Experimental Study on The Seismic Assessment of Pile Foundation in Volcanic Ash Ground

Takuya EGAWA, Satoshi NISHIMOTO & Koichi TOMISAWA
Civil Engineering Research Institute for Cold Region, Public Works Research Institute, Japan

15 WCEE
LISBOA 2012

SUMMARY:
Centrifugal model tests of pile foundation in volcanic ash and sandy ground were conducted to investigate the seismic behavior of pile foundation during earthquake motion. From the results of element tests of volcanic ash soil and centrifugal model tests, it was found that liquefaction phenomenon was also occurred in volcanic ash ground as well as in sandy ground. Comparison of dynamic behavior of pile in volcanic ash and sandy ground during liquefaction are discussed in the paper.

Keywords: Volcanic ash soil, liquefaction, pile foundation, centrifuge model test

1. INSTRUCTIONS

Many structure foundations built during Japan's high-growth period were constructed before aseismic design methods had been established. As a result, aging and deformation are observed in earthquake history records in some cases, and it is necessary to prepare maintenance, management and aseismic design methods based on appropriate assessment of ground properties.

In Japan, which is a volcanic country, volcanic products are accumulated over extensive areas. In Hokkaido in particular, 40% of the total land area is covered with unconsolidated volcanic products, and the types and properties of volcanic ash soil are diverse (Hokkaido Branch of Japanese Geotechnical Society, 2010). While pile foundations in volcanic ash ground are designed based on the specifications of sandy soil (Japan Road Association, 2002; Architectural Institute of Japan, 2001; Railway Technical Research Institute, 2000), volcanic ash soil has peculiar mechanical characteristics due to particle breakage (Miura and Yagi, 1997; Iitake, 1978; Takada et al., 1997). The results of past studies revealed that the bearing capacity of pile foundations in volcanic ash ground is smaller than the design value based on sandy soil (Tomisawa and Miura, 2007).

Large earthquakes in recent years have also caused liquefaction of volcanic ash ground, resulting in increased large-scale ground deformation and other types of damage (Hokkaido Branch of Japanese Geotechnical Society, 2010). Accordingly, it is desirable to clarify the seismic behavior of volcanic ash ground and establish appropriate seismic assessment methods.

This study accurately assessed the seismic behavior of volcanic ash soil, including its liquefaction mechanism, and aseismic design techniques of structure foundations. In order to investigate the seismic behavior of pile foundation during earthquake, centrifugal model tests of pile foundation in volcanic ash and sandy ground was conducted. The excess porewater pressure occurred in volcanic ash and sandy ground, bending moment of a pile foundation, response displacement of pile top obtained from the tests were compared and discussed in this paper.

2. CENTRIFUGE MODEL TEST

Figure 1 shows the outline of the centrifuge model tests. Photo 1 shows the condition of the model test after ground saturated. The tests were conducted in 50g of centrifugal acceleration. The input earthquake motion was sine wave (20 waves) with frequency 1.5Hz and maximum acceleration 750 gal (level 2 motion, Japan Road Association, 2002). Model piles with 10mm in diameter and 0.2mm in thickness were used in the tests in order to simulate the prototype piles with 500mm in diameter and 10mm of thickness (1/50 model). Strain gauges were pasted on twelve places of the inner part of the pile in order to measure the bending moment of the pile during level 2 earthquake motion. A weight of 400g (prototype 490kN) was loaded on the top of the pile to simulate the substructure loading.

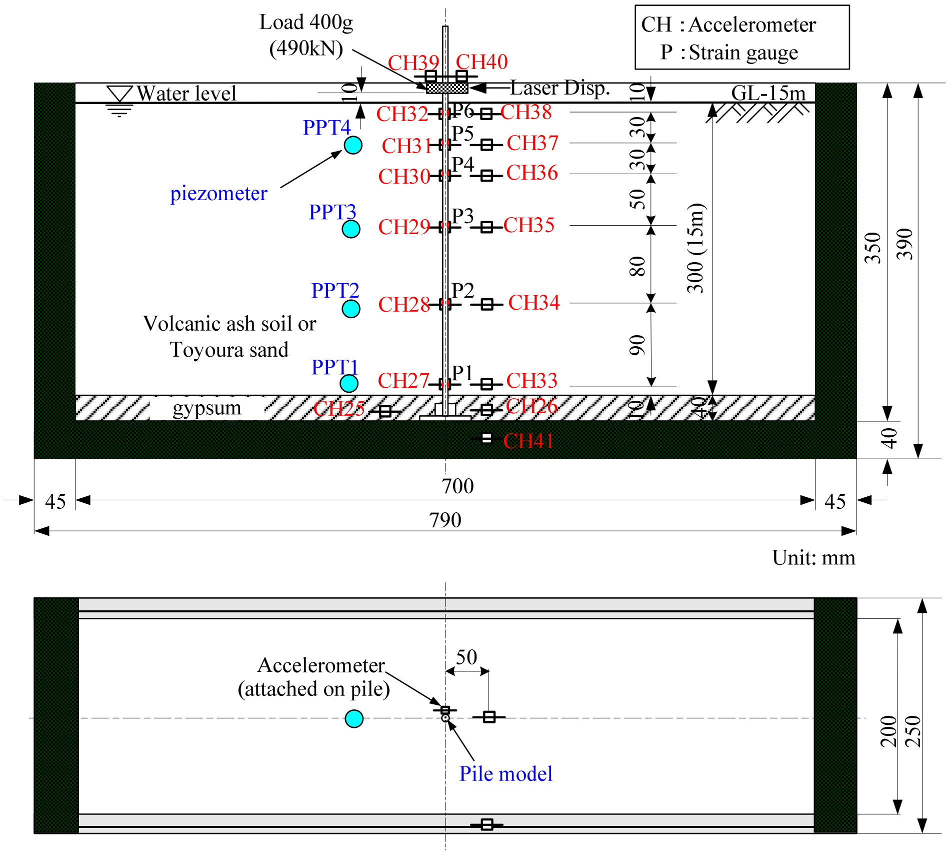
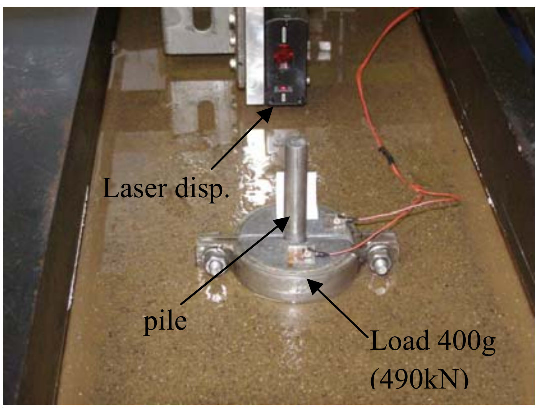
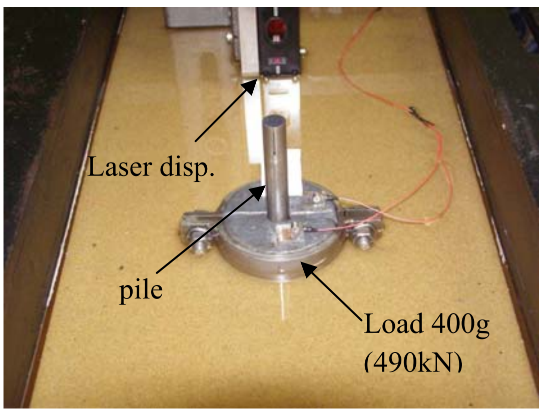


Figure 1. Outline of centrifuge model tests





(a) Case1 (Volcanic ash soil) **(b)** Case2 (Toyoura sand)

Photo 1. Condition of the models after saturation

Materials used in the tests were volcanic ash soil and Toyoura sand which their physical properties are shown in Table 1.

The volcanic ash soil material used in the test was Shikotsu pumice flow deposit (spfl) taken from Sapporo city in Hokaido area.

According to the standard of Japan Road Association, 2002 (V Seismic Design), the physical properties of volcanic ash ground shown in Table 1 ($FC \leqq 35\%$, $D_{50} \leqq 10$mm and $D_{10} \leqq 1$mm) is a material requires to evaluate the liquefaction of the ground.

Table 1. Physical properties of the materials

Materials No.	Toyoura sand	Volcanic ash soil (Spfl)
Sand fraction (%)	99.9	67.1
Silt fraction (%)	0.1	24.2
Clay fraction (%)	0.1	8.7
Fine fraction content (%)	0.1	32.9
Maximum grain size D_{max} (mm)	0.425	0.85
50% diameter on the grain size diagram D_{50} (mm)	0.164	0.143
10% diameter on the grain size diagram D_{10} (mm)	0.115	0.00699
Uniformity coefficient U_c	1.6	29.9
Coefficient of curvature U'_c	0.907	2.6
Soil particle density ρ_s (g/cm^3)	2.643	2.434

Table 2 shows the test cases and test condition. Density of Toyoura sand ground (Dr=40%, ρ_d=1.449g/cm^3) was determined by controlling the N value of sand ground to be the same as volcanic ash ground (Figure 2). Ground was saturated by using silicone fluid which its kinetic viscosity is 50 mm^2/s (KF-96-50cs). Water level was fixed at the same as ground level as shown in Figure 1.

Figure 3 shows the comparison of liquefaction strength between volcanic ash soil and Toyoura sand. It is shown that liquefaction strength of Toyoura sand is smaller then that of volcanic ash soil. This might be caused by the relative density of Toyoura sand specimen is smaller than volcanic ash soil.

Figure 4 shows the response acceleration of the soil container base obtained from the input earthquake motion level 2. The maximum of response acceleration is approximately 750 gal.

Table 2. Cases of centrifuge model test

Case name	Conditions of the models	Input earthquake motion
Case 1	Volcanic ash soil, Dr=84.7%, ρ_d=1.097 g/cm^3	Sine-wave 20 waves, Frequency 1.5Hz, Maximum acceleration 750gal
Case 2	Toyoura sand, Dr=40%, ρ_d=1.449g/cm^3	Sine-wave 20 waves, Frequency 1.5Hz, Maximum acceleration 750gal

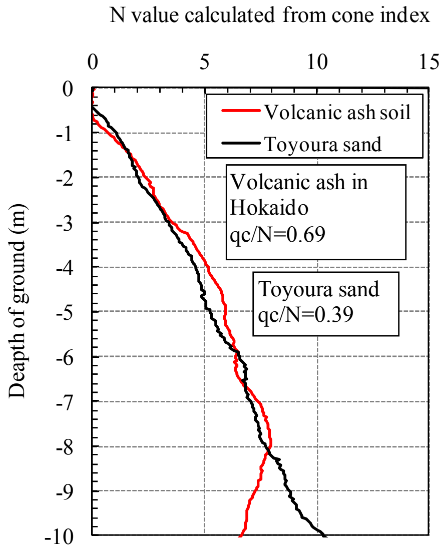


Figure 2. Results of cone penetration tests

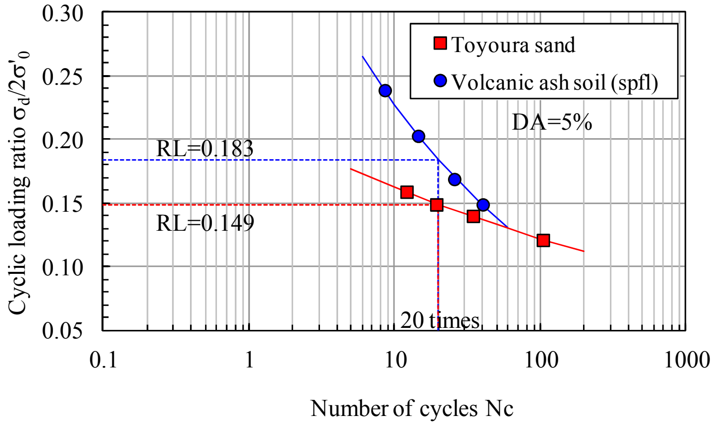
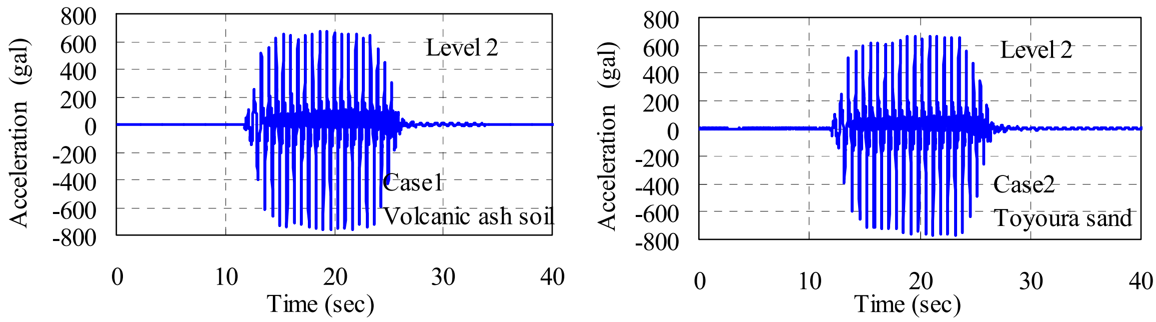


Figure 3. Comparison of liquefaction strength



(a) Case1

(b) Case2

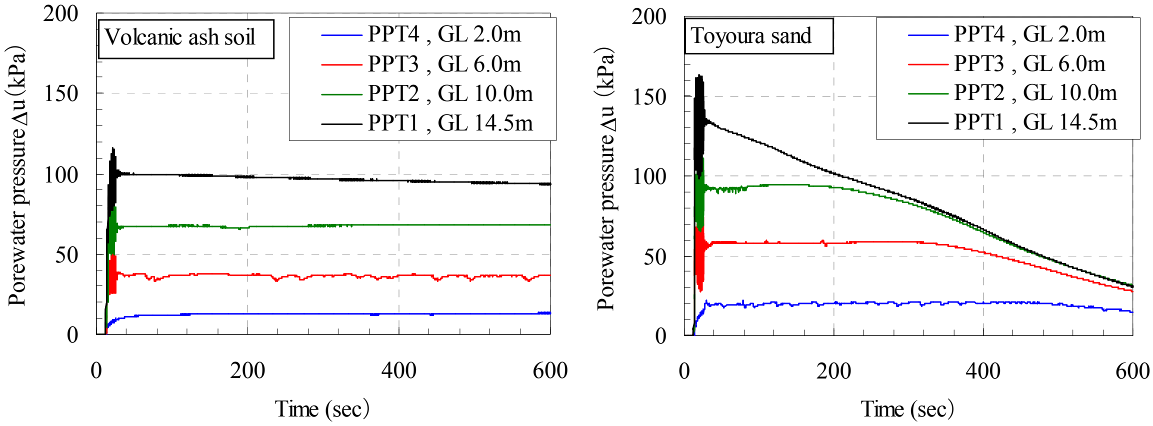
Figure 4. Response acceleration of base

3. TEST RESULTS

3.1. Excess porewater pressure

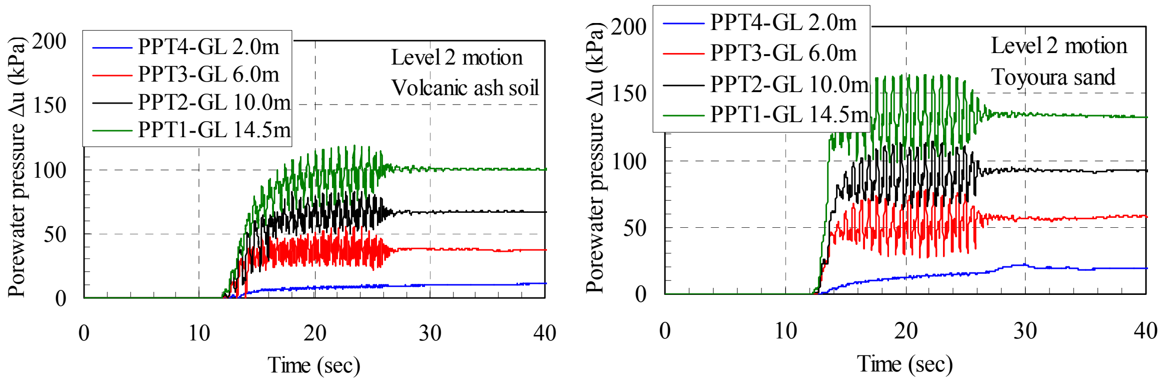
Figure 5 shows relationship between excess porewater pressure and time (from 0sec to 600sec) obtained from result of Case1 compares with result of Case2. It is shown that the excess porewater pressure occurred during earthquake motion obtained from Case 1 is lower than that of Case 2. The difference of excess porewater pressure between Case 1 and Case 2 is considered to be caused by the difference of relative density of both grounds. The dissipation process of porewater pressure of Toyoura sand ground (Case 2) is faster than volcanic ash ground (Case 1). The cause of this difference is considered to be due to the difference of fine fraction content of both materials as shown in Table 1. In Table 1, fine fraction content of volcanic ash soil is 32.9 % which is larger than the fine fraction content of Toyoura sand (0.1%). This might caused the permeability coefficient of volcanic ash soil become smaller than Toyoura sand. As a result, dissipation of excess porewater pressure of volcanic ash ground is slower than Toyoura sand ground.

Figure 6 shows relationship between excess porewater pressure and time (from 0sec to 40sec) obtained from result of Case 1 compares with result of Case 2. Figure 7 shows the ratio of excess porewater pressure and effective stress plots against time. If the ratio of $\Delta u/\sigma_v'$ is larger or equal to 1.0 ($\geqq$1.0) then the ground is judged to be liquefaction. In Figure 7, it is shown that volcanic ash ground is judged to be a liquefaction ground as well as Toyoura sand ground. However, the ratio of $\Delta u/\sigma_v$' of Case 1 is smaller than the result of Case 2. This result showed the same tendency as liquefaction strength of both materials shown in Figure 3.



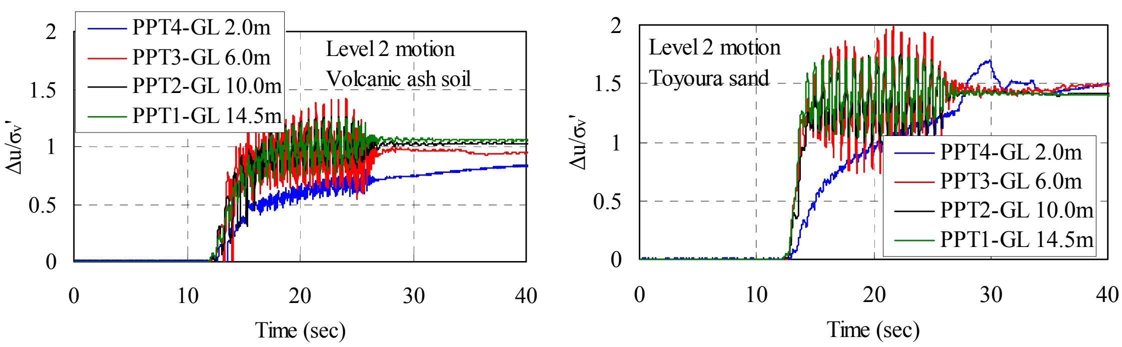
(a) Case1 (volcanic ash soil) **(b)** Case2 **(**Toyoura sand)

Figure 5. Excess porewater pressure occurred in ground during level 2 earthquake motion (up to 600 sec)



(a) Case1 (volcanic ash soil) **(b)** Case2 **(**Toyoura sand)

Figure 6. Excess porewater pressure occurred in ground during level 2 earthquake motion (up to 40 sec)

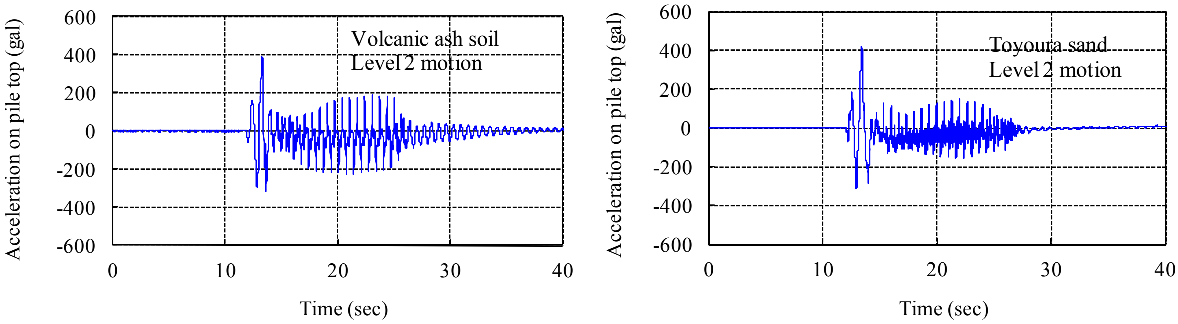


(a) Case1 (volcanic ash soil) **(b)** Case2 **(**Toyoura sand)

Figure 7. Ratio of excess porewater pressure and effective stress

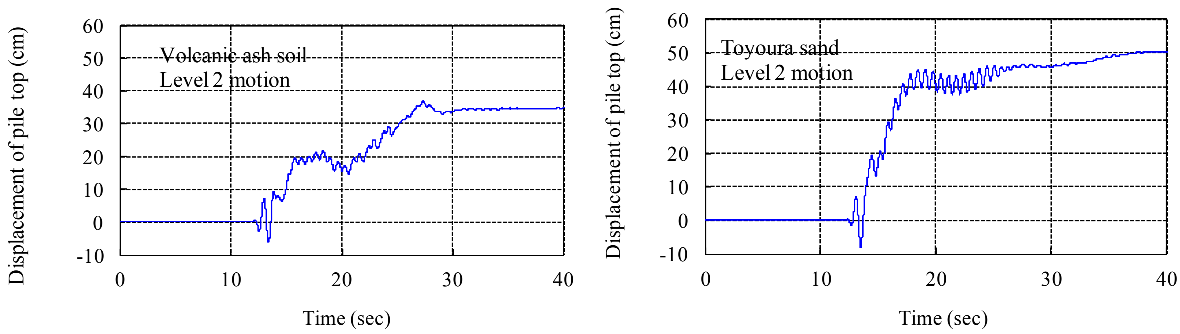
3.2. Response acceleration and displacement of pile top

Figure 8 presents response acceleration of pile top obtained from the tests. It is shown that maximum response acceleration of pile top (at 13sec) obtained from the model test of volcanic ash ground (Case 1) is smaller than the result of the test of Toyoura sand ground (Case 2). Figure 9 shows response displacement of pile top measured by laser displacement gauge. The response displacement of pile top obtained from Case 1 is smaller than result of Case 2. This result showed the same tendency with response acceleration of pile top as shown in Figure 8. However, maximum response displacement of pile top during shaking obtained from both tests presented large displacement (35cm for volcanic ash ground, 50cm for Toyoura sand ground).



(a) Case1 (volcanic ash soil) **(b)** Case2 **(**Toyoura sand)

Figure 8. Response of acceleration on pile top

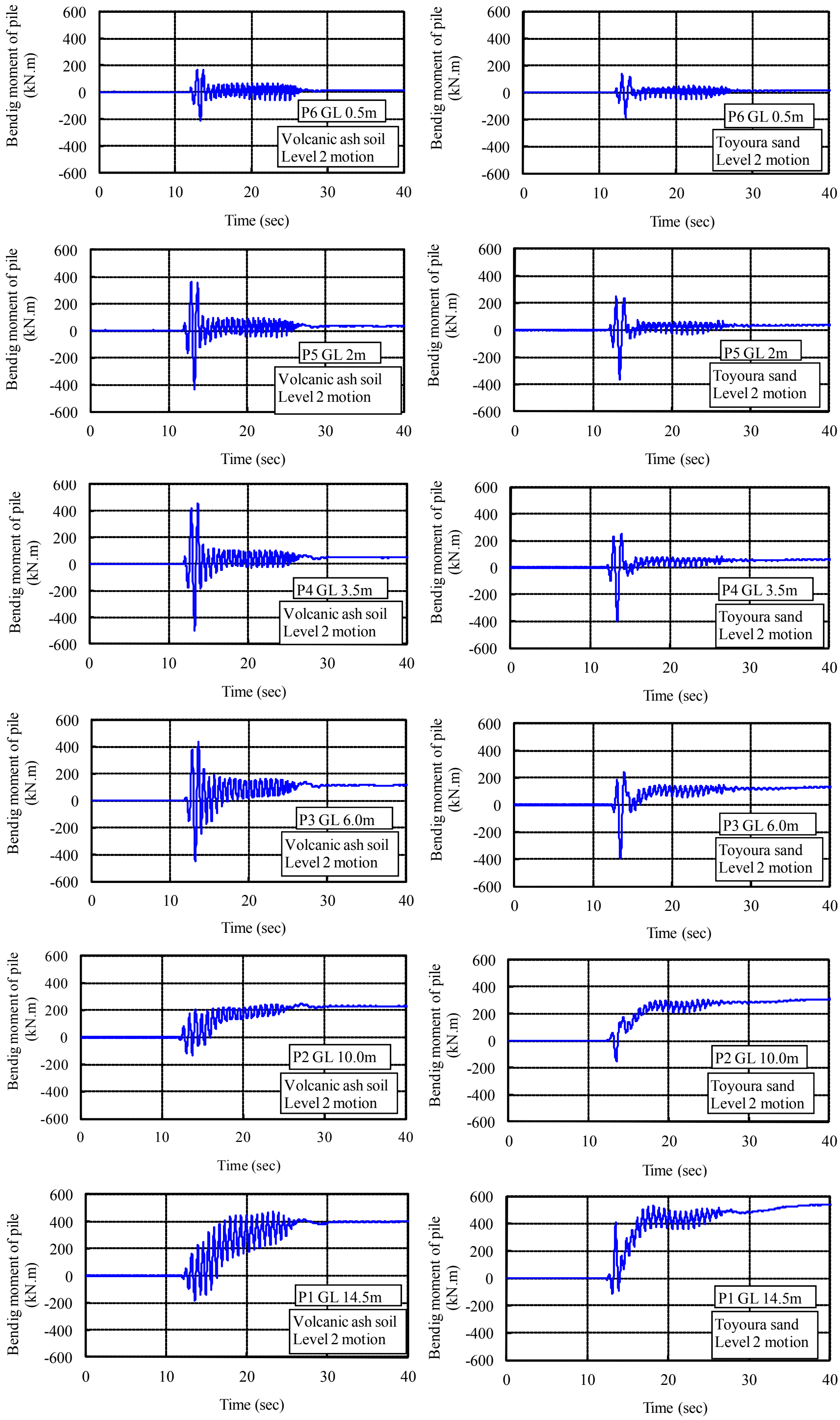


(a) Case1 (volcanic ash soil) **(b)** Case2 **(**Toyoura sand)

Figure 9. Response of displacement on pile top (measured by laser displacement gauge)

3.3. Bending moment of pile

Figure 10 shows the bending moment of pile during liquefaction of volcanic ash ground and Toyoura sand ground. Figure 11 presents the maximum value of bending moment plotted from Figure 10. It is found that the value of bending moment of pile obtained from both cases become large at the depth of 10m. This result indicated that during liquefaction, deformation of the pile concentrated at the bottom part of the pile.



(a) Case1 (volcanic ash soil) **(b)** Case2 (Toyoura sand)

Figure 10. Bending moment of pile during level 2 earthquake motion (during liquefaction)

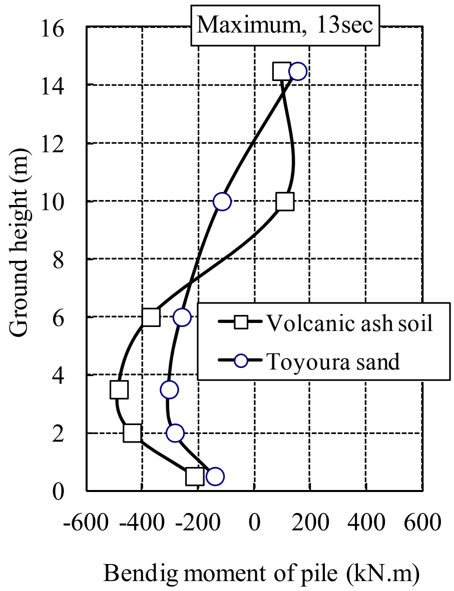


Figure 11. Maximum bending moment of pile in depth direction

4. CONCLUDING REMARKS

This study aimed to clarify the behavior of pile foundation during liquefaction of volcanic ash ground by centrifugal model tests. The centrifuge model test of Toyoura sand ground was also conducted in order to compared with the test result of volcanic ash ground. The main concluding remarks drawn from this study are summarized as follows.

(1) According to the Japanese standard, volcanic ash material (spfl) used in the model test is required to evaluate the liquefaction. The result of cyclic triaxial test showed that liquefaction strength of the volcanic ash soil is RL=0.183.
(2) Excess porewater pressure obtained from centrifuge model test showed that volcanic ash ground was liquefied during earthquake motion level 2 as well as Toyoura sand ground. However, due to relative density of volcanic ash ground is larger than Toyoura sand ground, porewater pressure occurred in volcanic ash ground is smaller than that of Toyoura sand.
(3) The maximum response displacement of pile top obtained from centrifuge model test was 35 cm for volcanic ash ground and 50 cm for Toyoura sand ground. Bending moment of pile is concentrated on bottom part of the pile.

This study introduced some fundamental data on the behavior of pile foundation and mechanism of liquefaction of volcanic ash ground by centrifuge model tests. However, to propose the countermeasure method for pile foundation during liquefaction, further experimental study and analytical study are necessary to conduct.

REFERENCES

Architectural Institute of Japan (2001). Recommendation for the Design of Building Foundations. 259-281.
Hokkaido Branch of Japanese Geotechnical Society (2010). Committee on Property and Use of Volcanic Ash Soil in Hokkaido, Volcanic ash soil for engineers, 1-80.
Hokkaido Branch of Japanese Geotechnical Society (2010). Committee on Property and Use of Volcanic Ash Soil in Hokkaido, Volcanic ash soil for engineers, 113-130.
Iitake, S. (1978). Consistency properties of Kanto loam, *Journal of the Japan Society for Civil Engineers*. **No.277:** 85-94.

Japan Road Association (2002). Specifications for Highway Bridges and instruction manual. **Vol. IV:** Substructures, 243-433.

Japan Road Association (2002). Specifications for Highway Bridges and instruction manual, **Vol. V:** Seismic Design, 1-133.

Miura, S. and Yagi, K. (1997). Particle breakage of granular volcanic ash caused by compression and shearing and its evaluation. *Journal of the Japan Society for Civil Engineers*. **No.561**: **III-36**, 257-269.

Railway Technical Research Institute (2000). Design Criteria for Railway Structures and instruction manual, foundation structures/retaining structures, 201-264.

Takada, M., Kitamura, R. and Kitada, T. (1997). Evaluation of dynamic properties of secondary Shirasu ground. *Journal of the Japan Society for Civil Engineers*, **No.561**: **III-38**, 237-244.

Tomisawa, K. and Miura, S. (2007). Bearing capacity and deformation behavior of pile in volcanic ash ground. *Journal of the Japan Society for Civil Engineers*, **C**, **Vol.63**: **No.1**, 125-139.