Proceedings of the XVIII ECSMGE 2024
GEOTECHNICAL ENGINEERING CHALLENGES
TO MEET CURRENT AND EMERGING NEEDS OF SOCIETY

ISBN 978-1-032-54816-6
DOI 10.1201/9781003431749-113

ECSMGE 24
XVIII EUROPEAN CONFERENCE ON SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

Analysis and modelling of Resonant Column test results on reconstituted specimens of a saturated pyroclastic soil

Analyse et modélisation des résultats d'essais en colonne résonante sur des échantillons de sol pyroclastique saturé reconstitué

G. Nicodemo*, N. Poursalimi, V. Foresta, S. Ferlisi
Department of Civil Engineering, University of Salerno, Fisciano (SA), Italy

**gnicodemo@unisa.it*

ABSTRACT: In the last decades, several efforts have been made by the Scientific Community to properly investigate the role played by the factors that affect the soil dynamic behaviour and define reliable interpretation models. With the general intent to provide a further contribution on the topic, this paper shows the results of laboratory tests carried out with a Resonant Column (RC) equipment on reconstituted specimens of a pyroclastic soil widely diffused in the Campania region (southern Italy) in saturated conditions. In particular, the laboratory tests are specifically aimed at investigating the effects of confining stresses on the soil shear stiffness modulus (G) at either very small or small shear strains (γ). The test results are also used to calibrate the values of parameters involved in modelling the G–γ relationships for the saturated pyroclastic soil under consideration.

RÉSUMÉ: Au cours des dernières décennies, plusieurs efforts ont été faits par la communauté scientifique pour étudier correctement le rôle joué par les facteurs qui affectent le comportement dynamique du sol et pour définir des modèles d'interprétation fiables. Avec l'intention générale de fournir une contribution supplémentaire sur le sujet, cet article montre les résultats d'essais de laboratoire effectués avec un équipement de colonne résonnante (RC) sur des spécimens reconstitués d'un sol pyroclastique largement diffusé dans la région de la Campanie (Italie du Sud) dans des conditions saturées. En particulier, les essais de laboratoire visent à étudier les effets des contraintes de confinement, de l'historique des contraintes et du fluage sur le module de rigidité de cisaillement du sol (G) à des déformations de cisaillement très faibles ou faibles (γ). Les résultats des essais sont également utilisés pour calibrer les valeurs des paramètres impliqués dans la modélisation des relations G–γ pour le sol pyroclastique saturé considéré.

Keywords: Pyroclastic soil; dynamic behaviour; resonant column; shear stiffness; small strains.

1 INTRODUCTION

Soil shear stiffness modulus (G) is one of the key parameters that must be considered to properly address issues concerning geotechnical dynamic analysis and design. For a given soil under certain confining stresses (p), its value is mainly related to the current shear strain (γ) level. In this regard, Hardin and Black (1968) separated the range of shear strains in which the shear stiffness modulus is almost constant (G_0) from that in which G decreases as shear strains increase.

To provide a further contribution to the topic, this paper shows the results of laboratory tests carried out with a Resonant Column (RC) equipment on reconstituted specimens of a pyroclastic soil widely diffused in the Campania region (southern Italy) in saturated conditions. In particular, the laboratory tests are specifically aimed at investigating the effects of confining stresses on the soil shear stiffness modulus (G) at either very small or small shear strains (γ).

The test results are also used to calibrate the values of parameters involved in modelling the G–γ relationships for the saturated pyroclastic soil under consideration.

2 TESTED SOIL AND RC DEVICE

In this work, reconstituted specimens of pyroclastic (ashy) soil originating from the strombolian activities of the Vesuvius volcano (Campania region, southern Italy) were tested.

The reconstituted specimens were prepared according to the following steps (Figure 1). The oven-dried soil was taken and mixed with a fixed water content equal to 15%. Then, the specimen was reconstituted in a cylinder mould (of 140 mm height and 70 mm large) by compacting three layers of soil, utilizing the moist tamping technique, up to attain a target porosity of 53% corresponding to the minimum

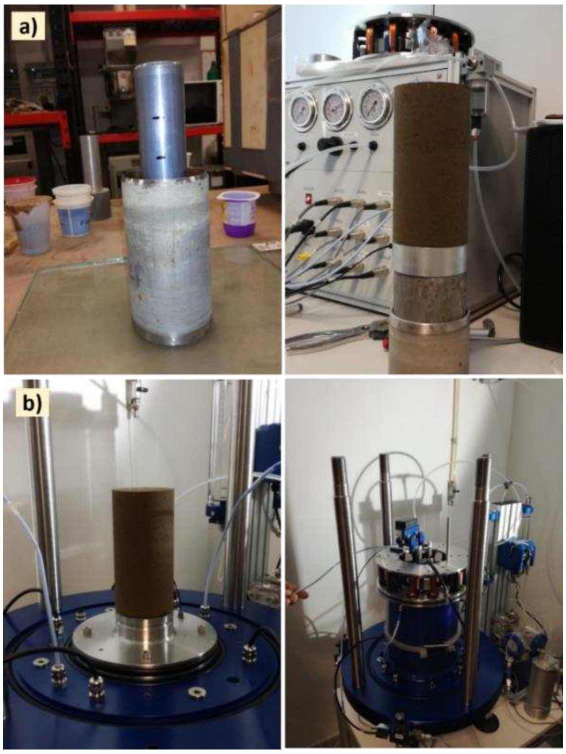

Figure 1. a) Soil specimen preparation; (b) its assemblage within the RCTS equipment.

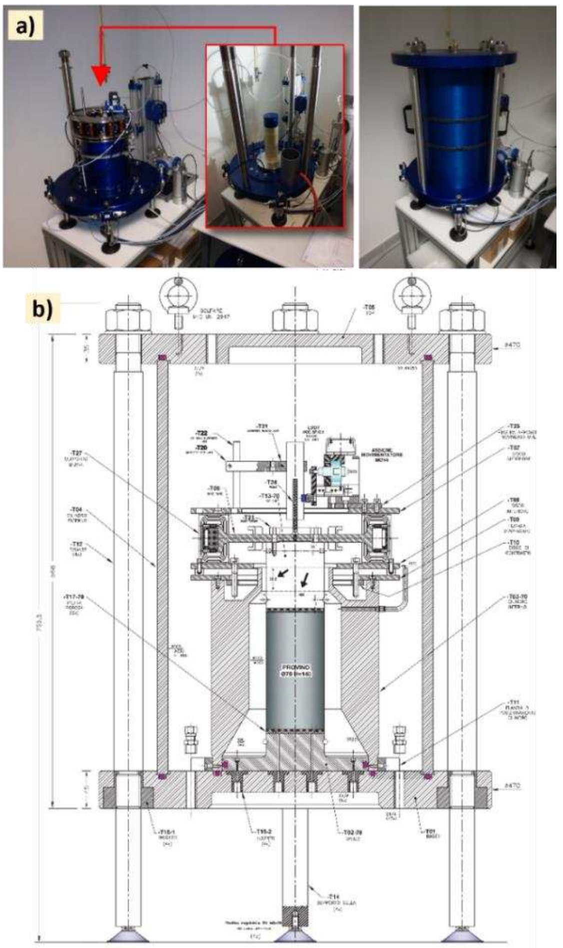

Figure 2. a) Photos and b) detailed scheme of the used RCTS equipment.

value measured in the field (Bilotta et al., 2005).

To reach a uniform distribution of both the water content and matric suction inside the specimen, a time lasting (at least) 24 hr was needed.

The soil testing system consists of a combined suction-controlled Resonant Column-Torsional Shear (RCTS) equipment generally aimed at retrieving the stress-strain behaviour of soil under cyclic loading in either saturated or unsaturated conditions. The RCTS apparatus is comprised of three main parts, namely: the excitation system, the electro-dynamic drive system, and the measurement system (Figure 2).

To pursue the main goal of this work, only the RC component was used. In particular, the reconstituted soil specimen is held fixed at the base. The electrodynamic drive system applies a torsional stress M_t (torque) to the top of the specimen by regulating a sinusoidal electrical signal V (voltage) with a torque/voltage ratio equal to 0.268 Nm/V. The drive consists of a plate with eight magnets, twelve drive coils and a power current amplifier. Each coil is shaped so that the magnets can move as the soil specimen volume changes during the consolidation phase.

3 EXPERIMENTAL TESTS

The experimental tests were carried out on saturated soil specimens to check the performance of the equipment and to gather information about the initial small-strain shear modulus (G_0) and the G–γ relationships.

Once the soil specimen is assembled in fixed-free conditions, the experimental procedure involves considering different steps. In the first step, the saturation is made by increasing the pore water pressure (u_w) at the base of the specimen, from a negative (initially measured) to a positive value, by contextually increasing the cell (p) and the pore air (u_a) pressures step by step (keeping constant their difference at 5 kPa as initial consolidation) until the measured u_w reach the minimum controlling value. The u_w can be measured/applied at the base drainage line while the u_a is applied at the top drainage line of the specimen. In this way, by first regulating u_w to the value reached by u_a and then opening the water drainage valve, the amount of water in the sample is forced to change until the difference between u_a and u_w is zero within the entire specimen. The saturation phase ends when water is visible in the upper drainage line. At this point, the two drainage lines are closed and a B-test is carried out.

As for the consolidation phase (second step) and the dynamic phase (third step), they are differentiated according to the aim of the test. In particular, in the case with the aim of G_0 estimation, gradually increasing confining stresses – at a rate of 4 kPa per hr – from 5 kPa to 200 kPa (loading phase) and vice versa (unloading phase) are applied to the soil specimen. Two RC tests per hr (1 each 30 min) are performed during the ramp of confining stress, with constant voltage amplitude (0.001 V) and variable frequencies (range of 20 to 80 Hz), to find the resonance frequency of the soil specimen. Resonance frequency allows computing shear-wave propagation velocity and the corresponding shear modulus G, while the measured rotation of the specimen free head allows computing the corresponding shear strains.

On the other hand, if the goal is to retrieve the $G–\gamma$ relationship, the soil specimen is subjected to multiple consolidation stages imposing confining stresses equalling 20 kPa, 40 kPa, 60 kPa, 80 kPa, 100 kPa, and 150 kPa. In such a case, the dynamic tests are performed with a range of different voltage amplitudes starting from 0.001 V until a maximum of 2.8 V according to the confining stress (to minimize the destructive effects on the specimen), which allows investigating a range of shear strains from 10^{-4} % to 5×10^{-1} %.

4 RESULTS

The analysis and modelling of test results were focused, separately, on the linear and non-linear soil behaviour based on the attained shear strain level. In the former case, information about the initial shear stiffness modulus (G_0) values was gathered; in the latter case, the soil response in terms of $G–\gamma$ relationship was retrieved. The obtained test results are plotted in the graphs of Figure 3 and Figure 4. As expected, the experimental data highlight that the confining stress has a direct effect on the soil dynamic response – in both linear and non-linear parts – with a general increase of the shear stiffness (G) as the confining stress increases; while G gradually decreases when the applied shear strain (γ) increases.

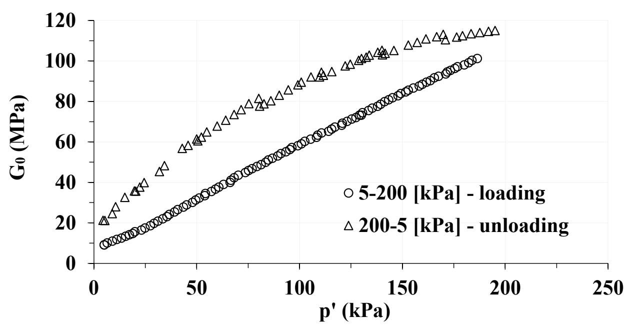

Figure 3. Initial shear stiffness modulus (G_0) vs. confining stress in loading and unloading phases.

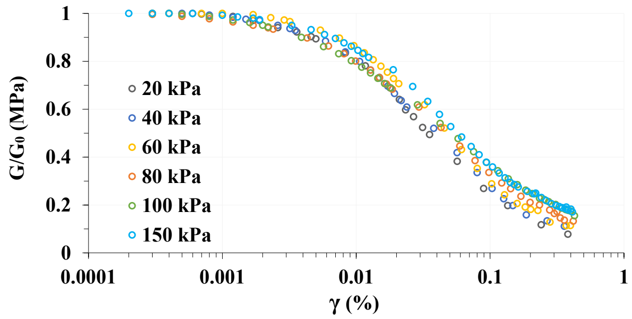

Figure 4. Normalized shear stiffness modulus (G/G_0) vs. shear strain (γ) for different confining stresses.

To model the non-linear soil behaviour, several mathematical formulations were proposed in the scientific literature. In particular, Darendeli (2001) suggested adopting a hyperbolic equation – like the one given by Hardin and Drnevich (1972) – which involves using a reference shear strain (γ_r) at $G/G_0 = 0.5$ and a curvature coefficient (α):

$$\frac{G}{G_0} = \frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}} \tag{1}$$

Figure 5 highlights that the Equation (1) allows modelling with sufficient accuracy the test results shown in Figure 4 for the different confining stresses and over a wide range of shear strains.

Nevertheless, to have a best-fit model of the shear modulus reduction curve for the investigated pyroclastic soil, a modified equation was proposed by introducing a further curvature parameter (β):

$$\frac{G}{G_0} = \left(\frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}}\right)^{\beta} \tag{2}$$

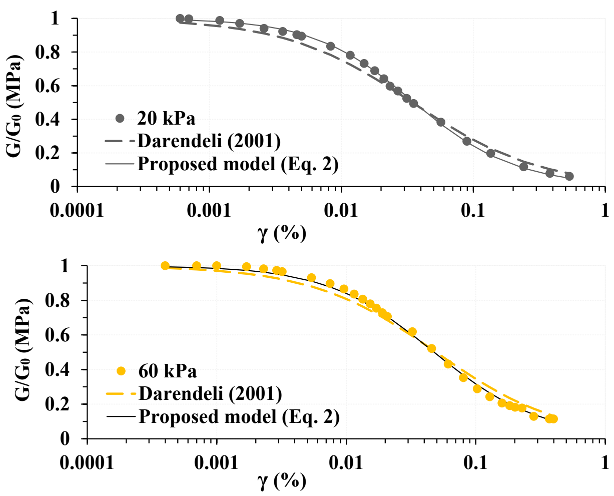

Figure 5. Application of the Darendeli (2001) model and proposed model on the test results in terms of normalized shear stiffness modulus vs. shear strain for different confining stresses values.

In Equation (2), each dimensionless parameter was estimated as a function of the confining stress (p) using Equations (3), (4) and (5) for α, β and, γ_r respectively:

$$\alpha = -7 \times 10^{-5} p^2 + 0.0057p + 1.0747 \quad (3)$$

$$\beta = -1 \times 10^{-5} p^2 + 5 \times 10^{-5} p + 0.8007 \quad (4)$$

$$\gamma_r = -3 \times 10^{-6} p^2 + 4 \times 10^{-4} p + 0.0188 \quad (5)$$

As for the initial shear modulus (G_0), the mathematical formulation given by Rampello et al. (1995) was first considered:

$$G_0 = A \cdot p_{atm}^{1-n} \cdot (p')^n \cdot (OCR)^m \cdot f(e) \quad (6)$$

In Equation (6) p_{atm} is the atmospheric pressure, A is a dimensionless stiffness index, n is a stiffness coefficient expressing the rate of sensitivity of G_0 on the current stress state, p' is the mean effective stress, OCR is the over-consolidation ratio and $f(e)$ is a suggested function of void ratio e ($f(e) = \frac{(2.97-e)^2}{1+e}$).

By plotting the curve relating $f(e)$ vs. e (-), it can be observed that the suggested function does not provide a good match with the test results (Figure 6). Thus, Equation (6) was adapted for the pyroclastic soil under consideration by introducing two new dimensionless parameters (B and C):

$$f(e) = \frac{(B-e)^C}{1+e} \quad (7)$$

On the other hand, the parameter A was calibrated according to the equation (Kokusho,1987):

$$G_0 = A \cdot f(e) \cdot (p')^n \quad (8)$$

Finally, the values of A = 0.18, B = 4.80, and C = 1.89 were obtained. The good match of data coming from Equation (7) to the test results is shown in Figure 6.

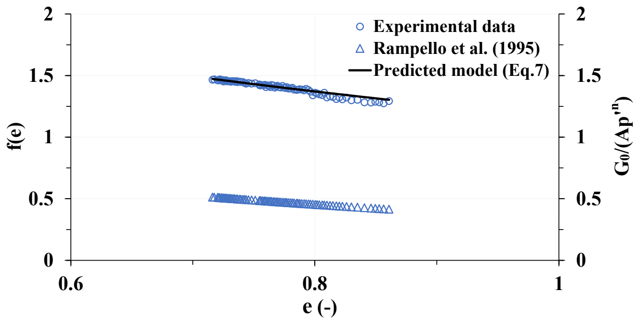

Figure 6. Comparison of the f(e) vs. e experimental data with the mathematical formulation given by Rampello et al. (1995) and the new predicted model proposed in this work.

On the other hand, to model the loading and unloading curves in the G_0–p' graph, the m parameter related to over-consolidation was calibrated too so obtaining a value equal to 0.37 (Figure 7).

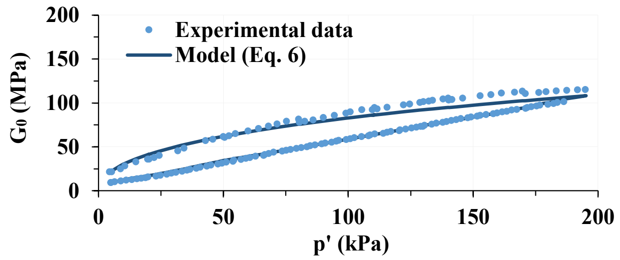

Figure 7. Experimental and modelled $G_0 - p'$ curves.

5 CONCLUSIONS

This paper showed the preliminary results of laboratory tests carried out with a Resonant Column (RC) equipment on reconstituted specimens of a pyroclastic soil widely diffused in the Campania region (southern Italy). The collected experimental data allowed providing new mathematical formulations useful to model either the linear or nonlinear behaviour of the investigated soil in saturated conditions under different confining stresses.

REFERENCES

Bilotta, E., Cascini, L., Foresta, V., and Sorbino, G. (2005). Geotechnical characterization of pyroclastic soils involved in huge flowslides. *Geotechnical and Geological Engineering*, 23:365-402. https://doi.org/10.1007/s10706-004-1607-3.

Darendeli, M.B. (2001). Development of a new family of normalized modulus reduction and material damping curves. *Ph.D. Thesis, University of Texas at Austin,* pp. 362.

Hardin, B.O., and Drnevich, V.P. (1972). Shear modulus and damping in soils: measurement and parameter effects (Terzaghi lecture). *Journal of the soil mechanics and foundations division,* 98(6): 603-624. https://doi.org/10.1061/jsfeaq.0001756.

Kokusho, T. (1987). In-situ dynamic soil properties and their evaluations. In *Proc. of 8th Asian Regional Conference on Soil Mechanics and Foundation Engineering,* Kyoto (Japan), Vol. 2, pp. 215-240.

Rampello, S., Silvestri, F., and Viggiani, G. (1995). The dependence of G_0 on stress state and history in cohesive soils. In: *Pre-failure e-failure deformation of geomaterials – Measurements and application*, IS-Hokkaido, Sapporo 1994, Balkema, Rotterdam, Vol. 2, pp. 1155-1160.