A New Approach for Liquefaction Assessment of Pumiceous Sands

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ABSTRACT

Sands consisting of pumice particles are a common soil type in several parts of the Waikato Basin and Bay of Plenty in the North Island of New Zealand. These pumice sands are highly crushable, compressible, and lightweight due to the vesicular nature of the particles. Pumiceous soils are often seen to be 'problematic' because their behaviour is considerably different compared to that of normal sands. Accordingly, the available empirical correlations for ordinary soils may not be applicable for the liquefaction assessment of pumiceous soils. Therefore, it is imperative to follow appropriate design recommendations for the liquefaction assessment of these soils. The development of several large-scale solar farms in the Eastern Bay of Plenty region requires more reliable approaches to assess the liquefaction susceptibility of pumiceous sands as some of the solar farm locations have thick layers of pumiceous deposits present across the site. The key geotechnical risk for solar farms in the Bay of Plenty is often related to soil liquefaction and the effects this may have on the foundations for various structures and the solar arrays. This paper will present a case study where several seismic cone penetration tests (sCPT) were performed and the obtained CPT and shear wave velocity (V_s) profiles were used for a site-specific liquefaction assessment. The results of the liquefaction assessment were made using the methods which reflect current good industry practice in New Zealand. In addition, a second assessment was carried out using recently developed pumice-specific correlations, which provided significantly improved cyclic resistance ratios (CRR) for pumiceous sands. In this paper, the important differences in using the newly developed and more advanced methods for liquefaction assessment of pumiceous deposits are discussed and presented.

Keywords: Pumice soil, particle crushing, liquefaction assessment, shear wave velocity

1 INTRODUCTION

Sands consisting of pumice particles are found in several parts of the Waikato Basin and the Bay of Plenty. These pumice sands are highly crushable, compressible, and lightweight, due to the vesicular nature of the particles, making engineering assessment of their properties complex and challenging. These unique characteristics of pumice particles significantly affect their behaviour under seismic conditions and are very different from the seismic response of 'normal' sand (Asadi et al. 2022a & b; Orense et al. 2020). The presence of crushable and porous pumice components in the naturally occurring pumiceous sands causes a softer response of those particles, and therefore, they have lower shear wave velocity (V_s). During seismic loading, pumice particle crushing results in significant increased particle-to-particle contacts. Subsequently, the irregularly shaped pumice particles can provide good interlocking between pumiceous sand particles, causing higher liquefaction resistance (Asadi et al. 2019). Following standard geotechnical engineering practice, pumiceous sands typically have lower V_s compared to the 'normal' sands; therefore, they are expected to have low liquefaction resistance which is not generally correct according to the comprehensive studies conducted by Asadi et al. (2022).

After the 1987 Edgecumbe Earthquake, manifestations of soil liquefaction, such as sand boils and ejected materials, have been reported for pumiceous deposits in many locations, including two sites reported by Orense et al. (2020), one in Whakatane and another in Edgecumbe. According to the case study of the 1987 Edgecumbe Earthquake published by Orense et al. (2020) at the above sites, the simplified empirical-based methods (for example, using CLiq software) would predict the occurrence of liquefaction for pumiceous deposits. However, while their assessments were consistent with the observed occurrence of liquefaction at the sites, the severity of liquefaction-induced damage, as indicated by the very low factor of safety against liquefaction, would not explain the minor-to-moderate degree of damage reported at the sites. This finding can be correlated to the conflicting response observed from pumiceous sands during dynamic testing as they typically have low V_s while providing relatively high liquefaction resistance. Accordingly, Asadi et al. (2022a) proposed a new methodology for the liquefaction assessment of pumiceous sands containing different pumice content (PC). This paper assesses a site-specific liquefaction assessment of the pumiceous deposits using advanced existing methods and the newly developed pumice-specific correlations.

2 PROJECT OVERVEW AND SITE DESCRIPTION

Lodestone Energy Limited plans to develop several solar farms in the Eastern Bay of Plenty region. The projects generally rely on several thousands of foundation piles to support the solar PV modules and the associated infrastructure. In addition, other civil and infrastructure works are required such as earthworks, access roads, stormwater, power structures, and buildings and infrastructure such as buried cables. For the assessment, management and mitigation of key project risks, including ground risks, it is vital to develop site-specific ground models to better identify the potential geohazards for each site. The focus of this paper is the liquefaction assessment of the soil deposits present at one of the proposed solar farm sites (per client request, the name and the location of this site need to be kept anonymous).

2.1 Field testing

Various deep investigations, including borehole logs, test pits and cone penetration tests (CPT) were carried out to develop a site-specific ground model, to assign geotechnical design parameters and to provide recommendations allowing the assessment the geotechnical hazards. Further to the site investigation methods mentioned above, more advanced testing methods were performed to allow the assessment of the liquefaction potential of pumiceous soils, i.e., eight (8) seismic cone penetration tests (sCPT) to 20m depth. Such tests provide shear wave velocities for these soils, which are more suitable for a liquefaction assessment of pumiceous soils according to the MBIE Module 3 (2021).

2.2 Geological units

The proposed site is underlain by up to 2.5 km of volcanic, marine and non-marine sediments and significant ignimbrite deposits. The Holocene geology comprises sediments from marine incursion and non-marine sediments such as volcanic tephra, re-worked tephra, peat and back swamps and alluvium deposited by the Rangitāiki River. The geology from the site investigation has been interpreted into a geological model, and this is summarised in Table 1 (listed from youngest to oldest).

2.3 Assessment of pumiceous layers thickness and pumice content (*PC*)

The thickness of the pumiceous sand layers was estimated across the site by correlating the CPT tip resistance (q_c) to the known extent of pumice units in the nearby borehole logs, and therefore, using the tip resistance of CPT results to understand the extend of the pumiceous layers thickness for the investigated sCPT profiles. The estimated thickness of the pumiceous soil layers at the eight (8) sCPTs test locations are ranging from 1.5 to 2.5 m thickness for Unit 3 and 2.5 to 3.5 m thickness for Unit 4.

The method of Asadi et al. (2019) need to be followed to estimate the pumice content (*PC*) of the pumiceous sands found on site. However due to the in-depth experience of the author of this paper (also the same author of the Asadi et al. (2019) approach)) with the soils in the Bay of Plenty, the *PC* of the samples herein has been visually estimated using the borehole log material, as well as using the index properties of the soils (obtained from laboratory testing) to confirm the estimated values of the *PC of each sample*. These assessments indicate that the *PC* content in the granular Pumiceous (Unit 3) is about 40-50%, and about 50-60% in the Pumiceous Tephra (Unit 4)

Table 1: Design engineering geotechnical units.

Unit No.	Name	Material Description
Unit 1	Recent Ashfall / Alluvium	Silt dominated, firm-stiff, caps the full extent of the site, encountered below the topsoil and typically extends 1.0 – 2.0 m bgl.
Unit 2	Swamp Deposits	Cohesive, fibrous peat and organic silt, thin (1-2m thickness), highly compressible, present across most of the site.0.5 to 1.0 m thick.
Unit 3	Granular Pumiceous	Loose to medium dense, fine to coarse pumice sand and gravel with varying silt, present across most of the site with 2 – 4m thickness.
Unit 4	Pumiceous Tephra	Very Stiff / medium dense to dense pumiceous medium-coarse silt and fine sand, moderately hard to dig with excavator. Non-plastic, liquefies when disturbed during drilling. Occurs in 3 - 5m thick layers across the entire site.
Unit 5	Non-Pumiceous Alluvial/Estuarine	Soft silt with minor clay, non-pumiceous and more plastic than above units. Predicted to underly the full site, minimum 2m thickness.

2.4 Seismic design parameters

The seismic design parameters for liquefaction assessments are summarised in Table 2 for serviceability limit sate (SLS) and ultimate limit state (ULS) conditions. These parameters have been determined using the guidance provided in MBIE Module 1 (2021). Table 2 also presents the ground water level (GWL) used for the liquefaction assessment carried out in this paper.

Table 2: Design input parameters for liquefaction assessment

Seismic State Limit	Annual Probability of Exceedance (AEP)	Peak Ground Acceleration (PGA)	Moment Magnitude (M_w)	GWL (m bgl)
Scenario 1 (SLS1)	1/25	0.11g	6.1	1.0
Scenario 2 (ULS)	1/100	0.22g	6.1	1.0
Scenario 3 (ULS)	1/500	0.44g	6.1	1.0

3 METHODS OF LIQUEFACTION ASSESSMENT

3.1 Existing empirical methods

The primarily liquefaction assessment was carried out using GeoLogismiki software package CLiq (version 3.0.3.4) following two standardized methods as follows: (1) to analyse CPT profiles: Boulanger and Idriss (2014); and (2) to analyse V_s profiles: Andrus & Stokoe (2000), and Kayen et al. (2013).

3.2 Pumice-specific empirical method

Asadi et al. (2022a) proposed a step-by-step procedure, a newly developed V_s-based method, to assess the liquefaction susceptibility of pumiceous sands. The key aspects of this method are set as below:

- Determine the V_s profile of the soil deposit.
- Obtain disturbed soil samples, e.g., from borehole logs.
- Asses the thickness of pumiceous layers.
- Quantify / estimate pumice content - use Asadi et al. (2019) method or soil properties.
- Assess liquefaction resistance - use the liquefaction assessment chart shown in Figure 1 or alternatively use the empirical correlations presented in Equations 1 and 2.

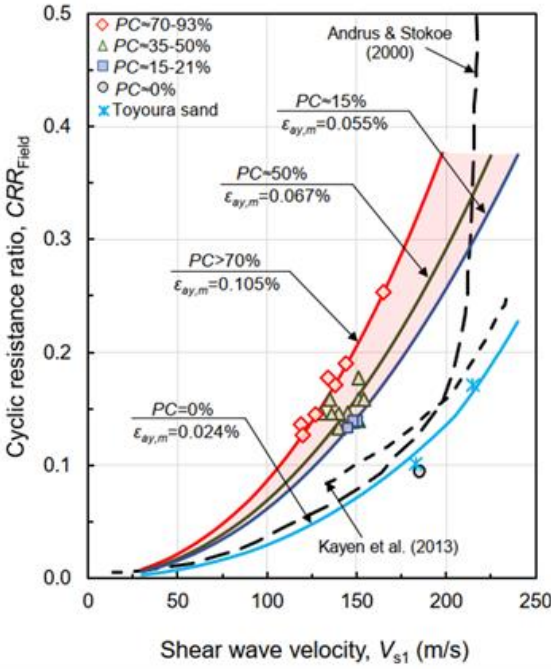

Figure 1. Liquefaction assessment chart for pumiceous sand and compared with the normal sand correlations (Asadi et al. 2022a).

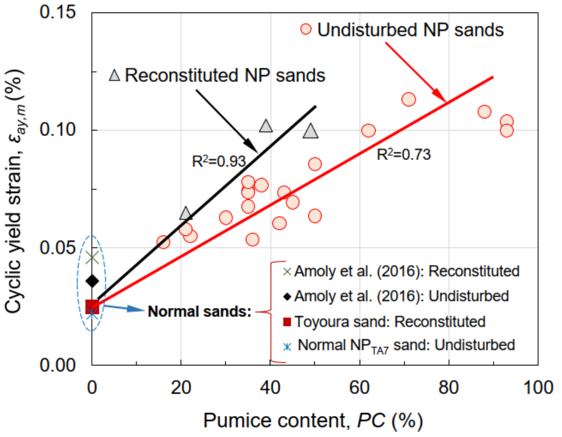

Figure. 2. The relation of cyclic yield strain factor with pumice content (Asadi et al. 2022b).

$$CRR_{(Field)} = \frac{3\rho}{(1+2K_0)\,P_a}\ \varepsilon_{ay,m\,(Field)}\ {V_{s1}}^2 \qquad (1)$$

where $CRR_{(Field)}$ is the liquefaction resistance of soils in field condition; ρ is soil's bulk density; K_0 is the lateral earth pressure coefficient (ranging 0.4 to 0.5); P_a is the referenced overburden pressure (= 100 kPa); V_{s1} is the normalised V_s with respect to P_a; and $\varepsilon_{ay,m\,(Field)}$ is cyclic yield strain factor (in field conditions). The investigated $\varepsilon_{ay,m}$ (in laboratory conditions) for different pumice contents is shown in Figure 2; next, use Equation 2 to convert the $\varepsilon_{ay,m}$ (from Figure 2) to $\varepsilon_{ay,m\,(Field)}$ to use in Equation1.

$$\varepsilon_{ay,m\,(Field)} = r_c \left(\frac{1+2K_0}{3}\right)^{2n+1} \varepsilon_{ay,m} \qquad (2)$$

where r_c is a constant value to account for the multi-directional shaking effects in field (=0.95), and n is power coefficient of confining pressure (for $PC \leq 5\%$ use $n = 0.25$; $5\% < PC \leq 50\%$ use $n = 0.3$; and $PC > 50\%$ use $n = 0.35$). For further details see Asadi et al. (2022a) publication.

4 RESULTS AND DISSCUSSTION

Eight (8) seismic cone penetration tests (sCPT) were completed at the proposed solar farm site and have been used to determine the liquefaction susceptibility of the pumiceous soil layers (Unit 3 and Unit 4) following the procedures explained above.

4.1 CLiq analyses outputs (CPT-based results)

This section summarises the results of liquefaction analyses using CPT profiles only for the three seismic scenarios outlined in Table 2. The assessment of liquefaction induced free-field vertical settlements as the result of post-liquefaction volumetric reconsolidation strain is important to estimate the surface settlements for this project. Therefore, the post-liquefaction ground settlements, using CPT-based approaches, are estimated and reported in Table 3. These results are based on using the existing engineering methods (from CLiq) for the liquefaction assessment. The estimated settlements for Scenario 1 are significantly lower than those predicted for Scenario 2 and Scenario 3, predominantly due to the lower seismic design parameters, which are less than the estimated liquefaction triggering PGA value (the triggering PGA is at approximately 0.15g). The estimated ground settlements for Scenario 2 and Scenario 3 indicate comparable total settlements values. This similarity for Scenario 2 and Scenario 3 can be explained as the PGA values for both scenarios are much greater than the estimated liquefaction triggering PGA value where the Settlement-PGA curves tend to become horizontal, and the majority of liquefiable sediments have been liquefied by ground shaking. Note that the tabulated results under 'pumice-specific methods' shown in Table 3, will be explained in Section 4.4.

Table 3: Summary of total vertical ground settlement for SLS and ULS events.

sCPT No.	Total estimated vertical free field surface settlements (mm)					
	CLiq outputs			Pumice-specific methods		
	Scenario 1	Scenario 2	Scenario 3	Scenario 1	Scenario 2	Scenario 3
sCPT101	12	70	86	Nil	45	86 (70%)
sCPT102	27	92	111	Nil	25	111 (70%)
sCPT103	7.5	86	127	Nil	40	127 (70%)
sCPT104	6	61	81	Nil	25	81 (70%)
sCPT105	18	137	155	Nil	20	155 (70%)
sCPT106	19	126	148	Nil	30	148 (70%)
sCPT107	5	50	74	Nil	25	74 (70%)
sCPT108	12	120	131	Nil	30	131 (70%)

4.2 CLiq analyses outputs (V_s-based results)

The CLiq software was also used for liquefaction assessments using the V_s profiles of the soil deposits based on the above stated seismic input parameters. The CLiq software is not able to directly estimate the free-field ground settlement using the V_s-based methods. Alternatively, the factor of safety (*FoS*) against liquefaction is presented instead to illustrate the severity level of liquefaction for the different scenarios. The *FoS* = *CRR/CSR*, where *CRR* is the 'cyclic resistance ratio' (liquefaction resistance), and *CSR* is the 'cyclic stress ratio' (the shear stress ratio induced in the soil deposit from an earthquake).

The sensitivity of the liquefaction assessment results for the three seismic scenarios (Table 1) as well as alternative liquefaction assessment methods were assessed. The Andrus and Stokoe (2000) methodology estimated that the overall soil profiles (including pumiceous layers) are susceptible to liquefaction for all three seismic scenarios (with *FoS* < 1.0). Figure 3 shows the outputs of the Andrus and Stokoe (2000) method for sCPT102. In addition, the Kayen et al. (2013) method was applied, and it was estimated that these soil profiles are susceptible to minor liquefaction under Scenario 1 (1 < *FoS* < 1.1) and the site was assessed to be highly liquefiable under Scenarios 2 and 3 (*FoS* < 1).

4.3 Pumice-specific analyses outputs

Following the Asadi et al. (2022a) method, the sCPTs (V_s profiles only) were analysed to assess their liquefaction susceptibility, considering the thickness of the pumiceous layers as well as their estimated *PC*. Figure 3 presents the liquefaction assessment results for sCPT102 for the three seismic scenarios. The plot also shows the comparison between the use of pumiceous-specific correlations and the more generic Andrus & Stokoe (2000) method. At the location of sCPT102 the pumiceous layers are present between 2.3 to 7 m bgl. Therefore, for these specific depths, pumiceous-specific correlations were applied while ‘standard’ correlations were used for non-pumiceous soil layers above and below this horizon (the pumice-specific correlations can still be used for non-pumiceous layers with input *PC* = 0%; see the comparison in Figure 3 between the two methods for non-pumiceous layers). As evident from Figure 3, under seismic Scenario 1 and 2, the pumiceous layers are not liquefiable with *FoS* ranging from 2.0 to 3.9 and 1.2 to 2.0, respectively, whereas the pumiceous layers are estimated to be liquefiable under Scenario 3 with *FoS* ranging from 0.6 to 0.9. It should be noted that while the pumiceous layers are liquifiable under seismic Scenario 3, they have much higher *FoS* compared to the results of Andrus and Stokoe (2000) as demonstrated in Figure 3 (c).

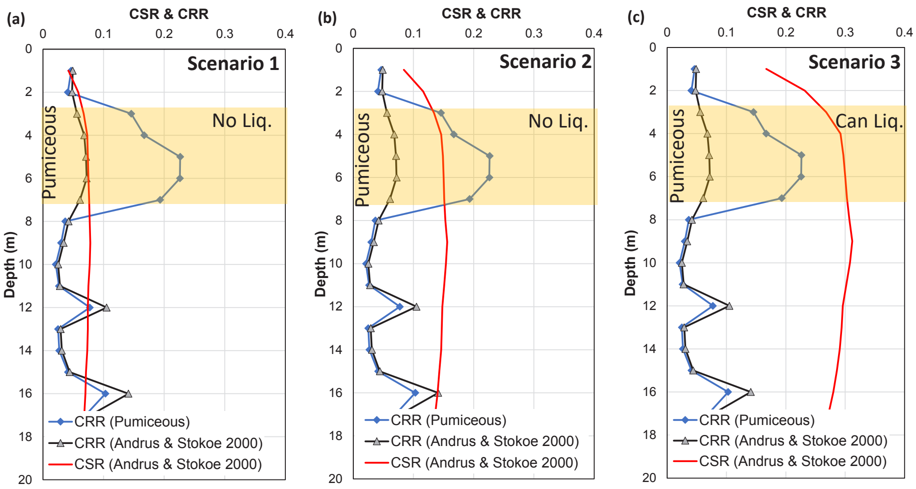

Figure 3. Liquefaction assessment results for sCPT102 profile: (a) Scenario 1, (b) Scenario 2, and (c) Scenario 3 (the shaded yellow zone represents the layers of pumiceous sand).

4.4 Design considerations

For sites with level ground conditions, the severity of ground damage caused by liquefaction is affected by the properties and thickness of the liquefied layers, and by the location (depth) of the liquefied layers within the soil profile. Ground displacements and liquefaction-induced settlements generally increase with the thickness of the liquefied layer, and with the proximity of the liquefied layer to the ground surface. The presence of a non-liquefiable crust at the ground surface may reduce the manifestation and damaging effects of liquefaction at the surface, as observed by Ishihara (1985) and more recently during the Canterbury earthquake sequence (Cubrinovski et al. 2019). Such beneficial effects of the surface crust should only be expected in cases where lateral spreading does not occur, and where the crust is sufficiently thick and robust to ensure reduced differential movements for relatively light structures. Ishihara (1985) developed criteria identifying conditions for the occurrence of liquefaction-induced free field settlements based on the thickness of the liquefied sand layer (H2) and the thickness of an overlying crust of non-liquefied soils at the ground surface (H1). These criteria are expressed in a chart comparing different thicknesses of H1 and H2 in which boundary curves for identification of ‘liquefaction-induced ground damage’.

Since the overall crust thickness of the top four soil units (Table 1) is approximately 7 m to 9 m thick across the Solar farm area, and these geological units are identified as non-liquefiable material for both, Scenario 1 and 2 (based on the Asadi et al. (2022) method), and the solar arrays are light weight structures, the Ishihara chart (refer to MBIE Module 3 (2021)) is used herein for ground induced damage assessment. Using the Ishihara chart and considering about 7 m - 9 m of non-liquefiable layers, this solar farm is expected to have a low potential for ‘liquefaction-induced ground damage’ under seismic Scenario 1 and 2. Furthermore, considering that the pumiceous layers are not liquefiable (under Scenario 1 and Scenario 2), the total ground settlement would be mainly a function of the settlement of the liquified material at depths below Unit 4. The estimated settlement values (from CLiq) for the material below the pumiceous layers is used to estimate the total ground settlement under Scenario 1 and 2 (these values are summarised in Table 3). The estimated post-liquefaction settlements are expected to be negligible for Scenario 1 and approximately 20 mm to 45 mm for Scenario 2. Regarding seismic Scenario 3, the post-liquefaction settlements are expected to be lower than those estimated from CLiq because the pumiceous-specific correlations estimated much higher *FoS* for Scenario 3 (and yet *FoS* < 1.0). The settlement values from CLiq analysed for Scenario 3 are then reduced by about 30% to incorporate the effect of pumiceous layers; this is in accordance with the differences between the *FoS* estimated by CLiq and Asadi et al. (2022) methodologies.

5 CONCLUSIONS

The presence of crushable, angular, and porous pumice components in natural pumiceous sands can result in conflicting responses when compared to ‘normal’ sands during seismic shaking, as pumiceous typically have low shear wave velocity (V_s) whilst having high liquefaction resistance. Considering the above observations and findings, the currently available correlations to assess liquefaction resistance developed from ‘normal’ sands would significantly underestimate the liquefaction resistance of pumiceous sands. Instead, the new correlations, specifically developed for pumiceous sands, were used in this project for more reliable liquefaction assessments of these crushable material. The application of a new method (Asadi et al, 2022a) to assess the liquefaction potential of pumicous sands significantly reduced the estimated ground settlements caused by soil liquefaction for the proposed solar farm project (in contrast with the outputs from CLiq software), which reduced the overall ground risk.

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