Cyclic deformation-strength evaluation of compacted volcanic soil subjected to freeze–thaw sequence

Satoshi Matsumura a,*,1, Seiichi Miura b, Shoji Yokohama b, Shima Kawamura c

aFoundations Group, Port and Airport Research Institute, 3-1-1, Nagase, Yokosuka 239-0826, Japan
bFaculty of Engineering, Hokkaido University, Kita 13 Nishi 8, Kita-ku, Sapporo 060-8628, Japan
cGraduate School of Engineering, Muroran Institute of Technology, 27-1 Mizumoto-cho, Muroran 050-8585, Japan

Received 4 October 2013; received in revised form 18 September 2014; accepted 20 October 2014
Available online 1 January 2015

Abstract

The aim of this paper was to clarify the effect of freeze-thawing on the cyclic shear behavior of a crushable volcanic soil that is widely deposited in Hokkaido, Japan. Using a triaxial apparatus, which was newly developed to simulate freeze–thaw cycles and cyclic loading, a series of cyclic triaxial tests was performed on compacted volcanic soils subjected to freeze–thaw cycles. The experimental results showed that freeze-thawing leads to a significant decrease in the liquefaction strength of densely-compacted volcanic soils. The decrement in liquefaction strength depended strongly on the behavior of the excess pore water pressure during cyclic loading. In addition, according to the variation in cyclic deformation behavior induced by the freeze–thaw sequence, it was anticipated that the freeze-thawing would be related to the change in the soil fabric of the specimens. However, the specimens compacted at a lower dry density seemed to be inconsistent with the above; i.e., the liquefaction strength became almost the same regardless of whether or not the specimens were exposed to the freeze-thaw sequence. Thus, it was concluded that the freeze-thaw sequence can degrade the dynamic properties of volcanic soil, but that such influences ought to be incorporated into a correlation with the dry density.

© 2015 The Japanese Geotechnical Society. Production and hosting by Elsevier B.V. All rights reserved.

Keywords: Freeze–thaw sequence; Cyclic deformation-strength characteristics; Liquefaction; Crushable volcanic soil; Compaction (IGC: D07/D08/D09)

1. Introduction

The island of Hokkaido, in Japan, is located in a cold region where varieties of volcanic soils are widely deposited. Therefore, the volcanic soils have been used as construction materials. However, recent large earthquakes, such as the Tokachi-oki earthquakes (1968 and 2003), have caused serious liquefaction-induced damage to a great number of soil structures and foundations composed of these volcanic soils with particle crushing. For example, Kazama et al. (2006) reported that liquefaction-induced damage was frequently observed in embankment structures constructed by filling in river channels or valleys.

The authors have similarly investigated the mechanical properties of the crushable volcanic soils deposited in Hokkaido. In previous studies, the following facts were derived: (1) particle crushing becomes remarkable in the stress path for which the effective mean principal stress increases, which triggers stronger contractive behavior and a significant decrease in shear strength (Miura et al. 2003), (2) dynamic properties, such as a small strain shear modulus and low
liquefaction strength, basically differ from those of Toyoura sand as a non-crushable soil (Yagi and Miura, 1998; Sahaphol and Miura, 2005) and (3) the N-value in the in-situ testing tends to be underestimated due to the effect of particle breakage (Miura et al. 2003). Thus, the geo-mechanical peculiarities attributed to particle breakage have been discussed here through a series of laboratory and in-situ testing.

On the other hand, the climatic conditions of Hokkaido probably cause changes in the mechanical performance of soil structures. It is a well-known fact that slope failures frequently occur in the snowmelt season and that differential settlements result from frost heaving. In terms of freeze-thawing in soil mechanics, numerous researches have addressed the effect of freezing on the liquefaction strength of clean sands, or soils mixed with sands and finer fractions, in order to discuss the validity of an in-situ freezing method as high-quality sampling (Walberg, 1978; Yoshimi et al., 1978; Ishihara et al., 1978; Singh et al., 1982; Goto, 1993). From a geotechnical standpoint, for disaster prevention in cold regions, attempts have been made by many researchers to identify the effects of freeze-thawing on the mechanical characteristics, e.g., hydraulic conductivity with glacier tills (Kim and Daniel, 1992; Konrad, 2010; Viklander, 1998), rainfall-induced slope failure behavior in laboratory modeling tests (Kawamura and Miura, 2013a, 2013b), monotonic deformation-strength characteristics (Ishikawa and Miura, 2011) and cyclic deformation properties with a small level of strain (Yamaki et al., 2009). However, cyclic deformation-strength characteristics, including the liquefaction behavior of volcanic soils subjected to the freeze–thaw sequence, are hardly known.

In this study, a new triaxial apparatus, which is capable of simulating freeze–thaw cycles and cyclic loading, was developed. Using this apparatus, a series of cyclic triaxial tests was performed on a compacted volcanic soil subjected to freeze–thaw cycles. In this paper, cyclic loading was applied to volcanic soils under fully saturated conditions, after freezing and thawing, to examine the fundamental results of the cyclic deformation-strength characteristics.

2. Testing method

2.1. Triaxial testing apparatus with freezing and thawing system

Figs. 1 and 2 and Photograph 1 show the schematic diagrams and view of a triaxial apparatus newly developed in this study, respectively. The main features are as follows: (1) to simulate a soil subjected to a freeze–thaw sequence by controlling the temperature and (2) to conduct liquefaction tests by applying stress-controlled cyclic loads.

In order that a sequence of freeze-thawing and cyclic loading be carried out with temperature and stress conditions controlled arbitrarily, a cooling system was equipped with a conventional cyclic triaxial apparatus, as shown in Fig. 2. An acrylic cylindrical cell (frost heave cell), fixed to the soil specimen and covered with a membrane 0.3 mm in thickness, constrains the lateral displacement throughout the freezing and thawing to simulate a one-dimensional freeze-thawing phenomenon. The maximum of the three temperature-controlled baths (T-C baths), used to circulate non-freezing fluids independently through a cap, a pedestal and an outer cell, is capable of one-dimensionally cooling the specimen set up in the triaxial apparatus at a desired temperature gradient. In this study, as shown in Fig. 2, T-C bath (B) operates to cool the specimen from the bottom end through the pedestal, while T-C bath (A) keeps the cell water replaced with the non-freezing
fluid constant at a temperature of 0 to 1 °C. At the same time, a set of platinum resistance thermometers, installed in both the cap and the pedestal, measures the temperature at the top and bottom ends of the specimen. Moreover, the amount of drainage during freezing and thawing can be monitored by a double-tube burette connected to the low-capacity differential pressure transducer (A) seen in Fig. 1. The axial displacement is monitored using an external displacement transducer (dial gauge), and its stress is measured using a load cell inside of the outer cell.

On the other hand, this apparatus was designed for unsaturated soil testing by employing the appropriate equipment, namely, (1) a set of transducers and pressure regulators to independently measure and control pore water pressure $u_w$ and pore air pressure $u_a$ of the specimen, (2) an inner cell and differential pressure transducer (B), shown in Fig. 1, to measure the volume change of the specimen and (3) a pedestal with a ceramic disk, which is free to be replaced with one setting in a porous plate used in this study. Some trial testing on unsaturated soils using these facilities would be required in the future.

Fig. 3 indicates the flow for each process of the testing. As mentioned above, the developed triaxial testing apparatus is expected to simulate the soil behavior in cold regions. Therefore, it is technically possible to prepare the specimens by the desired freeze–thaw sequence under a given moisture content prior to loading. However, it is still difficult to accomplish the identification of the individual effects of such factors as the degree of saturation, temperature and stress conditions. In this study, in order for the fundamental results to reveal the influence of freeze-thawing on the cyclic shear behavior, all of the specimens with or without a freeze–thaw cycle were fully saturated prior to cyclic loading. Hence, the results could be understood to reflect a critical state against a seismic capacity of the freeze-thawed soil.

2.2. Testing material

A coarse-grained volcanic soil was adopted in this study, which is deposited in the Komaoka district of Sapporo City in Hokkaido, as shown in Fig. 4 (referred to as ‘K soil’ in this paper). K soil belongs to a pumice flow deposit provided by Shikotsu caldera, which is classified into ‘Shikotsu pumice flow deposit: Spfl’ (Miura et al., 2005). Fig. 5 shows the particle images of the K soil in each section. The original oven-dried K soil seems to be composed of soil particles of various sizes. The coarse particles, 4.75 to 9.5 mm, were taken after being washed off to observe the surface of each particle. For microscopic frameworks in Fig. 5(III–V), taken by an electronic microscope, the degree of porosity differs in each grain, and these particles mix into the K soil well. The porous materials are eventually recognized as playing a key role in characterizing the mechanical behavior of crushable volcanic soils (Nakata and Miura, 2007).
Table 1 and Fig. 6 show the index properties and grain size accumulation curve for the K soil, respectively. Although the K soil contains negligibly few particles larger than 9.5 mm, the particles were removed to enable consistency regarding the limitation in triaxial specimen size prior to the specimen preparation. As shown in Table 1 and Fig. 6, the finer content and uniformity of the K soil were 31% and 48, respectively. However, volcanic soils, including K soil, Spfl, which have been utilized in residential embankments, have repeatedly liquefied due to earthquakes (e.g., Nansei-oki (1993), Toho-oki (1994) and Tokachi-oki (1968 and 2003)). Based on the liquefaction damage to compacted embankments, this study focuses on the cyclic behavior of K soil under certain compaction conditions on the compaction plane that was obtained from the compaction test by the A-c method (The Japanese Geotechnical Society, 2009a,d), as shown in Fig. 7. In the compaction test, the rammer that has 24.5 N and a falling height of 300 mm, and the 1.0 \times 10^{-3} \text{ m}^3 mold with a 0.1-m diameter, were used. The number of compaction blows reached 75, which means compactive effort equal to 550 kJ/m$^3$. Furthermore, in this method, the oven-drying of the soil prior to controlling the moisture content and reusing the soil, was not permitted. As a result, the optimum water content, $w_{\text{opt}}$ (%), and maximum dry density, $\rho_{\text{dmax}}$ (Mg/m$^3$), of the K soil were 40.5% and 1.059 Mg/m$^3$, respectively. The degree of compaction $D_c$ (%) was calculated on the basis of $\rho_{\text{dmax}}=1.059 \text{ Mg/m}^3$.

### Table 1: Index properties of K soil.

<table>
<thead>
<tr>
<th>Sample name</th>
<th>$\rho_s$ (Mg/m$^3$)</th>
<th>$I_p$</th>
<th>$D_{50}$ (mm)</th>
<th>$U_c$ (%)</th>
<th>$F_c$ (%)</th>
<th>Compaction (A-c)</th>
<th>$w_{\text{opt}}$ (%)</th>
<th>$\rho_{\text{dmax}}$ (Mg/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>K soil</td>
<td>2.512</td>
<td>NP</td>
<td>0.20</td>
<td>48</td>
<td>31.0</td>
<td>40.5</td>
<td>48</td>
<td>1.059</td>
</tr>
</tbody>
</table>

2.3. Specimen preparation for cyclic triaxial test

A compaction mold for the triaxial specimens was newly manufactured; it is 70 mm in diameter and 230 mm in height.
including a collar. It is possible to split the mold longitudinally so as not to disturb the specimen when it is taken out after compaction. Using the mold, cylindrical specimens were prepared by the following compaction method. The soil was moisture-controlled at $w_{opt}$ of 40.5% and compacted a total of 55 times in five layers by the same rammer as that used for the aforementioned compaction test. The mass of wet soil required compaction fixed at 200 g per layer. The initial dry density $\rho_{d0}$ (Mg/m$^3$) and the initial degree of compaction $D_0$ (%) were 1.027 Mg/m$^3$ and 97.0% in average (variation in ±3%), respectively. After compaction, the top end of each specimen was flattened to be 140 mm in height. In addition, in order to evaluate the effect of the difference in dry density on the experimental results, cyclic triaxial tests were performed on the $K$ soil specimens, which were compacted 16 and 32 times in four layers, respectively. The dry densities of the specimens were 0.940 and 0.988 Mg/m$^3$ in average, and corresponded to 88.8% and 93.3% for $D_0$.

Fig. 7 illustrates an effective stress path for specimens with the freeze–thaw sequence after setting up the triaxial apparatus. Furthermore, Fig. 9 presents the corresponding boundary conditions at each process shown in Fig. 8. The specimen was saturated so as to have more than a degree of saturation $S_r$ of 80%, which was referred to as “test method for frost heave prediction of soils” (The Japanese Geotechnical Society, 2009c) by the vacuum procedure for saturating (Rad and Clough, 1984) under the effective confining pressure $\sigma_{c}^{'}$ of 20 kPa. After that, the specimens progress to a freeze–thawing process, where the stress-strain state is described by (b) in Figs. 8 and 9. Throughout this process, the specimens maintain the stress state under the effective axial stress $\sigma_{a}^{'}$ of 20 kPa instead of $\sigma_{a}^{'}=20$ kPa. The specimens were allowed to deform one-dimensionally because of the frost heave cell restraining radial strain $\varepsilon_r$. Therefore, if $K_0$ denotes the coefficient of earth pressure at rest, an effective mean principal stress $p'$ during freeze–thawing should be represented as $p'=(1+2K_0)\sigma_{a}^{'}/3$. The specimens without the freeze–thaw sequence, or after thawing, are applied with back pressure to saturate while keeping $\sigma_{a}^{'}=20$ kPa. In this study, prior to cyclic loading, isotropic consolidation was performed under $\sigma_{a}^{'}=50$ kPa for enough time to finish the primary consolidation within 24 h. All of the specimens were fully saturated with a $B$ value of 0.95 or more. Afterwards, the stress-controlled cyclic loads were applied with the frequency of approximately 0.005 Hz until double-amplitude axial strain $DA$ reached 5% under undrained conditions with a constant total stress. Then, the $DA$ value was equal to the sum of the compression and extension strain described by $\varepsilon_{a \text{ comp.}}$ and $\varepsilon_{a \text{ ext.}}$, respectively, in Fig. 9(d). In this study, regarding the stress states, the freeze–thaw process was conducted under the $K_0$ consolidation state, while cyclic loading was applied under the isotropic consolidation state. The former is because it simulates the actual ground one-dimensionally freeze-thawed, as adopted in the “test method for frost heave prediction of soils” (JGS 0171-2009). On the other hand, the latter is the method for cyclic triaxial tests which have often been employed in general (e.g., The Japanese Geotechnical Society, 2009b).

In addition, in order to examine the degree of particle breakage, a sieve analysis was performed on the specimens after testing. In this study, the increment in finer content smaller than 75 μm, $\Delta F_c$ (%), defined in Eq. (1), is adopted as a parameter to identify the degree of particle breakage (Miura and Yagi, 1996).

$$\Delta F_c = F_c' - F_c$$  \hspace{1cm} (1)

where $F_c'$ and $F_c$ are the finer contents after and before testing, respectively.

2.4. Freeze-thawing process

The specimen with the frost heave cell was partially saturated in the range of $S_r$ higher than 80% in the triaxial cell, as mentioned above. Afterwards, the cell water was replaced with the non-frozen fluid and circulated and cooled by the T-C bath (A) (see Fig. 2) until each temperature of the top and bottom ends of the specimen was maintained at 0 to 1 °C. On the other hand, the effective axial stress $\sigma_{a}^{'}$ was kept constant at 20 kPa throughout freezing and thawing. In order to avoid supercooling, thermal shock was applied at the bottom end of the specimen prior to freezing. Subsequently, a freezing
front rose from the bottom end up through the pedestal at a certain cooling velocity by the T-C bath (B) (see Fig. 2). Then, de-aired water was supplied and then drained from the top end of the specimen through the double-tube burette, which measures the amount of drainage during freezing and thawing.

The freezing velocities \( V_f \) (mm/h), which are calculated in Eq. (2), were in the range of 0.45–1.26 mm/h for all of the freeze-thawed specimens.

\[
V_f = \frac{\Delta H}{t} \tag{2}
\]

where \( \Delta H \) (mm) and \( t \) (h) are the maximum amount of axial displacement induced by freezing and the required time to cause the corresponding \( \Delta H \) value, respectively. After ensuring that the axial displacement due to freezing became constant, both of the T-C baths were turned off in order to thaw the specimens. After 24 h at a temperature of around 20°C, the frost heave cell was immediately removed under the effective confining stress of 20 kPa by providing vacuum pressure through the specimen. Finally, the specimens which had experienced the freeze-thawing sequence, as well as the non-frozen ones, were fully saturated and consolidated prior to cyclic loading.

3. Experimental results and discussions

3.1. Mechanical behavior of specimens subjected to freeze-thawing

Figs. 10 and 11 show the time histories of axial strain \( \varepsilon_a \) (%), drainage \( \Delta V \) (ml) and each temperature (°C) during freeze-thawing with the specimens compacted at the initial degree of compaction \( D_{ci} \) of 97.0 and 88.8%, respectively. Then, the temperature histories applied to each specimen were almost exactly the same. It is noted that the positive values in axial strain \( \varepsilon_a \) and drainage \( \Delta V \) mean the compression strain and to drain, respectively. Furthermore, because the specimens were laterally restrained so that the one-dimensional freeze–thaw phenomenon could be achieved, the axial strain \( \varepsilon_a \) due to freeze–thawing corresponded to the volumetric strain. From the behavior of the densely compacted specimen, shown in Fig. 10, it is seen that the specimen continues significant dilation, and causes a freezing expansion rate \( \varepsilon_f \) (%) of 4.9% as the temperature of the pedestal drops, and is kept constant. The \( \varepsilon_f \) value is calculated by Eq. (3), namely,

\[
\varepsilon_f = \frac{\Delta H}{H_i} \tag{3}
\]

where \( H_i \) is the specimen height before freezing. Such dilation advances as the specimen absorbs water. After showing a subtle discharge at the beginning of freezing, the water tends to be continually provided into the specimen up to 10 ml. As a result, the specimen remains at a certain quantity of axial strain \( \varepsilon_a \) with the water absorbed even after the specimen has completely thawed. Meanwhile, the specimen which is loosely-compacted seems different in the freeze–thaw behavior (see Fig. 11). Namely, the specimen rapidly effuses water with swelling due to freezing for about the first 50 h, and then shifts to drainage behavior. Freezing expansion rate \( \varepsilon_f \) reaches 2.6%. Such different behavior in axial strain and drainage is attributed to the initial degree of compaction and may result from the difference in permeability. The degree of compaction significantly influences the permeability of the soil (Yokohama et al., 2014), namely, the permeability tends to decrease when the soil is compacted more densely. Such a tendency was recognized in the testing on the K soil as well (Matsumura, 2014). Therefore, even though almost exactly the same external temperature history to freeze and thaw can be applied to the specimens with different degrees of compaction, the behavior of each volume change and drainage during freeze–thawing may differ depending on the degree of compaction.

Thus, as the initial degree of compaction increases, freezing expansion rate \( \varepsilon_f \) tends to be higher, as shown in Fig. 12. From the figure, it is understood that the \( \varepsilon_f \) values of denser specimens become approximately twice those of the looser specimens. This fact implies that the freezing of soil leads to more significant changes in the soil fabric for specimens compacted at higher degrees of compaction. The above-mentioned tendency in volume change and moisture movement behavior during freeze–thawing agrees qualitatively with
those of the other freeze-thawed specimens, and the specimens at $D_{ci}$ of 93.3% shows the intermediate behavior of swelling and drainage in freezing, although the drawings have been omitted.

As shown in Figs. 10 and 11, axial strain $\varepsilon_a$ cannot return to the initial state of $\varepsilon_a=0$ even after the specimens have been completely thawed. This fact indicates that the specimens remain in the soil fabric changed by freezing after thawing. In order to identify volume changes due to freeze-thawing and consolidation, Fig. 13 shows the relation between the initial degree of compaction $D_{ci}$ and the degree of compaction after consolidation under the effective stress $\sigma_c'$ of 50 kPa (presented by $D_{cc}$ in the figure). In addition, regarding the freeze-thawed specimens, the degree of compaction after thawing with the frost heave cell kept on is plotted in relation to the $D_{ci}$ value. From this figure, it is clear that the freeze–thaw effect on volume change behavior appears. At a range over 90% in $D_{ci}$, the volume change occurs in the swelling side after thawing and consolidation, and the corresponding decrement in the degree of compaction becomes more significant as $D_{ci}$ increases. However, when the $D_{ci}$ is lower than 90%, conversely, the volume of the specimens changes toward the shrinkage side to a lesser extent regardless of whether or not the specimen is freeze-thawed after consolidation under $\sigma_c'$ of 50 kPa, while the freeze-thawed specimens still remain in swelling immediately after thawing. These facts agree well with previous findings, such as that of Viklander (1998). He reported that the void ratios are prone to converge at a certain unique void ratio between the initial ones whether an initial soil structure is loose or dense, as freeze–thaw cycles increase. In addition, the variation in volume becomes the most remarkable at the first cycle of the freeze–thaw sequence. Thus, the experimental results indicated that the volume change due to freeze–thaw sequence evolves on either side of swelling or shrinking depending on the initial degree of compaction.

Furthermore, based on the standpoint that the crushability of the constitutive particles contributes to the mechanical deterioration (Miura et al., 2003), the increment in finer content through a sequence of testing processes such as freeze-thawing, consolidation and cyclic loading was examined as depicted in Fig. 14. Although there is a variation in the experimental data, it appears that the specimens indicate higher crushability under the freeze-thawed condition, as they are more densely compacted. This seems to reflect the dilation behavior during freezing; i.e., the freezing of denser specimens can stimulate more changes in the soil fabric compared with looser specimens. Accordingly, it is understood that the increment in finer particles due to the freeze–thaw sequence shows positive correlations with the initial degree of compaction and the freezing expansion rate. However, this study is still not applicable to identifying the individual effect of increments in finer particles induced by freeze-thawing on the cyclic deformation-strength characteristics discussed below.

### 3.2. Effect of freeze–thaw sequence on cyclic shear characteristics

Figs. 15 and 16 show the time histories of deviator stress $\sigma_d$ and axial strain $\varepsilon_a$ up to 5% in the double-amplitude axial
strain $DA$, with the freeze-thawed specimen and the non-freeze-thawed one, respectively. The specimens were densely compacted at the initial degree of compaction $D_{ci}$ of 97.0% and applied cyclic loads at the cyclic stress ratio $SR$ of 0.26, which is calculated by $\sigma_d/2\sigma_c'$. In comparison to the number of loading cycles $N_c$ to cause the $DA$ value of 5%, it is clear that the specimen reached $DA=5\%$ with very few cycles of loading under the freeze-thawed condition. In other words, freeze-thawing leads to considerable deterioration in the liquefaction resistance of densely compacted $K$ soil. Furthermore, when focusing on the directions of the progressing deformation during cyclic loading, the non-freeze-thawed specimens tends to generate the extension strain dominantly as the $N_c$ value increases. In contrast, in the case of the freeze-thawed specimen, the axial strain seems to develop equally on the compression side as well as on the extension one. Such a tendency may lead to the anisotropic behavior of the $K$ soil specimens resulting from changes in the soil fabric induced by freeze-thawing, as will be discussed in detail later.

Fig. 17 illustrates the time histories of excess pore water pressure $\Delta u$ and effective mean principal stress $p'$ during cyclic loading, which are normalized by effective confining pressure $\sigma_c'$ of 50 kPa, respectively. These specimens correspond to Figs. 15 and 16. In terms of the $\Delta u/\sigma_c' - N_c$ relation, it is shown that the excess pore water pressure rapidly increases from the beginning of loading after freeze-thawing. As a result, the freeze-thawed specimen immediately causes initial liquefaction whereby $\Delta u$ reaches 95% of $\sigma_c'$. The trend for $\Delta u$, on the other hand, is consistent with the behavior of the effective mean principal stress $p'$ during cyclic loading. That is, the specimen exposed to the freeze-thaw sequence shows $p'$ decreasing at a higher rate, and this leads to liquefaction as the loading cycle increases, in comparison to the specimen not freeze-thawed.

Such a difference in the behavior of $\Delta u$ and $p'$, resulting from freeze-thawing, could be understood for each effective stress path of the freeze-thawed specimen and the non-freeze-thawed specimens shown in Fig. 18. These specimens are the same as those in Fig. 17. This figure indicates that the freeze-thawed specimen tends to reach liquefaction rapidly with a drastic decrease in the effective stress. This is similar to the typical case of loose clean sands in liquefying. On the other hand, the specimen not freeze-thawed is gradually liquefied with a moderate decrease in the effective stress as well as dense clean sands. According to Fig. 18, the difference in the decreasing effective stress becomes very conspicuous at the first cycle of loading. Thus, it is obvious that freeze-thawing plays a significant role in the evolving of the excess pore water pressure with a decrement in the effective stress under seismic action, and it contributes to the cyclic strength degradation, as elaborated in Figs. 15 and 16.

In order to discuss the variation in liquefaction strength due to the freeze–thaw sequence, Fig. 19 represents each relationship between the cyclic stress ratio $SR$ and the number of loading cycles $N_c$ to cause $DA$ to reach 1% and 5%, $\Delta u$ to increase to 95% of $\sigma_c'$ and $p'$ to decrease to 5% of $\sigma_c'$, respectively. From this figure, it is definitely found that the freeze–thaw sequence can remarkably degrade the liquefaction strength of $K$ soil regardless of the failure definitions of $DA$, 

![Fig. 17](image-url)
The relationship between the maximum $\Delta u/\sigma'_c$ at each cycle, where the $DA$ value reaches 1 and 5% and each cyclic stress ratio $SR$, is presented in Fig. 21. It clearly shows some difference at $DA=1\%$ depending on whether the specimen underwent the freeze–thaw sequence. The freeze-thawed specimens tend to reach a $DA$ of 1% with lower excess pore water pressure, and this tendency becomes remarkable as the value for $SR$ becomes higher. Furthermore, while the $DA$ value develops to 5%, the excess pore water pressure completely reaches the confining pressure with any $SR$ in both the freeze-thawed specimens and the non-freeze-thawed ones, because the $\Delta u/\sigma'_c$ values are over 1.0, as shown in Fig. 21. Thus, the freeze–thaw sequence may induce larger strain with lower excess pore water pressure due to cyclic loading before the initial liquefaction.

In order to obtain a better understanding as the cyclic deformation progresses in a different way, as shown in Figs. 15 and 16, Fig. 22 reveals the $N_c$-$DA$ relation with each cyclic stress. It is noted that the values in both axes are divided by each $N_c$ and $DA$ at $DA=5\%$. From this figure, the corresponding tendencies that depend on the freeze–thaw sequence can be found; i.e., the axial strain of the freeze-thawed specimens increases proportionately to some extent under any cyclic stress ratio conditions, as the loading cycle increases. On the contrary, the specimens that have not undergone the freeze–thaw sequence are likely to exponentially increase $DA$ from a certain $N_c$. Considering the points where the initial liquefaction, $\Delta u/\sigma'_c=0.95$, occurs, as described in Fig. 22, those for the specimens not freeze-thawed seem to be located where the cyclic deformation rapidly grows. However, such a tendency is not consistent with the freeze-thawed specimens, and the cyclic deformation for them develops at a relatively constant rate on the relation of $N_c$ vs. $DA$. Therefore, it can be said that the $K$ soil specimens tend to deform more remarkably by the cyclic action from the beginning of loading due to the freeze–thaw sequence.

Furthermore, Fig. 23 represents the ratio of extension axial strain $\varepsilon_{a,\text{ext}}$ to $DA=1\%$ and 5% with each cyclic stress, respectively, to discuss changes in anisotropic behavior induced by freeze-thawing. With respect to the specimens not freeze-thawed, the axial strain remains around 80 to 90% of $DA$ in the extension side at any $DA$ and $SR$, which means that the cyclic deformation develops dominantly toward the extension side. However, in any results of freeze-thawed specimens, the extension axial strain is lower than 80% of $DA$, and it does not always advance prior to the compression axial strain $\varepsilon_{a,\text{comp}}$. Such a trend becomes particularly significant at smaller strain levels and under lower cyclic stress. Thus, it is clear that the freeze–thaw sequence has an influence on not only the liquefaction strength, but also the cyclic deformation behavior.

The aforementioned tendency seems similar to the typical liquefaction behavior of clean sands that have anisotropic mechanical properties on the conventional liquefaction tests. In other words, the change in cyclic deformation behavior due to freeze-thawing shown in this study may be related to the structural anisotropy of the soil particles induced by a
peculiarity of the conventional liquefaction test. Then, the anisotropic behavior is associated with the configuration or orientation of the constitutive particles that are non-spherical under the gravity action, and the experimental peculiarity indicates that the effective mean principal stress $p'$ is not kept constant during cyclic loading. According to Oda (1972) and Miura and Toki (1984), it was verified that if a longitudinal axis of each particle dominantly deposited in a certain direction, is inclined at $\delta$ degrees to the maximum principal stress direction, as shown by the $\delta$-specimen in Fig. 24, the anisotropic mechanical behavior varies depending on the $\delta$ value. In the case of $\delta=90^\circ$, which is equal to the $90^\circ$-specimen in Fig. 24, for example, the axial strain on the extension side where $p'$ decreases tends to increase dominantly in the cyclic triaxial test. On the other hand, based on a $0^\circ$-specimen with $\delta=0^\circ$, the compression axial strain develops significantly due to cyclic loading. In addition, a specimen with a certain $\delta$ within $0^\circ$ to $90^\circ$ shows intermediate behavior on monotonic and cyclic deformation behavior. These are closely similar to the difference in cyclic deformation behavior of K soil compacted with or without freeze-thaw sequence. 

On the basis of the prior discussion about cyclic deformation behavior, it can be said that densely compacted K soil, which is similar to the $90^\circ$-specimen on the deposited direction, changes in soil fabric toward the $\delta$ value less than $90^\circ$ due to freeze-thawing in Fig. 24. Thus, because the compacted K soil specimens are prone to the strong anisotropy in liquefying, and the cyclic deformation behavior can be definitely affected by the freeze-thaw sequence, it is important to examine the relationship between the anisotropic fabrics of compacted soils and each stress direction such as freeze-thaw sequence and cyclic loading.

3.3. Effect of degree of compaction on cyclic shear behavior of freeze-thawed specimen

In the previous section, the remarkable change in cyclic strength-deformation behavior due to the freeze-thaw sequence was elucidated. However, according to the volume change and moisture movement behavior occurring during freezing and thawing, those can be strongly influenced by the initial degree of compaction. Figs. 25 and 26 compare the effect of initial degree of compaction $D_{ci}$ on the time histories of axial strain $\epsilon_a$ and excess pore water pressure ratio $\Delta u/\sigma_c^0$ up to 5% in $DA$, respectively. Both the freeze-thawed and the non-freeze-thawed specimens compacted, at each $D_{ci}$ of 97.0, 93.3 and 88.8%, were applied with cyclic loading within the SR range of 0.23 to 0.27. From Fig. 25, it is seen that freeze-thawing causes the drastic decrease in $N_c$ to reach $DA=5\%$ for the specimens with $D_{ci}$ of 97.0%, as explained in Figs. 15 and 16. The trend of cyclic strength deterioration seems consistent with the result of $D_{ci}=93.3\%$, and the change in the anisotropic behavior resulting from the freeze-thaw sequence can be
recognized as well. However, according to the results of the specimens loosely compacted at $D_{ci} = 88.8\%$, it shows little difference in the cyclic strength behavior despite freeze–thawing. Furthermore, the specimens in these cases indicate the comparatively-similar tendency in cyclic deforming regardless of freeze–thawing, unlike the above two cases different in $D_{ci}$. The aforementioned relation of the freeze–thaw effect with the initial degree of compaction agrees well with that in excess pore water pressure behavior during cyclic loading as described in Fig. 26. Therefore, the densely-compacted specimens which cause the cyclic strength deterioration due to freeze–thawing show that the excess pore water pressure increases at higher rates than those of the specimens not freeze–thawed. In contrast, when compacted at the $D_{ci}$ of 88.8%, the specimens with or without the freeze–thaw sequence basically correspond to each other in regards to the excess pore water pressure behavior.

Fig. 27 shows the $N_c$–DA relation as well as Fig. 22 with respect to each freeze–thaw sequence and degree of compaction. The specimens not freeze-thawed tend to follow a unique path on $N_c$ vs. DA similar to one another, even though compacted at a different degree of compaction. In addition, those results indicate the typical behavior that the cyclic deformation abruptly develops after the excess pore water pressure reaches 95% of the effective confining stress, as explained with Fig. 22. On the other hand, for the freeze–thawed specimens, the effect of freeze–thawing on the cyclic deformation behavior obviously appears while becoming lesser in the case of $D_{ci} = 88.8\%$. It is understood that although the $K$ soil specimens can be definitely influenced by freeze–thawing, the cyclic strength might be more susceptible to dry density (degree of compaction) than to the freeze–thaw sequence in the loosely-compacted condition.

Figs. 28 and 29 represent each liquefaction strength curve to attain $DA = 1\%$ and $5\%$, $\Delta u/\sigma_{c0} = 0.05$ and $p'/{\sigma}_{c0} = 0.05$ with the specimens compacted at 93.3 and 88.8% in $D_{ci}$, respectively. From Fig. 28, although the freeze–thawing effect on the cyclic strength becomes smaller than that of $D_{ci} = 97.0\%$, the freeze–
The thaw sequence triggers the cyclic strength degradation for the K soil specimens. However, if the specimens are compacted more loosely to become $D_i = 88.8\%$, little influence of the freeze-thawing on the liquefaction strength can be recognized.

Based on the liquefaction strength curves shown in Figs. 19, 28 and 29, Fig. 30 describes the relations between the degree of compaction after consolidation $D_{cc}$ and each $SR_{20}$ to reach the $DA$ values of 1\% and 5\%. Some clear differences in the cyclic strength behavior caused by freeze-thawing are indicated: i.e., (1) freeze-thawing disturbs the increase in cyclic strength accompanied with the increase in degree of compaction, (2) the $SR_{20}$ value for $DA = 1\%$ can hardly increase under the freeze-thawed condition, while compacted more densely, unlike the specimens not freeze-thawed. According to this study, it is emphasized that the cyclic strength characteristics of K soil can remarkably deteriorate when exposed to the freeze–thaw sequence. However, such effects of freeze-thawing ought to be incorporated into the relation with the initial degree of compaction prior to freezing.
4. Conclusions

From the results of a series of cyclic triaxial tests on K soil specimens with or without the freeze–thaw sequence, the following conclusions were derived:

(1) The freeze–thaw sequence causes volume changes with the water supply, drainage and particle breakage, and each behavior differs depending on the initial degree of compaction $D_{ci}$. Densely compacted K soil tends to swell more significantly when absorbing water due to freezing, and the specimen remains in dilation even after thawed and consolidated. At degrees of compaction lower than 90%, however, the specimen causes less dilation with drainage due to freezing, and it shrinks after being thawed and consolidated.

(2) The freeze–thaw sequence causes the excess pore water pressure to rise rapidly, and decreases the cyclic shear strength for the densely compacted K soil to 5% with the number of loading cycles $N_c$ of 20, $S_{R20}$, drops to around 70% due to freeze–thaw sequence compared with the specimens not freeze-thawed.

(3) Freeze-thawing triggers changes in the cyclic deformation behavior. The deformation of the freeze-thawed specimens tends to continue to constantly evolve as the loading cycles increase, while that of the specimens not freeze-thawed seems to increase exponentially as soon as the $\Delta u$ value reaches 95% of $\sigma'$, In addition, anisotropic behavior in liquefying changes due to freeze–thaw sequence.

(4) The effect of freeze-thawing on the cyclic shear characteristics varies depending on the initial degree of compaction $D_{ci}$. That is, such effects of freeze-thawing, as mentioned above, can decrease at lower degrees of compaction.

Acknowledgements

The authors would like to express their gratitude to Mr. Hiroshi Minami formerly of Hokkaido University who helped us to conduct the experiments. This study was undertaken with the financial supports of KAKENHI (Grant-in-Aid for Scientific Research (A) no. 23241056), Japan Society for the Promotion of Science.

References


