Rock joint shear tests.
Methods, results and relevance for design

J. Muralha
National Laboratory for Civil Engineering - Portugal

Abstract: Recent regulations introduced a limit state approach to geotechnical design by using representative values of the actions and of the strength parameters, partial safety factors that affect them, and by including safety margins in the calculation models. Moreover, Eurocode 7 stresses the importance of making use of test results for establishing ground parameters, and so rock joint shear tests are set to play a relevant role in the assessment of the shear strength required in the design of important projects, such as concrete dams, large bridge foundations, slopes, or underground excavations. In this keynote, joint shear tests are described, along with a presentation of their equipments and different procedures. Test results and calculations for the assessment of relevant shear strength parameters are illustrated, and several topics regarding sampling and variability are discussed. Opportunity is taken to present a practical apparatus allowing to perform simple shear tests (push tests) under very low normal stresses with advantages over tilt or pull tests.

Theme: Laboratory testing

Keywords: rock joint, shear strength, shear tests

1 INTRODUCTION

Certain projects, such as concrete dams, rock slopes, or relatively shallow underground works, require the design of geotechnical works in rock masses where stresses are low when compared with the intact rock strength. In these cases, stability is structurally controlled by the shear strength of kinematically unfavourable rock 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 done by means of shear tests (Goodman 1976; Hoek & Brown 1977, Muralha 2007).

Recently, Eurocode 7 "Geotechnical design – Part 1: General rules" (CEN/TC 250 2004) brought to Rock Engineering design the limit state approach, as it introduces 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 with rules that 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 mainly to common civil engineering works, it establishes a comprehensive framework for the design of any kind of structure, such as underground caverns, tunnels, slopes, and dam or large bridge foundations.

Concerning ground properties, such as the shear strength of rock joints, Eurocode 7 refers that the values of the geotechnical parameters to be used in design should be obtained from test results, either directly or through correlation, theory or empiricism, and from other relevant data. It also refers that characteristic values should be selected as cautious estimates of the parameters affecting the occurrence of the limit state under consideration. If it involves a failure mechanism affecting 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 can be assessed directly or thy may be determined from the characteristic values using partial safety factors, which have not yet been defined for large Rock Engineering projects.

As in other fields of Rock Mechanics, testing techniques and methodologies have not progressed as fast as computer capacities have allowed complex constitutive models to be implemented in commercial codes. The evolution of the design of concrete dam foundations can be used as an example to show the relevance of rock joint shear tests.

It is quite curious to notice that this same issues were already being addressed almost 40 years ago: "*As regards the methods for analysing dam foundations, one of the frequently put is: in what way (physical) models can still be useful, considering the spectacular progress of numerical methods, …, and the rapid generalization of their use in design?*" (Rocha, 1974).

Nowadays, discrete element models, like 3DEC (Itasca 2006), can be used for the design analysis of concrete dams regarding limit states involving shear failure along foundation discontinuities is presented (Lemos, 2011). This code displays special features allowing to simplify the analysis. The rock mass is represented by deformable blocks of polyhedral shape, which are internally divided into a finite element mesh of tetrahedral uniform strain elements, and for the concrete body of the dam the code allows the use of 20-node solid brick elements, much more efficient for the accurate representation of the bending behaviour. This finite element mesh can be imported into 3DEC after being previously generated outside. Regarding the rock mass, the representation of rock discontinuities requires a necessarily simplified approach.

Typically, individually identified major features, as faults and dykes, are inserted at their known locations with their respective orientations, and each of the most significant joint sets is represented by a few selected joint planes. The purpose of the analysis - stability assessment - directs the selection of the number and location of joints, as the intention is not to recreate in detail the joint structure, but to identify the possible failure modes and their likelihood. Experience and lessons from classical papers (e.g. Londe 1973) have shown that, in practice, a small number of joints from each set is normally sufficient to disclose the most relevant mechanisms (Fig. 1).

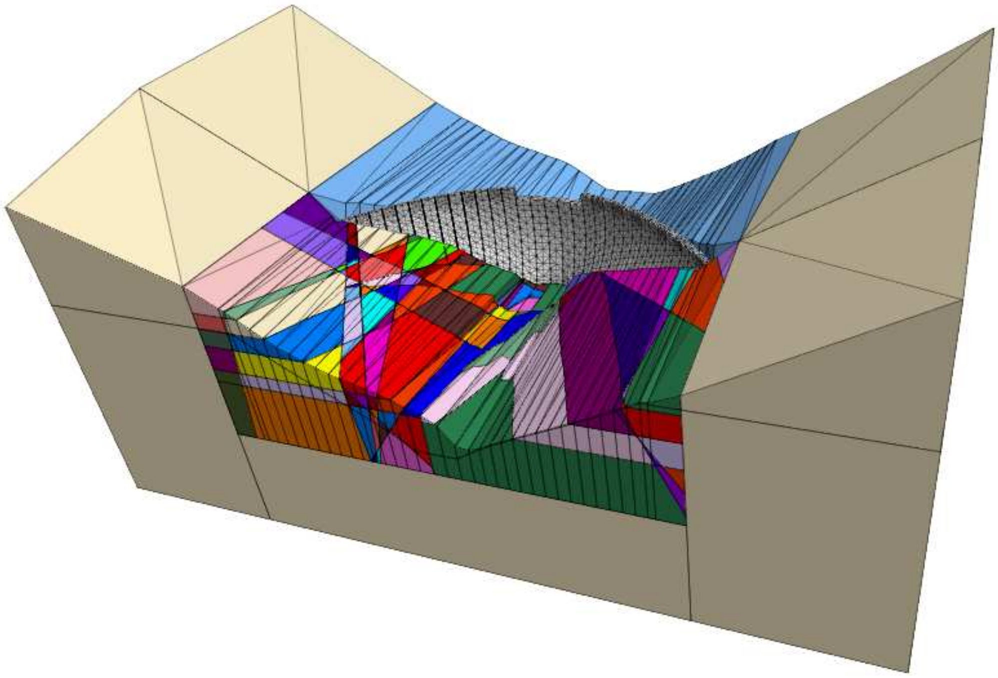

Figure 1. Discrete element model for the design analysis of limit states involving shear failure along foundation discontinuities of an arch dam (Lemos, 2011).

Rock joint shear tests have not experienced similar progresses as the improvements in computer capacities and the development of commercial software with complex constitutive models requiring several parameters that are not easily evaluated. Therefore, defining the relevant limit states of a given project, establishing the respective adequate conceptual models and defining the design parameters based on shear tests emerge as crucial tasks, for which contribution of experienced practitioners and designers is essential. In accordance, this paper intends to present the evolution of rock joint shear tests, their methods, and to discuss the relevance of the results they can provide for Rock Engineering design. Particular attention is paid to the path that starts with the testing methodologies, goes through the assessment of the results, and ends at the correct estimation of the shear strength parameters of the rock joints.

2 ROCK JOINT SHEAR TESTS

The basic principle of shear tests is to subject a set of joint samples to various normal stresses and to determine the shear stresses (strength) required to induce a certain shear displacement. Preferably, the shear strength of rock joints should be evaluated using the results of in situ tests, as they inherently account for any possible scale effect. However, they are especially difficult to execute, notably time consuming in what concerns the preparation of the specimens, and accordingly quite expensive. As a consequence, only very few can be performed for a given project, which makes it impossible to evaluate shear strength parameters with statistical significance. Even so, in situ shear tests can assume particular relevance when the limit states are conditioned by weak or filled joints (Barla et al. 2011, and Alonso et al. 2011).

Laboratory direct shear tests have been performed since Rock Mechanics early days more than 50 years ago (e.g. Natau, 1980; Franklin, 1985), using different types of equipments and apparatuses, following various procedures and evaluating distinct parameters.

2.1 Apparatuses and equipments

Determination of shear strength of rock joints is generally performed using direct shear apparatuses derived from similar equipments developed for the same kind of tests in soils. Though there are many variations in the way specimens are prepared, mounted, and loaded, commonly, direct shear testing machines incorporate (Fig. 2):

- A stiff testing system, including a stiff frame against which the loading devices can act and a sufficiently rigid sample holder to prevent distortion during the test. A stiff system allows the prescribed shear displacement rate to be maintained and allows the post-peak behaviour of the joint to be properly recorded.
- A specimen holder such as a shear box, shear rings, or similar device where both halves of the specimen are fastened. It must allow relative shear and normal displacements of the two halves of the discontinuity. Frictional forces on the perimeter of the sample holder must be minimized via rollers or other similar low friction devices.
- Loading devices to apply the normal and shear loads on the specimens at adequate rates such that the resultant of the shear load goes through the centroid of the sheared area to minimize rotation of the specimen.
- Devices to measure the normal and shear loads applied to the specimen and the normal and shear displacements throughout the test.

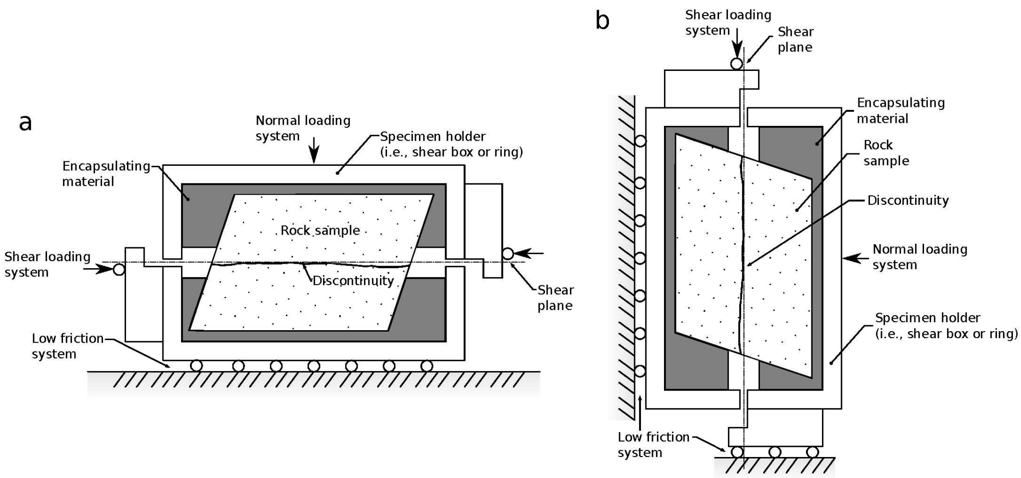

Figure 2. Schematic illustrating arrangement of laboratory direct shear specimen: (a) conventional horizontal arrangement and (b) alternative vertical arrangement.

Regarding the loading devices, the applied shear forces are usually provided by actuators (hydraulic, pneumatic, mechanical (gear-driven), etc.) with or without closed-loop control. Shear force actuators and connecting parts should be designed to ensure that the shear load is uniformly distributed over the discontinuity plane to be tested with the resultant force acting parallel to the shear plane through its centroid. Moreover, the design must provide a shear travel large enough for residual shear strength to be reached. Generally, a shear travel around 10% of the specimen length is sufficient. The applied constant normal load or constant normal stiffness is usually provided by actuators (hydraulic, pneumatic, mechanical, etc.) with or without closed-loop control. Normal force actuators and connecting parts should be designed to ensure that the load is uniformly distributed over the discontinuity plane to be tested. They should accommodate travel greater than the amount of dilation expected in the test and ensure the applied normal load is uniformly distributed over the test horizon with the resultant force acting perpendicular to the shear plane through its centroid. A cantilever system can also be used to apply a constant normal dead-weight load for tests under null normal stiffness, while a spring can be used to maintain a constant normal stiffness condition. Keeping constant the normal load or the normal stiffness is essential during shear tests. In accordance, the normal loading component of the apparatus must be devised to maintain the applied force or stiffness within a specified tolerance (2%).

Concerning the recording of the loads and displacements throughout the tests, the normal and shear forces are measured directly by load cells or indirectly by pressure gauges or transducers with accuracy better than 2%, and displacement transducers are used to measure the displacements. A minimum of two transducers are required: one mounted parallel with the shear plane to measure the shear displacement and one mounted vertically at the centre of the specimen to measure normal displacement. Preferably, two transducers should be used to measure shear displacement such that sway of the specimen is measured, and three to four transducers should be employed to measure horizontal displacement, such that pitch and roll of the test specimen can be evaluated. It is common practice to perform almost continuous measurements (sampling rate greater than 1 Hz) of these parameters using some kind of computer based data acquisition equipment, which is acceptable for quasi-static loading conditions commonly followed in laboratory joint shear tests.

2.2 Test specimens

Sampling, handling and storage of rock joint samples for shear tests require certain precautions aiming at minimizing disturbances.

Since Eurocode 7 underlines semi-probabilistic concept for design, it is advisable to choose a statistically representative number joint samples from the same joint set or from the same shear horizon to allow adequately characterizing its shear strength. Previous results of shear tests of joints from the same joint sets pointed out to a relatively high dispersion of the shear strength (Muralha, 1995). As coefficient of variation values for the shear strength around 25-30% can be frequently found, a statistically significant number of tests easily reaches values of about 20.

Joint samples for shear tests should preferably be collected in situ specifically for this purpose. However, common practice is to use samples found in boreholes cores, because they are more simply available. These type of samples present several disadvantages, though they generally are the only ones available: they display areas with small sizes (around 50 cm^2); they present an oval or circular shape in which the sample length is difficult to calculate and the border regions in the middle zone of the samples do not play any role in the mobilization of shear strength; finally, it is not easy to obtain samples with lengths greater than 10 cm from small cores, which is a value that allows the sample to reveal its roughness at this scale.

As a rule, the length of the test plane (measured along the shear direction) should be at least 10 times the maximum asperity height, and the width of the test plane (measured perpendicularly to the shear direction) should have at least 50 mm, allowing to collect samples from NQ cores. Concerning the shape of the sheared area, rectangular shapes are the most suitable because any error in the evaluation of the area results in an error in the shear strength.

If feasible, the length of the halve of rock sample that moves during shear should be slightly smaller than the other halve. This practice keeps the joint surface continuously supported during shear displacement, and does not demand any kind of area correction during the test.

In the field, the sample dip, dip direction, and other relevant geological features should be recorded. If possible, the absolute orientation relative to the test horizon should be marked on the sample, enabling the shear direction in laboratory to correspond to a particular in situ displacement direction of interest, which is relevant for instance in slope stability studies where the failure mechanism and its corresponding kinematics are well defined.

In the cases where the shear direction is not defined during sampling, the joint shear test is performed conservatively along the direction that displays the minimum strength. It is common practice to define this direction simply by pushing by hand the joint sample.

Figure 3 shows the relations between the friction coefficients of rock joints determined by pull tests along direction 1, chosen as the direction of minimum shear strength, and the direction 2, the opposite direction. Different markers refer to different joint sets. The graph shows clearly that the shear strength for direction 2 is larger than for direction 1.

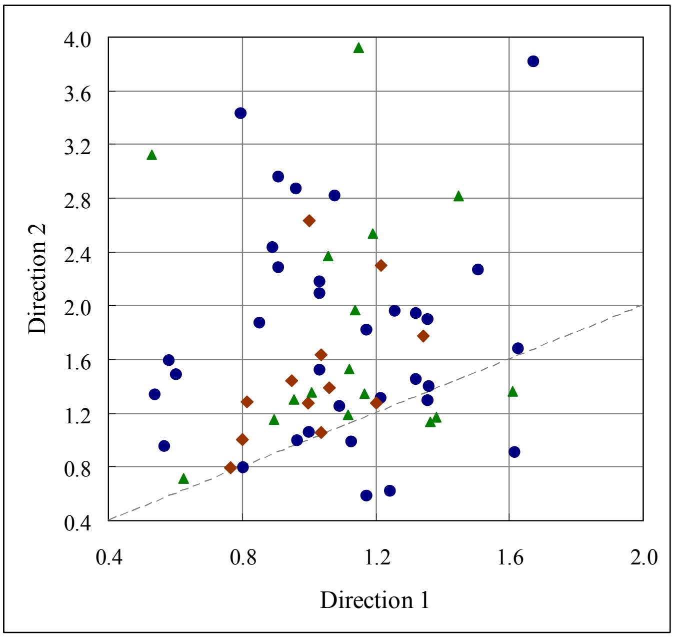

Figure 3. Friction coefficients determined by pull tests in the same rock joint for opposite directions.

In this particular case, friction angles would be conservatively reduced by values ranging between 35 and 50% if just the test results provided by direction 1 instead of all results were considered. It is likely that this decrease in the shear strength parameters would not be so important if the shear tests were performed under higher normal stresses . Nonetheless, this reduction represents an additional, although unaccounted, safety margin introduced in the design.

2.3 Procedures

Shear strength of rock discontinuities can be determined by tests under constant normal loading conditions (CNL), or under constant normal stiffness conditions (CNS). Under constant normal stiffness conditions, a single strength determination usually includes the testing of multiple specimens with differing initial normal loads and constant normal stiffness and measuring the shear and normal stresses and respective displacements resulting from a prescribed rate of shear displacement.

Under constant normal loading conditions, a single strength determination usually includes the application of several constant normal loads or stresses on multiple samples and measuring also the shear and normal stresses and the respective normal and shear displacements.

To define the shear strength of a given test horizon or rock joint, at least three, and preferably five, specimens should be obtained and tested in the same direction. Alternatively, under constant normal loading, a single specimen can be tested repeatedly under different constant normal stresses. This latter approach will usually result in a more conservative strength estimate as the incremental damage caused by prior tests under lower normal stresses will artificially decrease the shear resistance in subsequent tests under higher normal stress. To mitigate this effect, the normal stresses should be applied from the lowest to the highest. For a single rock joint, at least three, and preferably five, different normal stresses should be tested in the same direction. This latter option allows reducing the number of joints to be tested in order to define a statistically significant failure envelope, and so generally multi-stage test procedures are followed.

Two possible procedures for performing several CNL shear tests under different normal stresses using the same rock joint are generally followed: with or without repositioning of the joint in its initial natural position before each sliding. An example of a test without repositioning is presented in Figure 4. This figure shows that the test starts with an increase of the normal stress up to the 0.5 MPa (1), followed by the shearing of the joint displayed by the increase of the shear stress and corresponding shear displacement (2); when the shear stress appears to reach a constant value, shear displacement is stopped and the normal stress is increased to 1.0 MPa (3) and a second shearing is applied (4); the same procedure is followed for a normal stress of 1.5 MPa (5) and (6), and for the last normal stress of 2.0 MPa (7) and (8).

Multi-stage shear test with repositioning of the joint in its initial position is another widespread procedure that can be described as follows:

- Loading-unloading cycles up to a high normal stress, at least the largest normal stress that will be applied during the test;
- Loading of the joint up to the first (lower) normal stress;
- Shearing of the joint under this normal stress maintained constant through out;
- Removal of all wear debris and cleaning of the joint surfaces;
- Placing of the joint in its initial and mated position;
- Repetition of first five steps with the second normal stress;
- Repetition of first five steps with the third normal stress;
- Repetition of first steps with the last normal stress.

In both multi-stage procedures, the first sliding takes place under the lowest normal stress and the following slidings are performed under increasing normal stresses. This practice tries to minimize the influence of large shear displacements that wear the joint surface breaking the roughness and asperities. To minimize this effect, the normal stresses should not cover uniformly the expected range of stresses (e.g. 0.5, 1.0, 1.5 and 2.0 MPa); instead, a geometric progression should be used. For instance, if the design analysis foresees stresses in the joints of about 3 MPa, the following set of normal stresses would be adequate for the tests: 0.4, 0.8, 1.6 and 3.2 MPa. Barton's peak shear strength criterion, that includes a log relation between the JCS and the normal stress, supports this principle of doubling the normal stress for the following shearing.

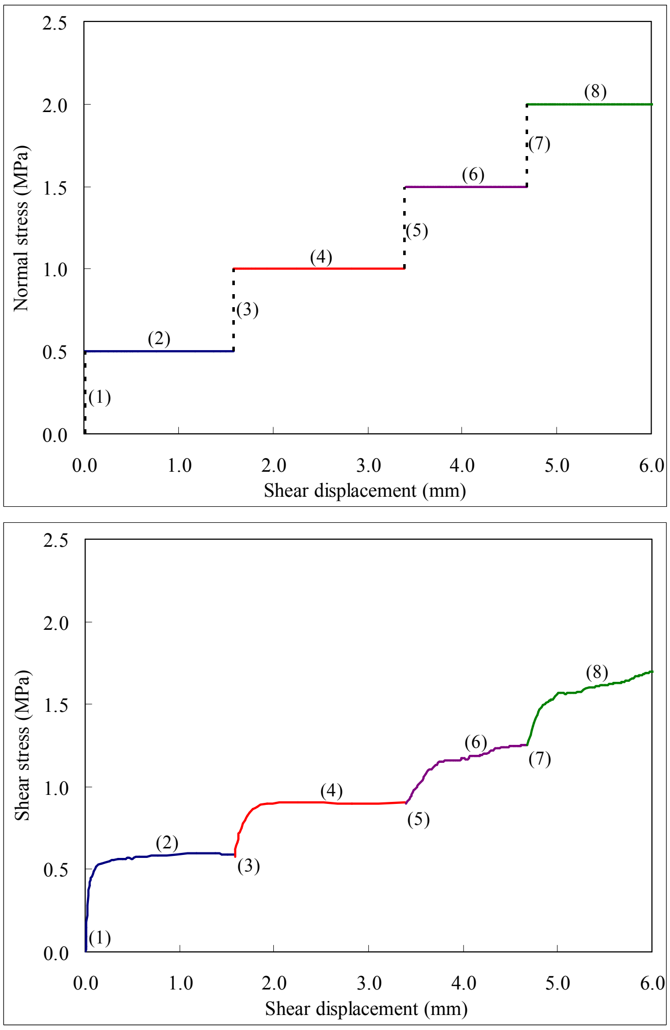

Figure 4. Test procedure without repositioning.

In the procedure with repositioning, in order to reach conditions as similar as possible before all slidings, it is very important to perform previously to each sliding a loading-unloading cycle up to a high normal stress. Figure 5 presents an example with plots of the last normal loading cycles of a joint performed prior to each shearing. Though between each graph the joint was sheared, the plots display a close fit to the same hyperbolic closure curves, revealing that the joint initial position prior to each shearing is identical.

2.4 Results

Results of rock joint shear tests can be summarized in the shear strength vs shear displacement and normal displacement vs shear displacement graphs (Fig. 6). Despite the fact that just the former are required for the evaluation of the shear strength, both graphs should be considered and analysed altogether.

Preferably, sampling to perform rock joint shear tests to assess the shear strength of rock joints for an important project should be specifically performed, allowing to collect around 20 samples from each of the major joint sets defined in the geologic survey. As already mentioned, in important projects using joints samples from small borehole cores is not advisable.

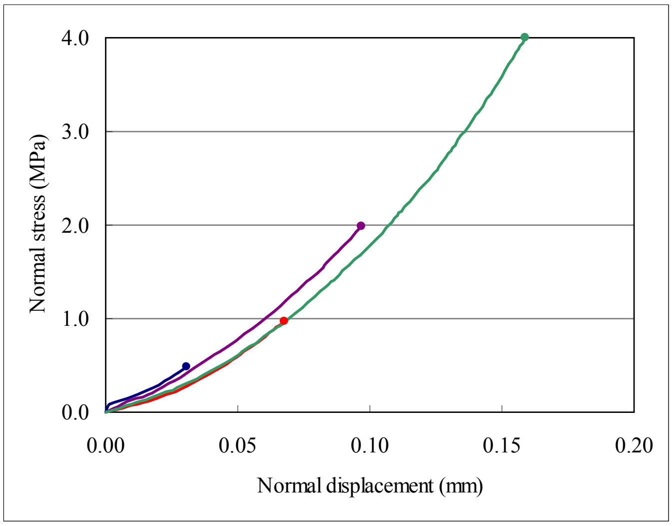

Figure 5. Normal pre-loading closure curves of a rock joint.

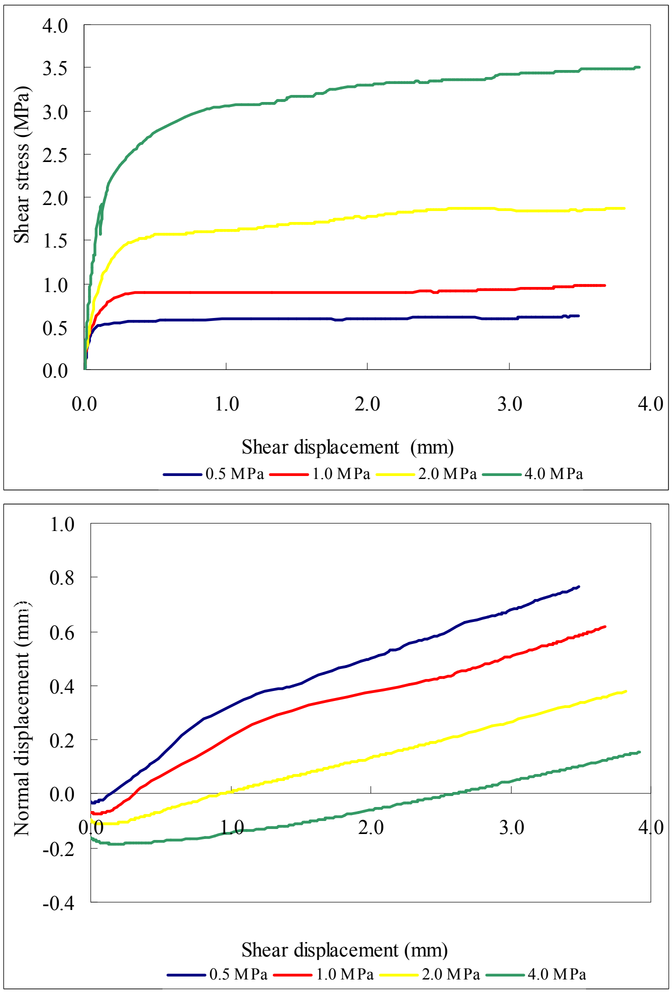

Figure 6. Example with results of a rock joint shear test.

As an example, the strategy and the results of a set of shear tests used for the site characterization study of an arch dam are presented. In this case, a portable drilling rig was used inside exploratory adits to extract 150 mm cores containing purposely chosen joint samples from the three major joint sets (X for schistosity, V for sub-vertical, and H for sub-horizontal). This sampling procedure allowed performing 54 laboratory shear tests (18 on discontinuities from joint set X, 20 from joint set V and 16 from the joint set referred to as H). This kind of sampling enabled to collect relatively large joints with about 200cm^2.

It is very important to carry out all tests under the same normal stresses σ_n, in order to perform simple to calculate simple descriptive statistics enabling to determine the average shear strength τ and standard deviation s_τ for each group of slidings at the same normal stress. Figure 7 displays the results of the shear tests of joints from the joint set X, and Table 1 compiles the averages and standard deviations of the shear strength of all joint sets for each applied normal stress.

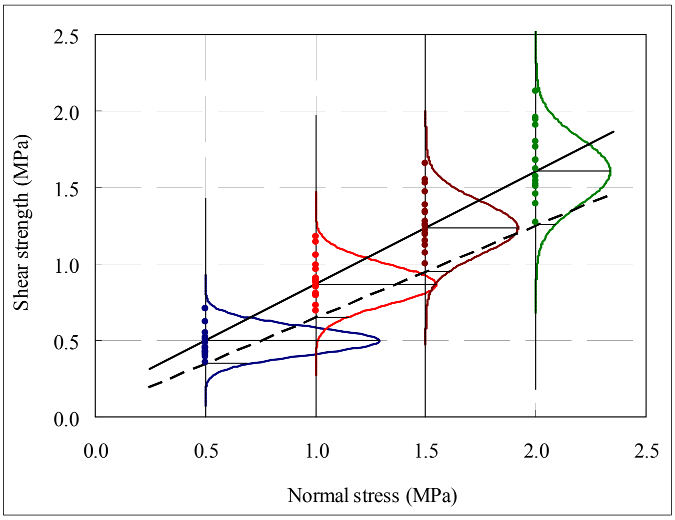

Figure 7. Average and 95% characteristic linear envelopes for the shear tests of joint set X.

Table 1. 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 any given of probability of being exceed, such as 50 and 95%, are easily computed, enabling to define average and characteristic linear envelopes. Figure 7 shows the normal distributions estimated from the tests results with the parameters presented in Table 1.

2.5 Calculations

The shear strength of rock joints can be denoted by appropriate failure envelopes. Usually a linear Coulomb envelope is used. However, extrapolation for low normal stresses of this linear criterion have to be performed with caution, as issues regarding rock joint (apparent) cohesion are debatable and yield unconservative shear strength estimates. Alternatively, Barton's failure criterion, which is starting to be included in codes (CFGB, 2002), can be favourably used.

To investigate the relations between both failure criteria, a set of joint tests was performed. Firstly, procedures defined in Barton & Choubey (1977) were followed to calculate JRC, JCS and ϕ_r, followed by regular shear tests on the same joints to evaluate the corresponding apparent cohesion and friction angle.

For the determination of JRC, instead of tilt or pull tests, push tests were performed with a specifically designed apparatus. The special feature of this equipment is a hard plastic block that is pulled over roller bearings (Fig. 8), and pushes the upper halve of the joint sample without any kind of rotation or overturning that can be caused by the pull force if it is not parallel to the joint mean surface.

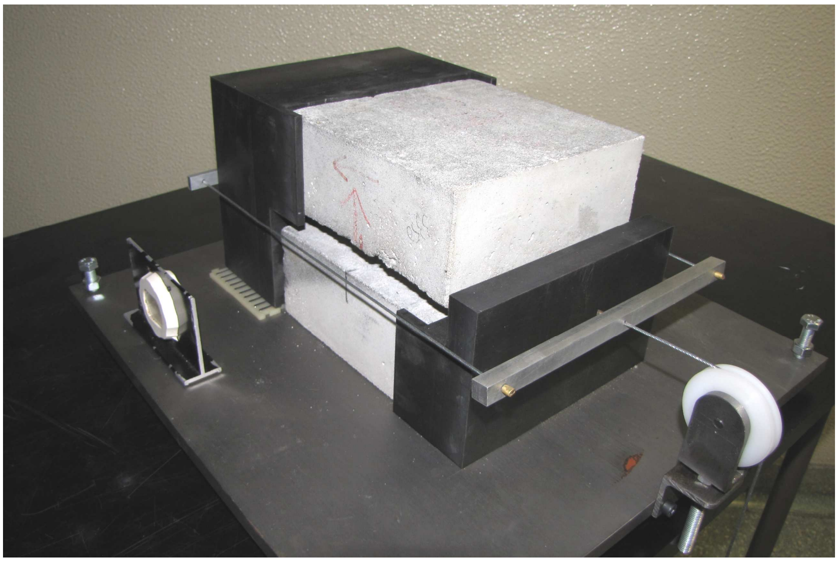

Figure 8. Push test apparatus.

The research showed that evaluating Barton's criterion parameters using push/pull tests and Schmidt hammer rebounds would render higher shear strength estimates than the values obtained from regular shear tests. Figure 9 presents the average results of the shear tests and both failure criteria (Barton and Coulomb); latter being defined from the tests results. This relation can be justified by the fact that the push tests were performed under very low normal stresses (around 5 to 10 kPa), and extrapolating their results for higher normal stresses (a couple of magnitudes higher) can be problematic.

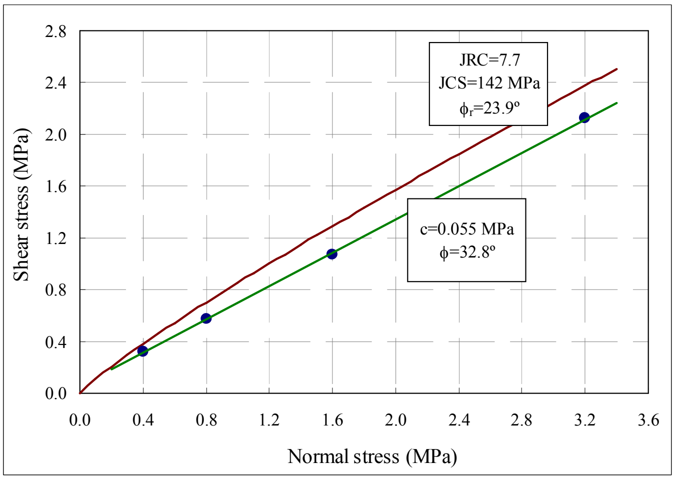

Figure 9. Comparison of Barton's criterion estimates and shear test results.

In order to try to overcome this difficulty attempts to estimate JRC, JCS and ϕ_r directly from the results of the shear tests were conducted. Straightforward regression techniques can not be used, as the parameters are not linearly independent. So, Barton's failure criterion equation was modified as follows

$$\tau = \sigma_n \tan\left[a - JRC \log_{10} \sigma_n\right] \quad (1)$$

With a being given by

$$a = JRC \log_{10} JCS + \phi_r \quad (2)$$

Using test results, both shear tests and push tests, Equation (1) allows to determine JRC (equal to 3.1) and a, and equation (2) provides a relation between JCS and ϕ_r. Considering the Schmidt hammer tests performed on the joints and on sawed surfaces of the rock, and shear tests of the same sawed surfaces, which rendered a ϕ_b value of 30.1°, a JCS value of 205 MPa and a ϕ_r value of 27.8° were determined. Figure 10 shows estimates from both failure criteria to the shear tests results equally displaying good fits.

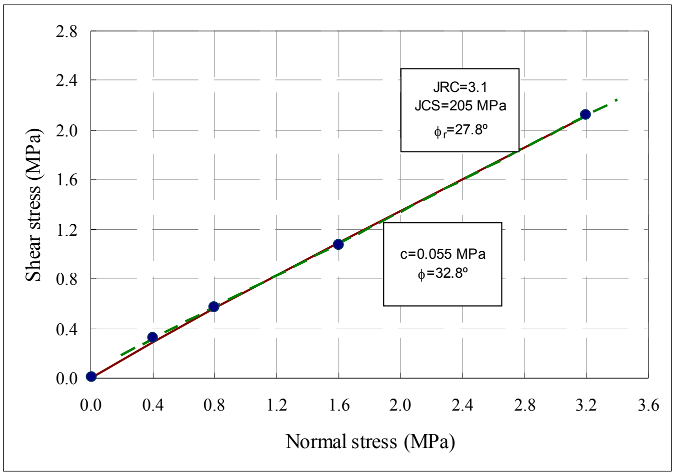

Figure 10. Coulomb and Barton envelopes determined from the same shear tests.

Other options for the determination of JRC, JCS and ϕ_r are to plot the test results as functions of $\log_{10}(JCS/\sigma_n)$; however, in this case, instead of the shear strength, the friction angles or friction coefficients are considered (Fig. 11). It has to be pointed out that the example displayed in this figure refers to a case where the fits between the test results (dots) and the graphs corresponding to the evaluation of JRC, JCS and ϕ_r by Barton & Choubey procedures unusually show remarkably low deviates.

These graphs also show that performing regressions with different types of linearization can render quite different results. In this particular case, it is rather different to minimize deviates of the shear strength, friction angle or friction coefficient.

3 CONCLUDING REMARKS

Recent regulations introduced a limit state approach to Rock Engineering design by using representative values of the actions and of the strength parameters, partial safety factors that affect them, and by including safety margins in the calculation models. Moreover, it stresses the importance of making use of test results for establishing ground parameters.

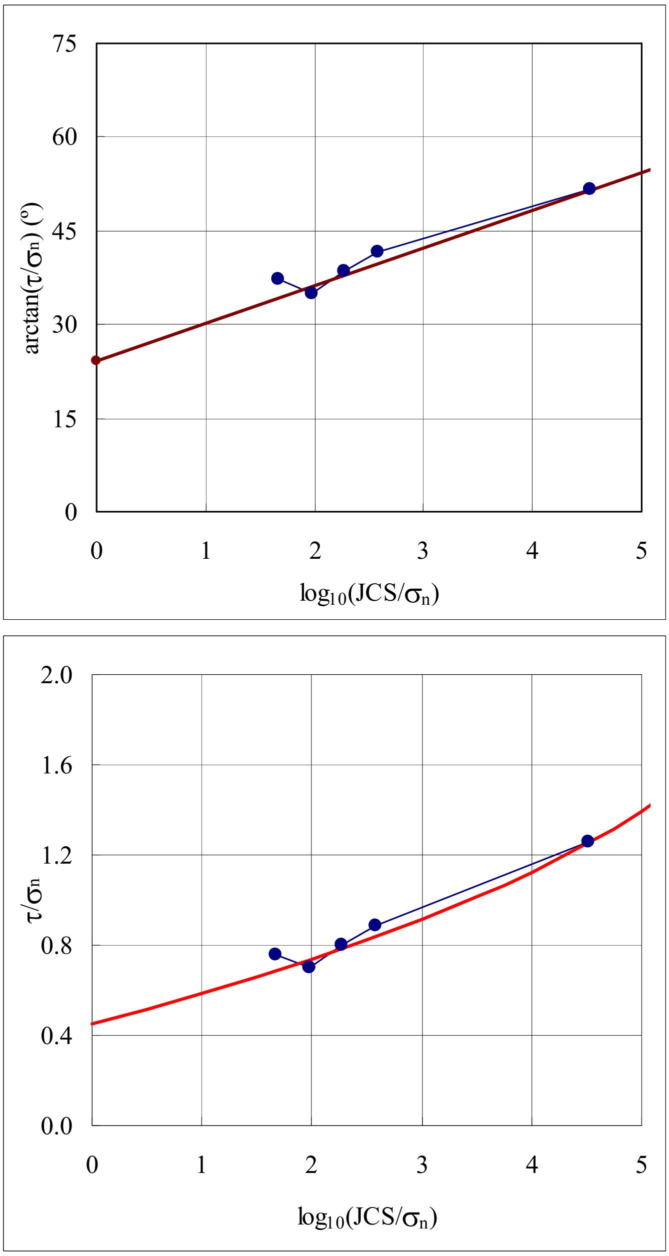

Figure 11. Different types of plots of the shear strength of rock joints.

Rock joint shear tests play a relevant role in the assessment of the shear strength required in the design of important projects; therefore internationally accepted standard procedures are required. As several different types of apparatuses and equipments are worldwide available, it is essential to perform a wide inter-laboratorial testing program to estimate variability regarding test results.

Joint sampling is another issue that affects shear strength variability. It was referred that generally shear tests were performed along low strength directions; however, it has also to be noted that weathered (low JCS) and gauge filled joints are very difficult to sample and test. Both these subjects have to be addressed at the design stage when the characteristic parameters are chosen.

The compromise between time and cost disadvantages of in situ tests and the problems regarding scale effects and the extrapolation of the results of small laboratory tests to large real discontinuities must be considered as well.

The pros and cons of perform several repetitions on the same joint sample must be carefully weighed, taking in consideration the effects on the results of roughness wear and the savings that can be achieved.

Coulomb and Barton failure criteria both comprise intuitive physical parameters that address rock joint shear strength from different points of view; moreover, they are well established among Rock Engineering practitioners. Coulomb linear envelopes main disadvantage refers to the questionable extrapolation to low normal stresses and the unconservative consideration of apparent cohesion. To cope with this uncertainty, codes and regulations generally apply high or even drastic partial safety factors to reduce or annul cohesion. On the other hand, Barton criterion allows considering shear strength for low normal stresses, but implementing partial safety factors to its parameters is almost impossible due to their non-linearity. Efforts should be set on trying to relate the parameters of Coulomb and Barton criteria in order to make use of the advantages of both.

REFERENCES

Alonso, E.E., Pinyol, N.M., & Pineda, J.A. 2011. Foundation of a gravity dam on layered soft rock. Shear strength of bedding planes in laboratory and large “in situ” tests. Proc. 15th European Conf. Soil Mechanics and Geotechnical Engineering – A. Anagnostopoulos et al. (ed), Athens, Greece. IOS Press, Amsterdam.

Barla, G., Robotti, F. & Vai, L. 2011. Revisiting large size direct shear testing of rock mass foundations. 6th International Conference on Dam Engineering – Pina, C., Portela, E. & Gomes, J. (eds), Lisbon, Portugal. LNEC, Lisbon.

Barton, N. & Choubey, V. 1977. The shear strength of rock joints in theory and practice. *Rock Mechanics*, **10**: 1-54.

CEN/TC 250. 2004. EN 1997-1: Eurocode 7 Geotechnical design - Part 1: General rules. Comité Européen de Normalisation, Brussels.

Comite Français des Grands Barrages, 2002. Justifications des barrages poids.

Franklin, J.A. 1985. A direct shear machine for testing rock joints. *Geotechnical Testing Journal*, **8** (1): 25-29.

Itasca 2006. 3DEC - Three-Dimensional Distinct Element Code, Version 4.0, User’s Manual. Itasca Consulting Group, Minneapolis, USA.

Lemos J.V. 2011. Discontinuum models for dam foundation failure analysis. 11th ISRM Congress, Harmonising Rock Engineering and the Environment – Qian & Zhou (eds), Beijing, China. Taylor & Francis Group, London.

Londe, P. 1973. Analysis of the stability of rock slopes. *Quarterly Journal of Engineering Geology*, **6**:93–127.

Muralha, J. 1995. Statistical description of shear parameters of rock joints. 8th ISRM Congress, Tokyo, Japan. Balkema, Rotterdam.

Natau, O.; Leichnitz, W. & Balthasar, K. 1980. Construction of a computer-controlled direct shear testing machine for investigations in rock discontinuities, 4th ISRM Congress, Vol. 3, Montreux, Switzerland. Balkema, Rotterdam.

Rocha, M. 1974. Analysis and design of the foundations of arch dams. Int. Symp. On Rock Mechanics applied to dam foundations, General Report – Theme III, Rio de Janeiro, Brazil.