



MONOTONIC AND CYCLIC UNDRAINED TORSIONAL SIMPLE SHEAR BEHAVIOR OF ASO KUMAMOTO PUMICE SOIL

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ABSTRACT: The 2016 Kumamoto Earthquake Sequence caused a number of geodisasters in the Mount Aso Caldera, including a large-scale flow-type slope failure known as the Takanodai landslide. Between April and October 2016, the authors conducted a series of geotechnical damage surveys and field investigations in the landslide area, and retrieved samples of volcanic soils to be characterized in the laboratory. In an attempt to provide insights into the failure mechanism of the Takanodai landslide, this paper reports on preliminary results of monotonic and cyclic undrained torsional simple shear tests carried out on reconstituted specimens of Aso pumice soil. The test results revealed that Aso pumice soil has the tendency to show a flow-type failure behavior characterized by an abrupt develop of large shear strains exceeding 50% when subjected to monotonic torsional shear loadings. On the contrary, under cyclic shear stress condition, a progressive build-up of excess pore water pressure was observed until the stress state reached the failure envelop. It was followed by a flow-type failure with a rapid development of large shear strains. It is also found that liquefaction resistance of Aso pumice soil (evaluated as the number of loading cycles to achieve double amplitude shear strain of 7.5%) can be compared to that of medium dense (relative density of 55-60%) specimens of Toyoura sand and Christchurch sand that do not exhibit a flow-type failure behavior.

Key Words: *landslide, large shear strain, liquefaction, pumice soil, torsional shear test*

INTRODUCTION

On 16 April 2016, a moment magnitude (M_w) 7.0 earthquake struck the Island of Kyushu, Japan. Two major foreshocks (M_w 6.2 and M_w 6.0) contributed to devastation in Kumamoto City, Mashiki Town and in the mountainous areas of the Mount Aso Caldera (Dang et al., 2016; Mukunoki et al., 2016; Kiyota et al., 2017; Chiaro et al., 2017, among others). In the Mount Aso Caldera, traces of medium to large scale landslides and rock falls were frequently observed. Among many landslides, the earthquakes caused the failure of a gentle slope near the Aso Volcanological Laboratory of Kyoto University. This large-scale runout slope failure, known as the Takanodai landslide (Dang et al, 2016; Mukunoki et al., 2016), destroyed at least 7 houses and killed 5 people, threatened many other houses and blocked several roads.

As reported by Dang et al. (2016), the Kumamoto prefectural government had created “landslide hazard maps” based on national standards for landslide prevention using the following criteria: 1) steep areas at least 5m high with a slope of 30° or more, 2) areas below a rapid mountain stream that has formed as alluvial fan, and 3) areas where landslides have occurred or are at risk of occurring. As a result, the slopes in the Takanodai housing complex did not meet any of these criteria (i.e. no past evidence of landslide and the slope angle was less than 30°) and, thus, were not considered as a landslide hazard zone.

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Earliest field observations suggested that the key soil to cause the slope failure could be the orange-colored pumice soil deposit (Mukunoki et al., 2016; Chiaro et al., 2017). Moreover, the sensitivity ratio of shear strength of this volcanic soil before and after the earthquake as well as the water pressure buildup could be the cause for the flow type slope failure. However, further site investigations and laboratory studies were required to clarify the mechanisms for such a flow-type failure of gentle slope.

Between April and October 2016, as a part of an international research collaboration, the authors conducted a series of geotechnical damage surveys and field investigations in the Mount Aso Caldera, and retrieved samples of volcanic soils to be characterized in the laboratory. One of the primary objectives of this research effort was to provide new understandings into the failure mechanism of the Takanodai landslide and evaluate the liquefaction potential of the Aso pumice soil believed to be the key soil responsible for the activation of the landslide. To this scope, a series of monotonic and cyclic undrained torsional simple shear tests were carried out on reconstituted specimens of Aso pumice soil. In this paper, main field observations and initial laboratory test results are presented and discussed.

FIELD OBSERVATIONS

Field observations indicated that the Takanodai landslide was a mobile earth slide that developed into a flow-type slide on a low angle slope (around 10-15°), with a travel angle from landslide crown to debris toe of approximately 6°. As shown in Figure 1, the landslide travelled in three different directions from a common source. The landslide was about 100m in width, 300-600m in length and 5-10m in depth. Relatively large intact blocks of soil, grass and trees (Photos 1 and 2) travelled towards the toe of the slope suggesting a translational movement of the soil. Tension cracks and scarps above the head scarp adjacent the Aso Volcanological Laboratory were widely observed, indicating the potential for head scarp retrogression and/or new incipient landslides to occur. As shown in Photo 3, the slip surface was identified and traces of orange-colored pumice soil were clearly noted on it.

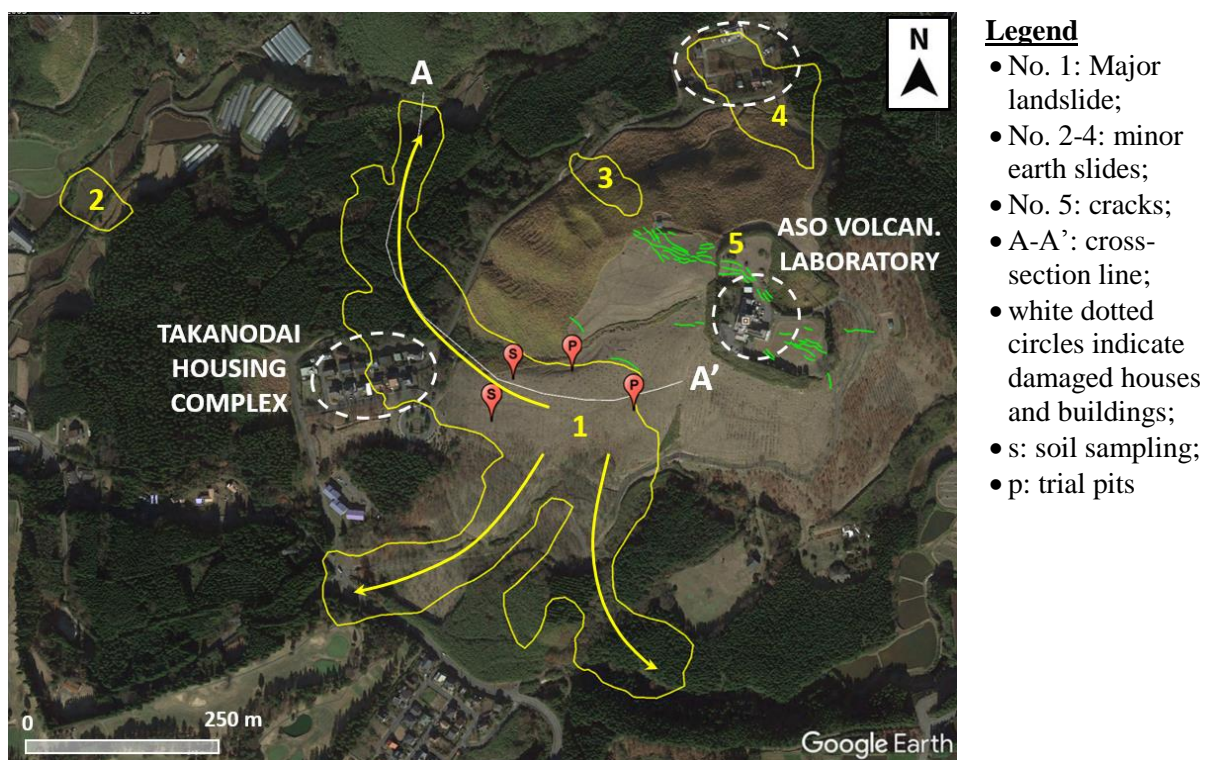


Figure 1. Google Earth image showing the location of the Takanodai landslide near the Aso Volcanological Laboratory and its impacts on houses and buildings

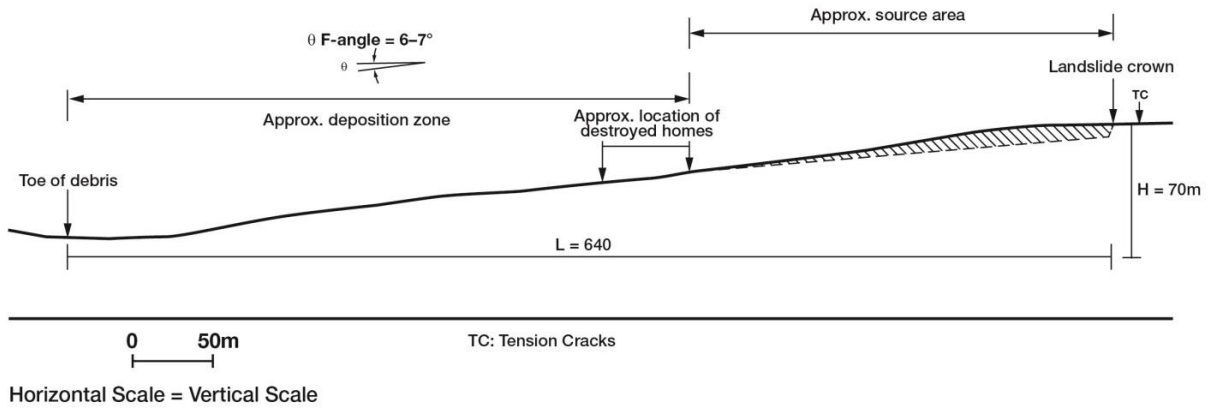


Figure 2. Cross-section A-A' through the Takanodai landslide (Chiaro et al., 2017)



Photo 1: View of the Takanodai landslide (looking downhill), with damaged houses and the large debris avalanche at Aso Ohashi Bridge visible in the distance



Photo 2: View of the Takanodai landslide (looking uphill), with the Aso Volcanological Laboratory visible in the distance

Two small trial pits were excavated across the assumed slide surface. Soil exposed in the trial pit is shown in Photo 4, and a borehole describing a typical soil profile up to the depth of 14m is reported in Figure 4. The soil consists of (1) Kuroboku volcanic ash with organic contents (black color); (2) volcanic ash Akaboku (red/brown color); (3) pumice soil; and (4) soft/weathered rock. It is worth to mention that water seepage within the pumice soil layer was clearly observed, indicating that the Akaboku ash soil (clay-like soil) is much less permeable than the pumice soil.



Photo 3. Traces of pumice soil on the slip surface

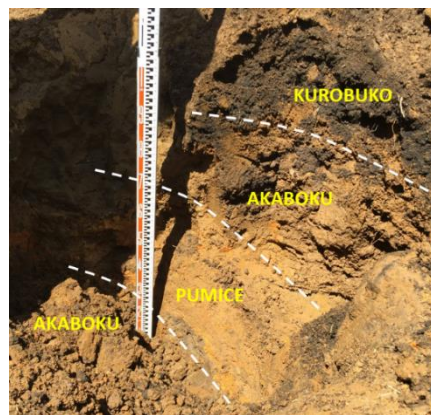
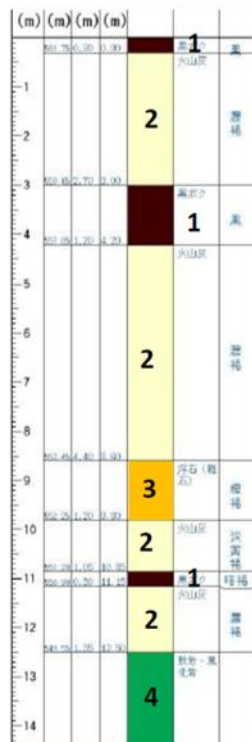


Photo 4. Soils exposed in wall of a trial pit excavated in the northern flank of the landslide

TEST APPARATUS AND PROCEDURE

Laboratory testing was carried out using the fully automated torsional apparatus shown in Figure 4, which has been developed in the Institute of Industrial Science, University of Tokyo (Kiyota et al., 2008). Such a device is capable of achieving double amplitude shear strain levels exceeding 100% by using a belt-driven torsional loading system that is connected to an AC servo motor through electromagnetic clutches and a series of reduction gears. Torque moment and axial load are measured by a two-component load cell, which is installed inside the pressure cell. The axial load and torque moment capacities are 8kN and 0.15kNm, respectively. Difference in pressure levels between the cell pressure and the pore water pressure are measured by a high-capacity differential pressure transducer (HCDPT) with a capacity of over 600kPa. On the other hand, volume change during the consolidation process is measured by low-capacity differential pressure transducer (LCDPT). A potentiometer with a wire and a pulley is employed to measure the rotation angle of the top cap and, thus, the large torsional deformation. Shear stress amplitude is controlled by a computer, which monitors the outputs from the load cell, computes the corresponding stress value and controls the device accordingly. It should be noted that the measured shear stress is corrected for the effects of the membrane force by using the hyperbolic equation proposed by Chiaro et al. (2015).



Legend

- 1) Kuroboku soil;
- 2) Akaboku soil;
- 3) Pumice soil;
- 4) Soft/weathered rock

Borehole data taken from <http://geonews.zenc-hiren.or.jp/api/2016KumamotoEQ/index.html>

Transducers:

- ① Two-component load cell
- ② Large vertical displacement transducer
- ③ High capacity differential pressure transducer (confining pressure)
- ④ Low capacity differential pressure transducer (volume change)

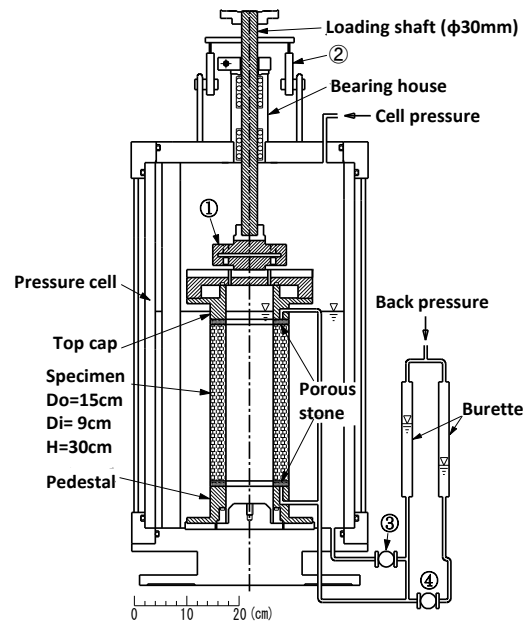


Figure 3. Typical borehole data around Aso Volcanological Laboratory

Figure 4. Torsional shear apparatus employed in this study (after Kiyota et al., 2008)

Several hollow cylindrical specimens with dimension of 150 mm in outer diameter, 90 mm in inner diameter and 300 mm in height were tested (Table 1). All the tests were performed on Aso pumice soil samples collected from the Takanodai landslide area (see sampling location in Figure 1). The soil had an in-situ dry density of approximately 0.6g/cm^3 , a natural water content of 117%, and a specific gravity of 2.3 (average value).

Similar to other volcanic soils, Aso pumice soil has a highly varying specific gravity of individual grains caused by occluded air pockets. Accordingly, conventional specimen preparation methods are not suitable (Hyodo et al., 1998). For instance, in the case of water pluviation volcanic soils tend to segregate, with some grains floating on the water surface. Using the wet tamping method, crushing of

particles may become very significant. Air pluviation also has been found to be problematic. By trial and errors, in this study, the following procedure was found most appropriate to build hollow cylindrical specimens with uniform density. To minimize the particle crushing during the specimen preparation, each specimen was prepared in 15 layers of equal height by spooning the soil (prepared at its natural water content) into the mold and subsequently applying a very gentle tamping until a target density was achieved.

Due to high internal void ratio, saturation of volcanic soil requires a rigorous de-airing process (Hyodo et al., 1998; Pender et al., 2006). In this study, therefore, the double vacuum method (Ampadu, 1991) was employed before and during percolation of water into the specimen. By doing so, and applying a back pressure of 200kPa, a high degree of saturation was ensured i.e. Skempton's 'B' values were greater than 0.95. After completing the saturation process, the specimens were isotropically consolidated by increasing the effective mean stress state (p_0') up to 100kPa (representative of in-situ stress conditions).

As summarized in Table 1, several strain-controlled (shear strain rate of 0.5 %/min) cyclic and monotonic undrained torsional simple shear tests were performed. In the case of cyclic tests, the loading direction was reversed when the amplitude of shear stress reached the target value. Whereas, in some monotonic tests, the static shear corresponding to the in-situ sloping stress condition (i.e. shear stress component induced by slope inclination) was applied by means of drained monotonic torsional shear loading before undrained monotonic shearing (Chiaro et al., 2012). In all the tests, during the undrained shearing phase, the vertical displacement of the top cap was mechanically prevented with the aim to simulate as much as possible the simple shear condition that ground undergoes during horizontal seismic excitations.

Table 1. List of undrained cyclic torsional simple shear tests performed in this study

Test	Type	Dry density (g/cm ³)	Shear stresses (kPa)		Mean effective stress (kPa)
			τ_{cyclic}	τ_{static}	p_0'
ASO #1	Monotonic	0.56	---	0	100
ASO #2		0.60	---	25	100
ASO #3	Cyclic	0.63	20	0	100
ASO #4		0.58	25	0	100

TEST RESULTS AND DISCUSSION

Undrained monotonic torsional simple shear tests

Figures 5 and 6 show the results of monotonic undrained torsional simple shear tests, in terms of effective stress paths and stress-strain relationships. For the sake of comparison, typical monotonic torsional shear behavior of Toyoura sand is illustrated in Figure 7.

As shown by Figs. 5a and 6a, under the stress conditions used in this study, Aso pumice soil exhibited a purely contractive behavior, during which the mean effective stress (p') decreases while shear stress (τ) progressively increases up to a transient peak stress (in correspondence of the instability line or IL line). The peak stress state is accompanied by an unstable behavior (Lade, 1993) with a sudden loss of strength and a large development of pore water pressure and shear strains.

In addition, by looking at the stress-strain responses (Figs. 5b and 6b), strain-softening behavior (i.e. decrease in shear strength owing to shear strain increase) can be observed after the transient peak stress. Moreover, Aso pumice soil showed a tendency to deform under a nearly constant stress (residual shear strength).

In order to investigate the effects of slope inclination on the undrained response of Aso pumice soil, a specimen was pre-sheared by applying 25kPa of initial static shear (Fig. 6). The overall, behavior of Aso pumice soil subjected to monotonic shearing loading did not change significantly, in the sense that it remained purely contractive and strain-softening followed by large strain development

was observed after the peak stress state. However, an increase in both the peak shear strength and the residual strength was noticed.

When compared with the response of Toyoura sand shown in Fig. 7, it seems that Aso pumice soil behaves similarly to a medium-dense sand specimen. However, some difference can be seen. For example, while a medium-dense sand specimen usually behaves first contractive and then dilative (p' increases after reaching the phase transformation line), Aso pumice soil behaves purely contractive and the shear stress suddenly drops after reaching the failure envelop line (FL). Therefore a clear PTL line cannot be established for Aso pumice soil. Moreover, while after the quasi-steady stage medium-dense sand experiences a limited strain development, Aso pumice soil undergoes large strain development. This different behavior may be attributed to: 1) the low crushability of sand particles (hard grained material) as compared to that of Aso pumice (crushable volcanic soil); and 2) different soil structure.

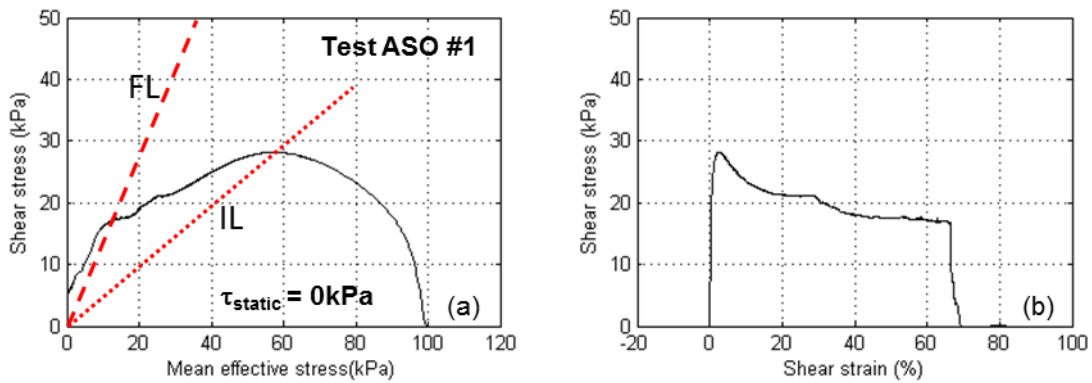


Figure 5. Test ASO #1: a) effective stress path; and b) stress strain relationship

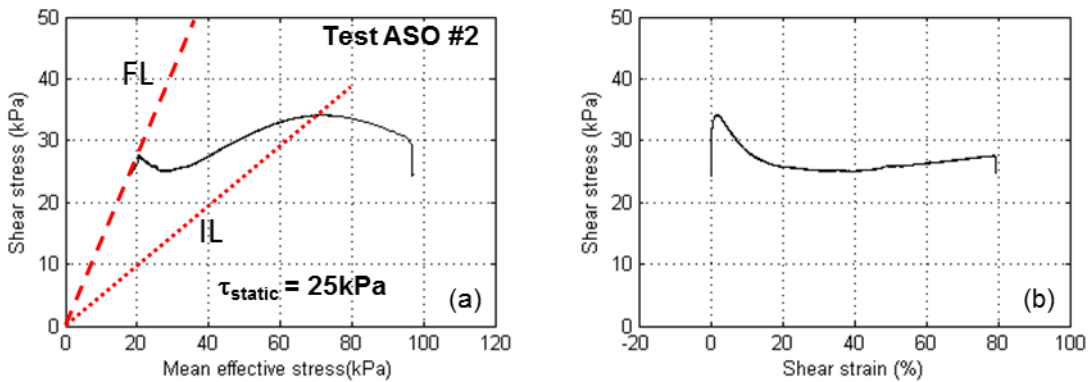


Figure 6. Test ASO #2: a) effective stress path; and b) stress strain relationship

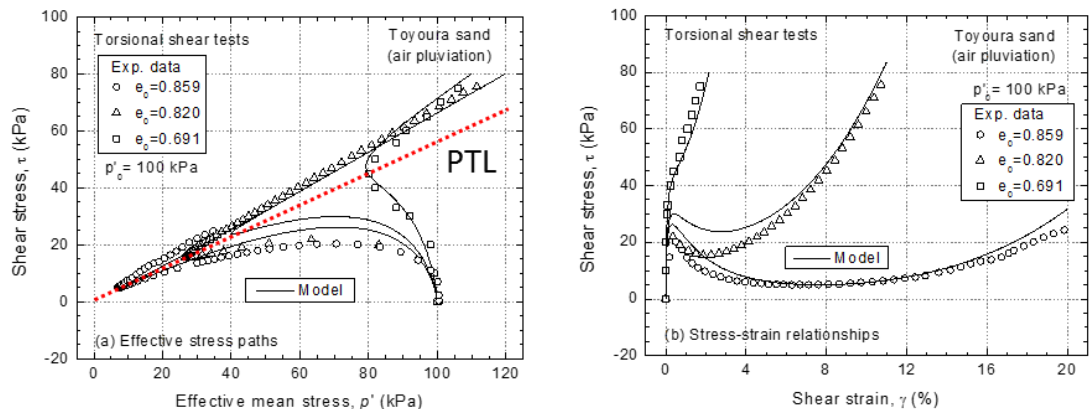


Figure 7. Monotonic undrained torsional shear behavior of Toyoura sand (Chiaro et al., 2013)

Undrained cyclic torsional simple shear tests

Undrained shear tests were performed using two level of cyclic stress ratio ($CSR = \tau_{cyclic}/p_0'$) of 0.20 and 0.25. Figures 8 and 9 show typical effective stress paths with their associate stress-strain plots. In both tests, Aso pumice soil showed a cyclic mobility failure type where the effective stress path progressively moves toward the zero- p' conditions but the cyclic shear strain increases up to large values. In particular, initially the shear strain were very small and then suddenly exceed values of double amplitude shear strain (γ_{DA}) of 50% or more during the last few cycles of loading as p' approached zero. Hyodo et al. (1998) presented similar behavior for the crushable Shirasu soil (a volcanic soil from southern Kyushu) subjected to cyclic triaxial shearing.

It is noted that, the undrained cyclic behavior observed in this study for Aso pumice soil is quite different from that of a liquefied sandy soil for which large strain are developed under stress condition of zero- p' and zero shear stress (Fig. 10).

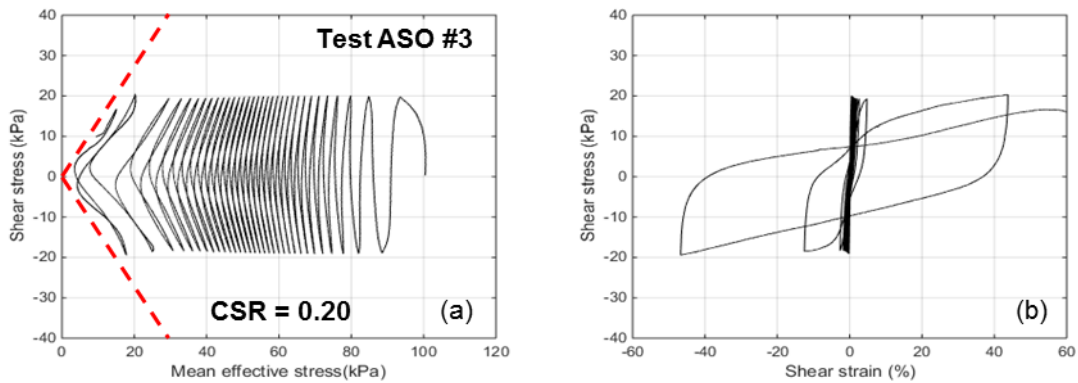


Figure 8. Test ASO #3: a) effective stress path; and b) stress strain relationship

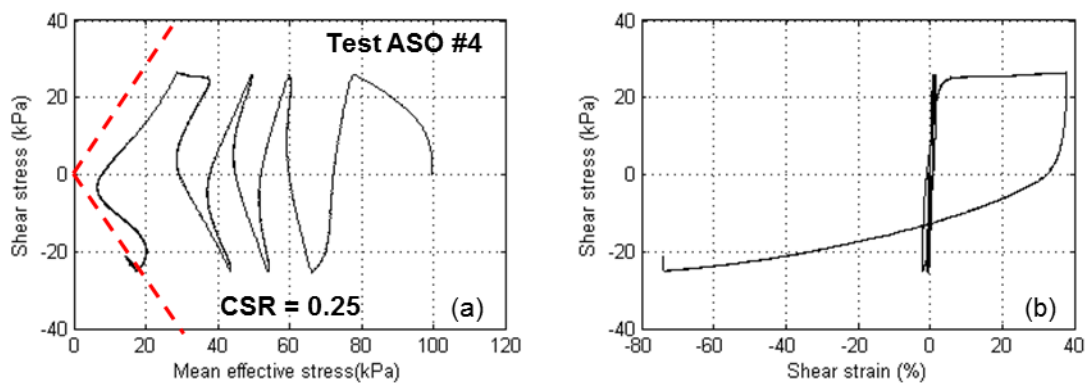


Figure 9. Test ASO #4: a) effective stress path; and b) stress strain relationship

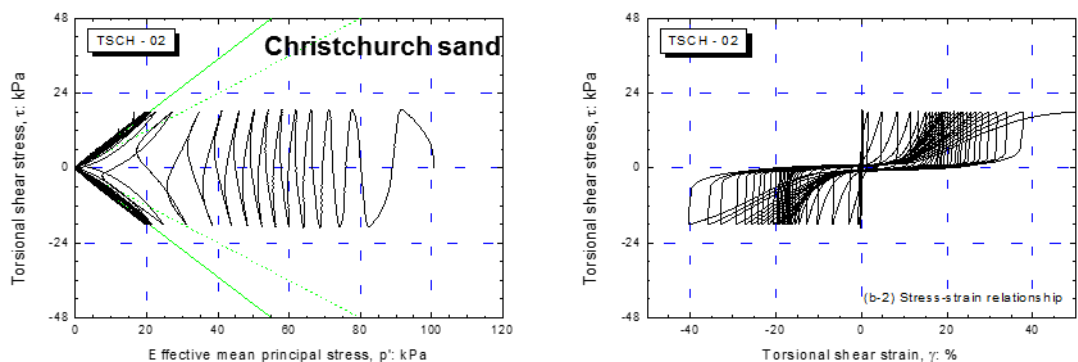


Figure 10. Cyclic undrained torsional shear behavior of Christchurch sand (Chiaro et al., 2015)

For completeness, the deformation characteristics of Aso pumice specimen at various states of shearing during Test ASO #3 are shown in Photo 5. At state 2, corresponding to a shear strain (γ) of about 7.5 %, the deformation was uniform throughout the specimen height. At state 3 ($\gamma = 15 %$), the deformation was still rather uniform. However, at stage 4 ($\gamma = 30 %$), it appears that strain localization started to develop in the upper part of the specimen.

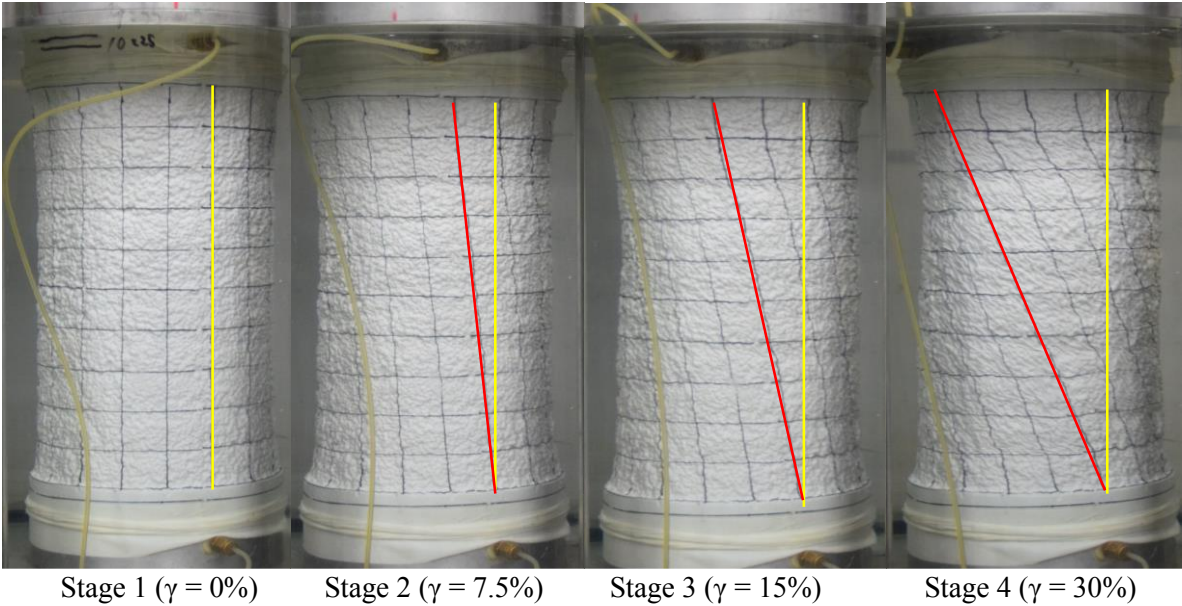


Photo 5. Typical deformation of Aso pumice specimens in cyclic undrained torsional shear tests

Resistance against cyclic strain accumulation

Conventionally, the resistance to liquefaction or to cyclic strain accumulation of soils is evaluated as the number of cycles to develop a specific amount of double amplitude shear strain (γ_{DA}) during cyclic loading. Accordingly, Fig. 11 reports the liquefaction resistance of Aso pumice soil corresponding to a at double amplitude shear strain (γ_{DA}) of 7.5%, which is equivalent a double amplitude shear strain of 5% in triaxial tests. In the same figure, liquefaction resistances of Toyoura sand and Christchurch sand (both hard grained soils) are reported for comparison.

Despite the differences in cyclic behavior previously described, it appears that the liquefaction resistance of crushable Aso Pumice soil can be closely related to that of less-crushable medium-dense sandy soils.

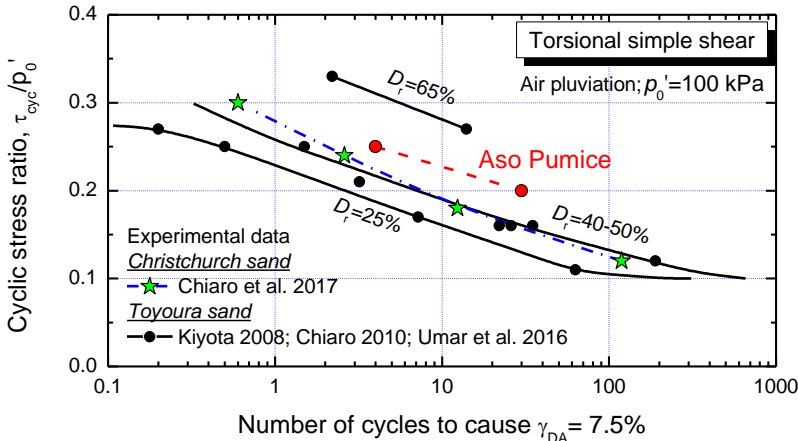


Figure 11. Cyclic strength curves for double strain amplitude 7.5%

Earthquake-induced failure of problematic volcanic soils is very common in Japan (e.g. Miura and Yagi, 1995; Ishihara and Harada, 1996) and, in particular, failures are known to have occurred in Shirasu, which is a volcanic soil frequently found in large areas of southern Kyushu (Yamanouchi, 1968). However, to date, very limited research has been undertaken to comprehensively understand the cyclic undrained behavior of such problematic volcanic soils and their failure mechanisms. In addition, due to its peculiar properties i.e. high crushability and compressibility, high internal void ratio (Wesley et al., 1999 and Wesley, 2001) existing correlating to estimate the liquefaction characteristics of hard grained (less crushable) soils are not applicable to highly crushable volcanic soils. Therefore, it is essential to conduct detailed and comprehensive investigations that enable the understanding of the failure mechanism of volcanic soil under cyclic shear loading and establishment of soil-specific liquefaction criteria suitable for this problematic soils.

CONCLUSIONS

Following reconnaissance damage surveys, a thin deposit of crushable pumice soil was understood to be responsible for the Takanodai landslide occurred in Minami Aso during the 2016 Kumamoto earthquakes sequence. However, there were discussions in regard to the potential failure mechanisms. Therefore, in an attempt to provide insights into the failure mechanisms of the Takanodai landslide, several monotonic and cyclic undrained torsional simple shear tests were carried out in this study on reconstituted specimens of Aso pumice soil.

The test results revealed that Aso pumice soil has the tendency to show a post-peak flow-type failure behavior characterized by an abrupt develop of large shear strains exceeding 50% or more when subjected to monotonic torsional shear loadings.

On the contrary, under cyclic shear stress condition, a progressive build-up of excess pore water pressure was observed until the stress state reached the failure envelop. It was followed by a flow-type failure with a rapid development of large shear strains.

Moreover, it is found that liquefaction resistance of Aso pumice soil (evaluated as the number of loading cycles to achieve double amplitude shear strain of 7.5%) can be compared to that of medium-dense (relative density of 55-60%) specimens of Toyoura sand and Christchurch sand (less crushable materials) that in a dense density state do not exhibit a flow-type failure behavior.

This study also highlighted the need for detailed and comprehensive investigations that enable the understanding of the failure mechanisms of volcanic soil under cyclic shear loading and the establishment of soil-specific liquefaction criteria suitable for problematic volcanic soils.

REFERENCES

- Ampadu, S. I. K. (1991): Undrained behavior of kaolin in torsional simple shear, *Ph.D. Thesis*, Dept. of Civil Engineering, University of Tokyo, Japan.
- Chiaro, G., Alexander, G., Brabhaharan, P., Massey C., Koseki J., Yamada, S. and Aoyagi, Y. (2017): Reconnaissance report on geotechnical and geological aspects of the 2016 Kumamoto Earthquake, Japan, *Bulletin of the New Zealand Society for Earthquake Engineering* (accepted).
- Chiaro G., Koseki J. & De Silva L.I.N. (2013): A density- and stress-dependent elasto-plastic model for sands subjected to monotonic torsional shear loading, *SEAGS Geotechnical Engineering Journal*, **44**(2): 18-26.
- Chiaro, G., Kiyota, T. and Miyamoto, H. (2015): Large deformation properties of reconstituted Christchurch sand subjected to undrained cyclic torsional simple shear loading, *Proc. of the 2015 NZSEE Annual Conference*, Apr. 10-12, Rotorua, New Zealand, 529-536.
- Chiaro, G., Koseki, J. and Sato, T. (2012): Effects of initial static shear on liquefaction and large deformation properties of loose saturated Toyoura sand in undrained cyclic torsional shear, *Soils and Foundations*, **52**(3): 498-510.
- Ishihara, K. and Harada, K. (1996): Cyclic behavior of partially saturated collapsible soils subjected to water permeation. In Ground failures under seismic conditions, *ASCE Geotechnical Special Publication*, **44**: 34-50.

- Dang, K., Sassa, K., Fukuoka, H., Sakai, N., Sato, Y., Takara, K., Quang, L. H., Loi, D. H., Tien, P.V. and Ha, N.D. (2016): Mechanism of two rapid and long runout landslides in the 16 April 2016 Kumamoto earthquake using a ring-shear apparatus and computer simulation (LS-RAPID), *Landslides*, **13**(6): 1525–1534.
- Hyodo, M., Hyde, A. F. L. and Aramaki, N. (1998): Liquefaction of crushable soils, *Geotechnique*, **48**(4): 527-543.
- Kiyota, T., Sato, T., Koseki, J. and Mohammad, A. (2008): Behavior of liquefied sands under extremely large strain levels in cyclic torsional shear tests, *Soils and Foundations*, **48**(5), 727-739.
- Lade, P.V. (1993): Initiation of static instability in the submarine Nerlerk berm, *Canadian Geotechnical Journal*, **30**(6): 895-904.
- Miura, S. and Yagi, K. (1995). The effect of stress history on the dynamic characteristics of a coarse-grained volcanic soil. *Proc. Of the Symposium on Properties of Volcanic Ash Soil related to Design and Construction, Japanese Geotechnical Society*, 229-236 (in Japanese).
- Mukunoki, T., Kasama, T., Murakami, S., Ikemi, H., Ishikura, R., Fujikawa, T., Yasufuka, N. and Kitazono, Y. (2016) : Reconnaissance report on geotechnical damage caused by an earthquake with JMA seismic intensity 7 twice in 28h, Kumamoto, Japan. *Soils and Foundation*, **56**(6): 947-964.
- Pender, M. J., Wesley, L. D., Larkin, T. J. and Pranjoto, S. (2006): Geotechnical properties of a pumice sand, *Soil and Foundations*, **46**(1), 69-81.
- Wesley, L. D. (2001): Determination of specific gravity and void ratio of pumice materials, *Geotechnical Testing Journal*, **24**(4), 418–422.
- Wesley, L. D., Meyer, V. M., Pranjoto, S., Pender, M. J., Larkin, T. J. and Duske, G. C. (1999): Engineering properties of pumice sand, *Proc. of the 8th Australia-NZ Conference on 'Geomechanics'*, Hobart, TAS, **2**: 901–908.
- Yamanouchi, T. (1968): Ground failure due to the Ebino earthquake. Report of the JSSMFE Technical Committee on Shirasu. *Tsuchi-to-kiso JSSMFE*, **16**(9): 47-59.