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Cyclic behavior of soft offshore clays at small to large strains

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***Abstract* – Two soft offshore clays, recovered from the Adriatic shelf and the Tyrrhenian shelf margin (Italy), were subjected to cyclic simple shear tests. The influence of the cyclic shear strain amplitude as well as of vertical stress, void ratio and stress history on stiffness and damping properties of the soils, in a wide strain range (i.e. from 10^{-4} % to more than 1%), is examined and discussed. The role of soil structure on the cyclic behavior is also explored.**

I. INTRODUCTION

The cyclic behavior of offshore clays controls several offshore geotechnical problems such as foundation of structures and stability of submarine slopes subjected to earthquake excitation or wave action.

The behavior of soils under cyclic loading is commonly expressed in terms of stiffness and damping properties. These properties have been extensively studied for onshore soils ranging from very stiff to very soft and have been related to several variables such as shear strain amplitude, confining stress, stress history, number of loading cycles and strain rate [e.g. 1-3]. Conversely, limited studies on cyclic properties of very soft offshore marine clayey sediments have been carried out [4-5].

In this paper, the results of cyclic simple shear tests on very soft clay sediments from the Adriatic shelf and the Tyrrhenian shelf margin (Italy) are presented. Cyclic properties were measured in a wide strain range: the small-strain stiffness expressed by the maximum shear modulus (G_0) as well as the equivalent shear modulus (G_{eq}) and the damping ratio (D) from medium to large strains are investigated. The influence on the above mentioned parameters of the cyclic shear strain amplitude (γ_c) as well as of vertical stress, void ratio and stress history is examined and discussed. The role of soil structure on the cyclic behavior is also explored.

II. TESTED SOILS

A. Vasto clay

The Vasto study area is located in the Central-Southern Adriatic Sea some 14 kilometers offshore the coast of the Abruzzo Region (Fig. 1). All over the Adriatic shelf a well-defined erosional unconformity, formed during the last glacial sea level lowstand (about 20 kyears to present), separates the overconsolidated Quaternary sequences from the overlying sedimentary units deposited during the last sea level cycle. They are formed by back-transgressive units (20-8 ka to present) mainly silty-sandy, overlaid by muddy deposit (8 ka to present) [6]. The latter, which is the target of the geotechnical characterization, has a thickness up to 25 m in the middle shelf (40-50 m water depth) and decreases towards the basin reducing to a 2-3 m-thick drape at the shelf break.

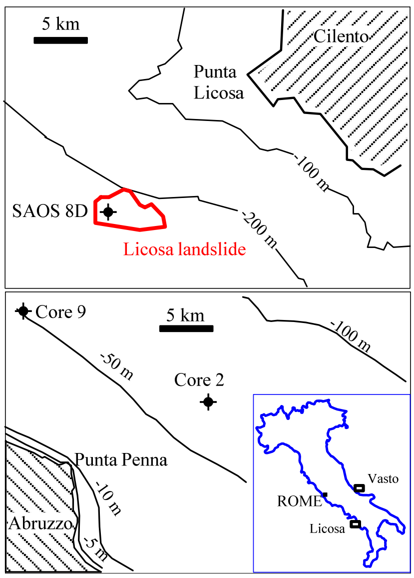

Fig. 1. Location of the studied areas and sampling sites

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The specimens utilized for the tests were recovered from the uppermost part of the pelitic unit in the cores 2 and 9 (Fig. 1). In order to reduce sampling disturbance typical of standard gravity coring, sampling was carried out through a 130 mm vibrocorer driven from the seabed into the sediment without vibration at low penetration rate under its sole own weight (about 16 kN). Due to the homogeneity of the depositional conditions in the study area, the material recovered in the cores 2 and 9 (that are 11 km apart) can be considered as being virtually the same. It is a clay and silt of medium plasticity (CF=47%-50%, PI=19-30) having a specific gravity of 2.75. The natural water content is close or slightly above the liquid limit W_L (Table 1). The material is normally consolidated with the exception of the uppermost 1 m where it is slightly over-consolidated probably due to bioturbation and thixotropy [7].

Incremental loading oedometer tests were conducted on samples retrieved at three different depths (0.5, 1.0 and 2.0 m). Average values of the compression index, Cc, and swelling index, Cs, are 0.47 and 0.07 respectively. The test carried out on the sample at 1 m depth is shown in Fig. 2 using the normalizing void index $I_V = (e - e_{100}^*)/C_c^*$ [8], where e is the void ratio of natural soil, C_c^* and e_{100}^* are the compressibility index and the void ratio at a vertical effective stress of 100 kPa for the reconstituted material, respectively. These latter intrinsic properties (i.e. independent on the soil state) were computed from the void ratio at liquid limit (e_L) using the empirical correlation proposed by Burland [8]. The resulting I_v-log σ'_v curve was compared to the Intrinsic Compression Line (ICL) representing the normalized behavior in oedometer compression tests of the reconstituted soil and the Sedimentation Compression Line (SCL) obtained by Burland from the regression of data in in-situ conditions of natural soils. The distance between ICL and SCL is a measure of the effect of fabric and bonding (i.e. structure) of the skeleton of natural soil.

From Fig. 2 it can be observed: i) at low σ'_v the Vasto curve is intermediate between ICL and SCL; ii) above $\sigma'_v \approx 200$ kPa the curve is about on the ICL, i.e., loading has completely disrupted soil fabric.

B. Licosa clay

The Licosa submarine translational landslide [9] involved an area of 20 km^2 at a mean water depth of 250 m (Fig. 1). The slide scar is 6 km wide and the thickness of failed sediment does not exceed 9 m; the total estimated volume of mobilized sediment ranges between 80 - 100 x 106 m^3. The failure, occurred about 11 ka BP, along a 1.5° - 3° dipping plane, involved a shelf-margin progradational alternation of sands, sandy silts and clayey silts with coarse-grained volcaniclastic intercalations deposited during the Last Glacial Maximum.

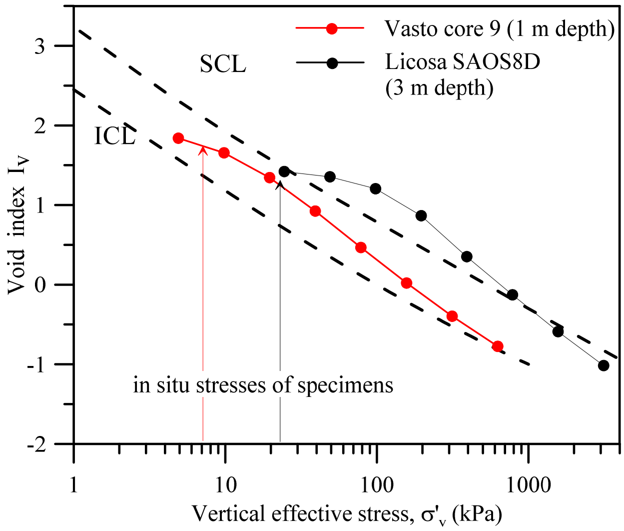

Fig. 2. Normalized oedometer compression curves of Licosa and Vasto clays compared with the ICL and SCL lines (see text)

Specimens were recovered below the sliding surface from the core SAOS8, located within the landslide scar (Fig. 1). Sampling was carried out through a gravity piston corer, with high accuracy (the sample is "very good to excellent" according to Lunne et al. [10]).

The tested soil is silt with clay (CF=32%) of medium plasticity (PI=27) having a specific gravity of 2.68. The natural water content is about 54%, close to the liquid limit W_L (Table 1).

Table 1. Relevant physical properties of tested soils

Soil	CF(%)	w_L(%)	PI	w (%)	e_0 (-)
Vasto	48	54	29	62*-58**	1.635*-1.523**
Licosa	32	55	27	54	1.456

* DSDSS test A, ** DSDSS test B

Incremental loading oedometer tests as well as direct shear test were conducted on sample SAOS8D recovered at 3 m depth. The material is slightly over-consolidated with a pre-consolidation pressure of about 80-100 kPa compatible with the thickness of sediment removed by the slide. The values of the compression index, C_c, and swelling index, C_s, are 0.48 and 0.07 respectively. The compression curve in term of void index I_V is shown in Fig. 2 together with the compression curve of Vasto clay and ICL-SCL lines. The curve of the Licosa clay plots significantly on the right of that of Vasto clay: at low σ'_v the Licosa I_V falls on or above the SCL thus suggesting that the soil has a significant structure. Post-yield destructuration is attained only at high vertical stress (1600-3200 kPa) where the curve crosses the SCL and seem to slowly converge to the ICL.

C. Final remarks

The two tested soils are characterized by similar plasticity and initial void ratio, though Vasto clay has a higher clay fraction. A remarkable difference regards soil structure: looking at the I_v-log σ'_v curves of Fig. 2, Licosa clay is undoubtedly more "structured" with respect to Vasto clay over the whole stress range investigated with cyclic tests (up to $\sigma'_v \approx 640$ kPa, see later). This difference can be ascribed only partially to the over-consolidation of Licosa clay (this "sign" should be obliterated above $\sigma'_v \approx 100$ kPa). A relevant role is played by depositional rate and water stillness: generally the higher is the rate and the stronger are the sea currents, the closer are the oedometer curves to ICL [8]. In this respect estimated average deposition rates of Vasto and Licosa sediments are about 3 and 0.1 mm/y, respectively. However other two factors may affect structure of Licosa clay: the influence of silt/sand particles on arrangement of particle aggregates and the presence of frequent coarse grained horizons which favor consolidation. Clarification will be given by microstructural analyses and by further insight in chemical-physical conditions of the depositional environments.

III. TESTING PROGRAM

Soils were tested in a double specimen direct simple shear (DSDSS) device constructed at the University of Rome "La Sapienza" [11] on the basis of the prototype designed and built at the University of California at Los Angeles [12]. The peculiarity of the device, consists in the simultaneous shearing of two specimens of the same soil. Due to its specific configuration and to the large stiffness of the device components, all the problems associated with false deformations and system compliance are negligible, thus enabling the measurement of soil properties even at very small strains (≈0.0004%).

At the completion of primary consolidation under the specified vertical load, the specimens are subjected to several steps of strain-controlled cyclic shearing gradually increasing the magnitude of cyclic shear strain amplitude γ_c. This allows to define the variation of equivalent shear modulus (G_{eq}) and damping ratio (D) with γ_c. The shape of cyclic straining is about sinusoidal with a frequency usually ranging from 0.1 to 0.3 Hz.

For the Vasto clay, the test program included two series of tests, hereafter referred as A and B (Table 2). For the A series tests, a total of 12 stages were conducted (A1-A12) according to the loading- unloading- reloading-second unloading sequence reported in Table 2. In the first loading phase, tests at σ'_v lower than 80 kPa were not carried out because the forces applied, at very small strains, are comparable with the accuracy of the load cell. In Table 2 the values of overconsolidation ratio (OCR) and void ratio (e) at the end of consolidation of each stage are reported. The cyclic shear strain amplitude γ_c varied between 0.0004% and 0.04%, i.e. slightly below the volumetric shear strain threshold, which for soils of similar plasticity is about 0.04% [13]. For B series tests, only 2 loading stages with σ'_v = 80-160 kPa were carried out (Table 2), with γ_c varying from $4*10^{-4}$% to 1%.

For the Licosa clay, the test program was applied to one specimen, with a total of 10 stages including loading-unloading-reloading-second unloading (Table 3). OCR and void ratio at the end consolidation (e) for each stage are reported in Table 3. Stages at lower σ'_v (<100 kPa) were not carried out again for the limited accuracy of the load cell at very small stresses. The maximum γ_c was 0.05% (below volumetric shear strain threshold) with the exception of the last stage, when γ_c reached 4%.

Table 2. Vasto clay: summary of cyclic testing conditions

stage	sequence	σ'_v(kPa)	γ_c (%)	e (-)	OCR(-)
A1	LD	80	$4*10^{-4}$ - 0.04	1.233	1
A2	LD	160	$4*10^{-4}$ - 0.03	1.100	1
A3	LD	320	$4*10^{-4}$ - 0.03	0.942	1
A4	UNLD	160	$4*10^{-4}$ - 0.03	0.953	2
A5	UNLD	80	$4*10^{-4}$ - 0.03	0.971	4
A6	UNLD	40	$4*10^{-4}$ - 0.03	0.996	8
A7	UNLD	20	$4*10^{-4}$ - 0.03	1.017	16
A8	RELD	80	$4*10^{-4}$ - 0.04	0.992	4
A9	RELD	320	$5*10^{-4}$ - 0.04	0.916	1
A10	RELD	640	$5*10^{-4}$ - 0.04	0.793	1
A11	2nd UNLD	160	$5*10^{-4}$ - 0.04	0.823	4
A12	2nd UNLD	40	$5*10^{-4}$ - 0.04	0.887	16
B1	LD	80	$4*10^{-4}$ - 1.1	1.298	1
B2	LD	160	$4*10^{-4}$ - 1.1	1.191	1

Table 3. Licosa clay: summary of cyclic testing conditions

stage	sequence	σ'_v (kPa)	γ_c (%)	e (-)	OCR(-)
1	LD	100	$3*10^{-4}$ - 0.09	1.373	1
2	LD	200	$3*10^{-4}$ - 0.05	1.241	1
3	LD	400	$4*10^{-4}$ - 0.12	1.060	1
4	UNLD	200	$5*10^{-4}$ - 0.05	1.068	2
5	UNLD	100	$5*10^{-4}$ - 0.02	1.079	4
6	UNLD	50	$5*10^{-4}$ - 0.05	1.099	8
7	RELD	200	$5*10^{-4}$ - 0.05	1.079	2
8	RELD	400	$5*10^{-4}$ - 0.05	1.047	1
9	2nd UNLD	100	$4*10^{-4}$ - 0.05	1.069	4
10	2nd UNLD	25	$4*10^{-4}$ – 4.0	1.109	16

IV. SMALL-STRAIN SHEAR MODULUS

It is well established that small-strain stiffness G_0 is influenced by the mean effective confining stress (σ'_m), OCR and void ratio (e). Many attempts have been made to describe the dependency of G_0 on the above mentioned parameters, mostly in the form:

$$G_0 = S\,F(e)\,\sigma'^{\,n}_{m}\;\sigma_r^{1-n}\;OCR^{m} \qquad (1)$$

where F(e) is a function of void ratio, σ_r is a reference stress usually taken as the atmospheric pressure ($p_a \approx 100$ kPa) and S, n, and m are non-dimensional stiffness parameters accounting for the nature of soil. The various relationships mainly differ in the choice of the void ratio function. Some of these functions were summarized by Jamiolkowski et al. [14] and, more recently, by Likitlersuang et al. [15].

In particular, Hardin [16] proposed $F(e) = 1/(0.3+0.7e^2)$ for sands and clays while for fine grained soils Jamiolkowski et al. [17] assumed $F(e) = e^{-x}$. These latter Authors used the product of vertical and horizontal effective confining stresses ($\sigma'_v * \sigma'_h$) instead of mean effective confining stress (σ'_m):

$$G_0 = S\,e^{-x}\,\sigma'^{\,n}_{v}\;\sigma'^{\,n}_{h}\,\sigma_r^{1-2n}\;OCR^{m} \qquad (2)$$

Moreover, these Authors found that with the adopted void ratio function (x is on average 1.3), the effect of OCR may be considered negligible (i.e. m~0 in Eq. 2).

An alternative way to take into account the dependency of G_0 on state and stress history under isotropic conditions was proposed by Viggiani [18] and Rampello et al. [19]. They expressed G_0 only as a function of two independent variables, the isotropic mean effective stress p' and the isotropic overconsolidation ratio R. In fact, the influence of void ratio (or specific volume v) can be expressed as a function of p' through the compressibility relationship, which is uniquely determined for normally consolidated states and accounted for by R for overconsolidated states. This approach was extended to oedometer compression by Lanzo et al [7]:

$$G_0 = S^*\,\sigma'^{\,n^*}_{m}\;\sigma_r^{1-n^*}\;OCR^{m^*} \qquad (3)$$

where S*, n* and m* are non-dimensional stiffness parameters equivalent to S, n and m in equation (1). It can be demonstrated [7] that:

$$OCR^{m^*} = (\sigma'_{ve}/\sigma'_{v})^{c} \qquad (4)$$

where σ'_{ve} is the equivalent vertical consolidation stress:

$$\sigma'_{ve} = \sigma_r\,10^{(e_r - e)/C_c} \qquad (5)$$

e_r being the void ratio at $\sigma'_v=\sigma_r$ while C_c is the compression index. The parameter c in eq. 4 is equal to m*/ Λ [7] where Λ is related to soil compressibility:

$$\Lambda = (C_c - C_s)/C_c \qquad (6)$$

being C_s the swelling index.

The G_0 values measured for both clays can be used to verify the applicability of the above relationships. G_0 values are first plotted in Fig. 3 as a function of σ'_v. Closed symbols refer to normally consolidated states (NC) whilst open symbols refer to overconsolidated states (OC). For both normally and overconsolidated states G_0 increases almost linearly with σ'_v. However, in agreement with literature data, it can be observed that the G_0 values corresponding to normally consolidated states plot on a straight line that is steeper than the lines corresponding to the overconsolidated states. The difference between the slope of NC and OC branches is more pronounced for Licosa clay.

The G_0 values normalized according to the Hardin, Jamiolkowski et al. and Lanzo et al. relationships, assuming $\sigma_r = p_a = 100$ kPa, are reported in Fig. 4. The horizontal $\sigma'_h=K_0\sigma'_v$ and mean effective stresses $\sigma'_m = (\sigma'_v+2\sigma'_h)/3$ were computed by estimating K_0 (coefficient of earth pressure at rest) through the relationships proposed by Mayne and Kulhawy [20]. All the relationships strongly reduce the scatter between the data points which are approximately distributed along straight lines. This circumstance is confirmed by the value of the coefficient of linear regression r^2 of all data, which range from 0.968 to 0.979.

The values of the regression parameters for both clays and all the relationships are summarized in Table 4 and compared with reference values obtained from literature for continental clays. Using Hardin relationship, the values of regression parameters for Vasto and Licosa clays are generally comprised in the literature range with the exception of m≈0 for Vasto clay. If reference is made to Jamiolkowski et al. relationship, S is significantly underestimated while n is overestimated. Literature data are obtained from Bender Element and Resonant Column tests [17], which apply a load frequency much higher than DSDSS. This condition leads to an overestimation of shear modulus [21] and hence of S with respect to DSDSS data. Moreover, it should be pointed out that in the literature data, the Authors imposed m=0 in the regression while in the present study we prefer to include the parameter m in a multiple linear regression analysis.

Looking at Fig. 4 and Table 4 some remarkable differences between the behavior of Licosa and Vasto clays can be observed. In general, although the non-dimensional stiffness parameter S (or S*) is comparable for the two tested soils, Licosa clay has a more pronounced dependence of G_0 on confining stress and OCR. As matter of fact, m and n values for Hardin and Jamiolkowski et al. relationships as well as m*and n* in Lanzo et al. procedure are higher for the Licosa clay. The reasons for such differences are not clear to the writers. Regarding the dependence on confining stress, it might be postulated that the destructuration observed in the Vasto clay as σ'_v increases (Fig. 2) causes a less pronounced increase of G_0 with confining stress (and hence a lower value of n) with respect to Licosa clay. In fact, the destructuration acts in the opposite direction of confining stress and tends to reduce the small strain shear modulus.

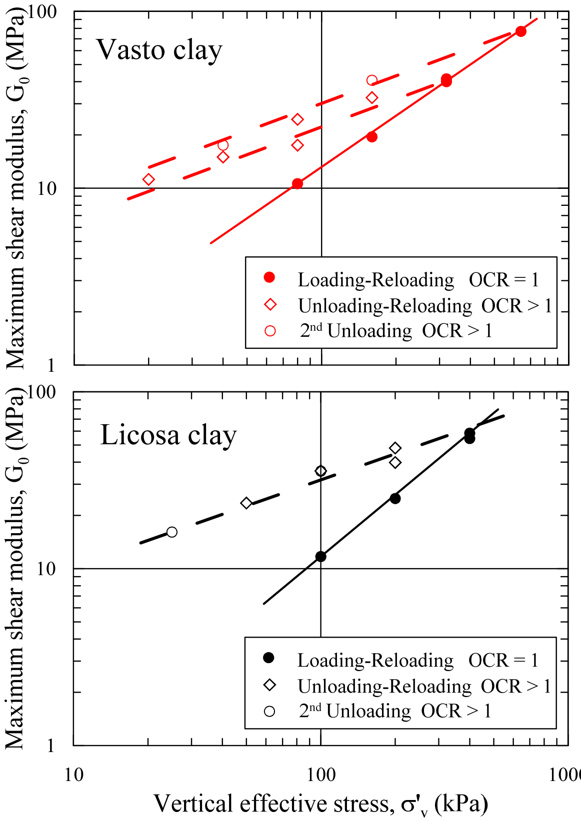

Fig. 3. Variation of small-strain shear modulus G_0 versus effective vertical stress for Vasto e Licosa clays

V. VARIATION OF SHEAR MODULUS AND DAMPING RATIO WITH STRAIN AMPLITUDE

The average equivalent shear moduli (G_{eq}) pertaining to the different steps were normalized with respect to G_0 to obtain the standard G_{eq}/G_0. The variation of G_{eq}/G_0 with γ_c is presented for Vasto stages B1 and B2 in Fig. 5. For the Licosa clay data are plotted for loading stage 1, characterized by $\sigma'_v = 100$ kPa comparable with Vasto stage B1, and stage 10 (2^{nd} unloading sequence, $\sigma'_v = 25$ kPa) the only conducted to a shear strain amplitude (≈4%) higher than the volumetric threshold. Data points plot between the curves suggested by Vucetic and Dobry [23] for clays with PI=30 (reproduced in the same plot) and the curves proposed by Darendeli [24] for the same values of PI at a mean isotropic confining stress of 100 kPa. Moreover, a more linear behavior can be observed for Licosa clay with respect to Vasto, especially at medium-high shear strains (above ≈0.1%). The effect of confining pressure and OCR seems to be almost negligible (for Licosa clay stage 1 and 10 are characterized by OCR=1 and 16 respectively).

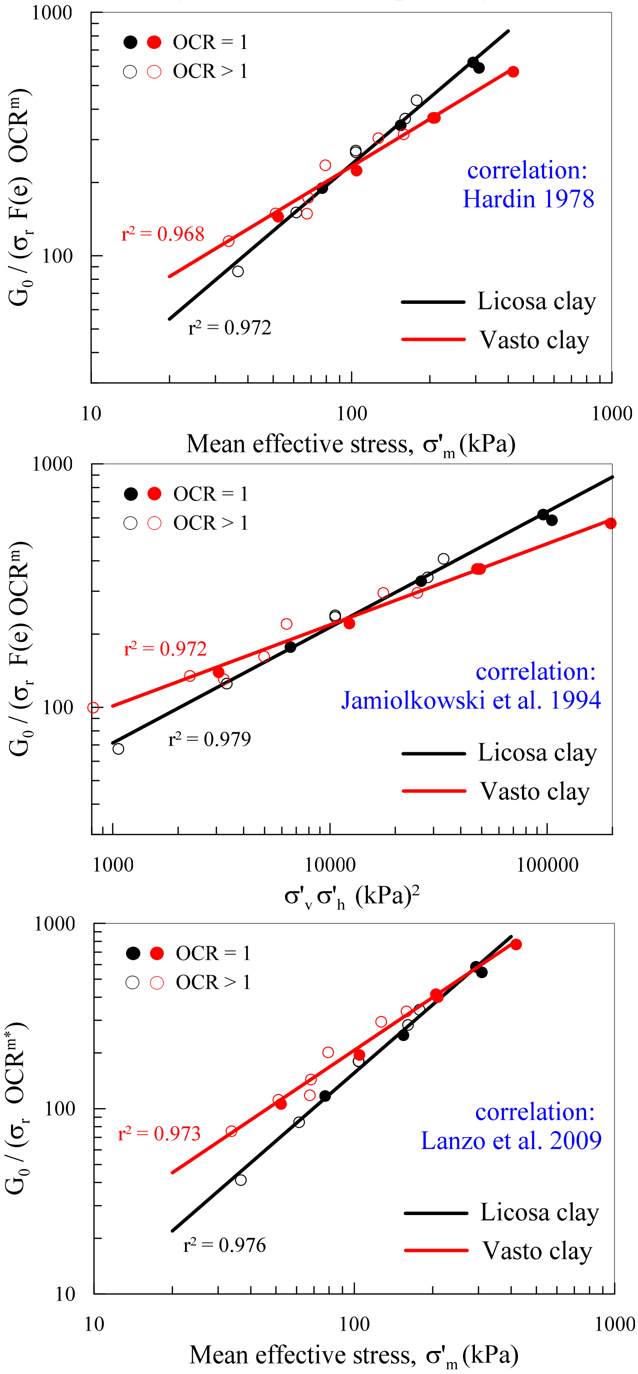

Fig. 4. Normalized G_0 versus confining stress according to different literature correlations

Table 4. Parameters of empirical correlations

correlation	Soil	S	n	m
Hardin (1978)	Vasto clay	233	0.65	0.00
	Licosa clay	238	0.91	0.28
	*Literature [22]**	*80-260*	*0.7-1*	*0.2-0.3*
Jamiolkowski et al. (1994)	Vasto clay	218	0.33	0.05
	Licosa clay	213	0.47	0.36
	*Literature [17]***	*440-810*	*0.2-0.3*	*0*

* continental clays, range of values for PI≈30
** G_0 from Bender Elements and Resonant Column tests, continental clays with PI=15-75, some highly structured

correlation	Soil	S*	n*	m*
Lanzo et al. (2009)	Vasto clay	207	0.95	0.14
	Licosa clay	156	1.22	0.49

In Fig. 6 the variation of D with γ_c for the same stages of Vasto and Licosa tests is presented. The D values are plotted together with the D-γ_c curves proposed by Vucetic and Dobry [23] for PI=30 and those proposed by Darendeli [24] for the same values of PI at a σ'_m = 100 kPa. With the exception of small strain range, Darendeli curves well represent the variation of experimental data of Vasto clay. Conversely, the variation of D of Licosa clay is better reproduced by the curves proposed by Vucetic and Dobry, significantly shifted downward with respect to Darendeli at medium-high shear strains. In other words, a significant less dissipative behavior is shown by Licosa clay especially above ≈0.1%. As observed for normalized shear modulus curves, the effect of confining pressure and OCR is almost negligible.
In conclusion, the plots of the normalized shear modulus and damping ratio versus shear strain amplitude of tested soils are essentially reproduced by standard literature curves of on-shore cohesive soils of similar plasticity. However, significant differences can be observed between the two clays: a more linear and less dissipative behavior is exhibited by Licosa, especially at medium-high shear strains (above ≈0.1%). Considering the negligible effect in confining stress and OCR and that the two clays have the same PI (the most important factor controlling the G_{eq}/G_0 and D curves), this difference could be tentatively related to the difference in soil structure highlighted in Fig. 2.

VI. CONCLUSIONS AND OPEN ISSUES

Shear modulus and damping properties of two soft offshore clays, characterized by similar plasticity and different soil structure, were investigated by means of cyclic simple shear tests over a wide range of shear strain amplitude, vertical stress, void ratio and stress history.

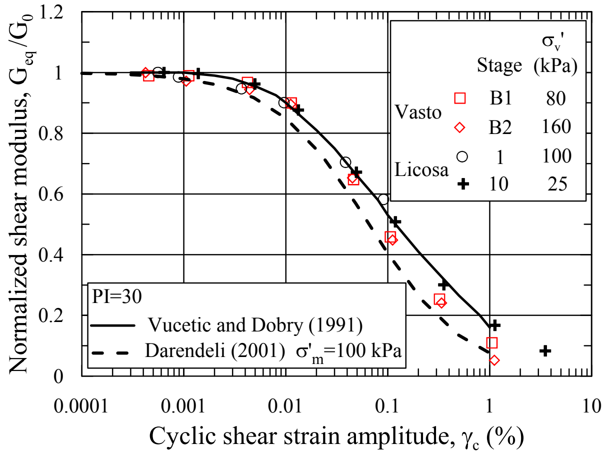

Fig. 5. Normalized equivalent shear modulus (G_{eq}/G_0) versus cyclic shear strain amplitude (γ_c)

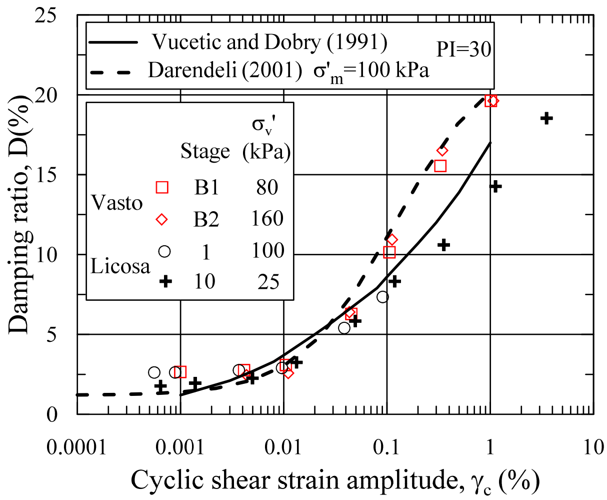

Fig.6. Variation of damping ratio (D) with cyclic shear strain amplitude (γ_c)

The dependence of the small-strain shear modulus G_0 on stress and history parameters (σ'_m, e and OCR) was first examined. Different simple normalization relationships were used to reduce the data, namely to divide G_0 by a function of OCR and/or by some functions of void ratio. Once normalized, the G_0 values plotted versus confining distributed in all cases along straight lines with negligible scatter. Non-dimensional stiffness parameters of normalization relationships are influenced not only by plasticity index, as shown by previous literature studies: a relevant role is played by soil structure. In particular, the more "structured" Licosa clay shows a stronger dependence of G_0 on confining stress and OCR.
Regarding the cyclic properties at medium to large strains, significant difference can be observed among the two tested soils, even if the G_{eq}/G_0 and D curves are essentially captured by standard literature curves built for on-shore soils. In particular, a more pronounced soil

structure results in a more linear and less dissipative behavior especially at medium-high shear strains (above ≈0.1%).

A testing program is envisaged to clarify the role of structure and dissipate some uncertainties deriving from the measuring devices. New load cells have been mounted on the DSDSS apparatus and will be used for performing tests at low vertical stresses, comparable to the in situ stresses of shallower samples. Tests on reconstituted specimens are to be conducted. Results will be compared to those from natural sediment specimens in order to definitely observe how much influence the sediment structure has on cyclic shear behaviour.

REFERENCES

[1] F. Tatsuoka, D.C.F. Lo Presti, Y. Kohata. Deformation characteristics of soils and soft rocks under monotonic and cyclic loding and their relationships. In Proc. of the 3rd Int. Conf. Recent Adv. Geotec. Earth. Eng. & Soil Dyn., 1995, state of art report, vol. 2, pp. 851-879.

[2] K.H. Stokoe, M.B. Darendeli, R.D. Andrus, L.T. Brown. Dynamic Soil Properties: Laboratory, Field and Correlation Studies. In Proc. 2nd Int. Conf. on Earth. Geotechnical Eng., Lisbon, Portugal, Theme Lecture, 1999, vol. 3, pp. 811-845.

[3] S. Teachavorasiskun, P. Thongchim, P. Lukkunaprasit. Shear modulus and damping ratio of soft Bangkok clays. Canadian Geotechnical Journal, 2002, 39: 1201-1208.

[4] D.G. Anderson, A. Phukunhaphan, B.J. Douglas, G.R Martin. Cyclic behavior of six marine clays. Proc. ASCE Annual Conv., Houston, October 17-21, 1983.

[5] S. Frydman, M. Talesnick, G. Almagor, G. Wiseman. Simple shear testing for the study of earthquake response of clay from the Israeli continental slope. Marine Geotechnology, 1988, 7: 143-171.

[6] F. Trincardi, A. Asioli, A. Cattaneo, A. Correggiari, L. Vigliotti, C. A. Accorsi. Transgressive offshore deposits on the central Adriatic shelf: architecture complexity and the record of the younger dryas short-term event. Il Quaternario, 1996, 9: 753-762.

[7] G. Lanzo, A. Pagliaroli, P. Tommasi, F.L. Chiocci. Simple shear testing of sensitive, very soft offshore clay for wide strain range. Canadian Geotechnical Journal, 2009, 46(11), 1277-1288.

[8] J.B. Burland. On compressibility and shear strength of natural clays. Géotechnique 1990, 40(3): 329-378.

[9] M. Iorio, Liddicoat J., Budillon F., Incoronato A., Coe R.S., Insinga D.D., Cassata W., Lubritto C., Angelino A., Tamburrino S. Combined palaeomagnetic secular variation and petrophysical records to time-constrain geological and hazardous events: an example from the eastern Tyrrhenian Sea in the last 120 ka. Global and planetary change, 2014, 113, 91-109.

[10] T. Lunne, T. Berre, K.H. Andersen, S. Strandvik, M. Sjursen. Effects of sample disturbance and consolidation procedures on measured shear strength of soft marine Norwegian clays. Can. Geotech. J., 2006, 43, 72-750.

[11] B. D'Elia, G. Lanzo, A. Pagliaroli. Small-strain stiffness and damping of soils in a direct simple shear device. In Proceedings of the Pacific Conf. on Earth. Eng., Christchurch, New Zealand, 2003.

[12] M. Doroudian, M. Vucetic. A Direct Simple Shear Device for Measuring Small-Strain Behavior. Geotechnical Testing Journal, 1995, 18(1), 69-85.

[13] M. Vucetic. Cyclic threshold shear strains in soils. J. of Geotech. Eng., ASCE, 1994, 120(12), 2208-2228.

[14] M. Jamiolkowski, D.C.F. Lo Presti, E. Pasqualini E. Behaviour of Soils at Small Strains," Proc. Int. Symp. on Engineering Geology Problems in Seismic Areas, Bari, Italy. In: Geologia Applicata e Idrogeologia, 1986, vol XXI, pp. 213-238.

[15] S Likitlersuang, S. Teachavorasinskun, C. Surarak, E. Oh, A. Balasubramaniamc. Small strain stiffness and stiffness degradation curve of Bangkok Clays. Soils andFoundations, 2013, 53(4), 498–509.

[16] B.O. Hardin. The nature of stress-strain behaviour of soils. In Proc. Geotech. Eng. Div. Speciality Conf. on Erath. Eng. and Soil Dynamics, ASCE, Pasadena, 1978, vol. 1, pp. 3- 90.

[17] M. Jamiolkowski, R. Lancellotta, D.C.F. Lo Presti. Remarks on the stiffness at small strains of six italian clays. In Prefailure Deformation of Geomaterials, Balkema, 1994, vol. 2., pp. 817-836.

[18] G. Viggiani. Small-strain stiffness of fine-grained soils. Ph.D. thesis, City University, London, 1992.

[19] S. Rampello, F. Silvestri, G. Viggiani. The dependence of small strain stiffness on stress state and history of fine grained soils: the example of Vallericca clay. In Prefailure Deformation of Geomaterials, Balkema, 1994, vol. 1, pp. 273-279.

[20] P.W. Mayne, F.H. Kulhawy. K0-OCR relationships in soils. Journal of Geotech. Eng., ASCE, 1982, 108(GT6): 851-872.

[21] A. Cavallaro, G. Lanzo, A. Pagliaroli, M. Maugeri, D.C.F. Lo Presti. A comparative study on shear modulus and damping ratio of cohesive soil from laboratory tests. In Di Benedetto et al. (eds), Deformation Characteristics of Geomaterials 2003, Swets & Zeitlinger Publ., Netherlands, pp. 257-265.

[22] C. Mancuso, F. Silvestri, F. Vinale. Dinamica dei terreni. In: Manuale di Ingegneria Civile vol. 1, pp. 810-858, Zanichelli, 1996.

[23] M. Vucetic, R. Dobry, R.. Effect of soil plasticity on cyclic response. Journal of Geotech. Eng., ASCE, 1991, 117(1): 89-107.

[24] M. B. Darendeli. Development of a new family of normalized modulus reduction and material damping curves. Ph.D. Thesis, University, 2001.