Liquefaction characteristics of crushable pumice sand

Caractéristiques de liquéfaction des sables de pierre ponce sensibles à l'écrasement

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ABSTRACT: Because of their highly crushable nature, there are concerns on whether liquefaction evaluation procedures which were empirically derived from hard-grained sands are applicable to pumice sands. To understand their liquefaction characteristics, several series of undrained cyclic triaxial tests were performed on both reconstituted and undisturbed pumice specimens. Results showed that the effect of relative density on the liquefaction resistance of pumice sands was not as significant when compared to that of hard-grained sands. Correlations were also made between the liquefaction resistance from the cyclic tests on undisturbed specimens and those from field tests conducted at sites where the samples were obtained. CPT and sDMT-based empirical methods of estimating the cyclic resistance ratio did not agree well with the laboratory-obtained values, while that based on shear wave velocity produced better correlation. The results obtained can be used to evaluate the in-situ liquefaction potential of pumiceous deposits.

RÉSUMÉ : En raison de leur forte sensibilité à l'écrasement, il y a des doutes quant à savoir si les méthodes d'évaluation de liquéfaction, obtenues de façon empirique avec des sables à grains durs, sont applicables à la pierre ponce. Pour comprendre les caractéristiques de liquéfaction, plusieurs séries de tests triaxiaux cycliques non drainés ont été effectués sur des échantillons de pierre ponce remaniés et intacts. Les résultats ont montré que l'effet de la densité relative sur la résistance à la liquéfaction des sables de pierre ponce n'était pas aussi important comparé à celui sur sables à grains durs. Des corrélations ont également été faites entre la résistance à la liquéfaction obtenue par les essais cycliques sur échantillons intacts et les résultats des essais in-situ menés sur les sites d'échantillonnage. Les méthodes empiriques d'estimation du rapport de résistance cyclique, basées sur des essais CPT et SDMT, ont sous-estimé les valeurs obtenues en laboratoire, tandis que celles basées sur la vitesse des ondes de cisaillement ont produit de meilleures corrélations. Les résultats obtenus peuvent être utilisés in-situ pour évaluer le potentiel de liquéfaction des dépôts de pierre ponce.

KEYWORDS: Pumice deposits; undrained cyclic test; liquefaction; cyclic resistance ratio; particle crushing.

1 INTRODUCTION

The recent earthquakes in Christchurch have demonstrated the impact of soil liquefaction to the built environment (e.g., Orense et al. 2011). With the central government, local councils and community residents in New Zealand now fully aware of the devastating effects of earthquakes in general and of soil liquefaction in particular, attention has shifted to the seismic performance of local soils, i.e., whether soils in certain localities will undergo the same degree of liquefaction as the Christchurch soils did.

Pumice deposits, which originated from a series of volcanic eruptions centred in the Taupo and Rotorua regions, are found in several areas of the North Island. They are frequently encountered in engineering projects and their evaluation is a matter of considerable geotechnical interest. Because of their lightweight, highly crushable and compressible nature, they are problematic from engineering and construction viewpoint. Moreover, no information is available as to whether empirical correlations and liquefaction procedures derived for hard grained soils are applicable to pumice deposits because there has been very little research done on their characteristics.

The authors presented preliminary results of an experimental programme they conducted to investigate the undrained cyclic characteristics of undisturbed and reconstituted pumiceous soils through cyclic triaxial testing (Orense et al. 2012). This paper further discusses more undrained cyclic triaxial results. Moreover, geotechnical investigations, including cone penetration testing (CPT) and seismic dilatometer testing (sDMT), were conducted at the sites where the undisturbed pumice samples were obtained. Finally, the validity of the conventional methods of evaluating the liquefaction resistance developed for hard-grained sands was examined to see if they are applicable to crushable soils like pumice.

2 MATERIALS USED

Two sets of materials were used in the triaxial tests. The first set consisted of undisturbed pumiceous soils obtained through push tube sampling at two sites in Waikato in central North Island: (1) at Carrs Rd in Hamilton; and (2) Mikkelsen Rd in Waihou. The samples from Carrs Rd site, which were obtained at depths between 8.0-8.5 m using 60 mm push tubes, were completely to heavily weathered ignimbrite. The closest SPT N-value was 18 (at depth=15m). The undisturbed Mikkelsen Rd samples were sourced at three depths: 3.0-3.3m, 6.0-6.6m and 12.0-12.4m, using 60mm push tubes. The SPT N-values were 11 and 13 at depth=4.5m and 6.6m, respectively. Because of its loose nature (some cores were lost), the push tubes were placed in a freezer for 1-2 days before the samples were extracted. Although the two sets of samples thus taken may have been "disturbed" one way or the other by the sampling, the degree of disturbance may be considered insignificant. In addition, care was taken during handling and transport; thus they are referred to as 'undisturbed" in this paper.

The other set of materials used was commercially-available pumice sand. This is not a natural deposit but was derived by processing sand from the Waikato River. The particles were centrifugally separated from the other river sand particles so that the samples consist essentially of pumice grains.

The properties of the tested soils, obtained using methods based on NZ Standards (1986), are shown in Table 1 and the grain size distribution curves are shown in Figure 1.

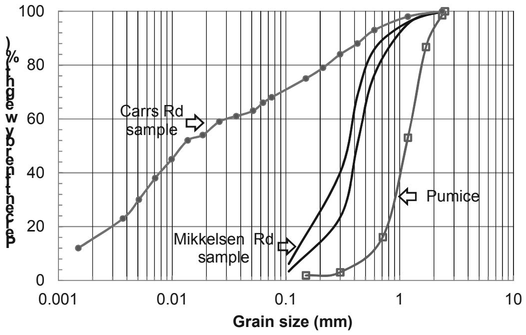


Figure 1. Grain size distribution curves of soils used in the tests.

Table 1. Properties of soils used.

Material	Specific Gravity	Maximum void ratio	Minimum void ratio
Carrs Rd	2.52	N/A*	N/A*
Mikkelsen Rd	2.49	1.165	0.717
Pumice sand	1.95	2.584	1.760

*Not applicable since sample has very high fines content

3 EXPERIMENTAL METHOD

3.1 *Undrained cyclic triaxial tests*

The undisturbed soil samples obtained from the site using 60mm push tubes were carefully transported to the laboratory. In case the soil sample was deemed stable, they were extracted from the tube using hydraulic jack. In some cases, the soil sample was deemed loose and extracting them straight away will destroy the structure and fabric; this was noted in the soil samples taken from shallow depths (3 m and 6 m) at Mikkelsen Rd site. In these cases, the tubes with the soil sample inside were placed in a freezer for 1-2 days. Then, the frozen specimen was extruded from the sampling tube. Trimming was carried out at the two ends of the specimens for the preparation of square ends. The height of the specimen used was 120mm for the Carrs Rd samples, and 100mm for Mikkelsen Rd samples (due to sample unavailability). Filter papers were placed at the ends to prevent clogging of the porous discs. The specimen was placed inside a rubber membrane and, for frozen specimens, they were allowed to thaw prior to testing. Saturation of the specimen was ensured by allowing water to enter the specimen by increasing the back pressure. B-value check was carried out to confirm that fully saturated condition had been achieved. Specimens were then isotropically consolidated at the target effective confining pressure, σ_c'.

For the reconstituted specimens, it was not easy to completely saturate the pumice sand because of the presence of voids from the surface to the particle interior. For this purpose, saturated specimens were made using de-aired pumice sands, i.e., sands were first boiled in water to remove the entrapped air. To prepare the test specimens, the sand was water-pluviated into a two-part split mould which was then gently tapped until the target relative density was achieved. Next, the specimens were saturated with appropriate back pressure and then isotropically consolidated at the target effective confining pressure, σ_c'. B-values > 0.95 were obtained for all specimens. The test specimens were 75mm in diameter and 150mm high.

The cyclic loading in the tests were applied by a hydraulic-powered loading frame from Material Testing Systems (MTS). A sinusoidal cyclic axial load was applied in the tests at a frequency of 0.1Hz under undrained condition. In addition to the axial load, the cell pressure, pore pressure, volume change and axial displacement were all monitored electronically and these data were recorded via a data acquisition system onto a computer for later analysis.

3.2 *Field tests*

In this paper, the results of two sets of field tests are discussed: cone penetration testing (CPT) and seismic dilatometer testing (sDMT). These tests were performed near the two sites where the undisturbed soil samples were obtained. The CPT was performed every 100mm depth interval, while the sDMT was carried out at every 500mm interval, with the first reading taken at 1m depth from the ground surface. An electrically-operated Autoseis hammer was used to generate a shear wave that propagated through the ground. The shear wave signals were recorded by the geophones in the seismic module and were sent back to a computer system as seismographs for analysis purposes. The seismographs from both geophones were shown as similar waves but with the time lag due to the fact that one of the geophones is 500 mm deeper than the other. A computer program allowed the two seismographs to be re-phased and so that the actual travel time difference of the shear wave could be calculated. The shear wave velocity of the soil layer between the two geophones was calculated from the interval between the two geophones divided by the difference in travel time.

4 TEST RESULTS AND DISCUSSION

4.1 *Effect of density on reconstituted pumice specimens*

Orense et al. (2012) discussed the effects of relative density on the liquefaction resistance of reconstituted pumice sands. The curves for dense pumice specimen (D_r=70%), loose pumice specimen (D_r=25%), as well as for the undisturbed Mikkelsen Rd sample obtained (at 6.0-6.6m), corresponding to double amplitude axial strain ε_{DA}=5% are reproduced in Figure 2. The slope of the curve for loose sand is gentle when compared to that of dense sand, with the latter having higher cyclic resistance. On the other hand, the slope of the curve for undisturbed sample is as gentle as the loose reconstituted samples, but the CSR (=$\sigma_d/2\sigma_c$', where σ_d is the deviator stress) is about three times higher.

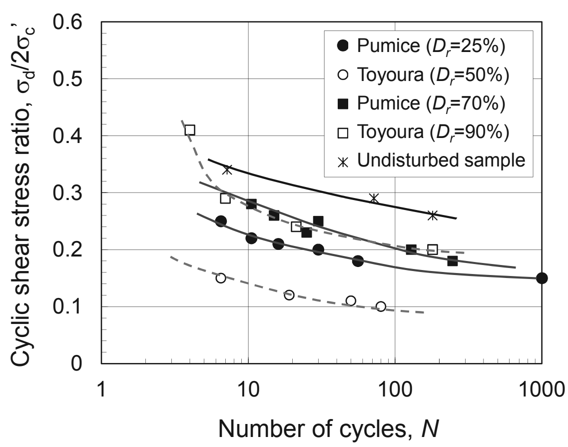


Figure 2. Cyclic resistance curves for the samples used.

Also plotted in the figure are the cyclic resistance curves for loose (D_r=50%) and dense (D_r=90%) Toyoura sand, as reported by Yamamoto et al. (2009). Comparing the curves for Toyoura sand and for reconstituted pumice sands, two things are clear: (1) loose specimens have gentle cyclic resistance curves, while dense specimens have curves rising sharply as the number of cycles decreases; and (2) while the effect of relative density is very pronounced for Toyoura sand, the effect of relative density on pumice specimens appear to be not as remarkable.

4.2 *Effect of confining pressure*

Next, the influence of effective confining pressure on the liquefaction resistance of reconstituted pumice sands was investigated. For this purpose, dense pumice sand specimens (initial void ratio, e_i=1.90-2.00) were subjected to three different levels of effective confining pressure, σ_c'=35, 100 and 500 kPa under different levels of cyclic shear stress ratio, CSR (=$\sigma_d/2\sigma_c$'). Figure 3 illustrates the confining pressure dependency of liquefaction resistance for reconstituted pumice. It can be seen that the curves are almost parallel to each other, with the liquefaction resistance increasing as the confining pressure decreases, consistent with the observations made on natural sands (e.g., Rollins and Seed, 1988). The value of the correction factor for overburden stress K_σ (CSR causing ε_{DA}=5% in 15 cycles under any confining pressure normalised to the corresponding value of CSR at σ_c'=100 kPa) is equal to 1.16 for σ_c'=35 kPa and 0.88 for σ_c'=500 kPa. These values appear to coincide with those reported for reconstituted natural sands (e.g., Boulanger and Idriss, 2004).

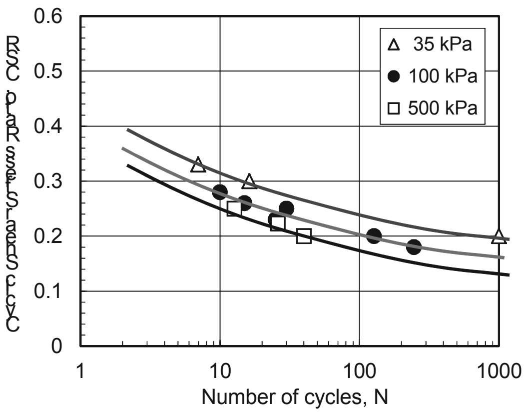


Figure 3: Comparison of liquefaction resistance curves for reconstituted dense pumice sands under different σ_c'.

4.3 *Development of particle crushing during cyclic loading*

The level of particle crushing during undrained cyclic testing has been reported by Orense et al. (2012). They noted that under the confining pressures considered, pumice undergoes remarkable particle crushing when subjected to cyclic shear. As cyclic shearing and particle crushing occur, the soil structure is gradually stabilized, resulting in higher cyclic shear resistance, even exceeding that of Toyoura sand. The cyclic shearing and the associated particle breakage resulted in stable soil structure for both dense and loose cases, and therefore, the effect of density was not as remarkable when compared to the cyclic behaviour of Toyoura sand, a hard-grained sand.

To elucidate further the development of particle crushing during a cyclic loading, a series of tests were performed such that the tests were terminated after a specified number of cycles afterwhich sieve analyses were performed. For these tests, virgin samples were used at each test. A confining pressure of σ_c'=100 kPa was considered, with the void ratio set at e_i=1.90-2.00. For CSR=0.10, the sieve analyses were carried out: (1) on the virgin samples; (2) after the end of consolidation stage; (3) after N=10 cycles; (4) after N=100 cycles; and (5) after N=1000 cycles. On the other hand, for CSR=0.20, sieving was done (1) after N=10 cycles; and (2) after N=83 cycles where initial liquefaction (pore pressure ratio, r_u=100%) occurred.

The grain size distributions of the specimens after the tests were determined. Particle crushing occurred, but with the level of CSR and the number of cycles applied, it was difficult to use the grading curves to make reasonable comparison. Instead, a method of evaluating particle crushing originally proposed by Miura and Yamanouchi (1971) was used which involves the quantification of the surface area of the particles. The specific surface of the particles was measured by first sieving the soil using 2.5 mm, 2.0 mm, 1.18 mm, 0.5 mm, 0.212 mm, 0.15 mm and 0.063 mm sieve sizes. For this range of particle sizes, the specific surface area (in mm^2/mm^3) is calculated as:

$$S = \sum \frac{F}{100} \cdot \frac{4\pi (d_m/2)^2}{(4/3)\pi (d_m/2)^3 G_s \gamma_w} \cdot \gamma_d \qquad (1)$$

where $d_m=(d_1 \cdot d_2)^{0.5}$, d_1 and d_2 are adjacent sieve sizes (e.g., 0.50mm and 0.212 mm), F is the % by weight retained on the sieve, G_s is the specific gravity of the particles, γ_w is the unit weight of water and γ_d is the dry unit weight of the specimen.

Figure 4 shows the development of the surface area S for the different tests described above. Firstly, it was observed that consolidation at 100 kPa effective confining pressure did not induce appreciable particle breakage to the pumice particles; however, the cyclic shearing did. Secondly, the degree of particle crushing increased with the amplitude of applied CSR. For the test with CSR=0.20, the increase in surface area during the initial stage of cyclic loading was small; however, as the liquefaction stage was reached (N=83), the surface area increased remarkably because large strains occurred with associated translation and rotation of particles causing the higher degree of crushing. For CSR=0.10, the state of liquefaction did not occur even when N=1000 cycles. Particle breakage was more or less gradual, with almost linear variation with the logarithm of N.

4.4 *Comparison between laboratory and field data*

Cone penetration tests (CPT) and seismic dilatometer tests (sDMT) were performed at the Mikkelsen Rd site and Carrs Rd site to supplement the undrained cyclic triaxial tests conducted on the undisturbed samples taken from these sites. The field tests were performed as near as possible to the sampling site. Correlations between the cyclic resistance obtained from the laboratory tests and the in-situ parameters were performed to confirm which method was appropriate for pumice. Note that undisturbed soil samples were obtained at three elevations at Mikkelsen Rd site, while samples from Carrs Rd site were taken only at a single depth; hence, emphasis is placed on the former. In addition, the results presented herein may be appropriate only for the two sites investigated and further tests are necessary to confirm their applicability to other pumiceous sites.

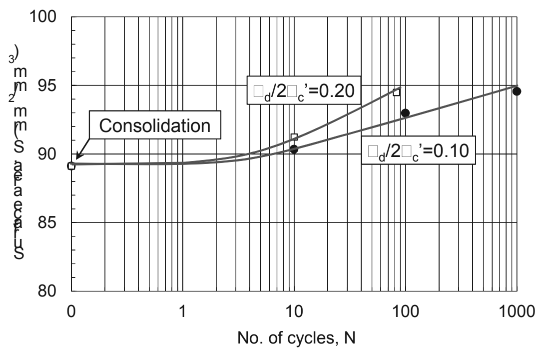
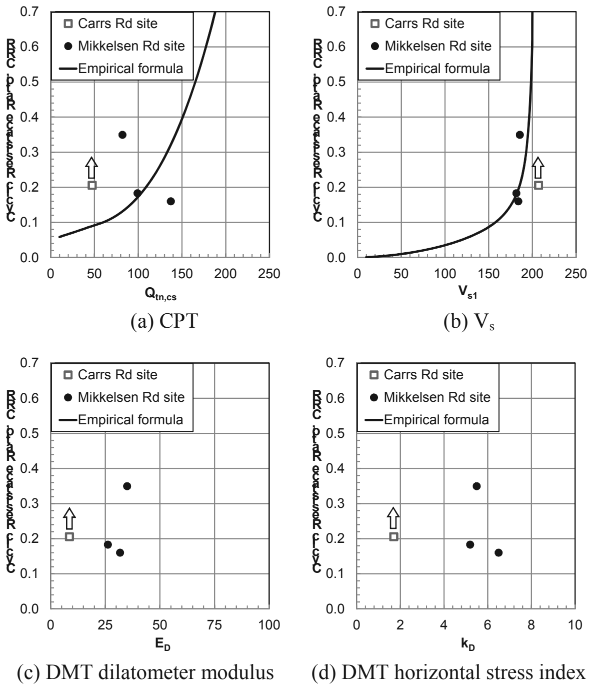


Figure 4. Relationships between specific surface area and number of cycles during cyclic undrained tests.

In the comparison, the liquefaction resistance of the undisturbed samples is specified in terms of the magnitude of

CSR required to produce ε_{DA}=5% in 15 cycles of uniform load application; herein, this is referred to as $(CRR)_{triaxial}$. The conditions the laboratory specimens were subjected to were different from those in-situ and corrections need to be applied to the laboratory-obtained values before comparing with the in-situ liquefaction resistance, $(CRR)_{field}$. Due to space constraints, these corrections are not presented in detail here; suffice it to say that the following corrections were incorporated: (1) correction due to difference in consolidation stress, C_1; (2) correction due to sample disturbance, C_3; (3) correction due to densification during handling, C_4; and (4) correction due to loading direction, C_5. Moreover, all results are expressed in terms of σ_c'=100 kPa using K_σ interpolated from Figure 3.

Figure 5 shows the plot of the $(CRR)_{triaxial}$ vs $(CRR)_{field}$ estimated from the following empirical formulas: (a) from normalized CPT tip resistance, $Q_{tn,cs}$ (Robertson and Wride, 1998); (b) normalized shear wave velocity V_{S1} (Andrus and Stokoe, 2000); (c) dilatometer modulus, E_D (Tsai et al. 2009); and (d) horizontal stress index, k_d (Tsai et al. 2009). Note that for Carr Rd specimen, only ε_{DA}=2% was achieved in the tests and therefore, $(CRR)_{triaxial}$ should be higher than the value measured, as indicated by the arrow sign in the figure. It can be seen that penetration-based methods (CPT and DMT) do not correlate well with the laboratory-obtained cyclic resistance. It is hypothesized that the shear stresses during penetration were so severe that particle breakage formed new finer grained materials, the mechanical properties of which were very different from the original pumice sand. On the other hand, empirical method based on shear wave velocity seemed to produce good correlation with liquefaction resistance of pumiceous soils. Although the V_s in this research was obtained from SDMT where the penetrating rod may have induced particle breakage in the adjacent zone, the shear waves travelled through the intact grains and not on the crushed ones.



(a) CPT (b) V_s

(c) DMT dilatometer modulus (d) DMT horizontal stress index

Figure 5: Comparison between laboratory obtained CRR and those from field-derived parameters.

It should be mentioned that only 4 tests were performed in this study, and more tests are required to validate this observation. Detailed studies on the percentage of pumice in the soil specimens may also be warranted.

5 CONCLUSIONS

In order to investigate the liquefaction characteristics of pumice sands, several series of undrained cyclic triaxial tests on reconstituted and undisturbed pumice specimens were performed as well as geotechnical investigations at sites of pumiceous deposits. The major results are as follows:

(1) Although relative density has some noticeable effect on the cyclic resistance of pumice, it was not as significant when compared to that observed for hard-grained sands.

(2) As the confining pressure was increased, the liquefaction resistance curve of reconstituted pumice specimens was shifted downward and the resistance reduced, consistent with the observations made on hard-grained sands.

(3) During the initial stage of shearing, the increase in surface area (as a result of particle crushing) was small; however, as the liquefaction stage was reached, the surface area increased remarkably because large strains occurred with associated translation and rotation of particles causing the higher degree of crushing.

(4) Among the in-situ methods tested, the empirical method based on shear wave velocity seemed to produce good correlation with liquefaction resistance of pumiceous soils.

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