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Undrained cyclic strength of undisturbed pumiceous deposits

M.S. Asadi[1], M.J. Pender[2] and R.P. Orense[3]

[1]Department of Civil and Environmental Engineering, University of Auckland, Private Bag 92019, Auckland Mail Centre, Auckland New Zealand; PH(+64 9) 373-7599; FAX (+64 9) 373 7462; email:masa679@aucklanduni.ac.nz
[2]Department of Civil and Environmental Engineering, University of Auckland, Private Bag 92019, Auckland Mail Centre, Auckland, New Zealand; PH(+64 9) 373-7599; FAX (+64 9) 373 7462; email:m.pender@auckland.ac.nz
[3]Department of Civil and Environmental Engineering, University of Auckland, Private Bag 92019, Auckland Mail Centre, Auckland, New Zealand; PH(+64 9) 373-7599; FAX (+64 9) 373 7462; email:r.orense@auckland.ac.nz

ABSTRACT

This paper presents experimental data on the cyclic properties of undisturbed fluvial deposits which include pumice particles, from the Huntly region in the North Island of New Zealand. A proposed highway development in the region and future infrastructure development require information about the seismic behaviour of these pumiceous deposits. To obtain reliable results from laboratory tests, it is necessary to obtain high-quality undisturbed samples because soil fabric, age, and stress history play important roles in determining liquefaction resistance. Obtaining high-quality undisturbed cohesionless materials is a difficult and challenging task; for this work gel-push sampling, a new method developed in Japan which is a lower cost alternative to ground freezing for obtaining high-quality undisturbed samples, was employed. The undisturbed pumiceous specimens were subjected to undrained cyclic loading in a triaxial apparatus, followed by monotonic loading. The cyclic results showed that the excess pore water pressure ratio approached unity well before the double amplitude cyclic axial strain approached 5 %. In terms of post-cyclic monotonic response, the undrained shear strength of liquefied pumiceous deposits was similar to that without prior cyclic loading history, which indicates that the pumiceous deposits recover their strength with post-liquefaction shearing. Scanning electron microscope (SEM) imaging was done on sieved fractions of the soil which showed that some coarse materials have the appearance of pumice while the fine materials are composed of shards of crushed pumice, possibly the result of particle crushing during shear.

Keywords: pumiceous deposits, undrained cyclic tests, undisturbed samples, gel-push sampling, liquefaction, particle crushing.

1 INTRODUCTION

Soil types are variable across New Zealand as well as the world, so it is necessary to consider the behaviour of soil in each area uniquely. For instance, due to New Zealand's tectonic location, volcanic soils, including pumiceous sands, are found in several areas of the North Island. They originate from a series of volcanic eruptions centred in the Taupo and Rotorua region of the central North Island (Pender et al. 2006; Wesley 2001). As a consequence of infrastructure development in the North Island of New Zealand, many engineering projects frequently encounter pumiceous materials, so there is a need to understand how these deposits behave under seismic loading.

Because of the vesicular nature and presence of internal voids, pumice particles are highly crushable, compressible and lightweight (Kikkawa et al. 2012; Orense & Pender 2013; Orense et al. 2012). Pumice particles are highly crushable and fragile not only because they are porous but also because of their angular shape (Kikkawa et al. 2013). As shown in Figure 1, pumice sand particle can be distinguished from other particles because of their unique appearance.

Owing to their characteristics, pumice deposits are problematic from engineering point of view. As Orense et al. (2012) indicated, commercially-available pumice sands under seismic and monotonic loading behave differently from hard-grained materials because they crush easily. As a consequence, for example, relative density does not have a significant effect on the liquefaction resistance of pumice; however relative density plays an important role on the liquefaction resistance of hard-grained materials.

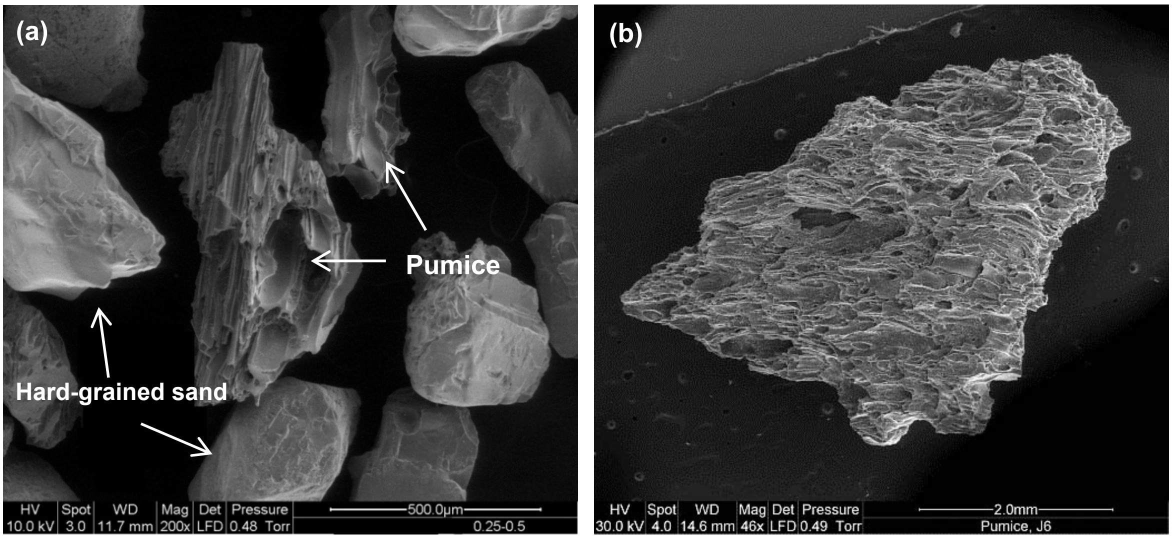

Figure 1. (a) A pumice sand particle (centre of image), with another smaller pumice to the above right, (b) close-up of the complex surface texture of a pumice particle.

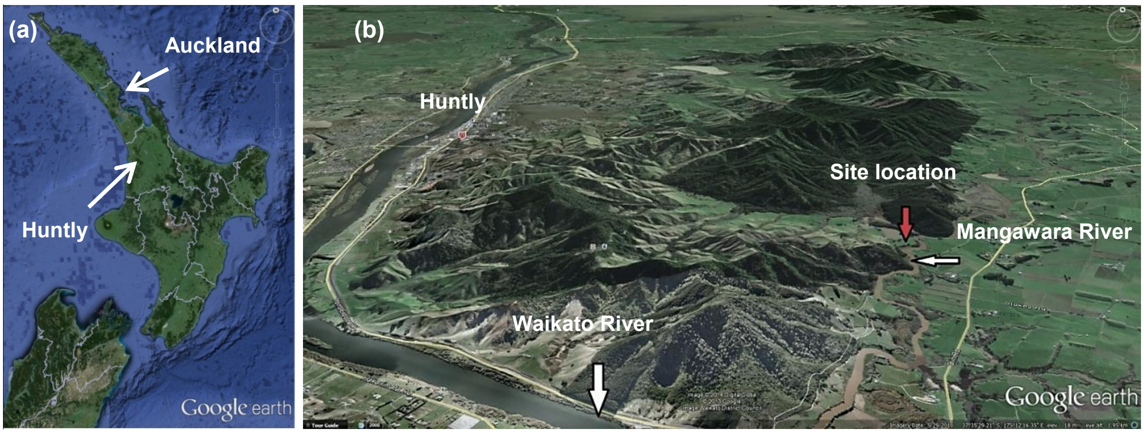

Figure 2. (a) Location of Huntly in North Island of New Zealand; (b) site location south-east of Huntly.

As a starting point in understanding the cyclic properties of undisturbed natural deposits of pumice, results of cyclic undrained triaxial tests on undisturbed pumiceous specimens obtained by gel-push sampling are presented.

2 MATERIAL USED

2.1 Site location

Undisturbed samples were obtained at a site which is known to have high pumice content. Figure 2 illustrates the site location which is near the town of Huntly in the North Island. The materials were sourced at the depth of 3.9 m to 6.8 m. The materials at this site are fluvial deposits and include pumice particles. According to the boring log, the materials at the depth of 3.9 to 6.8 m are described as grey very loose, fine to medium pumice sand with the SPT N-value of 0 and 6 at the depth of 4 m and 6 m respectively. From the CPT data, the cone resistance of the material is variable (0.5 to 3 MPa) along the above mentioned depth.

2.2 Sampling technique

Obtaining high-quality undisturbed sample of cohesionless material is a very difficult and challenging task. The best method for obtaining undisturbed samples is to freeze the ground (Hofmann et al. 2000) using liquid nitrogen before retrieving out large diameter samples by rotary coring; however in this study, gel-push sampling was implemented to obtain undisturbed samples. Gel-push sampling is a new method and a lower cost alternative to ground-freezing to obtain high-quality undisturbed samples.

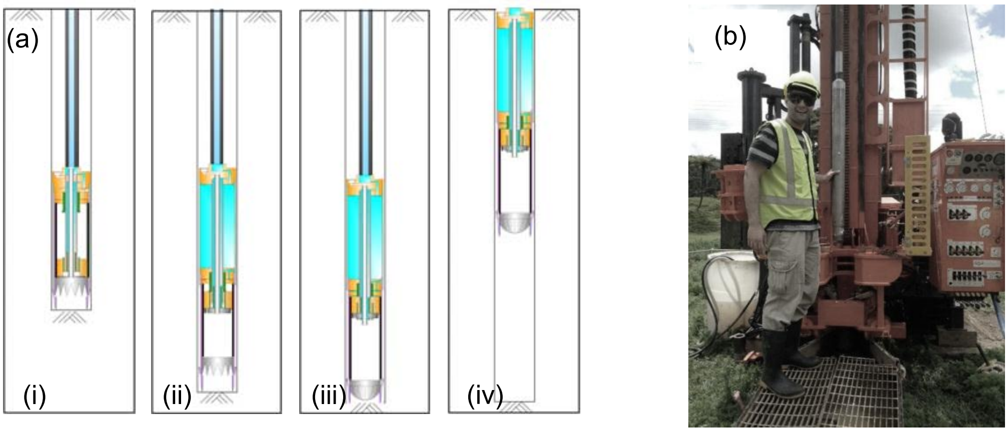

Figure 3. (a) Schematic illustration of the gel-push sampler at different stages of sampling procedure; (i) pushing down the sampler; (ii) hydraulic advancement of the sampling tube into the undisturbed soil; (iii) closure of core catcher; (iv) removal to the surface (after (Lee & Chen 2013). (b) Sampling barrel and drilling rig.

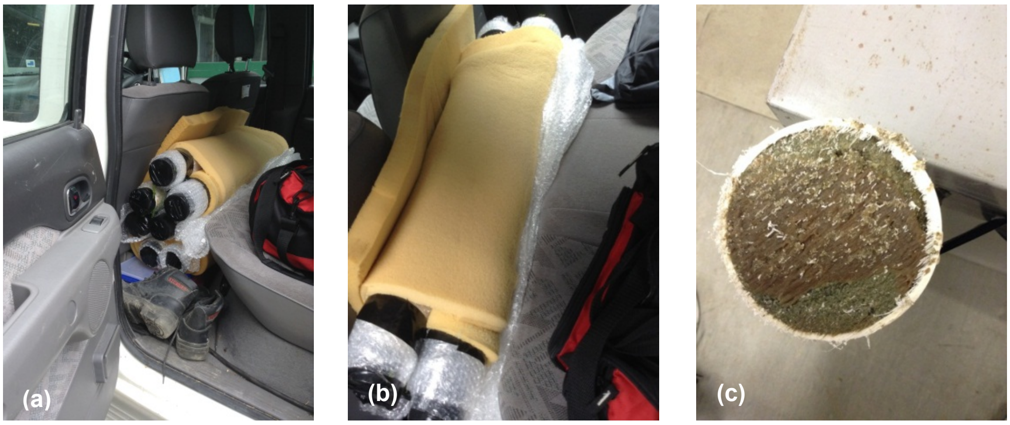

Figure 4. (a&b) Transportation of sample tubes from the site to the laboratory; (c) one end of the sample sliced through to check that transportation had not caused disturbance.

Gel-push sampling technique was developed in Japan and currently being used in Christchurch to obtain undisturbed soil samples (Taylor & Cubrinovski 2012). As the name suggests, this involves the injection of a water-soluble polymeric lubricant (gel) from the sampler shoe to lubricate and reduce friction between the cut sample and the tube, both during sampling as well as extrusion in the laboratory (Lee & Chen 2013; Taylor & Cubrinovski 2012). Similar to a regular piston sampler, the sampler is lowered down to the target depth prior to pushing the tube hydraulically to the undisturbed soils; the operation of gel-push sampler is illustrated schematically in Figure 3.

According to Lee and Chen (2013), the gel-push sampling technique is a capable tool for acquiring high quality undisturbed samples of loose deposits of non-plastic silty sand under high ground water table. Consequently, it is expected that the gel-push technique would obtain materials having lower degrees of disturbance than the conventional push-tube technique.

3 SAMPLE PREPARATION AND EXPERIMENTAL METHOD

After obtaining undisturbed soil samples from the site, the samples were transported to the laboratory with as little vibration as possible by putting the samples between the front and back seats, using a foam mattress to buffer them, and driving carefully (Figure 4). To check on possible disturbance during transportation, the day after the samples were transported back to the laboratory in Auckland, the end of some of the samples was sliced off with a bandsaw. The fact that the sample cross-section filled the tube suggests that there was no serious disturbance during the transportation.

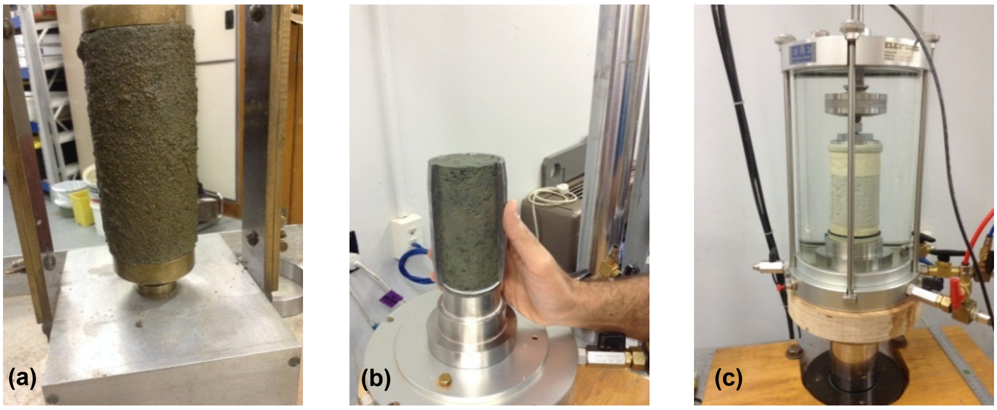

Figure 5. (a) Sample trimming and (b&c) setting-up in the triaxial cell.

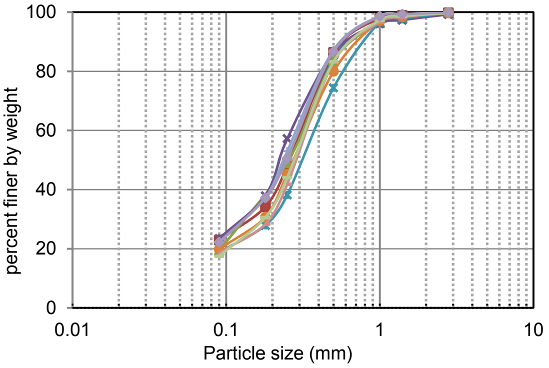

Figure 6. Grain size distribution curves of the pumiceous sand samples used in the tests.

Prior to performing the cyclic tests, the specimens were obtained by cutting a section from the PVC sample tube (71 mm internal dia.) using a band saw and then extruded manually, and subsequent manual trimming using a soil lathe to achieve the target size. Filter papers were placed at the ends to prevent clogging of the porous discs and then a sample membrane applied over the sample (Figure 5). The specimen was saturated by subjecting it to back pressure of 600kPa. A fully saturated condition was ensured by checking that B-value of more than 0.96 was obtained for all specimens. The specimens were then subjected to isotropic consolidation at effective confining pressure of 100 kPa and 250 kPa.

A hydraulic-powered loading frame applied the cyclic loading for the tests. All the tests were subjected to a sinusoidal cyclic axial load at a frequency of 0.1 Hz under undrained condition, during which the axial loads and displacements were recorded. Furthermore, the values of cell pressure, back pressure and volume change were electronically measured through a data acquisition system onto a computer for analysis.

4 TESTS RESULTS

4.1 Particle size distribution

The grain size distribution curves of the material used are shown in Figure 6. It is noted the percentage of fine particles varied a little along the depth from 3.9m to 6.8m. However such variation is assumed not have any significant effect on the results of cyclic tests.

4.2 Cyclic triaxial tests results

The plots of double amplitude axial strain and excess pore water pressure ratio ($r_u=u/\sigma_c'$, in which u is excess pore water pressure and σ_c' is confining pressure) against normalized number of cycles N/N(at ε_{DA}=5%) are shown in Figure 7. The specimens were subjected to two different consolidation pressures (100kPa and 250kPa) as well as two different cyclic stress ratios ($CSR=\sigma_d/2\sigma_c'$, in which σ_d is deviator stress and σ_c' is confining pressure) for each consolidation pressure.

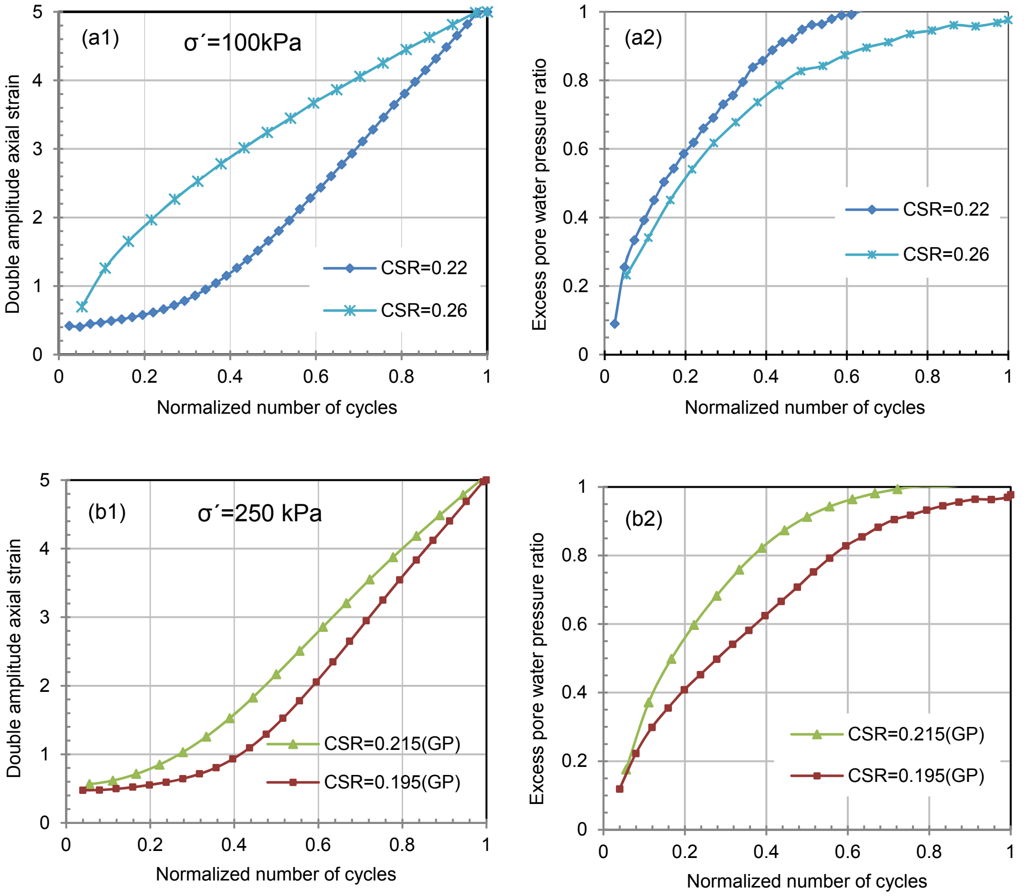

Figure 7. (a1, b1) Double amplitude axial strain ε_{DA} *and (a2, b2) excess pore water pressure ratio, plots against normalized number of cycles N/N* $_{\varepsilon DA=5\%}$ *from cyclic undrained tests.*

The trends of strain with the number of cycles are highly reliant on the values of the applied CSR. When the materials were subjected to higher CSR, the strain development was much faster. This behaviour can be attributed to the possible particle crushing of pumiceous materials during cyclic loading. Under the same confining pressure, higher CSR mean the materials were subjected to higher level of shearing and consequently more crushing occurred. Furthermore, the effect of consolidation pressure (within the range investigated) on strain development was found to be important. The curves of excess pore water pressure ratio against normalized number of cycles showed that the rate of the development of excess pore water pressure was much faster than the rate of strain development. Further investigation is required to gain an insight on this behaviour.

4.3 Cyclic shear resistance

According to Figure 8a, the pumiceous soil's vulnerability to liquefaction increased as a consequence of decrease in confining pressure; this behaviour is consistent with the observation made on commercially available pumice sand and hard-grained materials (Orense et al. 2012).

Figure 8b plots the cyclic resistance curves for loose (D_r=50%) and dense (D_r=90%) Toyoura sand and undisturbed pumiceous deposits. It is noted that the undisturbed pumiceous deposits are more resistant to liquefaction compared to loose Toyoura sand. For instance, if the liquefaction resistance is defined in terms of the cyclic stress ratio (CSR) corresponding to 15 cycles, then loose undisturbed pumiceous deposits have almost similar liquefaction resistance to dense Toyoura sand and twice that of loose Toyoura sand.

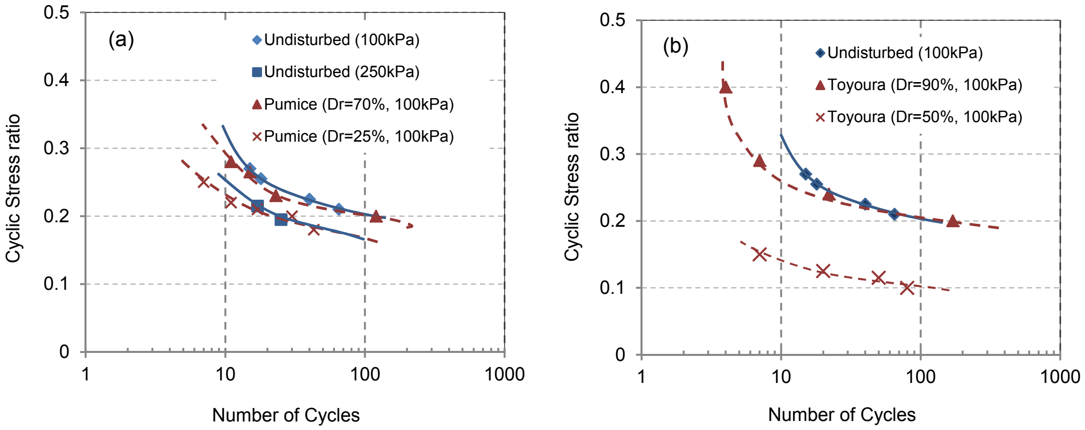

Figure 8. Cyclic resistance curves for (a) the undisturbed pumiceous specimens compared with results for pumice sand and (b) Toyoura sand (Orense et al. 2012).

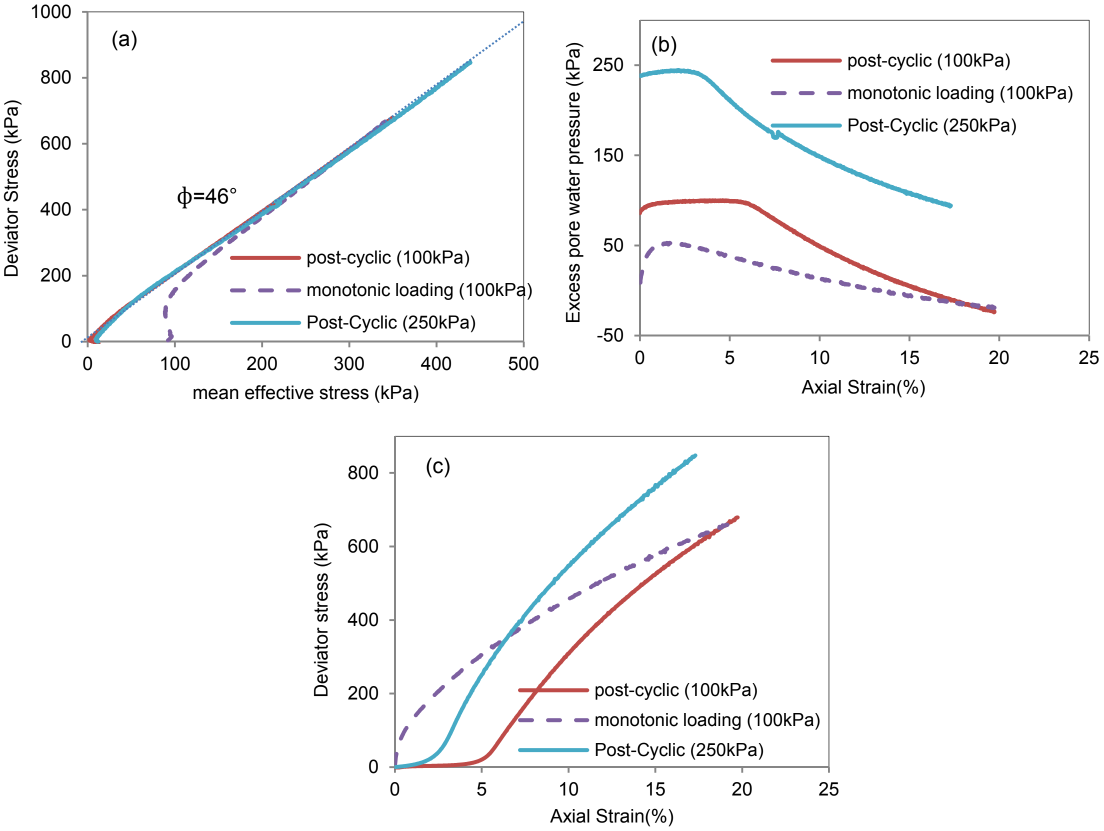

Figure 9. (a) Effective stress paths for static and post-cyclic monotonic loading; (b) pore pressure response during post-cyclic monotonic loading and monotonic loading; (c) stress-strain relation.

Comparing the cyclic resistance of undisturbed pumiceous deposits with pure pumice sands, the cyclic resistance of dense pumice sand is the same as that of loose undisturbed pumiceous deposits (100 kPa consolidation pressure) as indicated in the Figure 8a. However, loose pumice sands were more vulnerable to liquefaction compared to loose undisturbed pumiceous deposits. This can be explained by the fact that, soil fabric and stress history play an important role in the susceptibility of the soil to liquefaction.

4.4 Post-cyclic monotonic loading results

The undrained shear strength of liquefied pumice deposits was compared to that without prior cyclic loading. During the post-cyclic monotonic loading, a strongly dilatant behaviour was observed (Figure 9a). Figure 9 indicates that liquefaction did not have any significant effect on the post-liquefaction strength of pumiceous deposits. The pumiceous deposits have an angle of internal friction of about

46°, which is much higher than that of hard grained sand which is 30°(Ishihara 1996). This big difference in the value of angle of internal friction can be as a result of particle crushing as well as angular shape of the particles. Furthermore, steady state of deformation did not occur for pumiceous sands (Figure 9c).

4.5 Verifying the presence of pumice particles in natural soils

Crushability is the most important feature of pumice particles (Pender et al. 2006). To examine the occurrence of particle crushing during the tests, particle size distribution (PSD) tests were performed on the material before (using extra material from the trimming of the undisturbed specimens) and after the cyclic tests. As shown in Figure 10, particle crushing occurred during the cyclic tests and it is apparent that the confining pressure (similar CSR applied to specimens) had a significant effect on the degree of breakage. As a consequence of high consolidation pressure, the materials were subjected to higher level of shearing and subsequently more particle crushing occurred.

Scanning electron microscope (SEM) imaging was done on sieved fractions of the soil to get some indication about the amount of pumice in the natural soils at Huntly site. The SEM micrographs (Figure 11) showed that all coarse materials have the appearance of pumice while the fine materials are composed of shards of crushed pumice, possibly the result of particle crushing during shear.

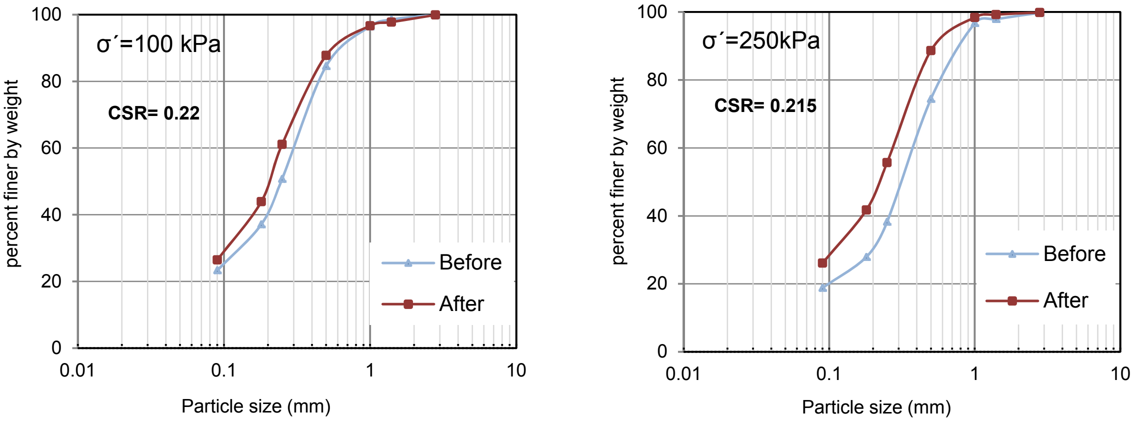

Figure 10. Grain size distribution curves before and after the cyclic tests under 100 and 250 kPa confining pressure.

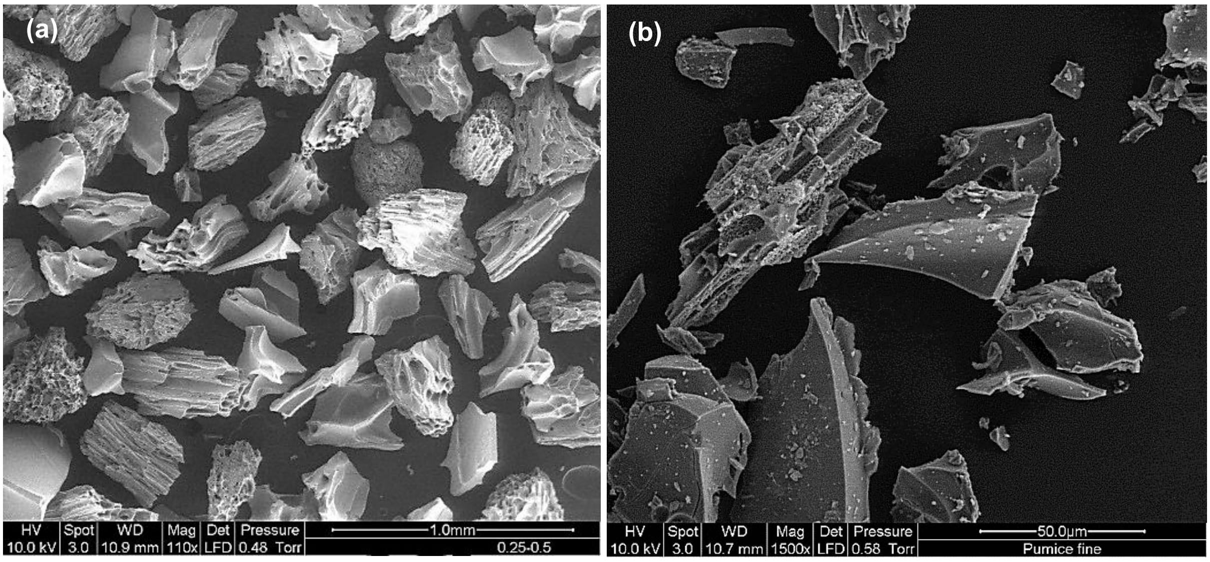

Figure 11. Scanning electron microscope images: (a) of material retained on the 250 micron sieve and passing the 500 micron sieve; (b) of material passing the 90 micron sieve.

5 CONCLUSIONS

In order to investigate the cyclic properties of pumiceous soils from the Huntly area, undrained cyclic triaxial tests were performed on material obtained by gel-push sampling. The following are the major conclusions:

1. Specimens trimmed from the gel-push samples were found to have liquefaction resistance similar to or better than that of dense Toyoura sand (Fig. 8).
2. Specimens trimmed from the gel-push samples were found to have liquefaction resistance similar to or better than that of pumice sand at a relative density of 70% (Fig. 8).
3. Increasing effective consolidation pressure reduces the liquefaction resistance of the pumiceous soil (Fig. 8).
4. Post-cyclic shearing of pumiceous soil has an effective stress path which approaches that for monotonic shearing and has a friction angle of 46 degrees (Fig. 9).
5. Crushing of the pumice particles occurs during shearing, which becomes more significant as effective consolidation pressure (under the same CSR) on the specimen increases (Fig. 10).
6. The cyclic results showed that the excess pore water pressure ratio approached unity well before the double amplitude cyclic axial strain approached 5 %. (Fig. 7).
7. Scanning electron microscope imaging on material after cyclic testing showed a large proportion of pumice particles in the 250 to 500 micron size range. The sub 50 micron particles appear to be shards of broken pumice particles (Fig. 11).

6 ACKNOWLEDGMENTS

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