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## THE COHESION OR DILATION EFFECT ON THE SHEAR STRENGTH OF GRANITIC RESIDUAL SOIL

*The paper reports laboratory investigations carried out in granitic residual soil from Covilha, to evaluate the stress-strain-dilatancy behaviour to provide a new model. Some soil tests are intended only to classify soil into broad groups. Other tests are done to examine the mechanical behaviour of soils and particularly to investigate their strength and deformation during loading. Drained and undrained triaxial tests were performed to study soil fabric effects on the mechanical behaviour of a granitic residual soil. It was revealed that the dilatancy of the saturated soil contributes to shear stress and strength at lower confining pressures. The cohesion is apparent for small stress and we need to introduce the evaluation of volumetric strain to understand the effect of the dilatancy in the peak strength.*

**Keywords** Dilatancy • Laboratory Tests • Granitic Residual Soil • Shear Strength

### 1. Introduction

The resulting soil, before the transport has occurred, is named residual soil. Its features are similar to decomposed rocks due to the succeeded structure and fabric, which depends on the alteration degree that occurred. It has specific mechanical properties, depending on the alteration degree, chemical ratio, structure and void ratio. Burland [1] defined the term “structure” of a natural soil as consisting of the spatial arrangement of soil particles and inter-particle contacts named fabric, and “bonding” between particles, which can be progressively destroyed during plastic straining, giving place for the term “destruction”. When the granitic residual soil from Covilha is destructured he has mechanical behavior similar to a granular soil.

Granular materials with different densities respond in different ways to applied shear stress. At low confining pressures, loose sand will compress and dense sand will dilate during the shear. In the present paper it was found that the granitic residual soil at low confining pressures that dilate exhibits, stable its behaviour until the failure surface has been reached. The dilation nature of structured samples decreases with increasing confining pressures, and only contractive behaviour is observed at high confining pressures, like a dense sand mechanical behaviour. The effect of initial soil fabric, which plays an important role at lower confining pressures, is not significant at high pressures where the fabric is controlled by the contraction and the crushing of the grain structure. Structured samples tested under drained shear show a transition from brittle/dilatants behaviour to a ductile/compressive response, as confining stress increases. However, the effects of bonding on

stress-strain-volumetric response of natural and artificially cemented geomaterials revealed that the dilation of the intact soil is inhibited by presence of cement component [2] [3].

The results of a study on the behaviour of granitic residual soil from Covilha at low and high effective confining pressures (25 to 400 kPa) under drained (CD) and undrained (CU) conditions in triaxial compression are presented here. The experimental results show consistent patterns of stress-dilatancy behaviour in both tests. The effect of dilatancy in peak strength appears like apparent cohesion and these concepts are found in critical state observation onto planes deviator stress versus effective medium stress ( $q:p'$ ) and specific void index versus effective medium stress ( $v:ln p'$ ). The critical state line is unique in the  $q: p'$  space, dependent on the frictional effect on shear strength. In the  $v: ln p'$  space the average isotropic state line (ISL) does not seem to be parallel to the critical state line determined from CU and CD triaxial tests but these lines tend to converge at higher confinement stresses. This behaviour is linked to evaluation of the dilatation effects on strength of this soil, implying the evaluation of volumetric deformations and a stress-dilatancy analysis.

### 2. Soil Type and Sample Preparation

All triaxial compression tests in the experimental program were performed on cylindrical samples of residual soil. The soil used in this study was a granitic residual soil taken from a deposit located in Covilha, Portugal. The in situ water content of the soil has 17%. The soil tended to form aggregates larger than 20 mm in diameter

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in the natural state. The classification tests on the soil were conducted in accordance with the procedures described in British Standards Institution [4]. The results are summarized in Table 1.

Identification and physical parameters of the granitic residual soil in Covilha Table 1

Index test	Granitic residual soil
<i>Grain size distribution</i> <sup>(1)</sup>	
Percentage of Gravel	20-38
Percentage of Sand	50-60
Percentage of Silt	9-14
Percentage of Clay	3-6
Effective size, D <sub>10</sub> (mm)	0.007-0.04
Coefficient of Uniformity, C <sub>U</sub> (-)	47.5-200
Coefficient of Curvature, C <sub>C</sub> (-)	1.2-4.1
Specific gravity, G <sub>s</sub> (-)	2.67
<i>(1) Desflocculating used: hexametaphosphate</i>	

The soil comprises 20 to 38% gravel, 50-60% sand, 9-14% silt, and 3-6% clay, which can be described as silty sand with gravel. The natural granitic residual soil belongs to the SW-SM group with gravel [5], with normal or low clayey activity, denouncing the presence of kaolinite with less expansive clay. The classification of soil may be done according to the Unified Classification System if we take precautions or make the necessary alterations on the methodology of the samples preparation and if this system is used together with the geological knowledge of the soils. The granulometric features of these soils have a slight importance, contrary to the sedimentary soils, concerning its mechanical behaviour to an interlocking fabric and degree of weathering [6].

The triaxial remoulded samples 100 mm in diameter and 200 mm in height were prepared by wet tamping method, according to standard proctor compaction procedures. The samples were compacted in a split mould in order to minimize its disturbance, using water contents and dry densities corresponding to the proctor maximum. The samples were compacted with water contents (*w*) of 13% in two groups of samples: one corresponded to the specific volume ( $v = 1 + e$ ) ranging from 1.397 to 1.417 (group: A-nc), and the other group (A-sc) corresponded to the specific volume ranging from 1.601 to 1.621. Probably, we have the loose soil structure formed by the compaction method used in these samples.

Undisturbed samples 100 mm in diameter and 200 mm in height were taken by driving thin-walled samplers in the residual soil. The samples were extruded at steady speed using a mechanical device with plunger of almost the same diameter as the inside sampler diameter applied to the bottom of the sample, in order to avoid disturbance and distortion of the soil. The specific volumes range from 1.409 to 1.556 and the degree of saturation from 50%

to 68%. Fig. 1 shows the specific volume versus degree of saturation of structured and remoulded samples.

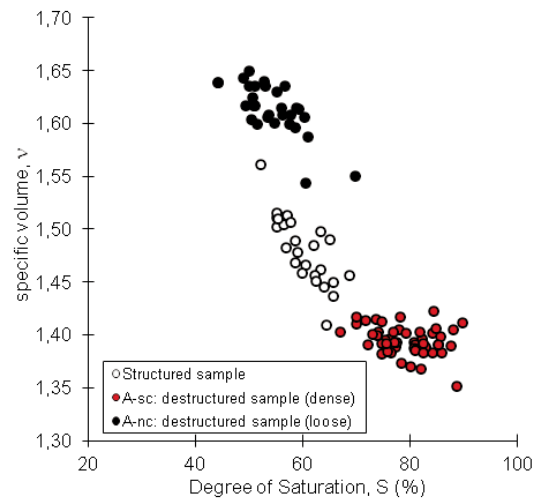


Fig. 1 Specific volume versus degree of saturation of structured and remoulded samples

The results from Fig. 1 clearly indicate high anisotropy of the granitic residual soil.

### 3. Mechanical Behaviour of Granitic Residual Soil

#### 3.1 The Intrinsic Behaviour

It is important to define the intrinsic mechanical behaviour of the soil which is obtained from the tests made on destructured or remoulded samples (group A-nc and group A-sc). The intrinsic behaviour is defined for the state on which the initial physical and structural properties of the soil have no influence on its mechanical behaviour. It's the critical state. On granular soils we need large volumetric compressive strains, where it is possible to get the rate of change as null at constant volume of normal and shear stress.

The projection of critical state line (CSL) onto  $q:p'$  plane may be described by:  $q = Mp'$ , where  $M$  (*capital mu*) indicates the gradient of critical state line. The projection of critical state line onto  $v:p'$  plane may be described by:  $v = \Gamma - \lambda \ln p'$ , where  $\Gamma$  (*capital gamma*) is defined as the value of specific volume ( $v$ ) corresponding to  $p' = 1.0$  kPa on the critical state line in the  $v: \ln p'$  space and  $\lambda$  is the slope of normal consolidation line (negative).

We have now considered separately the failure of samples which were initially isotropically compressed and then loaded in drained and undrained triaxial compression tests. It is striking that the lines of failure points in Fig. 2a appear to be similar for two families of tests and it is instructive to compare these directly. The

average isotropic state line (ISL) in Fig. 2b does not seem to be parallel to the critical state line determined from CU and CD shearing tests and these two lines tend to converge at higher stresses, implying that the state parameter ( $\psi$ ), proposed by Been et al. [7] for quantitatively measuring the dilatancy of soil, would probably decrease as the stress level increases.

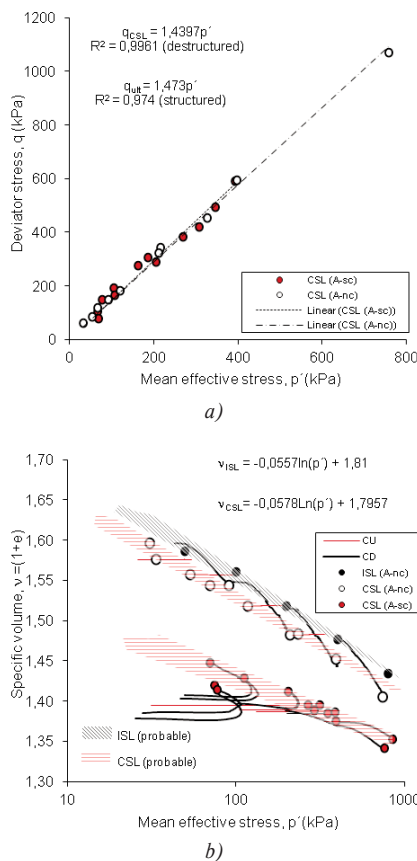


Fig. 2 Failure points for drained and undrained tests on granitic residual soil (loose and dense samples): a)  $q:p'$  space; b)  $v:lnp'$  space

He also defined the intrinsic compression line (ICL) as the one-dimensional consolidation slope of granitic residual soil that had been reconstituted from a liquid limit ( $w_L = 30\%$ ). By reconstituting the sample at high initial water content, the soil ideally loses all memory related to the soil structure [8]. Fig. 3 presents the “unique” ICL when the data were normalized by Burland (1990) parameter ( $I_v$ ) calculated by the following equation,

$$I_v = \frac{e - e_{100}^*}{e_{100}^* - e_{1000}^*} = \frac{e - e_{100}^*}{c^*} \quad (1)$$

which is based on the constants of intrinsic compressibility,  $e_{100}^*$  and  $e_{1000}^*$  (void ratio corresponding to effective vertical stress of

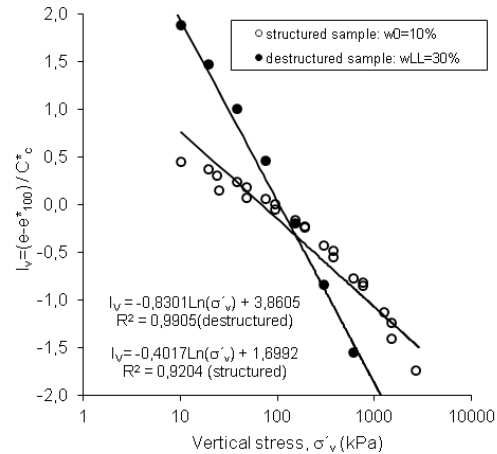


Fig. 3 Structured and destructured samples normalized with Burland's void index parameter [1]

$\sigma'_v = 100$  kPa - yield stress - and  $\sigma'_v = 1000$  kPa), and  $C_c^* = e_{100}^* - e_{1000}^*$ . Compared with a structured sample of granitic residual soil, it may, in fact, not be a truly intrinsic parameter of soil, but it is dependent on the sample fabric, alteration degree and preparation.

The measured high compressibility was probably due to the presence of crushable feldspar in the soil and the soil structure. The mineralogy and the fabric and, subsequently, the corresponding soil properties, may influence the degree to which the initial water content affects the compression curve of a structured soil and the intrinsic compressibility. This is responsible for a different CSL in  $v:lnp'$  space when the samples are sheared.

### 3.2 Stress Behaviour and Peak Strengths

Isotropic consolidated undrained and drained triaxial tests were performed on structured soils with the objective to get a complete understanding of the stress-strain-strength-dilatancy behaviour. The saturation of each sample was ensured by water flow followed by application of back-pressure. Radial and base drainage were adopted and 98% consolidation was obtained in less than 9 minutes. The rates of shearing adopted were 0.04 and 0.01 mm/min, respectively, for undrained and drained tests.

As it is shown in Figs. 4a and b, the stress-strain results of the drained test on the structured sample, the shear stress exhibits a peak in its  $q: \epsilon_a$  (deviator stress: axial strain) curve, and, therefore,  $q$  decreases and is still decreasing at the end of the test. The sample contracts slightly initially, but then expands strongly until the end of the test on the low confining pressure. The shapes of  $q: \epsilon_a$  curves for the undrained test are similar, though the values of  $q$  at failure are very different. The difference in pore water pressures at failure is the major cause of the large difference in the observed shear strength. As shown in Fig. 4a, the most stress-strain curves from undrained tests display peak deviator stresses and tests at

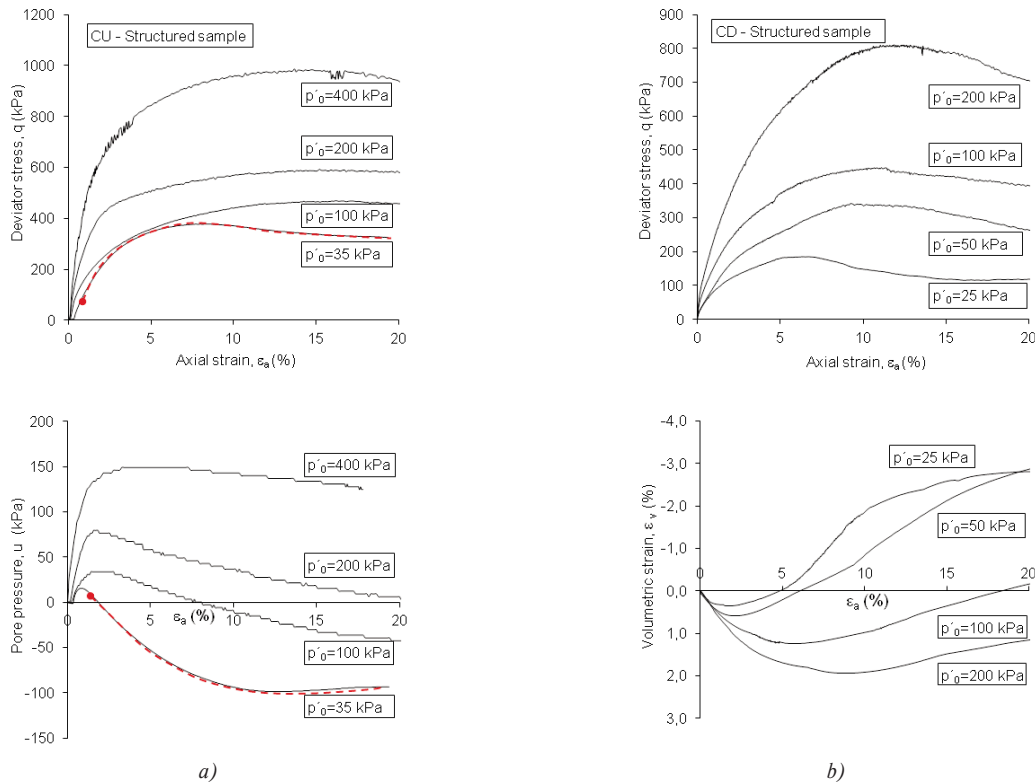


Fig. 4 Structured samples: a) stress-strain relationship; b) relationship between volumetric strain, excess pore-water pressure and axial strain

low confining stresses display negative pore pressures. The loss of shear strength was accomplished with an increase in the pore water pressure. In terms of energy it is suggested that the total work done by the stresses at the boundary of an element is partially dissipated in friction and partly in disrupting the structure of the soil.

This is a typical behaviour of sands. This pattern of behaviour is similar to that observed for clay, for overconsolidated samples of clay expand during shear and generate negative pore water pressures. This behaviour in structured soil is dependent on consolidation stress and variable bonding.

The effective deviator stress at failure in loose and dense remoulded samples for high confining stress is substantially larger than that for low confining stress; the ratios of principal stresses  $\sigma'_1/\sigma'_3$  are, however, almost the same for both cases. In structured samples, as shown in Figs. 5a and b, the ratios of  $\sigma'_1/\sigma'_3$  are not the same for both cases. For  $p'_0 = 25$  kPa in CD test or  $p'_0 = 35$  kPa in CU test, the ratios increased due to the dilation effect, having the peak about 2 and 4 % of axial strain and decreasing to a simple frictional effect. For high confining stress or during plastic straining inter-particle contacts and “bonding” between particles, which can be progressively destroyed and the strength is dependent on a simple frictional effect.

However, in structured samples, if the peak strength is frictional, and the gross yield (GY) represents the onset of major bond degradation, the soil must, therefore, undergo substantial destruction between the gross yield and the peak, as shown in Figs. 5a and b.

### 3.3 The Effect of Dilation on Granitic Residual Soil (Stress-Dilatancy Behaviour)

We have discussed the maximum possible value of  $(\sigma'_1/\sigma'_3)$  or  $(q/p')$  that a residual soil may resist at different states. We may now consider a drained test on granitic residual soil. The stress-dilatancy behaviour of granular soils plays a very important role on strength control; together with connections between particles there is the fabric effect, both leading the mechanical behaviour. In destructured material the peak strength generally coincides with maximum rate of dilation. In bonded materials (structured) the samples show dilation and brittleness at low confining stress, peak does not coincide with maximum rate of dilation. Natural and artificially cemented geomaterials revealed that dilation of the intact soil is inhibited by presence of cement components [9]. Fig. 6 shows the stress-strain relationship of a low confining pressure,  $p'_0 = 25$  kPa, in a drained triaxial test.

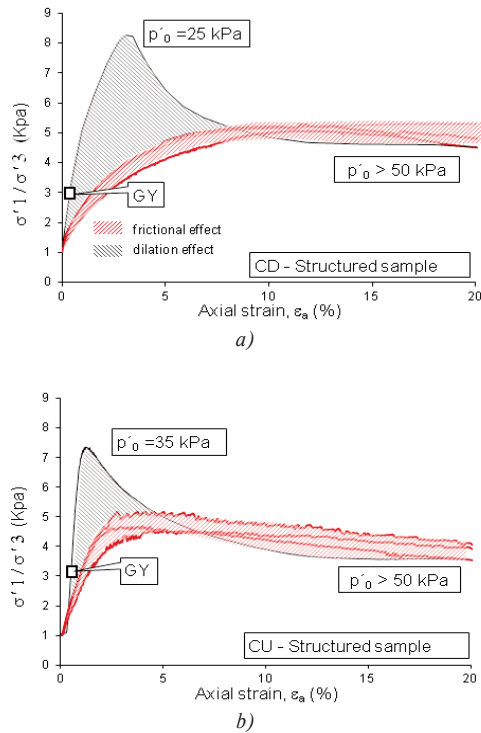


Fig. 5 Effect of initial stress ratio on ductility in structured samples: a) CD tests; b) CU tests

For the peak stress an increment of horizontal displacement  $\delta u$  in a simple shear deformation, the net work transferred in to the sample during the increment and the frictional work is,

$$T'_{yx} A \delta_h - \sigma'_y A \delta_v = \mu \sigma'_y A \delta_h \Leftrightarrow \frac{T'_{yx}}{\sigma'_y} = \mu + \left( \frac{\delta_v}{\delta_h} \right) \quad (1)$$

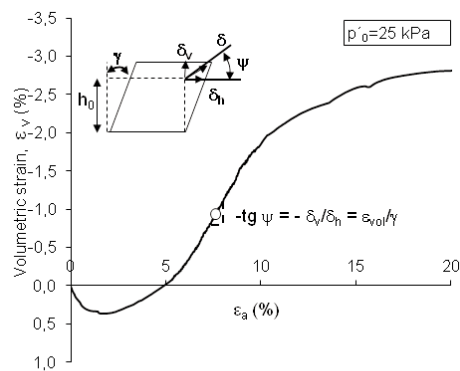
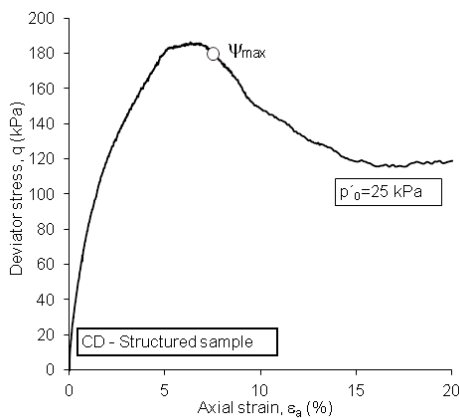


Fig. 6 Stress-strain relationship

We suppose that the coefficient of friction between the contacting faces is  $\mu$ . Now it is tempting to generalize where the invariants  $q$  (deviator stress) and  $p'$  (mean effective stress) are comparable with  $\tau$  (shear stress) and  $\sigma'$  (principal effective stress), respectively [10], and rewrite in terms of the equivalent invariant parameters this stress-dilatancy relationship:

$$\frac{q}{p'} = M + \frac{\delta \epsilon_v}{\delta \epsilon_h} \quad (2)$$

where

$$\eta = \frac{q}{p'}; M = \frac{6 \sin \varphi}{3 - \sin \varphi}; \psi = \frac{\delta \epsilon_v}{\delta \epsilon_h}$$

Each drained triaxial test analyzed gives the same unique straight-line relationship between the rate of dilation ( $\delta \epsilon_v / \delta \epsilon_h$ ) and the stress ratio  $\eta = q/p'$ , which can be calculated using Eq. 2. The stress-dilatancy evaluation was made with projection of maximum stress coefficients ( $\eta_{max}$ ) in function of maximum dilation ( $\psi_{max}$ ) for the destructured sample and the structured sample, as illustrated for granitic residual soil in Fig. 7. As the tests were terminated shortly after the peak, critical states cannot be identified from the stress-strain data, and a stress-dilatancy analyses can then be useful in the understanding of the underlying behaviour of the soil.

The internal frictional angle ( $\phi_p$ ) of peak strengths increase is associated with a dilatancy increase. When  $\psi = 0$  the soil mechanical behaviour is in CSL. The unique relationship in Fig. 7 indicates that the shearing behaviour is purely frictional so that the peak strengths are solely with dilation; the intercepts represents the CSL gradient,  $M$  and corresponds to a critical state friction angle,  $\phi'_{cs}$  of  $36^\circ$  to destructured sample and  $\phi'_{cs}$  of  $38^\circ$  to structured sample. These are unusual high values for sand that is predominately quartz, but this value is no constant over a very wide range of pressures and it may be related to the relatively high propor-

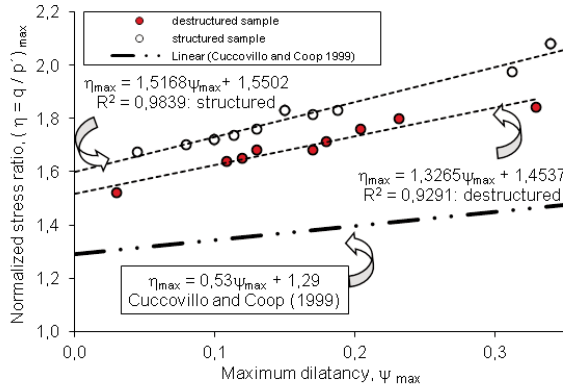


Fig. 7 Stress-dilatancy data for destructured dense sample, structured sample of residual granitic residual soil and the same relationship obtained from Cuccovillo and Coop (1999)

tions of other minerals present and the gradual destruction of the fabric and bonding.

The failure envelopes obtained in drained triaxial shear tests is shown in Fig. 8. Initially, the sample will compress slightly and then expand as the test proceeds and the stress paths move up towards the critical state line. Again, it is clear that the peak strength results from dilation as there is no evidence of a true cohesion interception.

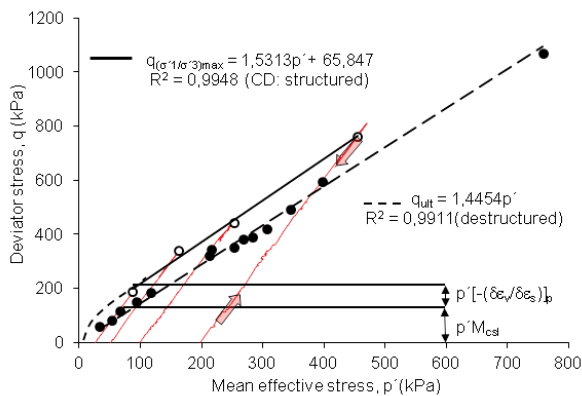


Fig. 8 Effective stress path for drained tests on saturated granitic residual soil: failures envelopes for  $(\sigma'_1/\sigma'_3)_{max}$  and ultimate and intrinsically criterion of rupture

The peak strength is the result of two effects: a) the frictional effect,  $\sigma' \tan \phi'$  or  $(q/p')p = M$  and b) the dilatancy effect,  $\sigma' [-(\delta \epsilon_v / \delta \gamma)]_p$  or  $p' [-(\delta \epsilon_v / \delta \gamma)]_p$ . After the peak, the soil strain softens, apparently following a straight line frictional trend on the stress-dilatancy plot, but as the stress ratio reduces, strain localization occurs so that the rate of dilation reduces more rapidly than the stress ratio, bringing the path inside the expected frictional relationship.

Fig. 9 shows a typical effective stress path of the undrained test in  $q:p'$  stress plane. The volumetric behaviour of the sample can be indirectly reflected from the shape of the effective stress paths. The variation of pore water pressure in loose sand at failure is different from that of structured samples.

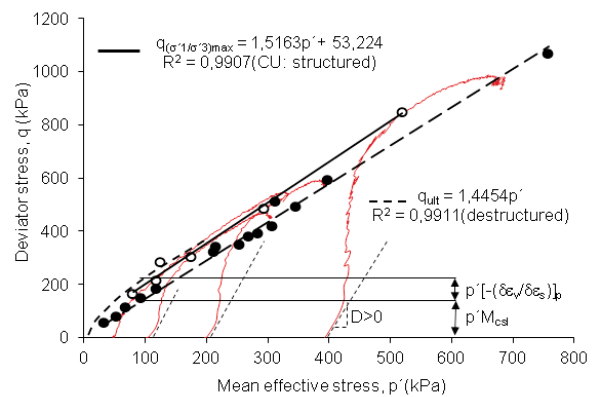


Fig. 9 Effective stress path for undrained tests on saturated granitic residual soil: failures envelopes for  $(\sigma'_1/\sigma'_3)_{max}$  and ultimate and intrinsically criterion of rupture

The loose sample has a positive pore water pressure at failure, while in structured samples this is greater and negative. In structured samples the negative variation in pore water pressure produce an increment in effective stress and the effective stress path moves towards the right-hand side ( $D > 0$ ). This is an indication of the tendency to dilate.

#### 4. Conclusion

The present investigation was performed to conduct a thorough examination of the effect of dilatancy on the drained and undrained stress - strain, volume change or pore pressure, and strength behaviour of intact granitic residual soil. The residual granitic soil contains some quantity of fines and its mechanical behaviour in drained and undrained triaxial shear tests is similar to that of sands for the same relative density and confinement stress.

However the classical models of soil mechanics cannot be used to describe the mechanical behaviour of residual soil, as the strength of the structure is independent of density. Based on the intrinsic compressibility test results presented in this study it appears that the value of  $e^*_{100}$  of the soil is dependent on the structure and initial water content of the soil sample. The intrinsic parameter  $I_v$  may not be a true intrinsic soil property because the strength of the soil depends on the gradual destruction of the fabric and bonding when the loading increases.

The effect of initial structure on the volumetric space dilatancy is most pronounced in the low pressure regime and this con-

tributes to soil strength. Again, it is clear that the peak strength results from dilation as there is no evidence of a true cohesion interception. The linear regression is not a necessarily the best fit. The relation stress-dilatancy of these soils plays a very important role in behaviour control; together with connections between particles there is a fabric effect, both ensuing to the mechanical behaviour. Then the Mohr Coulomb failure criterion is not availed in the simplest space of stresses because it is necessary to use the space of deformations together. The natural structure of bonded soils

has dominant effect on their mechanical response since the apparent cohesion/dilation component can dominate soil shear strength at engineering applications involving low stress levels. The interpretation of peak strength behaviour of this soil needs the combination:  $\phi_{peak} = \phi_{critical\ soil} + \psi$ .

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