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Liquefaction resistance of partially saturated soils from CPTs

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ABSTRACT: In this paper an experimental methodology to evaluate the undrained cyclic resistance of saturated and partially saturated sand from cone penetration tests is presented. The proposed methodology is based on the results of (1) undrained cyclic triaxial tests carried out on fully saturated and partially saturated sand samples and (2) centrifuge cone penetration tests. The tested soil is a natural sand retrieved from a site in Italy where liquefaction occurred in 2012. The results of centrifuge CPTs and undrained cyclic triaxial tests are interpreted in the frame of critical state soil mechanics and through a state parameter to calibrate a direct correlation between the cone resistance, q_c and the undrained cyclic resistance ratio, CRR, accounting for the effect of the degree of saturation on the sand cyclic resistance. Velocity of P-waves is assumed as relevant variable to check the saturation degree.

1 INTRODUCTION

Liquefaction occurs in saturated non plastic soils when seismic loading causes a pore-water pressure build up and a drop of the effective stresses. As a consequence, the soils lose their strength and stiffness and behave like a fluid. Even if the phenomenon has usually a short duration, its consequences on structures can be devastating.

The knowledge of the undrained cyclic resistance is therefore required to forecast the soil behaviour during an earthquake. Due to the difficulties (and costs) in obtaining good quality undisturbed samples of most liquefiable soils, the undrained cyclic resistance is usually deduced from field test results such as CPTs interpreted via semi-empirical correlations.

Several studies have demonstrated that the cyclic resistance of a soil significantly increases if its degree of saturation decreases even of few percentage points (Ishihara et al. 2001, Tsukamoto et al. 2002, Okamura 2006, Eseller-Bayat et al. 2013). However, CPT based methods of liquefaction assessment do not allow to account for the possible partial saturation of a soil deposit.

In this paper the results of a research to correlate the soil cyclic resistance of a saturated and partially saturated sand to the cone resistance is presented.

The tested sand was retrieved from a site in Italy (San Carlo, in the Ferrara Province) where liquefaction occurred during the 2012 Emilia seismic sequence.

The mechanical behavior of the San Carlo sand was studied through a series of monotonic and cyclic triaxial tests. The undrained cyclic tests were carried out on both saturated and partially saturated, reconstituted samples. Centrifuge cone penetration tests were also performed on reconstituted models of the same material. The results of centrifuge and undrained cyclic triaxial tests were interpreted through the state parameter ψ to calibrate a direct correlation between the cone resistance, q_c and the undrained cyclic resistance ratio, CRR. The effect of the partial saturation on CRR was accounted for. According to previous researches (Kokusho

2000, Tsukamoto et al. 2002) compression wave velocity is assumed as relevant variable to evaluate the degree of saturation and the relating cyclic strength.

2 TRIAXIAL TEST RESULTS

The testing soil is a natural sand retrieved at the site of San Carlo(SCS), in Italy, where extensive liquefaction occurred during the 2012 Emilia seismic sequence (Fioravante et al. 2013, Giretti and Fioravante 2017). SCS has 12.5% of non-plastic fines and a mean grain size D_{50} = 0.21 mm. Its specific gravity and minimum and maximum dry density are: G_s = 2.672, γ_{min} = 13.37 kN/m^3, γ_{max} = 16.95 kN/m^3, respectively. It is composed by 58.7% quartz, 17.3% feldspar, 17.4% calcite, 3.6% muscovite, 2.2% chlorite and 0.2% kaolinite, as deduced from x-ray diffraction. The mechanical behaviour of SCS was investigated through a series of monotonic and cyclic triaxial tests. The results of the tests carried out on saturated samples are described in Giretti and Fioravante 2017 and herein only briefly recalled. In this section, the behaviour of partially saturated SCS subjected to cyclic loading is mainly described.

The monotonic tests were performed both on undisturbed (Osterberg samples of acceptable quality for monotonic standard triaxial tests) and reconstituted saturated samples and consisted of Tx compression on isotropically consolidated specimens. The specimens where consolidated using a mean effective stress p'$_c$ ranging from 50 to 600 kPa. Failure was reached applying standard undrained compression stress paths. The strength parameter at critical state in compression loading resulted M_c = q/p' = 1.4 (q = stress deviator), which corresponds to an angle of shearing resistance at critical state φ'$_{cs}$ = 34.7°. The critical state conditions in the void ratio e-p' plane were fitted by a power function (Li and Wang 1998): e_{cs} = 0.99 - 0.12-(p'/ $p_a)^{0.59}$ (e_{cs} = void ratio at critical state).

The undrained cyclic tests (CTX) on saturated SCS were all carried out on reconstituted specimens (to avoid scattered results due to possible soil disturbance of undisturbed samples). The reconstitution was carried out by pluvial deposition in air of the dry sand at the target dry density. Saturation was achieved with CO_2 circulation, flushing of deaerated water and adequate back pressure. The value of the Skempton parameter, B at the end of the saturation was always higher than 0.98 and the complete saturation was confirmed by compression wave velocities measured after consolidation, as detailed in the following part of the paper. After saturation, the samples were isotropically consolidated at a mean effective stress p'$_c$ = 100 kPa. Only two samples were compressed at 150 kPa. At the end of consolidation, the saturated specimens were characterised by high, medium or low average void ratio, i.e. e_{avg} = 0.78, e_{avg} = 0.73, e_{avg} = 0.64, respectively. The state parameters (Been and Jefferies 1985) of the tested samples resulted ψ_{avg} = - 0.073 in the loose specimens, ψ_{avg} = - 0.134 in the medium dense samples, ψ_{avg} = - 0.226 in the dense samples. The samples were subjected to cyclic undrained loading. All the failure conditions (assumed as the states at which the double amplitude axial strain ε_a^{DA} 5%) are represented in Figure 1, in terms of applied cyclic stress ratio, CSR (CSR = q/2p'$_c$) and number of cycles, N. Different symbols are used to identify the three groups of void ratio (state parameter). In the Figure, the cyclic stress ratio from triaxial condition (CSR^{TX}) is corrected into cyclic stress ratio for simple shear conditions (CSR^{SS}) according to Ishihara et al. (1977) and (1985). The data in Figure 1 relating to a given ψ_{avg} were interpolated with a power function of N, which accounts for the dependence of the cyclic resistance on ψ as follows:

$$\mathrm{CSR}^{\mathrm{SS}} = \frac{\mathrm{a}(1-\psi)^{\mathrm{b}}}{N^{c(1-\psi)}} \tag{1}$$

where a = 0.115, b = 3, c = 0.145 = empirical constants determined by fitting experimental data. In Figure 1 are reported three curves computed using Equation 1 for the three groups of ψ_{avg}.

2.1 *Undrained CTX on partially saturated specimens*

The cyclic tests on partially saturated samples were all carried out on reconstituted specimens. The reconstitution was carried out by pluvial deposition in air of the dry sand at the target

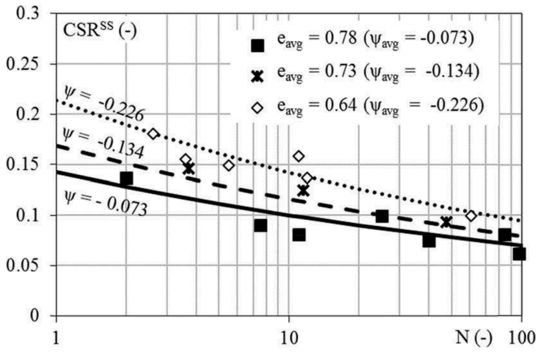

Figure 1. Cyclic strength of San Carlo Sand in simple shear condition.

dry density. After CO_2 circulation, the amount of deaerated water introduced in the samples by flushing was carefully measured and the water circulation was interrupted once the target degree of saturation, Sr = 80% or Sr = 90%, was reached. During this process, the Skempton parameter, B and the compression wave velocity, V_P were measured. Figures 2a and b show the measured V_P plotted versus the degree of saturation Sr and the B parameter. In Figure 2c, the B parameter is represented as a function of Sr. As shown in the Figures, V_P is in the range of 750-800 m/s when Sr = 80% (B = 0.1) and in the range 900-1200 m/s when Sr = 90% (B = 0.3-0.7). In fully saturate SCS samples (Sr > 97% and B > 0.98) V_P -1800 m/s. The data in Figure 2a can be interpolated, for V_P > 750 m/s, with the following logarithmic function:

$$Sr = 0.17\ln(V_P) - 0.29 \tag{2}$$

Even if the empirical constants of Equation 2 were derived from laboratory measures, Equation 2 can be used in situ to roughly estimate the degree of saturation from P-wave measures from field tests such as cross-hole and down-hole tests, seismic piezocone.

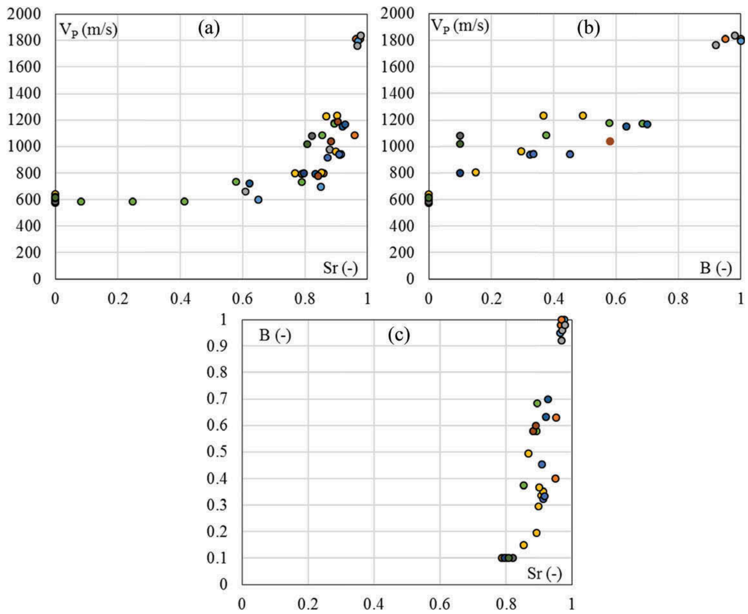

Figure 2. (a) Compression wave velocity V_P vs. the degree of saturation Sr, (b) V_P vs. Skempton parameter B; (c) B vs. Sr.

After partial saturation, the samples were subjected to an isotropic cell pressure of 100 kPa; the effect of partial saturation on mean effective stress was assumed negligible, i.e. p'_c = 100 kPa.

At the end of consolidation, the partially saturated specimens were characterised by an average void ratio e_{avg} = 0.7 and average state parameter ψ_{avg} = −0.16 (medium dense samples). In analogy to the samples tested saturated, the unsaturated specimens were subjected to cyclic undrained loading. As expected the cyclic stress ratio requested to induce 5% DA axial strain increased with decreasing degree of saturation.

Figure 3 shows the results of two undrained CTX tests carried out applying the same cyclic stress ratio (CSR^{TX} = 0.19) on a saturated (right side) and an unsaturated (Sr = 90%, left side) SCS samples. The applied stress deviator, q, the measured axial strain,ε_a and the measured excess pore pressure, u are plotted as a function of number of cycles, N in the Figures 3a, b and c, respectively.The results in Figure 3 highlight the effect of the partial saturation on the cyclic behaviour of SCS: the saturated sample reach failure after 4 cycles and large axial strains develop quite quickly. In the unsaturated sample the development of axial strains and excess pore pressure is slower and the liquefaction condition is achieved after 8 cycles. This is also shown in Figure 4, where the development of the pore pressure ratio $R_u = \Delta u/p'c$ is represented (Δu = excess pore pressure developed during the undrained cyclic loading). As to the samples with degree of saturation Sr = 80%, CSRTX larger than 0.35 were requested to reach liquefaction in less than 10 cycles.

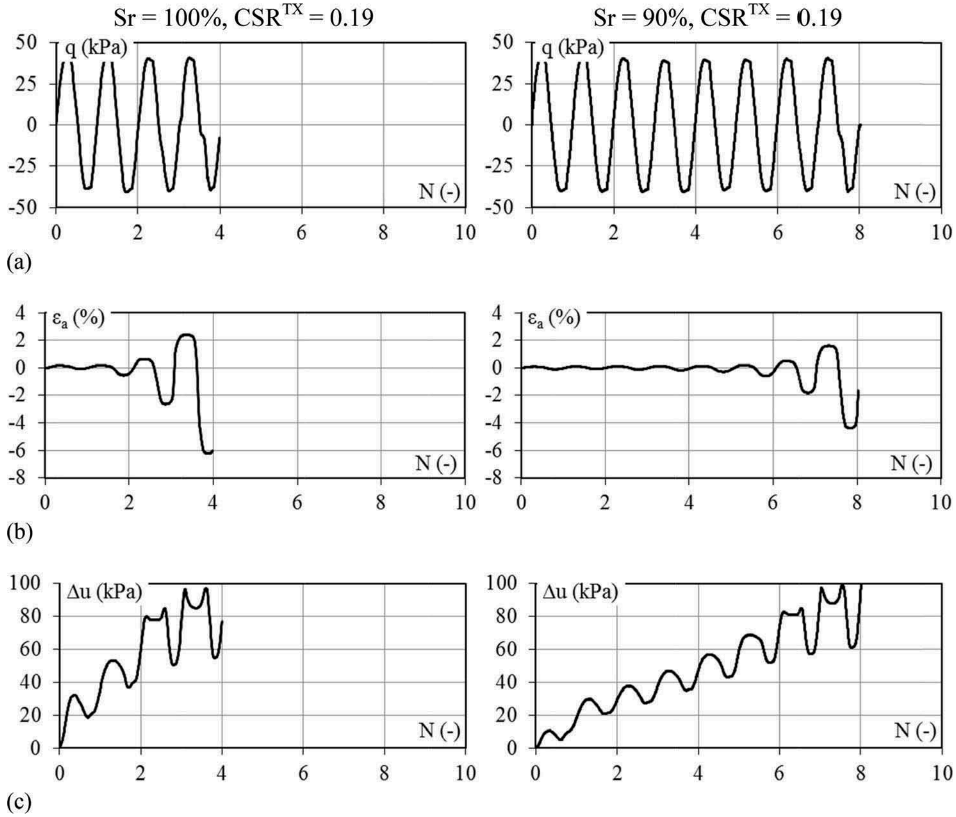

Figure 3. Undrained CTX tests on saturated (left side) and unsaturated (Sr = 90%, right side) samples: (a) stress deviator q, (b) axial strain εa and (c) excess pore pressure □u as a function of number of cycles N.

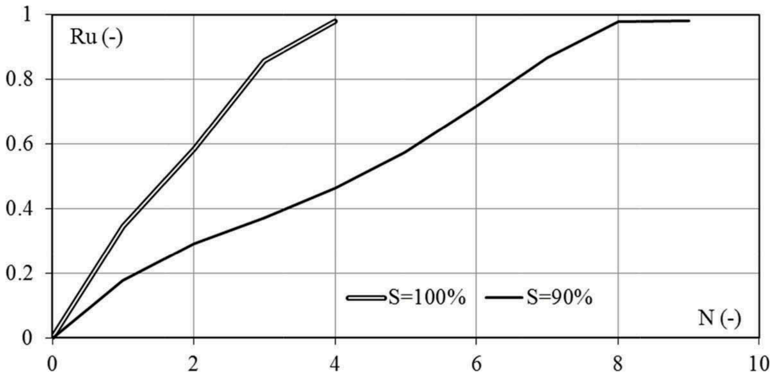

Figure 4. Pore pressure ratio measured during two undrained CTX tests carried out applying the same cyclic stress ratio (CSR_{TX} = 0.19) on a saturated and a partially saturated sample.

The increase in the liquefaction resistance, CRR, resulted almost independent on the number of cycles. The cyclic resistance of unsaturated SCS at Sr = 90% (V_P ≈900-1200 m/s) and 80% (V_P ≈ 750-800 m/s) is, in average, 1.2 and 2.2 times higher than the resistance of fully saturated SCS, as shown in Figures 5a, b and c where the CRR of unsaturated samples inducing ε_a^{DA} 5% is normalised against the relating value at full saturation and plotted versus the B parameter, the degree of saturation, Sr, and the compression wave velocity, V_P.

The experimental data in Figure 5b can be interpolated with an exponential function:

$$\mathrm{R} = \frac{\text{CRR at partial saturation}}{\text{CRR at full saturation}} = 60e^{-4.1\mathrm{Sr}} \tag{3}$$

which, combined with Equation 2, becomes:

$$\mathrm{R} = 197\mathrm{V_P}^{-0.7} \tag{4}$$

If the compression wave velocity is normalised to the maximum value at full saturation, V_{Psat}, Equation 4 can be re-written in non-dimensional form as follows:

$$\mathrm{R} = \left(\mathrm{V_P}/\mathrm{V_{P,sat}}\right)^{-0.7} \tag{5}$$

Equation 5 can be used as multiplier of the SCS liquefaction resistance to account for the effect of partial saturation. Equation 1 becomes then:

$$\mathrm{CRR^{SS}} = \left(\frac{\mathrm{V_P}}{\mathrm{V_{P,sat}}}\right)^{-0.7} \frac{\mathrm{a}(1-\psi)^{\mathrm{b}}}{\mathrm{N}^{\mathrm{c}(1-\psi)}} \tag{6}$$

Equation 6 allows to estimate the SCS cyclic resistance as a function of the state parameter y, and accounting for the degree of saturation through the P-wave velocity.

3 CENTRIFUGE TEST RESULTS

Four centrifuge CPTs on dry SCS models were carried out at a centrifugal acceleration of 100g using a miniaturised piezocone and the Istituto Sperimentale Modelli Geotecnici (ISMGEO) seismic geotechnical centrifuge (ISGC), which is a beam centrifuge 6 m in diameter, whose main characteristics are described by Baldi et al. (1988) and Airoldi et al. (2016).

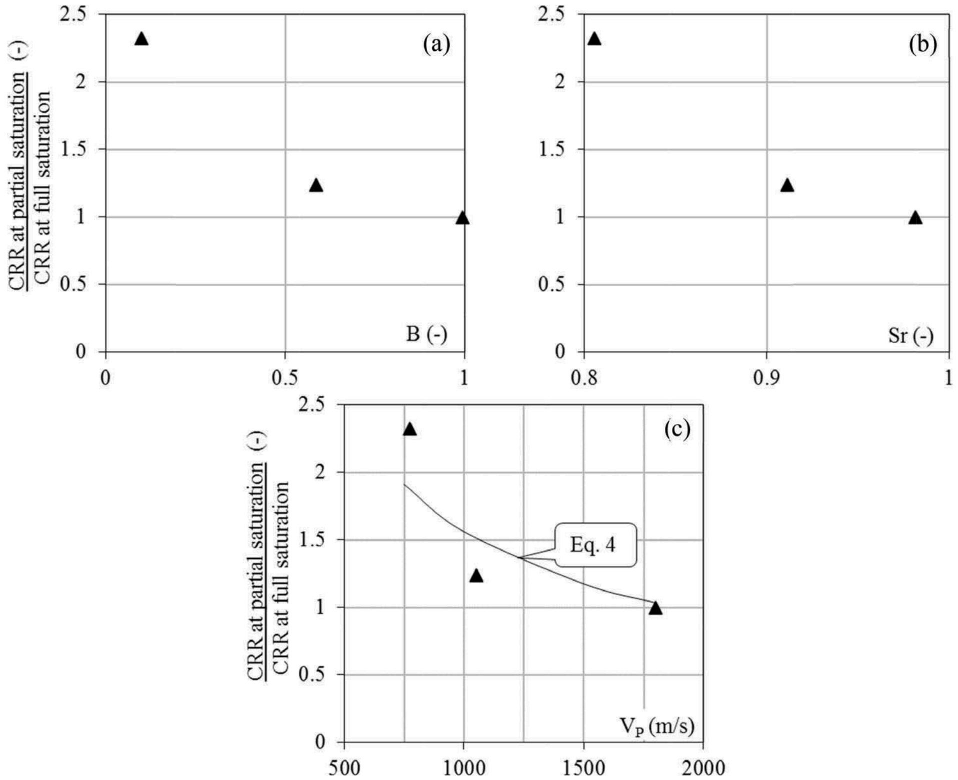

Figure 5. Cyclic resistance CRR of unsaturated samples normalized to the cyclic resistance of fully saturated samples plotted vs. (a) the Skempton B parameter, (b) the degree of saturation Sr,and (c) the compression wave velocity V_P.

The test results are described in Giretti and Fioravante, 2017 and herein only briefly recalled. The models were tested dry since, as shown by Bellotti et al. (1988) and Schmertmann (1976) on the base of calibration chamber tests on dry and saturated quartz sands, there is little influence of the saturation on the penetration resistance of silica sands.

The soil models were reconstituted at 1g by pluvial deposition in air of the dry sand. The pluviation was carried out within a rigid steel cylindrical container by means of a travelling sand spreader. The height of fall of the sand and the dimension of the spreader hole where calibrated in order to obtain the desired soil dry density. The models were instrumented with two linear displacement transducers (LDT) to measure the sand surface settlement.

When loaded in the centrifuge and accelerated to 100g, the model soil surface settled due to the self-weight of the sand, as monitored by LDTs. The total surface settlement of the soil model allowed to estimate the average compression index of the soil and to derive the profiles with depth of the void ratio (decreasing with depth) and the state parameter (increasing with depth). The in-flight average void ratio of the models ranged from 0.774 (low density model) to 0.615 (high density model).

The cone penetration test was carried out applying a penetration rate of 2 mm/s. Only one test per model was performed, in the central axis of each sample. The penetration was interrupted before reaching $20d_c$ of distance from the container bottom to avoid rigid boundary effects (Bolton et al., 1999).

The interpretation procedure adopted is described in detail by Fioravante and Giretti 2016 and Giretti and Fioravante 2017. The principal outcomes are:

- linear dependence of q_c on p’

$$q_c/p = \text{constant} = q_c* \quad (7)$$

exponential dependence of q_c* on ψ

$$q_c* = ke^{--m\psi} \quad (8)$$

where m = 7.45 and k = 27.1 are dimensionless fitting parameter.

4 LIQUEFACTION RESISTANCE OF SCS FROM CPT

As described in the previous sections, the results of both centrifuge and undrained cyclic triaxial tests carried out on SCS samples were interpreted in the frame of the critical state soil mechanics using the state parameter as independent variable governing both the cyclic stress resistance (Eq. 1) and the normalized cone resistance (Eq. 8).

Equations 1 and 8 can be combined into Equation 9 to obtain a direct correlation between q_c* and the cyclic resistance ratio at N cycles for simple shear condition, CRR^{SS} of saturated SCS:

$$CRR^{SS} = \frac{a\left[1 + \frac{1}{m}\ln\left(\frac{q_c^*}{k}\right)\right]^b}{N^{c\left[1+\frac{1}{m}\ln\left(\frac{q_c^*}{k}\right)\right]}} \quad (9)$$

where a, b, c, m and k are the fitting parameters of Eqs. 1 and 8.

For partially saturated conditions, P-wave velocity can be assumed as indicator of the soil degree of saturation (Eqs. 2 and 6) and Equation 9 can be rewritten as:

$$CRR^{SS} = \left(\frac{V_P}{V_{P,sat}}\right)^{-0.7} \frac{a\left[1 + \frac{1}{m}\ln\left(\frac{q_c^*}{k}\right)\right]^b}{N^{c\left[1+\frac{1}{m}\ln\left(\frac{q_c^*}{k}\right)\right]}} \quad (10)$$

Equation 10 allows to evaluate CRR directly from q_c for saturated and partially saturated SCS.

Equation 10 can be applied to evaluate the resistance to liquefaction of sandy deposits similar to that present at the reference site of San Carlo (unaged and uncemented alluvial sandy deposit, with limited non-plastic fine content and similar mineralogy). Its applicability requires the knowledge of the cone resistance and P-wave velocity, the latter easily achievable in situ by means of cross-hole or down-hole tests.

5 FINAL REMARKS

This paper presents a method for liquefaction assessment based on the results of an experimental research carried out using a natural silica sand retrieved from the site of San Carlo in the Ferrara Province (Italy), where extensive liquefaction occurred during the 2012.05.20 earthquake (moment magnitude M_w=6.1). San Carlo is located about 15 km SE of the epicentre; a peak ground acceleration PGA = 0.16g was estimated for this locality. The sandy layer which experienced liquefaction was formed by the fluvial activity of an Apennine river in the years between 1450 and 1770 and the sand is normally consolidated and uncemented.

The San Carlo Sand (SCS) was subjected to monotonic and cyclic triaxial tests and to centrifuge cone penetration tests with the main aim of calibrating a specific correlation between the cone resistance q_c and the cyclic resistance CRR. In the first instance the correlation was defined for saturated SCS.

The observation that in areas closer to the 2012.05.20 earthquake epicentre, where much higher PGAs were measured (or estimated), sandy deposits of similar origin and age to that present at San Carlo did not liquefied, suggested the possibility of partial saturation of those soils. This hypothesis is supported by frequent reporting of gas leaks from the soil and presence of gas in the ground water just in the area few kilometres around the earthquake epicentre.

For this reason, the cyclic triaxial tests on SCS were repeated also on unsaturated SCS in order to establish the increase in its liquefaction resistance due to the partial saturation. The cyclic resistance of unsaturated SCS at Sr = 90% and 80% resulted, in average, 1.2 and 2.2 times higher than the resistance of fully saturated SCS.

The effect of partial saturation was accounted for in the q_c-CRR correlation calibrated for SCS. The degree of saturation is a variable not measurable in situ, so a method to link CRR of partially saturated SCS to the P-waves velocity is proposed in this paper.

The method can be used to evaluate the resistance to liquefaction of sandy deposits similar to that present at the reference site of San Carlo (unaged and uncemented alluvial sandy deposit, with limited non-plastic fine content and similar mineralogy).

The applicability to different sands requires the execution of a limited number of centrifuge and laboratory tests.

REFERENCES

Airoldi, S., Fioravante, F. & Giretti, D. 2016. The ISMGEO seismic geotechnical Centrifuge. Proceedings of the 3rd European Conference on Physical Modelling in Geotechnics (EUROFUGE 2016).

Baldi, G., Belloni, G., Maggioni, W. 1988. The ISMES Geotechnical Centrifuge. In Centrifuge 88, Paris, Corté J. F. Ed., Balkema, Rotterdam, pp 45–48.

Been, K. & Jefferies, M.G. 1985. A state parameter for sands. Géotechnique, 35(2),99–112.

Bellotti, R., Pedroni, S. & Crippa, V., Ghionna, V.N. 1988. Saturation of sand specimen for calibration chamber tests. Proc. ISOPT-1, Orlando, Vol. 2, 661–672.

Bolton, M.D., Gui, M.W., Garnier, J., Corte, J.F., Bagge, G., Laue, J. & Renzi, R. 1999. Centrifuge Cone Penetration Tests in Sand. Geotechnique, 49(4),543–552.

Eseller-Bayat, E., Yegian, M. K., Alshawabkeh, A. & Gokyer, S. 2013. Liquefaction response of partially saturated sands. I: Experimental results. J. Geotech. Geoenviron. Eng., 139(6),863–871.

Fioravante, V., Giretti, D., Abate, G., Aversa, S., Boldini, D., Capilleri, P.P., Cavallaro, A., Chamlagain, D., Crespellani, T., Dezi, F., Facciorusso, J., Ghinelli, A., Grasso, S., Lanzo, G., Madiai, C., Massimino, M.R., Maugeri, M., Pagliaroli, A., Rainieri, C., Tropeano, G., Santucci De Magistris, F., Sica, S., Silvestri, F., Vannucchi, G. 2013. Earthquake geotechnical engineering aspects of the 2012 Emilia-Romagna earthquake (Italy). In: 7th International conference on case histories in geotechnical engineering, April 29–May 4, 2013, Chicago

Fioravante, V.& Giretti, D. 2016. Unidirectional cyclic resistance of Ticino and Toyoura sands from centrifuge cone penetration tests. Acta Geotechnica, 11(4),953–968.

Giretti, D. & Fioravante, F. 2017. A correlation to evaluate cyclic resistance from CPT applied to a case history. Bulletin of Earthquake Engineering, 15(5),1965–1989.

Ishihara, K., Iwamoto, S., Yasuda, S. & Takatsu, H. 1977. Liquefaction of anisotropically consolidated sand. Proceedings 9th International Conference on Soil Mechanics and Foundation Engineering, Japanese Society of Soil Mechanics and Foundation Engineering, Tokyo, Japan, Vol. 2, pp 261–264.

Ishihara, K., Yamazaki, A., & Haga, K. 1985. Liquefaction of K0 consolidated sand under cyclic rotation of principal stress direction with lateral constraint, Soils and Foundations, 5(4): 63–74.

Ishihara, K., Tsuchiya, H., Huang, Y. & Kamada, K. 2001. ‘Recent studies on liquefaction resistance of sand: Effect of saturation.’ Proc., 4th Int. Conf. on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego.

Jefferies, M. & Been, K. 2006. Soil liquefaction. A critical state approach. Taylor and Francis, London.

Li, X.S. & Wang, Z.L. 1998. Linear representation of steady state line for sand. J. Geotech. Geoenviron. Eng., ASCE, 124(12),1215–1217.

Okamura, M. & Soga, Y. 2006. Effects of pore fluid compressibility on liquefaction resistance of partially saturated sand. Soils and Foundations, 46(5),695–700.

Schmertmann, J.H. 1976. An updated correlation between Relative Density D_R and Fugro-type electric cone bearing, q_c. Contract Report DACW 39-76 M 6646 WES, Vicksburg, Miss. 1976

Tsukamoto, Y., Ishihara, K., Nakazawa, H., Kamada, K. &Huang, Y.N. 2002. Resistance of partly saturated sand to liquefaction with reference to longitudinal and shear wave velocities. Soils and Foundations, 42(6),93–104.