The Seismic Properties of a New Zealand Sand

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Summary: The seismic site response analysis of sand deposits requires an understanding of the dynamic properties of the soils involved. Most of the dynamic soil data currently available in the literature has not been derived for New Zealand sands, and the relevance of this data to New Zealand pumice sands is rather unclear. An extensive experimental investigation of the dynamic response of a pumice sand was therefore undertaken, which investigated the liquefaction response from cyclic triaxial tests and the shear modulus response from bender element and dynamic torsion tests. The liquefaction results indicated that the liquefaction response was similar to that observed in quartz sands. The low strain shear modulus of the pumice sand was found to be significantly lower than that of quartz sands at similar relative densities, and the non linear constitutive relationship was markedly different from other sands, particularly in the mid strain range.

1. INTRODUCTION

A recent study by Holzer [3] 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 (US $ 4.1 billion) 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, have graphically demonstrated the results of site effects. Kobe (1995), Northridge (1994), and Mexico City (1985) all exhibited significant ground motion amplifications. Significant liquefaction induced damage was observed during the Kobe (1995) and Edgecumbe (1987) 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 tremendous 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 projects in the hydro development area 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 emphasized the need for further experimental study in this area.

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, known as Puni pumice sand. Stress controlled cyclic triaxial testing of representative loose and dense samples was used to investigate the liquefaction response of the pumice sand. The repeatability of the liquefaction testing and sample preparation methods was investigated. The non linear constitutive response of soils is very important in strong motion numerical dynamic analyses, and results are presented for the constitutive behaviour of the Puni sand. These results were derived from bender element tests, torsion tests and the triaxial tests.

All methods of seismic analyses depend to varying degrees on site specific soil information. In practice there is very often little laboratory or detailed site data available which leads to a reliance 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 local soils, and these results are also presented for the pumice sand.

2. GENERAL PROPERTIES OF THE PUMICE SAND

2.1 Summary of General Sand Properties

A summary of the general properties of the Puni sand discussed is shown in Table 1. These were determined from a combination of laboratory testing by the authors and the results of a previous study at Auckland University on the same sand [5].

Table 1. Summary of general Puni sand properties

Puni Sand Property	Value
D_{50}	0.76 mm
Uniformity Coefficient (D_{60}/D_{10})	2.64
Apparent solid density	2230 kg/m^3
Appropriate solid density, ρ_s [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 [5]
[2] in close agreement with [5]

The main factor to note from the above table is the two different values of solid density given. Larkin et al. [5] found that due to the vesicular nature of pumice particles, two values of solid density were required to be determined. The first, which may be termed the apparent solid density, was determined in the standard manner. It was found however that water was infiltrating the solid particles and this was giving an overly high result. For void ratio considerations, only the void areas surrounding the solid particles are relevant and so an "appropriate" solid density had to be determined. A different test procedure therefore had to be used by Larkin et al. to determine the appropriate solid density which didn't allow time for the water to infiltrate the solid particles.

In conclusion two different solid density values had to be used in this study for the calculations required. For void ratio and relative density calculations, ρ_s=1770 kg/m^3 was used. The bulk densities of all samples tested were calculated from ρ_s=2230 kg/m^3 where the absorption of water by the pumice particles had to be accounted for.

2.2 Particle Size Distribution

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

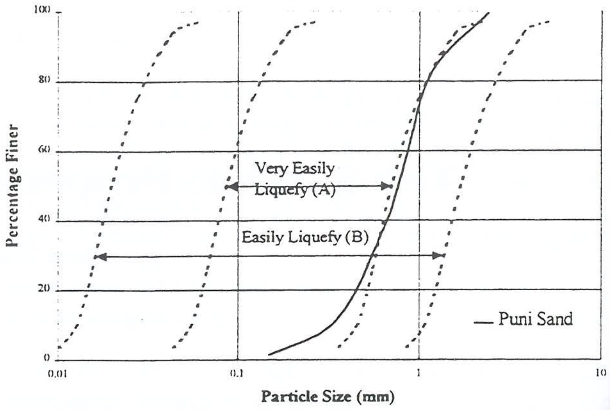

Figure 1 Particle size distribution curve of Puni sand showing the zones of liquefaction susceptibility (from [13])

The sand is classified as a well graded medium to coarse sand, with a small percentage of fines. The particle size ranges for sands that have been shown to be susceptible to liquefaction are also shown on Figure 1. The Puni sand falls within the readily liquefied zone of the graph and is hence a good choice for dynamic liquefaction testing.

3. LIQUEFACTION TEST RESULTS

3.1 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 constructed. For the loose test results reported in this paper, all samples were prepared by a dry pluviation method. Larkin et al. [5] 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 placed in the triaxial cell, flooded with carbon dioxide (CO_2) and de-aired water and left overnight. Saturation tests the next day indicated complete saturation of all samples had been achieved using this method.

3.2 Repeatability of Liquefaction Test Procedure

The repeatability of the sample preparation and testing method is a very important factor to consider 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 [9]. Both of these samples were then subjected to the same cyclic load controlled liquefaction test, the results of which are shown in Figures 2 and 3.

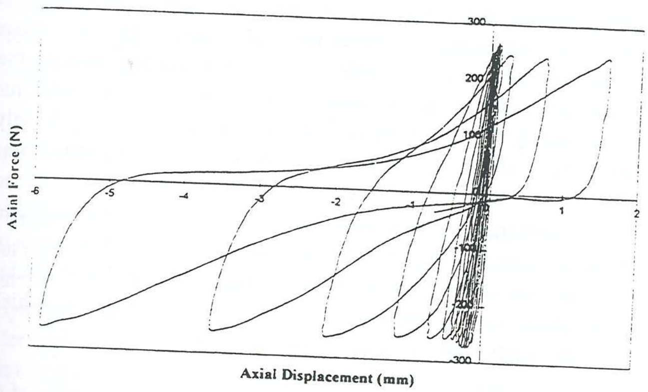

Figure 2 Axial load displacement plot for loose sample D (σ_3'=100kPa)

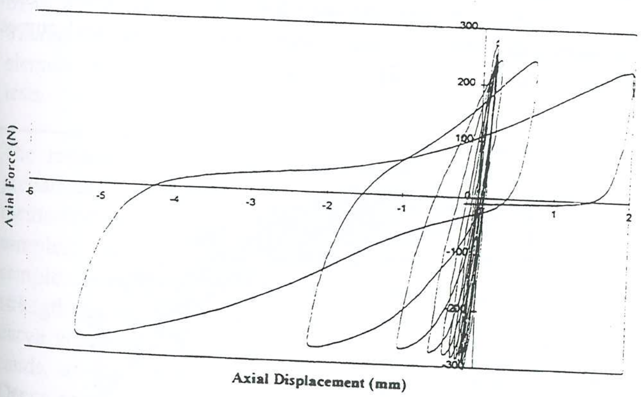

Figure 3 Axial load displacement plot for loose sample E (σ_3'=100kPa)

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.

3.3 Loose Sample Liquefaction 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. Figure 4 plots the liquefaction resistance curve of the loose samples.

Figure 4 also shows the liquefaction curve given by DeAlba et al. [2] that is often used to generate the shape of a liquefaction curve in the widely used empirical method of Seed and Idriss [10]. This data shows that the shape of the Puni sand curve is slightly flatter than the overseas data, particularly for the low stress ratio region where the number of cycles to liquefaction are large.

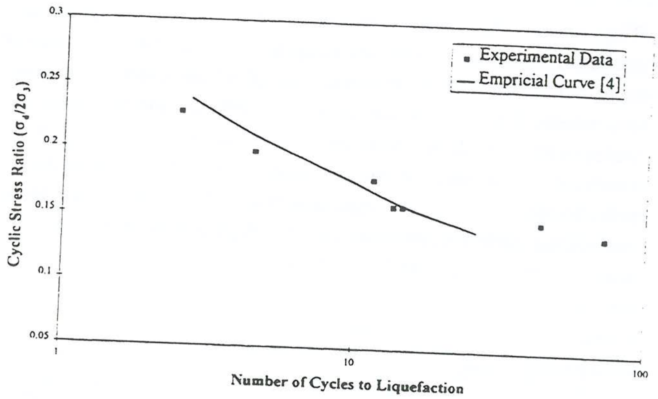

Figure 4 Loose Puni sand liquefaction curve and empirical shape

The normalised increment in pore pressure for all of the loose tests are shown in Figure 5. This shows 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 very similar behaviour for most of the samples. Figure 5 also shows the upper and lower bounds of the residual pore pressure curve collected by Seed et al. [11] for cyclic triaxial tests, and the Puni sand falls well within these boundaries. This indicates that the pore pressure response of the pumice sand is similar to other sands. This is important in the use of some numerical models in liquefaction analyses [6,8,11].

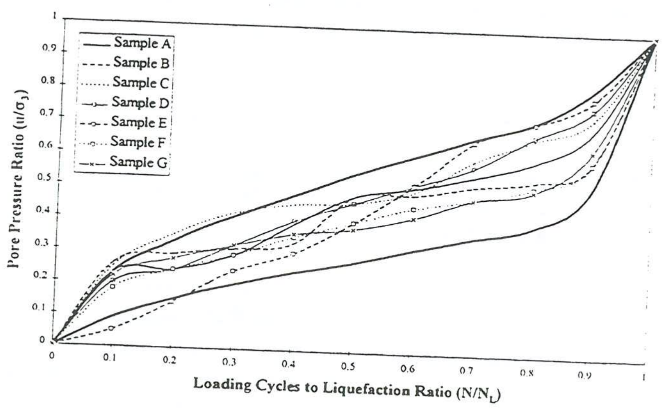

Figure 5 Pore pressure response of all samples

4. 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 non linear behaviour of the sand. These results are reported elsewhere [7], but some of the more interesting behaviour is presented and extended here.

The relationship between low strain shear modulus and confining pressure was investigated. The best fit power expressions derived from the test results (not shown here) had essentially the same power term for both the loose and dense state, with the difference being confined to a multiplying constant. The multiplying constant is clearly a function of relative density, and the experimental data for both the loose and dense state (represented by an expression known as K_2) may be replotted in the form of Figure 6.

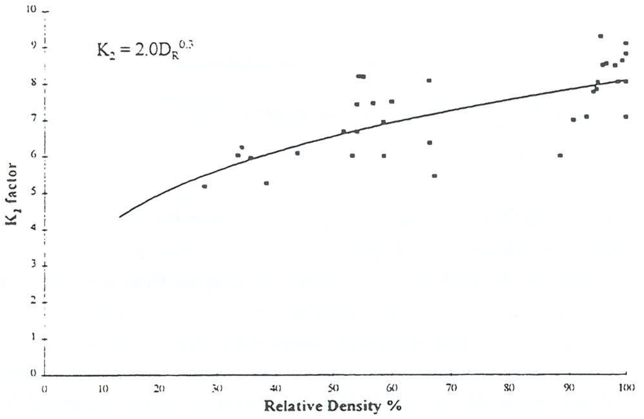

Figure 6 K_2 factor for G_{max} versus density from experimental data

The most commonly used expression relating confining pressure and low strain shear modulus is from Seed et al. [12], which is in the form (in S.I. units)

$$G_{max} = 6945K_2(p^/)^a \qquad 1$$

where G_{max} is in Pa
$p^/$ is the effective confining pressure in Pa
a,K_2 are constants

Typically a is taken as 0.5 and K_2 can be related to relative density or SPT blow count. For the Puni sand however, the power term (constant a) was found to be 0.6, and the K_2 term is derived from the best fit line of Figure 6. The full expression for the low strain modulus (D_R>10%) is therefore

$$G_{max} = 6945*2.0(D_R)^{0.3}(p^/)^{0.6} \qquad 2$$

where D_R is the relative density in % (D_R>10%)
$p^/$ is the confining pressure in Pa

In comparison to the expressions derived for quartz sands [12], the pumice sand exhibits significantly lower values of G_{max} for all relative densities. This ranges from 30% to 60% of the G_{max} values of quartz sands at the same relative densities. 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 New Zealand soils where possible.

5. NON LINEAR CONSTITUTIVE PROPERTIES

The constitutive properties of soils are very significant in seismic response computations [4] and requires close consideration. The available dynamic torsion test data on the form of the non linear shear strain versus shear modulus relationship for Puni sand from a previous study [7] only extends to shear strains in the region of 10^{-1}%. Shear strains of this magnitude are likely to be induced in moderate to large earthquakes, and very large earthquakes are 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 induced shear strains up to 1% in some cases.

The ratio G/G_{max} was determined by dividing the value of G obtained from the cyclic triaxial test results, tested at a confining pressure of 100 kPa, by the value of G_{max} determined from Equation 2. These large strain results were then plotted on the existing data from the torsion tests, which is shown in Figure 7. Also shown are the empirical curves given by Seed et al. [12] for sands.

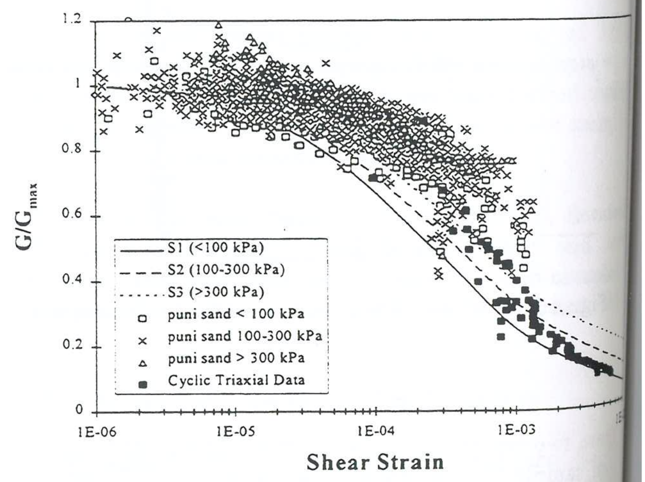

Figure 7 Normalised shear modulus versus shear strain response from all tests

This plot shows the scatter and trends of the constitutive behaviour of the sand. The 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. 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.

Torsion test results from volcanic ashes [1] 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.

6. 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 in dominated by overseas data derived primarily from quartz sands, it was important to investigate the dynamic behaviour of a New Zealand pumice sand. Most seismic design analyses rely to varying degrees on empirical soil relationships, and these therefore must be investigated for New Zealand conditions. 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 to consider. 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 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 non linear 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 non linear 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.

7. ACKNOWLEDGMENTS

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