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An experimental study of cyclic mobility and shear strain history depended on liquefaction of dense sandy ground using a shaking table

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ABSTRACT: A characteristic behavior in the case of liquefaction of the ground is cyclic mobility. Cyclic mobility is regarded as one of the important evaluation items of displacement and deformation when ground liquefaction occurs. This study conducted 1g field shaking table tests for dense sandy ground to consider cyclic mobility. The test results revealed that remarkable cyclic mobility occurred in the region of relatively small shear strain up to 1% to 2% or so if the value of the strain exceeded the past maximum strain and that such a tendency also appeared in the same manner if the shaking cases wereswitched and dissipation of excess pore water pressure was performed between the cases. Additionally, in the case where the shear strain reached about 3%, the results suggested the possible presence of a limit value of strainthat could cause cyclic mobility even if the strain was not exceeded.

1 INTRODUCTION

In general, negative dilatancy is generated in loose sandy ground by shear strain in the case of an earthquake, and the reduction in the effective stress associated with occurrence of excess pore water pressure leads to liquefaction. While liquefied ground loses rigidity and shear strain increases, soil particles try to get over others at this time to cause positive dilatancy, and the excess pore water pressure instantaneously decreases to recover the effective stress (Figure 1). The recovery of the effective stress restores shear rigidity and shear strength, which causes the phenomenon of instantaneous increase in shear stress and rise in acceleration. Such a phenomenon is called cyclic mobility (Castro 1975).

Since cyclic mobility is caused by the change in soil from compression to expansion (crossing the phase transformation line) by the given shear force, it is remarkable in easy-to-expand dense sand. In the case of complete liquefaction of loose sand, mobility is not caused until strain reaches several percent. Moreover, since the magnitude of the strain required for generation of the mobility sharply increases in conjunction with repetitive shear, it cannot be known whether the occurrence appears cyclic (Elgamal et al. 1998). In other words, even if the sand is loose to some extent, infinitely giving strain eventually exhibits mobility in theory. It can therefore be said that cyclic mobility should occur if the sand is not too loose to cause static liquefaction. However, response strain in the case of an actual earthquake is finite, and it is not very likely that cyclic mobility is caused (identified) in loose sand in the range. Meanwhile, in the case of dense sand, cyclic mobility occurs even if liquefaction is not caused yet or the strain is relatively small. Since cyclic mobility increases the effective stress to make rigidity greater as described above, it is a factor in determining the amount of ground displacement, especially residual displacement, of an earthquake (Adalier et al. 2004). In addition, since cyclic mobility occurs more easily in dense ground than in loose ground, dense ground has a greater influence on the amount of displacement of an earthquake. Therefore, this study conducted shaking table tests in a 1 g field as an approach to

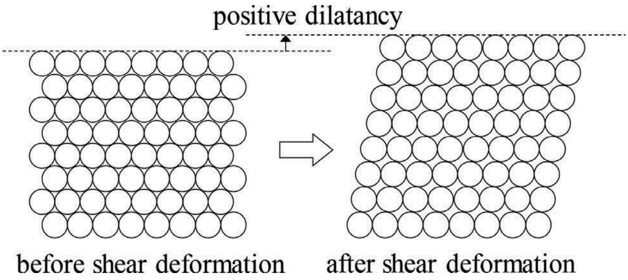

Figure 1. Image of Cyclic mobility

clarify deformation behavior of dense sand ground in the case of an earthquake and used the obtained response to consider the features of cyclic mobility.

2 ABOUT SHAKING TABLE TESTS

Figure 2 shows the laminar shear box used for the experiment. The shear box was composed of 16 aluminum movable frames piled up to create a model ground 700 mm in height, 800 mm in width, and 600 mm in depth inside. We devised the movable frames connected to each other with low frictional resistance sliders for as smooth a displacement as possible. The membranes attached onto the inside of the shear box for creation of saturated ground were 0.3 mm thick, which is the same as the thickness for indoor soil tests in consideration of preventing interference with deformation of the movable frames.

For the experiment, accelerometers, pore water pressure gauges, and displacement meters were placed in the positions shown in the figure. The accelerometers and the pore water pressure gauges were buried in the ground. As for the displacement meters, draw-wire displacement sensors were mounted on the fixed walls outside of the laminar shear box to measure the relative displacement of the shear frames from the bottom. The draw-wire displacement meters used had the mechanism to wind up the wire by springs, and the possible failure in tracking was anticipated if the displacement speed of the movable frame was fast. Therefore, the shear box was shaken at the shaking frequency of this experiment (5 Hz) in advance, and the measurements of the draw-wire displacement meters and laser displacement meters were compared. The results showed that the difference between the two was about 7% at the maximum values, and it was judged that the difference did not matter for examination of ground cyclic mobility.

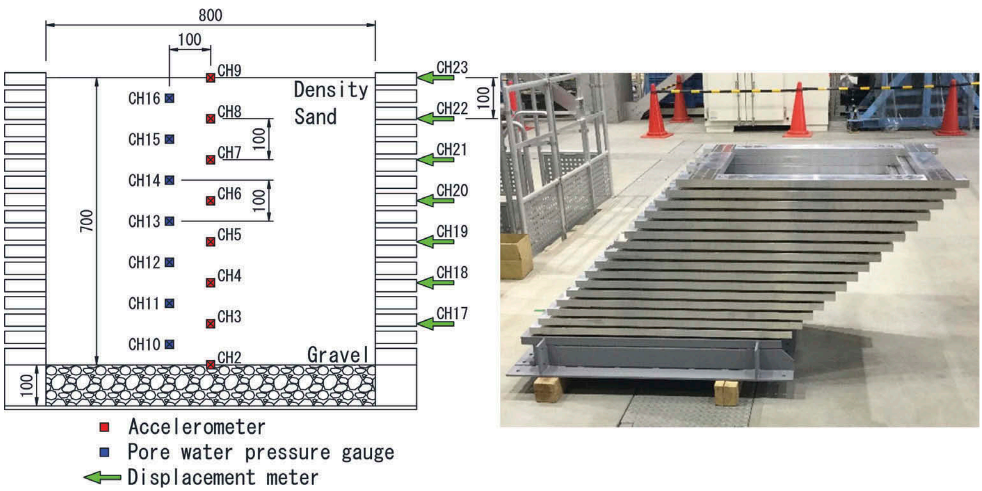

Figure 2. An experimental model of shaking table tests.

Table 1. Characteristics of sand using shaking table tests.

Specific Gravity (g/cm^3)	2.652
Maximum Grain Size (mm)	0.25
Fine Fraction Content (%)	40.4
Minimum Dry Density (g/cm3)	1.082
Maximum Dry Density (g/cm3)	1.497
Permeability Coefficient (cm/s)	1.17×10^{-3} (Dr=85%)

Table 2. Case of shaking table tests.

		Input motions	
Test Case	Relative Density of Model Ground (%)	Frequency (Hz)	Maximum Acceleration of The Table (gal)
1	85.4	5.0	400
2	85.4	5.0	400
3	97.1	5.0	800
4	100.1	5.0	800

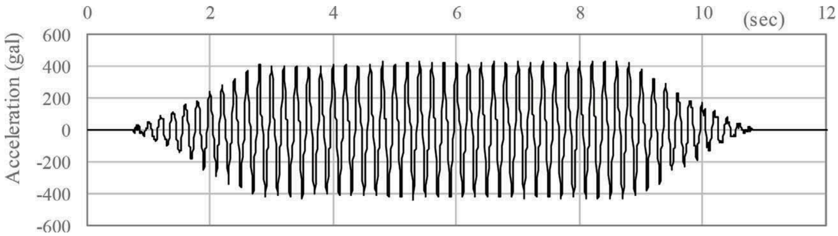

Figure 3. Input motion (Case1)

Table 1 shows the physical properties of the sand used for the experiment. Since this experiment was conducted in a 1 g field with a small shear box 700 mm high, the sand having a relatively small permeability coefficient was selected to prevent dissipation of pore water pressure before reaching liquefaction. Though the sand selected was mono-sized produced in a plant, many fine particles 0.074 mm or smaller were contained (fc = 40%), and the permeability coefficient was relatively small.

Table 2 shows the shaking test cases. Four cases of shaking tests were conducted for a single model ground, and Case 2 and subsequent cases were affected by the history of previous shaking. Table 2 shows the relative density of each case just before shaking. As the table indicates, the density of the model ground was increased by each shaking. The input wave was 5 Hz sine wave, and the maximum acceleration on the shaking table was 400 gal and 800 gal.

Figure 3 shows the shaking table waveform in Case 1 as an example of input wave. The input wave was 50 cycles of sine wave; the acceleration amplitude was gradually increased to the specified value in the first 10 cycles, kept constant in the next 30 cycles, and gradually decreased to zero in the last 10 cycles.

3 RESULTS OF THE TESTS

3.1 *Characteristics of cyclic mobility in case of triaxial tests*

Figure 4 shows an example of the relationship between shear stress and shear strain obtained by the cyclic undrained triaxial test of the sand. It was understood from the figure that (i) focusing on the relationship between stress and strain within one cycle of shear, stress sharply rose when strain reached a different magnitude per cycle to exhibit cyclic mobility, and (ii) the strain amplitude monotonically increased each time of repeated load. That is to say, the strain value at which shear stress reached the maximum of each cycle was greater than the strain value of the previous cycle. In other words, it was indicated that cyclic mobility would not occur unless the maximum shear strain generated in the past was exceeded, which is considered a characteristic phenomenon with regard to the occurrence of cyclic mobility. Except for these experimental results, triaxial tests or hollow cylinder torsional shear tests do not always show this increase in strain amplitude, and there is a report that it converged into a certain loop at some magnitude (Kawai et al. 2017). Anyway, both of them were the results of indoor tests, and it is possible that they were just a consequence of a boundary value problem that depended on the property as an element of the material or the test method. Therefore, this study focused on how such elemental behavior relates to the response of the ground as a whole (a set of elements) and considered the features from the 1 g field shaking table test results.

3.2 *Results of shaking table tests*

Figure 5 shows the time history of shear strain and excess pore water pressure ratio at GL-250 mm in Case 1. The shear strain was calculated as the average strain by dividing the relative displacement obtained from the multiple displacement time history measured at the shear frame by the height of the section in which the corresponding displacement meter was placed. The excess pore water pressure ratio was calculated by dividing the measurements of pore water pressure by the effective pressure given from above that was obtained from the density of the model ground.

The shear strain increased until about 3 seconds from the start of shaking and then decreased. The reason for the decrease is described later. As for the excess pore water pressure ratio, the negative amplitude of water pressure increased at the start of shaking, peaked a bit before 3 seconds, and then decreased. The amplitude almost vanished after 4 seconds and settled down to a constant value after 5 seconds. The reason why the excess pore water pressure ratio exceeded 1.0 is that the water pressure gauges sunk during shaking and the propagation of water pressure from lower layers, and the details are currently examined.

Figure 6 enlarges the horizontal axis of Figure 5 from 1.5 seconds to 4.5 seconds. The red circles and yellow triangles in the graphs indicate the minimum value of excess pore water

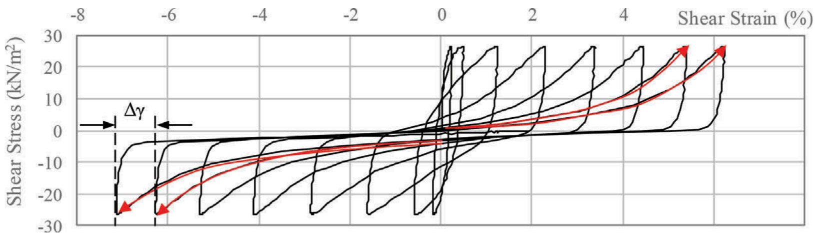

Figure 4. Relationship between shear stress and shear strain at cyclic undrained triaxial test

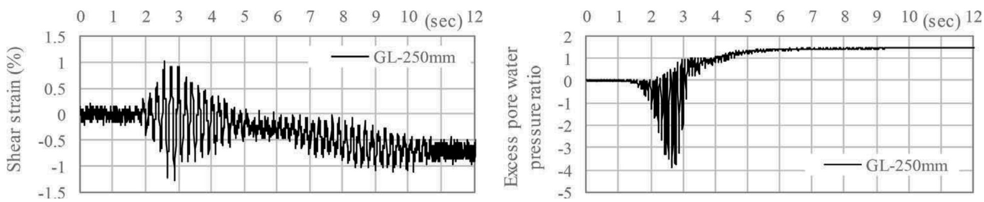

Figure 5. Time history of shear strain and excess pore water pressure ratio at Case1

pressure ratio within one shaking cycle and the shear strain value at the same time. The yellow plots are used when the shear strain is a positive value while the red plots are when it is a negative value.

As the figure shows, the negative peaks of the water pressure ratio corresponded to the positive and negative peaks of the shear strain, which means all of these responses are derived from cyclic mobility. The shear strain continued to increase until just after 2.5 seconds, then kept almost the same amplitude until 3.0 seconds, and decreased after 3.0 seconds. On the other hand, the negative excess pore water pressure ratio continued to increase until the shear strain reached the peak, and it decreased when the shear strain almost stopped increasing. The amplitude on the negative side quickly reduced after 3 seconds, and the time of the peaks did not match the peaks of shear strain any more after 3.5 seconds. According to the readings above, it can be judged that the behavior until around 3.5 seconds was due to cyclic mobility, although the amplitude of water pressure on the negative side (decreasing side) became considerably small after 3 seconds elapsed compared to the earlier amplitude. When the behavior of these negative excess pore water pressure values is examined, it started decreasing when the amplitude of shear strain became constant, and the values were extremely reduced when the shear strain decreased. This suggests that the apparent cyclic mobility did not occur unless strain exceeded the past maximum strain, which matches the features shown by indoor soil tests.

Figure 7 shows the distribution of displacement of the laminar shear box at the time when the trend in the shear strain increased (2.77 seconds) and when it decreased (3.985 seconds) as indicated by the plots of Figure 6. It can be known from the figure that the ground was deformed in the primary mode at 2.77 seconds and in the secondary mode at 3.985 seconds.

Very small white noise was applied before the sine wave was given to verify the transfer function of the ground, and it was identified that the natural frequency of the whole ground was 15 Hz. The shear modulus G of the ground was estimated from the natural frequency. If it is reduced to 1/10 by the shear strain that reaches 1% during shaking, the natural frequency at this time is 4.8 Hz. Thus, it is presumed that the reason why the strain at 3 seconds from the start of shaking and later decreased is the reduction in ground rigidity due to the increase in shear strain during shaking to switch the vibration mode from primary to secondary. Though Figures 5 and 6 focus on a particular layer (GL-250 mm) of the 700 mm thick model ground for consideration, the key to understand the behavior is to examine and consider the behavior of the whole ground model involving the upper and lower layers.

Figure 8 shows the waveforms of shear strain and excess pore water pressure ratio at GL-250 mm in Case 2. Each of the shaking frequency, amplitude, and wavenumber in Case 2 was the same as in Case 1 and the shear strain waveform in Case 1 at the same depth is also shown for comparison.

The shear strain gradually increased until the time after 3 seconds along the horizontal axis and then kept an almost constant amplitude later. However, it drifted to the minus side as a whole. Compared with the waveform at the same depth in Case 1, the strain value did not exceed the peak value (around 2.5 to 3.0 seconds) in Case 1 through all the time.

As for the waveform of the excess pore water pressure ratio, large negative excess pore water pressure as found in Case 1, which was apparently recognized as cyclic mobility, did not occur, and the positive excess pore water pressure was accumulated almost

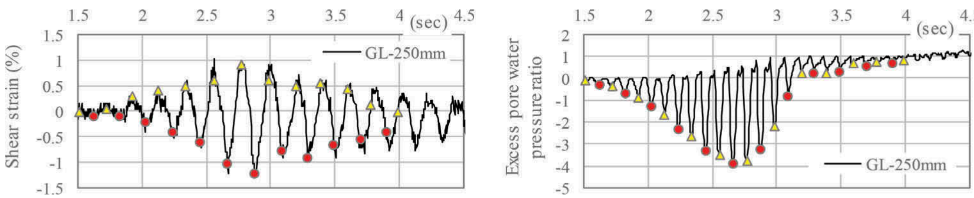

Figure 6. Time history of shear strain and excess pore water pressure ratio at Case1 (The section of t = 1.5 to 4.5 sec of Figure 6 is enlarged.)

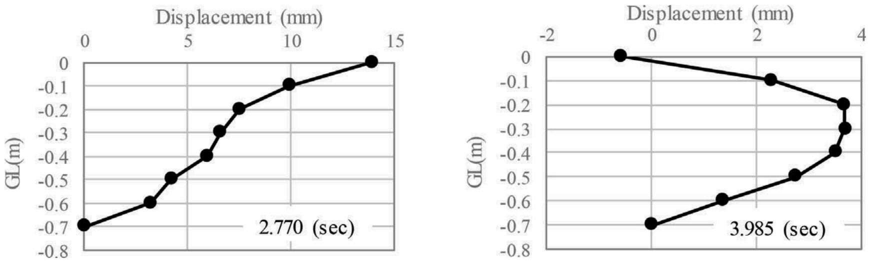

Figure 7. Depth distribution of horizontal displacement at shear box

monotonously. It is understood from this reading that, even if excess pore water pressure generated in the previous shaking case was dissipated and consolidation was performed, apparent cyclic mobility did not occur unless exceeding the maximum shear strain that was generated in the past. More specifically, when the actual ground is reviewed, the strain generated in the past seismic history is considered to affect the response to future earthquakes. Such behavior of dense sand ground after reconsolidation following liquefaction has not been discussed in detail, and the magnitude of shear strain that causes large negative excess pore water pressure and the effect of the strain history on it need to be studied afterwards by shaking table tests and indoor tests.

Figure 9 enlarges the horizontal axis of Figure 8 from 2.0 seconds to 4.0 seconds. When the amplitude was less than 0.5%, the negative peaks of the excess pore water pressure ratio did not correspond to the peaks of shear strain, and the occurrence of cyclic mobility was not recognized. However, when the strain amplitude exceeded 0.5%, the negative peaks of water pressure matched the peaks of the strain, and the occurrence of cyclic mobility was presumed. Note that it is difficult to consider that the negative water pressure to this extent has a large effect on the ground structure such as recovery of rigidity. In other words, the phenomenon itself is regarded as cyclic mobility but it is not a significant behavior enough to have an effect on ground deformation and other events.

Figure 10 shows the time history of shear strain and excess pore water pressure ratio in Case 3. While the shaking frequency and the wavenumber in Case 3 were the same as in Cases 1 and 2, the amplitude was doubled. The shear strain was amplified as the acceleration amplitude increased and kept a steady-state amplitude slightly above 3% after 3 seconds. This strain value exceeded the maximum strain value of Cases 1 and 2, and it was the largest value among the measurements, including previous shaking history.

While shear strain behaved like this, the amplitude on the negative side of the excess pore water pressure ratio increased in the first 3 seconds, and apparent cyclic mobility was exhibited. The reason is that the maximum shear strain of Case 1 was not surpassed in Case 2 but in Case 3. Meanwhile, when the shear strain amplitude came into the steady state, the amplitude of the excess pore water pressure ratio on the negative side decreased in the same behavior as shown in Cases 1 and 2.

It has been described until now that remarkable cyclic mobility did not occur if the shear strain value did not reach the previous maximum value regardless of whether the maximum value was produced during the current shaking or in the different shaking cases in the past, and the amount of the generated excess pore

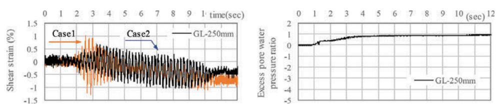

Figure 8. Time history of shear strain and excess pore water pressure ratio at Case2

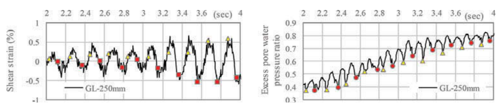

Figure 9. Time history of shear strain and excess pore water pressure ratio at Case2 (The section of t = 2.0 to 4.0 sec of Figure 8 is enlarged.)

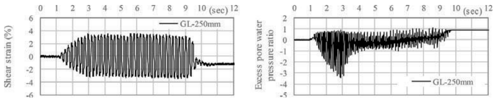

Figure 10. Time history of shear strain and excess pore water pressure ratio at Case3

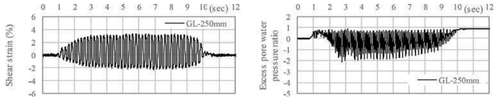

Figure 11. Time history of shear strain and excess pore water pressure ratio at Case4

water pressure on the negative side was small. However, Case 4 of shaking showed a slightly different tendency.

Figure 11 shows the time history of shear strain and excess pore water pressure ratio in Case 4. The acceleration of the shaking table in Case 4 was the same as Case 3, 800 gal. The maximum value of shear strain was 3% on the positive side and 2% on the negative side. Compared with Case 3, it was at the same level on the positive side and about 1% smaller amplitude on the negative side. It is inferred from the results obtained up to now that the degree of cyclic mobility should be small, and almost no excess pore water pressure on the negative side should be generated as verified in Case 2. In Case 4, however, a large excess pore water pressure ratio on the negative side reaching -2.0 occurred for +3% of shear strain and exceeding -1.0 for -2% of shear strain.

If the shear strain amplitude is the largest among the measurements, including past history, the amplitude value of the excess pore water pressure ratio on the negative side at the time should be about -3.0 to -4.0 as shown in Figures 5 and 10. Meanwhile, if the shear strain amplitude is almost equal to or lower than the past maximum value, the amplitude on the negative side of the excess pore water pressure should be quite small as shown in Figures 5 and 8. However, in Case 3 shown in Figure 10, about -1.0 of excess pore water pressure ratio maintained output even after the increase in shear strain amplitude was stopped, which is regarded as a different tendency from Cases 1 and 2. In addition, a large negative water pressure about -2.0 to -1.0 occurred in Case 4 as shown in Figure 11 despite equivalent or smaller values of shear strain compared with Case 3. In other words, though the shear strain in Case 4 did not exceed the past maximum value, negative excess pore water pressure occurred, and the rigidity and strength recovered.

Though the vibration given in Case 4 was identical to Case 3, the initial condition of shaking in Case 4 was different from the latter part of Case 3 in that excess pore water pressure was dissipated after Case 3 was conducted. While it was verified in Case 2, which was conducted after consolidation of Case 1, that dissipation of excess pore water pressure did not influence the occurrence of cyclic mobility, Cases 3 and 4 used twice the input acceleration amplitude of Cases 1 and 2 and showed greater values of shear strain generated. This is likely the reason why Case 4 had a different tendency from Case 2. In other words, the presence of a limit value of strain is suggested; even if shaking is maintained, cyclic mobility occurs as many times as the strain reaches a certain value, and the strain will not increase to exceed the value. On the other hand, the density was increased by 3% in Case 4 due to dissipation of excess pore water pressure after Case 3 was finished. It is also possible that the effect of the increased density was more dominative than that of the history of maximum shear strain, and the apparent cyclic mobility was exhibited without surpassing the maximum strain. More specifically, it is about $\Delta\gamma$ of triaxial tests mentioned in Figure 4. Generally speaking, denser ground has a smaller $\Delta\gamma$. When Cases 3 and 4 are compared, the magnitude of strain required to exhibit mobility in the early period of Case 4 was slightly suspended by the consolidation after Case 3 was finished, and the increase and decrease of shear strain and the increase and decrease of negative excess pore water pressure during shaking in Case 4 were influenced by $\Delta\gamma$ at the relative density compacted to 100%. It is thus judged that relatively large negative excess pore water pressure continued to occur in Case 4. Actually, the shear strain in Case 4 continued to increase until 6 to 7 seconds, though the extent was extremely small, and the negative excess pore water pressure increased or kept a certain value during the period. Once the shear strain stopped increasing to become constant, the negative excess pore water pressure seemed to decrease after that time the same as the latter section of Case 1. Though it is difficult to conclude the statement above only from the results of this experiment, these phenomena (threshold strain, $\Delta\gamma$, and magnitude and variation of negative excess pore water pressure) are important elements for prediction of seismic damage of ground, especially the residual displacement, and detailed examination is required.

We plan to conduct corresponding elemental tests and additional shaking table tests for the study on the presence of a limit value of strain amplitude and the relationship between cyclic mobility and strain amplitude more closely.

4 CONCLUSIONS

This paper has discussed cyclic mobility of dense ground according to the results of the 1 g field shaking table tests and the followings were suggested:

- Dense sand ground showed more remarkable cyclic mobility than loose sand ground, and the behavior was recognized from a small strain level.
- When the shear strain amplitude was the largest among the measurements, including past history, cyclic mobility was remarkable, and the amplitude value of the excess pore water pressure ratio on the negative side at the time was about -3.0 to -4.0.
- When the shear strain amplitude was almost equal to or lower than the maximum value in the previous history, the amplitude on the negative side of the excess pore water pressure was quite small, and this tendency was not affected by dissipation of excess pore water pressure.
- When the shear strain reached around 3%, a large negative water pressure about -2.0 to -1.0 occurred even if the shear strain amplitude generated was almost equal to or lower than the maximum value in the previous history.
- From the statements above, the possible presence of a limit value of strain is suggested; cyclic mobility occurs as many times as the strain reaches a certain value, and the strain will not increase to exceed the value.
- On the other hand, the test results seem to reflect the possible effect of the decrease in the extent of growth of the shear strain amplitude per repetition cycle ($\Delta\gamma$) due to the increase in density.

Since the limit value of strain and the extent of increase in shear strain amplitude are the important elements for prediction of seismic damage of ground, especially the residual displacement, we plan to conduct corresponding elemental tests and additional shaking table tests for the study on the presence of a limit value of strain amplitude and the relationship between cyclic mobility and strain amplitude more closely.

REFERENCES

Adalier, K., and Sharp, M. K. 2004. Embankment dam on liquefiable foundation-dynamic behavior and densification remediation, Journal of geotechnical and geoenvironmental engineering, 130(11): 1214–1224.

Castro, G. 1975. Liquefaction and cyclic mobility of saturated sands. Journal of the geotechnical engineering division, ASCE, 101, GT6: 551–569.

Elgamal, A. E., Dobry, R., Parra, E., and Yang, Z. 1998. Soil dilation and shear deformations during liquefaction, Proceedings of the 4th International conference on case histories in geotechnical engineering, St. Louis, Missouri, March 9-12: 1238–1259

Kawai, T., Jongkwan, K. and Kazama, M. 2017. Performance of various granular soils in most dense state, 3rd International conference on performance-based design in earthquake geotechnical Engineering, Vancouver, Canada.