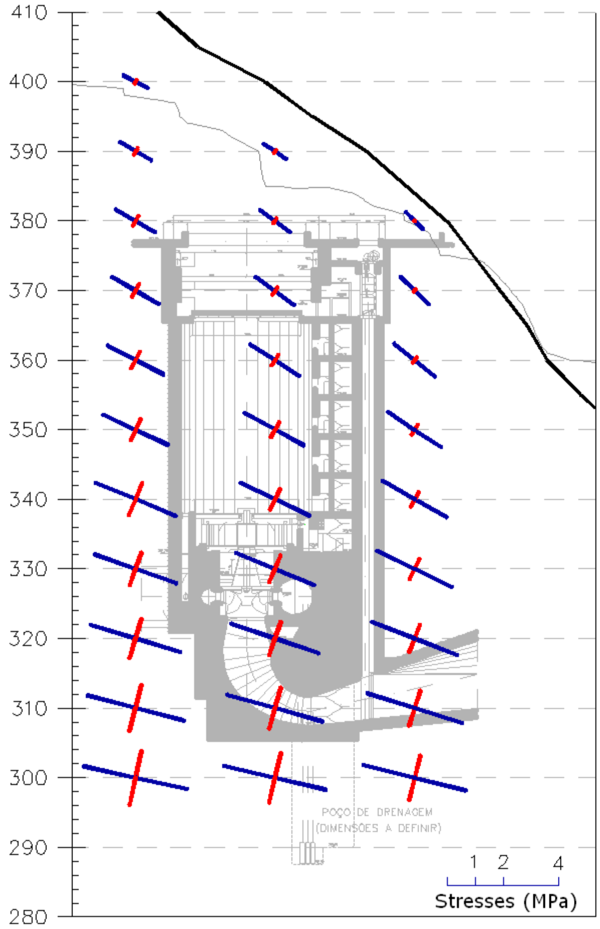

Figure 12. State of stress along the direction perpendicular to the river around the new powerhouse.

So, it does not allow making any statistical inference about stress variability. As a consequence, values of the in situ stresses to be used in design have to be carefully defined and it is advisable to use available mean results and to perform some judicious sensitivity analysis.

6 ROCK MASS DEFORMABILITY

Evaluation of rock mass deformability is usually made by in situ tests that apply loading-unloading cycles to establish its stress-strain behaviour. The most common tests are borehole expansion tests with flexible dilatometers or with borehole jacks, large flat jack tests and plate bearing tests (Wittke 1990). The development of these tests occurred since early Rock Mechanics years, and was driven by the geotechnical investigations for dam foundations. Field studies almost always include adits and boreholes, which enable performing tests in reachable rock mass zones and in inaccessible zones. An example of the application of large flat jacks (LFJ) and flexible dilatometer (BHD) tests will be presented (LNEC 1983).

Large flat jacks are reliable tests and have the considerable advantage of probing a large rock mass volume, around a few cubic meters, though they are time consuming and expensive, but cheaper than plate bearing tests for the same tested volumes and applied pressures. Dilatometer tests can be more widely used since they involve lighter equipment and so are less costly; however, explored rock mass volumes are much smaller, never beyond 0.2 m^3.

The issue of the rock mass volumes tested by both methods is directly related with the joint frequency and persistence, and with the representative elementary volume REV concept. The best approach to conciliate and optimize these advantages and drawbacks is to perform a few large flat jack tests in each river bank, to cover the whole foundations with dilatometer tests, and to correlate all the results gathered from both testing techniques. Results from large flat jacks tend to represent better rock mass deformability while dilatometer tests allow a statistical description.

Flat jacks are thin hydraulic jacks, 1.25 m long and 1 m wide, with a semicircular end, consisting of two 1 mm steel sheets welded along their contour. They are inserted in a 7-10 mm slot sawn in a rock mass exposure. After installing the flat jack inside the slot, pressure is applied inside the jack and the slot widening is measured by four displacement transducers placed inside each flat jack.

A LFJ test can be performed with a single jack or with arrangements of two or three parallel jacks, according to the rock mass REV. As a rule, tests are performed with two flat jacks.

Tests consist of a set of loading-unloading cycles to increasingly higher pressures. At the tip of the slot, tension stresses may appear during the tests if the pressure exceeds the in situ stress perpendicular to the slot plane plus the rock mass tensile strength.

Figure 13 presents the pressure versus average displacement plot of a LFJ test. At the beginning of the test, the rock mass displayed a stiffening behaviour resulting from the closure of rock joints that previously had widened due to the sawing of the slot. In the last loading-unloading cycle, the opposite occurred. The graph shows an increase of rock mass deformability, revealing that a tension crack developed at the tip of the slot. Subsequently, the test was completed with a final cycle up to the same pressure and with a 60 min creep stage.

Since positions of the displacement transducers are well defined, and assuming elastic and isotropic behaviour for the rock mass, the Young’s modulus may be easily calculated for any given stress variation, with the help of an interpretation model.

Figure 14 shows the evolution of the first loading modulus with the applied pressure. The increase of the moduli in the first cycles (up to 1 MPa) underlines the initial closure of joints in the rock mass; the moduli decrease in the last cycle (between 1.5 and 2 MPa) reveals the development of a tension crack.

Unloading and reloading moduli, also shown in Figure 14, are not affected by the initial joint closure, but the influence of the tension crack is noticeable. Average unloading-reloading modulus is often used to characterize the rock mass deformability.

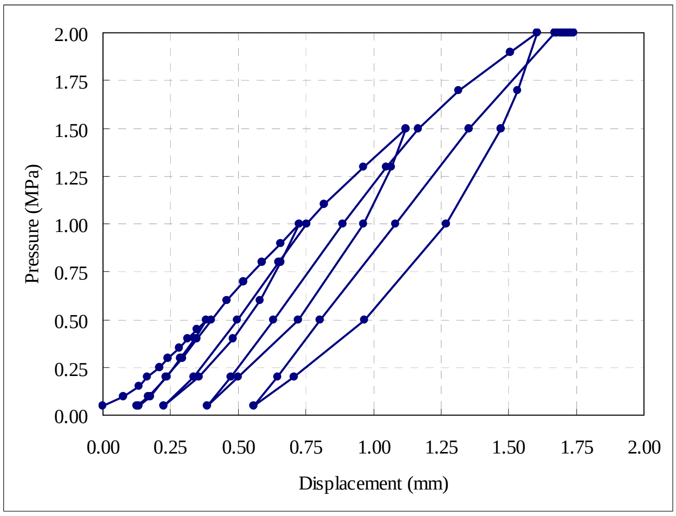

Figure 13. Pressure versus average displacement plot of a LFJ test.

Presentation of the results of a LFJ test as in Figure 14 allows designers to pick up the Young's modulus according to the stresses that are expected for the project.

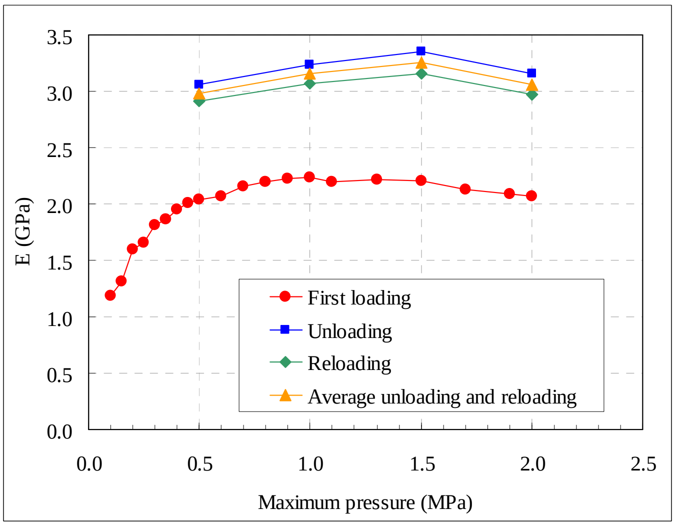

Figure 14. Young's moduli as a function of applied pressure.

Preparation of the LFJ tests is time consuming, so it seems important to take the maximum amount of results from them. So, it is advisable to perform a creep phase at the end of the regular test (Figure 13). Simple interpretations of the creep test results can determine if the rock mass displays significant long-term deformations that might be a concern for the design, in which case they should be taken into consideration.

BHD flexible dilatometer is an apparatus that uses a flexible rubber membrane to apply a uniform radial pressure to a 76 mm borehole stretch and measures the borehole wall displacements in four diametral directions using transducers that contact directly with the rock (Lamas et al 2009). These directions are equally spaced at 45º angles. When the dilatometer is positioned in a borehole, the positioning rods allow determining the attitude of the measuring directions.

Loading-unloading cycles are similar to those presented for the LFJ tests. Generally, three cycles up to increasingly higher pressures are performed. During the cycles, pressure and displacement readings may be taken continuously. However it is very important not to increase pressure continuously but in stages. This procedure ensures that the tests is not hastily performed and enables to take corrective precautions if the rock mass deforms too much and displacements are high. In such cases, unloading would be initiated as soon as possible and the remaining cycles would not exceed the highest pressure applied until then.

Similar to other deformability tests, direct results of a BHD test are the four graphs presenting the relations between pressure and the respective diametral displacements (Figure 15).

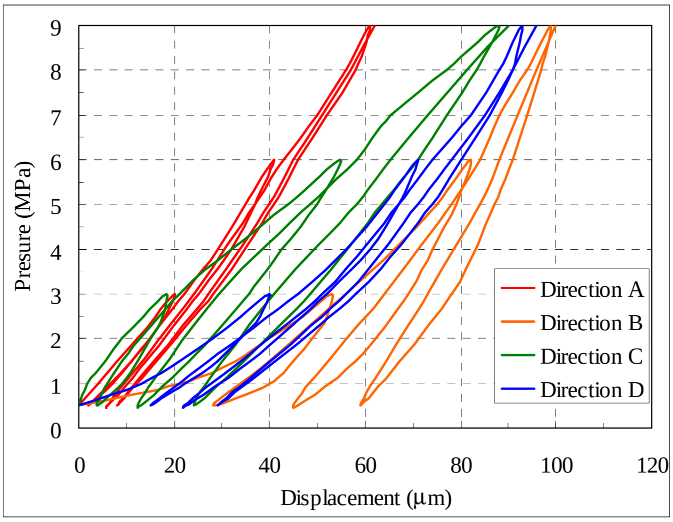

Figure 15. Pressure versus diametral displacements graphs of a borehole dilatometer test.

Measurement of the four diametral displacements allows to evaluate if the rock mass displays any anisotropic behaviour However, this assessment can be demanding, as, usually, in geological surveys, boreholes are executed along different attitudes.

The next step is to calculate the average diametral displacements and to plot these values against pressure. Assuming, again, an elastic isotropic behaviour for the rock mass, similar dilatometer moduli for the first loading, unloading and reloading cycles can be determined in the same way as for LFJ tests.

In dilatometer tests, the applied pressures induce a hoop tensile stress all around the borehole wall. In fractured rock masses, these tensile stresses may cause the opening of some joints approximately parallel to the borehole axis. Consequently, dilatometer test results are influenced by joint frequency and deformability, and may reveal certain variability. This is one of the reasons why these test results are referred to as dilatometer modulus.

The evolution of the dilatometer moduli with the applied pressure is plotted and included in the test report, as in the case of LFJ tests. These results are presented to the designer that is allowed to choose accordingly with the project circumstances that are to be considered and analysed. Nonetheless, a single value for the dilatometer modulus, corresponding to the average unloading and reloading modulus, is also forwarded for each test.

Joint presentation of the dilatometer moduli for all BHD tests in a geotechnical investigation allows a global statistical perception of the results. Figure 16 presents an example for a grey-schist rock mass that easily shows that there are no particular variations between different boreholes, and with depth along the same borehole.

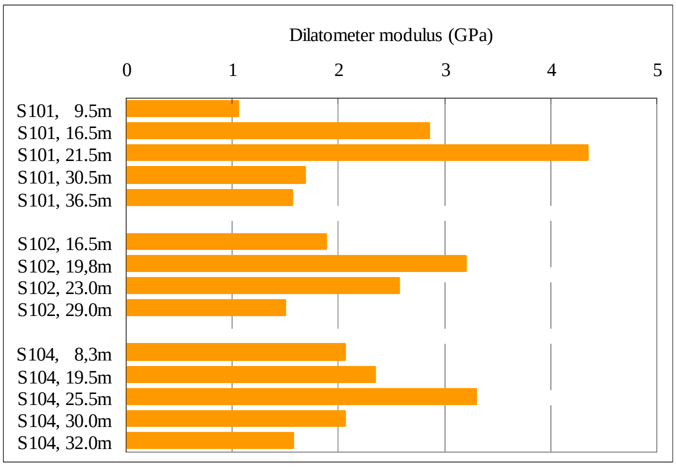

Figure 16. Results of all dilatometer test for a given project.

The expected stress levels and loading paths have to be taken into account in the estimation of mean parameters for the dilatometer modulus of any design analyses.

7 CONCLUDING REMARKS

Estimation of the geotechnical parameters to be used for the safety analysis of Rock Engineering projects is a demanding task. Eurocode 7 introduces a new perspective to the design of geotechnical works, allowing the same type of methodologies to be used for the safety assessment and design of all structural and ground parts of a project. It brings into play in the field of Geotechnical Engineering the concept of limit state design and semi-probabilistic approaches for safety assessment.

This paper presents several examples of the determination of the parameters required for the safety assessment of geotechnical works. All examples display results from common in situ and laboratory tests and studies performed for the site characterization of large projects. From each example, procedures to calculate characteristic values were referred. It should be borne in mind that the definition of the way how characteristic values are obtained is not intrinsic to the ground property described by that parameter. It must take into account how that property affects the occurrence of the limit state under analysis.

The concepts recently introduced by Eurocode 7 into Geotechnical Engineering are well established concepts, used since long by structural engineers. Many geotechnical engineers, mainly with a Civil Engineering background and often working mainly in Soil Mechanics problems, have already introduced these concepts in their way of performing the safety assessment and design of geotechnical works. Many rock engineers have also followed the same path, although they encounter obstacles and problems due to the inexistence or inadequacy of some Eurocode 7 provisions to several features required in Rock Engineering projects. However, the present status quo will be certainly overcome with the continuing application of these concepts to the field of Rock Engineering and with the resulting contributions that rock engineers will have to deliver.

AKNOWLEDGMENTS

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