Dynamic deformation characteristics of pumice sand

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ABSTRACT: To understand the strain-dependent shear modulus and damping ratio of reconstituted pumice sand samples, a series of dynamic deformation tests was performed using hollow cylindrical torsional shear test apparatus. The effects of confining pressure level and grading characteristics (or fines content) were investigated, and sieve analyses were performed after each test to check if particle crushing occurred. The results showed that the dynamic deformation characteristics of pumice sands were comparable with those of hard-grained sands, e.g., initial shear modulus and shear modulus ratio increased and damping ratio decreased with increase in confining pressure. The effects of fines content were consistent with those observed in other volcanic soils, e.g. Shirasu in Japan. Under undrained cyclic loading, reconstituted pumice sands did not undergo significant particle crushing, resulting in trends in strain-dependent modulus and damping ratio which were similar to those of hard-grained sands.

1 INTRODUCTION

Because of New Zealand's tectonic location, the seismic-resistant design of soil structures requires a clear understanding of the soil properties and behaviour under earthquake loading. Large-scale earthquakes, such as the recent Canterbury earthquake sequence, showed that amplified ground shaking from site effects was responsible for many of the ground damage observed. It is therefore important to consider ground shaking characteristics and local site amplifications in estimating ground response and in defining structural design loads. Toward this end, significant research effort has focused into understanding the dynamic properties of sandy soils. However, nearly all the work has been directed towards the properties of hard-grained (quartz) sands; very little research has been done on the dynamic characteristics of volcanically-derived sands.

New Zealand's active geologic past has resulted in widespread deposits of volcanic soils throughout the country. Pumice deposits are found in several areas of the North Island. They originated from a series of volcanic eruptions centred in the Taupo and Rotorua regions, called the "Taupo Volcanic Zone". The pumice material has been distributed initially by the explosive power of the eruptions and associated airborne transport; this has been followed by erosion and river transport. Presently, pumice deposits exist mainly as deep sand layers in river valleys and flood plains, but are also found as coarse gravel deposits in hilly areas. Although they do not cover wide areas, their concentration in river valleys and flood plains means they tend to coincide with areas of considerable human activity and development. Thus, they are frequently encountered in engineering projects and their evaluation is a matter of considerable geotechnical interest.

Pumice sand particles may be readily crushed against a hard surface by finger-nail pressure presumably, in part at least, because the particles have internal voids. Because of their lightweight, highly crushable and compressible nature, they are problematic from engineering and construction viewpoint. Moreover, no information is available whether empirical correlations and procedures derived for hard grained soils are applicable to pumice deposits because there has been very little research done to examine the dynamic characteristics of pumice.

The strain dependent shear modulus and damping characteristics of soils are very important input parameters in performing seismic ground response analyses. In this paper, we examined the dynamic deformation characteristics of reconstituted pumice sand with different grading properties and subjected to different confining pressures using a torsional shear test apparatus. Sieve analyses were also performed in order to confirm if particle crushing occurred after the tests. The results of the tests were compared with those observed in Toyoura sand, a hard-grained quartz-based sand.

2 TEST MATERIALS AND EXPERIMENTAL PROCEDURE

The material used in this study was the commercially available pumice sand, which has been used extensively in the Geomechanics Laboratory of the University of Auckland (e.g., Pender, 2006; Pender et al. 2006; Kikkawa, 2008; Kikkawa et al., 2009). In the tests, particles with grain size between 0.074mm – 2mm were used (referred herein as Type 1 pumice). To represent the effect of crushed particles on the dynamic deformation characteristics, some pumice particles were crushed and only particles smaller than 1mm were used (referred to as Type 2 pumice). This resulted in pumice sample with fines content, F_C=53.8%. The properties of the soils, obtained using methods based on Japanese Geotechnical Society (JGS) Standards (2000) are shown in Table 1, while the grain size distribution curves are illustrated in Figure 1. For comparison purposes, the properties of Toyoura sand, a sub-angular natural sand commonly used in experimental tests in Japan, are also indicated. Note that pumice is much lighter than Toyoura sand, and its specific gravity increases with decreasing particle size, consistent with the observation of Kikkawa et al. (2011). Moreover, the maximum and minimum void ratios of pumice are almost thrice of those of Toyoura sand.

Table 1. Engineering properties of samples used

Material	Specific Gravity	Maximum void ratio	Minimum void ratio
Pumice sand (Type 1)	1.95	2.584	1.762
Pumice sand (Type 2)	2.30		
Toyoura sand	2.64	0.968	0.628

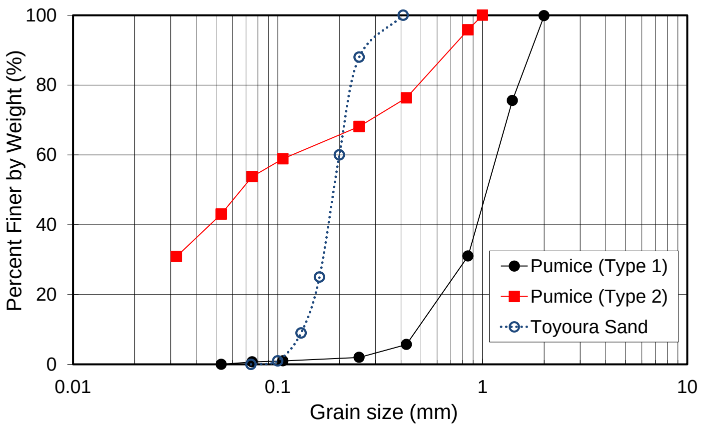
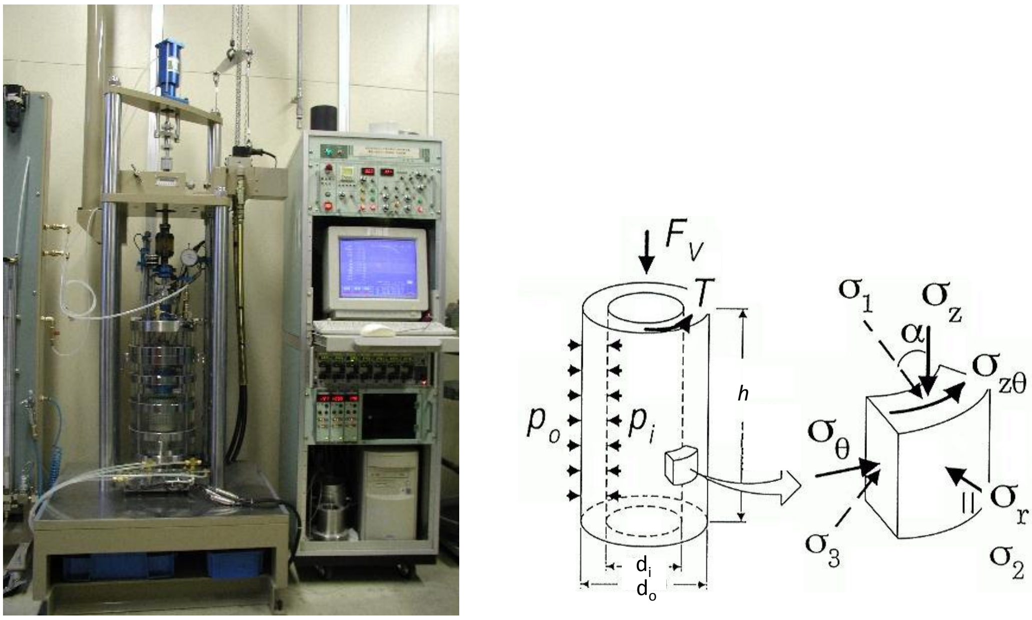


Figure 1. Grain size distribution curves of the samples used in the tests.

To investigate the strain-dependent shear modulus and damping ratio of reconstituted pumice sand, a series of undrained cyclic tests were performed. For this purpose, the hollow cylindrical torsional shear apparatus shown in Figure 2(a) was employed. The loading system consisted of a servo-controlled hydraulic actuator. The amplitude and frequency of load and wave form were controlled by a servo-controller. The data logging was done through an automatic computer-controlled system. Torsional displacement was measured by two gap sensors installed at the top cap and a potentiometer located outside the cell. The gap sensors had a sensitivity of $1\text{x}10^{-4}$ mm and were used to measure the



(a) (b)

Figure 2. Hollow cylindrical torsional shear tests: (a) apparatus used; and (b) stresses induced in a hollow cylindrical torsional shear element

displacements at shear strain levels less than 0.2%. For shear strain levels greater than 0.2%, the potentiometer was used. Torque was measured by a torque transducer installed inside the cell having a sensitivity of $1 \mathrm{x} 10^{-3}$ N m.

The hollow cylindrical specimens were prepared by air pluviation method. In this method, air dried sand was continuously pluviated into the annular space of the mould while maintaining a constant drop height. The hollow cylindrical specimen, with an inner diameter d_i=60 mm, outer diameter d_o=100 mm and height, h=100 mm, was enclosed laterally by two flexible membranes and vertically by rigid top and bottom caps. In order to ensure a high degree of saturation of the samples, carbon dioxide (CO_2) was circulated through the sample for approximately 2 hr. Then, de-aired water was slowly introduced into the sample through the bottom cap line under a low static pressure. The specimens were saturated by applying a back pressure of 200 kPa. The saturation time was maintained until the Skempton's pore pressure coefficient *B* exceeded 0.95. After full saturation, the sample was then isotropically consolidated to the target mean effective stress, *p'*. In all the tests, the sample was allowed to consolidate for one hour. The experimental programme is summarised in Table 2. Two types of pumice sands (different grading curves) and two levels of confining pressure (*p'*=20 and 100 kPa) were used.

Undrained cyclic torsional shear tests were carried out as per JGS 0543-2000 (JGS 2000; Tatsuoka et al. 2001). All the specimens were subjected to sinusoidal undrained cyclic loading at a frequency of 0.1 Hz and 11 cycles were applied in each loading stage. At the end of each loading stage, the excess pore-water pressure was dissipated by opening the drainage valves. In the calculation of shear stresses and shear strains, the volume of specimen immediately before the start of each loading stage was employed. The dynamic properties were calculated using the tenth cycle data. Figure 3 illustrates the computation of equivalent shear modulus, *G*, and damping ratio, *h*, for a particular single amplitude shear strain, γ_{sa}.

Table 2. Experimental programme

Case No.	Type of pumice sand	Confining pressure (kPa)	Void ratio	
			Before consolidation	After consolidation
Case 1	Type 1 (F_c=0%)	100	1.993	1.925
Case 2	Type 2 (F_c=54%)	100	1.386	1.298
Case 3	Type 1 (F_c=0%)	20	2.034	2.031

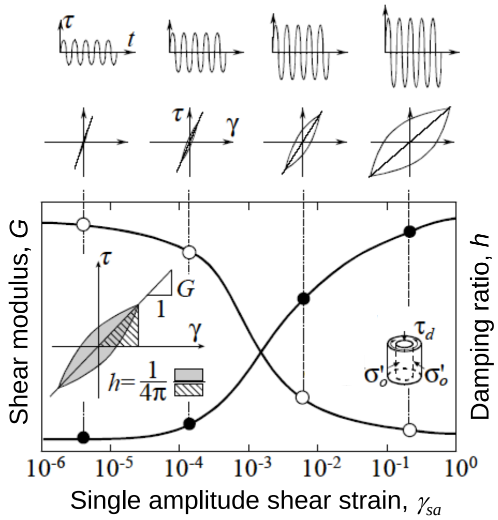


Figure 3. Schematic diagram showing the test procedure and calculation of strain-dependent shear modulus and damping ratio.

3 TEST RESULTS

3.1 Equivalent shear modulus

Figure 4 shows the plot of equivalent shear modulus, *G*, after ten cycles versus single amplitude shear strain γ_{SA} from 0.0001 to 1.0% for the three cases considered in this study. The modulus values

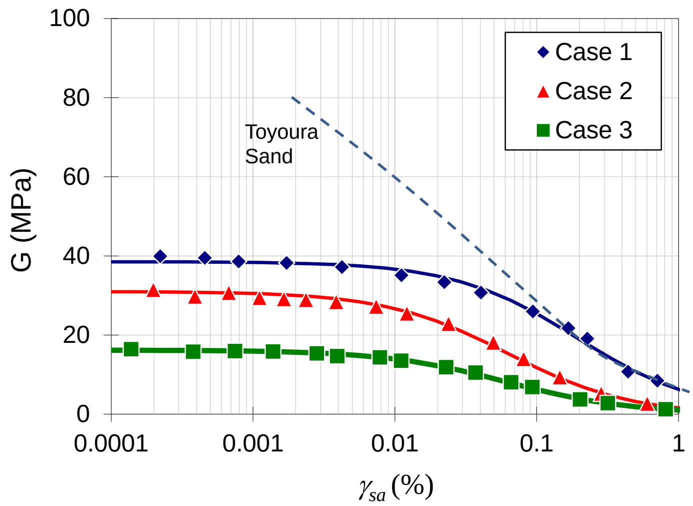


Figure 4. Equivalent shear modulus versus single amplitude shear strain. The curve for Toyoura sand was obtained by Chaudhary et al. (2002).

decrease with increasing shear strain down to about 1/5 to 1/10 of the initial value when the strain increases to a level of 0.5%. It is observed that the curves are influenced both by the grading curve (or fines content) and the level of confining pressure. Pumice sand with zero fines content (Case 1) show higher initial shear modulus than that with high fines content (Case 2). This is in agreement with the observation made by Hyodo (2006) on the behaviour of the volcanic soil Shirasu. Moreover, it is observed that as the effective confining pressure decreases (from p'=100 kPa in Case 1 to p'=20 kPa in Case 3), the initial shear modulus decreases. As pointed out by Ishihara (1996), this fact can be explained by considering the dependency of shear strength and initial shear modulus on the confining pressure.

Also indicated in the figure is the strain-dependent modulus obtained for Toyoura sand (relative density, D_r=50%, p'=100 kPa) using a similar testing apparatus (Chaudhary et al., 2002). As expected, the initial shear modulus of Toyoura sand is higher than that of pumice because of its hard-grained nature. Moreover, since the initial shear modulus is proportional to either $(2.17–e)^2/(1+e)$ or $(2.97–e)^2/(1+e)$, it is expected that pumice sand (e=1.925) would have lower initial shear modulus than Toyoura sand (e=0.798).

Figure 5 shows the initial shear modulus G_0 of pumice normalised by the function $(2.97–e)^2/(1+e)$ plotted against the effective confining pressure, p'. Although the data points are few, the normalised G_0 can be assumed to increase with p', with a proportionality constant of n=0.38. Comparing with known value of 0.5 for general sands (Ishihara 1996), it can be assumed that G_0 of pumice sand has lower dependency on p' as compared to hard-grained sands. More tests are necessary to confirm this.

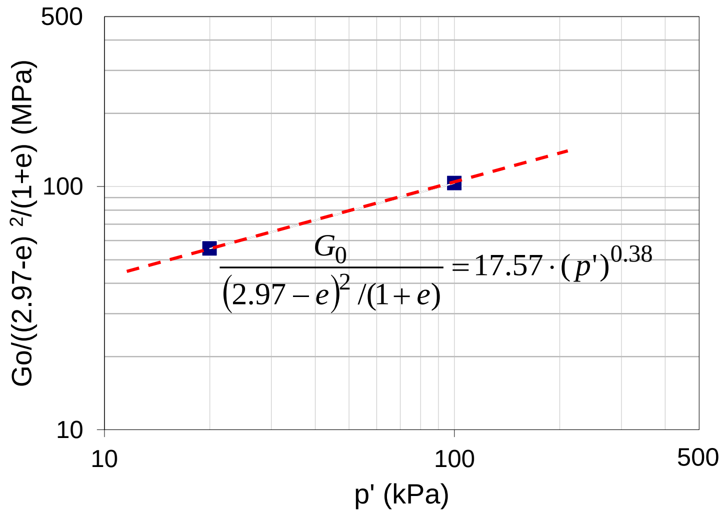


Figure 5. Effect of confining pressure on the initial shear modulus of pumice.

3.2 **Normalised shear modulus**

It is customary to represent the variation in shear modulus at any shear strain level by normalizing it with the initial shear modulus at a strain level equal to 0.0001%. This facilitates a comparison of the relationship of soils under different conditions. Figure 6 shows the strain-dependent normalized shear modulus G/G_0 for the three cases. It is evident from the plots that pumice sand with no fines content show stiffer elastic behaviour for a wider range of shear strains when compared to pumice sand with higher F_c. Again, this is in agreement with the behaviour of Shirasu (Hyodo 2006). Similarly, the rate of reduction in shear modulus of pumice with shear strain decreases with increase in confining pressure, consistent with the observation of Kokusho (1980) who conducted cyclic triaxial tests on Toyoura sand for confining pressures ranging from 20-300 kPa.

Also shown in the figure is the shear modulus ratio of Toyoura sand (p'=100 kPa) as reported by Iwasaki et al. (1978) using hollow torsional shear apparatus and considering different sample

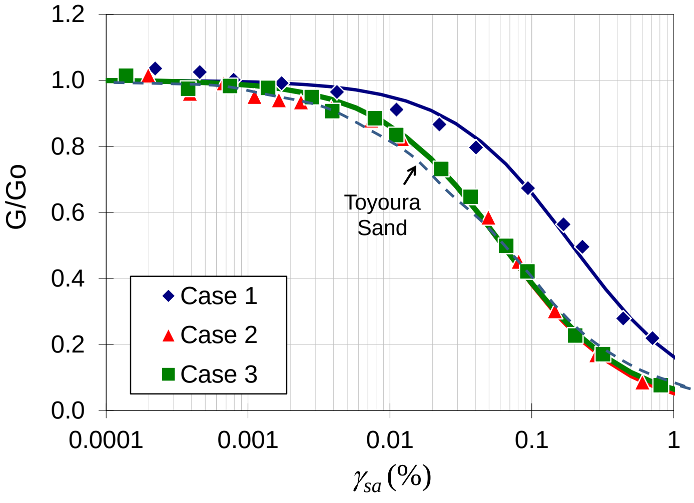


Figure 6. Normalised shear modulus versus single amplitude shear strain. The curve for Toyoura sand was obtained by Iwasaki et al. (1978).

preparation techniques. It is observed that the trend line is closer to Cases 2 and 3, although the reduction rate is a bit higher than the two cases.

3.3 **Damping ratio**

The relationship between damping ratio, h, and single amplitude shear strain, γ_{sa}, is illustrated in Figure 7. It is observed that at strain level less than 0.2%, the damping ratio of pumice is not influenced by the level of confining pressure and grading characteristics (or fines content); however, beyond 0.1%, damping ratio seems to increase with increase in fines content and with decrease in confining pressure. These trends are consistent with those observed for Shirasu (Hyodo 2006) and Toyoura sand (Kokusho 1980), respectively. Toyoura sand, on the other hand, has higher damping ratio at shear strain near failure, almost twice that of pumice, as shown by the plot obtained from the test results of Tatsuoka and Iwasaki (1978) using hollow torsional shear apparatus (p'=100 kPa) with samples prepared through different methods.

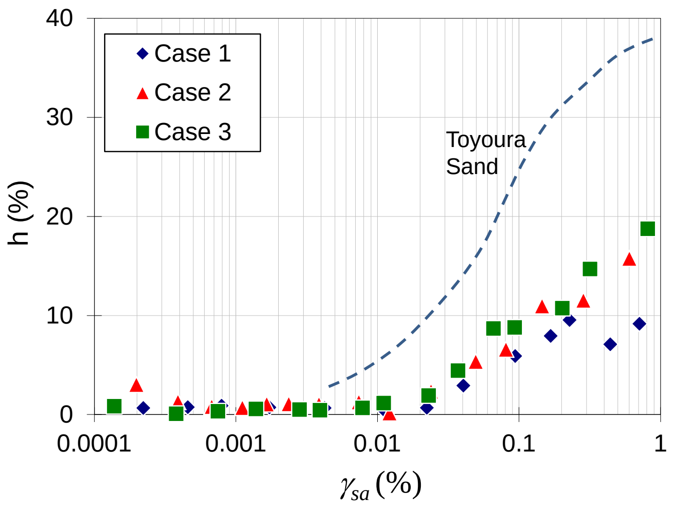


Figure 7. Schematic diagram showing the test procedure and calculation of strain-dependent shear modulus and damping ratio. The curve for Toyoura sand was obtained by Tatsuoka and Iwasaki (1978).

3.4 **Particle crushing**

At the end of each test, sieve analysis was performed to confirm if the pumice sands underwent particle breakage during the undrained cyclic tests. A comparison of the particle size distributions before and after the tests is shown in Figure 8. Although it is observed that there is slight reduction in the weight of particles larger than 0.2 mm and a corresponding increase in the weight of particles smaller than 0.2 mm, the difference is very small; hence, for practical purposes, we can say that pumice particles did not undergo particle crushing for the conditions adopted during the dynamic deformation tests. This may be reasonable considering the low level of shear stresses applied to the specimens under undrained condition.

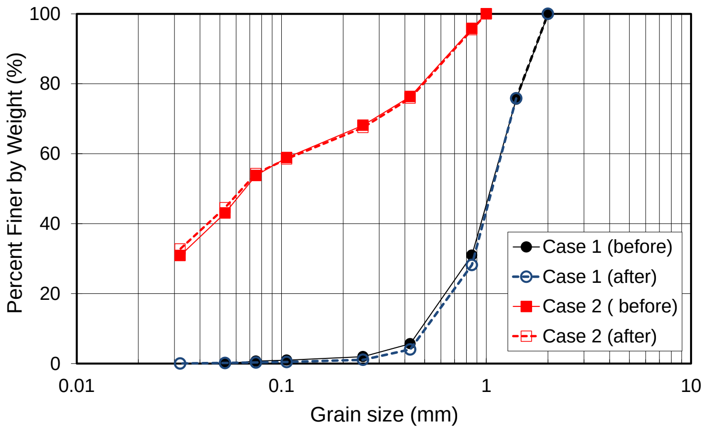


Figure 8. Grain size distribution curves of the samples used in the tests.

4 CONCLUDING REMARKS

To investigate the dynamic deformation characteristics of pumice sand, a series of cyclic undrained tests were performed using hollow torsional shear apparatus. The effects of grading characteristics (or fines content) and level of confining pressure on the strain-dependent shear modulus and damping ratio were examined. The following are the major conclusions obtained in this study:

- The shear modulus values decreased with increasing shear strain down to about 1/5 to 1/10 of the initial value when the strain increased to a level of 0.5%. The initial shear modulus increased with increasing confining pressure and lower fines content;
- The reduction rate in shear modulus ratio of pumice with strain increased with fines content, similar to the trend observed on the volcanic soil Shirasu. Moreover, the modulus ratio increased with confining pressure.
- The damping ratio of pumice seemed to be unaffected by grading curve and confining pressure at strain level less than 0.2%. At strain level near failure, the damping ratio of pumice was about half that of Toyoura sand.

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REFERENCES

Chaudhary, S., Kuwano, J., Hashimoto, S., Hayano, Y. and Nakamura, Y. 2002. Effects of initial fabric and shearing direction on cyclic deformation characteristics of sand. *Soils and Foundations*, Vol. 42 (1), 147-157.

Hyodo, T. 2006. Effects of fines on dynamic shear deformation characteristics of a volcanic soil “Shirasu”. *Master Thesis*, Yamaguchi University (in Japanese).

Ishihara, K. 1996. *Soil Behaviour in Earthquake Geotechnics*, Oxford Science Publications.

Iwasaki, T., Tatsuoka, F. and Takagi, Y. 1978. Shear moduli of sands under cyclic shear torsional loading. *Soils and Foundations*, Vol. 18 (1), 39-56.

Japanese Geotechnical Society 2000. *Soil Test Procedures and Commentaries*, First Revised Edition, Tokyo (in Japanese).

Kikkawa, N. 2008. Stress relaxation during Ko compression of pumice sand. *Proc. 8th ANZ Young Geotechnical Professionals Conference 2008*, Wellington, NZ, 5-8 November, 97-102.

Kikkawa, N., Pender, M. J., Orense, R. P. and Matsushita, E. 2009. Behaviour of pumice sand during hydrostatic and Ko compression. *Proc. Int. Conf. on Soil Mechanics and Geotechnical Engineering*, Alexandria, Egypt Vol. 1, 812-815.

Kikkawa, N., Orense, R.P. and Pender, M.J. 2011. Observations on microstructure of pumice particles using computed tomography. *Geotechnical Testing Journal* (submitted).

Kokusho, T. 1980. Cyclic triaxial test of dynamic soil properties for wide strain range. *Soils and Foundations*, Vol. 20 (2), 45-60.

Pender, M. J. 2006. Stress relaxation and particle crushing effects during Ko compression of pumice sand. *Proc. International Symposium on Geomechanics and Geotechnics of Particulate Media*, Yamaguchi, Japan, Vol.1, 91-96.

Pender, M.J., Wesley, L.D., Larkin, T.J., Pranjoto, S. 2006. Geotechnical properties of a pumice sand. *Soils and Foundations*, Vol. 46, (1), 69-81.

Tatsuoka, F. and Iwasaki, T. 1978. Hysteretic damping of sands under cyclic loading and its relation to shear modulus. *Soils and Foundations*, Vol. 18 (2), 25-40.

Tatsuoka, F., Shibuya, S., and Kuwano, R. 2001. *Advanced Laboratory Stress-Strain Testing of Geomaterials*, Balkema, Rotterdam, The Netherlands, 92–110.