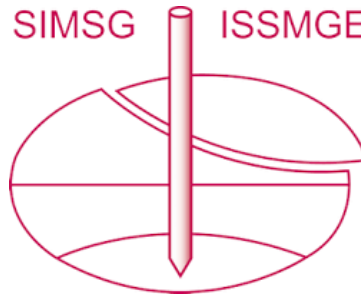


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Dynamic characterization of sands used as backfill in a reinforced earth wall

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ABSTRACT: The paper presents the results of the laboratory tests carried out to characterize two kinds of soils used for backfill of a reinforced earth wall that was built as a prototype in Brazil. To define the mechanical behavior of such backfill a series of static and dynamic laboratory tests were performed on dry reconstituted specimens. The static tests include direct shear tests performed on specimens reconstituted by the pluvial deposition method with different relative densities D_r . For the dynamic characterization, resonant column tests were performed to evaluate the equivalent shear modulus G and damping ratio D . The soil non linearity due to the decreasing of shear modulus and the increasing of damping ratio with shear strain was also evaluated.

1 INTRODUCTION

Reinforced earth walls are cost effective soil-retaining structures that can tolerate much larger deformations than reinforced concrete walls thus they exhibit a good performance during a seismic event. In spite of a very established technology to build reinforced walls, some uncertainties remain in the evaluation of the dynamic soil properties when a seismic response is required.

A full scale model of a mechanically stabilized earth wall was built in Brazil in order to evaluate the performance of such wall in static conditions and during an earthquake loading. The wall was built and instrumented at the Maccaferri establishment at Jundiai, Brazil. It was reinforced using pet straps placed in the soil backfill which were connected to the concrete facing panels. The straps were disposed within the backfill during construction. The backfill of the wall was divided into two parts. This allowed to use two kinds of soil separately as backfill that is a white sandy-soil and a red sandy-silty soil. The wall was instrumented with pressure cells, extensometers and strain gages to monitor the wall in terms of stress and deformations along pet straps and facing panels during construction and under static loading. A more detailed description of the instrumented wall can be found in Jayakrishnan (2013).

In order to make some predictions about the seismic behavior of such wall during an earthquake loading, a finite element analysis was then carried out (Capilleri et al., 2016, Capilleri et al., 2018, Capilleri et al., 2019 this issue). To this aim, the dynamic soil properties, in terms of stiffness and damping of the soil, are essential parameters for any dynamic soil-structure interaction. Since these properties often can be obtained from empirical curves deduced from laboratory tests, a series of static and dynamic tests were carried out at the geotechnical laboratory of the University of Catania (Italy).

2 GEOTECHNICAL CHARACTERIZATION

2.1 Geotechnical proprieties

The geotechnical characterization of the soil used as backfill of the full scale wall was performed taking two different samples from the backfill, that is a sandy soil BRA-1 (white soil) and a sandy-silty soil BRA-2 (red soil) as shown in Figure 1.

The particle size distribution curves of soil samples BRA-1 and BRA-2 were obtained according to the ASTM method for particle size analysis, with a series of standard sieves. (Figure 2)

The average particle size (D_{50}) is 1.00 mm for BRA-1 and 0.1 mm for BRA-2. The uniformity coefficient (C_u), defined as the ratio of D_{60} to D_{10} , is 5.6 for BRA-1 and 7.5 for BRA-2. The curvature coefficient, C_c , is 0.96 for BRA-1 and 0.53 for BRA-2, being C_c defined by the following equation:

$$C_c = \frac{(D_{30})^2}{D_{10} \cdot D_{60}} \quad (1)$$



Figure 1. Backfill behind the wall made with two different soils.

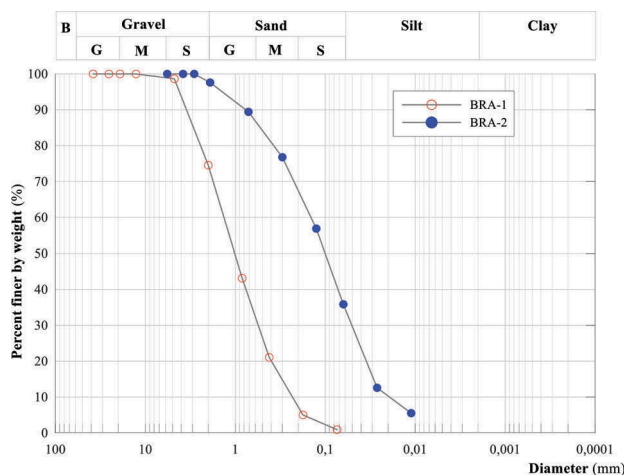


Figure 2. Grading curves for BRA-1 and BRA-2 samples.

Table 1. Geotechnical characteristics of tested samples.

Sample	D_{60} [mm]	D_{50} [mm]	D_{30} [mm]	D_{10} [mm]	C_u	C_c	G_s
BRA-1	1.40	0.90	0.58	0.25	5.60	0.96	2.70
BRA-2	0.15	0.10	0.04	0.02	7.50	0.53	2.74

Table 2. Average geotechnical parameters of shear strength.

	Unit weight γ_{AVERAGE}	Void ratio e_{AVERAGE}	Friction angle ϕ'_{AVERAGE}	Coesion c'
Sample	[kN/m ³]	[-]	[°]	[kN/m ²]
BRA-1	16,5	0,65	42	0
BRA-2	16,7	0,62	32	0

where D_{10} , D_{30} , D_{50} , D_{60} are the particles diameters respectively related to 10%, 30%, 50 % and 60% of passing. The specific gravity of soil, G_s , determined with the ASTM standard test method (D 854), are quite similar, being 2.70 for BRA-1 and 2.74 for BRA-2. Table 1 summarizes the geotechnical characteristics of the tested samples.

The maximum dry density was determined according to ASTM (4253-83). The maximum dry unit weight is $\gamma_{\max} = 16.47 \text{ kN/m}^3$ for BRA-2 and $\gamma_{\max} = 17.50 \text{ kN/m}^3$ for BRA-1. The minimum dry density was determined according to ASTM (4254-83) method which gave $\gamma_{\min} = 12.08 \text{ kN/m}^3$ for BRA-2 and $\gamma_{\min} = 14.61 \text{ kN/m}^3$ for BRA-1.

2.2 Direct shear tests

Direct shear tests were performed to determine friction angles at different relative densities (Banna et al., 2015). Specimens were reconstituted with the pluvial deposition procedure. The vertical stresses used were 100 kPa, 200 kPa, 300 kPa. Shear tests were carried out at different relative density varying from 25% to 90%, using the standard Casagrande box (6x6x2 cm) and a bigger Casagrande box (10x10x2 cm). Table 2 reports the average geotechnical parameters of shear strength that successively were utilized for the dynamic FEM analysis (Capilleri et al., 2019 this issue).

3 RESONANT COLUMN TESTS

The dynamic soil characterization was carried out to determine the dynamic parameters of soil. The equivalent shear modulus G_{eq} and the dynamic damping ratio D were determined by means of a Resonant Column Test (RCT). The Resonant Column Apparatus (RCA) was supplied at the Department of Civil Engineering and Architecture of the University of Catania (Capilleri et al. 2014, Castelli et al. 2016, Cavallaro et al. 2013, Cavallaro et al. 2018). The Resonant Column Test consisted in exciting one end of a confined soil specimen. The size of soil cylindrical specimens are: diameter $d = 50 \text{ mm}$ and height $h = 100 \text{ mm}$. The specimen was fixed at the bottom (fixed-free test) and excited in torsion by means of an electromagnetic drive system (Drnevich et al, 1978, Hall & Richart, 1963). Soil specimens were reconstituted in order to obtain a required relative density, $D_R = 55\%$ for BRA-1 and $D_R = 95\%$ for BRA-2. Specimens were subjected to an isotropic load achieved in a plexiglas pressure cell, using an air pressure source. Generally, the shear modulus G is constant until a certain limit strain has exceeded. This limit strain is known as the elastic threshold shear strain (γ_{te}). It is assumed that soil behaves elastically at strains smaller than γ_{te} . The elastic stiffness at a strain $\gamma < \gamma_{te}$ is defined as G_0 . Once the resonant frequency was determined, the velocity V_s of the propagating

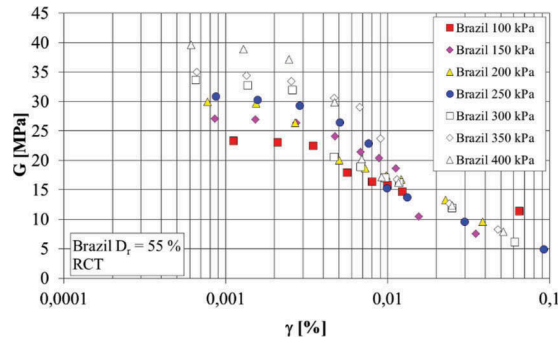


Figure 3. G - γ curves from RCT for BRA-1.

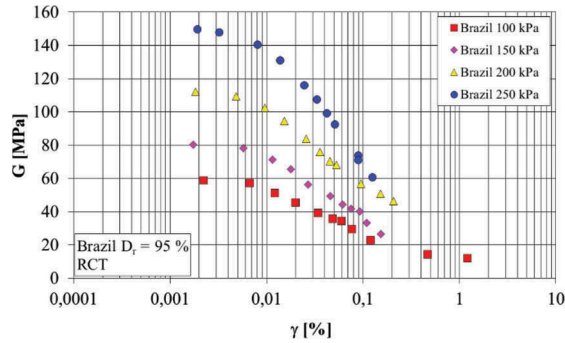


Figure 4. G - γ curves from RCT for BRA-2.

wave and the degree of soil damping were deduced. The shear modulus was then obtained from the shear wave velocity V_s and the density ρ of the specimen (Cascante et al., 1998, Rollins et al., 1998).

Figures 3 and 4 show the shear modulus G change versus the shear strain γ at different confining pressure p' for BRA-1 and BRA-2 respectively.

As expected, the shear modulus G is strongly dependent on the effective confining pressure p' . The elastic threshold γ_{te} was about 10^{-3} % for both samples. At small strains, the shear modulus G_0 is shown in Figure 5. In Figure 6 is reported the value of the damping ratio at small strains D_0 versus the effective confining pressure p' .

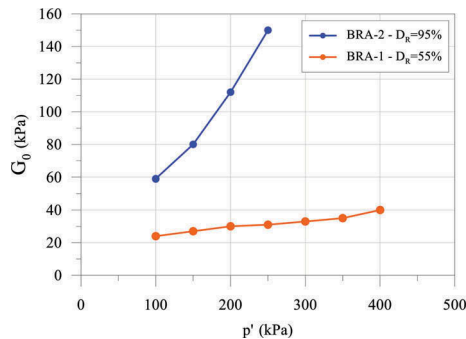


Figure 5. Plots of G_0 versus effective confining pressure p' for BRA-1 and BRA-2 samples.

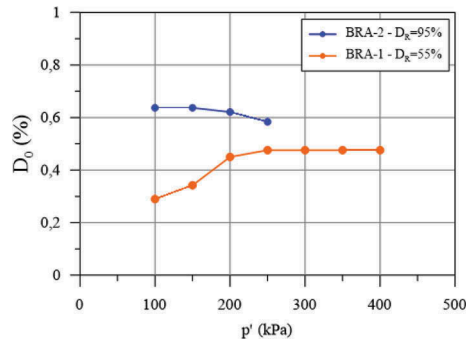


Figure 6. Plots of G_0 versus effective confining pressure p' for BRA-1 and BRA-2 samples.

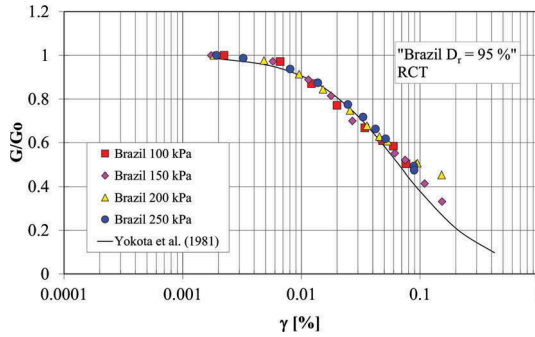


Figure 7. G/G_0 - γ curves from RCT for BRA-1.

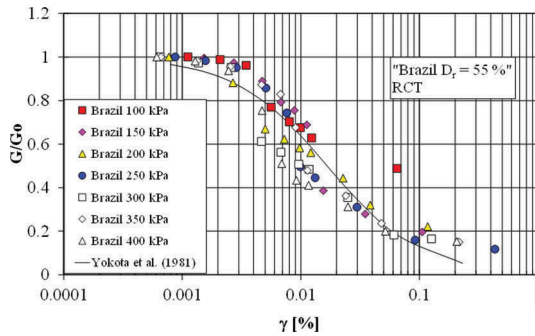


Figure 8. G/G_0 - γ curves from RCT for BRA-2.

The curves of the normalized shear modulus G/G_0 versus shear strain γ for both samples BRA-1 and BRA-2 are shown in Figures 7 and 8 respectively. The experimental results were used to determine the empirical parameters of the curve proposed by Yokota et al. (1981) to describe the shear modulus decay with shear strain, that is:

$$\frac{G(\gamma)}{G_0} = \frac{1}{1 + \alpha \cdot \gamma(\%)^\beta} \quad (2)$$

The damping ratio D was obtained during the resonance condition of the sample. Following the amplitude decay method, it was obtained during the decrement of free vibration

Table 3. Parameters to determine $G(\gamma)$ and $D(\gamma)$ according to Yokota et al. (1981).

Sample	α	β	η	λ
BRA-1	90	1,1	5,2	1,7
BRA-2	26	1,2	4,5	1,75

(Cavallaro et al., 2001). As suggested by Yokota et al. (1981), the damping ratio change versus the normalized shear modulus can be described by an exponential form defined by equation (3), that is:

$$D(\gamma)(\%) = \eta \cdot \exp \left[-\lambda \cdot \frac{G \cdot (\gamma)}{G_o} \right] \quad (3)$$

The parameters α and β in equation (2) and η and λ in equation (3), deduced for the samples BRA-1 and BRA-2, are reported in Table 3:

The damping ratio D versus the normalized shear modulus G/G_o is shown in Figures 9 and 10, for BRA-1 and BRA-2 respectively.

The experimental results for both $G/G_o-\gamma$ and $D-G/G_o$ and the theoretical curves from Yokota et al. (1981), are in good agreement.

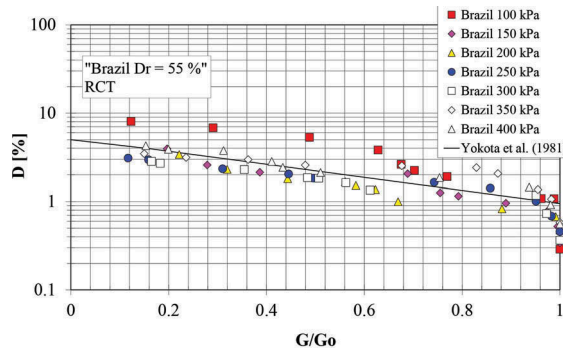


Figure 9. $D-G/G_o$ curves from RCT for BRA-1.

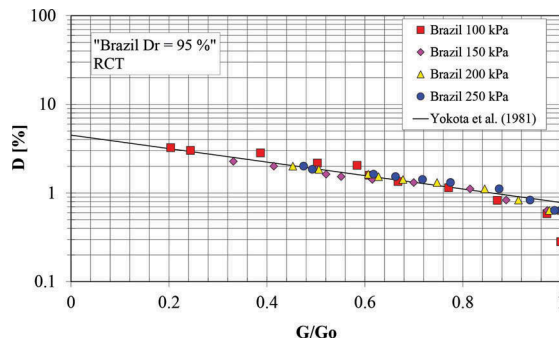


Figure 10. $D-G/G_o$ curves from RCT for BRA-2.

4 CONCLUSIONS

The evaluation of the non-linear behavior of the soil in terms of decay of the shear modulus and increase of the damping ratio with increasing strain level, assumes a relevant role in the seismic response of soil. In order to predict the seismic performance of an experimental reinforced wall a finite element analysis was required. To this aim, a dynamic characterization of the soil taken from the backfill of such wall was performed by laboratory tests. For the dynamic characterization, column resonant tests were carried out to investigate the soil dynamic parameters such as shear modulus G and damping ratio D and their dependence from shear strain at different confining effective pressures p' .

Results from resonant column test enabled to define the small strain shear modulus G_0 trend and empirical equations that have been used to describe the G and D variation with strain level. Based on the experimental results obtained, it is possible to draw that, according to many researches, G_0 values increase with increasing the confining pressure and indicate moderate influence of strain rate at small strain level where the soil behavior is supposed to be elastic. It has been found that the values of G_0 for the sandy-silty soil BRA-2 are greater than the ones found for the sandy soil BRA-1. Probably this was due to the relative density reached during RC test for BRA-2 ($D_R=95\%$) greater than that reached for BRA-1 ($D_R=55\%$). The elastic threshold shear strain was about $10^{-3} \%$ for both samples BRA-1 and BRA-2.

The Yokota et al., (1981) equations are in good agreement with experimental results and the parameters to be used were given for both BRA-1 and BRA-2.

In the range of effective confining pressures utilized, the damping ratio at very small strain D_0 does not present large variation ranging from 0,5%-0,7% for BRA-2 and 0,3%-0,5% for BRA-1.

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