CHARACTERISTICS OF LIQUEFACTION STRENGTH OF SAND UNDER OVERCONSOLIDATION AND LONG-TERM CONSOLIDATION CONDITIONS

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ABSTRACT :

If sandy soil deposits are subjected to overconsolidation or long-term consolidation, their liquefaction strength increases. However, only a few studies have investigated the effects of overconsolidation or long-term consolidation by means of laboratory element tests in order to simulate the in-situ ground condition more closely. In the present study, the liquefaction strength characteristics of overconsolidated sand samples were investigated under a K_0-stress condition by using a hollow cylindrical torsional shear apparatus based on the of the supposition that earthquake excitations affect the ground horizontally. Further, a surcharge method was used to simulate the stress histories of overconsolidation. The effect of overconsolidation on the liquefaction strength was studied on sand samples that were consolidated for a considerable period of time. Clean Toyoura sand and sandy soil collected from the sea bed containing fines were used as samples. The following behaviors were observed in the tests: 1) The liquefaction strength of both types of sand increased due to overconsolidation when the samples were subjected to long-term consolidation before the stress history of overconsolidation was applied to them. 2) The increase in the liquefaction strength of the two types of sand subjected to overconsolidation and long-term consolidation could be formularized by the overconsolidation ratio and the time taken for long-term consolidation.

KEYWORDS: liquefaction strength, overconsolidation, long-term consolidation, sandy soil

1. INTRODUCTION

If sandy soil deposits are subjected to overconsolidation or long-term consolidation, their liquefaction strength increases. Several laboratory element tests have been performed in order to understand the tendency to quantitatively increase the liquefaction strength, e.g., the tests in Tatsuoka et al. (1988), Nagase et al. (1996) and Nagase et al. (2000). However, the cyclic triaxial test was the typical element test conducted in the previous studies, and the stress conditions in the sand deposits during earthquakes could not be simulated precisely in these tests. For example, stress histories of overconsolidation and cyclic shear stress are applied to the horizontal ground in a state in keeping with K_0-stress condition.

Cyclic torsional shear tests were conducted in order to clarify the effects of overconsolidation on liquefaction strength by Ishihara et al. (1979). However, the effects of overconsolidation on the liquefaction strength obtained by the tests on sand under the K_0-consolidation condition in which the lateral displacement of the sample was restricted, were not perfectly clear in the study. Nagase et al. (2004) studied the effects of overconsolidation on the liquefaction strength of the sand samples under the K_0-stress condition and found that the increase in the liquefaction strength was greater in the test on sand overconsolidated under the K_0-consolidation than that on sand under isotropic and anisotropic consolidations.

In the Tatsuoka et al. (1988), the effects of overconsolidation and consolidation time on the liquefaction strength of Toyoura sand were also investigated. The liquefaction strength curve of loose sand with a relative density of 50%, subjected to a long-term consolidation of 1630 h was found to be almost equal to that obtained by the tests on the same loose sand overconsolidated with an overconsolidation ratio of 2. However, the effects of overconsolidation on the liquefaction strength of sand subjected to long-term consolidation were not investigated in the study.

In the present study, the liquefaction strength characteristics of the overconsolidated sand samples were investigated under the K_0-stress condition by using a hollow cylindrical torsional shear apparatus on the basis of the supposition that earthquake excitations affect the ground horizontally. Further, a surcharge method was used to simulate the stress histories of overconsolidation. The effect of overconsolidation on the liquefaction strength was studied on sand samples that were consolidated for a long term. Clean Toyoura sand and sandy soil collected from the sea bed containing fines were used as samples. The vertical displacement of the specimens was restricted in the cyclic loading in order to maintain the restriction of the lateral displacement of the specimens. The effects of overconsolidation and long-term consolidation on the liquefaction strength, observed in the sandy soil deposits that were subjected to the stress histories induced by the surcharge method, will be discussed in the following sections.

2. SAMPLES AND TEST PROCEDURES

Clean Toyoura sand was used as a sample material. The particle density of the soil, ρ_s, was 2.637 g/cm^3, and the maximum and minimum void ratios, e_{max} and e_{min}, were 0.973 and 0.609, respectively. The dredged soil samples collected from Hakata Bay with fine content ratios of 10% and 30%, which were called H1 and H2, respectively, were also used as sample materials in this study. The soil particle density, ρ_s, of these samples was 2.746 g/m^3. Further, the maximum and minimum void ratios of both the samples, e_{max} and e_{min} were 0.901 and 0.480, and 1.021 and 0.484, respectively. The plasticity index, I_p, was 32. Figure 1 shows the grain size distribution curves of the three samples. Hollow cylindrical specimens of 10 cm in outer diameter, 6 cm in inner diameter, and 10 cm in height were used in the cyclic torsional shear tests. Specimens with a relative density of 45% were prepared by an air-pluviation method. In all the tests, the specimens were saturated by circulating carbon dioxide, CO_2, and deaired water and the B-value was increased to 095.

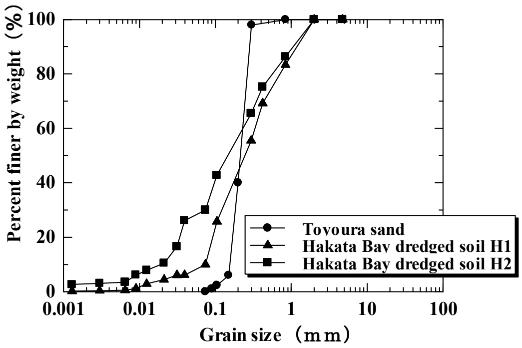

Figure 1 Grain size distribution curves

Table 1 Test conditions in Cases (A)–(C)

Case	Sample	Overconsolidation ratio $(OCR)_v$	Consolidation time (h)
(A)	Toyoura sand	1, 2, 3	1
(B)	Hakata Bay dredged soil H1	1, 2, 3	
(C)	Hakata Bay dredged soil H2	1, 2, 3	

Tables 1–4 show the test conditions used in the present study. Tables 1 and 2 indicate the test conditions in which the samples were subjected to overconsolidation or long-term consolidation independently during consolidation. Table 3 shows the test conditions in which the samples were subjected to the maximum confining stress during overconsolidation for a long period of time. Table 4 indicates the test condition, in which the samples were consolidated by using the initial effective vertical confining stress, σ_{v0}', for a long period of time at first and then they were overconsolidated. In all the tests, the specimens were consolidated under the K_0-stress condition.

The vertical displacement of the specimens was restricted in the cyclic loading. The stress histories of overconsolidation were applied to the specimens in the following manner: First, the specimens were consolidated by the initial vertical and horizontal effective confining pressures, σ_{v0}' and σ_{h0}',

Table 2 Test conditions in Cases (D)–(F)

Case	Sample	Overconsolidation ratio $(OCR)_v$	Consolidation time (h)
(D)	Toyoura sand	1	1
			6
			24
			72
			168
			672
			2016
(E)	Hakata Bay dredged soil H1	1	1
			6
			24
			72
			168
			672
(F)	Hakata Bay dredged soil H2	1	1
			6
			24
			72
			168
			672

Table 3 Test conditions in Case (G)

Case	Sample	Overconsolidation ratio $(OCR)_v$	Consolidation time (h)
(G)	Toyoura sand	2	1
			24
			72

Table 4 Test conditions in Cases (H)–(J)

Case	Sample	Overconsolidation ratio $(OCR)_v$	Consolidation time (h)
(H)	Toyoura sand	2	1
			6
			24
			72
			168
			672
(I)	Hakata Bay dredged soil H1	2	1
			6
			24
			72
			168
			672
(J)	Hakata Bay dredged soil H2	2	1
			6
			24
			72
			168
			672

respectively, where σ_{h0}' was equal to $K_0 \cdot \sigma_{v0}'$, in the case where the lateral displacement was prevented. After the consolidation, σ_v' was decreased to σ_{v0}', restricting the lateral displacement of the specimens. The horizontal effective stress, σ_h', ordinarily changes keeping the K_0-consolidation condition, while the vertical effective stress, σ_v', increases or decreases. The overconsolidation ratio, $(OCR)_v$, was defined as the ratio of the maximum vertical effective stress, σ_v', to the initial vertical effective stress, σ_{v0}', although the ratio, $(OCR)_v$, usually does not correspond to the ratio of the maximum horizontal effective stress, σ_h', to the initial horizontal effective stress, σ_{h0}', during overconsolidation under a K_0-stress condition. The long-term consolidation was applied to the samples by using the torsional shear test apparatus for a consolidation time shorter than 72 h and by using a simplified apparatus to simulate one-dimensional consolidation for a consolidation time longer than 72 h.

The K_0-consolidation and cyclic shear tests under the K_0-consolidation condition were performed by following the procedure in Nagase et al. (2004). During the K_0-consolidation, vertical stress was applied to the specimens at a rate of 1.96kPa/min. The lateral displacement was restricted to a value smaller than ±0.05%. In the cyclic shear tests, under the condition where the lateral displacement was restricted, the difference between the water level around the specimens in the inner cell and the reference water level in the bullet with a differential pressure gauge was restricted to a value smaller than ±0.05 ml by exerting lateral pressure. The differential water level was equivalent to the controlled value during the K_0-consolidation.

3. TEST RESULTS

3.1. Time Histories Observed in Cyclic Shear Tests

Typical time histories of the cyclic stress ratio, τ/σ_{v0}', the shear strain, γ, the excess pore water pressure ratio, $\Delta u/\sigma_{h0}'$, the vertical total stress, σ_v, the vertical effective stress, σ_v', and the horizontal effective stresses, σ_h', are illustrated in Figure 2 by using the data obtained from Case (A).

When the excess pore water pressure ratio, $\Delta u/\sigma_{h0}'$, increased in the process of cyclic loading to a value of 1.0, the shear strain, γ, increased suddenly, as shown in Figure 2. This phenomenon showed that the specimen was liquefied during the test. In the liquefaction stage, the vertical total stress, σ_v, corresponded to the initial horizontal effective stress, σ_{h0}', and both the vertical and the horizontal

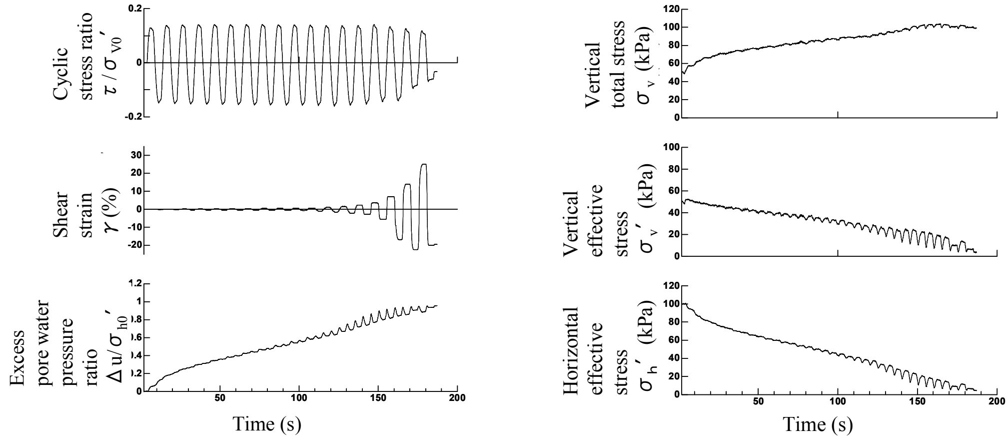

Figure 2 Typical time histories of test results observed in Case

effective stresses, σ_v', and, σ_h', coincided with zero. On the basis of these results, it can be realized that an isotropic stress state was simulated after the specimen was liquefied.

3.2. Liquefaction Strength Characteristics of Sand Subjected to Overconsolidation

Figures 3–5 show the relationships between the cyclic stress ratio, $R = \tau/\sigma_{v0}'$, and the number of cycles, N_c, to a double amplitude shear strain, DA, of 7.5% obtained from the cyclic torsional shear tests in Cases (A), (B) and (C) on Toyoura sand, Hataka Bay dredged soils H1, and H2 overconsolidated with an overconsolidation ratio of 1, 2 and 3, respectively. It can be seen from these figures that the cyclic stress ratio, R, was considerably large in the three samples, as the overconsolidation ratio, $(OCR)_v$, was large. Moreover, it might be considered that this increase in the cyclic stress ratio was based on the overconsolidation effect and the increase in the K_0-value due to overconsolidation, which induced the increase in the initial effective confining stress.

The cyclic stress ratio at the 20th cycle, which is called the liquefaction strength ratio, R_{l20}, was obtained from Figures 3–5 in order to calculate the ratio of increase in the liquefaction strength due to overconsolidation, $(R_{OC})_v$. $(R_{OC})_v$ was defined as the ratio of the liquefaction strength ratio of overconsolidated sand to that of the normally consolidated sand. Figure 6 indicates the relationship between R_{oc} and $(OCR)_v$. It can be observed from this figure that the ratio of the increase in the liquefaction strength, R_{oc}, was equivalent to $(OCR)_v^n$. Moreover, the value of n was 0.5 in Case (A) and was within a narrow range of 0.60-0.70 in Cases (B) and (C). It should be noted that the value of n did not correspond with the value of 0.25 obtained from the cyclic triaxial tests, which were performed by Tatsuoka et al. (1988), because the liquefaction strength ratio, R_{l20}, was affected by the consolidation conditions such as K_0-consolidation or isotropic consolidation. Further, the value of n

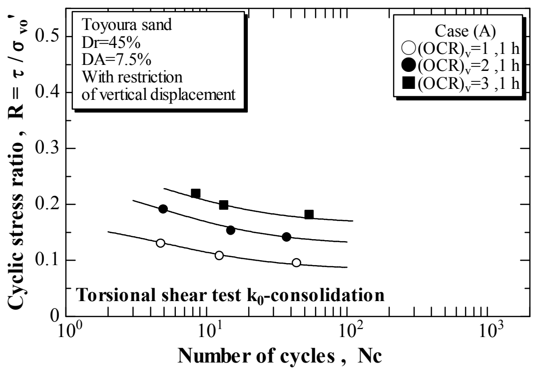

Figure 3 Cyclic stress ratio, R, versus number of cycles, Nc, to DA = 7.5%

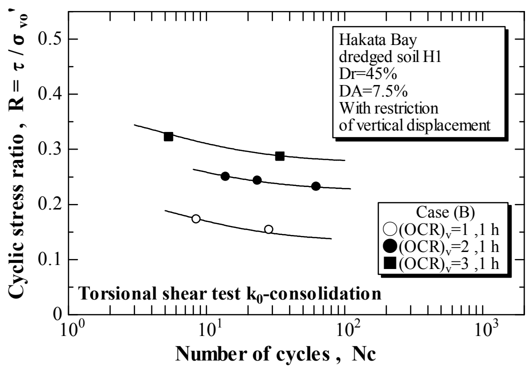

Figure 4 Cyclic stress ratio, R, versus number of cycles, Nc, to DA = 7.5%

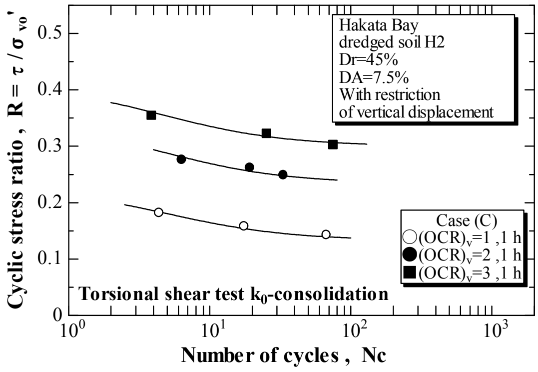

Figure 5 Cyclic stress ratio, R, versus number of cycles, Nc, to DA = 7.5%

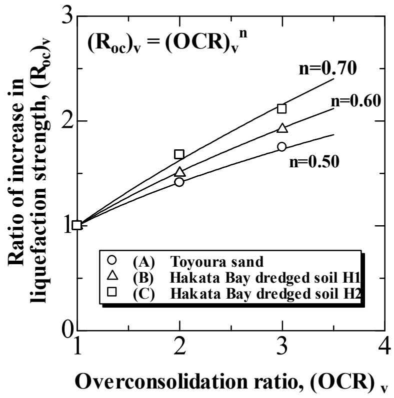

Figure 6 Ratio of increase in liquefaction strength, $(R_{Lc})_v$, versus overconsolidation ration, (OCR)v

increased as the fine content ratio, F_c, was high. Therefore, it may be considered from Figure 6 that the overconsolidation effect evaluated by the value of n was significant because the cohesive resistance increased due to the fines in the soil samples during overconsolidation and the structure of the particles was stabilized.

3.3. Liquefaction strength Characteristics of Sand subjected to Long-term Consolidation

Figures 7–9 show the relationships between the cyclic stress ratio, $R=\tau/\sigma_{v0}'$, and the number of cycles, N_c, to DA=7.5% obtained from Cases (D), (E) and (F), when the cyclic shear tests were conducted on the three samples subjected only to long-term consolidation. It can be observed from Figures 7–9 that the cyclic stress ratio increased as the consolidation time increased in the range of 1 h–2016 h for Toyoura sand and 1 h-672 h for Hakata Bay dredged soils H1 and H2. The ratio of increase in the liquefaction strength was large as the fine content ratio was high. The cyclic stress ratio at the 20th cycle, the liquefaction strength ratio, R_{l20}, was obtained from Figures 7–9 and the ratio of the increase in the liquefaction strength, $(R_{LC})_v$, which was defined as the ratio of the liquefaction strength ratio at any consolidation time to that at the consolidation time of 1 h under the condition of $(OCR)_v = 1$. The ratio of the increase in the liquefaction strength, $(R_{LC})_v$, is plotted against the consolidation time in Figure 10. The trend of the data shown in Figure 10 can be expressed by the equation $(R_{LC})_v = T^n$, and the value of n was estimated to be 0.04 for Toyoura sand and 0.067 and 0.075 for Hakata Bay dredged soils H1 and H2, respectively. The value of n was large as the fine content ratio, F_c, and the plasticity index, I_p, were high. A long-term effect also seemed to be more extremely accumulated in the samples when the cohesion of the sample was high.

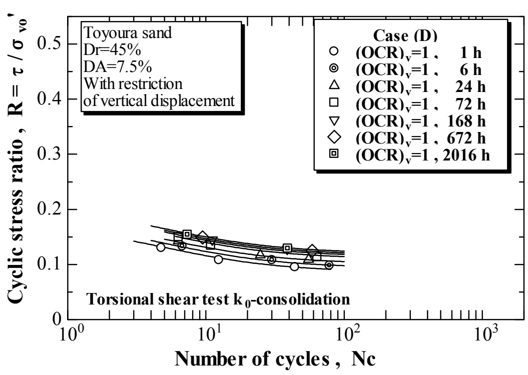

Figure 7 Cyclic stress ratio, R, versus number of cycles, Nc, to DA = 7.5%

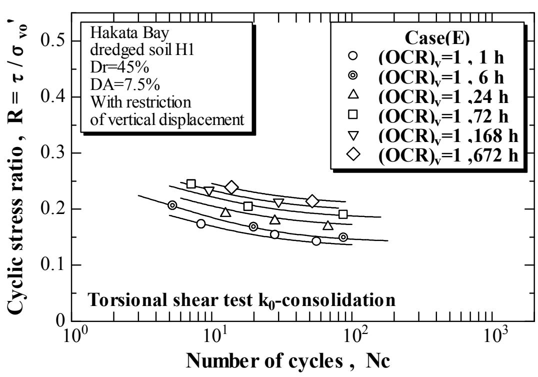

Figure 8 Cyclic stress ratio, R, versus number of cycles, Nc, to DA = 7.5%

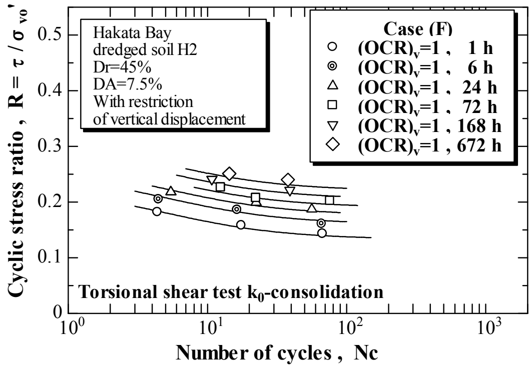

Figure 9 Cyclic stress ratio, R, versus number of cycles, Nc, to DA = 7.5%

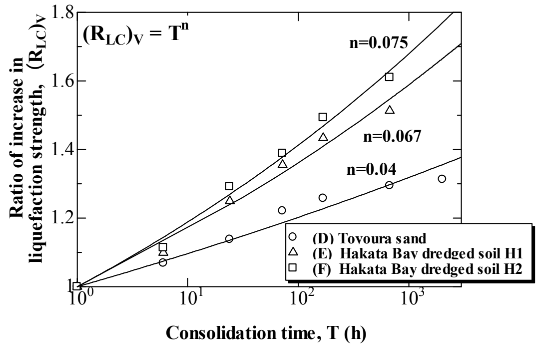

Figure 10 Ratio of increase in liquefaction strength, $(R_{Lc})_v$, versus consolidationtime, T

3.4. Liquefaction Strength Characteristics of Sand Subjected to the Maximum Confining Stress during Overconsolidation for a Long Period of Time

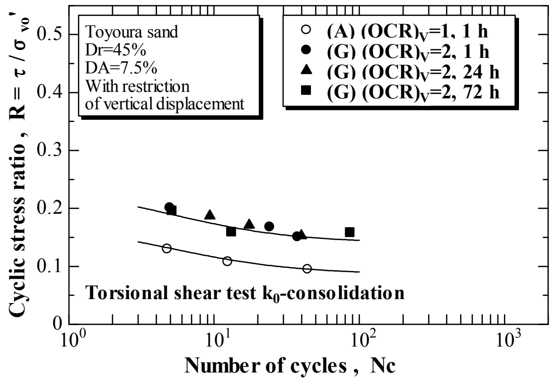

Figure 11 Cyclic stress ratio, R, versus number of cycles, Nc, to DA = 7.5%

Figure 11 shows the cyclic stress ratio, $R = \tau/\sigma_{v0}'$, versus the number of cycles to DA = 7.5% of Toyoura sand. This figure indicates the results obtained by the tests in which the samples were subjected to the maximum vertical stress during overconsolidation for a long period of time. After the long-term consolidation, the vertical effective stress was decreased to the initial vertical effective stress, σ_{v0}', before carrying out the cyclic shear test in Case (G). It can be observed from this figure that the cyclic stress ratio in Case (G) is greater than that in Case (A), in which the consolidation time, T, was 1 h and the overconsolidation ratio, $(OCR)_v$, was 1, and did not increase even if the consolidation time was increased. This tendency was also observed in Sawada et al. (2006). It might be considered that the effect of long-term consolidation on the liquefaction strength disappeared because of the reduction in the maximum confining stress to the initial confining stress. The effect of overconsolidation was governed in the specimens, instead of the effect of consolidation time. By comparing the liquefaction curve in Case (G) with that in Case (D), as shown in Figure 7, in the case of the maximum consolidation time of 2016 h, it can be recognized that the cyclic stress ratio in Case (D) was smaller than that in Case (G); however, the stress ratio in Case (D) gradually reached that in Case (G).

3.5. Liquefaction Strength Characteristics of Sand Subjected to Long-term Consolidation and Overconsolidation

Figures 12–14 show the cyclic stress ratio, $R = \tau/\sigma_{v0}'$, versus the number of cycles to DA = 7.5% for Toyoura sand in Case (H) and Hakata Bay dredged soils H1 and H2 in Cases (I) and (J), respectively. The differences between the cyclic stress ratio obtained under the consolidation time of 1h in Cases (A) and (H), in Cases (B) and (I), and in Cases (C) and (J) implied an increase in the liquefaction strength due to the overconsolidation effect. It was concluded by comparing the results in Cases (H) and (D), Cases (I) and (E), and Cases (J) and (F) that the cyclic stress ratio under the conditions of $(OCR)_v = 2$ and T = 1 (h) was usually greater than that under $(OCR)_v = 1$ and T = any consolidation time (h). From the three figures, it can be observed that the cyclic stress ratio obtained by the tests under the conditions of $(OCR)_v = 2$ increased with increasing consolidation time, T. Therefore, it is to be noted that the increase in the liquefaction strength due to overconsolidation was expected even in long-term in-situ ground deposits.

The liquefaction strength ratio, R_{l20}, was obtained from Figures 12–14 and the ratio of the increase in the liquefaction strength, $(R_{LC, OC})_v$, was defined as the ratio of the liquefaction strength ratio at any consolidation time to that at the consolidation time of 1 h under the condition of $(OCR)_v = 2$. The ratio of the increase in the liquefaction strength, $(R_{LC, OC})_v$, is plotted against the consolidation time in Figure 15. The tendency of the data indicated in Figure 15 can be expressed by the equation $(R_{LC, OC})_v = T^n$. The value of n was 0.04 for Toyoura sand and 0.021 and 0.027 for Hakata Bay dredged soils H1 and H2, respectively. The value of n was low when the samples contained fines: this shows a trend opposite to the increase in the liquefaction strength due to long-term consolidation, as shown in Figure 10. It can be concluded from Figures 10 and 15 that if the increase in the liquefaction strength due to the long-term consolidation was significant, the increase in the liquefaction strength due to the overconsolidation would decrease slightly because a considerable portion of the stabilizing particle structure was achieved by the long-term consolidation of the given specimens before they were overconsolidated.

From the results described above, it can be assumed that the increase in the liquefaction strength of sand subjected to overconsolidation and long-term consolidation could be formularized by using the overconsolidation ratio and the time taken for long-term consolidation as follows: $(R_{LC, OC})_v =$

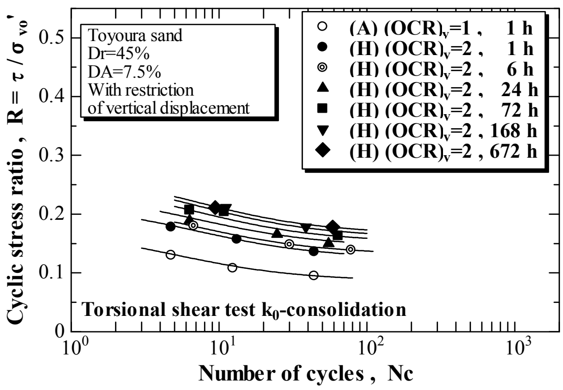

Figure 12 Cyclic stress ratio, R, versus number of cycles, Nc, to DA = 7.5%

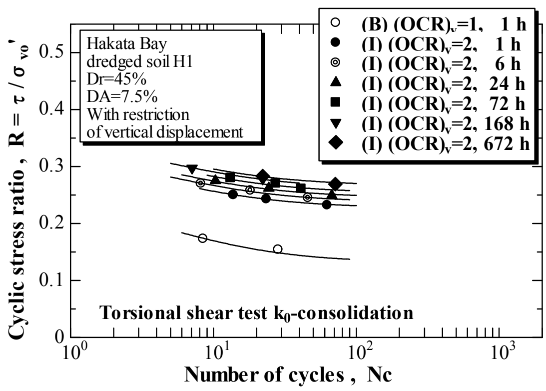

Figure 13 Cyclic stress ratio, R, versus number of cycles, Nc, to DA = 7.5%

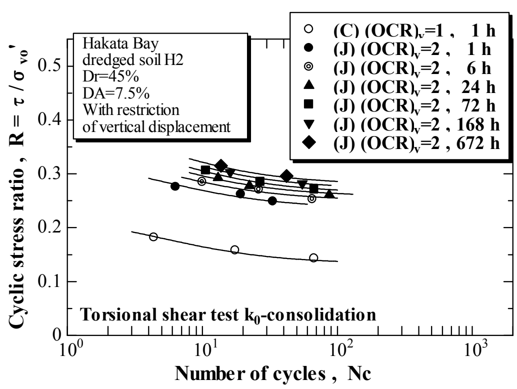

Figure 14 Cyclic stress ratio, R, versus number of cycles, Nc, to DA = 7.5%

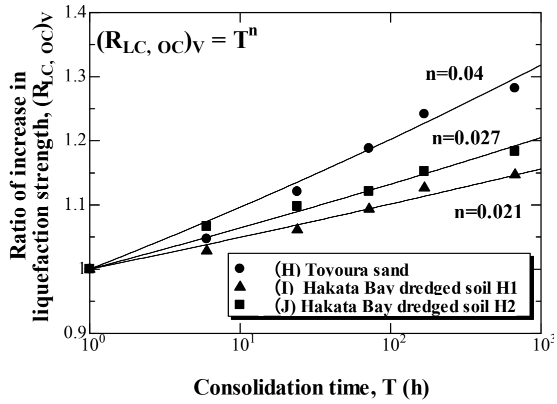

Figure 15 Ratio of increase in liquefaction strength, $(R_{Lc})_v$, versus consolidationtime, T

$(OCR)_v^{n1} \cdot T^{n2}$. The values of n1 and n2 were 0.5 and 0.04 for Toyoura sand with no fines, 0.6 and 0.021 for Hakata Bay dredged soil H1 with F_c = 10% and 0.7 and 0.027 for Hakata Bay dredged soil H2 with F_c = 30%, respectively.

4. CONCLUSIONS

Several series of cyclic torsional shear tests were conducted on clean Toyoura sand and Hakata Bay dredged soils H1 and H2, with fine content ratios of 10% and 30%, respectively, in order to investigate the liquefaction strength characteristics of sand samples subjected to overconsolidation and long-term consolidation under a K_0-consolidation condition. In the tests, the following behaviors were observed:

(1) The increase in liquefaction strength due to long-term consolidation was considerably high as the fine content ratio, F_c, and the plasticity index, I_p, were high. When the ratio of the increase in the liquefaction strength, $(R_{LC})_v$, was related with the consolidation time, T, and the relationship was expressed by $(R_{LC})_v = T^n$, the value of n was large as F_c and I_p were high.

(2) The liquefaction strength of the sand increased due to overconsolidation when the samples were subjected to long-term consolidation before the stress history of overconsolidation was applied to them. When the ratio of the increase in the liquefaction strength, $(R_{LC,\,OC})_v$, was also related to the consolidation time, T, and the equation was expressed by $(R_{LC,\,OC})_v = T^n$, the value of n was small for samples that contained fines.

(3) The increase in the liquefaction strength of sand subjected to overconsolidation and long-term consolidation could be formularized by using the overconsolidation ratio, $(OCR)_v$, and the time taken for long-term consolidation, T, according to the equation $(R_{LC,\,OC})_v = (OCR)_v^{n1} \cdot T^{n2}$.

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