

MECHANICAL BEHAVIOUR OF A POZZOLANA SOIL UNDER CYCLIC LOADINGS

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ABSTRACT

The paper summarizes an experimental campaign aimed at defining the mechanical behaviour of a pozzolana soil under cyclic loading. The material was sampled in a test site near Naples, Italy and was previously subjected to a detailed “conventional” investigation, in order to define its critical state condition. Experimental investigation was performed using a stress-path controlled triaxial apparatus. Due to some difficulties in sampling the soil in its natural state, tests were executed on remoulded samples reconsolidated under 1-D conditions starting from a water content equal to 1.2 w_L . Such preparation technique should lead to loose soil specimens, but instead the pozzolana soil has a moderate contractive behaviour. Such feature was confirmed by performing undrained cyclic tests, which were executed under controlled stress condition. Cyclic loading was applied at different amplitudes but keeping a constant loading frequency. The soil behaviour was interpreted in a traditional liquefaction framework, adopting a failure criterion related to a threshold deformation. In an attempt to testing the pozzolana soil in a looser state, a different preparation technique was adopted. The preparation consisted in putting the dry material in a mold filled with water. The resulting specimens were then frozen at -30°C. In this case, the experimental tests showed contractive behaviour and a liquefaction resistance remarkably lower than that of 1-D consolidated specimens.

Keywords: Liquefaction, triaxial tests, preparation technique, pore water pressure model.

INTRODUCTION

The analysis of the susceptibility to liquefaction of a given site requires, in general, the evaluation of the main characteristics of the expected seismic event (i.e., the triggering factor); the geotechnical characterization of the subsoil (i.e., the predisposing factor); and, the estimate of the consequent effects of the liquefaction. To evaluate the soil resistance to liquefaction two main approaches are usually followed: the first one is based on the results of laboratory tests, the second is focused on the results of in situ tests, together with observations of liquefaction behavior in past earthquakes. This paper summarises the results of a series of laboratory triaxial tests aimed at detecting the response of a pozzolana soil under large cyclic loadings and then its susceptibility to liquefaction. The

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material was obtained in a test site approximately 30 km SE of the city of Naples that recently experienced flowslides. Pozzolan materials constitute one of the main formations that are present in the city of Naples (see Santucci de Magistris and Evangelista, this conference). Pozzolan soils are, however, widespread in all Naples prefecture since they directly derives from the eruptions of the Somma-Vesuvius and Campi Flegrei volcanoes. Sometime pozzolan soils, in their natural state, have high value of porosity and a variable degree of saturation.

Several experimental investigations on the pozzolana soil from Naples prefecture were performed in the last decade at the University of Naples (see for instance Zingariello, 2006). However, the behaviour of this material under cyclic loading was never analyzed.

In this paper, first some physical characteristic of the soil are presented and then its behaviour under large cyclic loading is described. To better understand the obtained results, the general behaviour of the soil was pictured in the framework of the Critical State Soil Mechanics by mean of a series of monotonic tests. Finally, the soil behaviour was interpreted in a traditional liquefaction framework, adopting a conventional failure criterion.

PHYSICAL PROPERTIES AND EXPERIMENTAL PROCEDURES

The pyroclastic soil used for laboratory tests was taken, in a remoulded state, at the toe of the San Pantaleone hill, near the town of Nocera Inferiore (Salerno prefecture). This area was affected by flowslide events, the last one in 1997 triggered by a heavy rainfall.

In Figure 1 the grading curves of the tested material are reported. The natural material is a silty sand, that is well included in the threshold limits to evaluate the liquefaction susceptibility by Tsuchida, 1970 for soils having a uniform coefficient larger than 3.5. The gravelly fraction of the material subjected to mechanical tests was removed, so that the maximum particles diameter d_{max} is 2 mm.

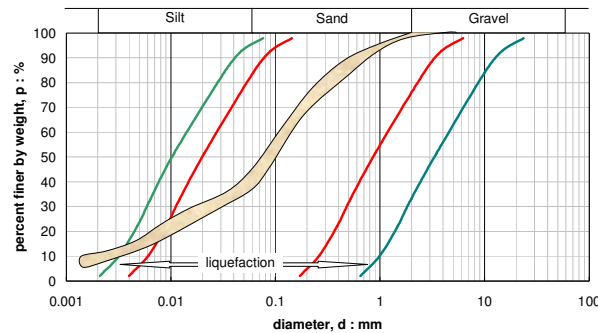


Figure 1. Grading curves of the San Pantaleone soil compared with the limit curves liquefaction by Tsuchida, 1970

The tested samples have specific gravity of soil solids $G_s = 2.66$, a liquid limit $w_L = 45.1\%$, a plasticity index $I_p = 10.9\%$. It was found an organic fraction of 5%. Maximum void ratio e_{max} (ASTM D4254-00) is 1.65, minimum void ratio e_{min} is 0.97. In its natural state the material has a high porosity; in situ measurements show a range varying from $n = 0.55$ to $n = 0.65$ (i.e., $e = 1.22 \div 1.86$, then even larger than e_{max}).

All mechanical tests were executed using a stress path controlled triaxial apparatus (Bishop and Wesley, 1975), slightly modified at the University of Naples (Aversa and Vinale, 1995). The axial stress in the soil specimen ($\varnothing = 38$ mm, $h = 76$ mm) was measured by means of an inner load cell, the axial strain by an external LVDT, and the volume change using a double-bellofram volume gauge.

To prepare the soil specimens for testing, two techniques were adopted: 1-D consolidation and freezing.

1-D consolidated specimens were prepared after mixing the dry disaggregated soil with water, to get a slurry having moisture content around 1.2 times the liquid limit. The slurry was one-dimensionally compressed up to a nominal $\sigma'_v=50$ kPa in a floating ring consolidometer, having an inner diameter of 15 cm and double drainage lines. Once consolidated, the sample was trimmed to the laboratory specimen size. In spite of the coarse nature of the soil particle, using this preparation technique specimen can stand without external confining pressure.

Frozen specimens were prepared putting the dry material in a mold filled with distilled water. The mold was simply made with an overhead transparent folded to make a cylinder having an inner diameter equal to 38 mm and an high of around 100 mm. The lower base of the mold was closed with a cap and then the soil was gently poured with a teaspoon. Then, the mold with the soil was frozen at -30°C . Before starting the mechanical test, the two bases were trimmed to have a specimen of 76 mm in high that was placed in the triaxial cell, after removing the folded overhead. To avoid a quick defrosting of the specimen and its subsequent collapse, the reservoir of the de-aired water used to fill the triaxial cell with and the triaxial chamber itself were packed with ice. Once the setup was completed the specimen was left for at least 24 hours under an effective isotropic stress state of 10 kPa before starting the consolidation stage, in order to allow for complete thawing of the material.

1-D consolidated specimens have initial specific volume v that varies in the range $2.16\div 2.24$ with a mean value of 2.21. The correspondent average porosity is the order of 55%. Instead, for the frozen specimens, the specific volume is contained in the range $2.37\div 2.38$, that correspond a porosity of 58%. It is evident that the second preparation technique is most efficient for obtaining specimens with similar and looser initial state. In this case, the limited scatter in initial porosity might be due to the fact that the material is prepared right at the dimensions of laboratory specimen, rather than being trimmed from a larger sample.

Table 1. Testing program

Test Number	Test Code	Preparation Technique	Max. consolidation pressure (kPa)	Shearing	
				Mode	Deviator Stress
1	CID 400	Consolidometer	400	CID	Monotonic To Failure
2	CIU 400	Consolidometer	400	CIU	Monotonic To Failure
3	CIU 100	Consolidometer	100	CIU	Monotonic To Failure
5	CP' 100	Consolidometer	100	CP'	Monotonic To Failure
6	CIU 200	Consolidometer	200	CIU	Monotonic To Failure
7	CID 200	Consolidometer	200	CID	Monotonic To Failure
8	CP' 200	Consolidometer	200	CP'	Monotonic To Failure
9	CID 100	Consolidometer	100	CID	Monotonic To Failure
11	CP' 400	Consolidometer	400	CP'	Monotonic To Failure
12	CLS 180C	Consolidometer	200	CLS	Cyclic 180 kPa
13	CLT 140C	Consolidometer	200	CLT	Cyclic 140 kPa
14	CLT 130C	Consolidometer	200	CLT	Cyclic 130 kPa
15	CLT 100C	Consolidometer	200	CLT	Cyclic 100 kPa
16	CLT 90C	Consolidometer	200	CLT	Cyclic 90 kPa
17	CLT 60C	Consolidometer	200	CLT	Cyclic 60 kPa
18	CLT 180C	Consolidometer	200	CLT	Cyclic 180 kPa
19	CLT 75C	Consolidometer	200	CLT	Cyclic 75 kPa
23	CLT 80F	Freezing	200	CLT	Cyclic 80 kPa
24	CLT 90F	Freezing	200	CLT	Cyclic 90 kPa
25	CLT 60F	Freezing	200	CLT	Cyclic 60 kPa

26	CID 200F	Freezing	200	CID	Monotonic To Failure
28	CLT 50F	Freezing	200	CLT	Cyclic 50 kPa

CID	Drained Shear	CP'	Constant mean effective stress	CLS	Cyclic Loading - Sinusoidal Law
CIU	Undrained Shear			CLT	Cyclic Loading - Sawtooth Law

Overall, twenty-two tests were performed. Generally, all specimens were isotropically compressed up to a mean effective stress of 200 kPa, even though in some cases a final consolidation stress of 100 or 400 kPa was reached. A stress rate of 10 kPa/hr ensures drained condition during this stage. At the end of compression, all specimens were left to consolidate for at least 24 hours.

Therefore, specimens were subjected:

1. triaxial compression monotonic loading under different stress paths up to the critical state; or,
2. undrained cyclic loading at a given stress frequency and amplitude until failure.

Type (1) tests were executed at a constant strain rate equal to 1%/h for undrained test and 0.1%/h for drained. Type (2) tests were executed under stress controlled condition applying a sawtooth time law of deviator stress having a single amplitude variable from 50 to 180 kPa that was applied at a fixed frequency of 1 cycle per minute (0.0167 Hz).

A summary of the testing program is reported in Table I.

BEHAVIOUR IN THE CRITICAL STATE SOIL MECHANICS FRAMEWORK

Before describing the behaviour of the San Pantaleone silty sand under cyclic loading, here its general behaviour is drawing in the framework of the Critical State Soil Mechanics. To this end, we used the initial isotropic compression stage for all the executed tests, and the monotonic shearing stage for specimens prevalently prepared with the 1-D consolidated procedure.

As underlined before, the most important difference for the specimens prepared with the two methods described above is the initial density or the initial void ratio. This is a key feature for evaluating the mechanical behaviour of the material. Particularly, as underlined for instance by Wood, 1991, it is important to compare the soil current state in the specific volume v : mean effective stress p' plane with respect to the critical state line, rather than using the relative density only.

Compressibility

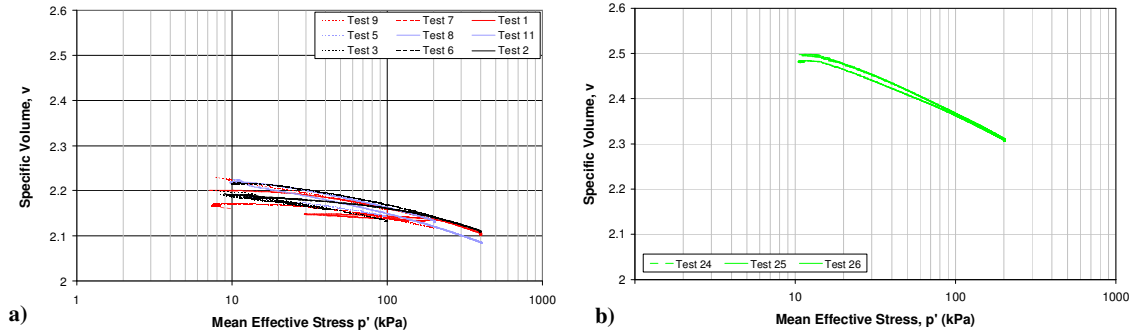


Figure 2. Compression curves for specimen prepared: a) with 1-D consolidometer; b) freezing

In Figure 2 the isotropic compression curves for the 1-D consolidated specimens (Figure 2a) and for the water sedimented frozen specimens (Figure 2b) are separately plotted in a semi-logarithmic plane. Apart from the different initial state, it clearly appears that the latter have a larger compressibility than the former. Fitting the experimental data (the first loading only) with the well-known relationships:

$$v = N - \lambda \ln p' \quad (1)$$

it results that for the water sedimented frozen specimens $N = 2.57$ and $\lambda = 0.069$ and for the 1-D consolidated specimens $N = 2.256$ and $\lambda = 0.023$.

It should be noted that for one 1-D consolidated specimen (test 1 of Table I) an unloading-reloading loop was executed and a clear different pattern in the $\langle v:p' \rangle$ plane can be detected.

As reported in several literature papers, the overall differences in compressibility might be attributed to the different initial void ratio and fabric that result from the different preparation techniques.

Shearing behaviour

In order to qualitatively compare the mechanical behaviour of the remoulded samples of the pozzolana soil prepared with 1-D consolidation and water sedimented freezing methods under shear stress, Figure 3 report the results of two consolidated-drained triaxial compression tests (CID), both at an isotropic consolidation pressure $p'_0 = 200$ kPa (Test 7, Test 26). The experimental data are plotted in terms of variation of deviator stress, q , and volumetric strain ε_v with the axial strain, ε_a .

In agreement with the previous considerations on the different initial state of the specimen reconstituted with the two preparation methods, the observed behaviour appears to be different. In particular, while the water sedimented frozen specimen has a ductile and contractive stress-strain response, the 1-D consolidated material exhibits first a contractive behaviour followed by dilation. The volumetric behaviour is reflected into the deviator stress q - axial strain ε_a curve: the peak strength was reached after the peak volume strain and approximately at axial strain of 9% while for $\varepsilon_a \approx 6\%$, the volumetric strain reaches the maximum value.

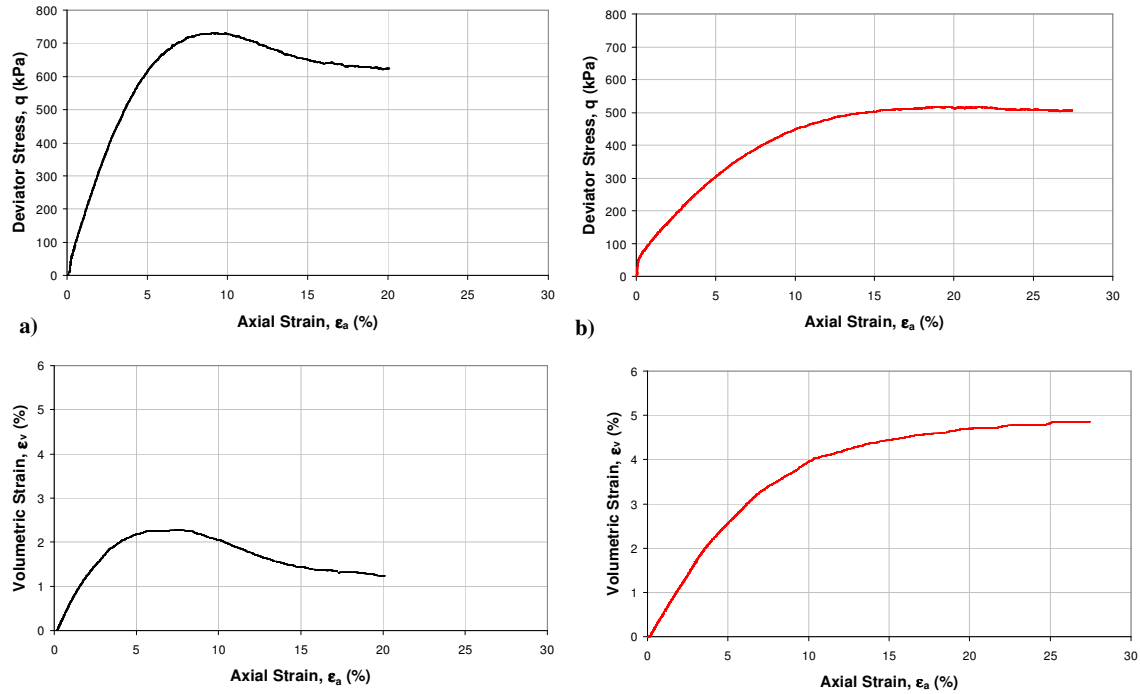


Figure 3. Typical stress-strain response in a conventional drained triaxial test at $p'=200$ kPa: (a) 1-D consolidated specimen and (b) water sedimented frozen specimen

A critical state condition seem be reached in the two tests here illustrated.

Figure 4 shows all the stress-strain response obtained from the tests on 1-D consolidated specimens with different effective stress-paths. The deviator stress q is normalized respect to mean effective stress p' . Except to two tests (Test 1 and Test 6), at large deformations all the experimental curves collapse into a unique horizontal line (failure conditions). The final point of this condition is assumed to be the critical state for the specimen.

In figure 5 are plotted the effective stress paths followed in the tests in the $\langle q:p' \rangle$ plane and the $\langle v : \ln(p') \rangle$ plane. To reduce scattering in the results, all the isotropic compression curves of the 1-D consolidated specimens were translated, so that they collapse into a unique point at $p'=100$ kPa. The open squares represent the critical state points (CSP).

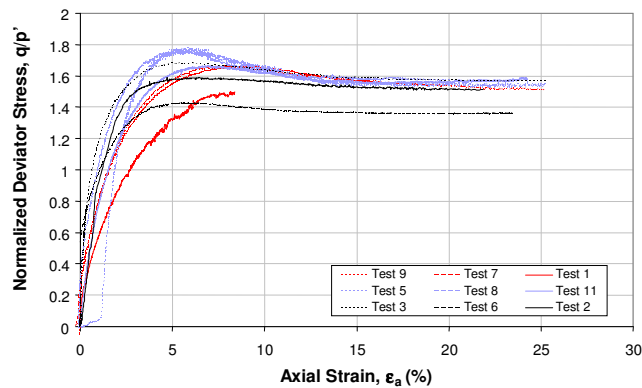


Figure 4. Stress-strain response for ODCM specimens in terms of normalized deviator stress (q/p') and axial strains

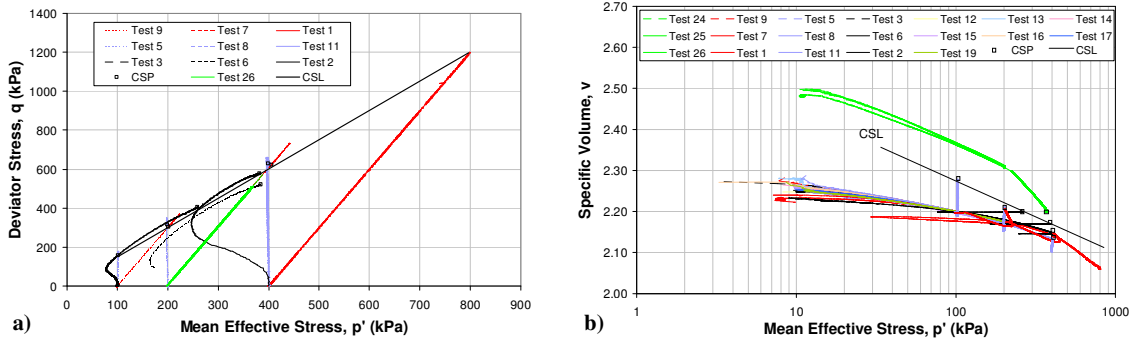


Figure 5. Behaviour of 1-D consolidated specimens in (a) $\langle q:p' \rangle$ and (b) $\langle v:p' \rangle$ plane

In the $\langle q:p':v \rangle$ three-dimensional space, the CSP describe a unique ultimate locus, regardless of the test pattern and the level of confinement. The critical state can be analytically expressed by:

$$\begin{aligned} q_{cs} &= Mp'_{cs} \\ v_{cs} &= \Gamma - \lambda_{cs} \ln p'_{cs} \end{aligned} \quad (2)$$

For this soil, it was found that $M=1.55$, corresponding to a critical state friction angle $\phi'_{cs} \approx 38^\circ$, $\lambda_{cs} = 0.0944$ and $\Gamma = 2.6642$.

Discussion

As it was previously underlined, the studied pozzolana soil has high porosity in its natural state. The 1-D consolidation preparation technique was then adopted, since it was able to produce specimens with a density close to the minimum standard density on a soil with a similar grading composition (Santucci de Magistris et al., 1998). Rather, this technique on San Pantaleone silty sand produces specimens having a relatively high density, i.e. their isotropic compression line lays below the critical state line, as can be observed looking at Figure 5b. Here, it was confirmed first that as the initial density changes from a looser to a denser state, the compression curve moves downwards and flattens. That implies a less compressible and more dilatant behaviour. It was then confirmed that the critical state line in the $v:p'$ plane has a slope similar to the isotropic compression line of very loose soils that was obtained, in this specific case, using the water sedimentation and then freezing technique.

It should be noted also that other preparation techniques, including wet tamping, Proctor compaction and air pluviation do not affect critical state behaviour of the San Pantaleone soil, both in $\langle q:p' \rangle$ and in $\langle v:p' \rangle$ plane (Parlato, 2006).

BEHAVIOUR OF SOIL UNDER CYCLIC LOADINGS

The mechanical behaviour of the San Pantaleone silty sand under cyclic loadings was detected by applying at each specimen a triangular loading history having a fixed amplitude and frequency. Such a loading shape allows to impose a fixed stress rate and to control the test in a smoother way, comparing with the more traditional sinusoidal loading.

To check if the loading shape might affect the subsequent mechanical behaviour, in Figure 6 the results of two tests (Test 12 and Test 18), performed in otherwise the same conditions but the loading shape, are compared. Only the first eight cycles are plotted. It can be seen that no macroscopic differences affected the two tests. This statement can be better understood looking at Figure 7, where the results of the previous two tests are summarized by means of the variation of the so called “equivalent parameters”, Young’s modulus and damping ratio, with the number of cycles. The values of the Young’s modulus are very close, except for the first cycle. Some differences might be seen for

the damping ratio, but the discrepancies could be attributing to the scattering of data that is somehow noticeable for the specimen loaded using a sinusoidal law.

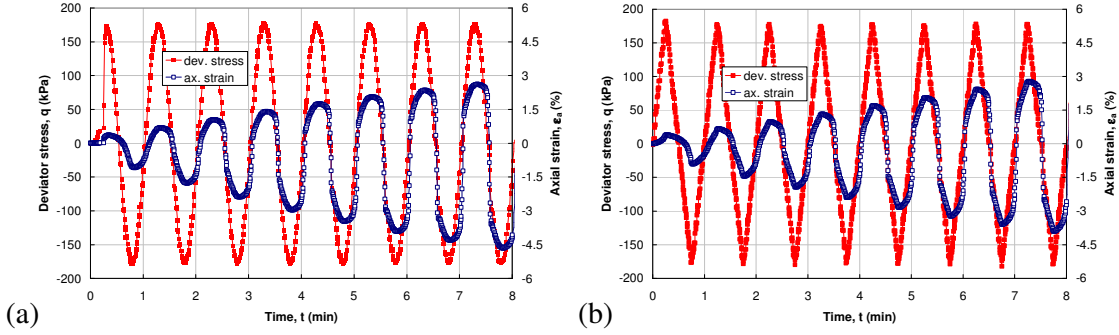


Figure 6. Time history of loading and axial deformation for: (a) Test 12, sinusoidal loading and (b) Test 18, sawtooth loading

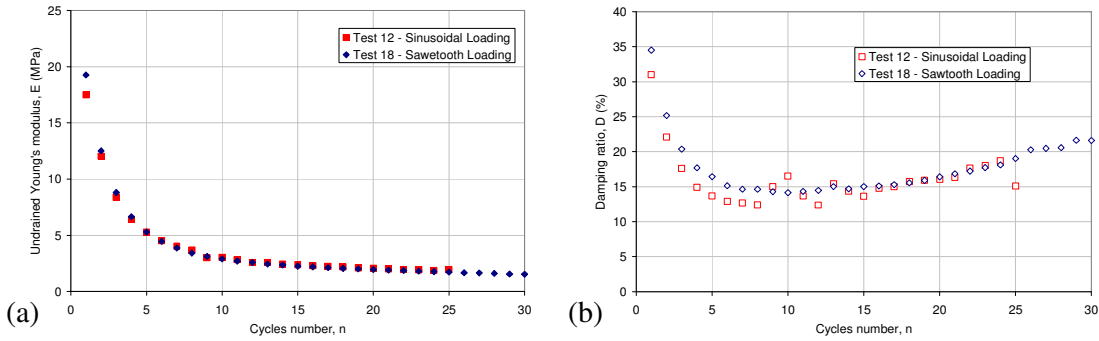


Figure 7. Variation for two specimens subjected to sinusoidal and sawtooth cyclic loading with the number of cycles for: (a) Young's modulus, E and (b) damping ratio, D

In Figure 8 instead, the comparison is made between the cyclic behaviour of two other specimens (Test 16 and Test 24) subjected to the same loading conditions but prepared with 1-D consolidation and with water sedimentation and freezing and then having different densities at the end of the isotropic compression stage. The comparison is made in terms of time histories of loading and axial strain, normalized excess pore water pressure $\Delta u_q/p'_0$ (Δu_q represents the pore pressure increment due to the distortional deformations only, thus subtracting the pressure variations that in a triaxial test with constant cell pressure are induced by the total stress variation), stress-strain relationship and effective stress path.

While the results of both tests are affected by cyclic degradation, some clear differences between the two specimens can be easily noticed, since the 1-D consolidated material reaches a failure condition after a number of cycles that is almost ten times larger than the frozen specimen. For the 1-D consolidated specimen axial strain starts growing after about 25 cycles, and except for the initial stages, stress-strain curves are clearly asymmetric: the absolute values of negative strain are larger than positive strain, at the beginning of the test, the opposite for a larger number of cycles.

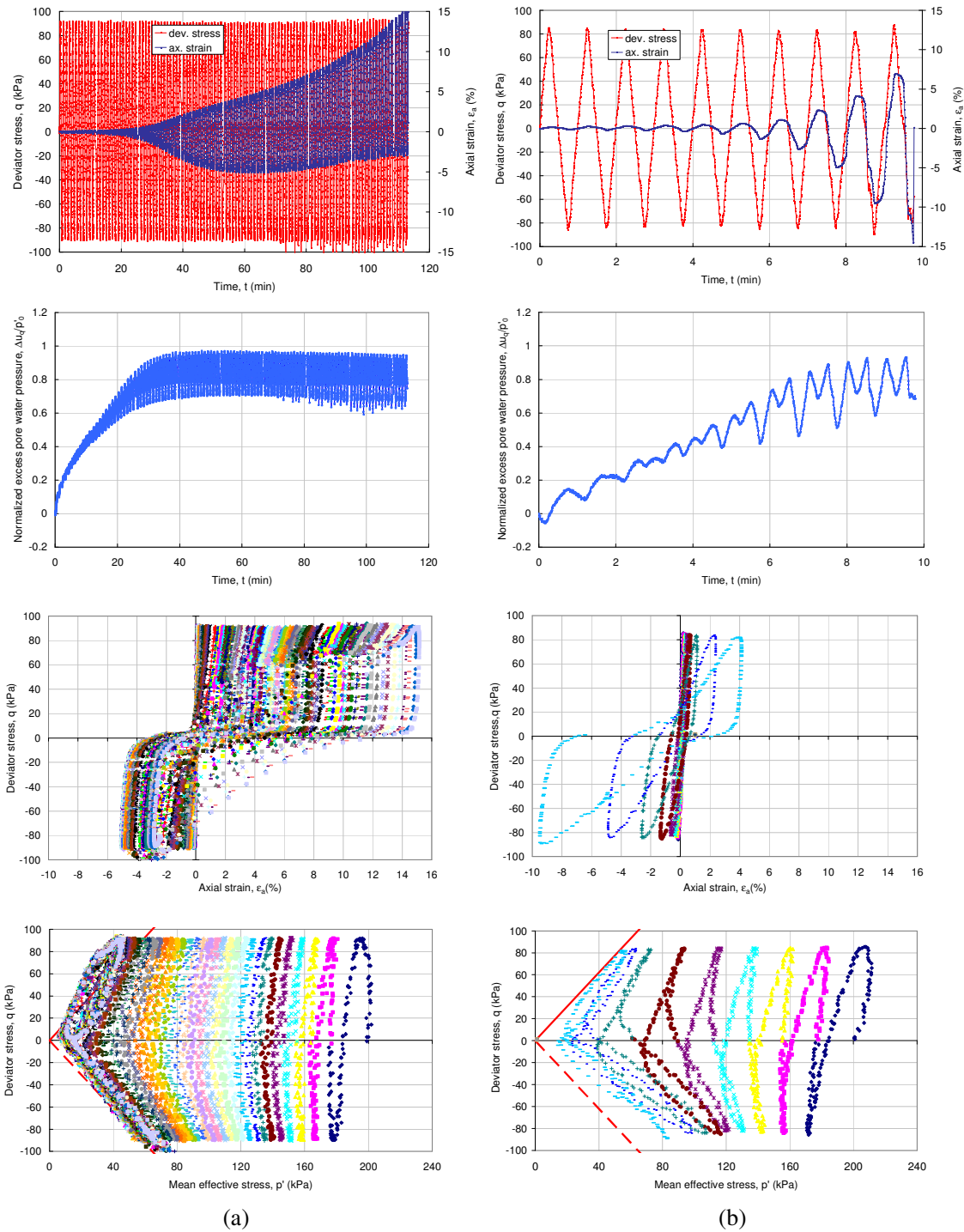


Figure 8. Comparisons in terms of deviator stress, axial strain and normalized excess pore water pressure time histories, cyclic stress-strain loops and stress paths between two specimens of Pozzolana Soil 1-D consolidated (a) (Test 16) and frozen (b) (Test 24)

Significant increment of pore water pressure can be observed after around 30 cycles. However, while axial strain continuing growing throughout the test, the pore water pressure reaches its maximum value after around 40 cycles. There, the normalized value of the $\Delta u/p'_0$ ratio, averaged during a cycle, reach the value of around 0.85. Due to the pore water pressure development, during the test the effective stress path moves leftward. The paths tend to slightly overcome the critical state line

obtained from the previous described monotonic compression test. Similar results were obtained on compacted silty sand (Santucci de Magistris & Parlato, 2003). Please notice, however, that the critical state line for extension tests is not available, at the moment, for the tested material.

The loose frozen specimen shows relevant axial deformation after about 8 cycles. Axial strains are almost symmetrical, even though at the tenth cycle the specimen fails during the extension load. Pore water pressure constantly rises during the test and, after the seventh cycle, the diagram becomes somehow irregular, confirming other literature data (see for instance Ishihara, 1996). Also, in this test the effective stress path moves leftward and the specimen fail touching the critical state line detected using monotonic tests.

If we consider another specimen (Test 25) prepared with the freezing technique but subjected to cyclic loading of a lower amplitude (60 kPa in this case) it can be more clearly understand how sudden deformations arise after a certain number of cycles, while pore water pressure develops in a more regular fashion (Figure 9).

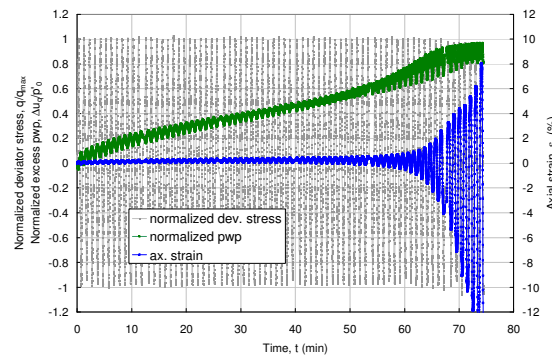


Figure 9. Normalized deviator stress, normalized excess of pore water pressure and axial strain time histories for a frozen specimen (Test 25) cyclically loaded with a low amplitude (60 kPa)

In analyzing the overall behaviour of the San Pantaleone pozzolana soil under cyclic loadings, it is necessary to establish an objective criterion to define the failure of the material. Then, the correspondent number of cycles that are necessary to bring the soil to fail can be evaluated.

The adopted criterion is based on setting a threshold value on the axial deformations, since in all the executed tests the excess pore water pressure ratio $\Delta u_d/p'_0$ never reach clearly the unit value. As previously illustrated, the deformations increase at increasing of the number of cycles, but a sudden growth of deformation is individuated for the water sedimentation and then frozen specimens while such increment is gentle for 1-D consolidated specimens. Therefore, the failure was conventionally established when a threshold deformation equal to 5% double amplitude is overtaken.

In Figure 10, for the studied pozzolana soil are reported the cyclic strength curves relevant for the specimens prepared with the two different techniques. The vertical axis of the diagram is the cyclic stress ratio CSR defined as the ratio of the maximum cyclic shear stress to the initial confining pressure; on the horizontal axis there is the number of cycles to produce failure or liquefaction. In the figure the two open red squares are due to two tests on 1-D consolidated specimens (Test 17 and Test 19) subjected to cyclic loading of an amplitude of 60 and 75 kPa that were stopped after 138 and 150 cycles respectively without reaching the conventional failure condition. Therefore the strength curve for 1-D consolidated specimen should be located above the points.

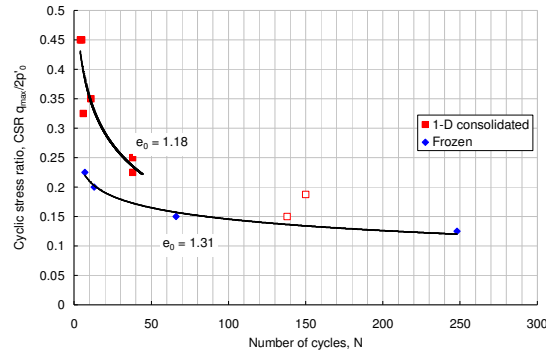


Figure 10. Cyclic strength curves for 1-D consolidated (red squares) and frozen (blue diamonds) specimens of San Pantaleone pozzolana soil

From the figure it can be directly appreciated the large differences in strength for the two set of data induced by the different porosity and fabric. If, for instance, a procedure to evaluate the liquefaction susceptibility of a site requires the evaluation of the soil strength after 15 cycles, this values moves from about 0.19 for 1-D consolidated medium dense specimens to 0.32 for the loose frozen specimens.

CONCLUSIONS

The paper summarises a series of triaxial tests aimed at analyzing the behaviour of a pozzolana soil under monotonic and cyclic loading, using two different preparation techniques, namely the 1-D consolidation method and the water deposition method followed by freezing.

The main finding of the research can be summarized in the following points.

1. 1-D consolidated specimens have an isotropic compression line that lies below the critical state line and then they have a shearing behaviour that is typical of that of medium-dense materials, both under monotonic or cyclic loading condition. This is in contrast with observations made on the behaviour of 1-D consolidated specimens of weathered granite of similar grading composition (Santucci de Magistris et al., 1998) that have a very loose initial state.
2. On the contrary, specimens prepared using water deposition and then freezing have an isotropic compression line located above the critical state line and then behave as loose soils.
3. No difference seems to exist between the cyclic behaviour of specimens tested under the same conditions except for the shape of the applied loading history (triangular or sinusoidal).
4. The initial density and the preparation technique influence in a relevant manner the cyclic strength, confirming what was already observed in several published paper.

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