54º Congresso Brasileiro do Concreto - CBC2012 - Oct / 2012
LABORATÓRIO NACIONAL DE ENGENHARIA CIVIL
@ 2012 - IBRACON - ISSN 2175-8182

Dam foundation failure analysis. Discontinuum models and rock mass parameters

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Abstract

Identification of potential failure mechanisms defined by the rock mass discontinuities or the concrete-rock interface plays an important role in the safety assessment of concrete dam foundations. The paper presents instances of application of discrete elements deformable block models for analysis of these failure scenarios, representing the rock mass foundation and the concrete dam. The rock mass characterization needs and the methodologies for obtaining the input parameters for these models are discussed.

keyword: Dam foundations, discontinuum models, rock mass parameters, safety analysis

1 Introduction

Several surveys promoted by ICOLD and other institutions have pointed out that the majority of the incidents and deficiencies experienced by concrete dams are originated by inadequate behaviour of the foundations. Though it was not the first of its kind (RODGERS (2006) presents thorough descriptions of the 1928 St. Francis dam failure), the Malpasset arch dam accident is the best known example of a structural collapse caused by sliding on rock discontinuities (LONDE 1987). As a consequence, the design of new concrete dams, or the safety assessment of existing dams, requires particular attention to be paid to the behaviour of the foundation rock mass.

This event, along with the Vajont slide, motivated a substantial body of research on rock foundations issues, namely on the hydro-mechanical behavior, and stimulated the development of new methods of safety assessment. Analytical or graphical methods, such as those proposed by LONDE (1973), became standard tools in arch dam design. With these techniques, it became possible to analyze the potential failure of rock blocks defined by the rock mass discontinuities, considering the installed water pressures and the dam loads. More recently, GOODMAN & POWELL (2003) applied Shi and Goodman's Block Theory to identify moveable blocks in concrete dam foundations. All these techniques, based on simple block mechanics, remained important for safety evaluation, while the finite element models became the preferred tools to analyze dam foundations under operating conditions and to predict stresses and displacements (WITTKE 1990).

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Presently, the designation of “discrete elements” (DE) covers a wide family of numerical methods (distinct elements, discrete finite elements, DDA, etc.), all sharing the concept of representing a discontinuous medium as an assembly of blocks or particles. They can be seen as a numerical tool that performs the same stability analysis as Londe’s method. However, not just static considerations are involved, but a full mechanical analysis is undertaken. A DE deformable block model, with internal meshes in the blocks, is capable of stress and displacement analysis as a finite element model, while retaining the ability to simulate in a straightforward manner failure modes defined by the rock discontinuities, and to be used as an engineering tool for dam foundation failure analysis.

The analysis of collapse mechanisms in dam foundations involves the representation of the discontinuities where sliding may take place. In this field of application, the block structure is better replicated in a numerical model by means of deformable blocks. In this way, a more realistic simulation of the distribution of structural loads is obtained, influenced by the foundation properties and their spatial variation, even with a fairly coarse block system. In the code 3DEC (ITASCA 2006), deformable blocks are obtained by internal discretization into a finite element mesh of tetrahedra. For dam foundation studies, these rock blocks are typically assumed elastic, with all the nonlinear behavior concentrated on the joints. For arch dams, the correct bending behaviour is more easily achieved with higher order elements, thus 3DEC allows 20-node bricks to be used for the concrete vault. In this type of model, the vertical contraction joints and the concrete-rock interface are also discontinuities which may be assigned general constitutive models. The Mohr-Coulomb model is the most widely used, but many other rock joint models exist.

2 Baixo Sabor Arch Dam

In this paper, the application of discrete element models to safety evaluation of arch dam foundations is presented, using the Baixo Sabor arch dam as a case study.

The Baixo Sabor Hydroelectric Project, presently under construction, is located in the north-east of Portugal in the lower branch of the Sabor river, a tributary of the right bank of the Douro river. The scheme is composed of two dams, a 123 m high arch dam upstream, and a 45 m high gravity dam downstream. This paper case study refers to the upstream arch dam, which has a crest length of 505 m and a total concrete volume of 670000 m^3. The full storage level is at elevation 234 and the reservoir capacity is 1095×10^6 m^3.

At the dam site, the river valley follows an essentially straight course for about 1 km, with the valley bottom, 40 m wide, approximately at elevation 127. The mean slope of the banks is about 40°. The dam is to be founded on a sound granitic rock mass. An extensive programme of rock mechanics testing, comprising field geologic surveys, in situ and laboratory tests, was undertaken.

It allowed detecting the main faults at the dam site and the joint network study identified four joint sets, being approximately orthogonal the three main ones. The main set is approximately sub-vertical and parallel to the river; the second set is sub-horizontal,

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slightly dipping upstream; and, the third one is sub-vertical, perpendicular to the valley axis. Figure 1 shows the main faults detected near the dam foundation, some of them being approximately parallel to the main joint sets.

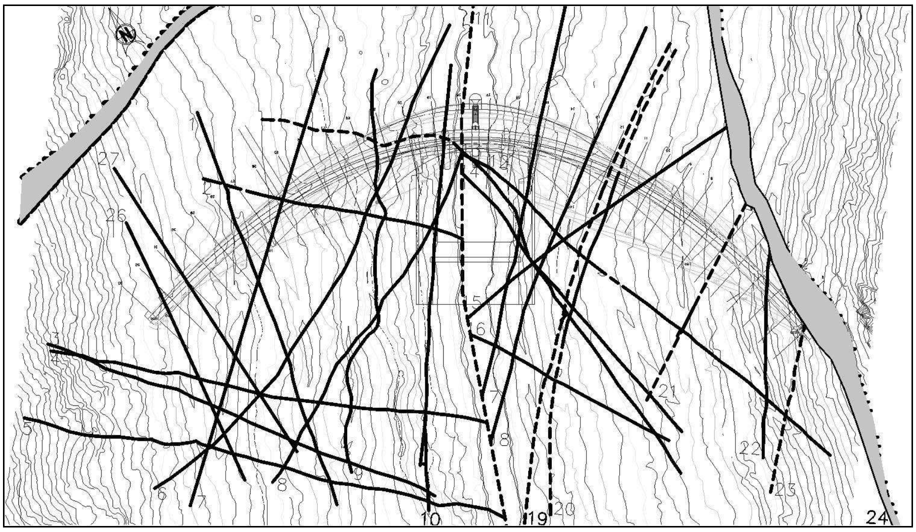
Figure 1 – Main faults detected in the rock mass

Regarding the deformability of the dam foundation rock mass, in situ large flat jack (LFJ) and borehole dilatometer (BHD) tests were performed, as well as uniaxial compression laboratory tests (LNEC 2004). Results of the four LFJ tests showed that the right margin displayed lower values of the rock mass deformability (E between 5 and 8 GPa) than the left margins (E between 13 and 21 GPa). Borehole dilatometer tests (38 tests were performed) revealed a mean value of 13 GPa for the rock mass deformability that can be considered similar to the results provided by the LFJ tests. Laboratory tests of the intact rock (30 tests) yielded values around 45 GPa for the Young’s modulus of the intact rock.

These results show that deformability is correlated with the fracturing intensity of the rock mass, which is higher in the right margin, thus increasing the deformability.

3 Joint network studies

Joint network studies are inevitable in geotechnical projects involving rock masses, whether it is an excavation slope, a tunnel, an underground cavern or a concrete dam. As for all other rock mass characteristics, the determination of the jointing parameters also requires some kind of sampling. 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 orientation; if the orientations 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 blasting is commonly used to execute the adits, care should be taken with damaged zones and 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 particular case of the Baixo Sabor dam, since no major heterogeneities were noticeable at the dam site, the sampling performed for the joint network study covered all available adits (three in each bank), referred to as GE1 to GE3 and GD1 to GD3. In each

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adit, six observation surfaces were selected along their walls. Figure 2 displays the 18 (3 x 6) observation zones on the walls of the left bank adits and shows that their locations were selected along a wide range of directions, in order to obtain a uniform spatial covering and in accordance to mitigate the effects of sampling bias.

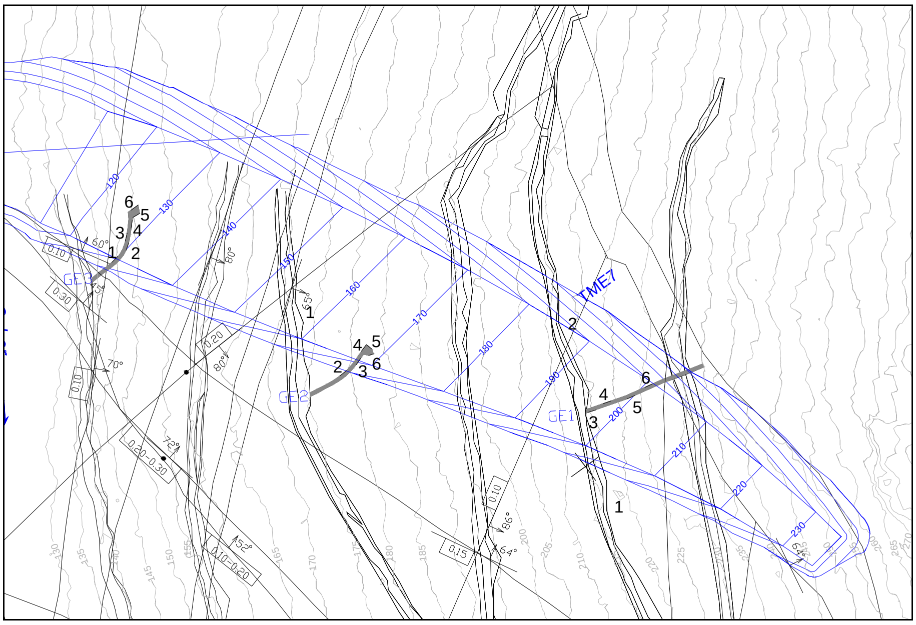

Figure 2 – Location of the observation zones in the left bank adits

In this case, all observation surfaces were 3.75 m^2 (2.5 m$\times$1.5 m) rectangles. Therefore, the joint survey covered a total area of 135 m^2. In comparison with the whole volume of the dam foundation, the total area sampled for the jointing study can be considered extremely small (less than 1% of the dam-foundation surface). Even so, this particular study identified and collected the data from a total of 2338 joints (1216 from the right bank and 1122 from the left).

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 established on equal area projections.

Subsequently, larger zones in the rock mass, which may still be assumed as 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.

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The main 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 36 observation surfaces starting with the standard equal density stereographic plots (Figure 3). 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 and their respective parameters were determined.

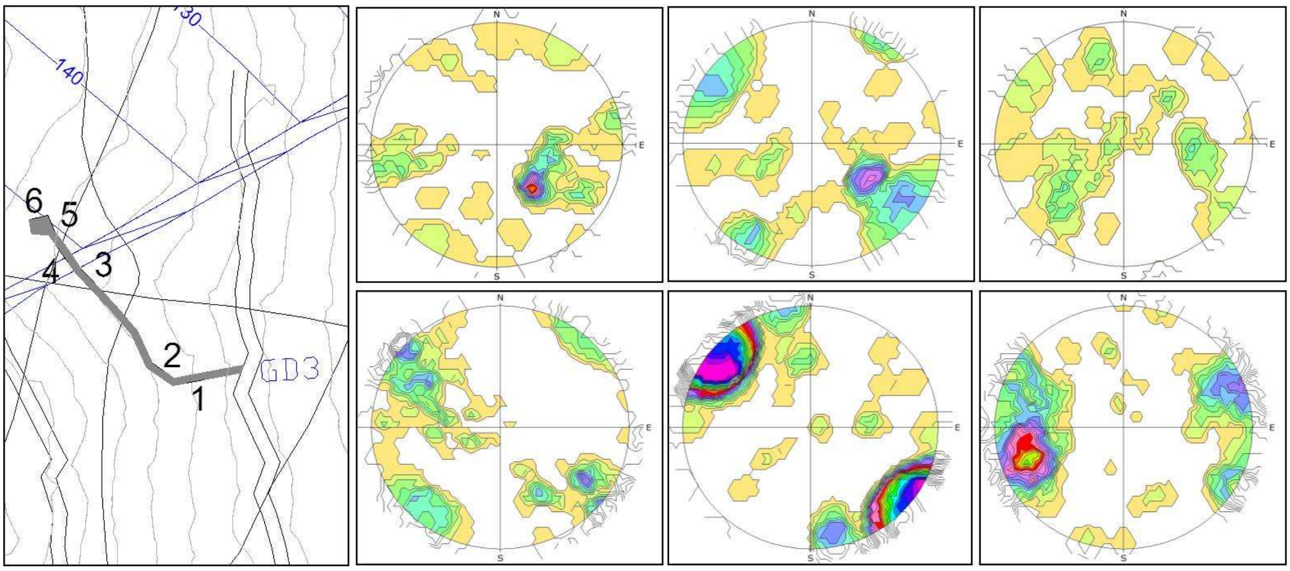

Figure 3 – Equal density stereographic plots of the six observation surfaces from adit GD3

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.

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

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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 lengths range between 2 and 4 m.

The results of the joint network study revealed that the most important joint set V_1 presented a mean attitude of N42°E;86°SE. This joint set alone is responsible for more than 50% of the total jointing in the whole rock mass. With lower degrees of relevance, another two joint sets were identified: discontinuity set H_2 with a mean attitude N32°W;09°NE with a 23% contribution to the jointing, and discontinuity set V_3 with a mean attitude N36°W;85°SW with a 17% contribution. A fourth joint set V_4 (N77°W;74°SW) was as well identified, but is only responsible for 7% of the jointing. The parameters of the statistical distributions of the attitude, intensity and area of joint sets X, V and H are presented in Table 1.

Table 1 – Geometric parameters of the most relevant joint sets

Joint set	Strike	Dip	Intensity	Spacing	Area
	(º)	(º)	(m^2/m^3)	(m)	(m^2)
V_1	132	86 SE	4.76	0.21	0.23
H_2	58	09 NE	1.96	0.51	0.25
V_3	234	85 SW	1.53	0.65	0.25
V_4	193	74 SW	0.56	1.80	0.32

Variability of the geometric parameters of the joint sets is not frequently mentioned, though it is quite important for the failure mechanisms of dam foundations. As Figure 3 allows to foretell that dispersion of the orientation of the joint sets is frequently higher than 20º, and ca even be higher in the case of sub-horizontal joint sets. It is well established that joint set attitude is more relevant for stability than the shear strength of the joints. In the case of blocks stability on natural or excavation slopes, orientation accounts for about 2/3 of the failure occurrences (MURALHA & TRUNK 1993). For dam foundations, this relevance may not be so strong due to the increased complexity of the kinematics of the failure mechanisms. Nevertheless, geometrical parameters of joint sets demand, not just the evaluation of the mean attitudes, but also the appraisal of the dispersion of the attitudes of the main joint sets.

Regarding persistence, design is usually performed considering that the joints are so persistent that they extend indefinitely. Since the probability of occurrence of joints decreases rapidly with the area, the joint probability required for large blocks to materialize is certainly very low, which may render the safety assessment quite conservative.

4 Joint Shear Strength

Even in the case of very high arch dams, generally bearing stresses acting on the rock mass are relatively low when compared with the intact rock strength leading to structurally controlled failure mechanisms. Consequently, their analysis requires the estimation of the shear strength of the rock joints, which is usually performed by means of shear tests (GOODMAN 1976; 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 foundation rock mass.

It is advisable to sample a statistically significant number of joint samples enabling the evaluation of shear strength variability. 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). Laboratory tests should be performed to estimate the shear parameters for a given joint set, defined during the joint network study. This means that 16 to 20 joint joints from each joint set should be sampled and tested specifically for this purpose.

In this case, joint samples were collected from borehole cores, purposely pertaining to the major joint sets (V_1 and H_2) and considering together sets V_3 and V_4. This sampling procedure allowed to perform 31 laboratory shear tests (15 on discontinuities from the sub-vertical set V_1, 7 from the sub-horizontal set H_2 and 9 from the sub-vertical sets V_3 and V_4). The areas of the joints were around 200 cm^2, and the normal stresses applied during slidings were 0.4, 0.8, 1.6 and 3.2 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.

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 4 presents this evaluation for joint set V_1, and Table 4 displays the values of the linear envelope parameters (apparent cohesion c and friction coefficient and angle ϕ) for the average values of each joint set.

Table 2 – Shear strength parameters of the most relevant joint sets

Joint set	Apparent cohesion	Friction coefficient	Friction angle
	(MPa)	(-)	(º)
V_1	0.095	0.762	37.3
H_2	0.122	0.780	38.0
$V_{3,4}$	0.124	0.821	39.4

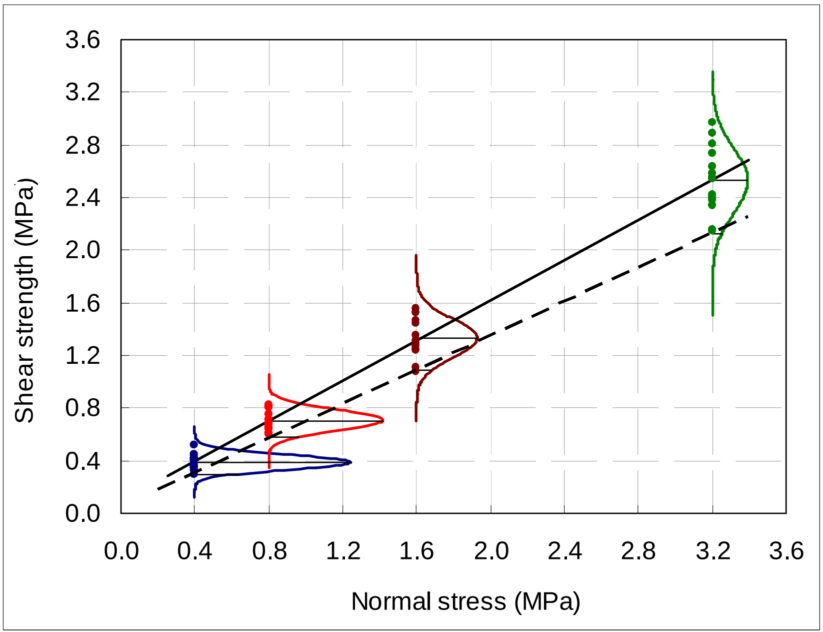


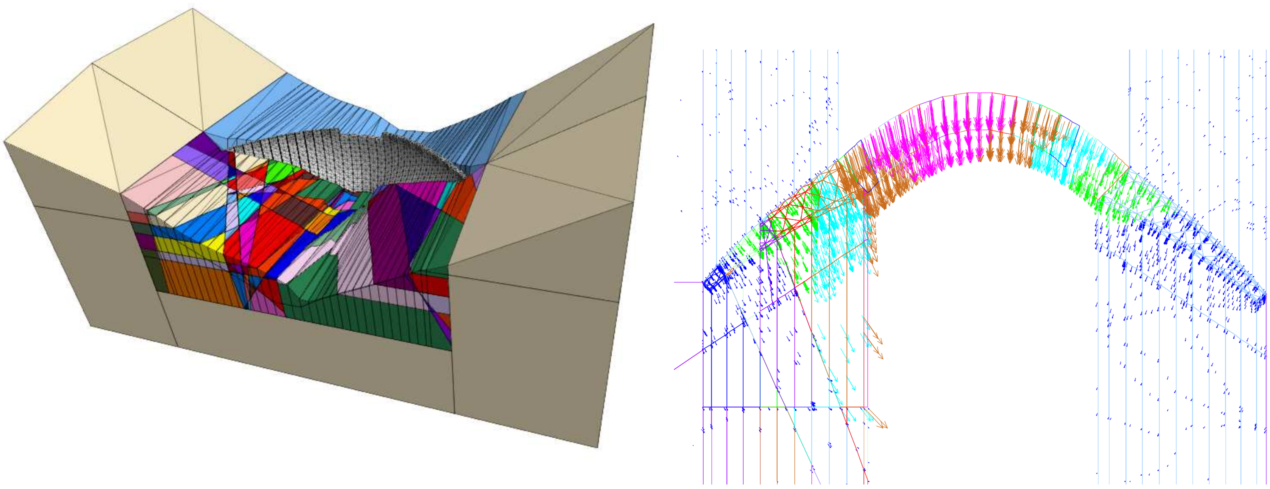
Figure 4 – Average and 95% characteristic linear envelopes for the shear tests of joint set V_1

Dots in Figure 4 refer to the results of the shear tests of all joints belonging to set V_1. They show a very low dispersion of the results, corresponding to coefficients of variation ranging between 9.8 and 14.5%.

Defining design or characteristic values has to take into due account the site specific features that are present in the failure mechanisms that is being analysed. In the case of dam foundations, failure mechanisms imply the occurrence of infinitely persistent and thus very large joints. As mentioned previously, rock joints with high areas are rare and thus have a very low probability of occurrence. As a consequence, design values for dam foundation failure analysis should not be reduced and mean or median values for the parameters should be used.

5 Numerical model

The numerical model of Baixo Sabor dam developed with 3DEC to study the failure mechanisms involving the rock mass is shown in Figure 5 a) (LEMOS & ANTUNES 2011). A simplified representation of the rock mass discontinuities was adopted, including the faults with known location depicted in Figure 1, and also joints representative of the various sets identified at the site. A Mohr-Coulomb joint constitutive model was applied in all the discontinuities, according to the experimental results discussed. The system was subject to the gravity loads, hydrostatic loads on the upstream dam face, and water pressures in all discontinuities. These joint water pressures play an essential role in the sliding failure modes through the rock mass. The safety assessment was performed by reducing the tangent of the friction angle by an increasing factor until a collapse mechanism develops, as shown in Figure 5 b), in which a typical of arch dam abutment can be seen, involving a rock wedge sliding in the right bank. The safety factor obtained, in the order of 2, indicates that this dam has a substantial safety margin.



(a) (b)

Figure 5 – (a) Discrete element model of dam and rock mass; (b) Displacements on horizontal cross section denoting failure mode on the right bank abutment.

6 Final remarks

A case study was presented of the safety assessment of a new arch dam. The importance of the field and lab geotechnical works is stressed, as reliable values are essential for input into elaborate numerical models. The ability of discrete element models to evaluate failure modes through the rock mass is also to be noted.

Aknowledgment

Permission by EDP Gestão da Produção de Energia, SA to present data for Baixo Sabor dam is gratefully acknowldeged by the authors.

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