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The Effect of Preloading on the Cyclic Liquefaction Strength Measured in the Laboratory

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Abstract: The effect of preloading on the liquefaction cyclic strength was investigated by cyclic shear tests where horizontal shear stress oscillated about a zero mean value on sands with varying fines content and at varying prestress ratios, densities and vertical stresses. Test results showed a marked increase of the cyclic soil strength with the prestress ratio. The effect is more pronounced for the looser specimens. An empirical expression predicting this effect is proposed. This expression is validated from results of a field test.

Key words: Cyclic liquefaction, shear stress, field test.

1. Introduction

Saturated sandy layers that are horizontal, or have a small inclination to the horizontal, run the risk of earthquake-induced liquefaction. Design codes (e.g. Ref. [1]) demand of the practicing engineer the estimation of the liquefaction risk. The factor of safety against liquefaction, estimated versus depth, is defined as the ratio of the in-situ cyclic soil strength by the cyclic SR (stress ratio) resulting from the design earthquake [1, 2]. The cyclic soil strength and cyclic SR are defined hereunder. In cases where the factor of safety against liquefaction takes values close to, or less than, unity, soil improvement is an effective way to mitigate the liquefaction risk by increasing the cyclic soil strength [3]. Preloading is a temporary loading applied at a construction site, to improve subsurface soils primarily by increasing density and horizontal stress [4]. The purpose of this work is to investigate the effect of preloading in the cyclic strength of different soils in the laboratory.

2. The Cyclic Strength of Sands

Prior to the application of cyclic loading, the shear stresses σ_{xz}, σ_{yz} and σ_{xy}, where x and y are horizontal mutually perpendicular directions, are zero. In addition, the effective normal vertical stress σ_{zz}, denoted as σ'_{vo}, equals the overburden effective pressure. Furthermore, the ratio of the effective horizontal stress, denoted as σ'_{ho}, to σ'_{vo}, is given by the coefficient of earth pressure at rest, Ko. As a result of an earthquake, dynamic loading is applied primary in the horizontal direction, as shear stress σ_{xz}.

When harmonic shear horizontal loading is applied, a cycle of loading is defined as the complete change of the shear stress σ_{xz} (i) from zero to τ_{cyc}, (ii) from τ_{cyc} to -τ_{cyc} and (iii) from -τ_{cyc} back to zero. Cyclic SR is defined as:

$$\mathrm{SR} = \tau_{cyc}/\sigma'_{vo} \tag{1}$$

In addition, cyclic shear strain during a loading cycle is defined as the maximum value of shear strain attained. Permanent strain, or permanent pore water pressure, is the part of the strain, or pore water pressure, that accumulates at the end of each cycle of loading.

As a result of cyclic horizontal harmonic loading on Ko-consolidated soil about a zero mean shear stress value, permanent pore pressure and cyclic shear strain build up with cycle number, while due to one-dimensional symmetry, considerable permanent horizontal

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movement does not accumulate [3]. Cyclic strength for *N* cycles of harmonic loading, SRN, is defined as the value of the cyclic SR causing liquefaction in *N* uniform cycles. Liquefaction is the state where the effective stress becomes very small and the cyclic shear strain very large. In a more functional definition this state is defined in the present work, similarly to Ishihara [5], as when the cyclic shear strain exceeds 2.5%.

In liquefaction analyses, a reference earthquake of magnitude M = 7.5, that corresponds to 15 cycles of uniform cyclic loading, is often used [1, 2]. For this reason, the cyclic strength SR_{15} will be used below as index to study the effect of preloading on the cyclic soil strength for level ground. Yet, it should be noted that it was observed that if the cyclic strength is taken as SR_{10} or SR_{20} instead of SR_{15}, the results presented below do not change considerably.

The cyclic strength is measured in the laboratory by cyclic undrained or constant-volume tests using devices which can be classified into three types: (a) Tests on the triaxial device with isotropic consolidation at stress $\sigma'_{co} = \sigma'_{vo}$ and then oscillation of the vertical stress between ($\sigma'_{co} + 2\tau_{cyc}$) and ($\sigma'_{co} - 2\tau_{cyc}$), (b) tests on the direct-shear or the simple-shear device where samples are subjected to one-dimensional consolidation at vertical stress σ'_{vo} and then to horizontal oscillation of the shear stress σ_{zx} between τ_{cyc} and $-\tau_{cyc}$ and (c) tests on the torsional-shear device where samples are subjected in the triaxial chamber to consolidation at vertical stress σ'_{vo} and any horizontal stress and then to horizontal angular oscillation of the shear stress $\sigma_{r\theta}$ between τ_{cyc} and $-\tau_{cyc}$ [6-8]. In tests (b) the ratio of the effective horizontal to the effective vertical stress prior to the application of cyclic loading is the coefficient *Ko*.

In all these devices [8] the effect of prestressing has been studied. The cyclic strength is correlated to the PR (prestress ratio), defined as:

$$PR = \sigma'_{v\text{-}c} / \sigma'_{vo} \quad (2)$$

where $\sigma'_{v\text{-}c}$ is the maximum effective vertical stress exerted during consolidation and σ'_{vo} is the effective vertical stress just prior to the application of cyclic loading.

Compared to tests under *Ko* consolidation, at tests under isotropic consolidation, prestress is performed under different horizontal stress, and thus cyclic soil strength takes a different value. Preloading affects the cyclic soil strength due to (a) the increased density under *Ko* conditions, (b) the increased horizontal stress and (c) changes in the sand fabric [8]. Cyclic direct-shear and simple-shear tests performed at different PR ratios can simulate all the above effects. Cyclic triaxial tests cannot simulate effect (b). For these reasons cyclic shear tests will be considered in the present study.

3. Effect of Preloading on the Cyclic Strength Measured in the Laboratory Measured Previously

A number of cyclic shear tests have been performed studying the effect of preloading on cyclic soil strength and based on the analyses of these tests and empirical expressions have been proposed. The most complete study is given by C. Stamatopoulos and A. Stamatopoulos [8] where the following equation predicting the increase in cyclic strength induced by preloading is proposed:

$$SR_{15}/SR_{15\text{-}1} = PR^{0.92-2.02SR_{15-1}} \quad (3)$$

where all the factors have been defined above.

4. Current Program of Laboratory Tests

In the current laboratory program, the materials used to prepare the mixtures tested were (a) a clean sand and (b) a non-plastic silt. The first material is a subwhite natural sand of quartz from the Egyptian Desert. Its quartzy grains are not lustrous but are well rounded, transparent and colorless. There are very few black grains of iron oxides or magnetic oxides of unknown provenance (less than 0.1%), whose existence could be due to manmade contamination. The non-plastic silt was obtained by grinding natural deposits of quartz from the area of Assirou near Thessaloniki, Greece. The materials were provided by Prof. Tika of AUTH (Aristotle University of Thessaloniki) [9]. Fig. 1 gives the grain size distribution of the sand and silt materials and maximum and minimum void ratio in terms of the fines content. The measured specific gravity of grains was 2.65 for the sand and 2.64 for the silt.

Sand-silt mixtures with different fines contents were prepared. A program of cyclic shear tests investigating the effect of prestress on cyclic was performed on specimens from these mixtures at different (a) initial void ratios (e_o), (b) vertical consolidation stresses (σ_v) and (c) fines contents (f).

In accordance with the previous discussion, in the current testing program the specimens were subjected to cyclic constant-volume loading oscillating about a mean zero shear stress in samples consolidated under *Ko* conditions in the shear device. Specifically, the device used was a Wykeham Farrance direct-shear apparatus in compliance with British Standard 1377. Undrained response was simulated by maintaining a constant specimen volume during shear by adjusting the vertical stress. The device and procedure used is similar to that described by C. Stamatopoulos and A. Stamatopoulos [8].

On the shear device all specimens had a height of 2 cm and a cross-sectional area of 6 × 6 cm^2. Tests were performed on specimens prepared at three densities that correspond to the loose, medium and dense states. Knowing the volume of the shear box and the desirable dry density the weight of the sand to be placed was calculated. Placement of oven-dried soil was performed in 2 to 4 layers with the use of rammers. In the case of the loose and medium dense specimens a wooden rammer 57 × 57 mm^2 was used and the number of layers was 2 to 4. In the case of the dense specimens, the rammer consisted of a steel prism 49 ×49 ×35 mm^3 and the number of layers was 4. In addition, it was necessary to slightly moisten the material to a water content of about 5% and apply onto the steel prism few light blows with a steel hammer. Then, all specimens were consolidated. Prior to shearing the samples, in the tests without preconsolidation, the vertical stress (σ'_v) increased from 0 to the vertical consolidation stress then consolidation was allowed. In the tests with preconsolidation, the vertical stress (σ'_v) increased from 0 to the maximum vertical consolidation stress then consolidation was allowed. Then, the vertical stress was decreased to the vertical stress of the test. In all cases the consolidation stress stayed enough time for compression to reach the d_{100} value, as determined by the logarithm of time method.

Table 1 gives the states of fines content, void ratio and consolidation stress studied. Fig. 2 gives typical results of a cyclic shear test. Fig. 3 gives typical liquefaction curves. Table 1 gives the cyclic soil liquefaction strength obtained in terms of PR for all states studied. Fig. 4a gives the cyclic liquefaction strength at 15 cycles of loading, SR_{15}, in terms of soil density, consolidation stress and PR ratio. It can be observed that, similarly to previous studies [8, 9], the cyclic strength for PR = 1: (a) increases as the void ratio decreases, (b) increases as the confining stress decreases and (c) increases as the PR value increases. Fig. 4b gives SR_{15} for σ = 150 kPa in terms of void ratio and fines content. It can be observed that for similar void ratio and confining stress, SR_{15} decreases as f increases until f = 0.25, but then increases. This is consistent with observations by Papadopoulou and Tika [9].

Fig. 5a plots the SR_{15} value in terms of the PR value. Only tests where more than two PR values on the samples with the same initial void ratio and consolidation stress are considered. It can be observed that SR_{15} increases with PR in a logarithmic manner. Thus, it can be assumed that the effect of PR on SR_{15} has the form:

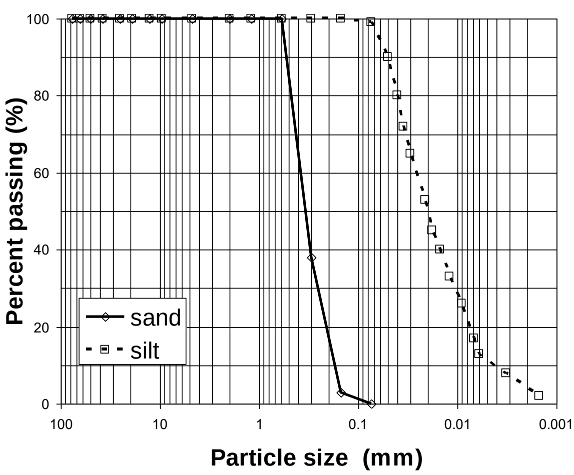
$$SR_{15} / SR_{15\text{-}1} = PR^{\Lambda} \quad (4a)$$

Fig. 5b gives the ratio of cyclic liquefaction strength at PR = 1 and 3 ($SR_{15\text{-}PR=3}$/ $SR_{15\text{-}PR=1}$) in terms of soil density, consolidation stress and fines content. Soil density is expressed in terms of $SR_{15\text{-}PR=1}$. The previous proposed Eq. (3) is given for comparison. It can be observed that the measured ratios are generally less than those predicted by the previously proposed equations, but show a similar trend. An expression that predicts the results is:

$$\Lambda = SR_{15}/SR_{15\text{-}1} = PR^{0.04/SR_{15}} \quad (4b)$$

Table 1 Current laboratory program. States studied and cyclic liquefaction strength in terms of *f*, e_o, σ and PR for all cases considered.

f	e_o	σ_v	e_f	SR_{15} (PR = 1)	SR_{15} (PR=2)	SR_{15} (PR=3)	SR_{15} (PR=4)
0	0.79	0.5	0.71	0.20		0.26	
0	0.79	1.5	0.66	0.15	0.18	0.21	0.22
0	0.67	0.5	0.60	0.20	0.26	0.27	0.30
0	0.67	1.5	0.56	0.18	0.20	0.25	0.27
0	0.58	0.5	0.52	0.24		0.33	
0	0.58	1.5	0.48	0.19		0.24	
0.15	0.67	0.5	0.59	0.15		0.21	
0.15	0.67	1.5	0.54	0.13		0.18	
0.25	0.72	0.5	0.63	0.14		0.18	
0.25	0.72	1.5	0.57	0.08		0.13	
0.25	0.67	0.5	0.58	0.16		0.21	
0.25	0.67	1.5	0.53	0.09		0.13	
0.25	0.66	1.5	0.52	0.12		0.17	
0.40	0.77	0.5	0.66	0.11	0.14	0.15	0.18
0.40	0.77	1.5	0.58	0.11	0.13	0.15	0.17
0.40	0.56	0.5	0.48	0.20		0.28	
0.40	0.56	1.5	0.43	0.15		0.19	
0.60	0.89	0.5	0.75	0.12		0.17	
0.60	0.89	1.5	0.65	0.08	0.10	0.13	0.15
0.6	0.56	1.5	0.40	0.16		0.22	
0.80	0.77	0.5	0.62	0.07		0.15	
0.80	0.77	1.5	0.53	0.10		0.14	
1	1.04	0.5	0.82	0.07	0.10	0.13	0.16
1	1.04	1.5	0.67	0.08	0.11	0.13	0.16
1	0.66	0.5	0.52	0.12		0.18	
1	0.66	1.5	0.43	0.13		0.19	



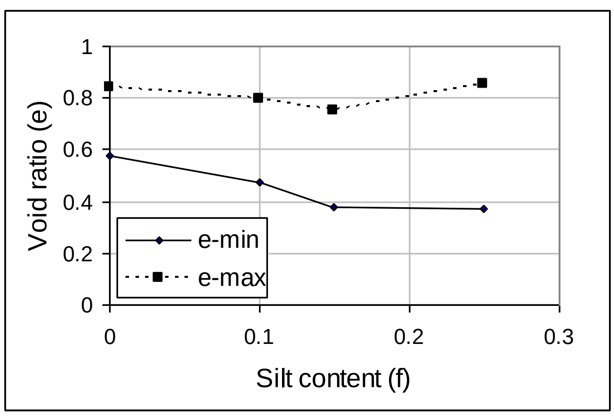


Fig. 1 Grain size distribution of the sand and silt used to form the mixtures of the present study and maximum and minimum void ratio in terms of the fines content.

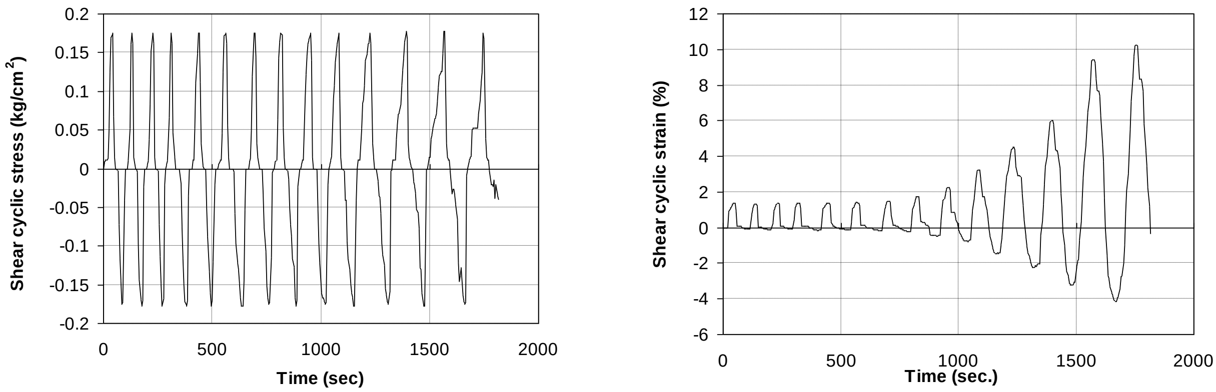


Fig. 2 Typical results of a shear test. The case of f = 0, e_o = 0.67, σ'_v = 150 kPa, PR = 3 is given.

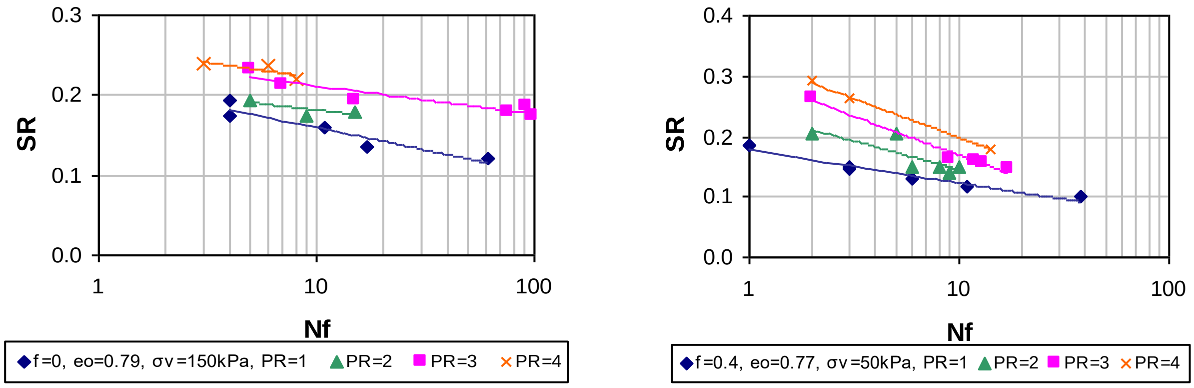
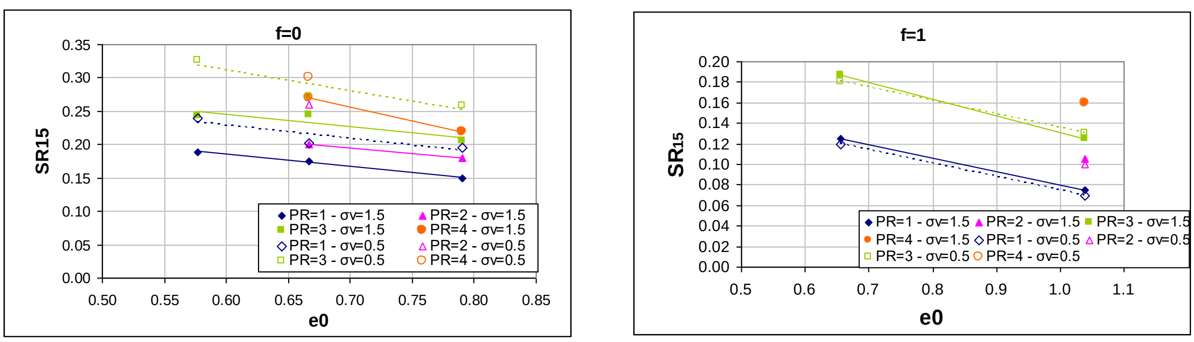
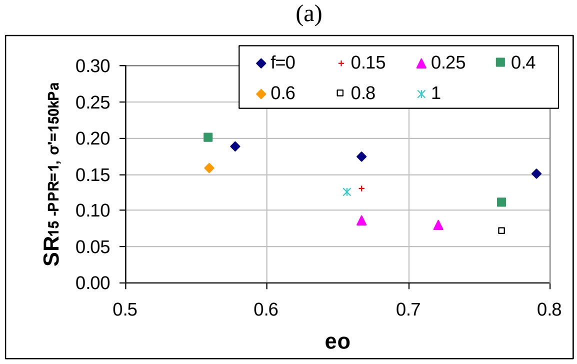


Fig. 3 Typical liquefaction curves. The cases with f = 0, e_o = 0.79, σ'_v = 150 kPa and f = 0.4, e_o = 0.77, σ'_v = 50 kPa are given.

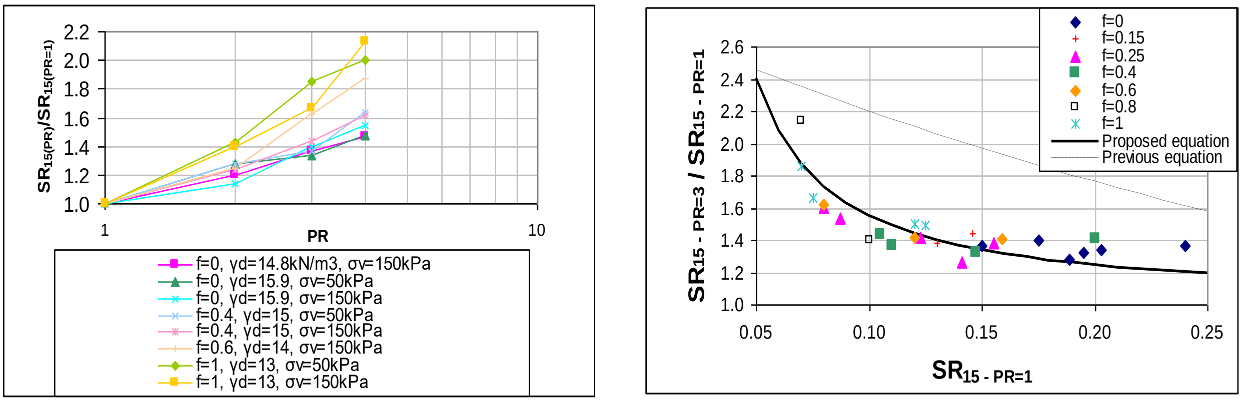


(a)



(b)

Fig. 4 (a) Constant-volume cyclic shear tests and cyclic liquefaction strength in terms of soil density, consolidation stress and PR ratio; (b) Effect of fines content on the cyclic soil strength.



(a) (b)

Fig. 5 (a) Effect of PR ratio on the liquefaction cyclic strength; (b) Ratio of cyclic liquefaction strength at PR = 1 and 3 in terms of cyclic liquefaction strength at PR = 1 and the fines content. The previous proposed Eq. (3) is given for comparison.

5. Application Procedure of Eq. (4)

A procedure can be proposed to predict the effect of preloading on increasing cyclic strength of new sites. It has the following steps: (a) estimate versus depth the field cyclic strength $SR_{15\text{-field-1}}$, (b) estimate versus depth the field prestess SR PR_{field}, and (c) estimate versus depth the field cyclic strength after the application of preloading, SR_{15}, by Eq. (4) assuming that $SR_{15\text{-}1}$ equals $SR_{15\text{-field-1}}$ and PR equals PR_{field}.

The factor $SR_{15\text{-field-1}}$ can be obtained versus depth from results of *in-situ* tests, such as the blow count measured in the SPT as described e.g. by Eurocode 8 [1]. The factor PR_{field} can be obtained versus depth using Eq. (4). If we suppose that the preload is applied for the time period necessary for the consolidation to take place, $\Delta\sigma'_v$, equals the corresponding total vertical stress $\Delta\sigma_v$. The stress $\Delta\sigma_v$ can be approximately obtained using the theory of linear elasticity (e.g. Ref. [1]).

6. Validation with Results of Field Tests

An elaborate field preloading study on a liquefaction-susceptible site was recently performed [9, 10]. The side was in Porto Romano, in the Albanian coast. Preloading was applied by a temporary embankment 9 m high. Prior to and after preloading, borings with standard penetration tests, cone penetration tests and geophysical studies were performed. Table 2 presents the soil layers that exist in the site and their properties. The water table was at a depth of 0.5 m. Table 3 gives the average measured N_{SPT}, qc and V_S before and after soil improvement in terms of soil layer. Only the two lower layers of Table 2 are susceptible to liquefaction. Table 4 gives the pre- and post-improvement cyclic liquefaction strength ($SR_{15\text{-bef}}$ and $SR_{15\text{-after}}$) based on field measurements and corresponding ratio $R(=SR_{15\text{-after}}/SR_{15\text{-bef}})$.

The factor PR_{field} was obtained versus depth as:

$$PR_{field} = (\sigma'_{vo} + \Delta\sigma'_v) / \sigma'_{vo} \qquad (5)$$

where σ'_{vo} is the overburden effective vertical stress after the application of the preload and $\Delta\sigma'_v$ is the maximum additional effective vertical stress applied during the preload estimated versus depth with linear elastic theory. Then, Eq. (4) was applied in the two layers at their average depth. The predicted increase in liquefaction cyclic strength of these two layers is in good agreement with the measured.

Table 2 Porto Romano field study. Soil layers that exist in the site and their properties.

Depth	Layer description	PI	Fines content	Cc	Cr	Initial void ratio	Total density (t/m^3)	φ' (for c = 0)
0-3.5 m	Silty clay	30%	77	0.52	0.02	1.40	1.66	28
3.5-7 m	Medium gravel with silty sand	< 5%	15	0.03	0.01	0.58	2.02	39
7-15 m	Fine sand and silt	< 5%	95	0.04	0.01	0.64	2.07	25

Table 3 Average measured N_{SPT}, qc and V_S before and after soil improvement in terms of soil layer and corresponding ratio.

Depth	Bef. (pre-improvement)			After (post-improvement)		
	N_{SPT}	qc (MPa)	V_S (m/s)	N_{SPT}	qc (MPa)	V_S (m/s)
	(a)	(b)	(c)	(d)	(e)	(f)
3.5-7 m	21.7	10.0	192	23.6	11.8	212
7-15 m	20.9	4.33	197	25.9	6.11	246

Table 4 Pre- and post-improvement cyclic liquefaction strength (SR15-bef and SR15-after) based on field measurements and corresponding ratio R(=SR15-after/SR15-bef).

	$SR_{15\text{-}bef}$			$SR_{15\text{-}after}$			R_M				R_{Pr}	R_{Pr}/R_M
Depth	N_{SPT}	qc	V_S	N_{SPT}	qc	V_S	N_{SPT}	qc	V_S	Ave	Eq. (4)	
3.5-7 m	0.50	0.42	0.39	0.55	0.48	0.46	1.10	1.14	1.19	1.14	1.09	0.96
7-15 m	0.38	-	-	0.43	-	-		-	-	1.13	1.07	0.95

7. Conclusions

A series of laboratory tests and numerical simulations are performed to investigate the effect of pre-loadings on the cyclic strength of liquefiable soils. Based on the test results, the empirical expression (Eq. (4b)) predicting the increase in cyclic strength induced by preloading was proposed. Based on Eq. (4) a procedure predicting the increase in cyclic liquefaction strength induced by preloading in the field is proposed. This procedure was validated by results of a case study where preloading was applied in the field.

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References

[1] European Standard. 2003. *Eurocode 8: Design of Structures for Earthquake Resistance*. prEN 1998-5, December.

[2] Seed, H. B. 1979. "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground during Earthquakes." *Journal of Geotechnical Engineering* 105 (2): 201-55.

[3] Committee on Earthquake Engineering, Commission on Engineering and Technical Systems, National Research Council. 1985. *Liquefaction of Soils during Earthquakes*. Washington, D.C.: National Academy Press.

[4] Stamatopoulos, A. C., and Kotzias, P. C. 1985. *Soil Improvement by Preloading*. New York: John Wiley & Sons.

[5] Ishihara, K. 1996. *Soil Behaviour in Earthquake Geotechnics*. Oxford, UK: Clarendon Press.

[6] Seed, H. B., and Peacock, W. H. 1971. "Test Procedures for Measuring Soil Liquefaction Characteristics." *Journal of the Geotechnical Engineering Division* 97 (8): 1099-119.

[7] Ishihara, K., and Takatsu, H. 1979. "Effects of Overconsolidation and K_0 Conditions on the Liquefaction Characteristics of Sands." *Soils and Foundations* 19 (4): 59-68.

[8] Stamatopoulos, C., and Stamatopoulos, A. 2007. "Effect of Preloading on the Cyclic Strength of Reconstituted Sand." *Ground Improvement* 11 (3): 145-55.

[9] Papadopoulou, A., and Tika, T. 2008. "The Effect of Fines on Critical State and Liquefaction Resistance Characteristics of Non-plastic Silty Sands." *Soils & Foundations* 48 (5): 713-25.

[10] Stamatopoulos, C., Balla, L., and Allkja, S. 2013. "Improvement of Dynamic Soil Properties Induced by Preloading Measured in the Field." In *Proceedings of the 2nd International Balkans Conference on Challenges of Civil Engineering (BCCCE)*, 23-25 May 2013, EPOKA University, Tirana, Albania.