THE DYNAMIC PROPERTIES OF A PUMICEOUS SAND

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ABSTRACT

The seismic site response analysis of sand deposits requires an understanding of the dynamic properties of the soils involved. Most dynamic soil data available in the literature has been derived for sands which do not contain pumice. Consequently, the relevance of this data to the behaviour of pumice sands is unclear. An extensive experimental investigation of the dynamic response of a pumice sand was therefore undertaken. The liquefaction response obtained from cyclic triaxial tests, and the shear modulus variation with strain amplitude observed in bender element and dynamic torsion tests were examined. The cyclic triaxial test results indicated that the liquefaction response was similar to that observed for quartz sands. However, the low strain shear modulus of the pumice sand was found to be significantly less than that of quartz sands at similar relative densities, and the nonlinear stress-strain behaviour was markedly different from that of other sands, particularly in the mid strain range.

INTRODUCTION

A recent study by Holzer [5] stated that approximately 98% of the US $5.9 billion in property damage from the 1989 Loma Prieta earthquake was caused directly by ground shaking. Amplified ground shaking from site effects was responsible for approximately two thirds of that property damage. Another 2% of the damage cost was attributed to permanent ground deformations. It is clear from this that ground shaking characteristics and local site amplification must be considered if realistic structural design loads and hazard mitigation are to be achieved.

Both site amplification and liquefaction behaviour may be included under the general category of site effects. Many recent earthquakes, in addition to Loma Prieta [1], have demonstrated graphically the results of site effects. Kobe (1995) [15], Northridge (1994) [12], and Mexico City (1985) [2], all exhibited significant ground motion amplifications. Significant liquefaction induced damage was observed during the Kobe (1995) and Edgecumbe (1987) [18] earthquakes. These recent events amongst others have prompted a renewed research interest in the field of liquefaction and site response behaviour.

Earthquake induced liquefaction is a major cause of strength loss in saturated sand deposits, with associated damage often significant. This damage can take the form of foundation bearing capacity failures, large and sometimes differential vertical settlements, lateral spreading and damage to underground services. In recent decades, a significant research effort has gone into understanding this behaviour. Nearly all of the work however has been directed towards the behaviour of quartz sands, with very little of the research effort focused on the dynamic and liquefaction behaviour of volcanically derived sands.

The active geologic past of New Zealand has lead to widespread deposits of volcanic soils throughout the country. The Taupo Volcanic Zone (TVZ) in the central region of the North Island (extending east to the Bay of Plenty) in particular has extensive deposits of volcanic ash, clays, and pumice sands. The Auckland region also has significant deposits of predominantly ash and tuff from the many small volcanic cones in the area, but few areas of significant volcanic sands.

Problems with sands of volcanic origin have been encountered in the past in some large geotechnical projects in New Zealand. The Edgecumbe earthquake of 1987 exhibited widespread liquefaction of these types of sand. Failures in hydro development projects have also occurred due to the high erodability of pumice sands. These events have highlighted the unique behaviour of some of the sands of volcanic origin in New Zealand and emphasize the need for experimental study.

This paper presents results from an experimental investigation of the liquefaction and general dynamic properties of a pumice sand taken from the Puni river in the Waikato. It is referred to herein as Puni pumice sand. Stress controlled cyclic triaxial testing of representative loose and dense samples was used to investigate the characteristics of the liquefaction response of the pumice sand. As the form of the nonlinear stress-strain relationship and low strain modulus properties of soils has a very significant effect on

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the outcome of strong motion numerical dynamic analyses, results are also presented for these dynamic properties of the Puni sand. These results were derived from bender element tests, torsion tests and the triaxial tests on the Puni sand. For all of the soil tests involved, the repeatability of the testing and sample preparation methodology was investigated.

All methods of seismic analysis depend to varying degrees on site specific soil information. In practice there is very often little laboratory or detailed site information available and so reliance is placed on a number of empirical relationships and correlations for much of the required soil data. It is therefore important to assess the validity of these commonly used empirical correlations for the dynamic behaviour of pumice sand.

REVIEW OF EXISTING DATA ON VOLCANIC SOILS

Pumice soils are characterized by a number of distinctive properties. They are generally lightweight, highly frictional, and exhibit high water absorption due to their vesicular nature. The coarser grained particles found in pumice sands are also highly degradable, compressible and erodable. These broad generalisations are a useful framework and description for these types of soils.

The experimental and empirical data in the literature on volcanic soils is very limited. Larkin et al [8] investigated extensively the engineering properties of the Puni pumice sand that is the subject of this study, and determined its physical, strength and the static stress-strain properties. Thrall [26] found significant deviations in the behaviour of an andosol ash soil when related to commonly used soil correlations.

Padilla [14] found that the pumice soils of Guadalajara, Mexico were very sensitive to vibration loading and exhibited very large and damaging settlements when subjected to railway line vibrations. Chan [3] investigated the dynamic soil properties of two volcanic ash soils sourced from the Rotorua region. This work found that the ashes exhibited a significant plateau of linear stress-strain behaviour, with a rapid fall off to strongly nonlinear behaviour at larger strain magnitudes. Pender et al [17] found that a volcanic ash from Tauranga was more sensitive to dynamic loading than an Auckland clay with similar soil classification properties.

GENERAL PROPERTIES OF PUNI SAND

The Puni sand used in this study is not a natural deposit but is derived by processing sand from the Waikato river. The pumice particles are centrifugally separated from the other river sand particles, so that the samples used for this study consist essentially of pumice grains.

A number of general soil classification tests were performed to quantify the Puni sand. These tests were to complement the work of Larkin et al [8] as the grading of the sand was slightly different from that used previously.

Particle Size Distribution

The particle size distribution of the Puni sand used for all samples reported in this study is shown in Figure 1.

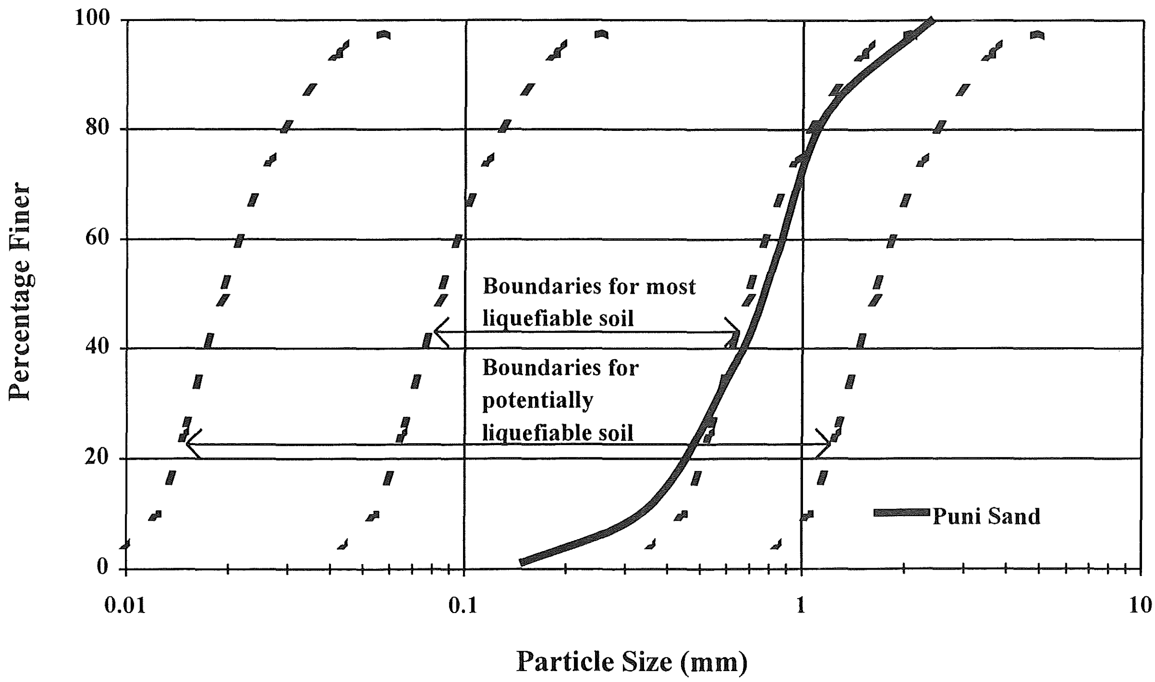

FIGURE 1 *Particle size distribution curve for Puni sand along with the zones of liquefaction susceptibility for conventional sands (from [27])*

The sand is classified as a well graded medium to coarse sand, with a small percentage of fines. The particle size ranges for sands not containing pumice that are known to be susceptible to liquefaction are also shown on Figure 1. The Puni sand falls within the readily liquefiable zone of the graph.

Density and Void Ratio

The density and void ratio properties of the sand are shown in Table 1. Larkin et al [8] found difficulty in determining the solid density of the pumice sand because of the vesicular nature of the particles. Modified experimental techniques were used to measure the appropriate "solid" density; in fact two densities are required - one for the particles when the vesicles are dry and the other when the vesicles are saturated with water. The first, termed the dry solid density ($\rho_{s\text{-}dry}$), is needed in estimating the interparticle relationships where the external vesicle spaces are of no influence, such as the relative density and void ratio calculations of both dry and saturated pumice sand samples. The second solid density value (ρ_s) is required when calculating the bulk density of saturated samples. From experimental work [8], the two values were found to be 1770 kg/m^3 for the dry solid density and 2230 kg/m^3 for the standard solid density of the pumice particles. The value of 2230 kg/m^3 is low for the solid density of a sand indicating that there are internal vesicles in the particles which water does not infiltrate on saturation.

Figure 2 shows the bulk density results determined experimentally for a range of confining pressures and initial relative densities. The bulk densities of the pumice sand samples are low in comparison to those of quartz sands and the range of measured bulk densities (1400-1560 kg/m^3) was small, demonstrating the effect of the vesicular nature of the saturated pumice particles.

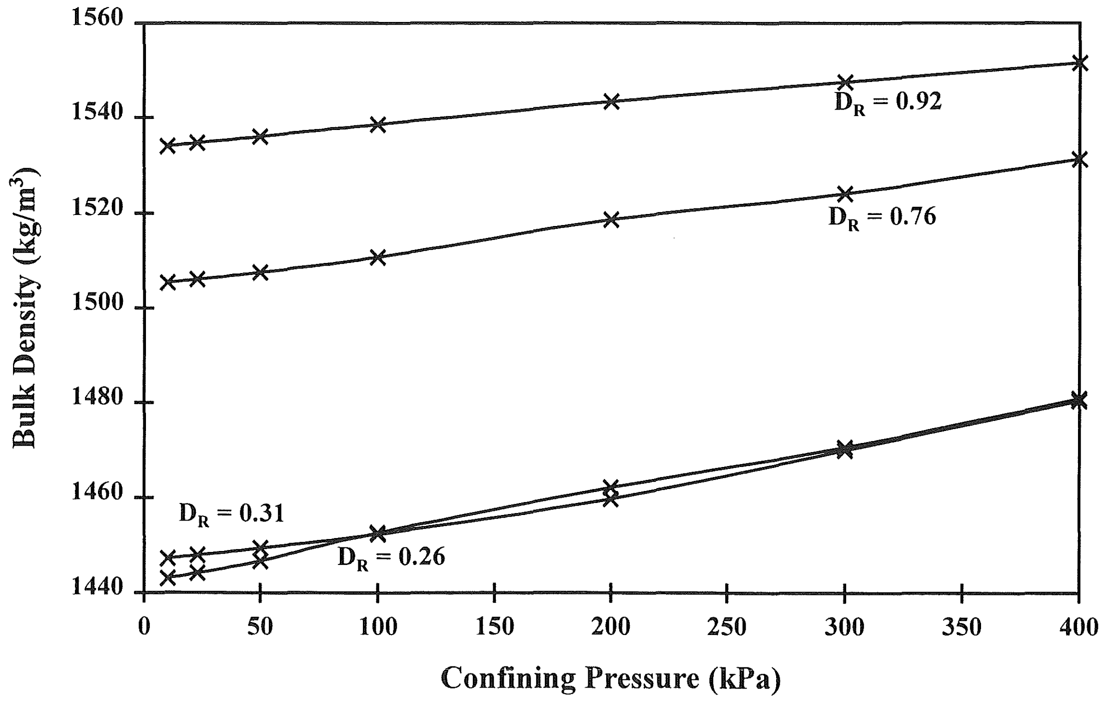

FIGURE 2 ***Experimental bulk density characteristics of the Puni sand***

The bulk density of all other prepared samples in this study was calculated from the measured initial dry density of the samples (ρ_{dry}) and solid density particles using the following expression, which was derived from the traditional geomechanics phase relationships:

$$\rho_{bulk} = \rho_{dry} + \rho_w \left(1 - \frac{\rho_{dry}}{\rho_s}\right) \quad (1)$$

where ρ_{dry} is the dry density of the test samples
ρ_w is the density of water (1000 kg/m^3)
ρ_s is the solid density of the pumice particles (2230 kg/m^3)

Permeability

Permeability can have a significant influence on the liquefaction response of a sand deposit due to pore pressure drainage and redistribution [9,10]. This is particularly relevant for pumice sands where the void ratios tend to be higher than for quartz sands. The permeability characteristics of the Puni sand was therefore investigated using a constant head permeameter test on 5 samples of varying relative density. Figure 3 summarises the permeability characteristics of the sand.

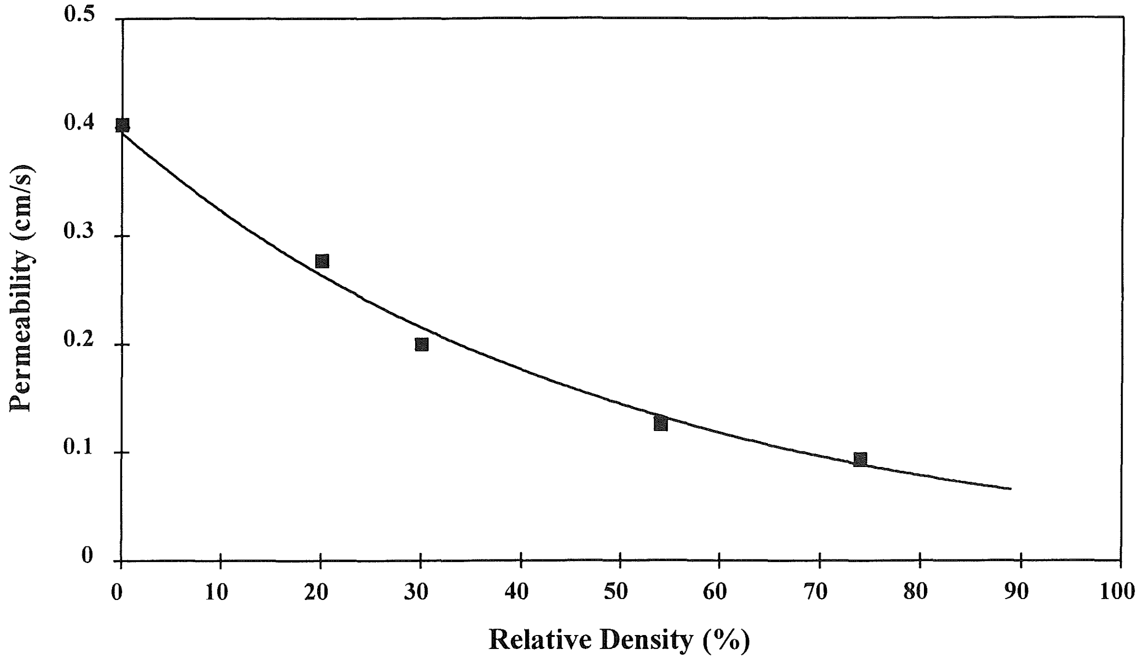

FIGURE 3 *Permeability of Puni sand*

Figure 3 also shows a best fit exponential interpolation function relating relative density to the coefficient of permeability over the range of relative densities, which is given by the expression:

$$k = 0.4e^{-0.02 D_R} \quad (2)$$

where k is the coefficient of permeability (cm/s)
D_R is the relative density (%)

The results show that the permeability of the Puni sand falls within the range of 0.08 to 0.4 cm/s, which is higher than the permeability obtained from the well-known Hazen's equation involving the value of D_{10}. For permeabilities in this range, the influence of drainage can be significant on the liquefaction response of the sand; it is therefore an important factor to consider in liquefaction analyses.

Strength Properties

Larkin et al [8] investigated extensively the strength properties of dry Puni sand samples and determined the strength and critical state parameters. Additional undrained triaxial tests were performed on dry and saturated samples for the sand investigated in this study as it was of slightly different composition than that investigated by Larkin et al. The friction angles, listed in Table 1, determined from this testing are in close agreement with those given in reference [8].

A listing of the general properties of Puni sand discussed above is given in Table 1.

TABLE 1 Summary of Puni sand properties

Puni Sand Property	Value
D_{50}	0.76 mm
D_{10}	0.32 mm
Uniformity Coefficient (D_{60}/D_{10})	2.64
Solid Density ρ_s [1]	2230 kg/m^3
Dry Solid Density $\rho_{s\text{-}dry}$ [1]	1770 kg/m^3
ρ_{dry} (maximum)	940 kg/m^3
ρ_{dry} (minimum)	745 kg/m^3
e_{min}	0.88
e_{max}	1.38
ϕ' (dense)[2]	41°
ϕ' (loose)[2]	39°
Permeability range	0.08 to 0.4 cm/s

[1] from reference [8]
[2] in close agreement with [8]

CYCLIC TRIAXIAL TEST RESULTS

Sample Preparation

The main difficulty in the laboratory testing of sands is sample preparation. The Puni sand is free flowing, which means all samples were reconstituted. For the loose test results reported in this paper, all samples were prepared by a dry pluviation method. Larkin et al [8] investigated a number of sample preparation methods, and concluded this method gave reliable and repeatable results for this sand. The loose samples had an average relative density of 33% using this method, which was repeatable for each sample to a tolerance of around 10%. Dense samples were formed using the method of vibro-compaction of the sand in the sample former, which yielded consistent dense samples.

Saturated samples were prepared in the usual manner within a former mounted on the base of the triaxial cell. On completion of the cell assembly the sample was flooded with carbon dioxide, followed by de-aired water. The samples were left overnight under a back pressure of 700 kPa, and saturation checks the next day indicated that complete saturation had been achieved using this method. All samples were then consolidated and tested at an effective stress of 100 kPa.

Testing Equipment

All stress controlled cyclic loading tests were performed under load control on an MTS closed loop electro hydraulic testing machine. An MTS function generator was used to generate a sine wave loading signal of 0.1 Hz for all reported tests.

Samples were tested in a conventional large triaxial cell, which allowed specimens of 100 mm in diameter and 200 mm in length to be tested. Specimens of this size have many advantages, the most important that they tend to be less influenced by any localised density inhomogeneities that may be present in the sample.

Load controlled undrained dynamic triaxial compression tests were initially performed on a number of loose Puni sand samples to investigate the liquefaction response of this sand. Some dense samples were also tested and these results are reported later. Initially however the reliability and repeatability of the sample preparation and testing procedure was verified.

Repeatability of Testing Procedure

The repeatability of the sample preparation and testing method is a very important factor when performing laboratory testing, and therefore two different loose samples were tested in the same manner to compare the results. The properties of the two samples (labeled D and E for convenience) are shown in Table 2.

TABLE 2 Material properties of the repeatability testing loose samples

	Sample D	Sample E
Dry Density (kg/m^3)	803.0	797.0
Bulk Density (kg/m^3)	1443.0	1440.0
Relative Density (%)	35.0	32.0

The results in Table 2 indicate that the dry pluviation method produces loose samples of similar density, which is very important in liquefaction testing [20]. Both of these samples were then subjected to a cyclic load controlled undrained triaxial test at the same cyclic stress ratio. The axial load versus axial displacement results are shown in Figures 4 and 5.

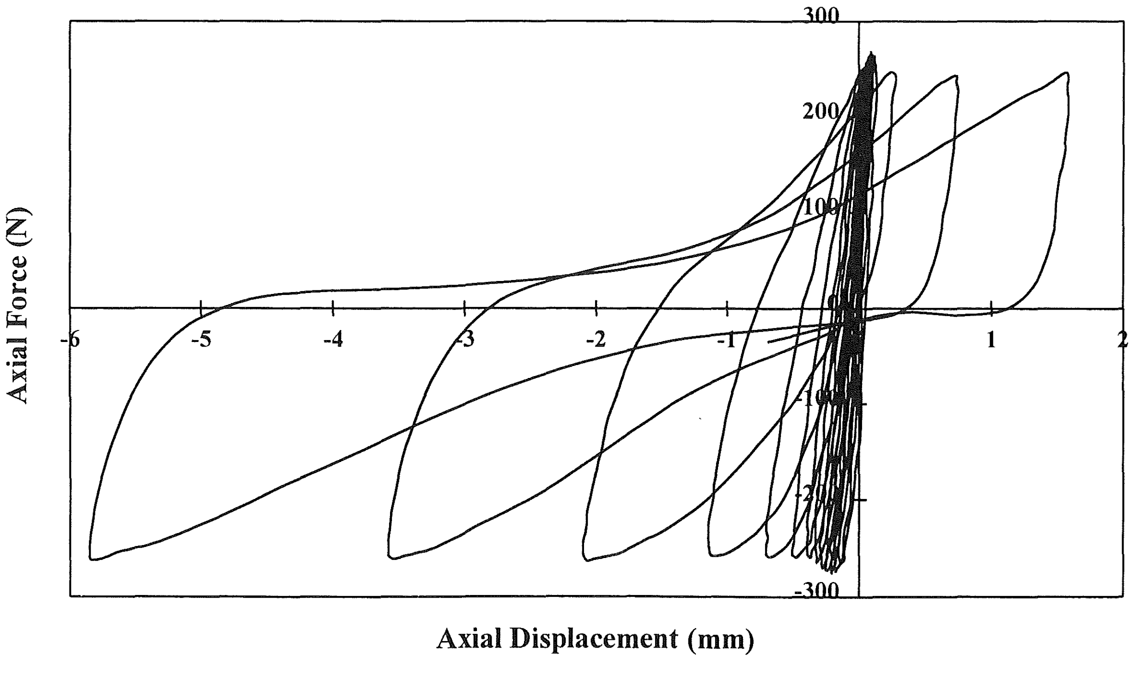

FIGURE 4 Axial load displacement plot for loose sample D (consolidation pressure, σ_3'=100 kPa)

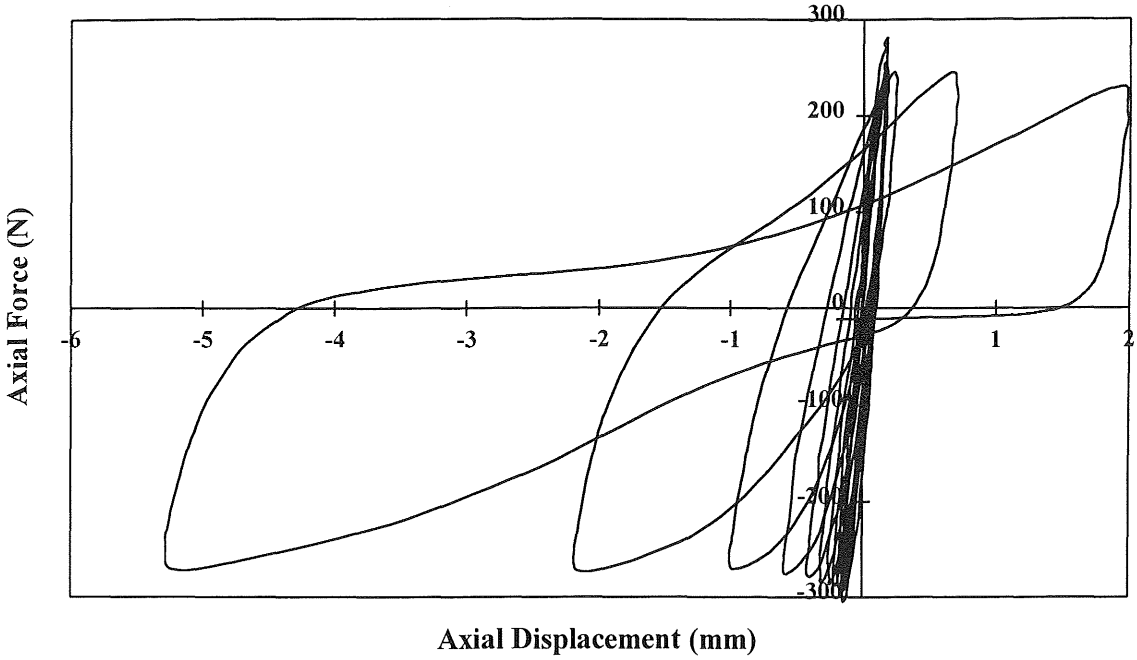

FIGURE 5 *Axial load displacement plot for loose sample E (consolidation pressure, σ_3'=100kPa)*

These two test results show a similar liquefaction response, with sample D experiencing liquefaction in 13.5 completed loading cycles, and sample E in 14.5 cycles. Both failed in the extension phase of loading, hence the extra ½ cycle.

Figures 6 and 7 show the total pore pressure response of the two samples as measured at the base mounted transducer.

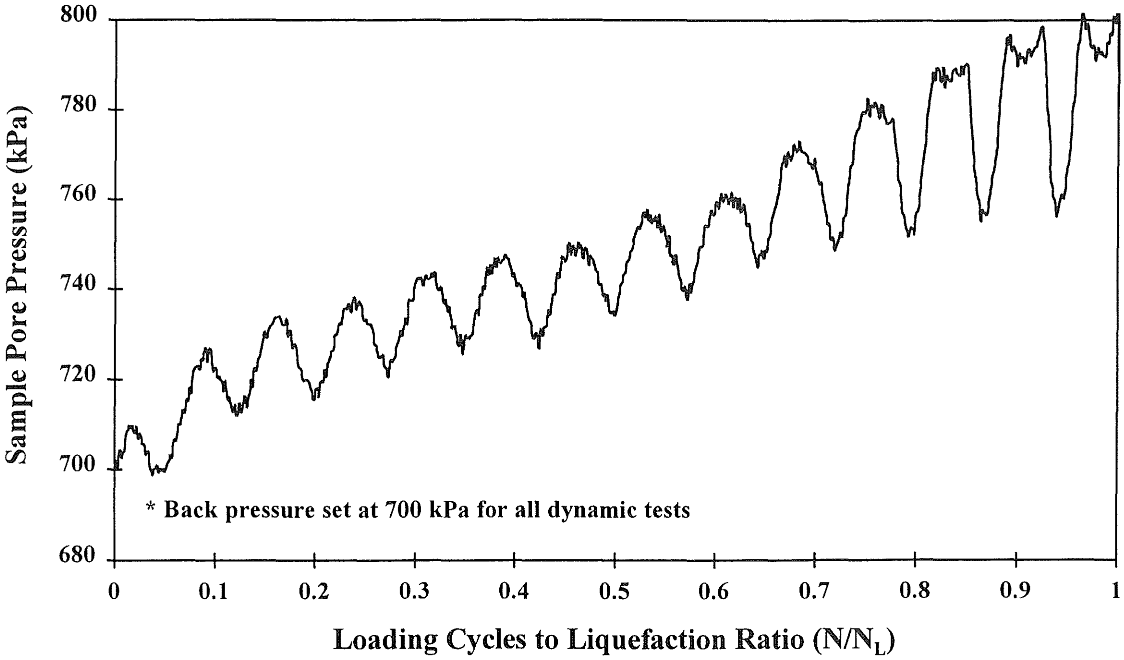

FIGURE 6 *Pore pressure response for loose sample D*

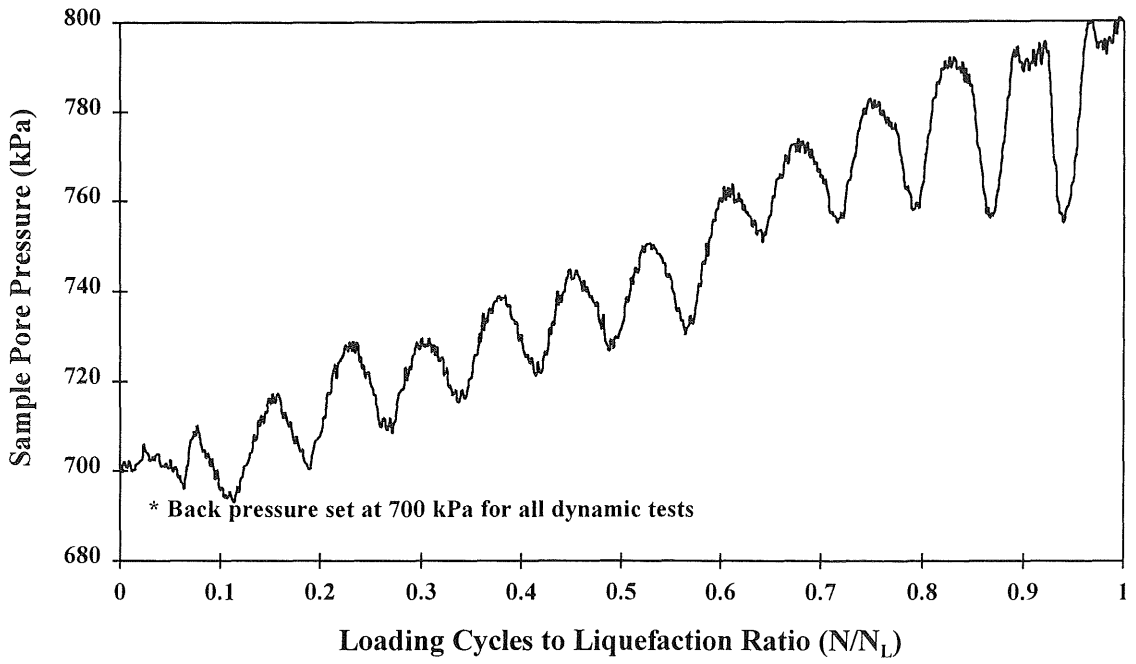

FIGURE 7 *Pore pressure response for loose sample E*

Both the load displacement and pore pressure results are very similar for the two samples tested, which gives confidence as to the repeatability of the testing procedure. This is particularly encouraging as the load controlled tests are inherently more difficult to duplicate than the displacement (strain) controlled test.

Liquefaction Test Results

A number of load controlled tests at a variety of stress ratios were performed to generate a liquefaction strength curve for the loose Puni sand. Table 3 summarises this test data.

TABLE 3 Summary of applied stress ratios for cyclic triaxial tests

Sample	Relative Density (%)	Stress Ratio ($\sigma_d/2\sigma_3'$)
A	38.0	0.23
B	32.0	0.20
C	33.0	0.18
D	35.0	0.16
E	32.0	0.16
F	33.0	0.15
G	29.0	0.14

It is clear from the above table that the densities of the samples are reasonably similar and therefore the liquefaction properties of the loose sand are able to be directly compared at different stress ratios. Figure 8 plots the liquefaction resistance curve determined from the loose samples.

Figure 8 also shows the liquefaction curve derived from large scale shake table testing by DeAlba et al [4] that is often used to generate the shape of a liquefaction curve in the widely used empirical method of Seed and Idriss [21]. This data shows that the shape of the Puni sand curve is slightly flatter than that in [4], particularly for the low stress ratio region where the number of cycles to liquefaction are large.

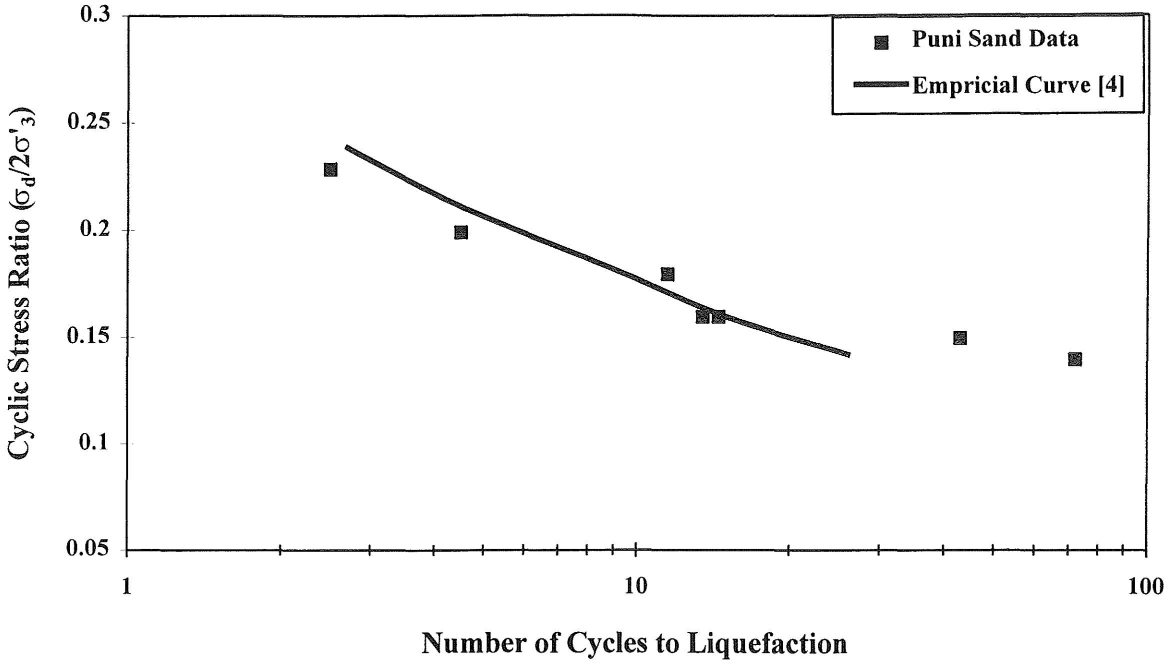

FIGURE 8 *Loose Puni sand liquefaction curve and empirical shape*

The normalised increments in pore pressure for all of the loose tests are shown in Figure 9. These originally took the form of those shown in Figures 6 and 7, but the in-phase dynamic component has been removed. This leaves the steadily increasing residual pore pressure component that is a result of the induced shear and potential volumetric strains. The residual pore pressure curves show similar behaviour for most of the samples. Figure 9 also shows that the Puni sand data falls within the upper and lower bounds of the cyclic triaxial residual pore pressure curves of Seed et al. [23]. Thus the pore pressure response of the pumice sand is similar to that of other sands and so established numerical models, [9,13,23], can be used in liquefaction analysis of these sands.

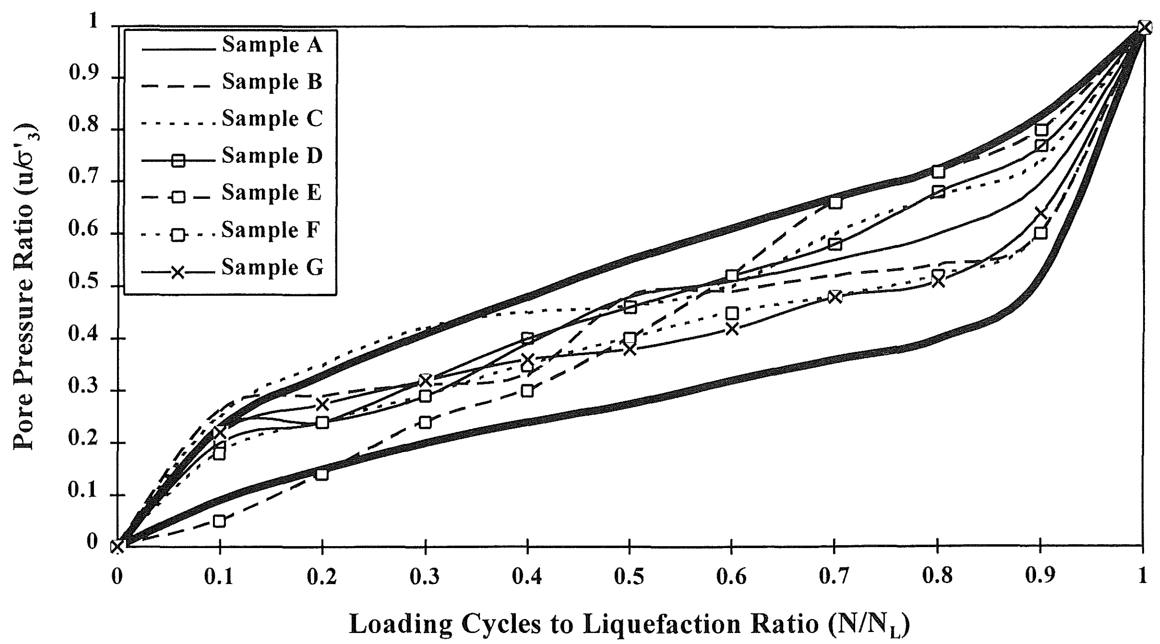

FIGURE 9 *Normalised pore pressure response of all samples, also showing boundaries of Seed et al. [22]*

Influence of Loading History on Liquefaction Response

A loose saturated sample (specimen H) was subjected to an initially small cyclic stress ratio ($\sigma_d/2\sigma_3'$=0.10) for 50 loading cycles and then reconsolidated. The reconsolidation produced a small volume decrease in the sample of 0.3%, indicating very little excess pore pressure had developed. When retested at a stress ratio of 0.16, which is the same as samples D and E, the number of loading cycles to liquefaction increased from the average value of 14 to 23.5 cycles. This is a significant increase in the liquefaction resistance properties of the sample and indicates the loading history may be a significant factor in the cyclic triaxial liquefaction properties of Puni sand. Similar behaviour has been observed by other researchers for quartz sands [19]. It is difficult to extend this observation to the in situ behaviour of the sands as field experience has shown that sand deposits still liquefy even after a history of previous cyclic motions. It does however indicate that stage testing of samples is not a viable option in cyclic triaxial testing if reasonable results are to be achieved.

Cyclic Testing of Dense Samples

Liquefaction tests on a number of dense Puni sand samples were undertaken to investigate the liquefaction response of the dense form of the sand and possible evidence of grain crushing. Traditionally, dense sands are not susceptible to damaging liquefaction effects as the potential for volumetric compaction under cyclic loading is low. The dynamic response of one of the dense samples (specimen I, D_R≈80 %) is shown in Figure 10. This sample was tested at a high value of the cyclic stress ratio ($\sigma_d/2\sigma_3'$) =0.32. Several trials of other dense samples at loading magnitudes similar to the loose samples showed very little response, which necessitated the higher stress ratio. The test was stopped after 35 cycles when it was clear no failure was imminent.

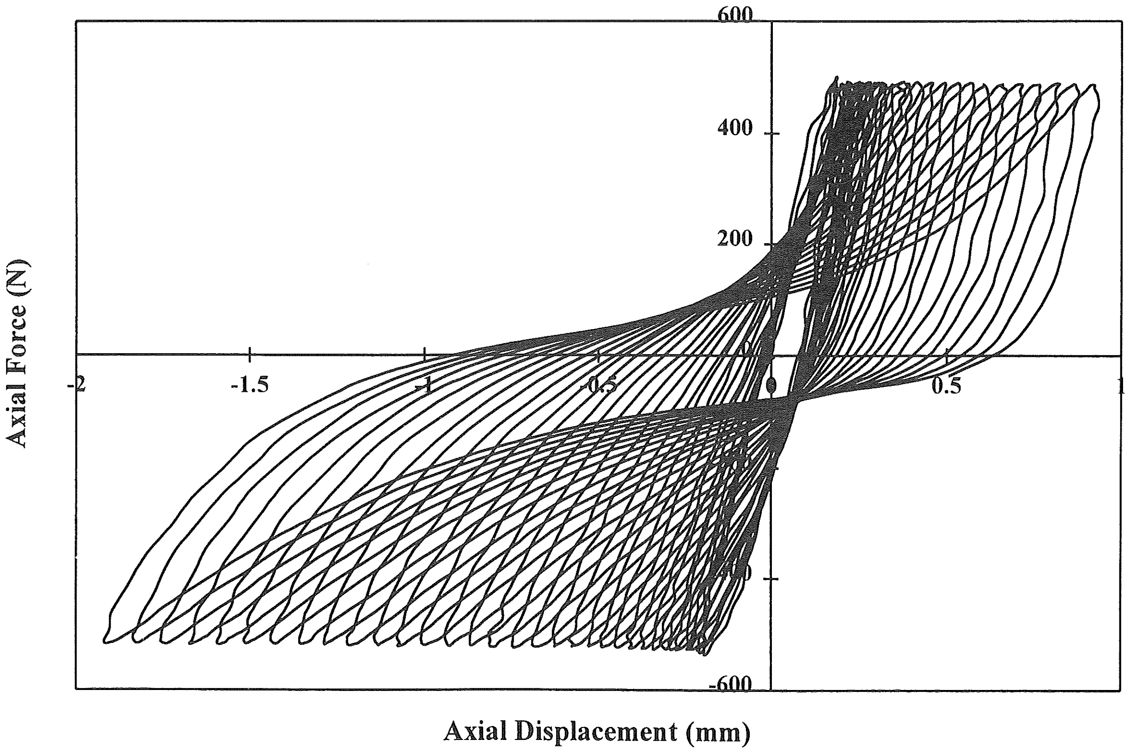

FIGURE 10 Axial load displacement response for dense sample I

This plot illustrates clearly the limited strain potential exhibited by dense sands, rather than the sudden failure of the loose samples. This behaviour can be explained with the aid of Figure 11 which shows the excess pore pressure response of the sample.

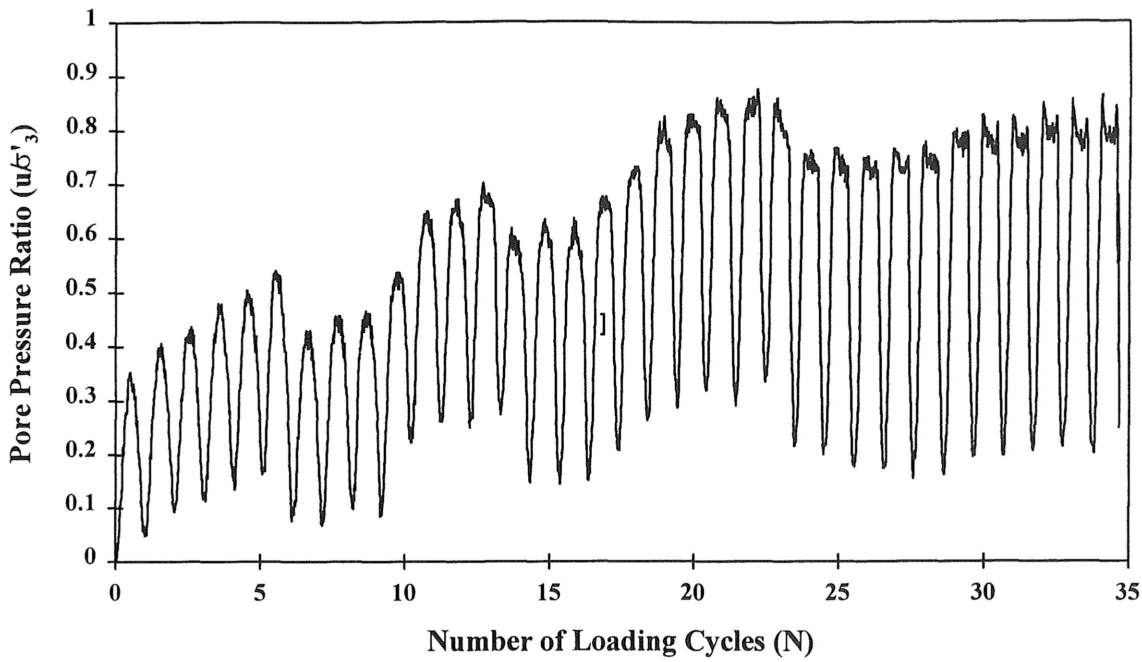

FIGURE 11 ***Dense sand normalised pore pressure response***

This figure shows clearly the characteristic liquefaction response of a dense sand. The mean pore pressure ratio doesn't exceed about 50%, due to the low potential volumetric strains induced in dense samples. Each loading cycle also illustrates the strongly dilatant behaviour of the sand to shear loading, which significantly retards the cyclic excess pore pressure response. Both these factors retard the magnitude of the induced strains, leading to the term limited strain potential. There is therefore no sudden failure as in the loose samples because the higher pore pressures only develop for short phases during each loading cycle.

Grain Degradation

As Larkin et al [8] found significant crushing of the soft pumice grains during their triaxial testing programme this is an potentially an important factor in the dynamic response of Puni sand. The applied deviator stresses during the cyclic triaxial testing programme were only in the order of 20 to 50 kPa, which was significantly lower than the 500 kPa to 2000 kPa range of Larkin et al, but the cyclic nature of the deformation was an added factor.

A number of the loose and dense sand samples tested were subjected to a particle size analysis after the liquefaction testing. No detectable grain crushing was evident in any of the samples. This prompted additional cyclic testing of two dense samples, which were subjected to dynamic loads well in excess of those that would be expected during a large earthquake, and again no grain crushing or degradation was observed. It may be concluded therefore that the stresses involved in liquefaction loading are not sufficient to promote significant grain degradation. Without significant particle degradation, the pumice sand tends to exhibit similar liquefaction behaviour to quartz sands at similar relative densities.

Analysis Of Liquefaction Behaviour

A common method for assessing the liquefaction potential of a site is that of Seed and Idriss [21]. The basis of their approach is to determine the liquefaction potential based on the particular penetration resistance profile of a site. From evidence of liquefaction from past earthquakes, Seed and Idriss [21] and Robertson and Campanella [19] developed empirical design charts which correlate SPT and CPT penetration resistance respectively to the simple shear stress ratio required to induce liquefaction in a sand subjected to 15 cycles of loading (which is considered equivalent to a magnitude 7.5 earthquake). Using other empirical data, a liquefaction resistance curve may be generated for the sand (as outlined above) to then determine the liquefaction resistance for other magnitudes of earthquake loading.

Typically the induced shear stresses in a site from the seismic loading are then evaluated, using either simple analytical or more complex numerical methods, and a factor of safety against liquefaction is determined. More complex coupled numerical methods exist to evaluate the liquefaction potential of sites, but they all require some method of determining the required liquefaction parameters. Often empirical relationships between liquefaction potential and penetration resistance are the only economic method of determining the required parameters [7].

The basis of the Seed and Idriss method is the relationship between penetration resistance and the susceptibility of a sand to liquefaction. It is therefore interesting to investigate this relationship using the experimental liquefaction results gained during this work and the CPT penetration resistance recently investigated for the Puni sand in large chamber testing at the University of Auckland [28]. The empirical liquefaction resistance versus CPT penetration curves published by Robertson and Campanella [19] are shown in Figure 12, with the cyclic stress ratio converted from the simple shear to the equivalent cyclic triaxial stress ratio. In New Zealand we are reliant on cyclic triaxial test data and must use empirical factors to convert between the two different cyclic shear stress ratios. The general form of the correlation between the two is

$$\frac{\tau}{\sigma'_{vo}} = C_r \frac{\sigma_d}{2\sigma'_3} \qquad (3)$$

where C_r is the correlation factor

A number of expressions for the correlation factor C_r exist in the literature derived from experimental testing, with most investigators finding it to be in the range of 0.6 to 0.7 for quartz sands. A value of 0.63 for this factor [20] was used to convert the original liquefaction resistance chart [19] to an equivalent cyclic triaxial stress ratio versus CPT penetration resistance chart.

Figure 12 also shows the position of the Puni sand result on this chart, with the triaxial stress ratio at 15 cycles determined from Figure 8 and the CPT penetration resistance for the Puni sand taken from the results of Wesley et al. [28]. The value of cone resistance was determined from large scale chamber testing of the Puni sand at a similar relative density to the loose cyclic triaxial results and at a confining pressure of 100 kPa.

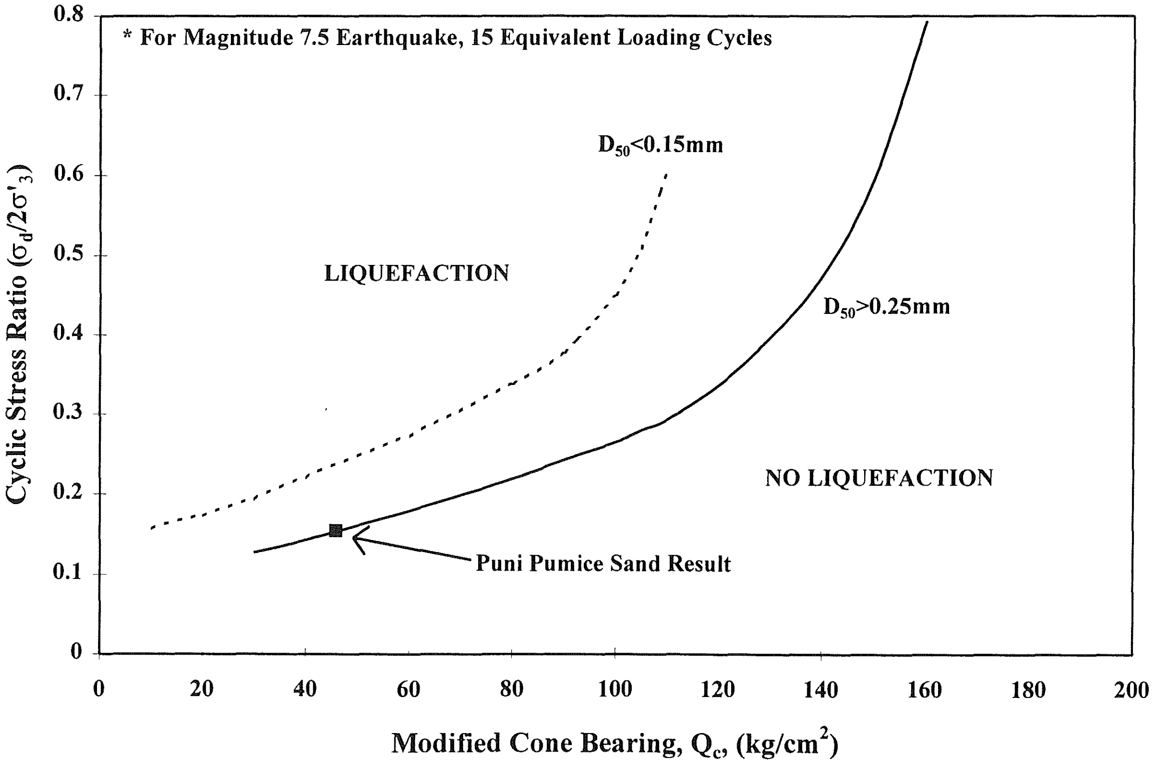

Figure 12 *Empirical CPT versus cyclic triaxial stress ratio relationship for liquefaction resistance [19]*

Figure 12 shows that the experimental result for the Puni sand plots on the curve for medium to coarse sands. This does not suggest that the Puni sand exhibits the same relationship between liquefaction resistance and penetration resistance as quartz sands because the empirical curves are a lower bound estimate. This result indicates that using the empirical method to estimate the liquefaction resistance of a Puni sand deposit would give no factor of safety in the analysis.

LOW STRAIN SHEAR MODULUS PROPERTIES

An accurate assessment of the relationship between low strain shear modulus and confining pressure is very important in seismic site analyses. A number of Puni sand samples were tested in the free vibration torsion test equipped with bender elements to determine the low strain shear modulus and nonlinear behaviour of the sand. These results are reported elsewhere [11], but some of the more interesting behaviour is presented and extended here.

Initially the relationship between low strain shear modulus and confining pressure was investigated for a range of samples. Figures 13 and 14 show the experimentally determined confining pressure versus G_{max} curves for the loose and dense samples respectively, with lines of approximate best fit are also shown.

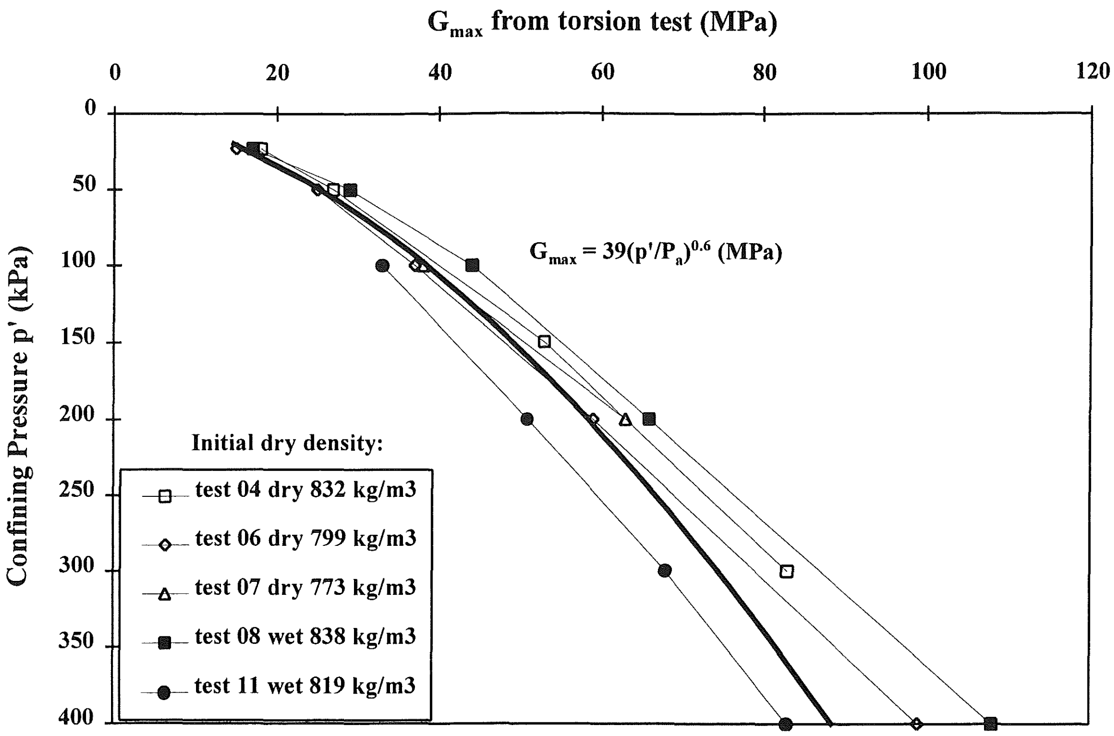

Figure 13 *Shear modulus versus confining pressure for loose samples (from [11])*

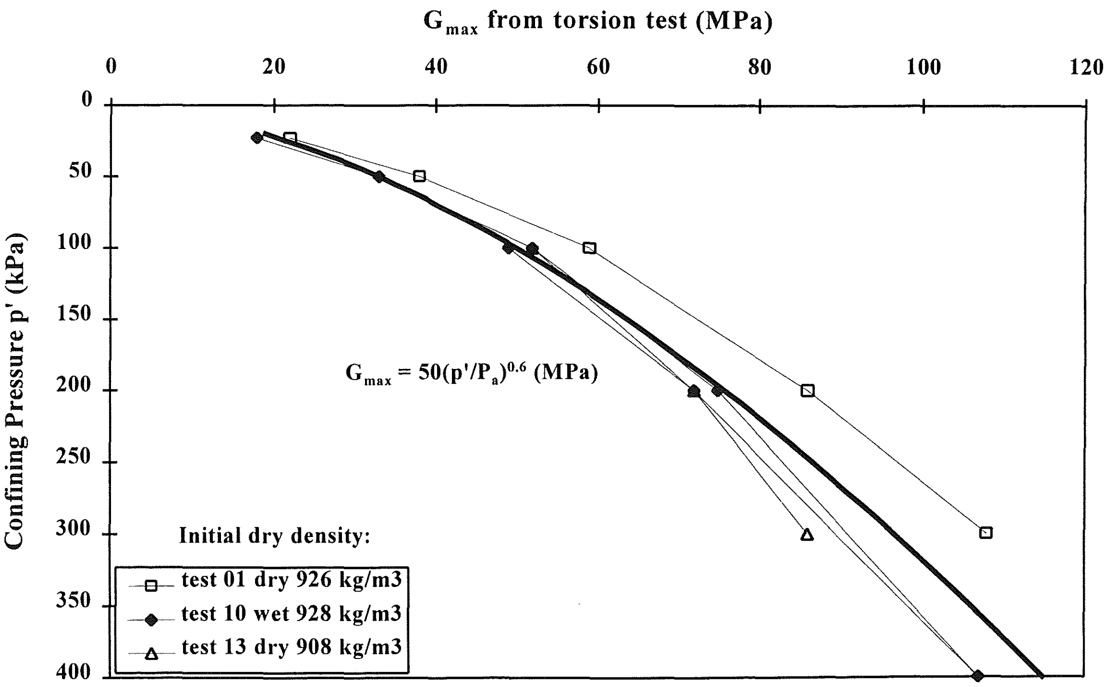

Figure 14 *Shear modulus versus confining pressure for dense samples (from [11])*

The best fit expressions shown on the above figures have the same power term for both the loose and dense state, with the difference being confined to the multiplying constant. The multiplying constant is clearly a function of relative density, and the data for both the loose and dense state may be replotted in the form of Figure 15.

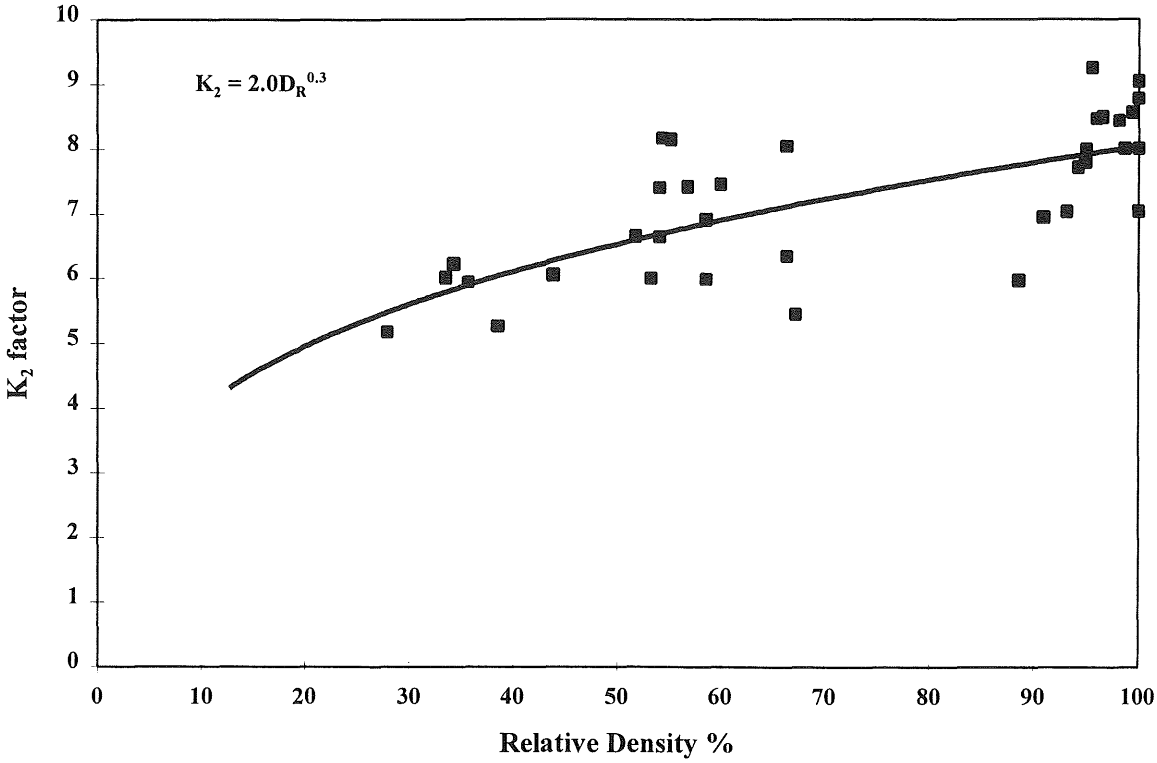

Figure 15 ***K_2 factor for Gmax versus density from experimental data***

The most commonly used expression relating confining pressure and low strain shear modulus is from Seed et al [24], which takes the form (in S.I. units)

$$G_{\max} = 2.2\,K_2\left(\frac{p'}{P_a}\right)^{0.5} \qquad (4)$$

where G_{max} is in MPa
p' is the effective confining pressure
P_a is atmospheric pressure in the same units as p'
K_2 is a material constant

Typically K_2 can be related to relative density or SPT blow count. For the Puni sand however, the power term is 0.6, and the K_2 term is derived from the best fit line of Figure 15. The full expression for the low strain modulus (D_R>10%) of this sand is therefore

$$G_{\max} = 14(D_R)^{0.3}\left(\frac{p'}{P_a}\right)^{0.6} \qquad (5)$$

where G_{max} is in MPa
D_R is the relative density in % (D_R>10%)

In comparison to the expression derived for quartz sands [24], the pumice sand exhibits significantly lower values of G_{max} for all relative densities, ranging from 30% to 60% of the values of G_{max} for quartz sands. Ideally the low strain shear modulus should be determined in situ by recording shear wave velocities, but this data is often not available and the practitioner must rely on correlations with penetration resistance or relative density. The above result emphasises that it is important to use appropriate correlations for volcanic soils where possible.

NONLINEAR CONSTITUTIVE PROPERTIES

The constitutive properties of soils are very significant in seismic response computations [6] and requires close consideration. The dynamic torsion test allows the nonlinear constitutive properties to be experimentally determined from the period of the decaying torsional free vibration response of a sample at different strain amplitudes. The available dynamic torsion test data on the form of the nonlinear shear strain versus shear modulus relationship for the Puni sand presented in a previous paper [11] only extended to shear strains in the region of 10^{-1}%. Shear strains of this magnitude are likely to be induced in moderate to large earthquakes, with very large earthquakes likely to induce larger strains in soft ground. Therefore the form of the constitutive relationship of the Puni sand at large strains was determined from the cyclic triaxial test results, which are plotted for shear strains up to 1%.

The large strain results determined from the triaxial test were plotted on the existing data from the torsion tests [11], which is shown in Figure 16. This plot shows the scatter and trends in the shear modulus - strain amplitude behaviour of the sand. The normalised shear modulus value used as the y axis was determined by dividing the value of the nonlinear shear modulus obtained from the torsion and triaxial test results at various shear strain magnitudes by the value of G_{max} determined from equation 5 for the various Puni sand samples.

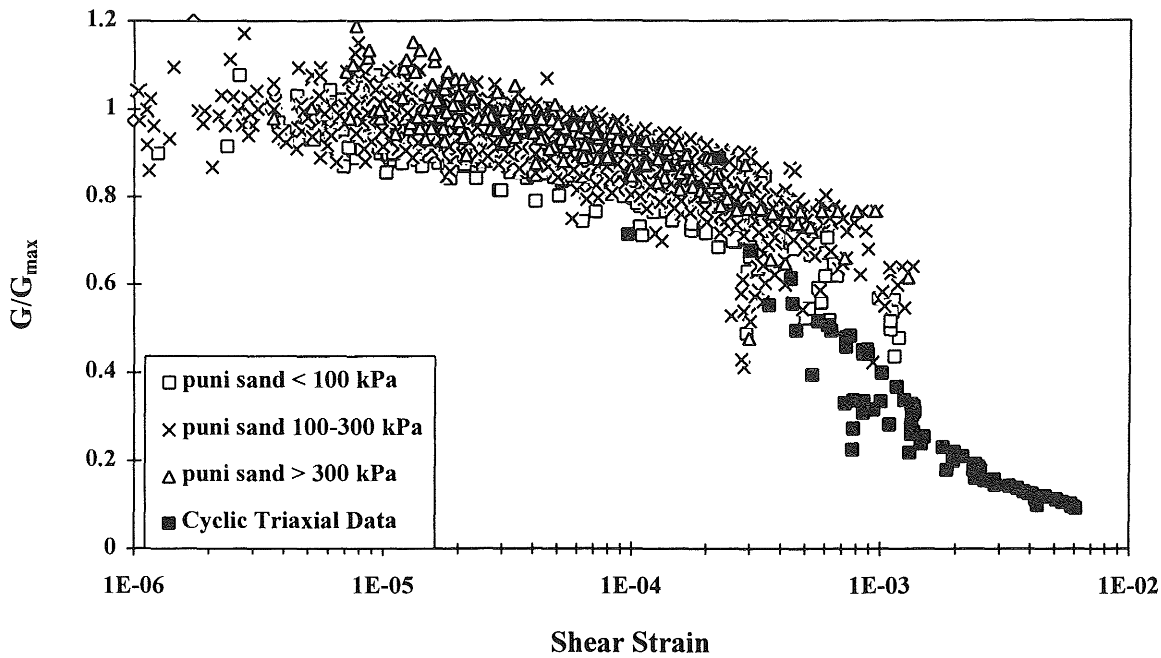

Figure 16 *Normalised shear modulus versus shear strain response from all tests*

A best fit line for the Puni sand from the data of in Figure 16 along with the well known shear modulus strain amplitude curves for sands and clays [24,25] is shown in Figure 17. Only one general best fit line was drawn, even thought there is evidence from these results that the confining pressure had some influence on the form of the relationship. Puni sand plots significantly above the empirical sand curves in the medium strain range (torsion test results), and then shows a rapid decay of the curve in the larger strain zone (cyclic triaxial test results). There is the difficulty here of comparing results from two different testing methods, but there appears reasonable agreement where the two sets of results overlap (Figure 16). These results indicate that the pumice sand exhibits more linear, less damped behaviour than quartz sands over the medium to low strain range, and tend to behave in a similar manner to quartz sands in the larger strain range. From Figure 17 it can be seen that the constitutive behaviour of the Puni sand is similar to a clay of relatively low plasticity.

Torsion test results from volcanic ashes [3] exhibited an extended shear modulus plateau, with a rapid decay after this plateau. The Puni sand shows similar behaviour, but the plateau appears to extend to a higher strain range. Based on this limited data it is not possible to conclude whether this large plateau and rapid decay behaviour is indicative of volcanically derived soils as a whole, but a trend does appear to be emerging. More work is clearly required to conclude a firm trend however.

Figure 18 shows a typical test result for the variation in the damping ratio with stain amplitude. Clearly this data exhibits much more scatter than shear modulus data, a not uncommon finding for damping curves.

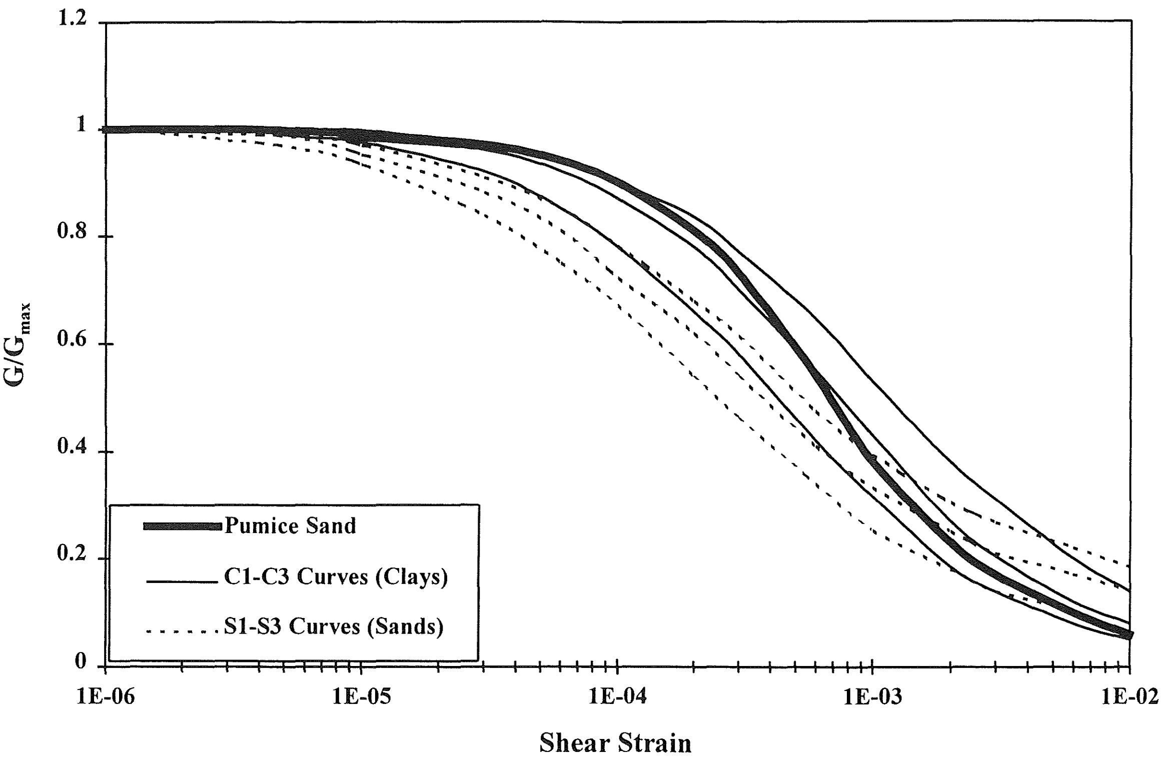

Figure 17 *Nonlinear behaviour of the Puni sand compared to literature data [24,25]*

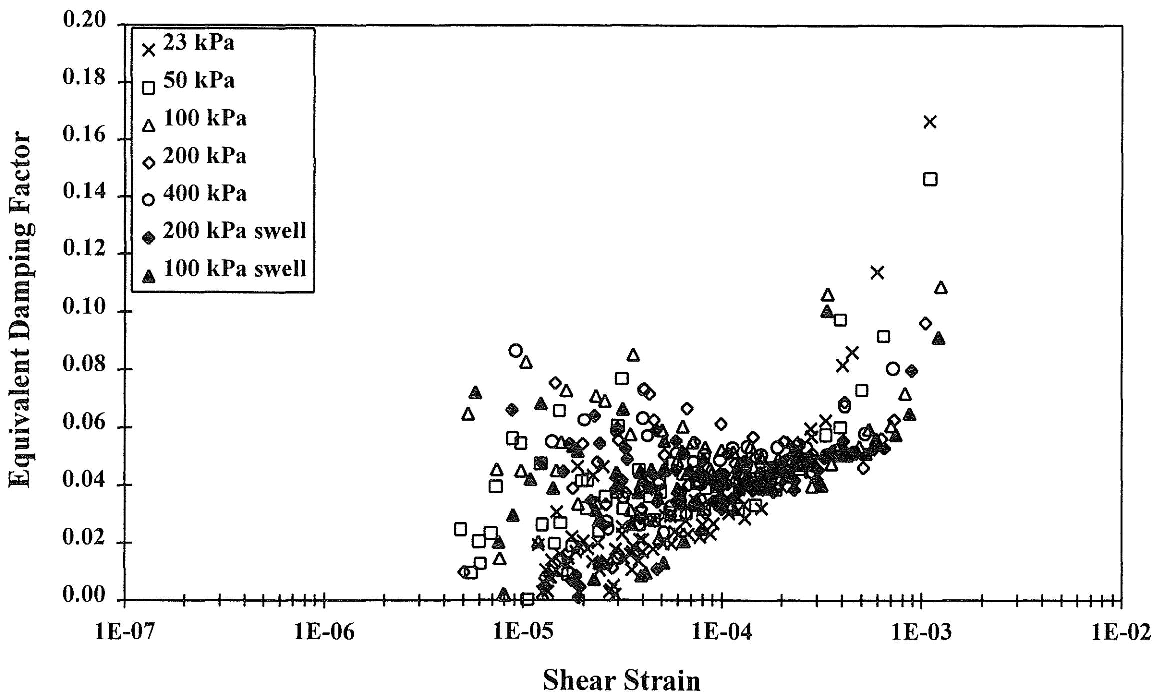

Figure 18 *Typical equivalent damping ratio versus shear strain for Puni sand sample*

CONCLUSIONS

The aim of this study was to investigate the seismic properties of a New Zealand pumice sand, such as are found extensively in the upper and central North Island. As the literature on dynamic soil properties is dominated by overseas data derived primarily from quartz sands, it was important to investigate the dynamic behaviour of a pumice sand. Most seismic design analyses rely to varying degrees on empirical soil relationships, and these therefore must be investigated. An extensive testing programme was undertaken to characterise the dynamic behaviour of the pumice sand, including cyclic triaxial liquefaction tests, free vibration torsion tests, bender element tests and a number of other general soil classification tests.

The repeatability of the dynamic triaxial test and sample preparation techniques was established, indicating that the method of dry pluviation sample construction gave consistent samples. A liquefaction testing programme on Puni sand samples was then undertaken to generate a liquefaction strength curve for the sand at a range of stress ratios. This curve was found to be of a similar shape to that of quartz sands, as were the excess pore pressure generation curves. Dense samples clearly illustrated the expected behaviour of limited strain potential.

No significant grain crushing was detected in any of the samples tested under dynamic conditions, even when the magnitudes of applied loading were significantly in excess of those expected during an earthquake. This finding indicates that one of the characteristic properties of pumice sands can perhaps be disregarded when considering its liquefaction response. This may explain the relatively classical liquefaction behaviour of this sand.

The low strain shear modulus response of a sand is a very important property. A number of free vibration torsion tests and bender element tests were incorporated in this study. These results showed that the shear modulus was proportional to the effective confining pressure raised to the power 0.6, which is significantly higher than the frequently used 0.5 power term. For similar relative densities, the low strain shear modulus of the pumice sand was found to be significantly lower than that expected of quartz sands. A specific expression was derived for the low strain modulus behaviour of the Puni sand.

The form of the nonlinear constitutive relationship is very important in numerical seismic analyses, and the free vibration test and cyclic triaxial tests were used in this regard. These results showed that the Puni sand exhibits more linear behaviour over the medium strain range that the literature suggests, with a rapid fall off to strongly nonlinear behaviour at larger shear strains. A previous study on volcanic ash showed similar trends. Based on the limited available data a trend does appear to be emerging, but clearly more work is required in this area if firmer conclusions are to be drawn.

The largest unknown feature of the behaviour of pumice sand is its response to penetration testing. As most dynamic analyses tend to rely on either SPT or CPT in situ penetration results this is a very important area for further research. At present the University of Auckland is involved in a research project on the penetration resistance of pumice sands, which may help to reduce some of the uncertainty in the seismic analyses of these types of soils in the future.

ACKNOWLEDGMENTS

The financial assistance of the Earthquake Commission and the NZNSEE (Postgraduate Scholarship) is gratefully acknowledged. The authors also wish to thank Mr Graham Duske, Dr Vaughan Meyer and Mr Jien Ma for their contributions to various aspects of the experimental work.

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