Influence of non-saturation and particle size distribution on the stiffness of granite materials

Influencia de la no saturación y granulometría en la rigidez de materiales de naturaleza granítica

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RESUMEN

En este artículo se presenta la rigidez de dos materiales con el mismo origen geológico, una arena con limo, llamada arena Perafita (D_{50}=0.36mm) y un agregado granítico (0/12.5, D_{50}=0.52mm). La rigidez para deformaciones muy pequeñas ($\varepsilon<10^{-5}$) de los materiales compactados es estudiada mediante ensayos triaxiales de precisión (con medidas locales de deformación). La influencia de la no-saturación en la rigidez se ha estudiado también mediante ensayos triaxiales con medición de la presión negativa del agua en los poros. Estas medidas permiten el análisis del comportamiento mecánico de materiales no saturados por lo que se refiere a tensiones efectivas. Con este propósito fueron usados dos procedimientos: el de Terzaghi y el de Bishop. Los resultados obtenidos revelan, para cada material, una relación única entre la rigidez vertical y la tensión efectiva vertical (ratio de vacíos constante = 0.5), independientemente del grado de saturación de las muestras estudiadas.

ABSTRACT

In this paper, the stiffness of two materials with the same geological origin is presented, a silty sand, called Perafita sand (D_{50}=0.36mm) and a granite aggregate (0/12.5, D_{50}=0.52mm). The stiffness in very small strains ($\varepsilon<10^{-5}$) of the compacted materials is studied by means of precision triaxial tests (with local strain measurements). The influence of non-saturation on the stiffness is also studied through triaxial tests with measurement of negative pore water pressures. Such measurements allow the analysis of the mechanical behaviour of the unsaturated material in terms of effective stresses. For this purpose two approaches were used: the Terzaghi's and Bishop's approaches. The results obtained show, for each material, a unique relationship between vertical stiffness and vertical effective stress (constant void ratio = 0.5), independently of the saturation degree of the studied samples.

Keywords: Unsaturated soils, Triaxial tests, suction, effective stress, Terzaghi's and Bishop's

1 INTRODUCTION

Nowadays the great importance of the initial Young's modulus or shear modulus (E_0 or G_0) is well established, since it is a reference value for several soil parameter correlations and numerical analyses. The investigation done in this field reveals that in the small strain domain the properties of the soils are not significantly affected by the cyclic, dynamic or static character of the actions. For this reason, the initial modulus of soils can be determined by dynamic (resonant column), cyclic or static tests (precision triaxial test).

The soil behaviour, that interests the serviceability of structures depends on the maximum strain, which differs from work to work. In agreement with Biarez et al. (1999), the strain level is in the order of 10^{-4} in the subgrade soils and unbound granular materials of pavements structures, around 10^{-3} for tunnels, about 10^{-2} for foundations and nearly 10^{-1} for embankments on soft soils. More generally, it can be concluded that the majority of structures, including buildings, presents values of strain in the order of 5×10^{-3}. Therefore, the domain of the small strains is very important for the serviceability design of the structures. The linear elastic domain is generally found for strain levels lower than 10^{-5}. From this value up to around 10^{-4}, the behaviour of soil is reversible but not linear, which means that the permanent strains are negligible although, in a loading-unloading cycle the stress-strain curve presents hysteresis (Jardine et al., 1984).

The main factors that affect the initial Young's modulus of the granular materials are the void ratio and stress level. The influence of stress level is given by a power law. In agreement with several authors (Jiang et al., 1997; Hoque and Tatsuoka, 1998; Gomes Correia et al., 2001) the power law that describes the vertical Young's modulus (E_v) in the very small strain domain is only a function of the vertical stress, σ_v, and not dependent on the horizontal stress, σ_h, and it is given by equation (1). Concerning the void ratio function, equation (2) proposed by Hardin and Richart (1963) is usually used.

$$E_v = C(\sigma_v^{'})^n \tag{1}$$

$$F(e) = \frac{(B-e)^2}{(1+e)} \tag{2}$$

Another very important aspect is the evaluation of soil behaviour in terms of effective stresses. When the soil is saturated or with a high degree of saturation, the effective stresses can be evaluated using Terzaghi's effective stress concept (equation (3)). When the soil has a low degree of saturation, it is necessary to use another model, such as Bishop´s. In this simple model, the effective stress is given by equation (4), where χ is a factor that depends on the saturation degree and u_a and u_w are the air and water pore pressures, respectively.

$$\sigma_v^{'} = \sigma_v - u_w \tag{3}$$

$$\sigma^{'} = \sigma - u_a + \chi(u_a - u_w) \tag{4}$$

The previous aspects are used in the present study dealing with the stiffness evaluation of a granite aggregate (0/12.5) by means of precision triaxial tests. Furthermore, this study also analyses the influence of the particle size distribution on the stiffness of two materials with the same geological origin, a granite aggregate (0/12.5, D_{50}=0.52mm) and a silty sand, called Perafita sand (0/6.4, D_{50}=0.36mm).

2 MATERIALS AND TEST PROCEDURE

Table 1 shows the geometric characteristics of the particle size distribution curves for the two materials (granite aggregate and Perafita sand). Figure 1 shows the modified Proctor curves of the two materials as well as the conditions of compaction of the granite aggregate samples.

To carry out the study on the granite aggregate two kinds of tests were done: triaxial tests with measurement of the negative pore water pressure (TTu) to estimate the effective stress, and precision triaxial tests (PTT) to evaluate the stiffness of the material in the small strain domain. All the tests were done in the Soils, Structures and Materials Mechanics Laboratory of the Ecole Centrale Paris (ECP) in France.

Table 1 – Geometric characteristics of the particle size distribution curves.

	Granite aggregate (0/12.5)	*Perafita Sand*
D_{max}	12.5	6.35
D_{10} *(mm)*	0.040	0.033
D_{30} *(mm)*	0.23	0.185
D_{50} *(mm)*	0.52	0.36
D_{60} *(mm)*	0.78	0.560
C_U	20	17
C_C	1.7	1.9

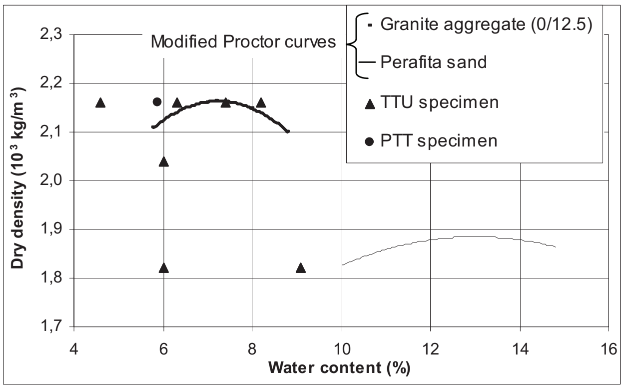

Figure 1- Modified Proctor curves and state conditions of the compacted specimens for the triaxial tests.

2.1 *Triaxial tests with negative pore water pressure measurements (TTu)*

The measurement of the pore pressure was done in a standard triaxial cell modifying the pedestal to accommodate a semi-permeable porous stone (Figure 2-a). This porous stone has the capability to prevent the passing of the air into the water circuit. A thin layer of kaolinite slurry is placed between the soil specimen and the porous stone (Figure 2-b) in order to guarantee a perfect contact of interfaces.

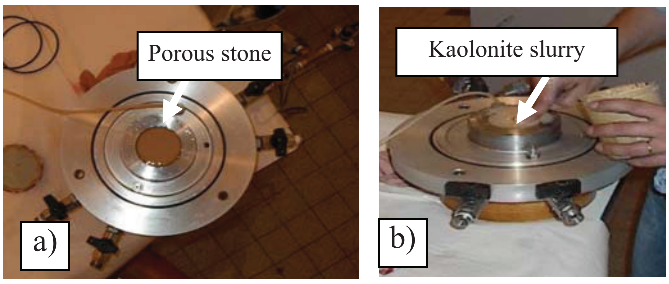

Figure 2 – a) Porous stone; b) – Kaolinite slurry.

The consolidated undrained tests with measurement of pore water pressure were performed in two steps: (i) isotropic consolidation under confining stresses of 0, 26, 52 and 78 kPa (the changes in negative pore water pressure were recorded until equilibrium was reached), and for the last one (ii) axial compression, loading at constant strain rate up to failure.

2.2 *Precision Triaxial test (PTT)*

The granite aggregate, was tested using a single specimen, with 150 mm of diameter and 300 mm of height, through a precision triaxial test. Figure 3-a shows a picture of the axial displacement transducers used for axial strain measurements (LDT - Local Deformation Transducer; Goto et al., 1991). Nowadays, this equipment is also operational at University of Minho in Portugal (Gomes Correia et al., 2006 - Figure 3-b).

The test procedure consists in submitting the specimen to different confining stresses (13, 52, 65 and 78 kPa). For each confining pressure five loading-unloading cycles of very small amplitude were applied. This amplitude was controlled to ensure that the cycles were closed and linear, in order to evaluate the elastic Young's modulus. For the latest confining stress (78 kPa), the sample was submitted to a loading-unloading cycle of great amplitude. During the unloading process, very small loading-unloading cycles of vertical stress were performed at different steps. Figure 4-a shows the test procedure for the latest confining pressure and Figure 4-b illustrates a typical result for a loading-unloading cycle of very small amplitude.

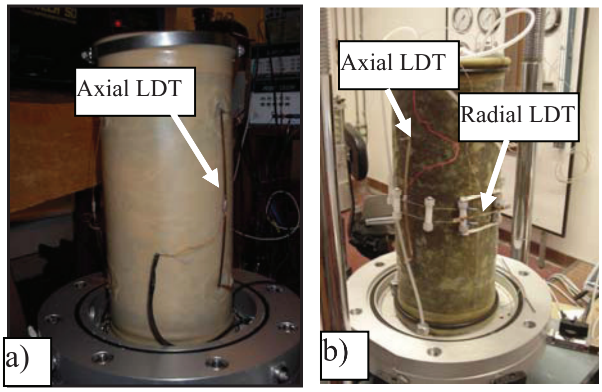

Figure 3– Specimen and LDT's: a) –at ECP; b) – at University of Minho.

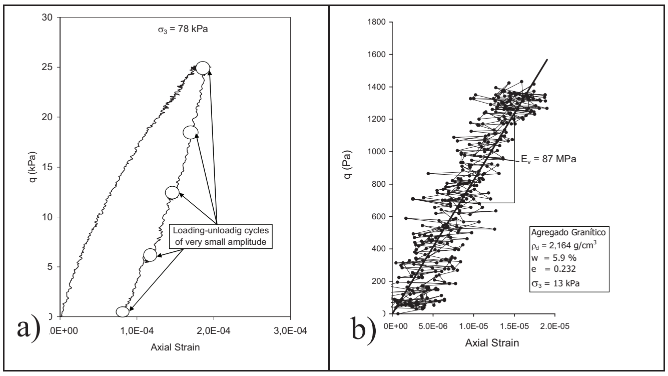

Figure 4 – a) Test procedure for 78 kPa confining pressure; b) Young's modulus (E) evaluation on loading-unloading cycles of very small amplitude.

3 TEST RESULTS

3.1 *TTu*

The aim of the triaxial tests with measurements of the negative pore water pressure was mainly to estimate the effective stresses. For this estimation two concepts were applied: Terzaghi's ($p' = p - u_w$) and Bishop's ($p' = p - \chi . u_w$). Figure 5 shows the average trend line evaluated for all the compacted samples at various water contents and densities (see Figure 1). When analyzing each approach individually, values of 2.29 and 2.31 were obtained for the peak envelopes (ratio between maximum deviatoric stress (q) and effective mean stress (p')), for Terzaghi's and Bishop's approaches, respectively. These values are very similar because the granite aggregate has low negative pore water pressures and the specimens are of low compressibility under the tested water content conditions.

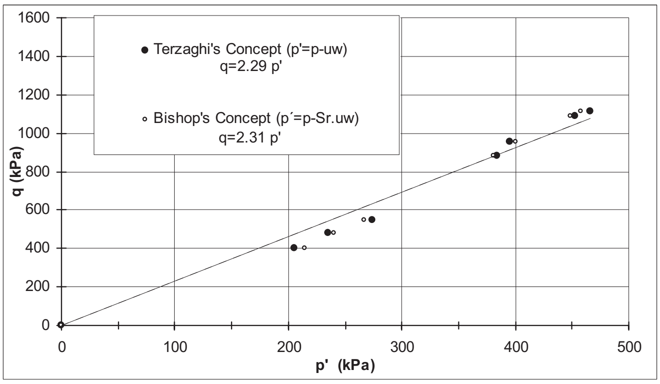

Figure 5 – Peak envelope of stress paths for compression triaxial tests.

Based on these data, the effective stress concept provides a simple means to model the failure criterion of the soil, in agreement with previous analyses (Biarez et al., 1994; Fleureau et al., 2003; Reis Ferreira 2003).

3.2 *PTT*

The very small strain Young's moduli were obtained on one hand by very small loading-unloading cycles in the isotropic state, and, on the other hand, on the unloading path at different anisotropic stress levels (Figure 4-a). Young's modulus (E_V) is calculated as illustrated in Figure 4-b. The analysis has been done both in total stresses and in effective stresses. The values were normalized for a p_a stress, of value equal to 100 kPa.

As shown on Figure 6 the results follow a power law given by equation (5), similar to equation (1), but incorporating the stress normalizing value (p_a). To interpret the results in terms of effective stresses, the negative pore water pressure values were derived from the measurements made in the TTu tests on the specimens at the same water content and void ratio, under the same mean stress (Reis Ferreira, 2003). Once again, the effective stress was estimated using the two approaches.

$$E = C\left(\frac{\sigma'_v}{p_a}\right)^n \tag{5}$$

Figure 6 shows the results obtained in terms of total and effective stresses. As we can see, the granite aggregate behaviour is similar for both approaches. This happens because the values of negative pore water pressure are small, the material is of low compressibility and the saturation degree of the sample is high (69%).

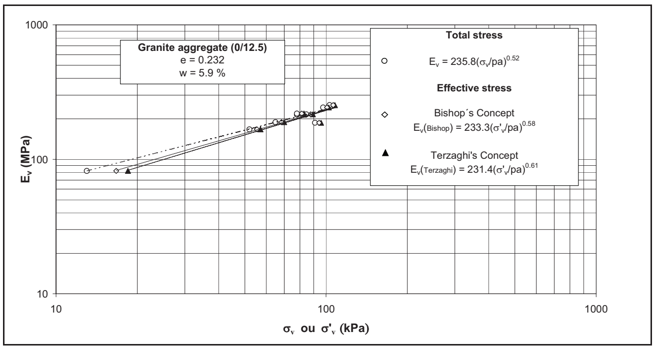

Figure 6 – Influence of total and effective stresses on the elastic Young's modulus.

4 COMPARISON OF STIFNESS IN TWO MATERIALS WITH SAME GEOLOGIC ORIGIN

The influence of the particle size distribution on stiffness was analysed comparing the results obtained on the granite aggregate (0/12.5mm; D_{50}=0.52mm) with the results obtained on Perafita sand (D_{50}=0.36mm). The latter results were presented by Fleureau et al. (2003) and were obtained under different conditions, using different experimental devices, on samples of different sizes and for water contents from 6.5% to 18.3%.

All moduli values were corrected to eliminate the effect of the different initial void ratio (e_0) of the tested specimens. For this, a normalised modulus corresponding to a reference void ratio of 0.5 was derived using expression 2. Following Iwasaki et al. (1978), a value of 2.17 was adopted for the parameter B (equation (2)).

Results presented on Figure 7 show that Perafita sand has Young's modulus values higher than the granite aggregate (0/12.5), for the same effective stress. However, the use of equation (2) to take the changes in void ratio into account must be made with precaution, as shown by Kokusho and Yoshida (1997, referred by Gomes Correia et al., 2001). These authors measured the shear velocity (v_S) of dense gravels having densities varying from very loose to very dense states; they reported that the relationship between the v_s values and void ratios of gravels having largely different uniformity coefficient (Uc) values is not unique under otherwise same conditions, but is largely affected by the gradation characteristics. That is, the rate of increase in E_v with a decrease in void ratio becomes larger when Uc increase. To clarify this point, there is an ongoing study at University of Minho to study the most adequate equation for this kind of materials.

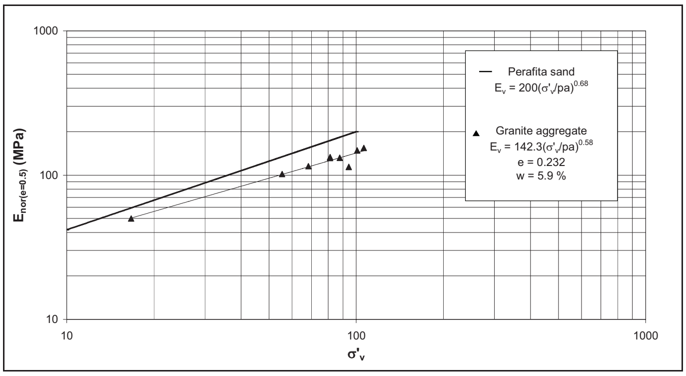

Figure 7 - Influence of particle size distribution on the normalised Young's modulus.

5 CONCLUSIONS

Stiffness properties of two materials with the same geological origin, a granite aggregate (0/12.5, D_{50}=0.53mm) and a silty sand, so called Perafita sand (D_{50}=0.36mm), were evaluated. In the very small strain domain ($\varepsilon<10^{-5}$) the stiffness was determined by means of precision triaxial tests, using local strain measurements (LDT´s - Local Deformation Transducer (Goto et al., 1991)).

The influence of non-saturation on the stiffness was studied through triaxial tests with measurements of negative pore water pressures. Such measurements allowed the analysis of the mechanical behaviour of the unsaturated material in terms of effective stresses. For this purpose two approaches were used: Terzaghi and Bishop. For the granite aggregate (0/12.5) the estimated effective stresses values, using the two approaches, were similar. This was due to the small values of negative pore water pressure and the low compressibility of the material under the water content test conditions.

The results obtained for the granite aggregate (0/12.5) show a unique relationship between elastic vertical Young's modulus and the vertical effective stress. Fleureau et al., 2003 also found for Perafita sand a unique relationship between vertical stiffness and vertical effective stress (constant void ratio = 0.5), independently of the saturation degree of the studied samples. These findings show the relevance of the use of effective stress analysis in the small strain behavior of geomaterials and the importance of negative pore water pressure measurements for the state conditions of the samples.

The influence of the particle size distribution on the stiffness was studied by comparing the results obtained on the granite aggregate (0/12.5 mm; D_{50}=0.52mm) with those obtained on Perafita sand (D_{50}=0.36mm). The stiffness values obtained for Perafita sand were higher than those for the granite aggregate, for the same void ratio and stress level. However, this result must be considered with some reserves, until the question, of which is the void ratio function that best describes the behaviour of materials with very low values of void ratio and high values of Uc, be clarified.

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