Undrained cyclic and post-liquefaction behaviour of natural pumiceous soils

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ABSTRACT
Sands containing pumice particles are widely distributed over Waikato basin, North Island in New Zealand. These pumiceous deposits, due to the vesicular nature and presence of internal voids in the pumice particles, are highly crushable, compressible and lightweight which make them problematic from engineering point of view. In this paper, in order to investigate the cyclic behaviour of natural pumiceous soils, the results of several series of undrained cyclic triaxial tests and post-liquefaction monotonic tests on reconstituted specimens are reported. For comparison purposes, similar tests are performed on specimens of hard-grained Toyoura sand. The test results illustrate that natural pumice-containing soils show significantly different cyclic behaviour when compared to Toyoura sand. For instance, the liquefaction resistance of pumice soils is considerably higher than that of Toyoura sand at the same relative density. Dense Toyoura sand has approximately similar resistance to that of medium dense pumiceous soils. During the cyclic triaxial tests, pumice soils start to deform from the start of the cyclic loading and axial strain gradually increases to reach 5% double amplitude. In contrast, Toyoura sand initially undergoes a significant number of cycles with negligible deformation followed by a sudden increase in deformation in a few cycles to reach 5% double amplitude. Furthermore, undrained post-liquefaction test results indicate that pumice soils recover their shear strength with the application of shear stress at smaller axial strain when compared to Toyoura sand. To supplement the triaxial tests, the materials are sieved before and after the tests to investigate the occurrence of particle crushing; the results indicate that, indeed, particle crushing occurred. This is complemented by image analyses on both pumice and Toyoura sand particles using scanning electron microscopy to distinguish the crushable volcanic soils from the hard-grained sands.

1 INTRODUCTION

It is well-known that earthquakes can have disastrous effects on infrastructure, buildings and society. One reason for earthquake damage is related to soil failure under seismic loading. For instance, widespread liquefaction was one of the main causes of damage following the 2010-2011 Canterbury Earthquake Sequence in New Zealand (Cubrinovski et al. 2010; 2011). Moreover, post-liquefaction behaviour of soil caused much damage in San Fernando dam as a result of pore water pressure dissipation and redistribution which triggered post-earthquake movements (Vaid & Thomas 1995). Consequently, the importance of understanding the geotechnical characteristics and seismic behaviour of various local soils when designing earthquake-resistant structures needs to be established.

Volcanic soils, including pumiceous sands, are found in several areas of the North Island, New Zealand. They originated from a series of volcanic eruptions centred in the Taupo and Rotorua regions, called the Taupo Volcanic Zone (Pender et al. 2006). Powerful eruptions and airborne transport resulted in airfall deposits; these were subsequently eroded and the material transported in a fluvial environment and mixed with other materials before deposition in the Waikato Basin. Consequently, many engineering projects in the region frequently encounter pumiceous materials, so there is a need to understand how these deposits behave under seismic loading.

Pumice is characterised by a number of distinctive properties. Pumice particles are highly crushable, compressible and lightweight as a result of the vesicular nature and the presence of internal voids (Orense et al. 2012). Due to these features, pumice deposits are problematic from an engineering point of view. Another important characteristic of pumice is its unique appearance (Figure 1) with high surface angularity that enables the particles to have very high angle of internal friction (Asadi et al. 2015; Kikkawa et al. 2013). Although some studies have investigated the dynamic properties and cyclic behaviour of crushable soils, such as carbonate sand (Elmamlouk et al. 2013) and commercially-available pumice sands (Orense and Pender 2014), little is known about the seismic response of natural pumiceous soils, which are a mixture of pumice sands with other constituents. Furthermore, the undrained post-liquefaction behaviour of natural pumiceous soils has not been studied sufficiently for earthquake-induced displacement assessment.

In this paper, several series of undrained cyclic triaxial tests were conducted on reconstituted natural pumiceous soils and Toyoura sand. In addition, to understand the post-liquefaction behaviour of these materials, some of the liquefied specimens were subjected to undrained monotonic loading. The differences in the test results on the two materials were then analyzed, complemented by image analyses of the particles.

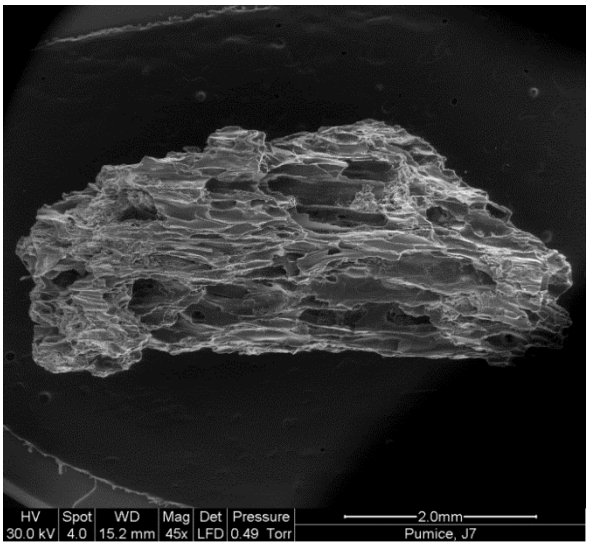

Figure 1. Typical scanning electron microscope (SEM) image of a coarse pumice particle

2 MATERIALS USED AND TEST PROGRAM

2.1 Material Used

The natural pumiceous soils which were utilized in this study were from two different sites in the Waikato basin (Figure 2). Bulk samples of these natural pumiceous soils were taken from the desired depth of 4.5m and 2m for NP1 and NP2, respectively. Moreover, for the purpose of comparison, another set of tests were performed on Toyoura sand which is known as a hard-grained, sub-angular material and commonly used in laboratory tests in Japan. The index properties and particle size distribution curves of the materials are illustrated in Table 1 and in Figure 3, respectively. The Japanese standard method (JGS 2000) was adopted to measure the index properties of the materials. Note that the pumiceous soils have lower G_s and larger void ratio range than Toyoura sand.

2.2 Sample Preparation

The reconstituted specimens for the multi-stage triaxial tests were prepared by moist tamping method, with the target specimen size of 63mm diameter and 126mm high. Prior to the sample preparation, the soil materials were mixed with water (approximately 20% of soil weight) to form uniformly moist samples. Then the specimens were prepared in a split mould with the membrane in place to achieve the target initial relative density for medium dense (Dr≈50%) and dense conditions (Dr≈80%). After the samples reached the target height, the top cap was positioned and the membrane sealed with O-rings, then the split mould was removed. Next, the specimen was saturated by subjecting it to a back pressure of 600 kPa. B-values greater than 0.95 were confirmed before the tests, indicating that the specimens were fully saturated.

2.3 Cyclic and Post-Cyclic Triaxial Tests

After consolidating the specimens with an effective confining pressure of 100 kPa, a servo-hydraulic loading frame applied the cyclic loading during the tests. All the specimens were subjected to a sinusoidal cyclic axial load with a frequency of 0.1 Hz under undrained condition. After the double amplitude axial strain reached 5%, the cyclic loading application was stopped (at the position corresponding to the start of the cyclic loading phase); then the specimen, without allowing for the excess pore water pressure to dissipate, was subjected to undrained monotonic loading at a rate of 0.1 mm/min. Furthermore, the axial load, displacement, cell pressure and back pressure were electronically recorded through a data acquisition system with a 16 bit A/D conversion into a computer for analysis.

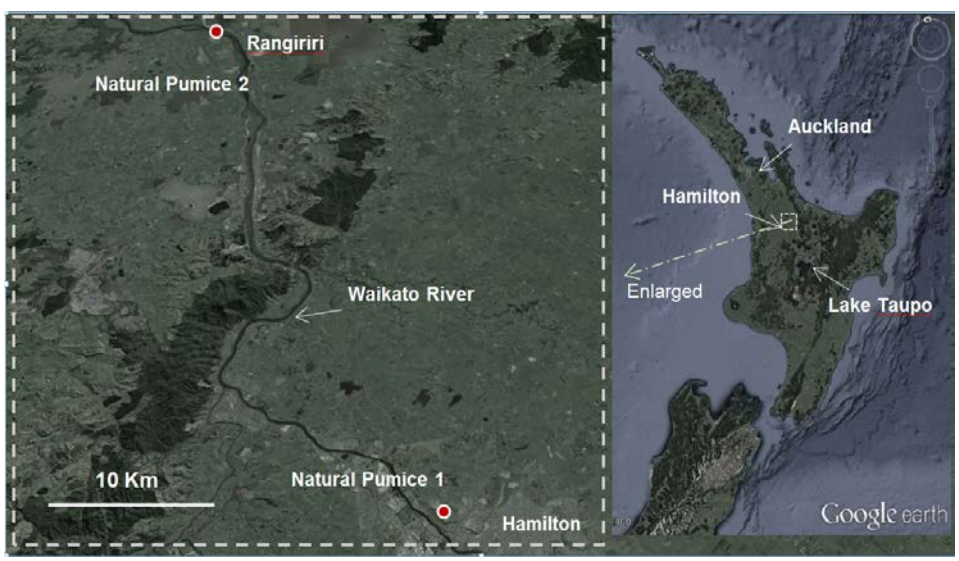

Figure 2. Locations where samples of natural pumiceous soils were obtained

Table 1. Index properties of natural pumiceous soils and Toyoura sand

Material	G_s	e_{max}	e_{min}
Natural Pumice 1	2.53	0.99	0.65
Natural Pumice 2	2.54	1.74	1.04
Toyoura Sand	2.66	0.89	0.61

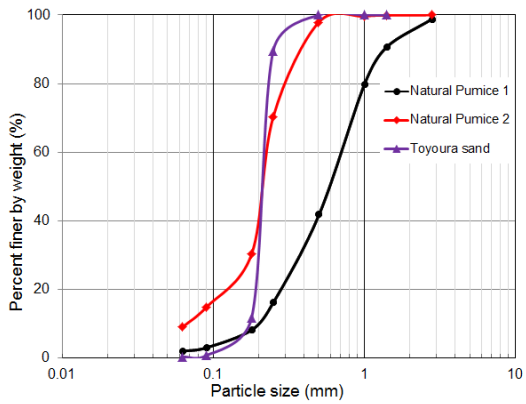

Figure 3. Particle size distribution curves of the materials tested

2.4 Particle Crushing

Crushability is the most important feature of pumice particles (Pender et al. 2006). In order to investigate the occurrence of particle crushing during the triaxial tests, wet particle size distribution (PSD) tests were performed on the materials before and after the multi-stage triaxial tests. To have a good representation of pumiceous soils for PSD tests, an adequate amount of natural pumiceous soils was taken from the prepared uniform sample before the test. After the triaxial test (cyclic + post-cyclic monotonic), a small portion was taken from the tested specimen in order to examine the occurrence of particle crushing. Next, Hardin's (1985) method for particle crushing assessment was used due to its simplicity and consideration of the entire PSD curve. According to this method, the relative breakage (B_r) is defined as:

$$B_r=B_t/B_p \quad (1)$$

in which B_t is the total breakage of the materials and equal to the enclosed area between the PSD curves before and after the test and has a lower limit of zero and a theoretical upper limit of unity. Furthermore, B_p is the potential breakage of materials, which is calculated as the enclosed area between the initial PSD curve and a given particle size boundary. In this study the threshold particle size for potential breakage calculation is considered as 0.063 mm.

2.5 Particle Shape Index

In order to distinguish the pumiceous sands from the other types of hard-grained soil particles, 2D scanning electron microscope (SEM) images were taken on different particle sizes of pumice soils and Toyoura sand at their most stable configuration. Subsequently, the methodology of Kikkawa et al. (2013) was followed to analyse the SEM images and quantify the soil particle shape characteristic through roundness coefficient (R_c), aspect ratio (A_r) and angular coefficient (A_c), which are defined as follows:

$$R_c=L^2/4\pi A \quad (2)$$

$$A_r=b/a \quad (3)$$

$$A_c=|R_c - 1 + A_r^2/(2A_r)| \quad (4)$$

In the above equations, L is the perimeter, A is the surface area, while a and b are the dimensions of the particles along the minor and major axes, respectively. Each of the aforementioned parameters (R_c, A_r and A_c) indicates an important feature of a soil particle. For instance, if the value of R_c is equal to one, the particle is circular and if it exceeds unity, the shape would change to ellipsoidal. Furthermore, if A_r exceeds one, the soil particle is more elongated. A higher value of A_c illustrates a more angular particle surface. While the interaction between particles within the triaxial specimen is essentially 3D, these 2D indices can very well characterize the shape of the particles.

3 TEST RESULTS

3.1 Cyclic Triaxial Test Results

Typical results of the undrained cyclic triaxial tests on Natural Pumice 1 and Toyoura sand under different relative densities (D_r≈80 % and D_r≈50 %) and σ_c'=100 kPa are plotted in Figure 4, where the results are depicted in terms of double amplitude axial strain (ε_{DA}) and excess pore water pressure ratio ($r_u=u/\sigma_c'$, in which u is excess pore water pressure and σ_c' is effective confining pressure) plotted against the normalized number of cycles N/N (at ε_{DA}=5%). These results were chosen as they represent almost similar number of cycles to liquefaction. Note that since the maximum diameter of the materials tested was 3mm, which is more than 20 times smaller than the diameter of the triaxial specimens, the effect of membrane penetration was deemed negligible.

There is a significant difference between the undrained cyclic behaviour of pumiceous soil and Toyoura sand. As shown in Figure 4a, the development of double amplitude axial strain (ε_{DA}) of pumiceous soil started from the beginning of the cyclic loading for both dense and medium dense specimens. In the dense case, deformation development is more significant compared with medium dense pumiceous soils; then gradually, in an approximately linear trend, this increased to ε_{DA}=5%. The axial strain development of pumiceous soil from the

beginning of cyclic loading is due to the compressibility and crushability feature of pumiceous material (Orense et al. 2012). The particle crushing of pumiceous soil during cyclic test is discussed further in Section 3.4. This cyclic behaviour is in contrast with the response of Toyoura sand, which is characterized by negligible straining at the start of loading followed by sudden strain development after a significant number of cyclic loading. The behaviour of Toyoura sand is consistent with the behaviour of other types of hard-grained sands reported in the literature, such as Christchurch sand (e.g. Taylor 2014).

The corresponding development of r_u for the tested materials is depicted in Figure 4b. It is noticeable from the graph that the dense pumiceous soil shows an immediate increase in r_u up to 0.9 during the first quarter of the cyclic loading, followed by a gradual change as r_u approaches 1.0; however, for the case of medium dense pumice, the increase is less significant. It is noteworthy that under high r_u, pumiceous materials are capable of undergoing significant number of cycles without any substantial deformation. This behaviour can be attributed to the high angularity of pumiceous materials, as discussed in Section 3.4. Conversely, in the case of Toyoura sand, once r_u reaches around 0.6, deformation suddenly takes place and, in a few numbers of cycles, the double amplitude axial strain reaches 5%.

3.2 Liquefaction Resistance

Figure 5 compares the liquefaction resistance curves of Natural Pumice 1, Natural Pumice 2 and Toyoura sand in terms of 5% double amplitude axial strain. As shown in the figure, the pumiceous soils are less susceptible to liquefaction compared to Toyoura sand. For example, if the liquefaction resistance is defined in terms of the cyclic stress ratio (*CSR*) corresponding to 15 cycles, then dense Toyoura sand has liquefaction resistance that is almost half of that of dense natural pumiceous soil and approximately the same as that of medium dense (D_r≈50%) natural pumiceous soils. The higher liquefaction resistance of pumiceous soils compared with Toyoura sand is partly due to the consequence of particle crushing during the cyclic test and rearrangement of soil skeleton resulting in more stable soil structure. More tests are planned to confirm this phenomenon. It is worth mentioning that Yamawaki et al. (2002) noted that as the values of R_c and A_r of various geomaterials increase, the liquefaction resistance would also increase. In the same vein, the higher angularity of pumiceous soils may be a reason for their high liquefaction resistance.

3.3 Undrained Post-Cyclic Monotonic Test Results

The post-cyclic monotonic triaxial test results on Natural Pumice 1 and Toyoura sand are depicted in Figure 6. The results are shown in terms of deviator stress increment and EPWP decrement with respect to axial strain development. As evident from the graphs, pumiceous soil and Toyoura sand both show hardening behaviour when monotonically sheared following liquefaction occurrence

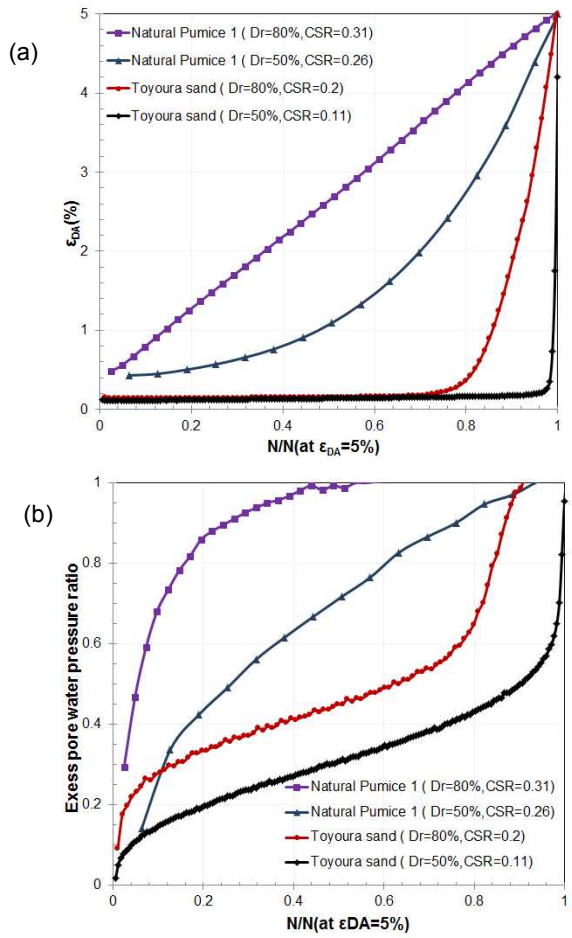

Figure 4. Comparison of the undrained cyclic behaviour of pumiceous soils and Toyoura sand in terms of: (a) double amplitude axial strain; and (b) excess pore water pressure ratio

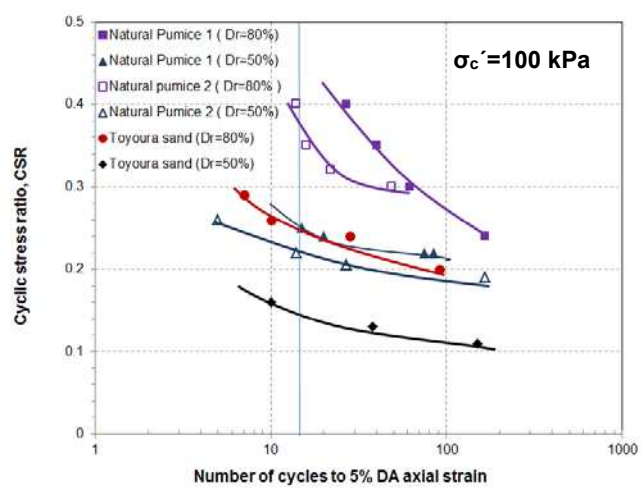

Figure 5. Liquefaction resistance curves of tested materials based on 5% double amplitude axial strain

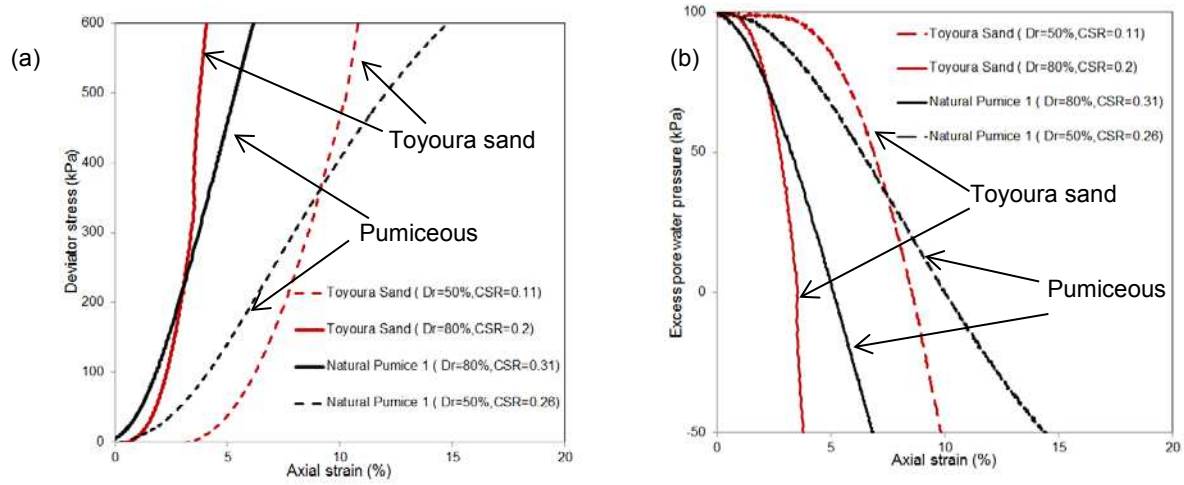

Figure 6. Post-liquefaction behaviour of pumice soils and Toyoura sand: (a) stress-strain relation; (b) pore water pressure behaviour

regardless of the initial relative density. Figure 6(a) illustrates that Natural Pumice 1 recovers its stiffness at considerably lower strain compared to Toyoura sand. For example, Natural Pumice 1 at D_r≈50% recovers its stiffness at approximately 0.5% axial strain; however, Toyoura sand (D_r≈50%) undergoes deformation of up to about 3% axial strain with almost zero shear resistance before the stiffness is gradually recovered. Furthermore, the initial relative density of Toyoura sand significantly affects the point of strain hardening (the strain when soil starts to recover its stiffness), which is consistent with the observation of Lombardi et al. (2014) on other types of hard-grained sands. However, the depicted data in Figure 6(a) show that for the case of pumiceous soils, relative density does not play an important role for the point of strain hardening; further tests are planned to confirm this observation.

3.4 Particle Crushing and Particle Shape Indices

Several series of PSD tests were performed on natural pumiceous materials before and after the multi-stage cyclic triaxial test in order to investigate the occurrence of particle crushing of pumice sand. Figure 7 illustrates that pumiceous materials crushed during the multi stage triaxial test (cyclic + post-cyclic monotonic). Using Hardin's (1985) method, the average relative breakage (B_r) of the two different pumiceous soils was investigated and the results are summarized in Table 2. As shown in the table, the degrees of crushability of pumiceous soils are different from each other. For instance, the relative breakage of Natural Pumice 2 is almost twice that of Natural Pumice 1 for the same relative density of about 80% and 50%. Furthermore, as indicated in the table, the initial relative density significantly affects the degree of crushability of natural pumiceous soils. The results show that, as a consequence of the increase in the initial relative density from 50% to 80%, the relative breakage of Natural Pumice 2 and Natural Pumice 1 increases from 0.046 to 0.079 and 0.027 to 0.035, respectively.

Moreover, several scanning electron microscope (SEM) images were taken of the particles from the natural pumice soils and Toyoura sand before the multi-stage triaxial tests to distinguish the crushable soils from hard-grained soils. From the SEM images shown in Figure 8, pumice sands have unique and irregular surface texture with lots of voids as compared to Toyoura sand which is more sub-angular. In order to distinguish these crushable soils from other hard-grained sands, the work of Kikkawa et al. (2013) was used to quantify the differences.

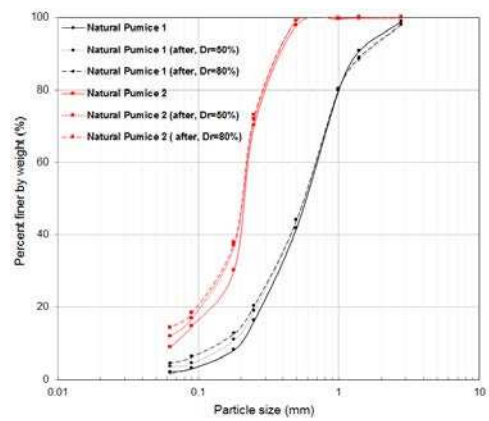

Figure 7. Typical particle size distribution of the materials before and after the multi-stage triaxial tests

According this method, the SEM images were initially binarized for the purpose of 2D image analyses. Subsequently, the following parameters were calculated from the cross sectional images of the soil particles using image analyzing algorithm: perimeter, surface area, and long and short axes along center of gravity. Consequently, these four parameters were utilised to calculate the roundness coefficient (R_c), aspect ratio (A_r) and angular coefficient (A_c). From the average results shown in Table 3, it can be seen that the particle shape indices of natural pumiceous sand are considerably different from those of Toyoura sand. For example, the value of R_c for natural pumiceous soils is approximately 1.5 times higher than that of Toyoura sand. Furthermore,

Table 2. Average degree of particle crushing of natural pumice soils after triaxial tests.

Soil type	Number of tests	B_t	B_p	B_r	D_r (%)
Natural Pumice 1	4	0.034	0.981	0.035	80
Natural Pumice 1	2	0.026	0.981	0.027	50
Natural Pumice 2	4	0.036	0.455	0.079	80
Natural Pumice 2	2	0.021	0.455	0.046	50

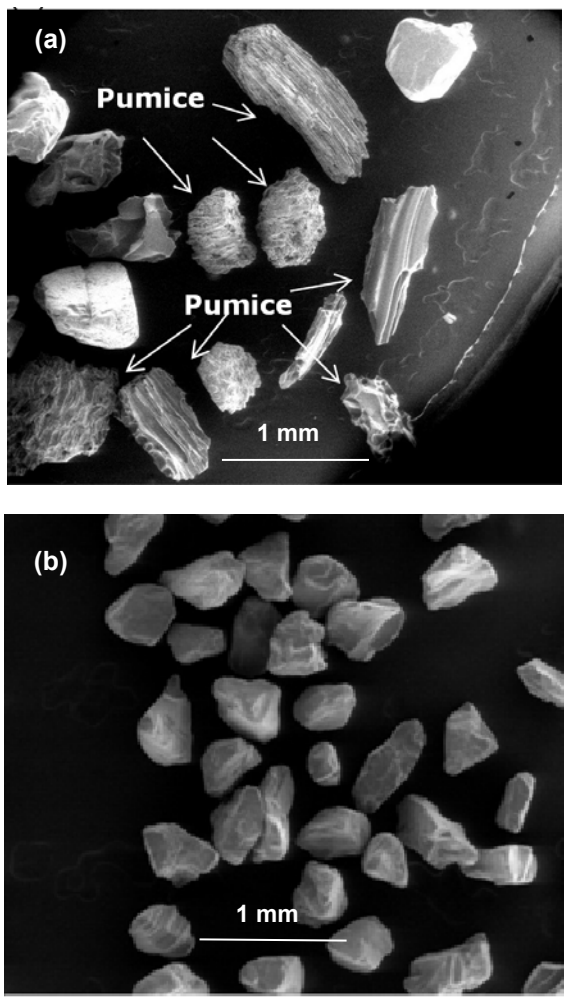

Figure 8. Scanning electron microscope images of: (a) Natural Pumice 2; and (b) Toyoura sand

the average A_r values for Natural Pumice 1 and 2 are 1.703 and 1.801, respectively, compared to 1.483 for Toyoura sand. The A_c values of the natural pumiceous soils were about 5 times higher than Toyoura sand.

4 CONCLUSIONS

In order to investigate the cyclic and post-liquefaction properties of pumiceous soils from the Waikato basin in North Island, New Zealand, several series of undrained cyclic triaxial tests and undrained post-liquefaction tests were performed on reconstituted pumiceous soils. Similar tests were also conducted on hard-grained Toyoura sand. Additionally, sieve tests were conducted on the materials before and after the multi-stage triaxial tests (cyclic + post-cyclic monotonic) for the purpose of confirming particle crushing occurrence. Moreover, to distinguish the pumice particles from hard-grained sand, SEM images of the soil particles were analysed to illustrate the differences. The following are the major conclusions in this study:

1. From the cyclic triaxial tests, pumiceous samples started to deform from the beginning of loading and gradually reached 5% double amplitude axial strain. However, for the case of Toyoura sand, strain development was negligible up to a significant number of cycles, after which a sudden increase occurred to reach 5% double amplitude axial strain.
2. During cyclic tests, dense pumiceous materials were capable of undergoing high excess pore water pressure ratio (r_u=0.8) without the occurrence of significant deformation. However, for Toyoura sand, as soon as r_u=0.6 was reached, a sudden increase in cyclic strain amplitude occurred.
3. The liquefaction resistance of dense natural pumiceous soil was almost twice of that of dense Toyoura sand. This dense Toyoura sand had approximately similar resistance as the medium dense pumiceous soil.

Table 3. Particle shape indices of pumice sand and Toyoura sand

Soil Type	Number of analysed soil particle images	Average R_c	Average A_r	Average A_c
Toyoura sand	50	1.258	1.483	0.179
Natural pumice 1	98	1.703	1.812	0.521
Natural pumice 2	93	1.801	1.989	0.555

4. In terms of post-liquefaction behaviour, pumiceous soils recovered their stiffness at considerably lower of strain hardening (strain when the soil started to recover its stiffness) of pumiceous soils.
5. During multi stage triaxial tests (cyclic + post-cyclic monotonic), crushing occurred in pumice particles. The analysed SEM images of the materials (before triaxial tests) showed that the pumice particles are angular and more elongated compared to Toyoura sand particles.

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strain that Toyoura sand. Furthermore, relative density appeared not to affect significantly the point

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