POST-CYCLIC LOADING SETTLEMENT OF SATURATED CLEAN SAND

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Abstract

Saturated sand deposits may experience significant settlement or reconsolidation with the dissipation of excess pore pressure in the post-cyclic loading phase even if liquefaction has not occurred. This paper presents results from an extensive and systematic experimental study on the factors governing the post-cyclic loading settlement of saturated clean sand. Strain-controlled, undrained, triaxial tests were performed on reconstituted specimens of Ottawa (C-109) sand. Effects of excess pore pressure, induced shear strain, consolidation stress, number of loading cycles, and relative density on post-loading settlement were investigated. It was found that the settlement potential during the post-loading phase is strongly correlated to the excess pore pressure; the larger the excess pore pressure the greater the settlement. The potential to develop excess pore pressure, however, decreases considerably with increasing consolidation effective stress in the range between 100 kPa and 400 kPa. Additionally, the level of induced shear strain and the number of loading cycles were confirmed to be important factors in the development of excess pore pressure and post-loading settlement. The results revealed a unique relationship between initial liquefaction and post-loading settlement. Specimens that were loaded up to initial

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liquefaction were found to experience about the same volumetric strain in the post-loading phase, irrespective of the induced shear strain.

KEYWORDS: Cyclic triaxial testing; Strain-controlled test; Excess pore water pressure; Cyclic settlement; Volumetric strain; Threshold shear strain; Liquefaction of sand

1. Introduction

Loose to medium dense sand deposits tend to densify when subjected to strong ground motion due to particle rearrangement and redistribution of voids within the soil. Dry soil deposits experience densification during shaking while saturated deposits mostly settle and densify after shaking with the dissipation of excess pore water pressure. Depending on the level of generated excess pore pressure, soil index properties, and other factors such as the thickness of soil layer and depth of the ground water table, post-shaking settlement (typically measured by volumetric strain and also referred to as reconsolidation settlement) may cause significant damage to any structure above or within the ground.

The amount of settlement for dry sands depends on the relative density, level of induced shear strain, confining pressure, and number of loading cycles [18]. For saturated sands, however, excess pore pressure generation is the key factor for determining the post-cyclic loading settlement [16]. In cases where large excess pore pressure generation or liquefaction occurs (i.e., $r_u > 0.9$, where $r_u = \Delta u / \sigma_c'$; Δu = excess pore pressure and σ_c' = initial effective confining pressure) significant settlement (i.e., 2% and greater volumetric strain) may be expected with dissipation of excess pore pressure in the post-cyclic loading phase [23]. For smaller excess pore pressure ratios and nonliquefaction cases, the volumetric strain is typically well under 1% [16]. Because the major soil strength and stiffness is preserved at relatively low excess pore pressures, the settlement due to dissipation of such low excess pore pressures is limited [1, 16, 22].

This study focuses on the generation of excess pore pressure as a key factor governing the post-cyclic loading settlement. The generation of excess pore pressure is controlled mainly by the level of induced shear strains [4]. However, most of the previous experimental studies on cyclic settlement of sands have been typically conducted through stress-controlled tests where the emphasis is on the stresses induced and not necessarily the strains. Further, in stress-controlled testing the induced shear strain can show great variation from cycle to cycle. As a result, the post-cyclic loading volumetric strain is correlated to the maximum induced shear strain. In this study, however, the generation and dissipation of excess pore pressure in clean sand were explored through strain-controlled cyclic tests that allow for more direct evaluation of the characteristics of post-cyclic loading settlement. Specifically, the following effects on the settlement were evaluated: (i) level of induced shear strain, (ii) consolidation effective stress, (iii) number of loading cycles, (iv) initial liquefaction, and (v) relative density.

2. Previous research

Lee and Albaisa [16] studied the settlement of sands caused by the dissipation of excess pore pressure through stress-controlled cyclic triaxial tests. The grain size, relative density, and excess pore pressure were reported as the key factors controlling the amount of the settlement of sand. However, no conclusions were drawn about the possible effect of the induced strains on the post-loading/post-liquefaction settlement.

Tatsuoka et al. [21] studied the influence of various parameters on volumetric strains after initial liquefaction through cyclic stress-controlled, undrained simple shear tests. They found that the amount of settlement significantly depends on the induced maximum shear

strain and density of soil. The settlement was found to be relatively insensitive to the overburden pressure.

Tokimatsu and Seed [22] compiled and analyzed previous data from Tatsuoka et al. [21]; Lee and Albaisa [16]; and Yoshimi and Hiroshi [25], and reported that the maximum shear strain is an important factor influencing the settlement after liquefaction. It should be noted that these studies referred to the maximum shear strain that developed in the sample during cyclic stress-controlled testing, and it is unclear whether the influence of the shear strains on settlement was due to the nature of stress-controlled testing (i.e., progressive increase in strain). Liquefaction resistance in stress-controlled testing is typically affected by the method of sample preparation and stress history [17], however Tatsuoka et al. [21] reported that these factors were less significant when dealing with post-liquefaction volumetric strain. They attributed this to the relatively large strains developed during liquefaction and suggested that the settlement behavior of sands is primarily controlled by the relative density and the induced maximum shear strain.

Duku et al. [2] presented results from an experimental study on seismic compression of dry and partially saturated sands. They concluded that seismic compressibility was significantly affected by relative density and confining pressure.

Ueng et al. [23] conducted a study on settlement of a saturated clean sand using a large biaxial laminar shear box. Various one- and multi-directional sinusoidal input motions were imposed by a shaking table at different frequencies and accelerations. They reported that post-liquefaction volumetric strain of the sand decreased with increasing relative density regardless of shaking amplitude, frequency and direction (1-D or 2-D shaking), but increased with shaking duration (i.e., number of loading cycles). Ueng et al. [23] also reported that a surcharge mass of about 1m thick overlying soil layer did not significantly

affect the settlement characteristics for sand; only slightly increased volumetric strains for liquefied specimens.

Few studies [18, 26, 24, 10, 11] employed strain controlled simple shear tests in soil settlement tests, but they either referred to the settlement behavior of cohesive materials or concentrated on different aspects of the soil settlement, such as dry soils and threshold values of shear strains, without a direct evaluation of the strain effect on post-cyclic loading settlement of fully saturated sand.

Most relevant previous research efforts and the significant findings of these studies are summarized in Table 1.

3. Experimental Program and Methodology

Effects of induced shear strain, consolidation stress, number of loading cycles, and relative density on excess pore pressure generation and post-loading settlement of clean sand were investigated in four different groups of testing. The test series are listed in Tables 2 to 5. The first set includes tests performed on specimens subjected to cyclic loading under effective consolidation stresses of 100kPa, 300kPa and 400kPa (Table 2). The variable parameter in the second group of tests (Table 3) is the number of loading cycles. In this series, specimens with identical properties (relative density, saturation, reconstitution technique) were subjected to different number of loading cycles ranging from 10 to 100 loading cycles. The third series (Table 4) were performed on medium dense sand and terminated with initial liquefaction. Finally, a separate series of tests was conducted on specimens with low relative density to investigate the influence of relative density on reconsolidation characteristics of clean sand (Table 5).

3.1 Soil Tested

The soil investigated in this study was Ottawa sand (C-109). Basic index properties of the soil were determined using pertinent ASTM procedures, and are summarized in Table 6. Ottawa sand is a commercially produced clean sand with a median grain size D_{50} of 0.42 mm. The particle size distribution of Ottawa sand is shown in Fig.1. It is classified as poorly graded sand (SP) according to USCS classification.

3.2 Cyclic Triaxial Testing Equipment

A state-of-the-art cyclic triaxial testing equipment was used in this research. The custom-made triaxial apparatus has a tilting frame that allows for in-place specimen preparation. This unique feature helps obtaining high quality specimens and thus more reliable testing results. A photograph and schematic diagram of the system used are shown in Fig. 2. The system is equipped with signal conditioning, servo amplifier, computer interface and data acquisition units. Normal loading, cell pressure application, and back-pressure saturation are controlled through servo valves. The triaxial cell accepts 101.6 mm (or smaller) diameter specimens.

Prior to executing the testing program, the equipment was calibrated and performance of the system was verified by conducting a series of calibration tests and comparing the results with previously published data. This benchmark testing ensured the reliability of the equipment and helped form a baseline in strain-controlled mode. Undrained cyclic triaxial tests were performed at shear strains varying from 0.003% to 0.5%. It is important to note that the high precision mini LVDTs located outside the triaxial cell (Fig. 2) were calibrated against an internal LVDT placed between the top and bottom platens. This was done to eliminate any possible compliance effect on the strain measurements. Monterey #0/30 sand was used for verification, because no reported data

were found in the literature on strain-controlled cyclic loading of Ottawa (C-109) sand. In “Liquefaction of soils during earthquakes” [3], Dobry reported results from several sands including Monterey sand. Thus, the results from this study were compared to those of Dobry, as shown in Fig 3. The response of Monterey #0/30 was found to be in between the upper and lower bounds of Dobry, indicating quite good agreement. Tests on Ottawa (C-109) sand were also conducted to form a baseline for this study. Interestingly, the results from Ottawa (C-109) sand were found to fall on the upper bound of Dobry. Based on the verification tests, it was concluded that the equipment produced high quality and reliable data through strain-controlled testing.

3.3 Specimen Preparation

In this study, the moist undercompaction method [14] was adopted for reconstituting the soil specimens. A predetermined amount of dry soil required to form a 101.6 mm diameter by 205 mm tall specimen was mixed with distilled, de-aired water such that an initial saturation of about 50% was achieved prior to placing the soil into the mold. The soil-water mixture was prepared in an air-tight container to prevent any loss of water due to evaporation. To ensure a uniform distribution of moisture, the soil-water mixture was left for about 10 hours in a sealed container prior to forming the specimen. The soil specimen was formed in 10 layers. A manual tamper was used to achieve the desired layer height while extra care was given to distribute the soil uniformly in the layer. After compaction, the tamper was removed and the layer surface was slightly roughened in order to obtain a good contact with the overlying layer and prevent the forming of a weak inter-layer shear plane. To compensate for densification during compaction, less compactive effort was applied to the bottom layers than the layers above them. Thus, approximately a uniform density distribution throughout the specimen was achieved through the whole compaction process. After compacting the final layer, the top cap was placed and twisted slightly to

ensure full contact between the cap and the specimen. Then, the drainage lines were connected, and 20 kPa of vacuum was applied to the specimen prior to dismantling the mold. O-rings at the top cap and base pedestal were applied and tightened further around the membrane with the help of hose clamps. The dimensions of the specimen were measured and recorded.

Following reconstitution of the specimen, the triaxial chamber was installed, and the cell was filled with tap water through the bottom cell line. It is important to leave the top cell line open while the water is being filled into the cell, otherwise the air pressure accumulating inside the cell can easily overconsolidate the sample. Vacuum inside the sample was released while increasing the cell pressure at the same rate to maintain a constant net effective pressure on the specimen. Next, carbon dioxide (CO₂) was percolated through the sample to expedite the saturation. The CO₂ displaces the air; and being much more soluble in water than air, it enables subsequent saturation steps to be carried out successfully. The de-aired water was flushed through the bottom drainage line of the specimen. The LVDTs were assembled and zeroed so that specimen deformations during the saturation could be monitored. Saturation was achieved by application of back-pressure of 100 to 200 kPa (pressure applied to the pore water within the specimen) while maintaining a constant effective stress of about 30 kPa on the specimen. Full saturation was assumed to have been achieved for a Skempton's B-value of 0.96 or higher.

Following back-pressure saturation, the confining effective stress was increased to the desired level and the specimen was allowed to consolidate; the volume change during consolidation was monitored and recorded. The consolidation was continued until no further volume change of the specimen occurred. This took about 30 to 45 minutes. Based on the measured volume change and recordings of the LVDTs, post-consolidation dimensions of the specimen were determined.

3.4 Cyclic Loading

To ensure undrained conditions, the drainage lines were closed prior to the application of cyclic loading. The desired level of axial strain was applied sinusoidally at a frequency of 0.2 Hz. The relatively slow loading rate was selected to ensure equilibration of pore water pressure and thus provide more accurate pore pressure measurements [9]. The test was terminated after a specified number of loading cycles (i.e., $N=10, 50$, or 100) or a specified r_u value (i.e., $r_u=1.0$). Each specimen was subjected to only one strain level; staged testing was avoided to eliminate the effect of previous straining on the excess pore pressure generation.

Results of a typical strain-controlled, undrained, cyclic triaxial test are illustrated in Fig. 4. A clean sand specimen is subjected to 0.3% shear strain for 50 loading cycles. The shear stress continuously drops with cyclic loading in response to increased excess pore pressure and decreased stiffness. The sample exhibits only about 20% of the initial shear strength (stress) after the 9th cycle during which the excess pore pressure ratio is 0.86. After 10 cycles, the excess pore pressure is almost equal to the confining pressure (i.e. $r_u \sim 1.0$) and the remaining shear stress decreases to negligible values. Beyond this point the sample is considered to have fully liquefied. It should be pointed out that the increase in excess pore pressure is the key parameter in determining the shear strength of the specimen at any given cycle. Fig. 4-d shows shear stress versus shear strain response of the specimen. The figure clearly illustrates degradation of stiffness. The specimen shows higher stiffness (about twice as high) in compression (positive shear stress and strain) than in tension in the initial cycles of loading. As the number of loading cycles increases, the stress-strain response flattens out showing almost no resistance to the applied stresses. It should again be

noted that this type behavior of stress reduction is primarily due to the increase in the excess pore pressures.

3.5 Post-loading Measurements

At the end of each test, the generated excess pore pressure was allowed to dissipate by permitting drainage into the graduated backpressure burettes. This allowed the specimen to reconsolidate. Due to the high permeability of the sand, the reconsolidation was completed in about 15 to 20 minutes. Volume and height change of the specimen in the post loading phase were recorded.

4. Results and Discussion

4.1 Excess Pore Pressure Generation under Different Consolidation Stresses

Three levels of effective consolidation stresses (100 kPa, 300 kPa and 400 kPa) were investigated from specimens formed at similar relative density (i.e., $D_r \sim 50-55\%$), as discussed previously and presented in [Table 2](#). Testing at different levels of consolidation stress was necessary to understand the effect of confining pressure on the generation of excess pore pressure and post-loading settlement. In the field, these consolidation stresses correspond to effective confining pressure of soil specimens located at different depths. Various factors, such as heterogeneous soil layers and/or varying densities and existence of a surcharge due to an above ground fill/structure, need to be considered in order to precisely estimate the corresponding depths.

To evaluate the influence of effective consolidation stress on medium dense sand ($D_r=50-55\%$), the excess pore pressure ratio as a function of number of loading cycles for shear strain levels of $\gamma_c=0.03\%$, $\gamma_c=0.06\%$, $\gamma_c=0.1\%$ and $\gamma_c=0.3\%$ are presented in [Fig. 5](#). It can be clearly seen from the figures that the excess pore pressure generation is influenced

significantly by the level of shear strain as well as the effective consolidation stress. The largest pore pressure ratios were consistently recorded in the specimens consolidated at 100 kPa effective stress at all levels of shear strain and for all loading cycles. At the small shear strain of 0.03% (Fig. 5a), the influence of consolidation stress is relatively insignificant in the initial cycles of loading. However, it becomes more and more pronounced with increased number of loading cycles indicating much less pore pressure ratios for the specimens consolidated at 300 kPa and 400 kPa. In the mid-range levels of cyclic shear strain of 0.06% and 0.1% (Fig. 5b and 5c), the influence of consolidation stress is noticeable from the early cycles and continues with growing impact throughout the loading. For specimens subjected to larger shear strain amplitudes (that eventually liquefied as shown in Fig. 5d), similar trends were observed, however the influence of consolidation stress is somewhat shadowed due to the high level of excess pore pressure ratios. The pore pressure reducing effect of consolidation stress can still be observed in the form of delayed liquefaction for specimens consolidated to higher effective stress. It took more than 30 cycles for both the 400 kPa and 300 kPa specimens to liquefy whereas the 100 kPa specimen took about 10 loading cycles to reach liquefaction. It is important to note that there is a slight variation in the relative density between different values of consolidation stress; the relative density somewhat increased with increasing consolidation stress. The effect of the variation in relative density is also included in the trends in Fig. 5, however the effect must be relatively small as shown later in Fig. 12.

In strain-based approach for liquefaction analyses [4], the excess pore pressure generation is typically defined as a function of shear strain at $N=10$. The value of N is related to the magnitude, M_w , of the earthquake; $N=10$ is used to represent $M_w=7.0$ earthquake [4, 20]. Such pore pressure generation curves for all levels of consolidation effective stress are shown in Fig. 6. The lower bound pore pressure generation curve

represents the response of specimens subjected to 400 kPa consolidation stress while the upper bound was obtained from specimens subjected to 100 kPa consolidation stress. The results from specimens subjected to 300 kPa consolidation stress consistently fall in between the upper and lower bounds. Dobry et al. [4] reported similar results from specimens of Monterey sand subjected to consolidation effective stresses of 26 kPa, 96 kPa, and 192 kPa.

The influence of consolidation stress on the generation of excess pore pressure is significant in the middle range shear strains ($\gamma_c > 0.03\%$); as much as 50% and even greater reduction in excess pore pressure ratio occurs with increasing consolidation stress from 100 kPa to 400 kPa.

The cyclic threshold shear strain (γ_{ct}) is defined as the level of strain below which no excess pore pressure develops [4, 5]. The data in Fig. 6 also indicate and confirm the findings of previous studies [5, 6] that the threshold shear strain is influenced by the consolidation effective stress. It is clear that the threshold shear strain tends to shift to the right with increasing consolidation effective stress. The threshold shear strain at 100 kPa consolidation effective stress is around 0.01% and it appears to increase to around 0.02% at 400 kPa consolidation effective stress.

4.2 Influence of Consolidation Stress on Post-Loading Settlement

To investigate the influence of the consolidation stress on the volumetric strain, post-testing volume change data of the tests performed under different consolidation effective stresses were analyzed. The volumetric strain was determined from the ratio between post loading volume change and the post consolidation (pre-loading) volume of the specimen ($\epsilon_v = \Delta V_{pt}/V_p$).

Volumetric strain as a function of the generated excess pore water pressure ratio at the 10th cycle is shown in Fig. 7. It can be seen that when the excess pore pressure ratio is smaller than 0.1, very limited settlement occurs regardless of the consolidation effective stress. In general, for excess pore pressure ratio larger than 0.1, appreciable volumetric strains (i.e., 0.1% to 1%) were experienced in the specimens consolidated to 400 kPa effective stress. Volumetric strains from the specimens consolidated to 100 kPa were much smaller (0.03% to 0.5%), and those of the specimens consolidated to 300 kPa were found to be in between with the exception of $r_u=0.32$ where greater volumetric strain occurred. The general trend suggests that for the same value of excess pore pressure ratio, the volumetric strain increases with increasing consolidation stress, and that the influence of consolidation stress becomes more pronounced at higher excess pressure ratios. Lee and Albaisa [16] reported similar results, however they found the influence of the effective confining pressure to be significant only for values of pore pressure ratio higher than 0.6.

Volumetric strain as a function of the induced shear strains at N=10 for all consolidation effective stresses is shown in Fig. 8. There is a unique correlation between volumetric strain and induced shear strain without strong and clear dependency on consolidation stress. The volumetric strain data from all levels of consolidation stress form a relatively narrow band between 0.06% and 0.3% shear strain, which may be approximated with the following correlation:

$$\varepsilon_v = 0.5394 \ln(\gamma_c) + 1.6308 \quad (\text{Eq. 1})$$

where ε_v = volumetric strain; and γ_c = induced shear strain. Additionally, the results in Fig. 8 indicate that there is no appreciable post-loading settlement up to the level of 0.03% shear strain. Therefore, this strain level can be considered as volumetric strain threshold [11]

below which little to no settlement occurs. It is important to note that this threshold level is higher than that based on excess pore pressure generation (i.e., 0.01%).

4.3 Influence of Number of Loading Cycles on Post-Loading Settlement

Tests that were performed with different number of loading cycles (Table 3) were evaluated in terms of post-loading settlement characteristics. Fig. 9 illustrates the post testing volumetric strains as a function of cyclic shear strain. It can be seen from the figure that the samples subjected to higher number of loading cycles consistently resulted in larger volumetric strains. Shear strains smaller than 0.03% did not cause appreciable volumetric strain confirming this level as volumetric strain threshold.

A strong correlation exists between the volumetric strains and the number of loading cycles. The settlement was doubled as a result of 50 loading cycles ($\epsilon_v=1.2\%$) compared to that of the test terminated after 10 loading cycles at 0.5% shear strain ($\epsilon_v=0.62\%$). It should be noted that specimens subjected to shear strains of 0.1% and greater, reached liquefaction ($r_u=1.0$) in all tests except that terminated at $N=10$. It was also found that further cyclic loading following initial liquefaction resulted in larger volumetric strains, as shown in Fig. 9. This is in agreement with the findings of Ueng et al. [23] where larger volumetric strains with increased shaking duration were reported. However, other previous research efforts through stress-controlled testing [16, 22] showed no clear correlation between settlement and any parameter, including cycles of loading, beyond initial liquefaction.

The impact of loading cycles on volumetric strain as a function of the generated excess pore water pressure ratio is shown in Fig. 10. Despite the same levels of generated pore pressure ratios, samples that were subjected to higher number of loading cycles consistently resulted in larger volumetric strains. It should be noted that the pore pressure data in this figure represents the measured values at the 10th loading cycle of each test. The

trends suggest that through strain-controlled testing a clear correlation between the number of loading cycles and post-liquefaction settlement exists. Volumetric strain was found to increase with increasing number of loading cycles for specimens that experienced liquefaction (i.e., $r_u \sim 1.0$).

4.4 Settlement after Initial Liquefaction

In addition to the tests performed with different number of loading cycles, four tests (Table 4) were conducted and terminated with the initiation of liquefaction (i.e., $r_u = 1.0$). The results of these tests together with those presented earlier in Fig. 9, are shown in Fig. 11. Unlike the results of the tests terminated after certain number of loading cycles, these series of tests showed no dependence between volumetric strains and induced shear strains. It is interesting to note that all of the tests produced almost identical amount of reconsolidation volumetric settlement. Additionally, the results of these tests allow differentiating between the liquefaction and non-liquefaction regions. The data that fall above the liquefaction line (shown as $r_u = 1.0$ in Fig. 11) show clear correlations after liquefaction between volumetric strain and parameters such as number of loading cycles and induced shear strain. Previous studies, which mainly investigated the soil settlement through stress-controlled testing, concluded that volumetric strain after the initial liquefaction is mainly controlled by the level of induced shear strains [21, 22, 12]. However, the results of the current study showed that not only the induced shear strain but also number of loading cycles will have significant impact on volumetric strain after initial liquefaction.

4.5 Influence of Relative Density on Excess Pore Pressure Generation and Post-Loading Settlement

Tests series given in Table 5 were performed on loosely formed specimens at relative density ranging between 33% and 39% with an average $D_r = 35\%$ under 100kPa

effective consolidation stress. Specimens in this category were subjected to cyclic shear strains varying from 0.02% to 0.5%.

[Fig. 12](#) illustrates the comparison of pore pressure generation curves for specimens of loose and medium densities. The data indicate that specimens prepared at lower relative densities experience slightly higher excess pore pressure compared to medium dense specimens subjected to the same shear strains. In general, the difference is about 10% over a broad range of shear strains. As discussed in the previous sections, the dominant factor on the generation of excess pore pressure is the cyclic shear strain but not the relative density. This is in agreement with the findings of Dobry et al. [4] where they reported a slight influence of D_r on r_u , and suggested, for practical purposes, a representative band irrespective of D_r . Hazirbaba and Rathje [7, 8] reported similar results on the effect of relative density from tests performed on Monterey #0/30 sand through cyclic simple shear testing. The insensitivity of the generation of excess pore pressure to factors (e.g., relative density) other than cyclic strain amplitude is considered to be the hallmark of the cyclic strain approach [13, 15].

Responses of medium dense and loose sands were also evaluated from specimens that experienced liquefaction. The number of loading cycles required to initiate liquefaction (N_L) as a function of the induced cyclic shear strain from all specimens that reached $r_u \sim 1.0$ are presented in [Fig. 13](#). At any given cyclic shear strain level within the range between 0.1% and 0.5%, the number of loading cycles required to initiate liquefaction is higher for medium density specimens (i.e., denser specimen takes more cycles to degrade and finally liquefy than loose specimen). It is interesting to note the strong correlation between N_L and γ_c , which is represented by a power equation regression model with R^2 value of 0.945 and 0.983 for medium dense and loose specimens, respectively. The number of loading cycles required to cause liquefaction was also found to be influenced by the consolidation effective

stress between 100 kPa and 300 kPa; at a given shear strain, N_L increases with increasing consolidation effective stress. N_L for 400 kPa and 300 kPa were found to be the same, as displayed in Fig 13.

The post-cyclic loading volumetric strains from the two series of tests (i.e. $N=10$ and $N=50$) at each range of relative density are displayed as a function of the induced shear strain in Fig. 14. The effect of relative density on post-cyclic loading settlement is very clear and distinct. Loose sand specimens in comparison to those of medium sand consistently experienced greater post-cyclic loading volumetric strains at all shear strain levels beyond 0.03%. The difference in volumetric strains due to relative density is as much as 80%. This was recorded at the shear strain of 0.5% from specimens subjected to 50 loading cycles; the post-loading volumetric strain of medium dense sand ($D_r \sim 50-55\%$) is 1.2% while that of loose sand ($D_r \sim 35\%$) is 2.16%. Specimens of both relative density ranges were found to experience lesser volumetric strains at $N=10$ than at $N=50$, which once again indicates the effect of number of loading cycles on volumetric strain. It is interesting to note that both medium dense and loose sand specimens did not experience any appreciable volumetric strain when subjected to shear strains smaller than 0.03%. Thus, the relative density, for the ranges considered in this study, appears to not affect the volumetric strain threshold.

Conducting tests on loose specimens allowed to further investigate the finding of “identical volumetric strain with initial liquefaction” as discussed in the previous section. Similar to previous results from medium dense sand, volumetric strains in loose specimens also showed no dependence to the induced shear strain levels, as shown in Fig. 15. However, slightly larger volumetric strains were obtained from the loosely formed specimens compared to medium dense specimens. This may be attributed to the larger voids and the higher excess pore water pressure generation in the loose specimens. It is also worth

to note that the amount of settlement after initial liquefaction observed in this study is in agreement with the results reported by Lee and Albaisa [16], where they showed that specimens with relative densities 35% to 50% produced volumetric strains in the range of 0.4% to 0.6%.

The equal amount of volumetric straining in liquified samples can be attributed to the micro-scale interaction of soil particles during a strain-controlled test. The increase in the pore pressure gradually decreases the normal effective stress. In strain-controlled tests, this corresponds to a decrease in bearing capacity of the sample (shear stress degradation, as illustrated in Fig. 4c). On the other hand, in stress-controlled testing the shear stress acting on the sample is held constant. In strain-controlled testing, when the test is terminated at the instant of initiation of liquefaction and the sample is allowed to drain, the relative displacement of the particles (and thus the volumetric strain) does not significantly differ with the amplitude of shear strain. For stress-controlled tests, however, the application of the same amount of stresses at each cycle cause the particles to displace to larger relative distances. This becomes more significant after the development of high excess pore pressures, particles are rearranged and reoriented in a bigger scale in order to resist to the same amount of stress being applied. Reorientation allows denser compaction of particles with large volumetric strain as the developed pore water pressure dissipates.

4.6 Comparison with previous results

A comparison between the post-cyclic loading volumetric strains obtained from this study and those reported by others is presented in Fig. 16. The comparison was done for both loose sand and medium dense sand. To be consistent with previously published results [12, 22, 23] responses from liquefied sand specimens were considered. The volumetric strains of liquefied specimens from this study were found to be in reasonable agreement with those reported by Ueng et al. [23] at both relative density ranges despite the different

testing techniques adopted in each study (i.e., cyclic-strain controlled testing versus shaking table testing). However, the values of volumetric strains suggested by Tokimatsu and Seed [22] and Ishihara and Yoshimine [12] for loose and medium sand are greater than those found as a result of this study. In fact, it is interesting to note that the largest volumetric strains have been reported by Ishihara and Yoshimine [12]. This is reasonable because of the large shear strains (e.g., in excess of 4% and even up to 14%) they considered in their study.

5. Conclusion

An extensive experimental investigation was conducted to study the characteristics of post-cyclic loading settlement of saturated clean sand. Strain-controlled, undrained triaxial tests were performed on reconstituted specimens of sand, and the generation and dissipation of excess pore pressure as a key factor governing the post-cyclic loading settlement were studied to particularly address the influence of consolidation effective stress, induced shear strain, number of loading cycles, initial liquefaction, and relative density.

The results from this study suggest that the potential to develop excess pore pressure decrease significantly with increasing consolidation effective stress in the range between 100 kPa and 400 kPa. However, the level of induced shear strain is also important. The influence of consolidation stress at the small shear strain level of 0.03% was found to be more pronounced at high number of loading cycles while in the medium to large shear strain range an appreciable effect is present from the early cycles of loading. The practical implication of this finding is that small strain-high cycle loading (e.g., moderate seismic events of long duration) has the potential to generate greater excess pore pressure in shallow

layers of soil and the excess pore pressure will decrease with depth. Similarly, the generation of excess pore pressure will decrease significantly with depth under large strain-low cycle loading (e.g., strong seismic events of short duration). The results of the tests in this study also show that the threshold shear strain is affected by the level of consolidation stress. The threshold shear strain at 100 kPa consolidation effective stress is around 0.01% and it was found to increase to around 0.02% at 400 kPa consolidation effective stress. In terms of in-situ conditions, the depth of soil layer and confinement appears to have a significant impact on both the potential to develop excess pore pressure and the level of threshold shear strain.

Post-cyclic loading settlement of sand was found to be influenced by the level of generated excess pore pressure, effective consolidation stress, shear strain, and the number of loading cycles. Appreciable post-loading settlements were recorded from specimens generating excess pore pressure ratio of 0.1 and higher at the 10th loading cycle. On the basis of the excess pore pressure, a clear trend indicating larger settlement with increasing consolidation stress was obtained. However, when correlated to shear strain, the settlement was found to be less influenced by the consolidation stress; the level of induced shear strain seemed to have dictated the post-loading settlement much more than the consolidation stress. The settlement results based on shear strain also indicated a threshold level of $\gamma_t = 0.03\%$ below which little to no post-loading settlement occurs. This threshold level was found not to be affected by the relative density of soil and is higher than that based on the excess pore pressure, which is around 0.01%.

Another factor affecting the post-cyclic loading settlement was the number of loading cycles. For the same level of shear strain or excess pore pressure, specimens subjected to larger number of loading cycles experienced significantly greater settlement in

the post-loading phase. This was the case for both medium and low relative density ranges considered.

Finally, initiation of liquefaction within the soil was found to have somewhat unique influence on the post-loading settlement. When medium dense specimens were loaded to the level of initial liquefaction (i.e., $r_u=1.0$), the volumetric strain in the post-loading phase was found to be on the order of 0.4% regardless of the induced shear strain. This trend was confirmed with another series of specimens that were prepared at a lower relative density. Loose sand specimens were found to experience a slightly larger volumetric strain of 0.45%. Thus, it can be concluded that Ottawa (C-109) sand at a relative density in the range of about 35% to 55% is likely to experience a volumetric strain of around 0.4% to 0.45% with initial liquefaction.

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TABLES

Table 1: Summary of most relevant previous research on settlement of sands.

Testing technique	Soil tested	Nature of testing	Major Findings	Reference
Simple shear	Dry sand	Strain controlled	Important parameters influencing the settlement were: (1) relative density, (2) magnitude of the cyclic shear strain, and (3) number of strain cycles	Silver and Seed, 1971
Triaxial	Saturated sands	Stress controlled	Amount of reconsolidation volumetric strain is a function of (1) grain size (2) relative density, and (3) developed excess pore pressure generation. Larger volumetric strains were observed with increasing confining pressures	Lee and Albaisa, 1974
Simple shear	Saturated sands	Stress controlled	Amount of settlement significantly depends on the induced maximum shear strain and density of soil.	Tatsuoka et al., 1984
No testing (Compilation of previous research results)	-	-	Maximum shear strain is an important factor influencing the probable settlements after liquefaction	Tokimatsu and Seed, 1987
Simple shear	Saturated sands	Strain controlled	Specimens subjected to cyclic straining smaller than threshold shear strain value, $\gamma_{cyc} < \gamma_{tv}$, experienced no permanent volumetric strain, while those tested at shear strains higher than the threshold value settled appreciably.	Hsu and Vucetic, 2004
Biaxial laminar shear box	Saturated sands	Stress controlled	Settlement without liquefaction was generally very small ($\epsilon_v \approx 0.1\%$). Significant volume changes (ϵ_v up to 8%) occurred only when there was liquefaction.	Ueng et al., 2010

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Table 2: List of tests performed under different confining pressures.

Test No.	Post Cons D_r (%)	e	γ_c (%)	B	σ'_c (kPa)	Note	$N_L = N$ to reach $r_u \sim 1.0$
CP1-1	51.4	0.626	0.001	0.97	100	N=50	N/A
CP1-2	51.4	0.626	0.003	0.97	100	N=50	N/A
CP1-3	51.4	0.626	0.005	0.97	100	N=50	N/A
CP1-4	51.4	0.626	0.007	0.97	100	N=50	N/A
CP1-5	51.4	0.626	0.010	0.97	100	N=50	N/A
CP1-6	51.4	0.626	0.030	0.97	100	N=50	N/A
CP1-7	52.7	0.622	0.060	0.96	100	N=50	N/A
CP1-8	52.3	0.623	0.100	0.99	100	N=50	$N_L=50$
CP1-9	51.1	0.627	0.200	0.96	100	N=50	$N_L=30$
CP1-10	51.8	0.625	0.300	0.97	100	N=50	$N_L=10$
CP1-11	53.8	0.619	0.500	0.96	100	N=50	$N_L=5$
CP2-1	55.9	0.612	0.001	0.96	300	N=50	N/A
CP2-2	55.9	0.612	0.005	0.96	300	N=50	N/A
CP2-3	55.9	0.612	0.007	0.96	300	N=50	N/A
CP2-4	56.6	0.610	0.010	0.96	300	N=50	N/A
CP2-5	56.0	0.612	0.020	0.97	300	N=50	N/A
CP2-6	53.8	0.619	0.030	0.98	300	N=50	N/A
CP2-7	56.2	0.611	0.050	0.98	300	N=50	N/A
CP2-8	57.6	0.607	0.060	0.96	300	N=50	N/A
CP2-9	55.0	0.615	0.100	0.98	300	N=50	N/A
CP2-10	55.3	0.614	0.200	0.96	300	N=50	$N_L=50$
CP2-11	52.3	0.623	0.300	0.96	300	N=50	$N_L=35$
CP2-12	54.4	0.617	0.500	0.96	300	N=50	$N_L=10$
CP3-1	55.3	0.614	0.010	0.97	400	N=50	N/A
CP3-2	56.4	0.611	0.030	0.96	400	N=50	N/A
CP3-3	55.2	0.614	0.060	0.98	400	N=50	N/A
CP3-4	55.0	0.615	0.100	0.98	400	N=50	N/A
CP3-5	58.6	0.604	0.200	0.96	400	N=50	$N_L=50$
CP3-6	54.3	0.617	0.300	0.96	400	N=50	$N_L=35$

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Table 3: List of tests performed with varying number of loading cycles.

Test No.	Post Cons D_r (%)	e	γ_c (%)	B	σ'_c (kPa)	Note	$N_L = N$ to reach $r_u \sim 1.0$
N10-1	51.2	0.626	0.02	0.98	100	N=10	N/A
N10-2	53.4	0.620	0.06	0.97	100	N=10	N/A
N10-3	54.9	0.615	0.10	0.96	100	N=10	N/A
N10-4	51.2	0.626	0.30	0.97	100	N=10	$N_L=10$
N10-5	52.3	0.623	0.50	0.98	100	N=10	$N_L=5$
N50-1	51.4	0.626	0.01	0.97	100	N=50	N/A
N50-2	51.4	0.626	0.03	0.97	100	N=50	N/A
N50-3	52.7	0.622	0.06	0.96	100	N=50	N/A
N50-4	52.3	0.623	0.10	0.99	100	N=50	$N_L=50$
N50-5	51.1	0.627	0.20	0.96	100	N=50	$N_L=30$
N50-6	51.8	0.625	0.30	0.97	100	N=50	$N_L=10$
N50-7	53.8	0.619	0.50	0.96	100	N=50	$N_L=5$
N100-1	51.8	0.625	0.10	0.96	100	N=100	$N_L=52$
N100-2	53.0	0.621	0.20	0.96	100	N=100	$N_L=30$
N100-3	51.1	0.627	0.30	0.96	100	N=100	$N_L=10$
N100-4	54.7	0.616	0.50	0.96	100	N=100	$N_L=4$

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Post Cons D_r : Post-consolidation relative density

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Table 4: Initial liquefaction testing of medium-dense sand.

Test No.	Post Cons D_r (%)	e	γ_c (%)	B	σ'_c (kPa)	Note	$N_L = N$ to reach $r_u \sim 1.0$
RU1-1	51.2	0.626	0.10	0.96	100	$r_u=1$	$N_L=50$
RU1-2	51.2	0.626	0.20	0.96	100	$r_u=1$	$N_L=29$
RU1-3	53.7	0.619	0.30	0.96	100	$r_u=1$	$N_L=9$
RU1-4	54.3	0.617	0.50	0.96	100	$r_u=1$	$N_L=5$

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Table 5: Tests performed with low relative densities.

Test No.	Post Cons D_r (%)	e	γ_c (%)	B	σ'_c (kPa)	Note	$N_L = N$ to reach $r_u \sim 1.0$
DR1	39.0	0.663	0.10	0.96	100	$r_u=1$	$N_L=44$
DR2	33.6	0.679	0.20	0.97	100	$r_u=1$	$N_L=10$
DR3	35.8	0.673	0.30	0.97	100	$r_u=1$	$N_L=5$
DR4	34.7	0.676	0.50	0.96	100	$r_u=1$	$N_L=2$
DR5	35.1	0.675	0.02	0.98	100	N=10	N/A
DR6	35.0	0.675	0.10	0.97	100	N=10	N/A
DR7	34.6	0.676	0.30	0.99	100	N=10	$N_L=5$
DR8	34.3	0.677	0.50	0.96	100	N=10	$N_L=2$
DR9	33.5	0.680	0.02	0.96	100	N=50	N/A
DR10	33.6	0.679	0.10	0.96	100	N=50	$N_L=42$
DR11	35.5	0.674	0.20	0.96	100	N=50	$N_L=10$
DR12	35.4	0.674	0.30	0.97	100	N=50	$N_L=5$
DR13	35.6	0.673	0.50	0.96	100	N=50	$N_L=3$

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Table 6: Index properties of clean Ottawa sand.

Index Properties of Ottawa Sand (C-109)	
Specific Gravity	2.66
Maximum Void Ratio	0.78
Minimum Void Ratio	0.48
D ₁₀ (mm)	0.25
D ₃₀ (mm)	0.36
D ₆₀ (mm)	0.45
C _u	1.8
C _c	1.15
USCS Soil Classification	SP

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Fig. 1. Grain size distribution of Ottawa (C-109) sand

Fig. 2. The cyclic triaxial testing system used

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Fig. 9: Post testing volumetric strain versus shear strain for samples subjected to different loading cycles

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Fig. 12: Comparison of pore water pressure generation curves of loose and medium dense specimens.

Fig. 13: Comparison of the numbers of loading cycles required to initiate liquefaction in loose and medium dense specimens

Fig. 14: Post testing volumetric strain versus shear strain from loose and medium dense sand subjected to different loading cycles

Fig. 15: Measured volumetric strains after initial liquefaction for loose and medium dense specimens.

Fig. 16: Comparison of volumetric strains of fully liquefied specimens with previous studies.

FIGURES

Fig. 1. Grain size distribution of Ottawa (C-109) sand

2(a) A photo of the testing system and set-up

2(b) Schematic diagram of the cyclic triaxial system and its components

Fig. 2. The cyclic triaxial testing system used

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Fig. 3. Verification and baseline testing results

4(a) shear strain versus number of loading cycles

4(b) pore water pressure ratio versus number of loading cycles

4(c) shear stress versus number of loading cycles

4(d) shear stress versus shear strain

Fig. 4. Typical results of a strain-controlled cyclic triaxial test ($\gamma_c=0.3\%$; $f=0.2$ Hz; $\sigma_c'=100$ kPa; and $D_r=51.9$)

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5(a) Development of excess pore pressure at $\gamma_c=0.03\%$

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5(b) Development of excess pore pressure at $\gamma_c=0.06\%$

5(c) Development of excess pore pressure at $\gamma_c=0.1\%$

5(d) Development of excess pore pressure at $\gamma_c=0.3\%$

Fig. 5. Development of excess pore pressure at various shear strains (a) $\gamma_c=0.03\%$, (b) $\gamma_c=0.06\%$, (c) $\gamma_c=0.1\%$ and (d) $\gamma_c=0.3\%$.

Fig. 6. Excess pore pressure ratio versus shear strain under different consolidation stresses (N=10).

Fig. 7. Measured volumetric strains versus pore pressures at N=10 for tests performed under different confining pressures.

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887 **Fig. 8.** Measured volumetric strains versus induced shear strains for tests performed under
888 different confining pressures.

Fig. 9. Post testing volumetric strain versus shear strain for samples subjected to different loading cycles.

Fig. 10. Post testing volumetric strain versus pore pressure ratio after 10 cycles.

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Fig. 12: Comparison of pore water pressure generation curves of loose and medium dense specimens.

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