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YIELDING OF ROCKFILL IN TRIAXIAL EXPERIMENTS



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ABSTRACT

In recent years, interest in rockfill mechanics has considerable increased, since it has been applied in many geotechnical structures throughout the world. Rockfill is commonly used in road and railway embankments, railway ballast and earth and rockfill dams. Whether its applied in concrete face rockfill dams or embankments for roadways or high-speed train lines, a precise prediction of expected deformations is needed. Since rockfill behaviour started being investigated that significant differences between rockfill and soils behaviour and characteristics were pointed out. A main difference is related to particle breakage, which depends on the strength of particles, particle size distribution, particle geometry and degree of weathering, stress level and relative humidity. Nonetheless, the effect of RH on rockfill behaviour has been neglected both in experiments and in constitutive modelling despite its fundamental effect. The importance of water action, for a wide class of rockfill materials, can be assessed analysing the behaviour of rockfill dams under reservoir impoundment or climatic conditions. This paper presents the results of triaxial tests performed on compacted specimens of a granite rockfill. Different aspects of the rockfill behaviour, important for constitutive modelling, were studied, namely the shape of yielding loci and the dilatancy rules. Dilatancy was described in terms of plastic work input and the shape of the yield locus, in a triaxial plane, was established by different experimental techniques. Yielding loci can be well represented by approximate elliptic shapes, whose major axis follows approximately the K0 line.

Keywords: Rockfill, triaxial tests, yielding loci, dilatancy.

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1. INTRODUCTION

In the second half of the nineteenth century, the use of rockfill as a construction material started with the building of timber-faced mining dams in California [1]. Since then, rockfill material became widely used in many geotechnical engineering applications, such as railroads, airports and railway embankments. Particularly, rockfill dams have been increasingly used due to their inherent flexibility, capacity to absorb large seismic energy and adaptability to various foundation conditions. They also became an economical option since the increasing use of modern earth and rockfill moving equipment and locally available materials.

Simultaneously, embankment settlements were considered one of the major problems and several field observations have been collected concerning the construction and operation of large rockfill dams [2]. During those observations (over many decades) slow accumulation of deformation has been registered, as well as, collapse settlements, mainly after the submersion of the upstream shells or after a heavy rainfall in downstream shells.

The most relevant limitation when testing rockfill materials is the practical impossibility to test the real material, due to their large dimensions. To overcome this limitation, rockfill tests are carried out using truncated particle size distributions (by elimination of the coarser particles), assuming that, nevertheless, these reproduce the actual material behaviour. Since the 60's, some laboratories around the world (particularly, in Portugal, United States, Mexico, Spain and Italy) have developed and built large equipment including oedometers, triaxial test cells, plane strain devices and torsional shear apparatus. The development of such equipment allowed to study the behaviour of several granular materials, with different strength and compressibility, and to characterise the influence of water, for different stress paths and suctions (suction can be related to the water effect on rockfill).

Collapse deformations has been registered by several authors [2-7] during one-dimensional compression tests when rockfill specimens were flooded. This phenomenon is characterised by a fast increase in the vertical deformation, similar to collapse settlements observed in dams, roads, railways and ports. In triaxial tests, partial collapse happened as well, when rockfill was wetted [8, 9] due to the combined effect of mean and deviatoric stresses. This phenomenon is attributed to the breakage of particles due to rock weakening induced by wetting. The reduction of strength of flooded samples was also identified when compared to dry samples [10]. Nobari and Duncan [11] referred the initial water content of the samples as being the most important factor regarding collapse deformation. It is responsible for determining the amount of collapse upon flooding, i.e., the larger the initial water content, the smaller the collapse deformation. They also observed that the flooding of an initially dry specimen, with a given stress state, caused deformations of collapse. Moreover, increases in loading after flooding this specimen, showed similar stress versus axial strain curves and volumetric strain versus axial strain curves to that of a sample flooded in the beginning of the test. Veiga Pinto [12] studied low strength materials in saturated conditions and performed several tests with isotropic consolidation observing the same behaviour. He also observed that increasing the confinement stress increased the contractive behaviour of the material, during the shear phase, and reduced the initial tangent Young's modulus of the stress-strain curve.

Tests performed in large scale triaxial apparatus showed that the dependence, between the strength envelope of rockfill materials and suction, is more perceptible in materials susceptible to particle breakage (schist and shales). On the other hand, limited sensitivity to water content changes was registered for hard, tough lithologies, with isotropic properties like limestones [13].

Oldecop and Alonso [2] performed oedometer tests of compacted slate gravel with relative humidity (RH) controlled conditions. They studied the effect of water on the macroscopic rockfill behaviour applying the concepts of fracture mechanics in the phenomenon of crack propagation in individual particles. They showed that, by controlling RH in a continuous manner, they were able to develop collapse deformations. This work was continued by Chávez and Alonso [14] and Chávez [15] which presented the results of RH control triaxial tests on the same compacted slate gravel and demonstrated the fundamental role of RH.

This paper presents the results of triaxial tests performed on compacted specimens of a granite. Different aspects of the rockfill behaviour, important for constitutive modelling, were studied, namely the shape of yielding loci and the dilatancy rules. Dilatancy was described in terms of plastic work input and the shape of the yield locus, in a triaxial plane, and was established by different experimental techniques. Yielding loci can be well represented by approximate elliptic shapes, whose major axis follows approximately the K0 line.

2. SUCTION-CONTROLLED TRIAXIAL TESTS

2.1. Specimens preparation

Oldecop and Alonso [2] studied the rockfill behaviour under vertical loads and Chávez and Alonso [17] extended the study to triaxial loading. They showed the influence of relative humidity, inside the rock pores, in the collapse behaviour of granular materials. The relative humidity controls the deformability of the granular materials [2]. Important changes in the relative humidity conditions lead to decrease in strength and increase in compressibility, which characterises the collapse settlements and deferred strains in time [18]. Due to rockfill permeability, the water is mainly stored in the accessible rock pores, instead of the voids between particles, unless the structure is submerged. Relative humidity conditions change mainly by rain, climatic variations, and reservoir filling.

For the triaxial tests, each specimen was compacted in eight identical layers, making a total height of 710 mm for a 308 mm diameter. The granular matrix, within the specimen, resulted in important voids between particles, which added to the particle irregularities and shape, often led to membrane punching during the test. For higher confinement stresses this effect was even more severe. Therefore, depending on the desired stress path, the specimen had to be involved in a double neoprene membrane with 1 to 2 mm thickness or, for higher stresses (> 700 kPa) a single steel wire mesh of 1 mm thickness would be placed between them. It is acceptable that the first membrane could be perforated and the second one ensured the water-tightness of the specimen [12].

In the top and bottom of the specimen hardened steel platens (30 mm thickness, 308 mm diameter) were placed and, between them and the specimen, a porous disk (10 mm thickness, 308 mm diameter). In order to avoid the discontinuity between the granular material and the porous disks, and to promote a better stress distribution between contacts (preventing excessive crushing of particles), a protection layer was also implemented. As in the oedometer test, this protection layer was constituted by gravel of the same material, with sizes ranging from 1/2” to 3/8”, with known weight. The neoprene membrane was fixed to the hardened steel platens (top and bottom) using a high strength steel strap, to achieve the water-tightness of the specimen.

The layers were compacted using a hardened steel platen (20 mm thickness, 300 mm diameter, 100 N weight) connected to a vibrator with the specifications presented in Table 1. Each layer was compacted during 12 minutes, which was found to be enough to obtain almost 100 % of maximum settlement [12].

Table 1 – Characteristics of the vibrator used to compact the triaxial and oedometer specimens. Adapted from [12].

Manufacturer	A. B. Vibro Verken, Sweden
Nominal frequency of vibration	2850 RPM
Dead weight	354 N
Rotational weight	2.5 kN
Diameter of the platen	150 mm

Suction control was attained using an air-flow circulation system developed in LNEC. The air from the saturated solutions was introduced in the bottom platen, circulated within the specimen and collected in the top platen closing the circuit. By allowing the saturated solution to reach equilibrium (by controlling its weight) it was possible to dry or wet the specimen, to a specific value of RH.

2.2. Dilatancy

Several researchers [17-19] stressed the importance of dilatancy during shear to characterise the behaviour of rockfill materials and develop elastoplastic constitutive models. Chávez and Alonso [17] performed a series of large scale suction controlled triaxial tests and developed a hardening plasticity model for rockfill. The Cam Clay model [20] is a reference for frictional materials that dissipate energy during compression and shear, such as rockfill. Considering the original associated model, dilatancy is given by:

$$d = \frac{\dot{\varepsilon}_v^p}{\dot{\varepsilon}_s^p} = M - \eta \tag{1}$$

where $\varepsilon_v^p = \varepsilon_1^p + 2\varepsilon_3^p$ and $\varepsilon_s^p = 2/3(\varepsilon_1^p - \varepsilon_3^p)$ represent the volumetric and deviatoric plastic strains and $\dot{\varepsilon}_v^p$, $\dot{\varepsilon}_s^p$ their rates, η is the stress ratio, q/p, and M is the slope of the critical state line.

In a rockfill material two fundamental phenomena influence deformation: the contact (asperities) breakage and the particle crushing, with the subsequent rearrangement (rotations and particles displacements) of the granular structure. The dominant effect of these phenomena is characterised by the positive dilatancy rates registered during the loading process.

In order to characterise the effect of confinement stress and vertical deformation, Alonso et al. [19] suggested that the work input could be a suitable variable to combine the contribution of the stress and deformation occurring in the specimens. Firstly, they plotted dilatancy against the plastic work during the tests, W^p, normalised with respect to the mean stress. However, they concluded that if dilatancy was only dependent on the plastic work, it would result in a continuous reduction of dilatancy during isotropic loading. Therefore, in order to obtain an infinite dilatancy rate under isotropic loading (only plastic volumetric deformations occur), they proposed to express dilatancy as a function of the following two variables:

$$d = f(\eta W^p/p, s) \tag{2}$$

where s represents the total suction. Figures 1 to 3 present the dilatancy against the product of the stress ratio, p/q, by the plastic work, W^p, normalised by the mean stress, p. The dilatancy rate decreases towards negative values as η increases, although at the cost of considerably high plastic work values. Furthermore, this dimensionless parameter, $\eta W^p/p$, has the ability of considering the effect of confining stress, inhibiting dilatancy as it increases.

Alonso et al. [19] suggested the following empirical equation to characterise the observed dilatancy:

$$d = (\alpha + \frac{\beta}{(\eta W^p/p)^2})^2 - \beta^2 \quad (3)$$

where α and β represent constant parameters that reproduce the experimental dilatancy. Figures 1 to 3 present the respective equations, adjusted to the experimental results, separating them by the respective confinement stress, but with different suctions.

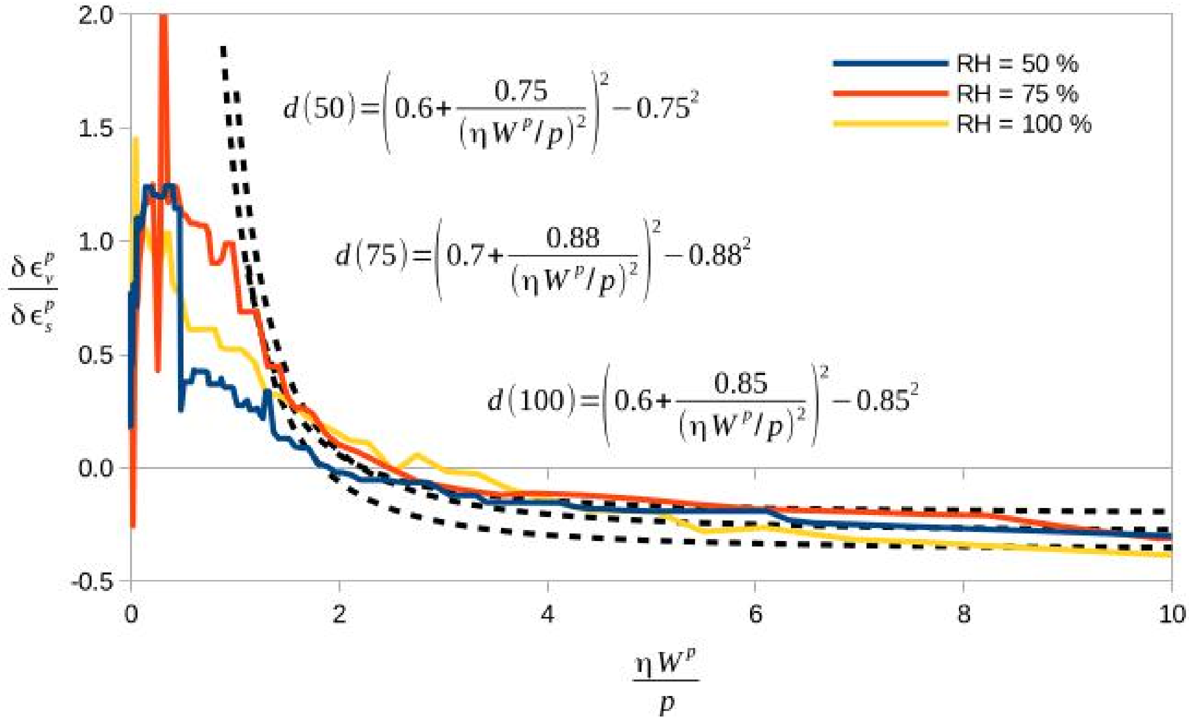


Fig. 1 – Dilatancy rate analysed with normalised plastic work. Tests performed with σ_3 = 0.2 MPa.

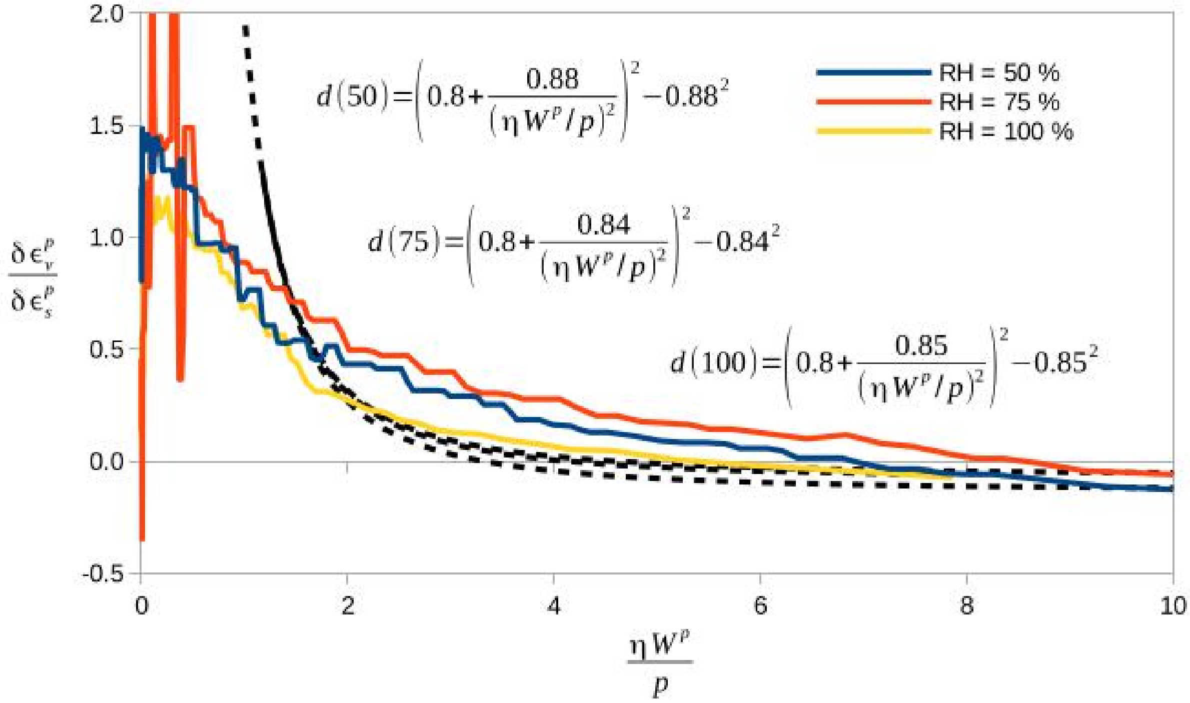


Fig. 2 – Dilatancy rate analysed with normalised plastic work. Tests performed with σ_3 = 0.4 MPa.

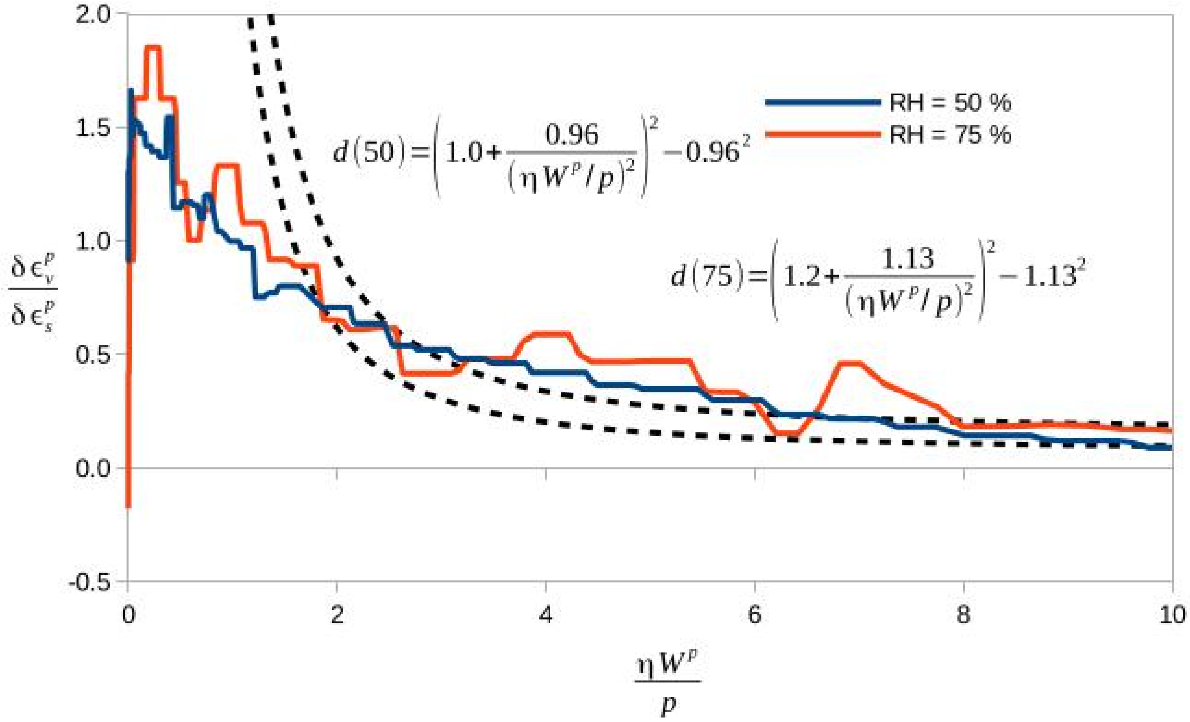


Fig. 3 – Dilatancy rate analysed with normalised plastic work. Tests performed with σ_3 = 0.75 MPa.

It can be clearly seen that suction has a limited influence on dilatancy. Generally, the specimens in equilibrium with a relative humidity of 75 % returned the larger values of dilatancy rate. Nonetheless, the increase in suction led to an increase in negative dilatancy rate, comparing the tests with relative humidities of 50 and 75 % with the flooded specimens. Similar results have been found by other researchers [15, 17, 19]. For Alonso et al. [19] the explanation of this limited influence, in the case of granular materials, might be that small variations in specimen density or particle arrangement may have the same significance as the applied suction.

Several conclusions can be pointed out after this analysis regarding dilatancy rate. The quantity of asperities and particle breakage, that took place within the rockfill material, is believed to be the main factors that control dilatancy [19]. Plastic work can be introduced to assess this quantity, combining together the stress ratio and mean stress. In consonance with the findings of other researchers [15, 17, 19], the effect of suction in dilatancy is slightly noticeable in the granite tested in this work. What is more, its effect is considerably smaller regarding other variables analysed.

2.3. Determination of the yield locus shape

In engineering, yielding separates elastic from plastic behaviour, with irrecoverable deformations. In granular materials, such as rockfill, this separation is rather complex due to changes of void ratio, e [21]. The yielding of a material can be defined by a pronounced change in the curve of the void ratio, e, with the mean stress, p.

Since the 1960s, simple elastoplastic models have been developed, leading to the formulation of the Cam Clay model by Roscoe [22, 23]. This model is described by a flow rule, a plastic potential and a hardening law. Later, several critical-state models have been developed, based on conceptual models of macro or micro behaviour, to simulate the mechanical behaviour of granular materials. They allowed a deep analysis and the prediction of the behaviour of such materials [24]. McDowell [25] presented a new family of yield loci derived using a new work equation and adopting the normality principle, based on specific micro-mechanical assumptions involving particle fracture and frictional rearrangement. By specifying a single parameter along with the friction dissipation constant of the critical state, it resulted in the original Cam Clay yield locus. An appropriate selection of this new parameter can allow the modelling of some yield surfaces, usually determined for isotropically consolidated clays and sands.

Alonso et al. [26] determined the shape of the yield loci of two well-graded granular materials: a quartzitic slate and a hard limestone, for different stresses and suctions. The geometry of the yield loci were described by irregular ellipses adopting the equation proposed by McDowell [25].

To the study of the granite of Montesinho dam, an experimental procedure was adopted to determine the shape of the yielding locus for specimens at equilibrium with three different relative humidities: 50, 75 and 100 %. Figure 4 ilustrates this procedure. It consisted in fixing the position of the yield locus, at a point in the (p , q) plane (point A), through isotropic loading. If the specimen is isotropically unloaded to point 1 and then axially reloaded, following a 1:3 stress path, it will yield at point B, with the loading path extending up to point C. The curved segment AB will be a part of the yielding surface. If the specimen is unloaded, firstly following an axial path to point 1 and, secondly, following an isotropic path to point 2, and then axially loaded to point E, passing by point D where the material yields again, it is possible to determine another point of a different yield surface. If this procedure continues by further unloading and reloading, other curved segments of the yield locus may be added.

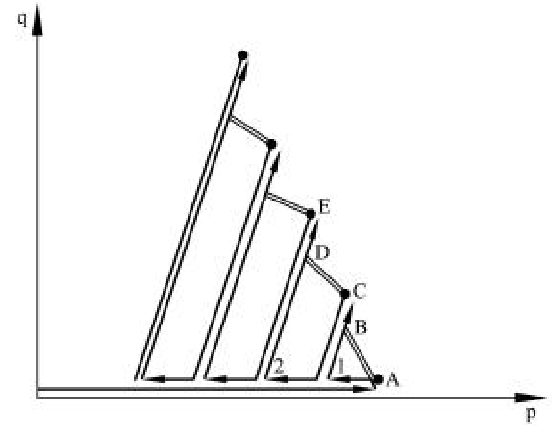


Fig. 4 – Experimental stress paths adopted to study the shape of the yield surfaces [26]

In order to reduce the set of curved segments into a common locus, the yield locus proposed by McDowell [25] was adopted:

$$\eta = q/p = M\left[(a+1)ln\frac{p}{p_0}\right]^{\frac{1}{a+1}} \quad (3)$$

where M represents the value of the stress relation η at the critical state (i.e. the slope of the critical state line), p_0 denotes the pre-consolidation mean stress and a represents a parameter, related to the departure from the shape of the original CamClay model (Figure 5).

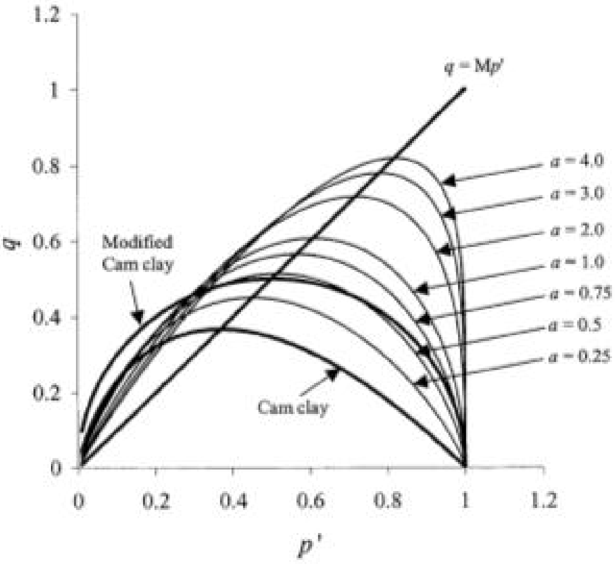
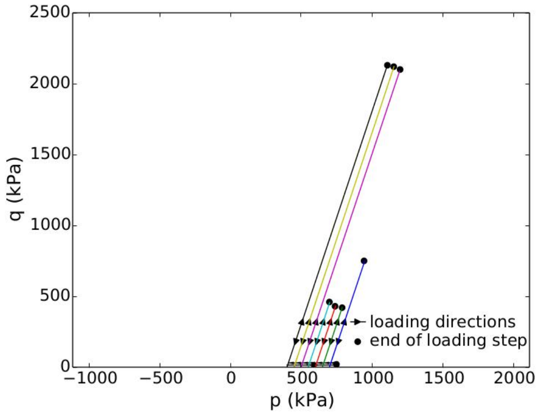


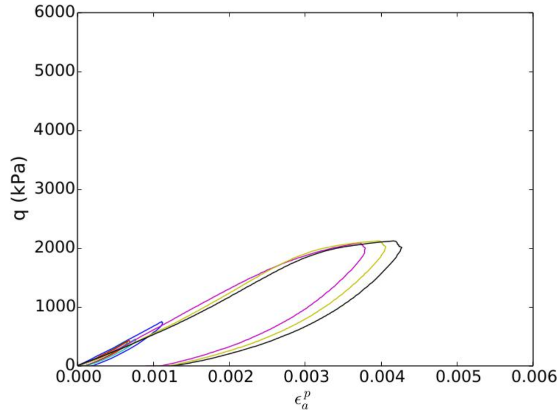
Fig. 5 – Experimental stress paths adopted to study the shape of the yield surfaces

This procedure allowed the determination of the shape of the yielding loci and helped to interpret experimental tests performed under different relative humidity conditions. The main objective was to analyse whether the shape of the yield surface would experience a change when suction decreased (RH increase). This question was approached by conducting three series of tests on specimens of the granite of Montesinho, in equilibrium with three different relative humidities (50, 75 and 100 %), following the same stress path.

Figures 6 to 8 present the stress paths applied to specimens at equilibrium with relative humidities of 50, 75 and 100 %, in the $(p,\ q)$ plane, and the respective specimen response, in the $(q,\ \varepsilon_a)$ plane. Each specimen was isotropically loaded up to 750 kPa (the limit of the equipment - 550 kPa in the flooded specimens due to the back pressure of 200 kPa) and the experimental procedure was performed in order to define the points of the successive yielding locus. The confinement stress was decreased 50 kPa in each step. For each specimen, it was possible to define the shape of the yield locus, for each suction stress, and the experimental procedure was generally performed seamlessly for this stress level.

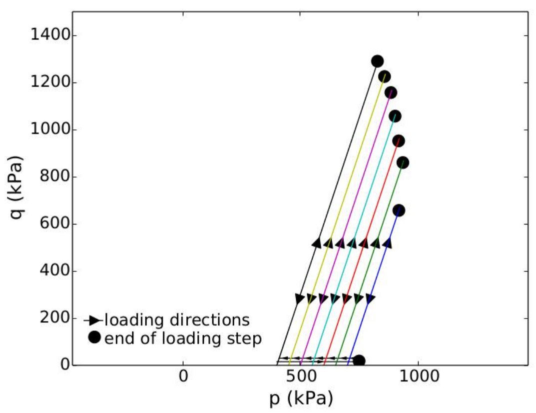


(a) Imposed stress path.

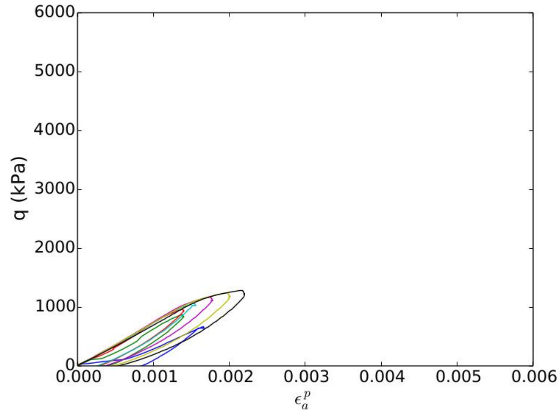


(b) Experimental response.

Fig. 6 – Determination of the shape of the yielding locus at 750 kPa confinement stress. RH = 50 %

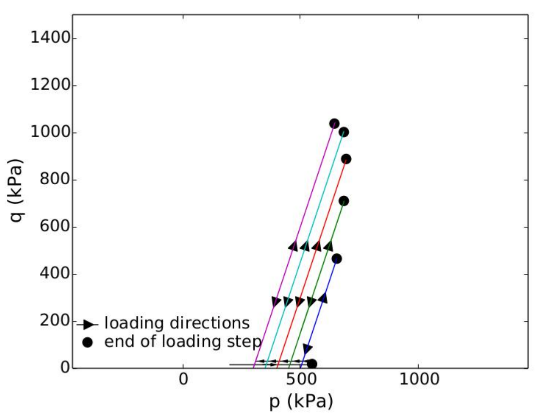


(a) Imposed stress path.

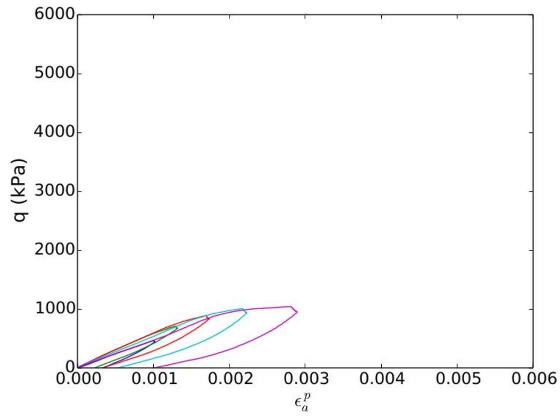


(b) Experimental response.

Fig. 7 – Determination of the shape of the yielding locus at 750 kPa confinement stress. RH = 75 %



(a) Imposed stress path.



(b) Experimental response.

Fig. 8 – Determination of the shape of the yielding locus at 550 kPa confinement stress. RH = 100 %

Every yielding point resulted in with a pair (p_0, a), allowing to apply Eq. 3. Following Alonso et al. [26], in order to determine a set of p_0 values, for each shape of the yield locus, a constant average value of a was adopted. After that, sets of obtained (p, q) values, defining the successive yielding points, were normalised by p_0, resulting in the yield curves plotted in plane $(q/p_0, p'/p_0)$. Figure 9 presents the procedure used to determine each yield compression point along the adopted stress path. The yield points were identified in the curve $v - ln(p)$, at the point where the slope changed.

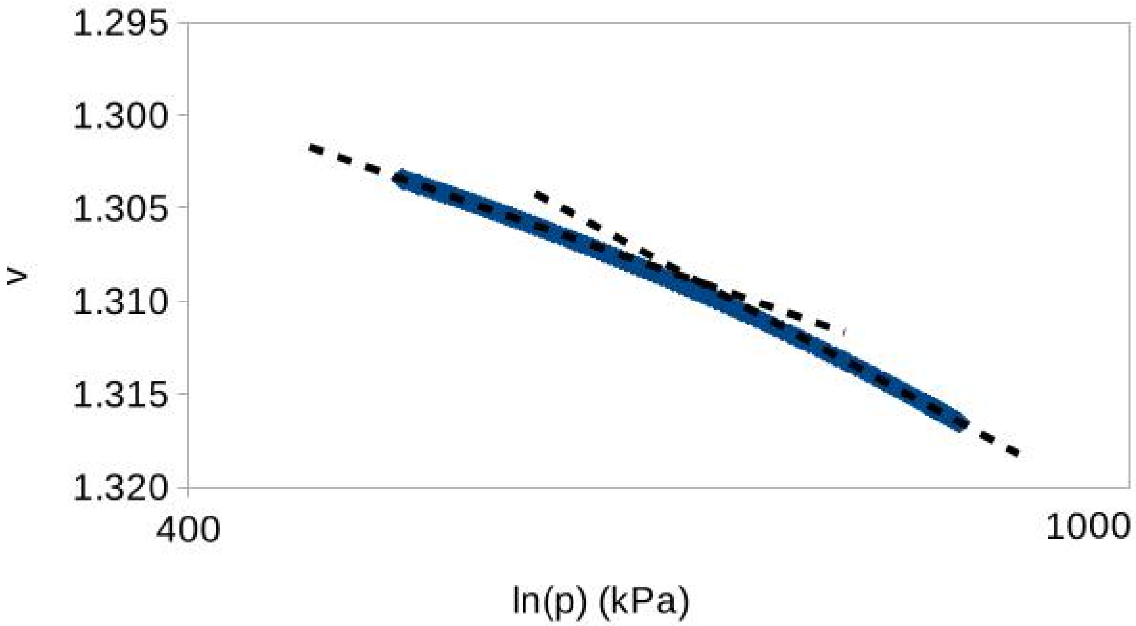


Fig. 9 – Experimental procedure adopted to study the shape of the yield surfaces [26].

Figures 10 to 12 present the normalised yield surfaces for each condition of equilibrium (RH = 50, 75 and 100 %). The points follow a single continuous curve. For Alonso et al. [26], this curve is explained by particle breakage.

For each case, Eq. 3 was estimated based on the experimental data, which is presented in Figures 10 to 12. The yield locus proposed by McDowell [25] performed perfectly for all cases.

The shape of the yield loci was not significantly affected by varying suction. Other studies performed over different materials [26] suggested that a sudden decrease in suction, such as flooding a dry specimen, have led to a shrinkage of the elastic domain. This can be explained by the abrupt increase of particle breakage, triggered by such an event, and the subsequent rearrangement (displacements and rotations) of the particles.

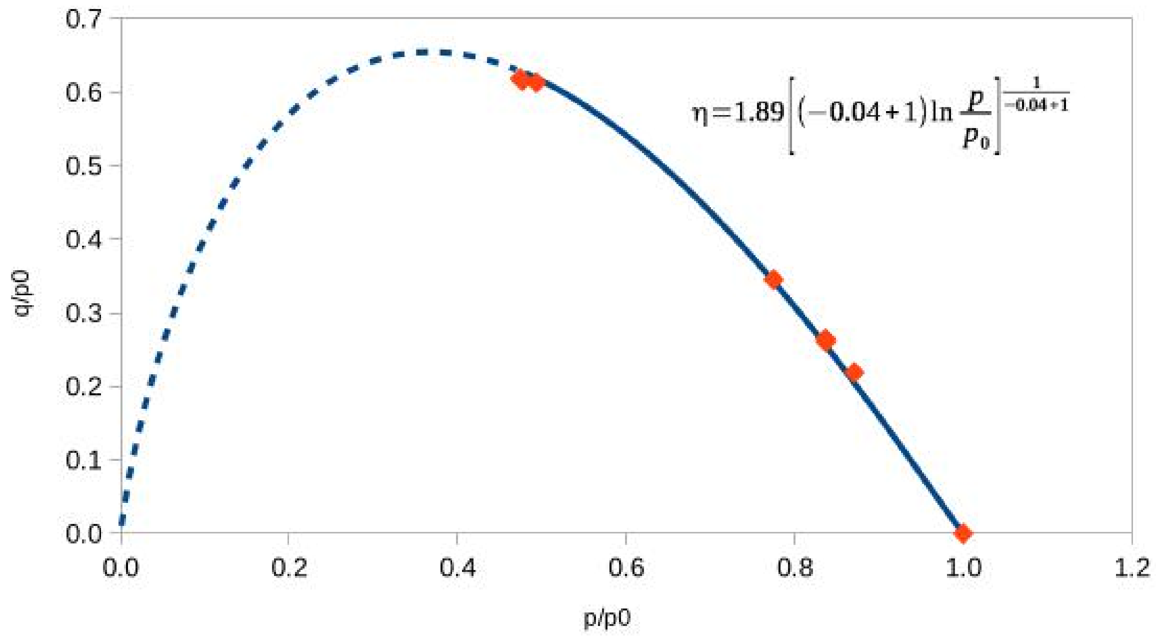


Fig. 10 – Normalised yield surfaces for triaxial conditions. Specimen in equilibrium at RH = 50 %, σ_3 = 750 kPa.

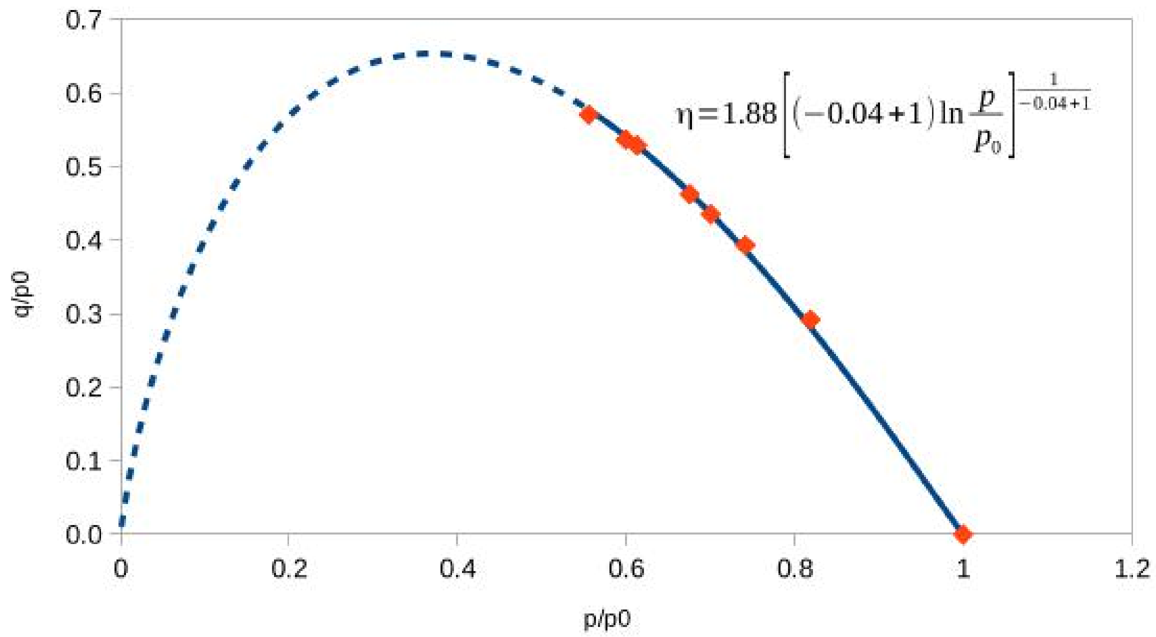


Fig. 11 – Normalised yield surfaces for triaxial conditions. Specimen in equilibrium at RH = 75 %, σ_3 = 750 kPa.

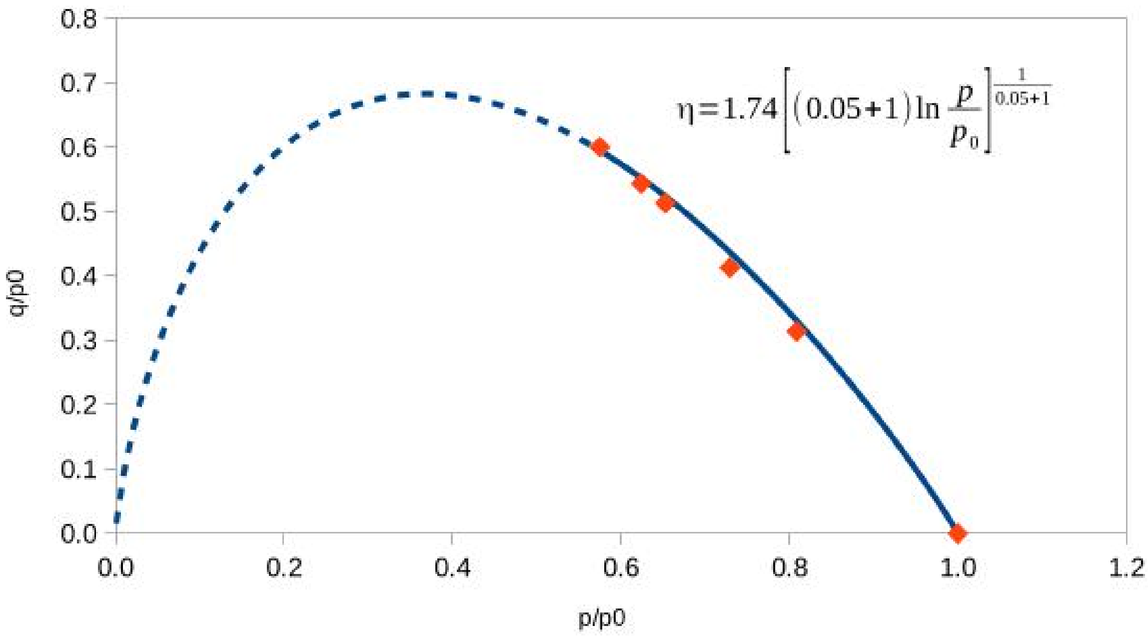


Fig. 12 – Normalised yield surfaces for triaxial conditions. Specimen in equilibrium at RH = 100 %, $\boldsymbol{\sigma}_3$ = 550 kPa.

3. CONCLUSIONS

Rockfill dams have experienced large collapse settlements during reservoir impounding or from direct action of rainfall. When numerical analyses fail to capture the relevant effect of weather conditions, settlements tend to be underestimated resulting in structural loss of serviceability and safety conditions, ultimately leading to considerable costs. The characterisation of this effect in the granite of Montesinho, will eventually allow its numerical modelling and influence the design solutions in further geotechnical structures. Therefore, this paper focused in the rockfill shear strength of the granite of Montesinho by performing an extensive analysis, that included triaxial tests with controlled suction. This work is a part of an extensive work that covered several topics, particularly, the failure envelopes considering the peak strength and the strength at constant volume. Among them the influence of suction on dilantancy and the shape of the yield loci was analysed. Regarding dilatancy, it was analysed considering a dimensionless parameter, $\eta W^p/p$, which had the ability of considering the effect of confining stress, inhibiting dilatancy as it increases, and suction. Results showed a small effect of suction in the behaviour of the tested material. To study the shape of the yield loci, an experimental procedure was implemented, consisting of cycles of increasing and decreasing the confining stress, complemented with cycles of shearing, yielding the material multiple times in order to define the points of the successive yielding locus. The yield points were identified in the curve $v - ln(p)$, at the point where the slope changed. The obtained locci were not significantly affected by varying suction.

Collapse effects induced by full saturation were introduced by means of a numerical algorithm, which reproduces a change in material properties (from a dry state to a saturated state). It requires two sets of constitutive parameters: one set for the dry material and a second one for the wet material.

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