Estimation of reliquefaction considering from soil element tests

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ABSTRACT

Because of water drainage, liquefied sand becomes denser after liquefaction. Therefore, such sand is not expected to liquefy again. However, repeated ground liquefaction has been reported from several earthquake events. We have considered some reasons for frequent reliquefaction: (1) the upper ground remains in a loose condition after liquefaction and (2) increased pore water pressure remains during subsequent earthquake events. This study investigated reliquefaction mechanisms using triaxial tests with bender elements (BE) and local small strain measurements (LSS) to explore other reasons for reliquefaction. The experimentally obtained results demonstrate that initial shear modulus $G_0$ is insensitive to specimens with shear and liquefaction histories, although the liquefaction strength varies considerably according to the stress and liquefaction histories. The LSS test results indicate that the elastic strain region of sand shrinks according to the liquefaction history while maintaining almost unchanged $G_0$. Furthermore, the sand particle orientation was evaluated from 2D images obtained using optical digital microscopy.

Keywords: element test, cyclic triaxial test, liquefaction, sand

1 INTRODUCTION

After liquefaction of sandy ground, the ground becomes denser during the dissipation of increased pore water pressure. Therefore, sand deposits having a liquefaction history are deemed as more robust against liquefaction than the original deposits. Nevertheless, reliquefaction has occurred frequently in the same ground during the 1983 Nihonkai-chubu Earthquake (Yasuda and Tohno, 1998), the 2010–2011 Christchurch earthquakes (Cubrinovski et al., 2012), and the 2011 Great East Japan Earthquake (Yasuda et al., 2012). Reports describe slight liquefaction occurring during the main shock but heavy liquefaction during aftershocks, which suggests that liquefied ground is more susceptible to liquefaction than the original ground.

Studies of several kinds have been conducted to assess reliquefaction phenomena. First are model tests using shaking tables: a loose state persists in the upper part of the liquefied layer after liquefaction in spite of increased density in lower parts (Sasaki and Taniguchi, 1982; Yamaguchi et al., 2008). Second are soil element tests: large residual deformation after liquefaction can reduce resistance against liquefaction (Finn et al., 1970; Yamada et al., 2010). Results of such reports have described the strain-induced anisotropy attributable to liquefaction characteristics. Third are numerical analyses (e.g. Morikawa et al., 2012). Such studies have examined increased pore water pressure generated during the main shock, which persists to some degree during aftershocks. This remnant pore water pressure reduces the liquefaction resistance.

Shear wave velocity $V_s$ has been used to assess liquefaction properties in laboratory soil tests and in field investigations because it can be found from simple, non-destructive tests. When $V_s$ is measured from a surface wave survey, the data are spatially averaged using an inversion process. Although $V_s$ has considerable uncertainty for that reason, the effectiveness of $V_s$ has been shown to evaluate ground improvement (Stokoe et al., 2014). Hatanaka et al. (1997) reported that $V_s$ is effective for the estimation of liquefaction resistance. However, Toyota et al. (2012) have reported no great difference between liquefied and non-liquefied areas in terms of the shear wave velocity distribution evaluated using the surface wave method.

The main objective of this study is elucidation of decreased liquefaction resistance during reliquefaction. Bender element (BE) tests and local small strain (LSS) tests, which are embedded in the triaxial apparatus,
were conducted to evaluate changes of the liquefaction resistance and the mechanical properties in a small strain range related to prior liquefaction. Furthermore, orientations of sand particles on the side surfaces of the specimens before and after liquefaction were compared using images obtained from optical digital microscopy.

2 EXPERIMENTS

2.1 Soil and specimen preparation

Toyoura sand, a Japanese standard sand having angular to sub-angular particle shape with mineralogy almost entirely comprising quartz, was used for this study. Its physical properties are mean particle diameter $D_{50}$ of 0.2 mm, uniformity coefficient $U_c$ of 1.48, maximum void ratio $e_{max}$ of 0.990, and minimum void ratio $e_{min}$ of 0.597. The specimen, with 50 mm diameter and 120 mm height, was prepared using air pluviation ($D_r=40\%$) for long-term consolidation tests and wet tamping ($D_r=40\%, 60\%, \text{ and } 75\%$) for liquefaction tests (Ishihara, 1993).

2.2 Testing methods

Test procedures including the LSS test were reported by Toyota and Takada (2017). However, for long-term consolidated specimen of the present study, the effective confining pressure $\sigma_0$ of 50 kPa was used. Back-pressure was not applied so that experiments would be able to continue even during a power failure.

3 COMPARISON BETWEEN LONG-TERM CONSOLIDATED SPECIMEN AND SPECIMEN WITH STRESS HISTORY

To assess the aging effects on liquefaction, loose specimens of $D_r=40\%$ were consolidated for a long period using the triaxial apparatus. Low confining stress of $\sigma_0=50$ kPa and a lack of back-pressure were chosen because consolidation was thereby continued even when the electric power supply was cut off.

For easy production of a specimen with similar mechanical properties to a long-term consolidated specimen, the specimen was subjected to stress history (Tatsuoka et al., 1988). In this study, an overconsolidation ratio (OCR) of 2 was chosen for the stress history. Then, the liquefaction strength and deformation characteristics were compared between the specimens after conducting long-term consolidation and the specimens with stress history.

3.1 Representative liquefaction behavior

Variables used are explained in the “NOTATION”. Figure 1 presents representative results of stress–strain and stress paths for liquefaction tests in 1-day (Fig. 1(a)), 126-day consolidated specimens (Fig. 1(b)), and OCR=2 specimens (Fig. 1(c)). The number of cycles to reach liquefaction was clearly greater for the long-term consolidation of 126 days. After reaching cyclic mobility, the shear strain extends greatly, which represents loose sand behavior and which is similar to the specimen consolidated for 1 day. B-value was checked after the liquefaction test. The measured B-value indicated 1 even after the long-term consolidation.

The stress–strain relations in the long-term specimen (Fig. 1(b)) and overconsolidated specimen (Fig. 1(c)) are mutually similar. Regarding the stress path, the increased pore water pressure in the first cycle is more readily apparent for the long-term consolidated
specimen than for the overconsolidated specimen. Although the cyclic undrained behavior is not perfectly equal between the long-term consolidated specimen and OCR=2 specimen, it is similar in the point of slow generation of pore water pressure. Therefore, in liquefaction tests, overconsolidated specimen might be substituted to long-term consolidated specimen, which is time-consuming for specimen preparation.

a long-term consolidated specimen should be used in the laboratory for the assessment of ground liquefaction. However, those are, respectively, high-cost and time-consuming works. Therefore, the liquefaction phenomenon is evaluated here using specimens with a stress history, which has similar mechanical properties to those of long-term consolidated specimens.

Figures 3(a)–3(c) portray differences of cyclic loading behaviors in specimens of similar density ($D_r=40\%$) for the stress history of “OCR=1 (no stress history),” “OCR=6 (overconsolidated),” and “OCR=6 + liquefaction (reliquefaction).” Stress–strain relations and stress paths of the specimen without stress history are depicted in Fig. 3(a). The mean effective principal stress $p'$ decreases gradually (pore water pressure increases) through cyclic loading. Subsequently, $p'$ decreases suddenly and reaches zero at $p'=40\, \text{kPa}$. At that time shear strain also expands to almost 5% because of the loose state of $D_r=40\%$. Regarding the specimen of OCR=6 stress history (Fig. 3(b)), more cycles are necessary to trigger liquefaction in spite of the similar specimen density ($\Delta e=0.006$ by OCR=6). Therefore, the stress history is effective to increase the liquefaction resistance, even for pure sand. However, extension of the shear strain after reaching the cyclic mobility is large, even in the OCR=6 specimen, which behaves just as it is in loose sand. Therefore, heavy damage will be triggered when liquefaction occurs because of its low resilience against deformation after reaching cyclic mobility.

This specimen, in which $p'$ became zero (i.e. liquefaction) during cyclic loading, was drained and was consolidated isotropically at $p'=100\, \text{kPa}$. Then cyclic undrained loading was applied to the specimen again (reliquefaction). Results for this specimen are presented in Fig. 3(c). It is interesting that fewer cycles are necessary to reach liquefaction in spite of the slight increase of the specimen density during drainage after the first liquefaction ($\Delta e=0.02$ by the first liquefaction).

Therefore, pre-liquefaction dampened the effect of stress history, which increases the liquefaction resistance. The number of cycles necessary to reach liquefaction is similar to that in the case without stress history (Fig. 3(a)), but the increase of pore water pressure in extensional stress becomes much greater than that in compression stress at the first cycle. This fact implies that the orientation of sand particles changes during the pre-liquefaction. Another difference is that the shear strain expansion that occurs after reaching cyclic mobility is more gradual than in the case depicted in Fig. 3(a). One reason related to the gradual expansion of shear strain is regarded as densification of the specimen caused by drainage after pre-liquefaction. Those figures show that the liquefaction resistance varies complexly according to the stress and liquefaction history, with almost no density variation.
specimens having various cyclic undrained loading histories. First, the specimens were isotropically overconsolidated at OCR=6. The liquefaction strengths of OCR=1 and 6 are presented in the figure for comparison. The specimens were subjected to cyclic undrained loading to a certain excess pore water pressure ratio, $\Delta u/\sigma_c$. These specimens were again consolidated isotropically at $p'=100$ kPa. Finally, CSR of specimens was measured using the specimens created as described above.

The CSR on the specimens for which $\Delta u/\sigma_c = 0.5$ and 0.7 are almost identical to those of the specimen for which OCR=6. However, for specimens for which $\Delta u/\sigma_c$ as high as 0.9 and 1, the CSR decays toward that of the specimen for which OCR=1. The specimen for which $\Delta u/\sigma_c$ is greater than 0.9 exhibits cyclic mobility during the cyclic undrained loading history. Therefore, the large shear strain is regarded as eliminating the increase of CSR induced by the stress history of OCR=6. When the cyclic undrained history continues to large shear strain (double amplitude (D.A.) axial strain =5%), the CSR decreases to less than that of the specimen of OCR=1, which indicates that a weaker particle fabric is created during liquefaction than that with the OCR=1 specimen produced using the wet tamping method.

Figure 4(b) depicts the same effects on OCR=1 specimens as those on OCR=6 specimens in Fig. 4(a). The CSRs on the specimens of $\Delta u/\sigma_c = 0.5$ and 0.7 are higher than that on OCR=1 specimens despite maintaining almost identical liquefaction strength in the OCR=6 specimens (Fig. 4(a)). Therefore, the cyclic undrained loading history, which is the small strain

4.2 Variation of liquefaction strength

Figure 4(a) depicts the cyclic resistance curve of
history before liquefaction, raises the liquefaction strength (CSR). However, for the same reason as that of the OCR=6 specimen described earlier, specimens for which Δu/σ₀ reaches 1 have CSRs that decay toward smaller values than those of the OCR=1 specimen.

Mullis et al. (1977) and Ladd (1977) described that the change of fabric was induced from the sample preparation and that its fabric affected the shear and liquefaction behavior. Toyota and Takada (2017) demonstrated that CSR of the specimen produced by wet tamping is stronger than that produced by dry deposition. Wet tamping method might create a stronger particle structure against liquefaction because the tamping history remains before liquefaction tests.

As an additional reason for the lower liquefaction strength during re liquefaction, it is noteworthy that the cross section on the top part of the specimen becomes slightly smaller than that on the bottom part because of "necking," especially after the occurrence of large cyclic shear strain. Actually, the differences in cross sections of the specimen between top and bottom are scarcely visible, even in the case of Δu/σ₀ =1, except for the case of D.A.=5%.

To remove or minimize the necking effect, tests of the same kinds were conducted using denser specimens (D₆=60% and D₆=75%). The initial cyclic undrained loading was terminated when Δu/σ₀ reached 0.9, which minimized the “necking” effect. The changes of the number of cycles to liquefaction through the processes of OCR=1, OCR=6, and OCR=6 + liquefaction are presented in Fig. 5. The liquefaction curves of D₆=40%, 60%, and 75% are made from CSR data of the specimens of OCR=1. The CSRs of the specimens of OCR=6 move to the right side. This increase is equivalent to the CSRs of denser specimens by 15–20% in relative density. However, these increases of cycles to liquefaction are lost in specimens that had liquefied once. In other words, the sufficient increase of pore pressure or large deformation might erase the stress-history-related memory of the specimen.

Fig. 5. Changes of the number of cycles to liquefaction through the stress and liquefaction histories.

4.3 Shear modulus

Precise local small strain (LSS) measurements were taken to clarify the mechanism of the change of liquefaction resistance on the specimens of OCR=1, OCR=6, and OCR=6 + liquefaction. However, in relation to the specimen of D₆=40%, it was difficult to conduct the LSS test after pre-liquefaction because large deformation occurred during pre-liquefaction. Therefore, only results of OCR=1 and OCR=6 specimens were obtained in the specimen of D₆=40%.

Figure 6(a) presents a comparison of secant shear moduli under drained conditions between OCR=6 and OCR=1 specimens. The difference between the two specimens appears to be that the value of initial shear modulus is maintained up to shear strain of about 0.003% in the OCR=6 specimen, but the value of the
Initial shear modulus starts to decrease at shear strain of about 0.001% in the OCR=1 specimen. Many researchers (e.g., Vucetic and Dobry, 1991; Jardine, 1992; Vucetic, 1994) have assessed the concept of threshold shear strain as representing a change in soil mechanical behavior. Results from Fig. 6(a) also indicate that the threshold shear strain will expand with increased OCR=6 in the sand. This threshold shear strain might be readily apparent in OCR=6 specimen (Fig. 6(a)), but it is not readily apparent in the OCR=2 specimen (Fig. 2). The secant shear modulus becomes greater only at shear strains of 0.005% – 0.07% in OCR=2 specimen. This behavior in the small strain range is expected to produce increased liquefaction resistance without increasing initial shear modulus $G_0$.

Figures 6(b) and 6(c), respectively present secant shear moduli under drained conditions for medium-dense ($D_t=60\%$) and dense ($D_t=75\%$) specimens. The initial shear moduli are almost identical in all cases of the same densities. The BE test results also indicate almost identical $G_0$, although a slightly greater $G_0$ was found for OCR=6 + liquefaction because slight densification is induced by drainage after liquefaction. The difference appears in the expansion of elastic strain region where the initial shear modulus is maintained as a constant value. Although the elastic strain region expands with stress history of OCR=6, degradation of the secant shear modulus starts from smaller shear strain in the specimen that had undergone pre-liquefaction.

The elastic strain regions are, respectively, about 0.003% and 0.005% in the OCR=1 specimens of $D_t=60\%$ and $D_t=75\%$ specimens. This difference in the elastic strain region might result from compaction during specimen preparation (wet tamping). However, the elastic strain region shrinks to about 0.001% by experiencing pre-liquefaction. The secant shear modulus degrades sharply with increased shear strain. Although the CSR is almost identical between OCR=1 and OCR=6 + liquefaction (Fig. 5), large difference appears in the secant shear modulus (Fig. 6). Those changes of shear properties in a small strain range are expected to decrease liquefaction resistance even with a slight increase of density after liquefaction.

5 SAND PARTICLE ORIENTATION

A difference appeared in degradation of secant shear modulus with shear strain, which is a main reason why liquefaction resistance varies during stress and liquefaction history. However, it remains uncertain why different degradation in secant shear modulus is engendered. Therefore, an image processing technique was used to qualify the particle orientation induced by stress and liquefaction histories. The pore water in the specimen was drained after triaxial tests. That specimen can stand by itself. Then the membrane that covered the specimen was cut vertically and removed carefully.

Next, 2D images of the specimen’s side surface were taken at different positions of the specimen using an optical digital microscopy to ascertain the orientation (inclined angle) of sand particles. Sand particles of the side surface were removed using a vacuum cleaner. Then, inside of the specimen also examined. However, obvious difference has not been observed between the side surface and the inside specimen. Therefore, total counting number including both the side surface and the inside specimen was used for the analysis.

Figures 7(a) and 7(b) respectively portray images and measurements of the inclined angle of sand particles. The inclined angle was defined as the direction of longer particle diameter rotated counterclockwise from the horizontal line. Its values ranged from 0 deg to 180 deg.

5.1 Comparison between OCR=1 and OCR=6 specimens

First, stress history effects (OCR=6) on the sand particles’ directions were examined. The distribution of inclinations of sand particles was depicted in a bar
5.2 Effects of liquefaction history

Next, the effect of liquefaction history on the sand particles’ directions was investigated. Figures 9(a) and 9(b) respectively exhibit the distributions of sand particles’ directions for $D_t=75\%$ specimens with OCR=1 and OCR=6 + liquefaction. Two peaks express wet tamping characteristics just as $D_t=75\%$ specimen before liquefaction. However, after liquefaction (Fig. 9(b)), sand particles tend to stand vertically because of an increased number of particles around 90 deg. The reason can be considered that upward infiltration flow will be generated during re-sedimentation induced by liquefaction. By contrast, this phenomenon was not able to be observed for the denser specimens, which are the $D_t=60\%$ and $D_t=75\%$ specimens. However, clear differences appear in the degradation of secant shear modulus in the $D_t=60\%$ and $D_t=75\%$ specimens (Fig. 6). More important and predominant factors will exist to explain the different degradation of the secant shear modulus.

6 CONCLUSIONS

The assessment of reliquefaction considering aging effects on liquefaction has become a crucially important topic since the 2011 Great East Japan Earthquake. Therefore, this study treated resistance against reliquefaction assisted by cyclic triaxial tests. Furthermore, the secant shear modulus of the specimen was measured respectively using the BEs and LSSs embedded in the triaxial test to ascertain its relation with liquefaction resistance.

The main results of this study are presented below.
1. Stress history and long-term consolidation (ageing
effect) both increase the liquefaction resistance of sand. However, once sand is liquefied, it behaves the same as sand without stress history.

2. Stress history effects resemble ageing effects in terms of mechanical properties. However, details show that although the linear elastic shear strain region mainly expands due to ageing effects, the secant shear moduli at 0.005% – 0.07% shear strain, which is greater strain than the linear elastic shear strain, becomes much greater due to stress history (OCR=2).

3. Liquefaction history decreases liquefaction resistance because of loss of the stress history effect. Preloading is effective as a countermeasure against liquefaction. However, once liquefaction occurs, heavy damage will occur during an earthquake just as it would in ground without countermeasures. Moreover, the ground becomes susceptible to liquefaction in subsequent earthquakes because this preloading is cancelled by prior liquefaction.

4. The elastic strain region shrinks through liquefaction while almost maintaining G0. When the specimen has a liquefaction history, the secant shear modulus degrades sharply with increased shear strain. This rapid degradation is induced by prior liquefaction, which is regarded as reducing liquefaction resistance.

5. Vr-based method to estimate liquefaction resistance is inaccurate because of insensitive of Vr to the specimen having shear and liquefication history.

6. Sand particles tend to stand vertically after liquefaction in a loose state. However, that tendency was not apparent in medium-dense and dense sands.

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NOTATION

G: Secant shear modulus
G0: Initial shear modulus
p*: Mean effective stress = (σ1+2σ3)/3
q: Deviator stress = σ1−σ3
Δu: Excess pore water pressure
εa: Axial strain
εr: Radial strain
εs: Shear strain =2/3(εa−εr)
σa: Axial stress
σr: Radial stress
σ0*: Effective confining stress