Response of pumice-rich soils to cyclic loading

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Abstract. Soils containing pumice are frequently encountered on engineering projects in the North Island of New Zealand. The presence of pumice is known to result in different material behaviours, including the resistance to cyclic loading. In this paper, results of triaxial testing on undisturbed specimens of dense, pumice-rich soils are presented, and examined to identify the apparent effects that differing pumice content has on the observed behaviours. It is shown that significant reductions in the cyclic resistance were observed in these soils compared with expectations for hard-grained materials, but that this effect appears to be fully developed with limited amounts of pumice in the soil. It is further shown that the undrained strength is significantly reduced by increasing amounts of pumice and that typical predictions of post-cyclic reconsolidation strains are unconservative in pumice bearing materials.

Keywords: Pumice, Cyclic loading, Undisturbed testing.

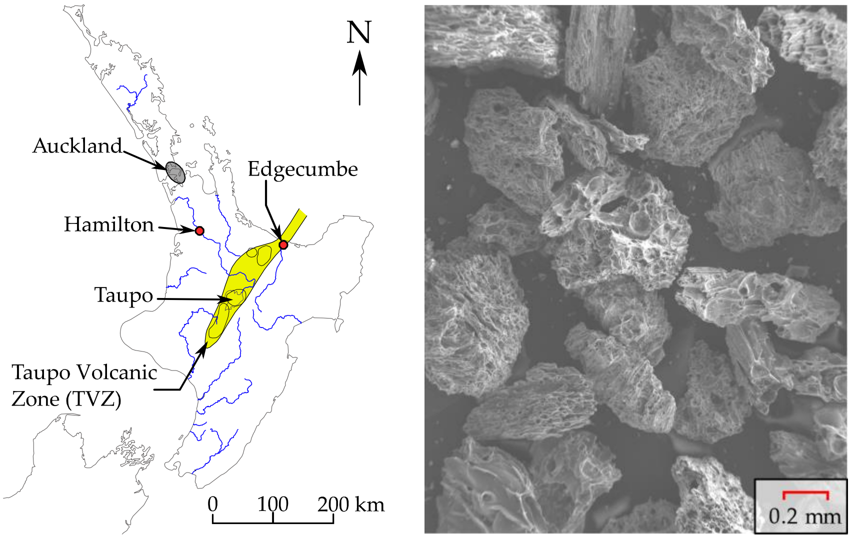
1 Introduction

In New Zealand, strong ground shaking associated with earthquakes forms one of the major geotechnical hazards that must be considered throughout the country. The Canterbury Earthquake Sequence (CES) of 2010-2011 served to highlight the disruptive effects of liquefaction to both structures and infrastructure in an urban setting, particularly in recent alluvial deposits and remains the focus of many research efforts. The impacts of the CES, as well as other earthquakes in the past 30 years, have also highlighted the importance of considering the exposure to earthquake-related phenomena across the country.

The North Island of New Zealand is home to a number of active volcanic fields including the Auckland Volcanic Field [1] and the Taupo Volcanic Zone (TVZ) [2]. The TVZ (shown in Figure 1a) is a North-East trending zone extending from near the centre of the North Island of New Zealand to the Pacific Ocean. The TVZ contains both the Taupo supervolcano, as well as a number of smaller clusters, and is one of the most productive active volcanic areas on the planet. The eruptive products of the TVZ are predominantly rhyolitic [2], producing large volumes of ignimbrites (containing pumice), which are found in both welded and unwelded forms, and cover extensive areas of the central North Island, existing either in their original depositional environment, or having been re-worked and redeposited in river systems. As a result, pumice-rich soils originating from the TVZ are frequently encountered in major engineering projects around the central North Island of New Zealand.

Pumice is a frothy foam formed by the expansion of gas in molten magma as it erupts from a volcano. As a result of their foam-type structure, pumice soil grains tend to be extremely light-weight, crushable, angular and have high surface roughness. Examples of a number of pumice grains are shown in the scanning electron microscope (SEM) images in Figure 1b from which the particular properties of pumice grains can be appreciated. In particular, it has been shown that the crushing strength of pumice grains is typically size-dependent (larger particles are weaker) and approximately 1 order of magnitude lower than typical silica-based materials [3].

The different nature of these materials have led a number of researchers to consider the particular engineering properties of these materials. Studies have shown that pumice based soils tend to develop high friction angles [4,5,6], lower stiffnesses [7], and higher compressibility compared with hard grained materials. Cyclic strengths of these materials have similarly been investigated, both in studies on undisturbed and reconstituted materials [8,9,10,7,11,6], which have highlighted the differences in the development of pore pressure and strains in these materials, as well as the differences in engineering properties which can arise depending on the source of the volcanic materials.



a) b)

Fig. 1. a) Map of North Island of New Zealand showing the Taupo Volcanic Zone and sampling locations. b) Representative photo of pumice soil grains collected from Hamilton

Characterising pumice-rich soil deposits remains an important and open question. In particular, it has been shown that the penetration resistance is significantly affected by the presence of pumice [12]. Hence, there are significant challenges in understanding not just the behaviours of soils containing volcanically-derived material but also how best to characterise these soil deposits using tools available to practice. As part of ongoing work to address this question, a programme of undisturbed sampling has been undertaken at a number of locations in New Zealand, and initial comparisons of the cyclic resistances to existing simplified procedures are discussed by [7,11]. In this paper, the behaviours of high-quality specimens obtained as part of this wider study are examined with a focus on the effect of pumice content and stress level on the observed behaviours.

2 Undisturbed Sampling & Laboratory Testing

2.1 Edgecumbe

Undisturbed sampling was undertaken close to Edgecumbe on the Rangitaiki Plains, and approximately 200 m from the present-day Rangitaiki River. At this location, [13] noted that there was only minor to moderate liquefaction during the 1987 Edgecumbe earthquake.

A conventional borelog indicated that the site consisted mainly of sands to a depth of 4m, and gravelly sands/sandy gravels from 4 to 7m. Pumice was noted in the log at all depths in both the sand and gravel fractions. Data from both cone penetration testing and shear wave velocity profiling are summarised in Figure 2 along with the sampling intervals which were targeted at this site. In this paper, the results from the samples in the sandy layers will be considered. Information regarding the cyclic resistance of the deeper layers is described in [7].

Undisturbed samples were obtained using the GP-TR technique. The GP-TR sampler is similar to the Mazier Core Barrel sampler, and introduces a lubricating polymer gel that coats the exterior of the soil sample as it enters the sample liner [14]. Recovery in the layer between depths of 3-4m was excellent, being close to the theoretical maximum. However it should be noted that the first sample (marked in red in Figure 2) was very soft immediately after sampling, which was considered indicative of a high degree of disturbance. The samples marked 2 and 3 in this depth interval were stiff after recovery and considered to be of very good quality. The open circles shown in Figure 2 indicate the laboratory estimates of shear wave velocity at the in-situ stress level. It can be seen that the measurements from samples 2 and 3 generally fall within 85 – 100 % of the insitu measurements, while those from sample 1 are between 65 and 85 %. Hence it appears that samples 2 and 3 have been recovered in very good condition using the GP-TR technique.

Following sampling, the soil samples were allowed to drain in a vertical orientation prior to uni-axial freezing from top to bottom using dry ice. Samples were kept frozen until testing in the laboratory.

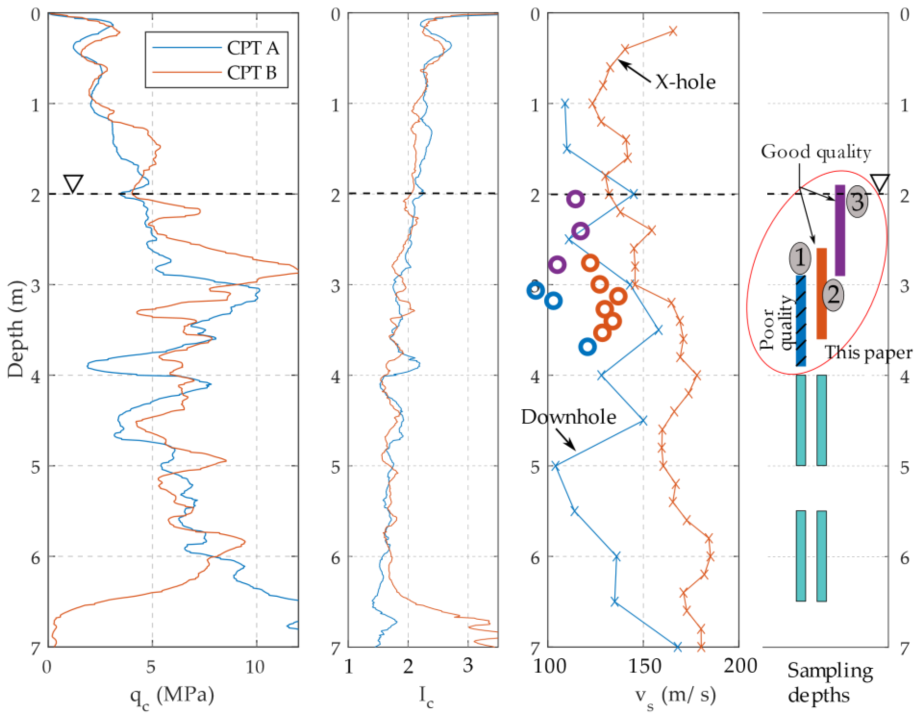


Fig. 2. CPT data, v_s profiles and sampling intervals at the Edgecumbe site.

Figure 3 summarises some key results from the characterisation testing performed after triaxial testing. Note that the sub-specimens from each sample are referred to using a letter (i.e. A to G), which is shown in the figure. It can be seen that the particle size distributions (PSD) of the specimens were relatively similar and confirmed that the specimens from samples 2 and 3 were clean sands with a median grain size of 0.2 mm. Pumice content was determined using a sink/swim method [15] and highlighted significant differences in the pumice content of these specimens which was not apparent before triaxial testing had been performed. As shown, specimens located between 2.85 – 3.7 m had much lower pumice contents than those between 2.1 m and 2.85m. Relative densities are shown for the specimens after consolidation, with limiting densities based on the Japanese standard method [16]. It is apparent that specimens obtained from samples 2 and 3 were extremely dense. When examining Figure 2, it is noticeable that the CPT trace shows an increase in the cone tip resistance between approx. 2.7 and 3.7 m in CPT A (shallower in CPT B). It is noticeable that in the post-test characterisation, the relative density of the specimens is quite similar from samples 2 and 3, while there is a marked reduction in pumice content at around 2.85m. This suggests that the variation in cone penetration resistance may be associated with the change in pumice content, rather than relative density (i.e. [12]).

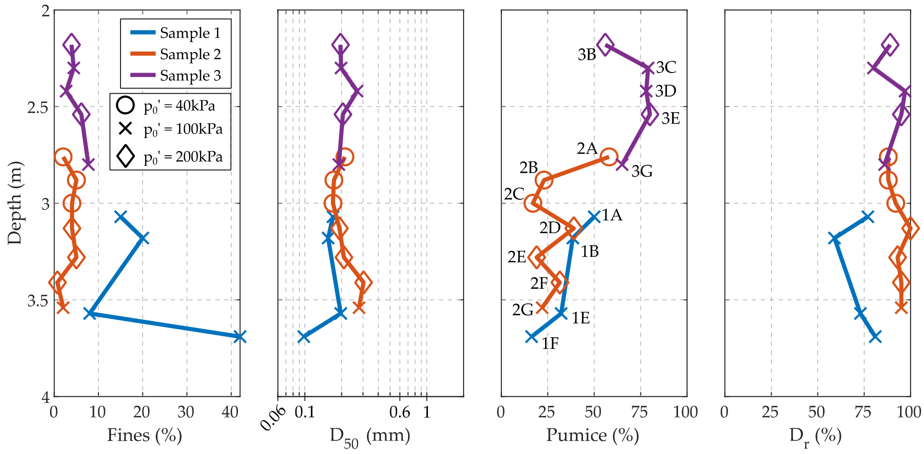


Fig. 3. Particle size, composition and relative density (D_r) of specimens from Edgecumbe site

Frozen specimens of 100 mm length and 500 mm diameter for triaxial testing were prepared using a circular saw, and a core drill (cooled with calcium chloride brine) to reduce the diameter. Small slots were carved in the base and top of the specimens using a hand held drill to accommodate bender elements. Frozen specimens were allowed to defrost in the triaxial apparatus under an isotropic cell pressure of 10kPa. After defrosting, the cell pressure was raised to 20 kPa. Specimens were saturated by percolating CO_2 and deaired water followed by raising the cell and back pressures (maintaining constant p') until a B-value in excess of 0.97 was achieved.

It was decided to test these specimens at mean effective stresses close to their expected in-situ vertical effective stress, as well as higher stresses to investigate whether larger stress levels affected the cyclic response due to crushing. Assuming a saturated unit weight of $14kN/m^3$, the in-situ effective stresses of these specimens are estimated to be in the region of 30 – 35 kPa. Hence specimens were isotropically consolidated to either 40 kPa 100 kPa or 200 kPa. Consolidated specimens were subjected to stress-controlled cyclic loading (CTX), or undrained monotonic loading (CIU). CTX tests were followed by either reconsolidation to the initial effective stress level, or a CIU test phase.

Response to monotonic loading

The stress paths and stress-strain response measured during the monotonic testing of the Edgecumbe specimens is shown in Figure 4. The figure shows that the specimens were dilative on shear, as would be expected for sands with high relative densities. In the interpretation of the CIU data, it is important to note that at the end of the experiments, test "3E" had positive excess pore water pressures, while tests "1E" and "2B" had developed significant negative excess pore water pressures so that in these two tests the final pore water pressure was -25kPa and – 50kPa. Hence it is possible that the end state of tests "1E" and "2B" are being affected by de-saturation, and that the true end

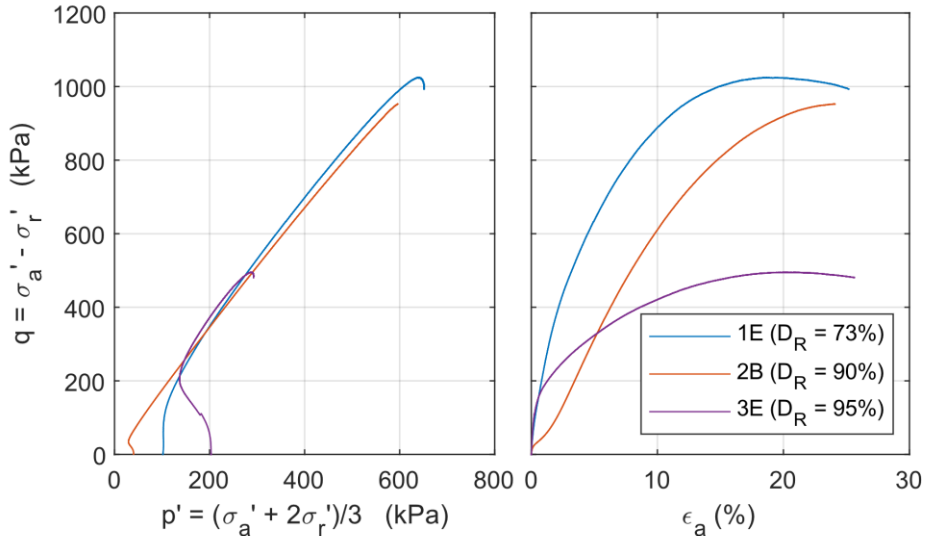


Fig. 4. Response of Edgecumbe specimens to CIU loading.

of the stress path would be at higher stresses than shown. Despite this, there is a clear separation in the end strengths of these tests, with tests 1E and 2B displaying similar ultimate strengths which are approximately double those observed in test 3E. It is interesting to note that despite having the highest relative density, test 3E also had the lowest strength, counter to expectations. Referring to Figure 3, it can be see that specimen 3E had a pumice content around 80% while 1E and 2B had pumice contents of 32% and 23% respectively. Hence it appears that increasing pumice content has the effect of significantly reducing the strength of the specimens in undrained shearing, likely as a direct result of crushing of the soil grains.

In these CIU tests, it is interesting to compare the mobilized friction angles at the maximum deviator stress, which is interpreted here as being representative of the critical state friction angle. Close examination of the data in these plots indicated that the mobilized friction angle in the tests was 39^o in tests 1E and 2B, while it was 42^o in test 3E. These values are relatively large compared with hard-grained sands, where the critical state friction angle might be in the region of $30\text{-}35^o$ [17]. This data is however consistent with other studies [4,18,5,6,12], and suggests that increasing pumice content has an impact on the critical state strength of these materials.

Response to cyclic loading

Cyclic testing on the specimens was performed at a range of cyclic stress ratios and confining pressures. Given the range of pumice contents in the specimens (found after cyclic testing was concluded), it was expected that there would be significant differences in both the effective stress paths and the stress-strain responses. Figures 5 and 6 show the responses from two tests, which were both conducted with an initial mean effective stress of 100 kPa, similar cyclic stress ratios (CSR)s of approximately 0.375

but with a large difference in the pumice content (22% vs 80%). The responses observed in these two tests are remarkably similar in many regards, with large excess pore pressures being developed relatively early in the loading sequence, while the peak-to-peak axial strains developed gradually over continued cycling. When considering these tests, it is important to note that cyclic failure was defined when the specimens had accumulated 5% double amplitude (DA) strain. Note that while these specimens reached 5% DA strain, and had quickly generated large excess pore water pressures, the behaviour is that of cyclic mobility (consistent with the behaviour of denser sands. i.e.[19]) rather than liquefaction (referring to a collapse of the soil specimen). It is also interesting to note that no localization of strain (i.e. necking) was observed in any of the tests, and strains appeared to develop uniformly throughout the specimens.

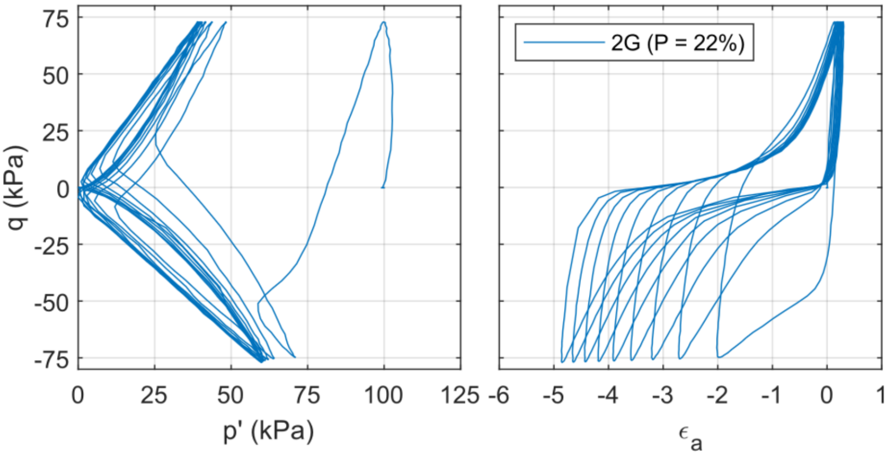


Fig. 5. Cyclic response of Edgecumbe specimen 2G with a low pumice content

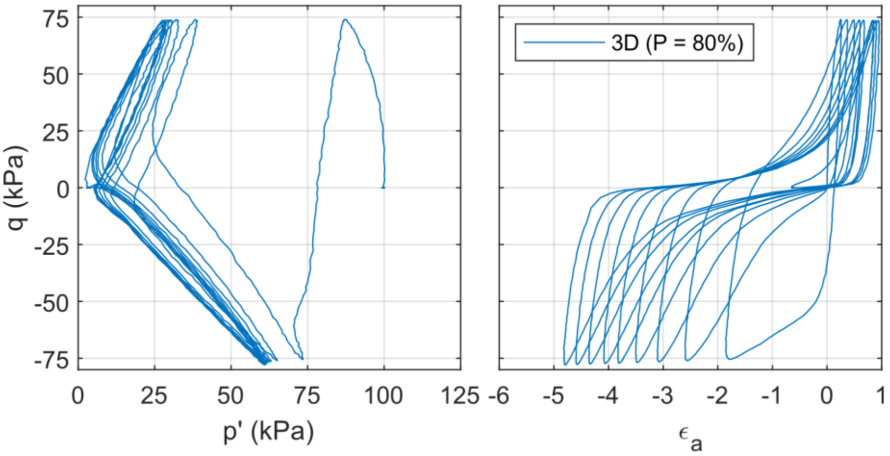


Fig. 6. Cyclic response of Edgecumbe specimen 3D with a high pumice content

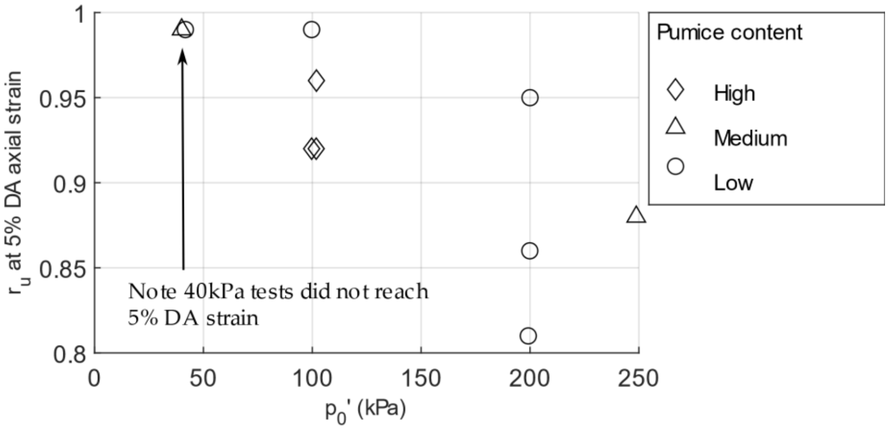


Fig. 7. Excess pore water pressures at 5% DA axial strain and q≈0kPa

When the cyclic responses of the tests of all specimens were compared, it was found that the main difference in the responses came with respect to the inclination of the stress paths once it began the cyclic mobility loops close to the origin. It can be observed that the specimens with high pumice content generally exhibited steeper effective stress paths after phase transformation, indicating more dilative behaviour which may be a result of the surface roughness and angularity of these particles creating better interlocking. Across the spread of the tests, it was also noted that there appears to be a trend that the increase in excess pore water pressure ratios (r_u) at 5% DA axial strain (measured at instances when $q \approx 0$ kPa) appeared to decrease slightly with increasing initial effective stress level (Figure 7). This same effect has been reported in testing by [5]. Surprisingly, no obvious trend appeared for excess pore water pressure ratios at the same instants when compared against the pumice content with this limited data set.

A summary of the number of cycles of uniform load required to cause the development of 5% double amplitude strain is shown in Figure 8. As shown, there were only two tests undertaken on specimens at 40kPa, at CSR values of 0.21 and 035. Less than 2% DA axial strain developed after more than 100 loading cycles in both tests. The shear wave velocities measured in these specimens were generally consistent with the other specimens from samples 2 and 3 (Figure 2), and hence did not have clear indications of cementation or other factors which would be significantly different to the other specimens. Hence further testing was not performed at this lowest stress level.

When viewing the results shown in Figure 8, two sets of data are shown for tests performed with initial mean effective stress, p_0' = 100kPa, corresponding to tests performed on specimens from sample 1 (open markers) and specimens taken from samples 2 and 3 (closed markers). It can be seen that the cyclic resistance of specimens from samples 2 and 3 is significantly higher than those from sample 1, which is due to the disturbance of sample 1 as noted earlier. The significant reduction in the cyclic resistance as a result of disturbance is a known effect (i.e. [20]) and lends confidence to the results from the remaining specimens.

It is clear that the increase in the initial mean effective stress from 40 kPa (close to the insitu vertical stresses) to 100 kPa has significantly reduced the cyclic resistance of the soil specimens. It can be seen that the specimens tested at the highest stress level (i.e. $p_0' = 200$kPa) had more scatter in the results, but fell largely in the same band as the results from the tests performed at 100 kPa, suggesting that the change in cyclic resistance due to stress level takes place at relatively low stress levels only. The causes for this effect are unknown at this time, but may be due to the inherent crushing strength of the pumice grains being exceeded at the higher stress levels. It might be expected that the increased stress levels would lead to greater crushing taking place, leading to a more contractive response. However, it was previously noted that a reduction in r_u seemed to occur with increasing stress levels, and therefore it is speculated that an additional mechanism causing the reduction in CSR is the localized breakage of the pumice grain cell walls allowing particles to slide past one another more easily.

A final observation to note is that in these results, the pumice content of the specimens (excluding those tested at $p_0' \approx 40$kPa) does not seem to make a large difference to the observed cyclic resistance, despite specimens having pumice contents varying approximately in the range of 20-80%. It is also noticeable that the cyclic resistance of these specimens (the CSR to cause failure in 15 cycles is approximately 0.31) is extremely low considering the relative density is around 90-100%. [20] reported that frozen samples of Niigata sand with $D_R \approx 80$-90% was around 0.9. Hence the cyclic resistance ratio (CRR) of the test specimens reported here is very low in comparison with hard-grained material and suggests that the major influences of pumice on the cyclic resistance of the soil takes place at relatively low pumice contents.

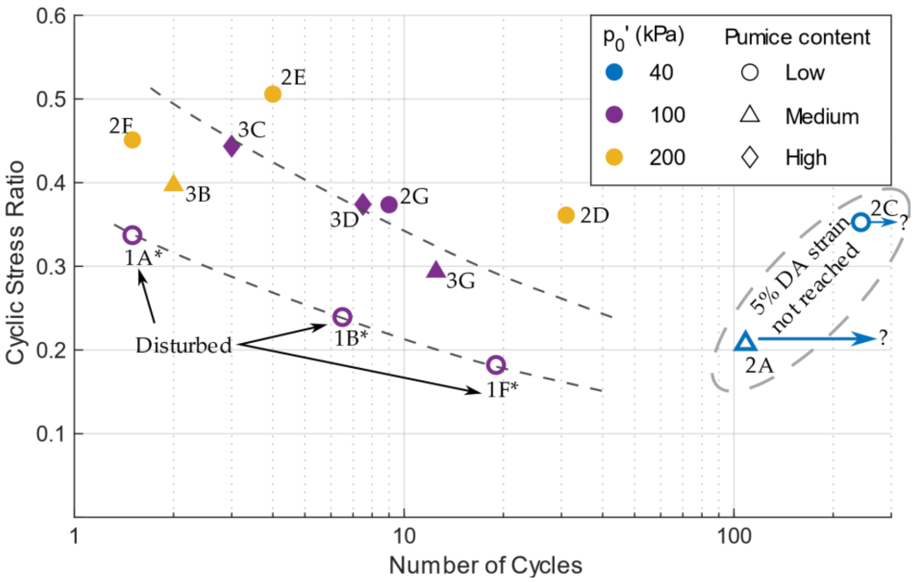


Fig. 8. Cyclic stress ratio required to cause 5% double amplitude axial strain.

2.2 Hamilton

Undisturbed sampling was conducted at a second site within the city of Hamilton (see Figure 1) approximately 60 m from the Waikato river. CPT and shear wave data collected at this site are shown in Figure 9 below, as well as the sampling intervals which were targeted. The targeted soils were located in a layer known as the Taupo Pumice Alluvium, which exists close to the river and is up to 30 m thick. This layer was formed as a result of a breakout flooding event after the most recent eruption of the Taupo volcano in approximately AD 250 [21].

The specimens which are discussed here were obtained from between 5 and 6 m below the ground surface, in a layer where cone resistance was approximately constant at 7 MPa. Sampling and testing procedures were identical to those at the Edgecumbe site. A comparison of the shear wave velocities measured in-situ and laboratory estimates are shown in Figure 9, which suggests that high-quality specimens were obtained.

Table 1. Index data for Hamilton specimens

Specimen	Depth (m)	Relative Density (%)	Pumice Content (%)	D_{50} (mm)	Fines Content (%)
HE	5.34	(not meas)	90	0.288	18
HD	5.46	(not meas)	91	0.192	20
HC	5.58	79	83	0.180	15
HB	5.7	67	90	0.147	21
HA	5.82	69	70	(not meas)	(not meas)

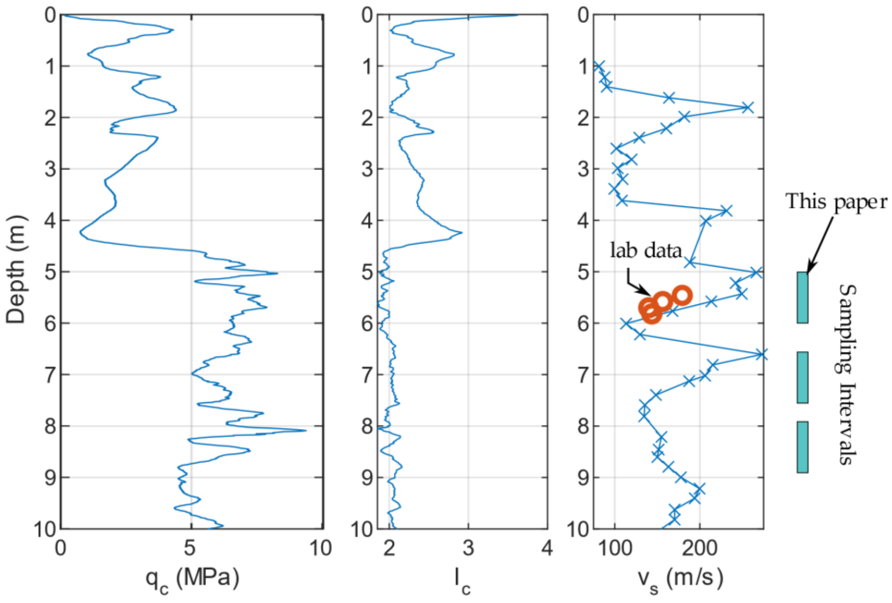


Fig. 9. CPT and v_s data for the Hamilton site

Post-test characterisation data is shown in Table 1. As shown, these specimens have a relative density of approximately 70%, have extremely high pumice contents, and are reasonably well graded sands with up to 20% fines and 20% gravel contents.

It is estimated that the vertical effective stresses of these specimens were close to 80-85 kPa. Triaxial testing of these specimens was therefore undertaken with mean effective stresses of approximately 90 kPa, and of the five specimens obtained from the sample at this depth, four were tested for cyclic resistance, and one was tested in undrained shear (CIU – specimen "HD"). In the previous section, the details of the monotonic and cyclic responses of Edgecumbe specimens were discussed. The specimens at the Hamilton site showed similar behaviours – the monotonic testing revealed an overall dilative response, as expected given its relative density, and the undrained shear strength of the specimen was comparable to that specimen "3E" in Figure 4. Under cyclic loading, the specimens from the Hamilton site developed large excess pore water pressures early in the cyclic loading, while the deformation gradually accumulated as a result of cyclic mobility. An overall summary of the cyclic resistances of the specimens from the Hamilton site is shown in Figure 10, along with the high quality specimens from the Edgecumbe site that were tested at 100 kPa (i.e. comparable to the testing pressures for the Hamilton specimens).

The data in Figure 10 shows similarity between the cyclic resistances of the specimens from the Hamilton and Edgecumbe sites when tested at similar initial mean effective stress levels, despite the differences in the physical location, gradation and relative density of the specimens. The reasons for the similarity are still being investigated, but given the similarity in the undrained strengths of the materials at the two sites, it is possible that the crushing strength of pumice grains is a controlling factor in the cyclic resistance of these materials.

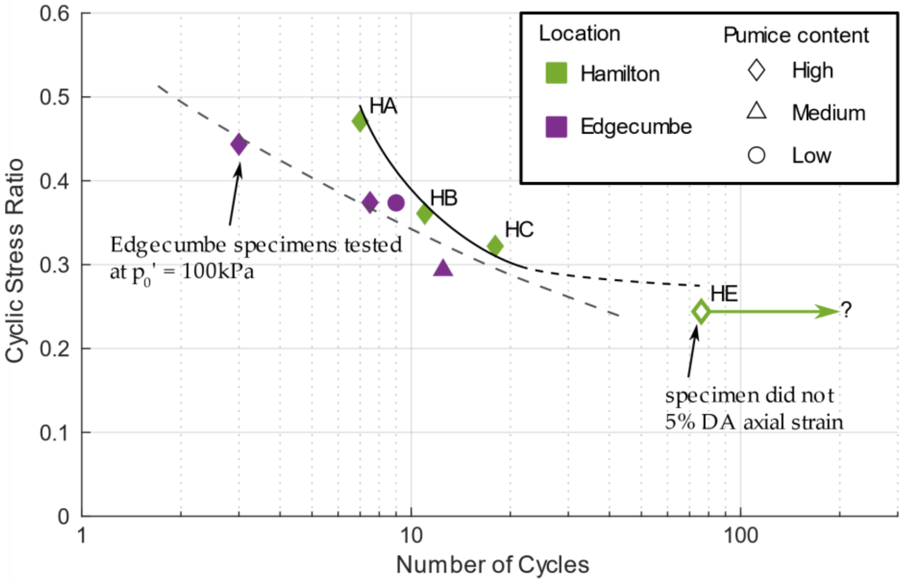


Fig. 10. Cyclic resistance of specimens from the Hamilton site

Post cyclic responses of test specimens

In the assessment of the potential consequences of liquefaction, it is common to estimate the free field settlement at a given site, often as the integration of expected one dimensional vertical strains due to the reconsolidation of the soil as excess pore water pressures dissipate. It is also the case that the amount of volumetric strain which takes place is of concern as it defines the volume of water that is flowing through the soil, potentially increasing the severity of shear induced deformations in the ground by preventing the recovery of effective stresses. The reconsolidation strains were therefore measured after a number of the cyclic tests, and have been plotted against the maximum cyclic shear strain in Figure 11, separated into bands of relative density and pumice content. Also shown in the figure are the relationships proposed by [22] for hard-grained soils at the mid-point of each band of relative density. It should be noted that some of the data points shown correspond to tests where multiple loading series were applied (i.e. specimens were subjected to cyclic loading followed by reconsolidation, followed by additional loading etc). In tests where additional loading sequences were applied, the specimens appeared to get stronger, and displayed none of the problems often encountered with hard-grained soils (i.e. no necking or localized loosening was observed).

The results shown indicate that the existing relationships linking volumetric strains to the maximum shear strain during cyclic loading are unconservative, especially as the relative density of the soil increases. As shown, the majority of the data belonged to the grouping of soils with relative densities higher than 80% (purple markers). Within this grouping, it can also be seen that there is a trend of increasing reconsolidation strains with pumice.

It may be recalled that there is significant variation in the initial mean effective stress of the specimens, and therefore the change in stress which the soil undergoes when it is reconsolidated. However, when the data was closely examined, it was not possible to determine a clear trend in the data which was associated with the different stress levels during testing. Hence it appears that the main factor causing the increase in volumetric strains during reconsolidation is the pumice content of the soil.

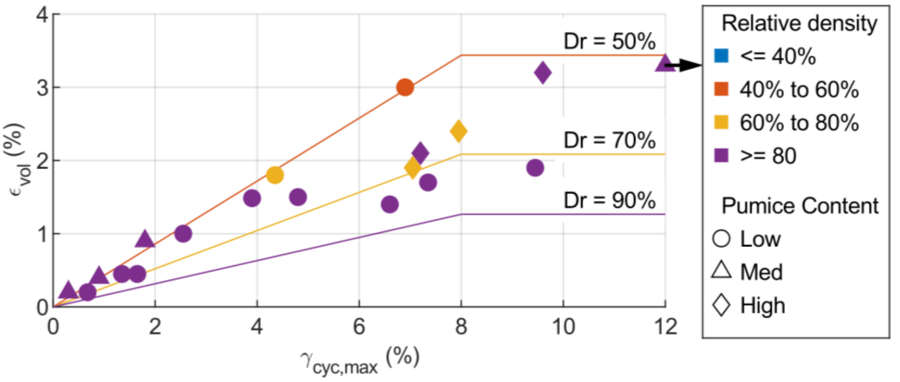


Fig. 11. Volumetric strains during reconsolidation of soil specimens. (solid lines show predicted strains based on [22])

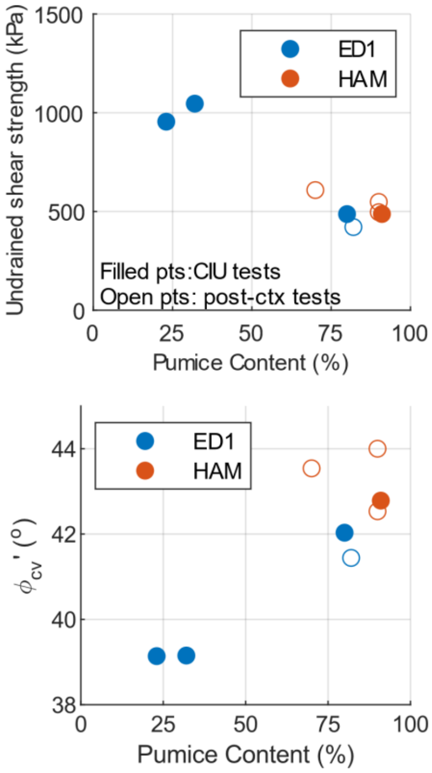


Fig. 12. Variation of undrained strength parameters with pumice content.

A final aspect of the testing on the undisturbed specimens considered the strength of the soils under undrained conditions. These aspects have been previously discussed in Section 2.1.1, and in particular highlighted that the strength of the specimens reduced with increasing pumice content. In addition to the monotonic testing which had previously been discussed, some of the specimens were subjected to undrained shearing after the cyclic loading in order to determine the ultimate strength of the specimens,and to verify whether the accumulated effects of cyclic shearing had any significant impact on the observed strength. The results from this phase of the testing are summarized in Figure 12, where both the undrained strength (left plot) and the effective stress friction angle at the undrained strength (ϕ'_{cv}) are plotted against the pumice content of the specimen. It can be observed that in both cases, the pumice content is strongly affecting these strength parameters; the ultimate strength being reduced strongly as a result of the crushability of the soil grains, while the mobilized friction angle at the point when the undrained strength was attained increases with the pumice content. This second result is likely the effect of the extreme angularity and surface roughness of the pumice particles, which exists for the soil grains in both their original and broken state resulting in the large friction angles with these materials under undrained conditions.

3 Concluding Remarks

In this paper, the results from a series of triaxial tests on high quality undisturbed samples have been presented, and represent the behaviours of dense, alluvial deposits containing a wide range of pumice contents.

It has been shown that there are significant effects arising from the presence of pumice which has affected the undrained strength of the soil, the cyclic resistance and the volumetric reconsolidation strains.

A key result from the testing is that while the undrained strength reduces over a wide range of pumice contents, the cyclic resistance may be affected over a very narrow range of pumice content. In other words, the major changes in cyclic resistance of pumice bearing soils may be observed at relatively low proportions of pumice (i.e. 0% to 30%) in the soil mixture.

Significant reduction in the cyclic resistance of the specimens from Edgecumbe was observed when the initial effective stresses were raised from 40kPa to 100 kPa and higher, suggesting that careful consideration must be given to stress level when evaluating the behaviour of these soils.

Finally, it should be acknowledged that the results from only two sets of undisturbed samples have been presented here, which have allowed some preliminary observations to be made regarding the effects of pumice content on the behaviour of undisturbed soil deposits. The behaviour of undisturbed samples are invariably affected by many factors which include fabric/structure, ageing and cementation as well as complex stress histories. While these additional aspects have not been investigated as part of this study they may have important influences on the results obtained. Further laboratory testing on pumice-rich soils, both in undisturbed and reconstituted states is therefore recommended to help clarify the effects of pumice content and stress level as well as the specific mechanisms which are responsible for these effects.

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