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CYCLIC SHEAR STRENGTH OF A VOLCANIC SANDY SILT SUSCEPTIBLETO FLOWSLIDE

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ABSTRACT

The paper provides insight into the cyclic behaviour of volcanic sandy-silt material susceptible to flowslides.

Flowslides and liquefied debris flows are typical phenomena in the Calabria region (Italy) following intense and/or prolonged rainstorms. In particular, a catastrophic event occurred in April, 2000, which caused very destructive impacts on the existing down-hill structures. Such phenomena involve sandy-silt materials of volcanic origin, covering the steep slopes located along the calabrian coast of the Messina Strait, near the city of Reggio Calabria.

These shallow deposits display key properties of volcanic materials, such as low plasticity, high porosity, low density and tendency to collapse upon load application.

The experimental site has been investigated through detailed stratigraphic borings, trial pits and Standard Penetration Tests (SPT). Undisturbed samples were carefully recovered by block and tube samples from material exposed in carefully dug trial pits.

To establish a more consistent theoretical frame for interpreting the observed cyclic behaviour, undisturbed samples were initially subjected to one dimensional compression tests and isotropically consolidated drained and undrained triaxial compression tests. Laboratory undrained cyclic simple shear tests in a modified NGI type apparatus provide a valuable means of assessing material susceptibility to liquefaction or cyclic failure of the recovered samples**.** Cyclic test results were carefully analysed with the aim of identifying the influence of combination of high limit liquid and low plasticity index on the observed behaviour.

The results gathered in the present research in terms of undrained cyclic resistance vs. SPT, fall well in the range of other field performance-based data of silty sands, sandy silts and silts with fine content (FC) equal to or higher than 35%, at a given equivalent earthquake magnitude.

Keywords: sandy silts, volcanic materials, cyclic behaviour, simple shear.

INTRODUCTION

Recent experiences with seismically induced flowslides in low-plasticity silts during seismic behaviour have highlighted the need for an improved fundamental understanding of their seismic behaviour (Ishihara, 1993; Amini and Qi, 2000; Chien et al., 2002; Thevanayagan and Martin, 2002; Nabeshima and Matsui, 2003; Boulanger and Idriss, 2004; Hyde et al., 2004). Flowslides in saturated silty soils can also develop under static conditions as a consequence of undrained deformational mechanisms that give rise to

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high excess pore water pressures up to triggering spontaneous static liquefaction (Yamamuro and Lade, 1999; Picarelli et al., 2006).

In a considerable part of the southern Italian region of Calabria, flowslides and liquefied debris flows are rather common phenomena following intense and/or prolonged rainstorms. In particular, large flowslides occur periodically between Scilla and Bagnara Calabra, two towns near Reggio Calabria. The most relevant event involving silty sand to sandy-silt volcanic materials, occurred on April 20 th 2000, when a train went off the rails and an industrial gas pumping plant was damaged. Furthermore, both railway and main road structures suffered some damage. It is plausible that these phenomena will become more and more numerous and hazardous under earthquake loading conditions.

The involved materials were characterised by peculiar properties of volcanic materials such as high initial void ratio and low plasticity index (Shimizu, 1998; Miura et al., 2003; Sahaphol and Miura, 2005; Picarelli et al., 2006; 2008a). Recent research on these materials (Picarelli et al., 2008b) showed that the initial soil density has an important role regarding the pattern of slope movement, i.e. the type of landslide which can be induced by failure under static conditions. The prediction of their behaviour under cyclic loading is fundamental for earthquake induced flowslide risk assessment. However, if realistic analyses are to be done, specific information is needed on their mechanical properties under both monotonic and cyclic loading conditions to overcome uncertainties connected with their unusual properties. This was highlighted by Boulanger and Idriss (2004) in a recent report: "when soils are characterized by unusual properties, such as a combination of high limit liquid and low plasticity index, it is recommended to perform an appropriate programme of in-situ and laboratory testing to evaluate stress-strain behaviour during monotonic and cyclic undrained shear loading". Furthermore, Hungr et al. (2001) pointed out that, for such types of materials, laboratory tests for assessing liquefaction susceptibility can be considered a suitable means of evaluating the potentiality for flowslide to occur.

In this paper, the cyclic stress-strain-strength type behaviour of the investigated material was characterized by means of undrained cyclic direct simple shear (DSS) tests on undisturbed block and tube samples under both stress and strain-controlled loading mode.

Taking into account the unusual features of tested material, attention is focused on the pattern of behaviour observed during cyclic loading ("sand-type" or "clay-type") leading to liquefaction or cyclic failure. (Boulanger and Idriss, 2004). Additionally, for a better and more complete recognition of tested material, both compressibility characteristics and monotonic shearing behaviour were also assessed.

GEOTECHNICAL INVESTIGATION AT SITE

The site under investigation is located at Melia di Scilla, a small village near Reggio Calabria on the calabrian coast of the Messina Strait (Figure 1).

Figure 1. Geographic map showing the location of the sampling site at Melia di Scilla.

On this site, during the catastrophic event of 2000, an important role was played by meteorological events, such as an intense and delayed period of rain that can lead to complete saturation of the soil even in the usually unsaturated upper zones. Photographs showing the effects of this event are presented in Figure 2. The type of landslides consists of a liquefied solid body which is very similar to the flow of a viscous fluid. (Hungr et al., 2001). The movement occurred on a steep slope with an angle ranging from 25° to 40 °.

(a)

(b)

Figure 2. Photographs showing (a) the crown scarp and (b) the downstream effects on the main road during the 2000 flowslide, at Melia di Scilla.

Deposits covering natural slopes in this area consist of unsaturated soil layers with an underlying crystalline bedrock of metamorphic origin, which appears rather weathered. Geological information supports the hypothesis that air-fall deposits of volcanic ashes may have accumulated in this area from the nearby eruptive centres (Etna, Stromboli active volcanos).
In general, fabric and state properties of these types of materials depend on their mode of deposition (Picarelli et al., 2006). Furthermore, in unsaturated conditions, suction can play a significant role on their mechanical behaviour exerting a marked influence on the stability of natural slopes.

Site investigation
The site was investigated through several stratigraphic borings and standard penetration tests (SPT) at different depths. It was possible (Figure 3) to identify layers of sandy-silt materials (volcanic ash) whose total thickness is a function of the slope angle and which are generally limited to a few meters. In the same Figure, the profile of the SPT number of blows, at 3 verticals carried out up to the depth of bedrock, is reported. N_{SPT} values are referred to an energy ratio (ER) of about 60%, in agreement with italian

practice. Afterwards, undisturbed sampling was performed by recovering block and tube samples by hand at shallow depths from material exposed in carefully dug trial pits. The pits were dug to a maximum depth of around 3.50 m.

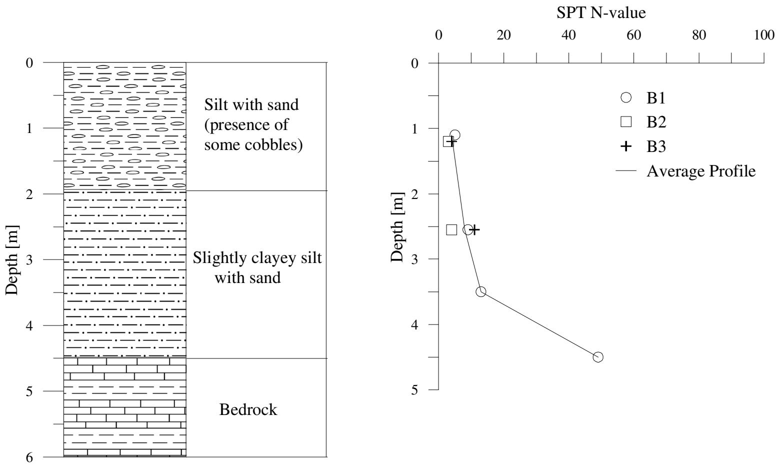


Figure 3. Stratigraphic profile and results of standard penetration tests carried out at investigated site.

LABORATORY INVESTIGATION

Material and test programme

A grain size analysis was performed on representative recovered samples. As shown in Figure 4, it is a well-graded material containing 66% silt and 28% sand. The fines content was equal to 72% with a low clay fraction (≈6%). The fine-grained component has low-plasticity characteristics. The basic properties were the following: specific gravity G_s=2.691; liquid limit LL=65.5%; plastic limit PL=54.3%; plasticity index PI=11.2%. According to the Unified Soil Classification System (ASTM D 2487) the material was classified as MH (inorganic silt of high compressibility).

Natural material displays some distinctive physical properties of piroclastic soils: high porosity, approximately 67%, and, low unit weight (γ_n=11.5 kN/m^3).

The compressibility characteristics of the natural soil material were investigated through oedometer tests carried out on samples retrieved from cubic samples at a depth of 1.50 m (Figure 5). It can be noticed that, for the tested MH material, the ratio between swelling index (C_s) and compression index (C_c) was found to be very low (0.04). Hydraulic conductivity from oedoemeter tests ranged from $8{\cdot}10^{-8}$ to $1.6{\cdot}10^{-9}$ m/sec.

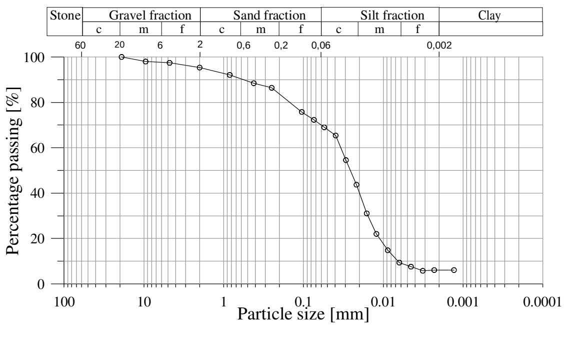


Figure 4. Grain-size distribution curve of tested material

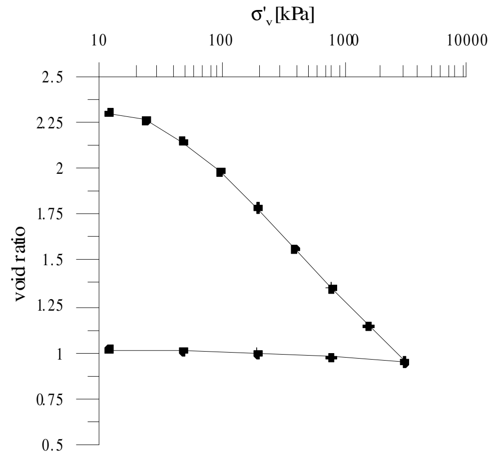


Figure 5. One-dimensional compression curve of tested material

The shear strength of recovered samples was investigated in monotonic drained and undrained triaxial compression tests. The undisturbed samples, recovered at a depth of about 1.50 m, were first saturated and then isotropically consolidated at different effective consolidation stresses $(\sigma_c^{'})$.
In order to properly simulate the in situ stress state acting on a soil element before and during an earthquake, undrained cyclic DSS tests under both stress and strain controlled mode were performed on samples recovered at a depth of 3.50 m. The laboratory test equipment is an NGI type DSS apparatus (Bjerrum and Landva, 1966) which was properly modified by an automated control system in order to perform monotonic and cyclic undrained tests under both stress and strain-controlled loading mode (Porcino et al., 2006). The test specimen was about 80 mm in diameter and 20 mm in height and was confined using a steel wire reinforced membrane. The important feature of this equipment is that cyclic undrained conditions are applied under constant volume conditions. The pore pressure in constant volume simple shear tests is always atmospheric and thus the change in total vertical stress during shear equals the excess pore pressure generated in an equivalent undrained test (Dyvik et al., 1987).
All cyclic tests were conducted at a frequency of 0.02 Hz. Cyclic tests under strain-controlled mode were performed at large strains, applying a uniform cyclic shear strain equal to 3.75%, generally assumed as a failure criterion in the laboratory tests.

Behaviour during shear
Figure 6 shows test results of drained triaxial compression tests carried out on the tested samples. In particular, in the same Figure, stress-strain curves (Figure 6a), volumetric response (Figure 6b) and stress-paths (Figure 6c) are depicted.
During the shearing phase under drained conditions, the material appears ductile and contractive (Figure 6a and 6b). This is consistent with the very loose state of testing material.
The peak failure envelope, coincident with ultimate state conditions, presents an internal friction angle equal to 35° in the range typically indicated for volcanic ash materials (33° to 40°), and the cohesion is zero. Consistent with compressional volumetric behaviour shown in Figure 6b, a large positive excess pore water pressure is expected to develop in corresponding tests carried out under undrained conditions.

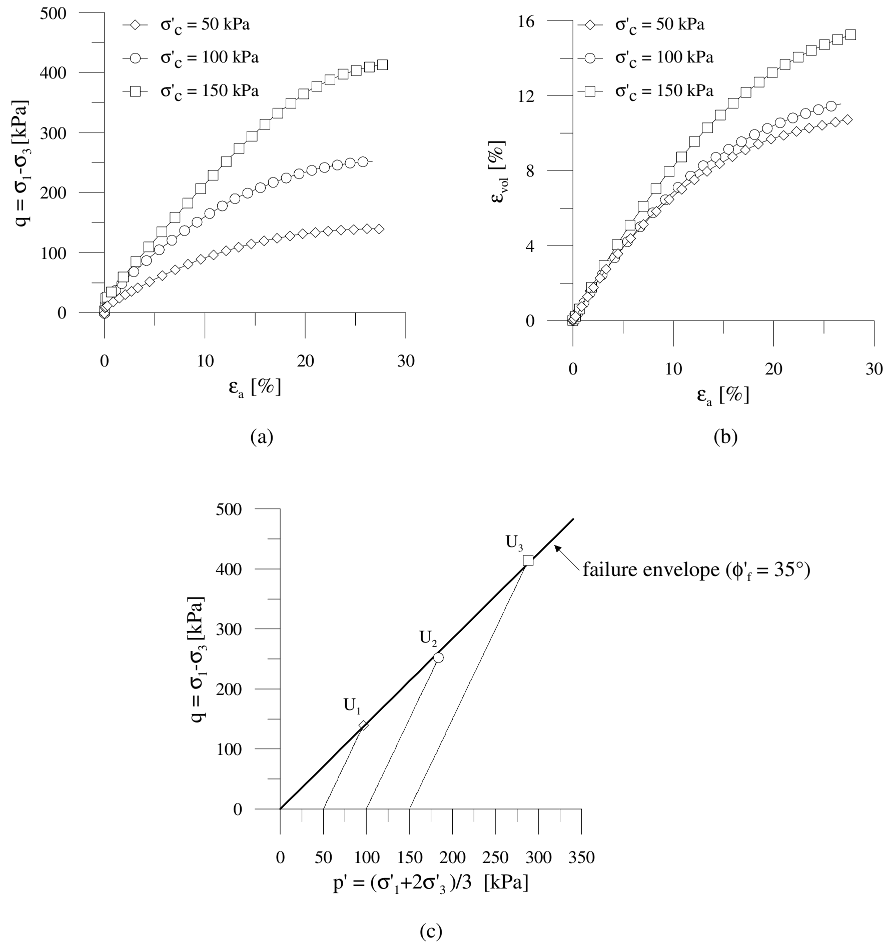


Figure 6. Drained triaxial compression test results of tested material

This is clearly displayed in Figure 7, reporting stress-strain curve (Figure 7a), pore water pressure response (Figure 7b) and stress-path (Figure 7c) from undrained monotonic triaxial compression tests. Results refer to a sample, isotropically consolidated at an effective consolidation stress $\sigma_c^{'} = 200\,\text{kPa}$. As can be argued from data in Figure 7b, values of the Skempton parameter (A) greater than unity were observed. This is indicative of the collapsible nature of highly porous structure upon load application.
The sample exhibits a brittle behaviour provoked by generation of a large positive excess pore water pressure induced by shearing, with a tendency of deviatoric stress to remain relatively constant as axial strain increased from about 5 to 20%.
It is worth noting that the pattern of response observed in the above tests is very similar to that observed for normally consolidated soft clays. This is consistent with Figure 8, in which the initial state, in terms of void ratio (e) and mean effective stress $(p^{'})$ with respect to Critical State/Steady State Line (CSL/SSL) position, is shown.
Data from one-dimensional compression tests were used for defining normal compression line while CSL/SSL was inferred from triaxial tests. In the same Figure, for the purpose of clarity, data points relative to initial (I) and ultimate (U) state conditions for two of the triaxial tests presented in Figures 6 and 7, are reported. The points have their initial conditions above CSL and the observed response during shearing is consistent with their initial state conditions.

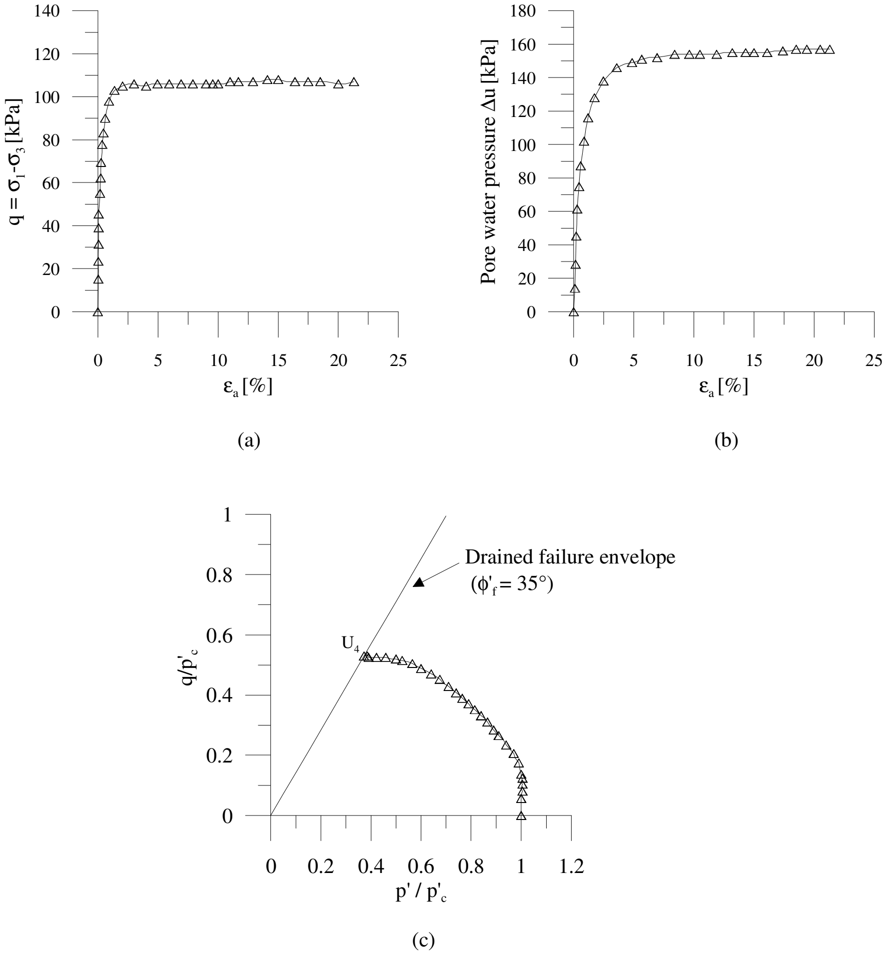


Figure 7. Undrained triaxial compression test results of tested material

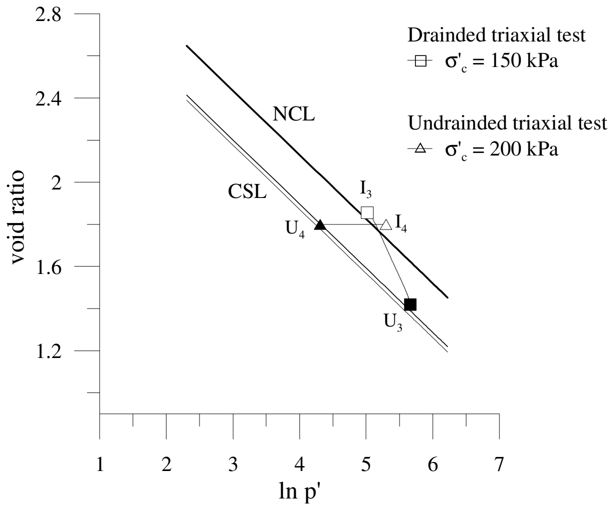


Figure 8. Normal compression line (NCL) and critical state line (CSL) in (lnp':e) compression plane.

It is worth noting that the NCL and CSL for tested material are essentially parallel; however, the slope of the lines assume typically high values ($\lambda = 0.30$).

Cyclic behaviour in undrained SS tests

Typical results from undrained cyclic simple shear tests under strain-controlled conditions are shown in Figure 9. Results are relative to an undisturbed sample, reconsolidated at the same in-situ effective overburden stress $(\sigma'_{v0} = 40\,\text{kPa})$. Cyclic shear strain controlled tests were performed applying a uniform cyclic shear strain amplitude, $\gamma_c = \pm 3.75\%$ for 30 cycles before stopping the tests.

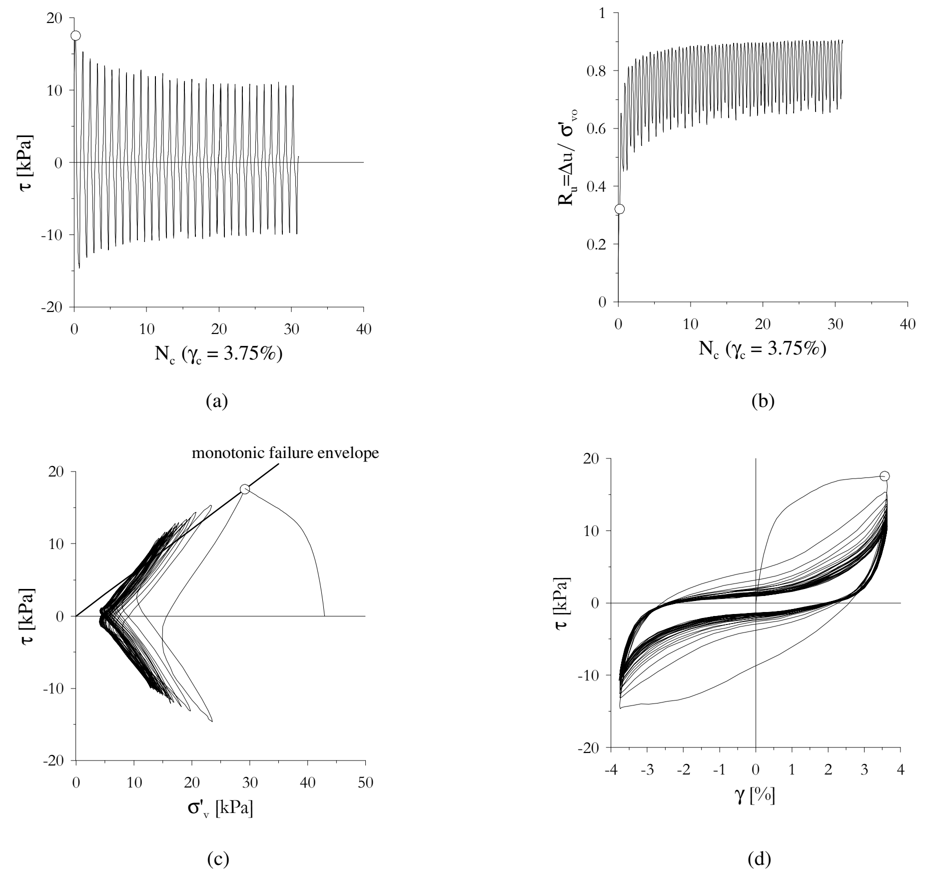


Figure 9. Cyclic undrained response under strain-controlled simple shear conditions ($\gamma_c = \pm 3.75\%$) on undisturbed samples of tested material.

In particular, the graphs in Figure 9 show the following: trend with number of cycles (N_c) of cyclically induced shear stress (τ) (Figure 9a) and pore water pressure ratio $(R_u = \Delta u / \sigma'_{v0})$ (Figure 9b), effective stress-paths followed during the tests (Figure 9c) and finally stress-strain loops (Figure 9d).

Figure 9a shows that the excess pore-pressure ratio rapidly increases at the beginning of cyclic strain application; afterwards, the pore pressure increases with the number of cycles at a very small rate with a tendency to remain close to a terminal value of $R_{u\,max} = 0.90$. Figure 9d presents typical stress–strain loops measured at different cycles and the initial backbone curve, corresponding to N_c=1, is clearly evident. It is remarkable to note that the amplitude of applied cyclic shear strain pushes the effective stress path up to the level of the same monotonic strength envelope, gathered from triaxial tests (Figure

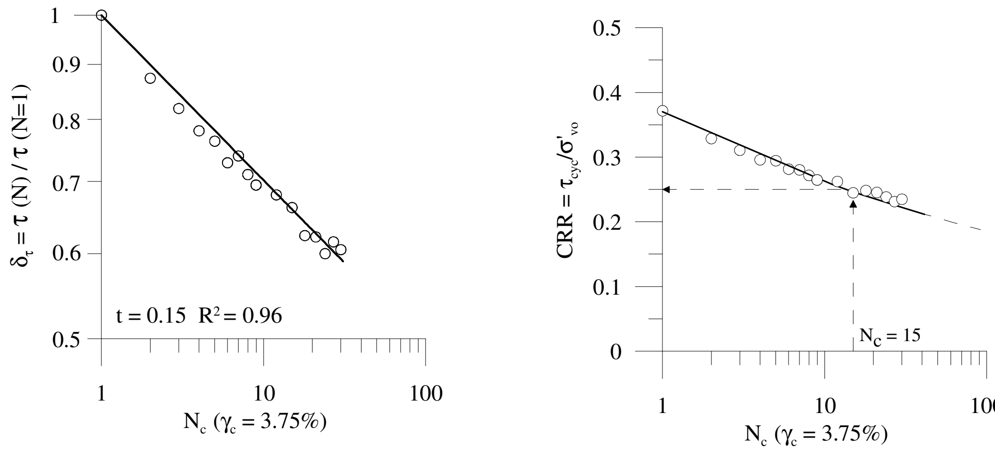
9c); afterwards, the effective stress path moves towards the origin without actually reaching a state of zero effective stress. During cyclic loading with a strain amplitude above a critical value, there occurs a continuous loss of strength and stiffness with the number of cycles. As a result, under strain controlled loading the measured cyclic stress level comes down with the number of cycles (Figure 9b and 9d).
To quantify such reduction, Idriss et al. (1978) introduced the concept of the degradation index, δ, for normally consolidated clays based on the results of cyclic strain controlled tests. This degradation index is considered a measure of the current value of the stiffness of the clay, expressed as a fraction of the initial stiffness. Therefore, in the case of a cyclic strain controlled simple shear test (γ_C is constant), δ can be expressed in terms of cyclic shear stresses as:

$$\delta_\tau = \frac{\tau(N)}{\tau(N=1)} \tag{1}$$

Based on experimental observation, Idriss et al. (1978) proposed the following expression to describe the trend of δ_τ with the number of cycles N_c

$$\delta_\tau = N_c^{-t} \tag{2}$$

t being a degradation parameter which controls the rate of degradation with the progression of cycles.
According to Equation 2, δ_τ vs. N_c plots along a straight line in a log–log scale for a given strain amplitude. The gradient t generally increases along with the cyclic strain amplitude and decreases when the plastic index (PI) increases. These features have been confirmed in our tests, as shown in Figure 10a. The value of t was found to be equal to 0.15 (R^2=0.96) for the tested material.
The cyclic strength curve versus number of uniform loading cycles for tested material is reported in Figure 10b. The plot shows that the slope of the CRR versus N relation is quite flat, indicating that the cyclic behaviour of the material is highly dependent on the cycles of loading. The observed results appear consistent with the established differences between sands and materials with fines for which the slope of the CRR versus N relationship tends to be flatter as plasticity increases (Boulanger and Idriss, 2004).



(a) (b)

Figure 10. (a) Degradation of cyclic shear stress with the progression of cycles; (b) Cyclic resistance ratio in simple shear tests on undisturbed samples of tested material.

The in-situ CRR of sandy-silt mixtures is evaluated in practice using semi-empirical correlations, such as those presented in Figure 11.With the aim of verifying the applicability of the established empirical correlations between cyclic strength characteristics (CRR) and in-situ test (SPT) results, the data point

relative to the present study was superimposed in Figure 11. Due to the deposit heterogeneity, a representative average $(N_1)_{60}$ value equal to 15, was properly assumed for the investigated layer in the depth range of about 2.00 to 4.00 m. Furthermore, in accordance with Idriss and Boulanger (2004), a limiting value of the overburden correction (or normalization) factor, C_n, was assumed at such depths.
Figure 11 also reports data points from case histories of silty sands, sandy silts and silts revised by Idriss and Boulanger (2004) together with the NCEEER Workshop (1997) curve for soils with a fine content $FC \geq 35\%$ and the new curve derived more recently by Idriss and Boulanger (2004). As can be seen, the results gathered in the present research in terms of undrained cyclic resistance vs. normalized SPT penetration resistance, fall well in the range of other field performance data and very close to the boundary curve for soils with a fine content greater than 35% recommended by NCEER (1997).

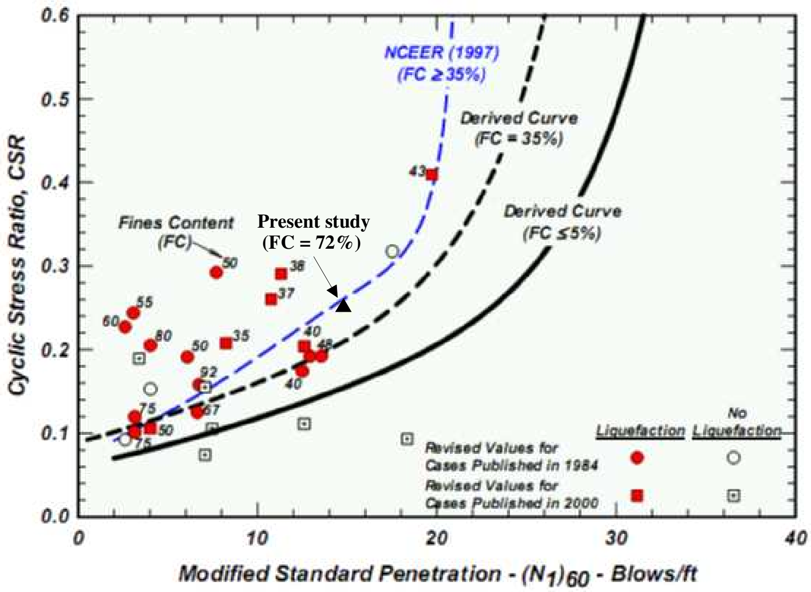


Figure 11. Comparison between data obtained from the present study with recommended SPT field performance correlations for estimating cyclic resistance ratio (adapted from Idriss and Boulanger, 2004).

CONCLUSIONS

The paper provides a contribution for the understanding of the undrained cyclic behaviour of a sandy silt of volcanic origin (ash) of low plasticity, covering the unstable slopes in a seismic area along the calabrian coast of the Messina Strait, subjected in the past to numerous rainfall-induced slope failures.
Due to some unusual properties of tested material, such as very high porosity and high liquid limit ($LL \cong 65\%$) compared with a low plasticity index ($PI \cong 11\%$), the prediction of its undrained cyclic response is not straightforward (“sand-type” or “clay-type”) (Boulanger and Idriss, 2004).
Summarising the results of the experimental investigation it was disclosed that:

- In-situ state of the examined material is above CSL in void ratio-effective stress plane; hence, during monotonic shearing, contractive response is observed in drained triaxial tests and, correspondingly, a generation of high pore water pressure in undrained triaxial tests.

- The critical/steady state friction angle of volcanic tested material is equal to 35°, which is in the range typically observed for volcanic ash materials.
- The pattern of behaviour under undrained cyclic straining of tested material is more consistent with that expected for "clay-like" soil than for "sand-like" soil (Boulanger and Idriss, 2004) and thus the material is susceptible to triggering of cyclic failure conditions.
- Undrained cyclic loading in SS tests (γ_c=±3.75%) have been observed to produce high excess pore water pressures that, however, are less than 100% (R_u=90%).
- A coincidence between the maximum cyclic shear strength (at N_c=1) required to cause a shear strain of 3.75% and the monotonic undrained shear strength in SS tests does exist.
- The rate of decrease of cyclic shear stress under cyclic straining was described by the expression (2), proposed by Idriss et al. (1978) for clayey soils. A linear trend of $\delta_\tau = \frac{\tau(N)}{\tau(N=1)}$ vs. N_c plot in a log–log scale was observed for the given 3.75% cyclic shear strain amplitude, with the gradient of the line being equal to 0.15.
- In the plot CRR vs. normalized standard penetration resistance $(N_1)_{60}$, the data point of tested material falls very close to the proposed cyclic resistance boundary curve for soils with $FC \geq 35\%$ recommended by NCEER (1997).

REFERENCES

Amini, F. and Qi. G.Z. (2000). Liquefaction testing of stratified silty sands. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 126, No. 3, pp. 208-217.

Bjeruum, L. and Ladva, A. (1996). Direct Simple Shear tests on a Norwegian quick clay. Geotechnique, Vol. 16, No. 1, pp. 1-20.

ASTM D 2487, (1993). Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System).

Boulanger, R.W. and Idriss, I.M. (2004). Evaluating the potential for liquefaction or cyclic failure of silts and clays. Report. UCD/CGM – 04/01. Center of Geotechical Modeling Department of Civil and Enviromental Engineering, University of California Davis, California.

Chien, L.K., Oh, Y.N. and Chang, C.H. (2002). Effects of fines content on liquefaction strength and dynamic settlement of reclaimed soil. Canadian Geotechnical Journal, Vol. 39 , No. 1, pp. 254-265.

Dyvik, R., Berre, T., Lacasse, S. and Raadim, B. (1987). Comparison of truly undrained and constant volume direct simple shear tests. Geotechnique, Vol. 37, No. 1, pp. 3-10.

Hungr, O., Evans, S.G., Bovis, M., Hutchinson, J.N. (2001). Review of the classification of landslides of the flow type. Enviromental and Engineering Geoscience, Vol. 7, No. 3, pp. 221-238.

Hyde, A.F.L., Higuchi, T. and Yasuhara, K. "Cyclic failure of low plasticity silt". Proc. International Conference on Cyclic Behaviour of Soils and Liquefaction Phenomena, Bochum, 137-146, 2004.

Idriss, I.M. and Boulanger, R.W. "Semi-empirical procedures for evaluating liquefaction potential during earthquakes". Proc. 11th Int. Conference on Soil Dynamics and Earthquake Engineering, and 3rd Int. Conference on Earthquake Geothecnical Engineering, Berkeley, California, USA, 1, 32-56, 2004.

Idriss, I.M., Dobry, R., Singh, R.D. (1978). Nonlinear behaviour of soft clays during cyclic loading. Journal of Geotechnical Engineering, ASCE, Vol. 104, No. GT12, pp. 1427-1447.

Ishihara, K. (1993). Liquefaction and flow failure during earthquakes. Geotechnique, Vol. 43, No. 3, pp. 351-415.

Miura, S., Yagi, K., Asonuma, T. (2003). Deformation-strength evaluation of crushable volcanic soils by laboratory and in-situ testing. Soils and Foundations, Vol. 43, No. 4, pp. 47-57.

NCEER, 1997. Proc. the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Technical Report NCEER-97-022, 1997.

Nabeshima, N. and Matsui, T. "Role of plastic and non plastic fines on cyclic shear behaviour of saturated sands". Proc. the 13th International Offshore and Polar Engineering Conference, Honululu, Hawaii, USA, 440- 444, 2003.

Picarelli, L. Evangelista, A., Rolandi, G. Paone, A. Nicotera, M.V., Olivares, L., Scotto di Santolo, A., Lampitello, S., Rolandi, M. "Mechanical properties of pyroclastic soils in Campania Region". Proc. Int. Symp. Characterisation and Engineering Properties of Natural Soils, Singapore, 4, 2331-2384, 2006.

Picarelli, L., Olivares, L., Comegna, L., Damiano, E. (2008a). Mechanical aspects of flow-like movements in granular and fine grained soils. Rock Mechanics and Rock Engineering, Vol. 41, No. 1, pp. 179-197.

Picarelli, L., Olivares, L., Avolio, A. (2008b). Zoning for flowslide and debris flow in pyroclastic soils of Campania Region based on "infinite slope" analysis. Engineering Geology Vol. 102, pp. 132-141.

Porcino, D., Caridi, G., Malara, M., Morabito, E. "An automated control system for undrained monotonic and cyclic simple shear tests". Proc. Geotechnical Engineering in the Information Technology Age GeoCongress06, 26 February – 1 March, Atlanta, GA, 2006.

Sahaphol, T. and Miura, S. (2005). Shear moduli of volcanic soils. Soil Dynamics and Earthquake Engineering, Vol. 25, pp. 157-165.

Shimizu, M. "Geotechnical features of volcanic-ash soils in Japan". Proc. the International Symposium on Problematic Soils, IS-Tohoku '98, Sendai, 28-30 October, Japan, 907-927, 1998.

Thevanayagam, S. and Martin, G.R. (2002). Liquefaction in silty soils-screening and remediation issues. Soils Dynamics and Earthquake Engineering, Vol. 22, pp. 1035–1042.

Yamamuro, J.A. and Lade, P.V. (1999). Experiments and modelling of silty sands susceptible to static liquefaction. Mechanics of cohesive frictional materials, Vol. 4, No. 6, pp. 515-564.