Laboratory investigation on nonlinear dynamic properties of core materials of Italian dams

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Keywords: Italian earth-core rockfill dams; Core material; Modulus reduction curve; Damping ratio curve; Laboratory investigation

Abstract. *This paper presents the results of a series of laboratory investigations into the dynamic properties of core materials of Italian earth-core rockfill dams. All tests have been performed on specimens of undisturbed samples reconsolidated in a wide range of effective confining stresses. Most of the data were obtained from cyclic simple shear tests and few data from resonant column tests. It is shown that the shifting of the* G/G_0-γ_c *and* D-γ_c *curves with plastic index and effective confining stress is not as significant as it is well established for natural fine-grained soils. Generic literature curves do not predict properly the dynamic behaviour of the core materials, especially in the small-to-medium strain range. The importance of conducting site-specific measurements in order to accurately model the behaviour of core materials for dynamic analyses of embankment dams is therefore highlighted.*

1 INTRODUCTION

A peculiarity of the infrastructural heritage of Italian dams is that most of them were built before or shortly after the middle of the last century. That means most of them are more than 50 years old. As the chance to build new dams in Italy is rather limited, consequently there is a strong need for the seismic re-evaluation of existing ones. In addition to their age, this need is dictated by other important reasons. The dams were generally designed using seismic design criteria and methods of analysis (e.g. pseudo-static method) that are considered not suitable today, especially under specific local circumstances (e.g., liquefaction). After the release in 2004 of the updated hazard map of the Italian territory [1], many dams may be nowadays located in areas of higher seismicity as compared to the seismicity considered at the time of their construction. A recent National Code on Dams [2], hereafter referred as NTD2018, has been issued very recently, which dedicates a specific section to the assessment of seismic safety of existing dams.

Different methods can be used for the seismic re-evaluation of existing dams, including pseudo-static, simplified (Newmark) and advanced dynamic analyses. These latter nowadays became extremely common in geotechnical earthquake engineering practice. However, these

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analyses still suffer from several uncertainties that can compromise the results, and consequently, possible intervention strategies. Among these, it is worth to mention the knowledge of: i) history of the dam, especially under earthquake actions ii) material properties under static as well as dynamic conditions iii) input motion and iv) the confidence on the limits of the methodology used.

In this paper, attention is focused on one of the source of uncertainties above mentioned, explicitly related to the dynamic behaviour of the core material of embankment dams. Basic fundamental parameters required for performing advanced analyses are the nonlinear deformation properties, that is the variation of normalized shear modulus reduction (G/G_0) and damping ratio (D) with the cyclic shear strain amplitude (γ_c), G_0 being the maximum shear modulus. Experimental data based on laboratory investigations carried out on samples retrieved from the core of zoned Italian dams are analysed and discussed.

2 ITALIAN EARTH-CORE ROCKFILL DAMS

Six earth-core rockfill Italian dams are considered in this study. These dams are generally constituted by a central core of fine-grained material protected by shells of sand and gravels on both sides.

Table 1 shows the list of the dams examined, the construction period and the maximum height. The dams are aged between 30 and 60 years since their construction. Maximum height is comprised between about 20 and 65 m, the only exception being San Pietro in Villa (H_{max}=6.3 m). The location of the dams is illustrated in Figure 1, superimposed to the hazard map of the Italian peninsula referred to the return period T_R=475 years. The values of maximum acceleration PGA for return periods of 475 and 1950 years are also reported in Table 1. For all dams, PGA for T_R=475 years is larger than 0.15g; more specifically, for Angitola $PGA_{TR=475yrs}$ =0.27g whereas for the other dams $PGA_{TR=475\ yrs}$ is comprised between 0.18 and 0.22g.

The experimental data presented in this study are obtained from different sources. Data of Angitola, Montedoglio, Polverina and San Pietro in Villa dams derive from the consultant activity of one of the Authors (Ground Engineering srl), data of San Pietro dam were taken from the literature [3] whereas data of Penne dam have been derived from tests carried out on a sample provided from a consultant engineering company (see acknowledgments).

Table 1: Italian earth-core rockfill dams considered

#	Dam name	Region	Construction period	H_{max} (m)	$PGA_{475\ yrs}$ (g)	$PGA_{1950\ yrs}$ (g)
1	Angitola	Calabria	1960-1966	22.6	0.270	0.468
2	Montedoglio	Toscana	1977-1986	64.3	0.217	0.346
3	Penne	Abruzzo	1966-1969	35.7	0.182	0.294
4	Polverina	Marche	1964-1965	27.5	0.196	0.309
5	San Pietro	Campania	1958-1964	49	0.211	0.395
6	San Pietro in Villa	Toscana	1980-1993	6.30	0.221	0.353

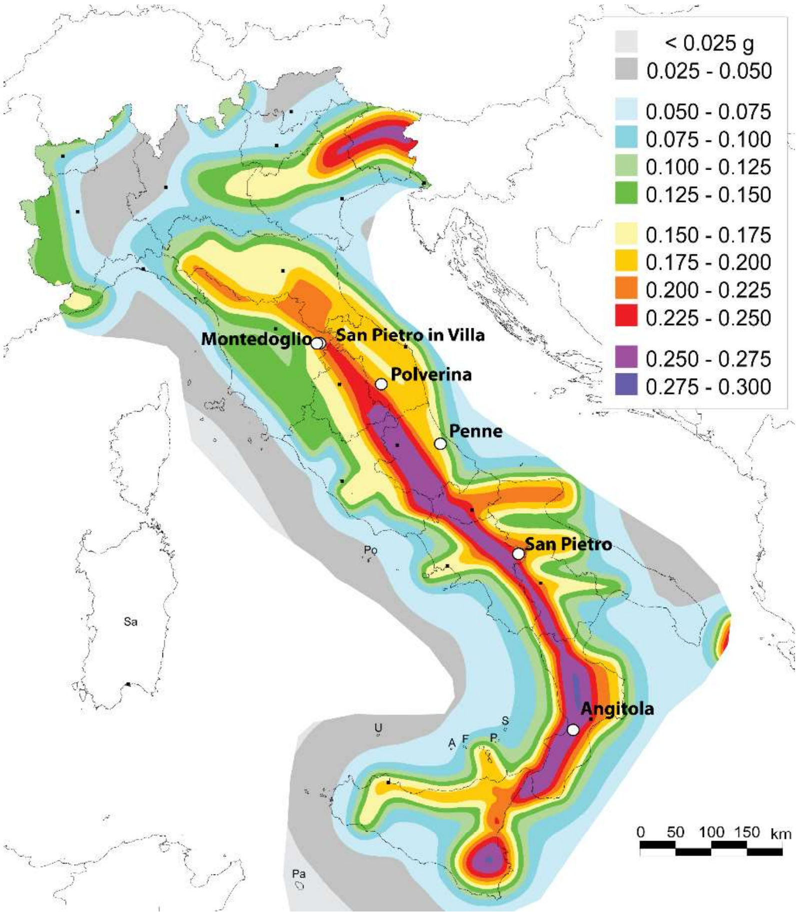

Figure 1: Location of the zoned dams examined, superimposed to the Seismic Hazard Map of Italy in terms of ground peak acceleration with 10% excess probability in 50 years, referred to rigid bedrock (Vs>800 m/s).

3 BASIC GEOTECHNICAL PROPERTIES OF CORE MATERIALS

Site investigations and geotechnical in-situ and laboratory tests were carried out to obtain the physical and mechanical properties of core materials of the six dams. In each dam boreholes were carried out, from the dam crest up to a maximum depth of 50 m, to evaluate the Vs profile in the core. Specifically, two boreholes were carried out at Montedoglio, Polverina, San Pietro and San Pietro in Villa dams (to perform cross-hole test) whereas at Angitola and Penne only one borehole was executed (to perform down-hole test).

In the following, only the physical properties as determined from laboratory tests carried out on the undisturbed samples retrieved from the cores are illustrated and discussed. Table 2 provides a summary of the main physical properties and state variables obtained on the samples subjected to cyclic and/or dynamic tests.

Grain size distributions, illustrated in Figure 2, indicate that core materials generally consist of silt, sand and clay in variable percentages; for Polverina sandy fraction is predominant, in the range 40-50%, in accordance to information from literature [4]. Clay fraction is always present for all tested materials, ranging between 20 and 40%. Figure 3 shows the profiles of unit weight (γ), water content (*w*), void ratio (*e*), degree of saturation (S_r) and clay fraction (CF) with depth. It can be noted that γ is about constant (20.1-21.3 kN/m^3) and also water content varies in a small range (*w*=15.2-23.3%) which is very close to the plastic limit (w_P) range (13.8-26.0%). Liquid limit can be found in the range 33-55% and plasticity index (PI) varies between 13 and 29. Degree of saturation (S_r) is between 90 and 100%. Consistency index (I_C) is generally higher than 1, indicating a stiff material. According to the Casagrande chart (Figure 4a), the core materials can be classified as inorganic clays of low to medium plasticity whereas on the activity chart (Figure 4b) they correspond to soils with low activity.

Table 2: Physical properties and state variables of the tested samples

#	Dam name	Test	Depth (m)	γ (kN/m^3)	w (%)	e_0 (-)	w_L (%)	PI (-)	CF (%)	I_C (-)	S_r (%)
1	Angitola	DSDSS	18.9	20.3	20.8	0.565	44	23	36	1.00	98.7
2a	Montedoglio	DSDSS	4.70	20.2	23.3	0.629	55	29	35	1.09	100.0
2b	Montedoglio	DSDSS	23.0	21.3	17.4	0.469	37	16	23	1.24	99.8
2c	Montedoglio	DSDSS	30.0	21.2	15.2	0.456	34	15	30	1.24	91.9
3	Penne	DSDSS	30.0	20.2	18.3	0.513	48.2	27.3	36	1.10	93.7
4a	Polverina	DSDSS	7.50	21.2	15.3	0.445	37.6	17.9	25	1.25	93.1
4b	Polverina	RC**	12.2	20.8	17.6	0.503	37.5	18.0	23	1.11	94.8
4c	Polverina	RC*	21.2	21.2	17.3	0.468	27.5	13.0	20	0.79	100.0
4d	Polverina	DSDSS	21.3	21.0	16.2	0.488	33.8	15.4	19	1.14	91.0
5	San Pietro	RC	15.0	20.1	23.0	-	42	22	32	0.9	-
6a	San Pietro in Villa	DSDSS	3.00	20.4	19.4	0.56	39	15	28	1.30	94.5
6b	San Pietro in Villa	DSDSS	6.00	20.7	18.4	0.56	33	14	11	1.00	94.5

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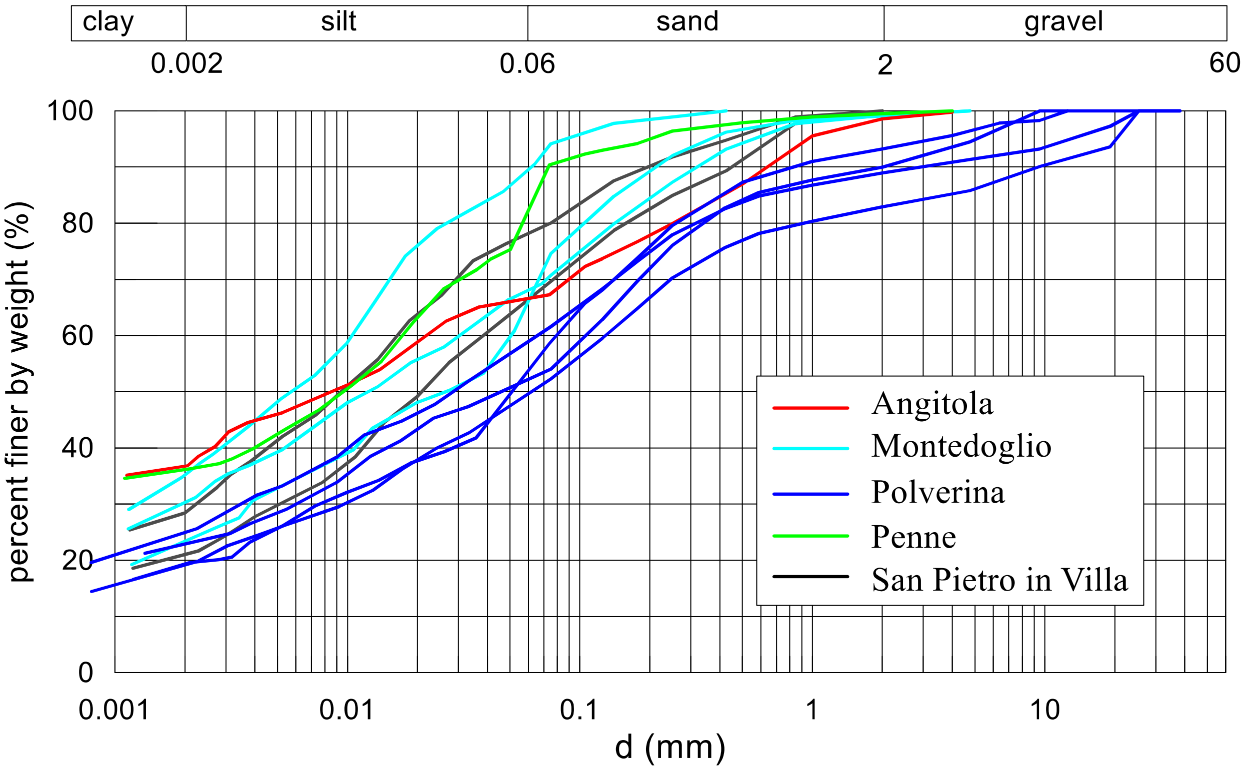

Figure 2: Grain size distributions of the core samples subjected to cyclic and dynamic tests

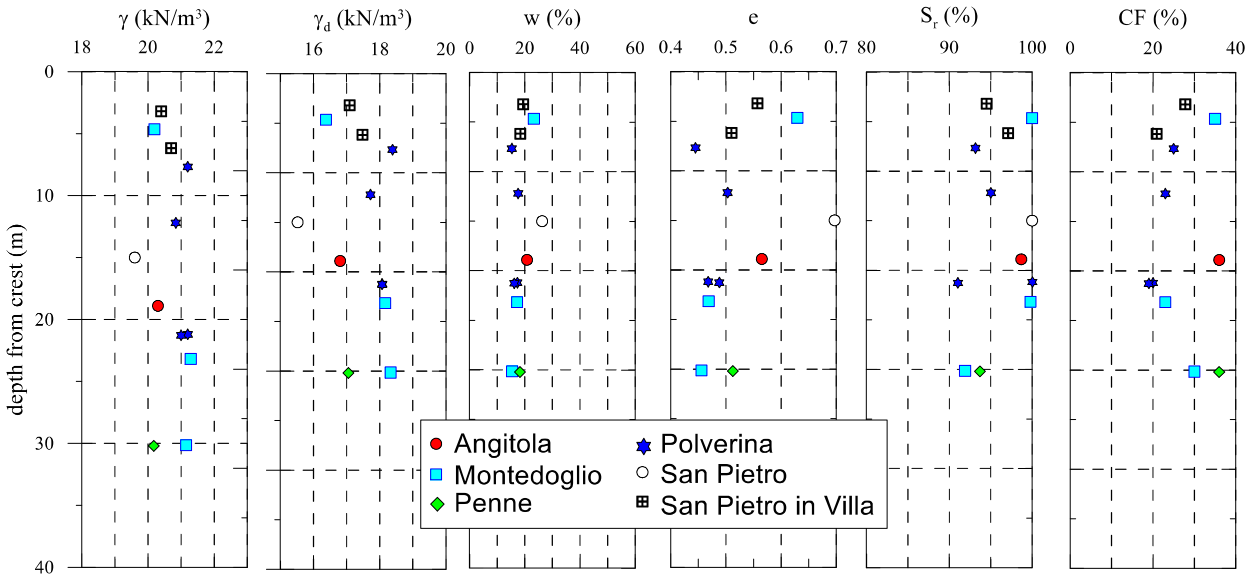

Figure 3: Variation of index and state properties of core samples with depth from the crest of the dams

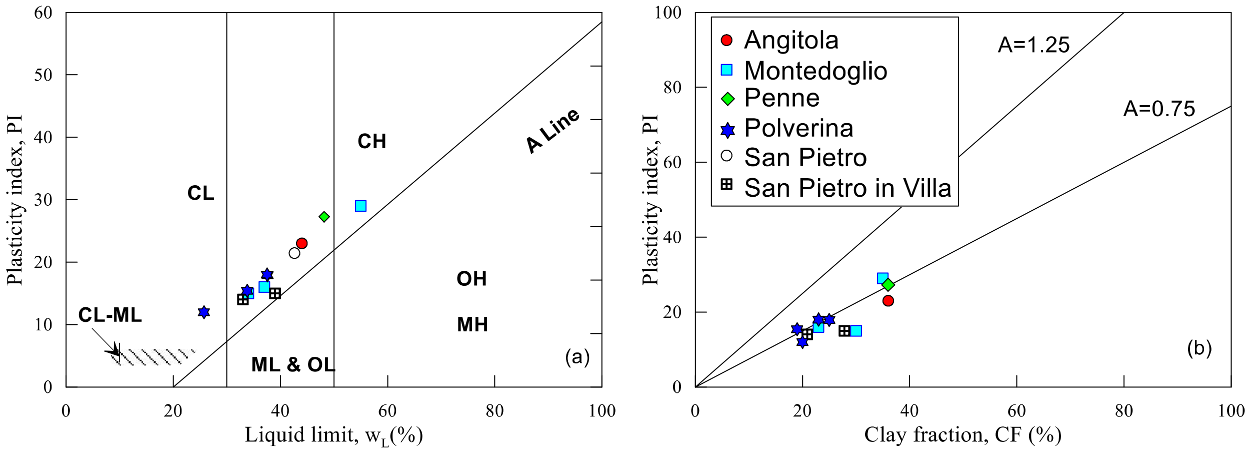

Figure 4: Casagrande and activity charts of core samples

4 CYCLIC AND DYNAMIC LABORATORY APPARATUSES

Deformation properties of core materials have been determined through laboratory cyclic and dynamic testing. As mentioned before, these properties are expressed in terms of normalized shear modulus reduction and damping ratio curves, that is G/G_0-γ_c and D-γ_c, respectively.

Most of the experimental investigations have been carried out at the Geotechnical Laboratory of the Department of Structural and Geotechnical Engineering of the Sapienza University of Rome (Faculty of Architecture). More specifically, cyclic simple shear tests have been conducted with a Double Specimen Direct Simple Shear (DSDSS) apparatus on Angitola, Montedoglio, Penne, San Pietro in Villa and Polverina dams (Table 3). For Polverina core material two RC tests have also executed, one at the Geotechnical Laboratory of the Department of Structural and Geotechnical Engineering of the Sapienza University of Rome (Faculty of Engineering) and the other by a private laboratory (ISMGEO). Finally, data from RC test have also been found in the literature on San Pietro dam. Usually the samples have been consolidated at the in-situ vertical or mean effective stress, variable between 50 and 500 kPa, but in some cases (e.g. Montedoglio and Polverina) very high confining pressure were also applied (up to 1500 kPa) in order to investigate the effect of confining stress

Table 3: Cyclic and dynamic tests carried out on core materials

#	Dam name	Depth (m)	Cyclic/Dynamic laboratory test	σ'_v (kPa)	σ'_m (kPa)
1	Angitola	18.9	DSDSS	250-500	166.7-333.3*
2a	Montedoglio	4.70	DSDSS	95-200	63.3-133.3*
2b	Montedoglio	23.0	DSDSS	430-800	286.7-533.3*
2c	Montedoglio	30.0	DSDSS	500-1000-1500	333.3-666.7-1000*
3	Penne	30.0	DSDSS	250-500	166.7-333.3*
4a	Polverina	7.50	DSDSS	150-300	100-200*
4b	Polverina	12.2	RC	-	125
4c	Polverina	21.2	RC	-	270-400
4d	Polverina	21.3	DSDSS	300-600-1200	200-400-800*
5	San Pietro	15.0	RC	-	150-300
6a	San Pietro in Villa	3.00	DSDSS	50-100	33.3-66.7*
6b	San Pietro in Villa	6.00	DSDSS	125-250	83.3-166.7*

* Estimated value

The DSDSS device was originally conceived, designed and constructed at the University of California at Los Angeles [5], specifically for investigating small-strain behaviour. In fact, due to its double specimen configuration, typical frictional problems that characterize the standard Norwegian Geotechnical Institute (NGI) direct simple shear device [6] were eliminated, thus allowing cyclic soil behaviour to be studied even at small strains. The DSDSS version used in this study is available at the Geotechnical Laboratory of the Faculty of Architecture, Sapienza University in Rome. A detailed description of this apparatus can be found in D'Elia et al. [7] and only a brief summary is provided hereafter. The layout of the apparatus is shown in Figure 5. Tests are carried out under constant volume conditions and a horizontal piston is used to apply the cyclic loading. The specimens are initially consolidated under anisotropic conditions. The mean effective stress σ'_m can be estimated under the assumption of oedometric conditions with a coefficient of earth pressure at rest (K_0) of 0.5. The anisotropic σ'_m is then evaluated as $\sigma'_m = (\sigma'_v + 2\sigma'_h)/3$. After the consolidation phase, cyclic loadings are applied with increasing cyclic shear strain amplitudes usually variable between 0.0004% and 1%. The test is performed under displacement control conditions. The secant shear modulus (G) and damping ratio (D) are directly measured from the cyclic stress-strain loop, according to the γ_c reached. The maximum shear modulus (G_0) can be estimated from the extrapolation of the experimental data at γ_c=0.0001%. For each step, 10 loading cycles are applied with a loading frequency of about 0.1 Hz. The apparatus is not instrumented for pore water pressure measurements.

As already mentioned, the device is capable of measuring the cyclic properties of soils in a very wide range of cyclic shear strain amplitudes, spanning from very small ($\gamma_c\cong$0.0004%) to very large ($\gamma_c\cong$3-4%) strains. As a matter of fact, the DSDSS device has been successfully used to investigate cyclic behaviour for a variety of soils and soft rocks (e.g. [8-10]. An example is shown in Figure 6 where the results of the DSDSS test carried out on the Penne core specimens (PI=27.3, σ'_v=500 kPa) are illustrated in terms of stress-strain loops, for increasing γ_c values from 0.0005% to about 3%, showing the large strain range investigated during the test.

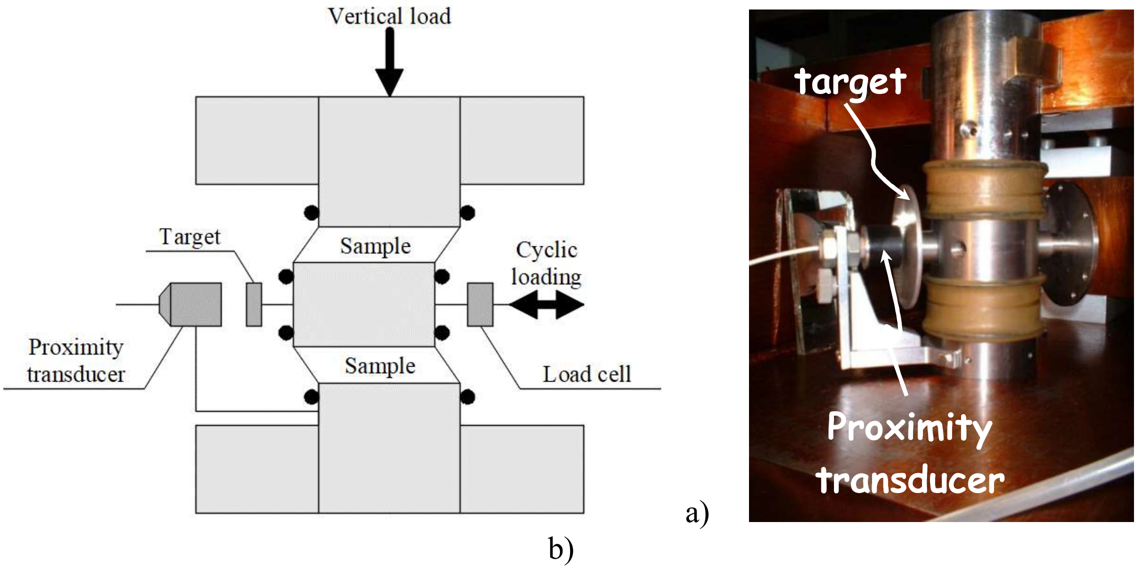

Figure 5 a) Layout of the DSDSS apparatus available at the Geotechnical Laboratory (Faculty of Architecture) of the Sapienza University of Rome; b) picture of the DSDSS apparatus.

The RC apparatus made available by the Geotechnical Laboratory of the Sapienza University of Rome (Faculty of Engineering) is a modified version of the first free-fixed type machine designed at the University of Texas at Austin [11] and is equipped with an electro-pneumatic motor that allows closed-loop feedback control of torsional load or angular deformation, a signal conditioning unit and a data logger. The cylindrical soil specimen is fixed at the bottom base and excited at the top using an electrical motor able to generate a torsional moment, constituted by eight drive coils encircling four magnets attached to a drive plate. This device and the specimen are placed into the compressed air cell for the application of the isotropic total pressure, while a drainage system allows control of the pore water pressure. In the RC test the specimen is dynamically excited applying a torsional oscillation at the top base while varying the frequency of the input signal in a range that can go from 10 to 250 Hz. The response of the specimen in terms of motion amplitude (rotation angle) is measured either by an accelerometer or by proximity displacement transducers, so that the fundamental mode of vibration is found in correspondence of the maximum response. The shear modulus is calculated from the resonant frequency according to the elasticity theory, while material damping can be determined from the half power bandwidth or from the free-vibration decay curve observed after stopping the excitation at resonance.
It is important to note that the RC and DSDSS confinement states are different, isotropic for RC (K_0=1) and anisotropic for DSDSS (assumed K_0=0.5) tests.

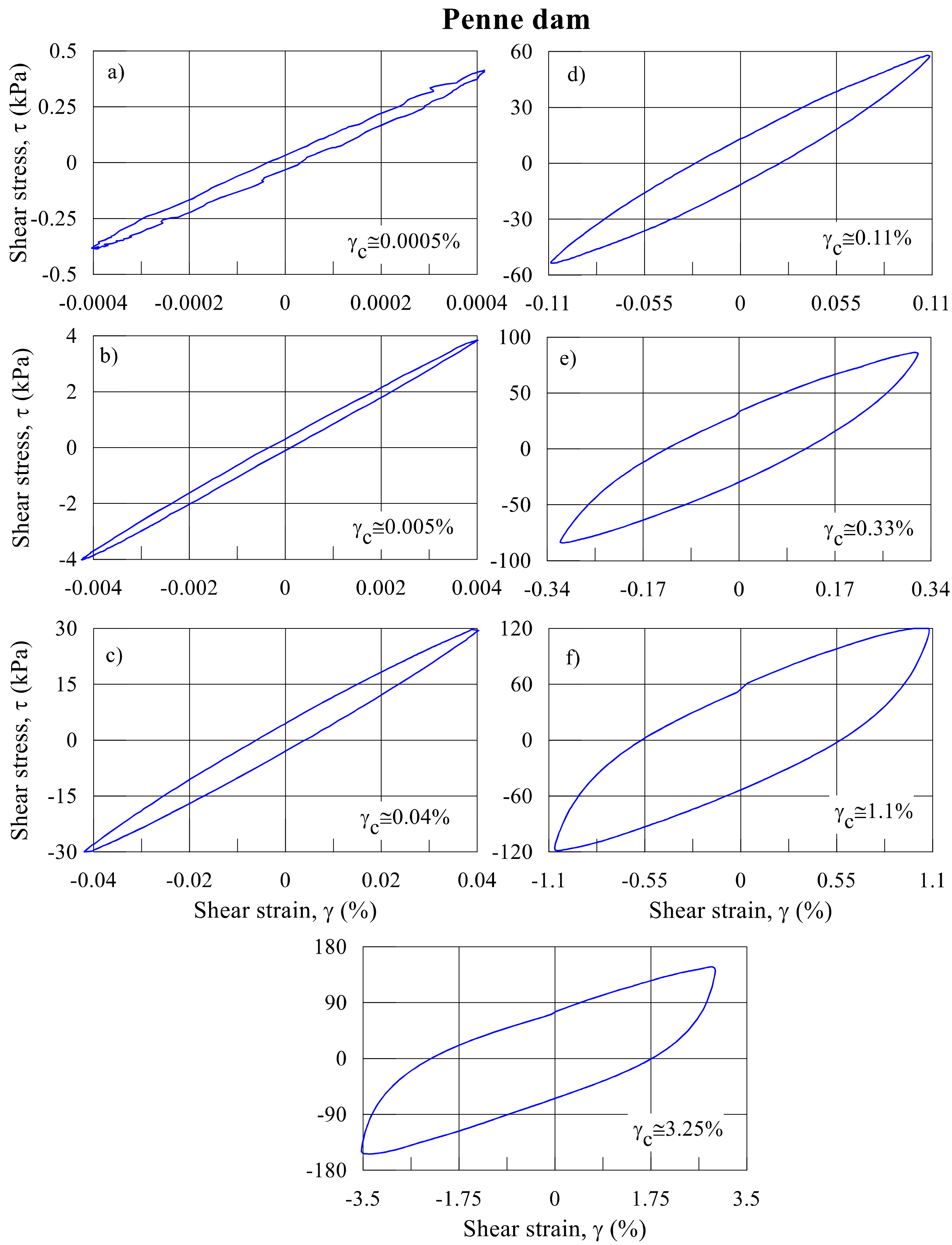

Figure 6 Cyclic stress-strain loops obtained from the Penne specimen (PI = 27.3) in DSDSS tests at σ'_v = 500 kPa for increasing levels of shear strain amplitude γ_c.

5 RESULTS OF CYCLIC AND DYNAMIC TESTS

The G/G_0-γ_c and D-γ_c data points of the tested core samples are illustrated in Figures 7 through 10. Specifically, Figure 7 and Figure 8 show the influence of effective confining stress and plasticity index for Angitola and Montedoglio dams, respectively. For Angitola (PI= 23) tests have been conducted at σ'_v=250 kPa and σ'_v=500 kPa and data points overlap almost completely; only a very small reduction of damping at small strains for increasing σ'_v can be noted. For Montedoglio, tests have been conducted on samples of different plasticity (PI=15, 16 and 29) effective confining stress (σ'_v=100-1500 kPa). It can be noted that all the curves describe a very narrow range, regardless of the different values of plasticity index in the range examined (PI=15-29). Also the effect of confining pressure is not very significant, considering the high σ'_v applied to the specimens; in particular the influence of confining pressure is negligible on the normalized stiffness curves, whereas some effect can be noted on damping ratio with D values shifting downwards as confining stress increases. To illustrate further the effect of PI and σ'_v on the G/G_0-γ_c and D-γ_c relationships, test results on Polverina dam are shown in Figure 9. These tests have been conducted on core samples with similar plasticity index (PI=13-18) consolidated at effective confining stresses variable in a wide range (σ'_m=100-800 kPa). Moreover, different apparatuses were used (i.e. DSDSS and RC), thus a comparison in terms of stiffness and damping ratio can also be made. In accordance with previous results it can be seen that the effect of effective confining stress is very limited on normalized stiffness whereas is more significant on damping ratio values, especially at smaller γ_c amplitudes. In fact, it can be seen from DSDSS tests that small-strain damping ratio varies between about 3.5% and 1% for σ'_v= 150-1200 kPa; small-strain damping ratio from RC tests are higher than those predicted by cyclic DSDSS tests. These differences can be presumably attributed to the different loading frequencies applied.

Figure 10 illustrates the experimental data obtained for all tested materials. It can be seen that the whole set of data fall in a narrow range and no clear trend with plasticity index can be identified, as commonly established for natural soils. Only for Polverina dam, the experimental trend shows a more nonlinear behavior in terms of stiffness reduction as compared the other materials. This is consistent with the grain size composition of Polverina core material which is characterized by a greater sandy fraction and the lowest values of plasticity index. However, damping ratio values fall within the range identified for the other core materials.

The above observations indicate that the effect of PI on the G/G_0-γ_c and D-γ_c of undisturbed core materials is not so evident and important as it is for natural fine-grained soils, at least in the range investigated (PI=13-29). Further, the effect of effective confining stress is also very limited. This behavior can be presumably due to the overconsolidation induced by compaction during construction. Similar kind of behavior has been recently observed also by Park and Kishida [12].

In Figure 10 the G/G_0-γ_c and D-γ_c data points are also compared with the V&D (Vucetic and Dobry) [13] empirical curves for similar PI. The V&D curves are not completely able to capture the variation of normalized stiffness with shear strains, at least up to about γ_c=0.05%, where the experimental data show a more linear behavior as compared to literature curves. Also for Polverina, which shows a slightly less linear behavior, the linear threshold is higher than that calculated by V&D curves. However, at larger shear strains experimental data fall within the band identified by V&D for soils of similar plasticity (PI=15-30). Comparison in terms of damping ratio shows that experimental data are lower than generic curves at small strains ($\gamma_c \cong$0.05%) whereas at moderate to large strains (γ_c>0.1%) experimental data are higher than V&D curves. In the damping ratio plot, experimental data from Polverina RC tests have not been included.

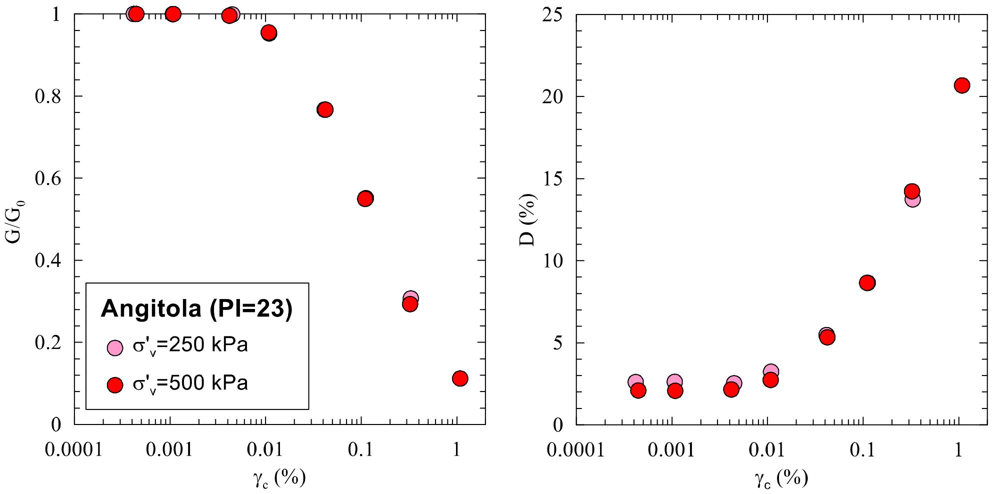

Figure 7 Modulus reduction and damping ratio data points of Angitola dam core from DSDSS tests..

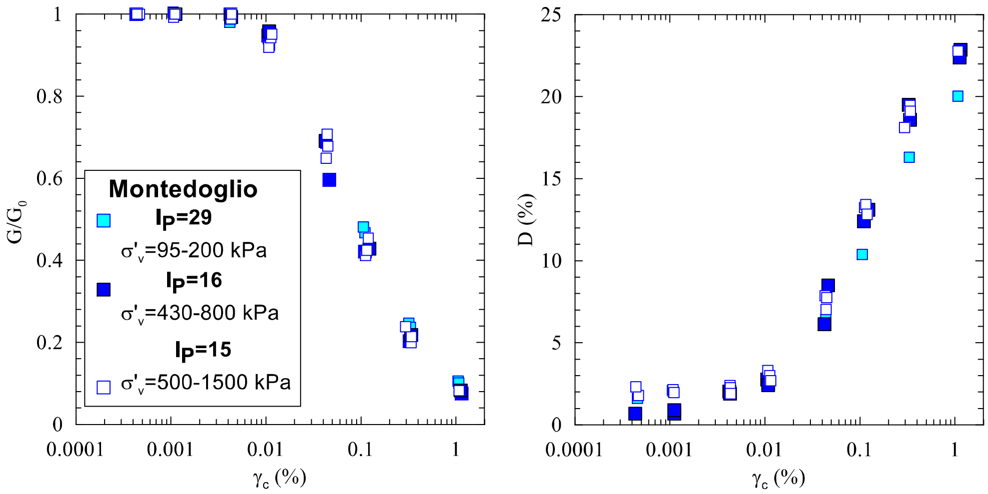

Figure 8 Modulus reduction and damping ratio data points of Montedoglio dam core from DSDSS tests.

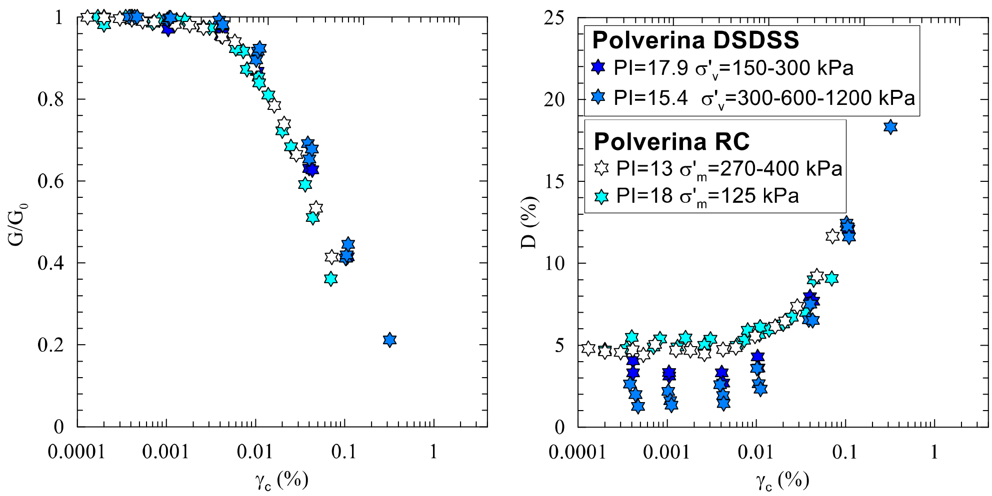

Figure 9 Modulus reduction and damping ratio data points of Polverina dam core from RC and DSDSS tests.

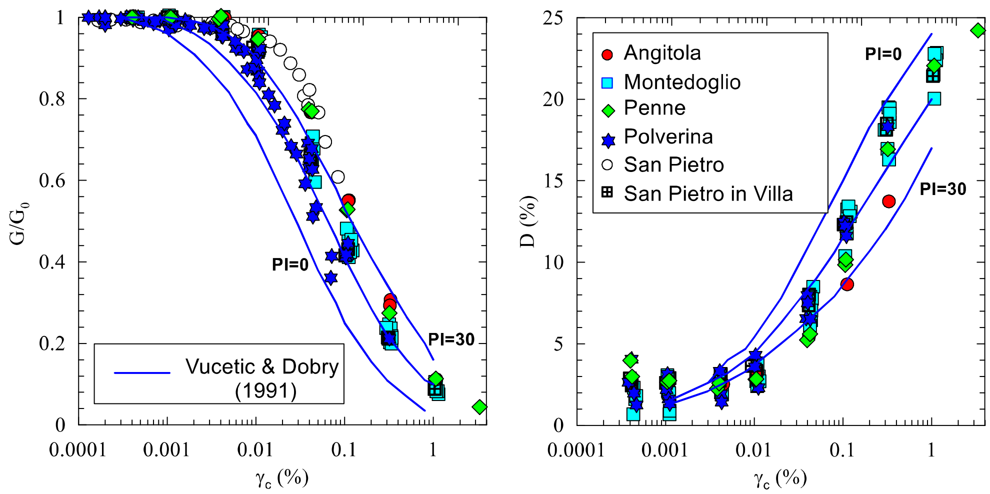

Figure 10 Modulus reduction and damping ratio data points of Italian core dam materials compared with empirical relations by Vucetic & Dobry (1991).

6 CONCLUSIONS

A series of laboratory tests have been illustrated aiming at investigating the main factors affecting the nonlinear stiffness and damping properties of undisturbed samples of core materials of six Italian zoned dams. The laboratory investigations comprised cyclic simple shear and resonant column tests. Based on the gathered experimental results, the importance of conducting site-specific measurements on dynamic properties of core materials, rather than using generic literature data to properly conducting numerical dynamic analyses, has emerged. More specifically, the following tentative conclusions can be drawn:

- normalized modulus reduction and damping ratio curves do not seem to follow the well-established trend for natural fine-grained soils in terms of plasticity index and confining stress, at least in the ranges investigated (PI=13-29 and σ'_m=100-800 kPa);
- it is speculated that this lack of clear shifting of the G/G_0-γ_c and D-γ_c curves as plasticity and effective confining stress incraese is due to the overconsolidation induced by compaction process during the construction of the dam;
- empirical correlations from literature (such as Vucetic and Dobry curves) are not completely capable to predict properly the G/G_0-γ_c and D-γ_c trend with PI and σ'_m, especially in the small-to-medium strain range; for this reason it is recommended conducting specific laboratoy tests to assess the nonlinear dynamic behaviour of core materials of embankment dams.

The above results have to be considered preliminary as more laboratory tests are needed to corroborate the indications that emerged from the study and to provide a greater generality to the results obtained.

ACKNOWLEDGMENTS

The writers want to express their gratitude to O&M Hydro (Engs. Rosella Caruana, Francesco Fornari, Alberto Frezza and Renzo Sparnacci) for allowing the publication of test results of Polverina dam. Eng. Stefano Cola of EAUT (*Ente Acque Umbre-Toscane*) is gratefully acknowledged for the permission to publish the data of Montedoglio and San Pietro in Villa dams. The permission to publish data for Angitola dam by Eng. Pasquale Cimbalo (*Consorzio di Bonifica "Tirreno Catanzarese"*) is also appreciated. Finally, Eng. Alessandro Masciotta of Studio Masciotta in Rome is gratefully acknowleged for having provided the undisturbed samples of the core of Penne dam.

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