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Experimental evaluation of shear wave velocity change induced by repeated liquefaction of Sofia sand by undrained cyclic triaxial tests

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ABSTRACT: The aim of this paper is to observe the effect of the soil skeleton variation on repeated liquefaction resistance of soils by means of experimental evaluation of the shear wave velocity change induced by re-liquefaction of natural coarse sand deposit located in Lozenetz area near the capital of Bulgaria, Sofia. The study has been conducted through cyclic undrained triaxial tests carried out on reconstituted specimens. The isotropically consolidated specimens have been tested at 90 % relative density (close to "in-situ" initial condition) and 100 kPa effective overburden stress. The paper presents two types of tests performed for each experiment: one is aimed to evaluate the liquefaction resistance several times on the same specimen (re-liquefaction) by means of CSR-N_C (CSR – cyclic stress ratio and N_C – number of loading cycles at axial strain double amplitude of 5 %) relation and the other to determine the shear wave velocity through dynamic measurement methods before and after each re-liquefaction phase.

1 INTRODUCTION

Studies (Wahyudi et al. 2015) after major earthquakes in New Zealand (Canterbury in 2010 and Christchurch in 2011) and Japan (Off the Pacific Coast of Tohoku in 2011) have found that soil liquefaction could be triggered multiple times in the same sites. This observation has gained major attention on the possibility of reoccurrence of multiple-liquefaction during future severe earthquakes. Researchers' studies prove that liquefaction not only can occur more than once in the same site but the devastating consequences from re-liquefaction might often be more severe than the ones caused by the initial liquefaction.

A well-known fact is that soil deposits prone to liquefaction are saturated "young" well-sorted sands. The resistance of the cohesionless soil to liquefaction depends on its density and structure as well as on the confining stress, duration and magnitude of the dynamic loading.

Generally, liquefaction leads to densification of the soil deposits which might mislead many to conclude that such event improves soil resistance to re-liquefaction in all cases. However studies (Villalta 2015) show that densification of soil due to liquefaction does not always act beneficially on its structure. Rearrangement of soil particles might render the soil structure more prone to liquefaction than it was before the initial liquefaction. In order to evaluate the resistance of cohesionless soils to multiple-liquefaction an experimental study has been performed. The study has been conducted through cyclic undrained triaxial tests carried out on specimens of natural coarse sand deposit from Lozenetz area (near the capital of Bulgaria – Sofia). Soil skeleton's variation has been observed by measurement of the shear wave velocity change.

Table 1. List of performed tests

Test No.	σ'_c [kPa]	D_r [%]	*CSR [-]*	*Stages*
Test 109	100	90	0.12	1
Test 110	100	90	0.15	3
Test 111	100	90	0.18	3

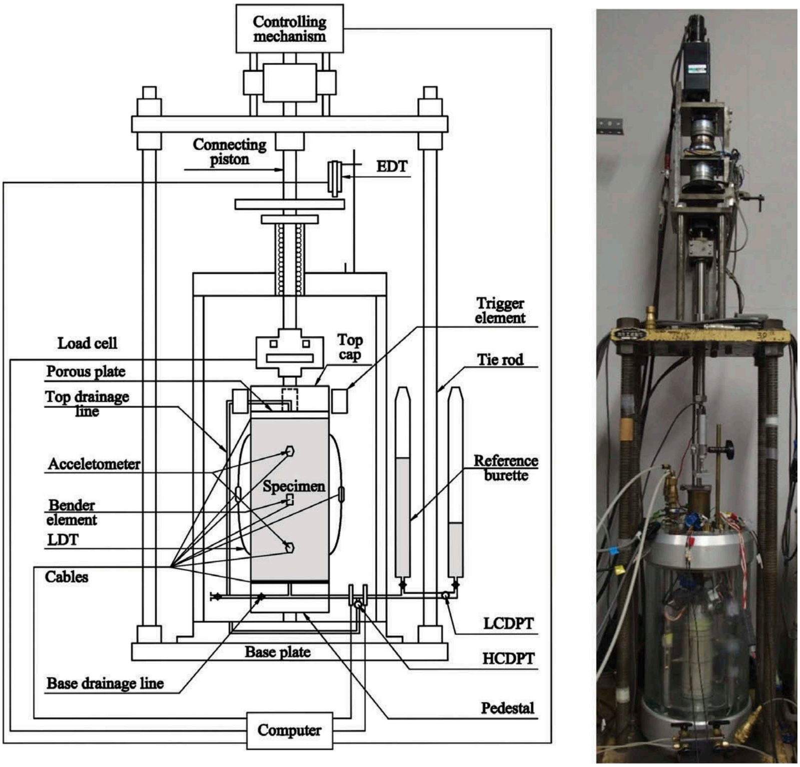

Figure 1. Unconventional triaxial apparatus (Geotechnical Laboratory of "Komaba" Campus of The University of Tokyo – Institute of Industrial Science)

2 EQUIPMENT AND SPECIMEN DESCRIPTION

2.1 *Apparatus*

All laboratory tests have been performed at the Geotechnical Laboratory of the University of Tokyo (Institute of Industrial Science – Komaba Campus) by means of unconventional triaxial apparatus as shown on Figure 1. Three undrained cyclic triaxial tests have been performed on one stage or three consecutive stages at 100kPa confining stress (σ'_c), relative density (D_r) of 90% and various cyclic stress ratio (*CSR*) – 0.12, 0.15 and 0.18 respectively. Summary of the performed tests is shown in Table 1. Basic physical and mechanical properties of the tested material are obtained by additional conventional tests.

2.2 *Tested material*

The tested material is cohesionless – typical Bulgarian sand from Sofia plateau (called "Sofia sand" herein). Sofia sand is beige yellowish soil from Lozenetz region the dominant minerals of which are: amphibole, epidote minerals, titanite, zircon, tourmaline and rutile (Angelova &

Table 2. Physical and mechanical properties of Sofia sand

Specific density	Dry density	Void ratio	Maximum void ratio	Minimum void ration	Relative density	Mean particle diameter	Fines content	Coefficient of uniformity	Angle of shearing resistance
ρ_s	ρ_d	e	e_{max}	e_{min}	D_r	D_{50}	F_C	C_U	φ
[g/cm^3]	[g/cm^3]	[-]	[-]	[-]	[%]	[mm]	[%]	[-]	[°]
2,68	1,40	0,918	1,390	0,866	90	0,22	4,24	2,19	38,46

Figure 2. Photograph of Sofia sand and grain size distribution curve

Yaneva 1998). Its physical and mechanical properties are shown on Table 2. A photograph of Sofia sand is shown on Figure 2 together with its grain size distribution curve.

2.3 *Specimen preparation*

Since the used material is sandy soil (cohesionless), the air pluviation technique has been adopted for the specimen preparation. Through slow pluviation it is possible to create a very uniform specimen of dry poorly graded coarse-grained soils. In this method the material is placed in a mold of 75 mm in diameter and 150 mm of height at a specific vertical distance (depending on the relative density which is aimed) above the specimen surface. The feed door is opened and the material is allowed to fall down in a slow constant stream. The hopper is continuously traversed across the specimen depositing a thin layer of material with each pass. The process is continued until the specimen mold is overfilled by 1 cm. In the end the top surface is formed by a straight edge. The specimen has been fully saturated by the “double vacuum” method. The degree of saturation of the specimen has been evaluated by measuring Skempton’s *B*-value (the value should be larger than 0.96).

3 TEST PROCEDURE

An AC servo-motor has been used in the loading system of the triaxial apparatus so that cycling loading under stress control could be applied accurately to the specimen in vertical direction. In order to measure the vertical stress (σ_1) a load cell is located above the top cap inside the cell so that the effects of piston friction could be eliminated. The vertical strain (ε_1) has been measured by external displacement transducer. The horizontal stress (σ_3) has been applied through the air in the cell which has been measured with high capacity differential pressure transducer.

The stress which has been applied on the specimens during the tests is given in Figure 3 (left). For the sake of reaching σ'_c = 100kPa of isotropic consolidation the stress has been increased in three consequential steps (50 kPa, 80 kPa and 100 kPa) in drained condition. The stress has been kept constant for 30 minutes at each step. When the final isotropic consolidation phase has been reached at σ'_c = 100 kPa the stress is kept constant in a drained condition until the vertical (axial) strains due to volume change cease. In the final step cyclic loading in an undrained condition has been applied up to a certain level deviator stress (σ_{dev}). The test has been terminated when a double amplitude of axial strain higher than 5% is registered ($\varepsilon_a = \varepsilon_{a,max} + \varepsilon_{a,min} > 5\%$) as shown on Figure 4 (left). According to many liquefaction assessment

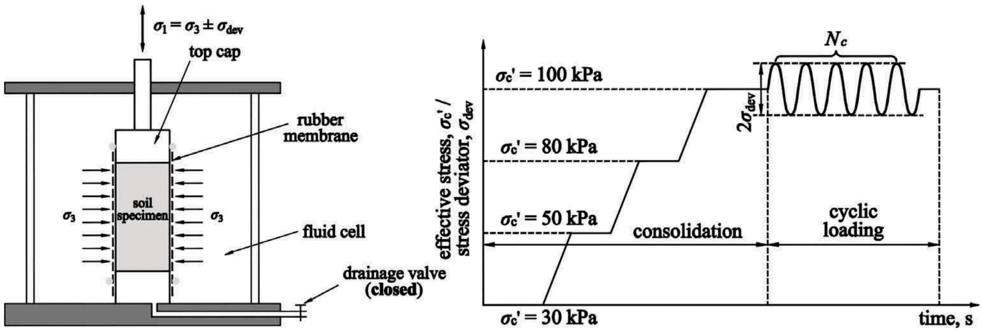

Figure 3. Schematic representation of triaxial cyclic loading and adopted test procedure for one stage

procedures the double amplitude of axial strain of 5% is considered as the starting point of liquefaction (another known criteria is a 95% reduction of the initial effective stress.

The adopted procedure for cyclic triaxial testing is shown in Figure 3 (right). After completing all aforementioned steps the first (initial) stage of liquefaction is considered as performed (called "Stage 1" herein). Thereafter the confining stress (σ'_c) is dropped down once again to 30 kPa and the condition is switched from undrained to drained (the loss of volume is measured at that moment). The whole test procedure is repeated two more times (called "Stage 2" and "Stage 3") in order to reproduce the re-liquefaction phenomena on the specimen. In each stage the consolidation pressure is applied in drained condition and the cyclic loading (deviator stress) is applied in an undrained condition.

Undrained shear strength in terms of cyclic loading is often normalized by the effective overburden pressure, hence the definition of cyclic stress ratio (*CSR*). *CSR* could be defined in various ways depending on the type of laboratory equipment which has been used. For the particular case (triaxial apparatus capable of applying cyclic deviator stress σ_{dev}) it is defined by the $\sigma_{dev}/2\sigma'_c$ ratio ($CSR = \sigma_{dev}/2\sigma'_c$). The number of cycling loads which generate 5% double amplitude of axial strain (N_C) is defined according to Toki et al. 1986 and Figure 4 (right):

$$N_c = 0.5\frac{[DA - DA(N_i)]}{[DA(N_i + 0.5) - DA(N_i)]} + N_i \tag{1}$$

Studies show that the number of cycling loads which generate 5% double amplitude (*DA*) of axial strain increases with the decrease of the cyclic stress ratio (*CSR*) and with the increase of the relative density (D_r). After a certain number of load cycles the CSR-N_C curve flattens and

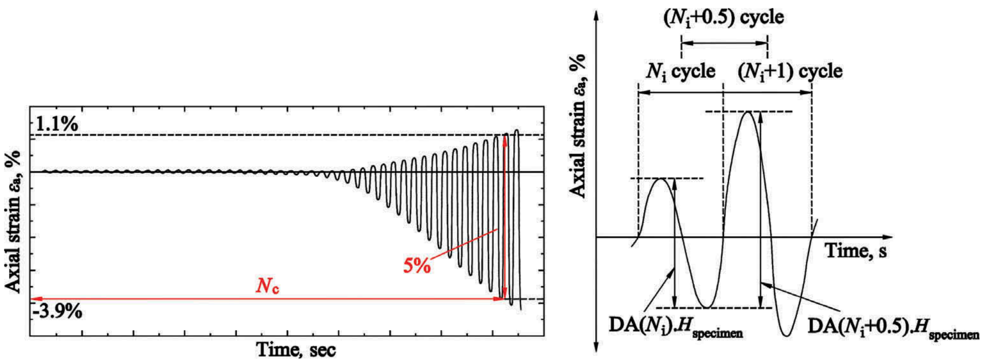

Figure 4. Time-history of axial strain and definition of number of loading cycles which generate *DA* of 5%

continues parallel to the abscissa. The geotechnical practice for obtaining CSR-N_C curves by means of triaxial tests implies that at least three tests are performed at the same effective (confining) stress (σ'_c) of 100 kPa with variation of the deviator stress (σ_{dev}) for the sake of defining several values of CSR.

4 DYNAMIC MEASUREMENTS

4.1 *Trigger elements – accelerometers method*

In order to generate shear waves a special type of source called "trigger elements" has been employed. The trigger elements are composed of multi-layered piezoelectric actuator made of ceramics (dimensions 10 mm x 10 mm x 20 mm, mass of 35 g and natural frequency of 69 kHz) and U-shaped thick steel bar to provide reaction force as seen on Figure 5. Trigger elements have been used in pairs in order to apply excitation equally. For the sake of receivng dynamic waves piezelectric accelerometers (cylindrical in shape with diameter of 3.6 mm, height of 3 mm, mass of 0.16 g and natural frequency of 60 kHz) have been used (glued on the side surface of the specimen at two different heights).

4.2 *Bender elements method*

Bender elements are small piezoelectrical transducers which either bend as an applied voltage is changed or generate voltage as they are bent. For the case of this study two bender elements have been glued on each side of the specimen so that shear waves could be transmitted and received in the cross section of the sample. There have been two ways for inducing shear waves in the cross section as it could be seen in Figure 6. In the first the wave could be propagated perpendicularly through the cross section and the second in parallel through the cross section.

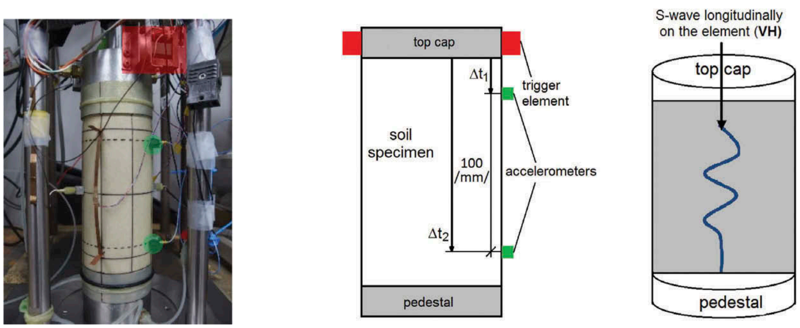

Figure 5. Measurement of shear waves by means of trigger-elements/accelerometers method

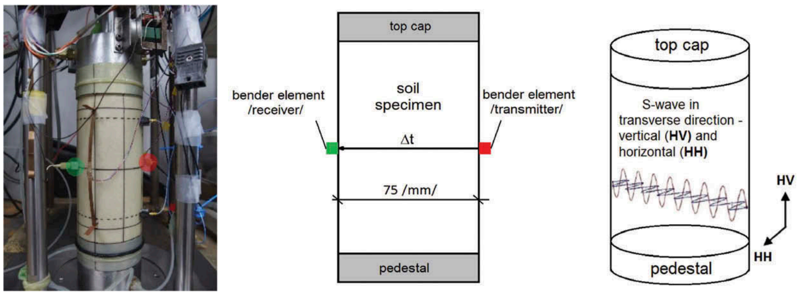

Figure 6. Measurement of shear waves by means of bender elements method

4.3 *Recording techniques for dynamic waves*

A digital oscilloscope has been employed for recording electrical outputs from accelerometers and bender elements with an interval of 10^{-6} sec. In order to obtain clear signals a stacking (averaging) technique which has been adopted in the oscilloscope and has been introduced instead of using filtering methods. The stacking number which has been used is 256 for the bender elements method and 128 for the triggers-accelerometers method.

4.4 *Testing procedure*

Each specimen has been kept under a saturated condition and subjected to isotropic consolidation. For all re-liquefaction stages as soon as the effective stress in the specimen (σ'_c) has reached 30 kPa, 50 kPa and 80 kPa dynamic measurements have been conducted in order to verify soil skeleton structure change before reaching the target confining stress of 100 kPa.

4.5 *Travel time definitions*

The propagation of shear waves through the soil specimen has been used to study the change of structure of the soil skeleton due to re-liquefaction. The two adopted methods involve measuring of arrival time of propagated waves from the source to the receiver, and as the distance between transducers is known, wave velocity can be determined.

In some cases shear waves are difficult to be identified due to near field effect, reflection and refraction of waves. These three factors complicate the detection of the accurate arrival point. There are a few known methods for estimation of the arrival time of waves, such as the cross-correlation method, time domain analysis, frequency domain approach, multiple reflections, wavelet analysis and variable path method.

Two different techniques have been adopted for this study – both related to the time domain analysis. One technique detects arrival time by visual pick and the other uses mathematical procedure (cross correlation) to match the first rise points of the signals. Both methods will be explained below.

Time domain techniques are a direct extraction of travel time based on the plots of the electrical signals versus time. A commonly employed technique for detecting arrival time is visual inspection of the received signal. Figure 7 shows typical shear waveform in time domain series obtained on Sofia sand.

The time lapse between major peaks in the input and output signals is considered as the travel time. As for the bender elements method Point 1 on Figure 7 (left) sets the first major peak of the input signal and Point 2′ or Point 2" (depending on the polarity of the bender elements) on the same figure sets the first major peak of the received signal.

The "first major rise to rise" approach has been adopted for shear wave velocity evaluation in the trigger elements-accelerometers method. The time lapse between the first major deflections of the two output signals from the accelerometers is considered as the travel time as

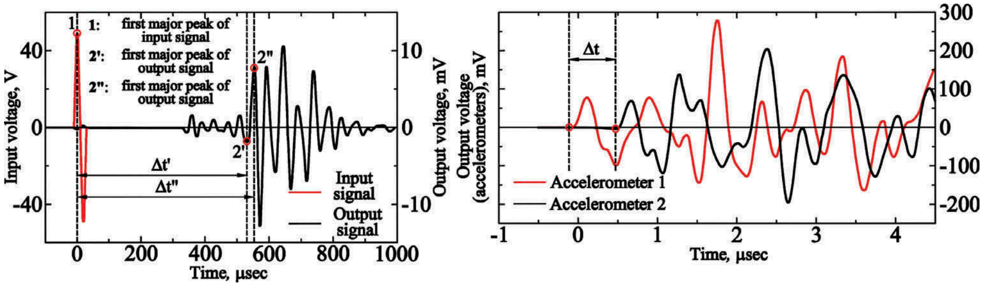

Figure 7. Example results from dynamic measurements and evaluation of travel time of shear waves

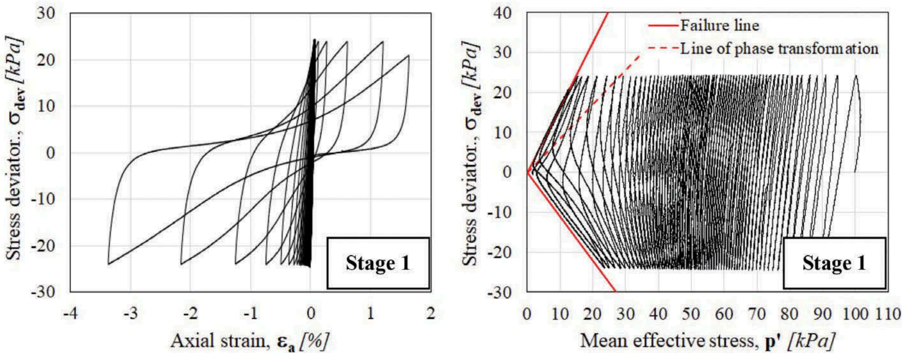

Figure 8. “Test 109” results – *CSR* = 0.120

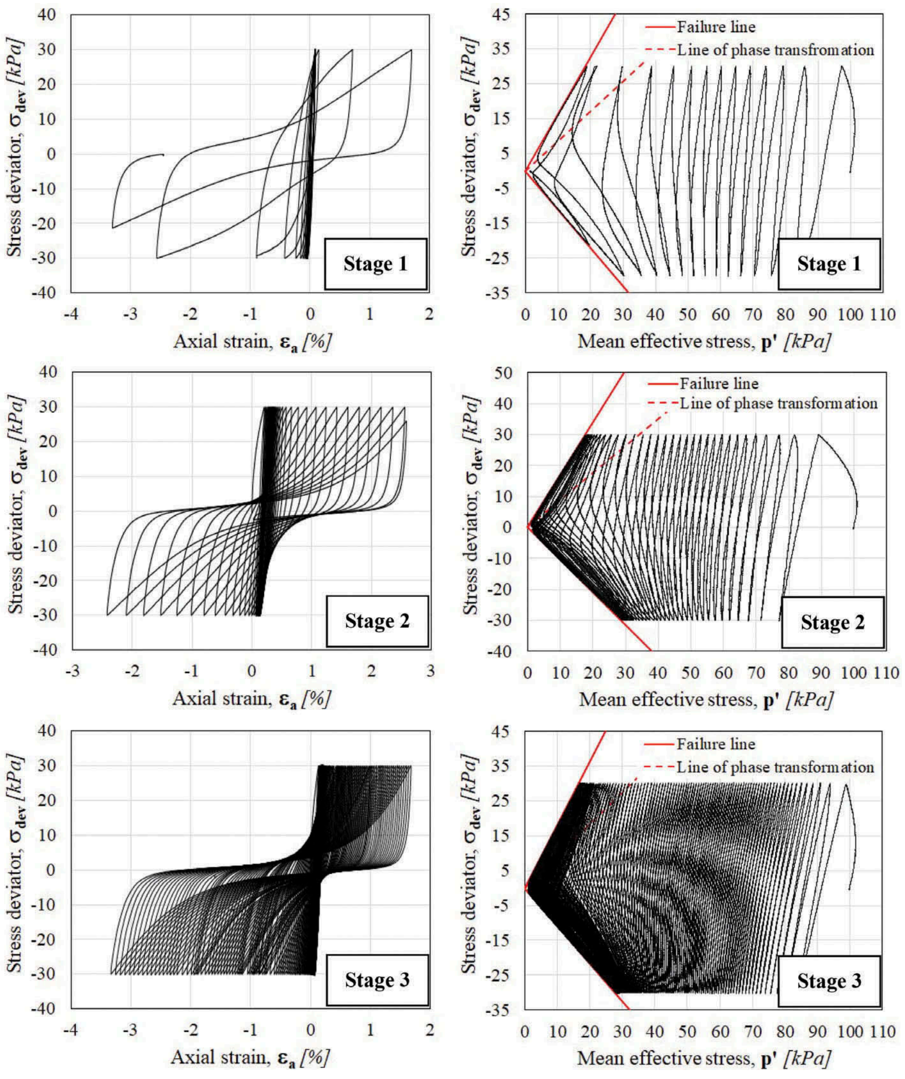

Figure 9. “Test 110” results – *CSR* = 0.150

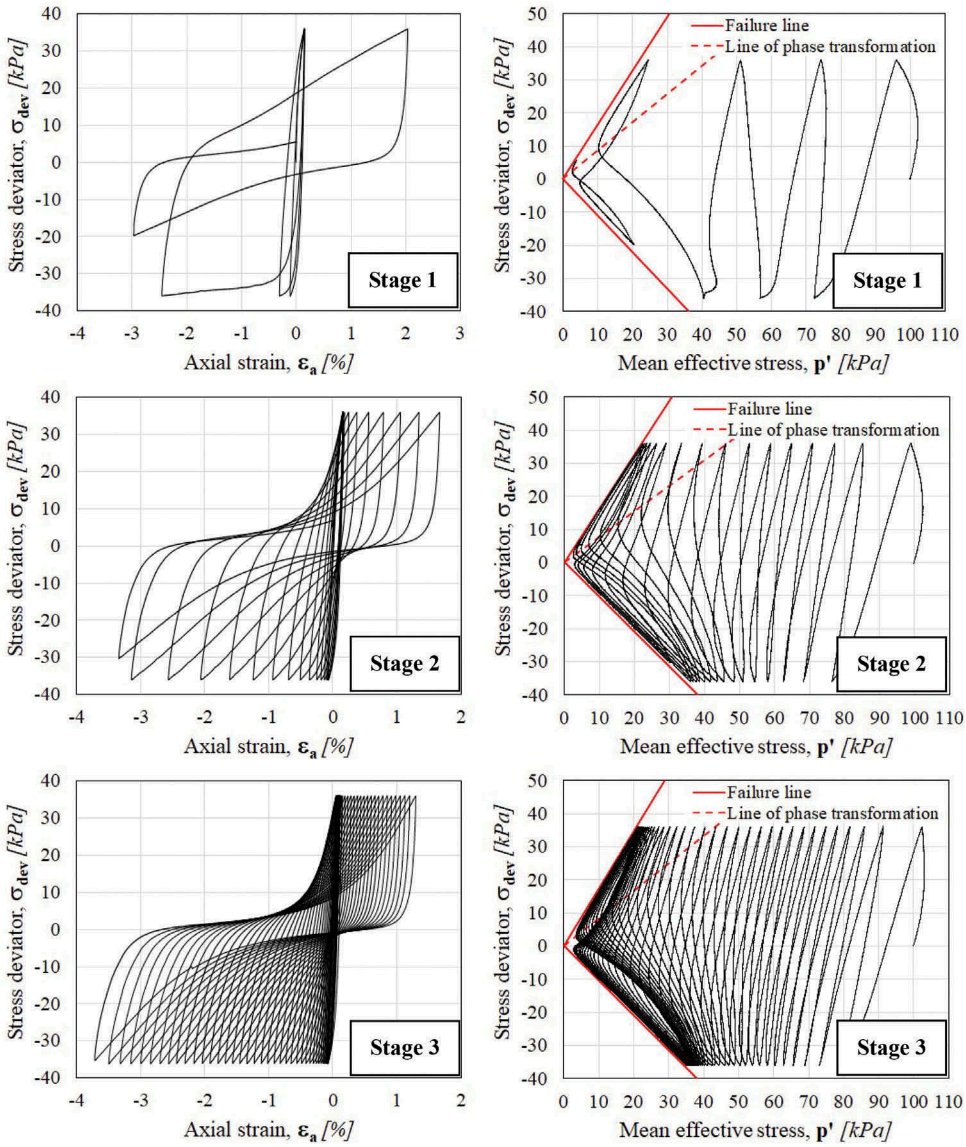

Figure 10. "Test 111" results – CSR = 0.180

shown Figure 7 (right). In order to mathematically obtain the inflection point (rise) a cross-correlation has been adopted.

4.6 *Void ratio function*

Due to change of relative density of the soil for each re-liquefaction stage the use of "void ratio function" ($f(e)$) has been used in order to eliminate the various void quantity effects on the results. A number of equations for $f(e)$ is suggested in the literature which allows for the direct comparison of results from tests which have been performed at several values of relative density. A commonly used equation for defining the "void ratio function" for cohesionless soils is (Hardin & Richart 1963):

$$f(e) = \frac{(2.17 - e)^2}{(1 + e)} \tag{2}$$

where e = void ratio.

5 RESULTS

Figures 8-10 show results from the three tests. "Test 109" consists of only one stage while "Test 110" and "Test 111" consist of three stages of repeated liquefaction.

Figure 11 shows the change of the CSR-N_C relation for three stages of re-liquefaction.

Figure 12 shows the dependency of multiple liquefaction resistance at three stages (represented by N_C-value) on the soil skeleton structure (represented by normalized by $f(e)$ small-strain stiffness) which has been defined in two ways by shear wave velocity measurement.

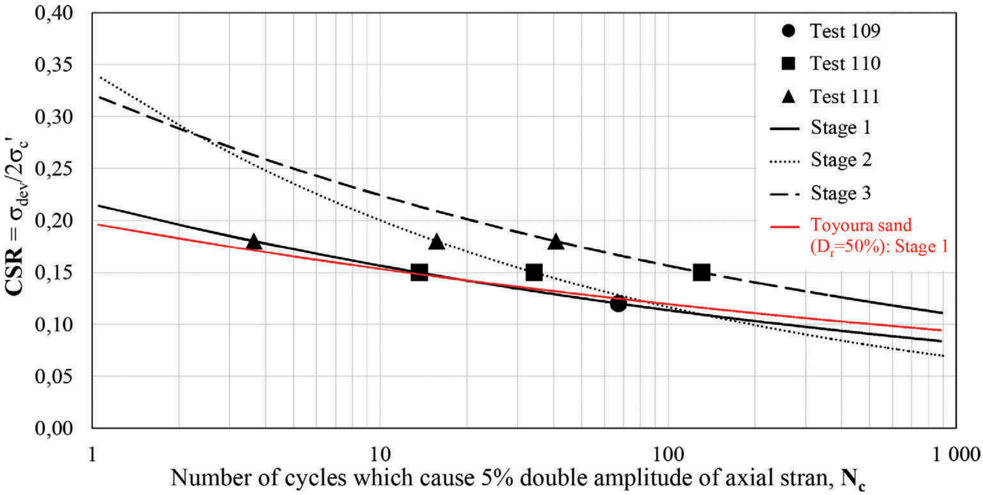

Figure 11. CSR-N_C relations for three stages of repeated liquefaction

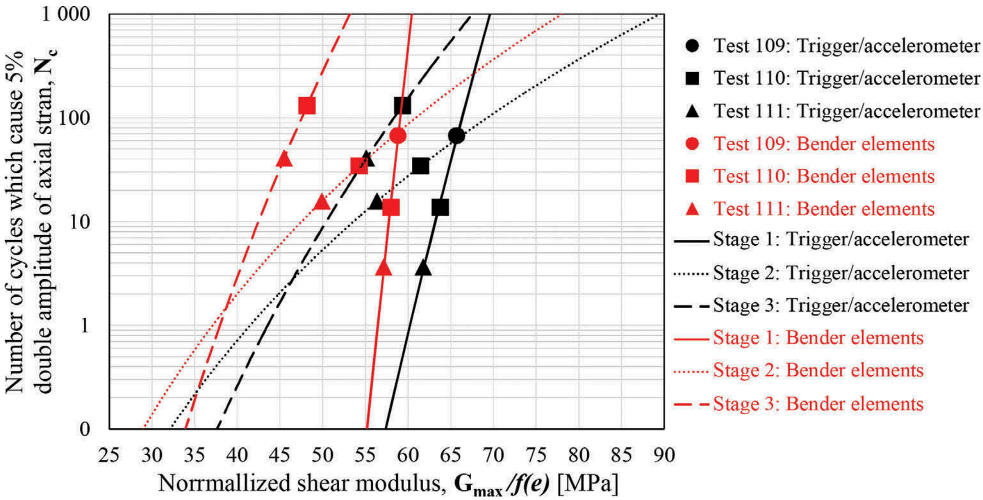

Figure 12. N_C-$G_{max}/f(e)$ relation for three stages of repeated liquefaction

6 CONCLUSIONS

A series of undrained cyclic triaxial tests together with dynamic measurements have been performed on Sofia sand in order to investigate its re-liquefaction resistance due to soil skeleton variation (structure change) caused by densification the reason for which is each subsequent liquefaction event (stage). The main observations are as follows:

- On the basis of small-strain stiffness of soil which could be easily determined "in-situ" (through cross-hole logging, down hole, suspension sonde method among other methods) as well as in laboratory (resonant column, torsional shear test, cyclic triaxial test, bender elements method among other methods) a prediction of the future re-liquefaction resistance of soil could be made.
- In the general case with the particular material which has been studied (Sofia sand) consecutive liquefaction improves its re-liquefaction resistance.
- CSR-N_C relation of Stage 2 of liquefaction crosses CSR-N_C relation of Stage 1 of liquefaction (cross-point: CSR = 0.108 and N_C = 133) and CSR-N_C relation for Stage 3 of liquefaction (cross-point: CSR = 0.280 and N_C = 2,30) which proves that soil particle rearrangement (densification) does not certainly mean resistance improvement.

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REFERENCES

Angelova, D., M. Yaneva, 1998. New data for the lithograph of neogene in the Sofia plateau. *Magazine of Bulgarian Geotechnical Society*, 59 (2),pp. 37-40 (in Bulgarian).

Hardin, B. O., F. E. Richart, 1963. Elastic wave velocities in granular soils. *Soil Mechanics and Foundations*, 89(1), pp. 33-65.

Toki, S., F. Tatsuoka, S. Miura, Y. Yoshimi, S. Yasuda, Y. Makihara, 1986. Cyclic undrained triaxial strength of sand by a cooperative test program. *Soils and Foundations*, 26(3), pp. 117-128.

Villalta, G., 2015. *Change of Shear wave velocities induced by repeated liquefaction*. Tokyo: Department of Civil Engineering, University of Tokyo.

Wahyudi, S., Koseki, J., Sato, T., and Chiaro, G. 2015. Multiple-Liquefaction Behavior of Sand in Cyclic Simple Stacked-Ring Shear Tests, *Int. J. Geomech*., 10.1061/(ASCE)GM.1943-5622.0000596, C4015001.