Applicability of field-based empirical methods for evaluating liquefaction potential of pumiceous deposits

R P Orense, L M Wotherspoon and M J Pender
Department of Civil & Environmental Engineering, University of Auckland, NZ
r.orense@auckland.ac.nz (Corresponding author)

S Van Ballegooy
Tonkin & Taylor Ltd., Auckland, NZ

M Cubrinovski
Department of Civil & Natural Resources Engineering, University of Canterbury, NZ

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ABSTRACT

Pumice materials, which are problematic from an engineering viewpoint, are widespread in many parts of the North Island. Following the 2010-2011 Christchurch earthquakes, a clear understanding of their properties under earthquake loading is necessary. For example, the 1987 Edgecumbe earthquake showed evidence of localised liquefaction of sands of volcanic origin. To elucidate on this, research was undertaken to investigate whether existing empirical field-based methods to evaluate liquefaction potential of sands, which were originally developed for hard-grained soils, are applicable to crushable pumice deposits. For this purpose, two sites were selected where liquefaction have been observed following the Edgecumbe earthquake. Test site #1 was located adjacent to the Whakatane Sewage Pump station, while Test site #2 was opposite the Edgecumbe substation. Manifestations of soil liquefaction, such as sand boils and ejected materials, have been reported at both sites. Field tests, including cone penetration tests (CPT), shear-wave velocity profiling through seismic CPT, and screw driving sounding (SDS) tests were performed at the sites. Then, considering estimated peak ground accelerations (PGAs) at the sites based on recorded motions following the 1987 earthquake and possible range of ground water table locations, liquefaction evaluation was conducted at the sites using available empirical chart-based approaches. The applicability of the current field-based empirical approaches in assessing the liquefaction potential of pumiceous deposits was then scrutinised vis-à-vis the observed liquefaction manifestation at the target sites.

1 INTRODUCTION

The 2010-2011 Christchurch earthquakes have highlighted the impact of soil liquefaction and associated phenomena to the built environment. A cursory review of the current state of research on soil liquefaction showed that nearly all the work on this topic has been directed towards understanding the properties of hard-grained (quartz) sands; very little research has been done on the dynamic characteristics of volcanically-derived sands.

However, it is known that New Zealand's active geologic past has resulted in widespread deposits of volcanic soils throughout the country. The M_L=6.3 1987 Edgecumbe earthquake, for example, showed localized patches of liquefaction of sands of volcanic origin across the Rangitaiki Plains (Pender & Robertson 1987). Pumice deposits are found in several areas of the North Island. Although they do not cover wide areas, their concentration in river valleys and flood plains means they tend to coincide with areas of considerable human activity and development. Thus, they are frequently encountered in engineering projects and their evaluation is a matter of considerable geotechnical interest.

Previous research by Wesley et al. (1999) showed that the penetration resistance (q_c) values obtained from cone penetration tests (CPT) on pumice sand were only marginally influenced by the density of the material. The reason for this behaviour is possibly because the stresses imposed by the penetrometer are so severe that particle breakage forms a new material and that the properties of this are nearly independent of the initial state of the sand. Thus, conventional relationships between q_c value and relative density, D_r, which in turn is correlated with liquefaction resistance, appear to be not valid for these soils. In addition, research results reported by Orense et al. (2012) and Orense and Pender (2013) indicated that penetration-based approaches, such as cone penetration tests and seismic dilatometer tests, underestimated the value of liquefaction resistance of the pumice deposits, confirming that any procedure where the liquefaction resistance is correlated with relative density will not work on pumiceous deposits. The same research showed that empirical method based on shear wave velocity seemed to produce good correlation with liquefaction resistance of pumiceous soils.

Admittedly, the above observations were obtained from limited number of test data and such conclusions have not been well-validated. With many consultants and practitioners constantly asking for advice on how to evaluate the liquefaction susceptibility of pumice deposits, there is indeed a need to clarify and address this issue.

This paper presents the results of recent investigation on the evaluation of liquefaction triggering of in-situ pumice deposits through field testing at two sites where liquefaction have been observed following the 1987 Edgecumbe earthquake. Using field-obtained data, attempts were made to explain the occurrence/non-occurrence of liquefaction at the target sites following the earthquake using available empirical chart-based approaches. The applicability of the current field-based empirical approaches in assessing the liquefaction triggering of pumiceous deposit was then scrutinised vis-à-vis the observed liquefaction characteristics of pumice sands.

2 SELECTED TEST SITES AND FIELD TESTING

Two sites where liquefaction had been observed during the 1987 Edgecumbe Earthquake were selected in this research. Test site #1 was located adjacent to the Whakatane Sewage Pump station, while Test site #2 was opposite the Edgecumbe substation (see Figure 1). Manifestations of soil liquefaction, such as sand boils and ejected materials, have been reported at both sites (Pender & Robertson 1987).

At both sites, borehole sampling, cone penetration test (CPT), seismic cone penetration test (sCPT), and screw driving sounding (SDS) were performed (see Figure 2). SDS is a new in-situ method that has recently been developed in Japan, where a rod is drilled into the ground at several loading steps at the same time as the rod is being continuously rotated. Details of this test are reported elsewhere (e.g., Orense et al. 2013.) The SDS test is fast, the machine is small in size and the implementation is relatively cheap, compared to other in-situ testing methods.

Boreholes from the two tests sites indicate the presence of fine to coarse sand layers intermittently mixed with pumice. At site #1, the presence of pumice sands were visible between 0.5-7m, while at site #2, pumice was mixed with fine to medium sand from 0.5-6.2m. The ground water table was located at about 2m from the surface at both sites.

The results of the field tests are shown in Figures 3 and 4 for site #1 and site #2, respectively. At site #1, the cone tip resistance was about q_c=4 MPa up to a depth of 5m and it increased to about q_c=8 MPa up to a depth of 10m. The soil behaviour type (SBT), derived from CPT data, indicated alternating layers of sand and silt mixtures up to a depth of 5m, and predominantly sand up to a depth of 12m. The shear wave velocity profile showed V_s ranging from 90-120 m/s up to a depth of 5m, after which V_s increased with depth, reaching 170 m/s at depth of 11m. During the SDS test, several parameters were measured every 25cm; these include torque, load,

(a) (b)

Figure 1: Target sites for the study: (a) Test site #1 near Whakatane Sewage Pump station; and (b) Test site #2 near Edgecumbe substation. Yellow hatched zones are the regions where liquefaction had been observed based on literature.

(a) (b)

Figure 2: Field testing at Target site #1: (a) CPT and sCPT; and (b) SDS test.

speed of penetration, depth of penetration and friction on the rod. An important parameter derived was the specific energy of penetration, E_s, representing the sum of the contribution of the torque and applied load for every load step normalised by the volume of penetration (Mirjafari et al. 2016). At site #1, E_s <25 N-mm/mm^3 up to a depth of 10m, below which stiff layer with E_s<50 N-mm/mm^3 existed.

At site #2, q_c was generally <8MPa, except at depths of 3.0-3.5m and >6.5m; V_s generally varied between 110-170 m/s. SDS indicated E_s< 30 N-mm/mm^3, except at depths of 3.0-3.5m and >6.5m. From all tests, a hard layer was apparent at depth of approximately 7m, where q_c >20 MPa, V_s >160 m/s, and Es>70 N-mm/mm^3; all tests were terminated at this depth.

3 EVALUATION OF LIQUEFACTION TRIGGERING

For the purpose of evaluating the liquefaction triggering at both sites during the Edgecumbe earthquake, six simplified empirical methods were employed: 3 CPT-based methods – i.e. those proposed by Boulanger and Idriss (2014), Robertson and Cabal (2012) and Moss et al. (2006); two V_s-based methods – i.e. those proposed by Andrus and Stokoe (2000) and Kayen et al. (2013); and the SDS-based method proposed by Mirjafari et al. (2016). Per the analyses of Mellsop (2016), the Edgecumbe earthquake, with moment magnitude M_W=6.5, induced the following peak ground accelerations (PGA): 0.29g in site #1 and 0.53g in site #2.

In terms of ground water table (GWT), Pender et al. (1987) reported the following: "*the earthquake occurred at the end of the summer and after a long period of dry weather. Most of the deposits underlying the Rangitaiki Plains are saturated, with the water table ranging from near surface in the coastal margin, to about 3 m below ground level in the Te Teko area. (In Edgecumbe), the top of the soil profile is a layer of about 3m thickness which is very loose, (and) at the time of the earthquake, the water table was probably towards the bottom of this*

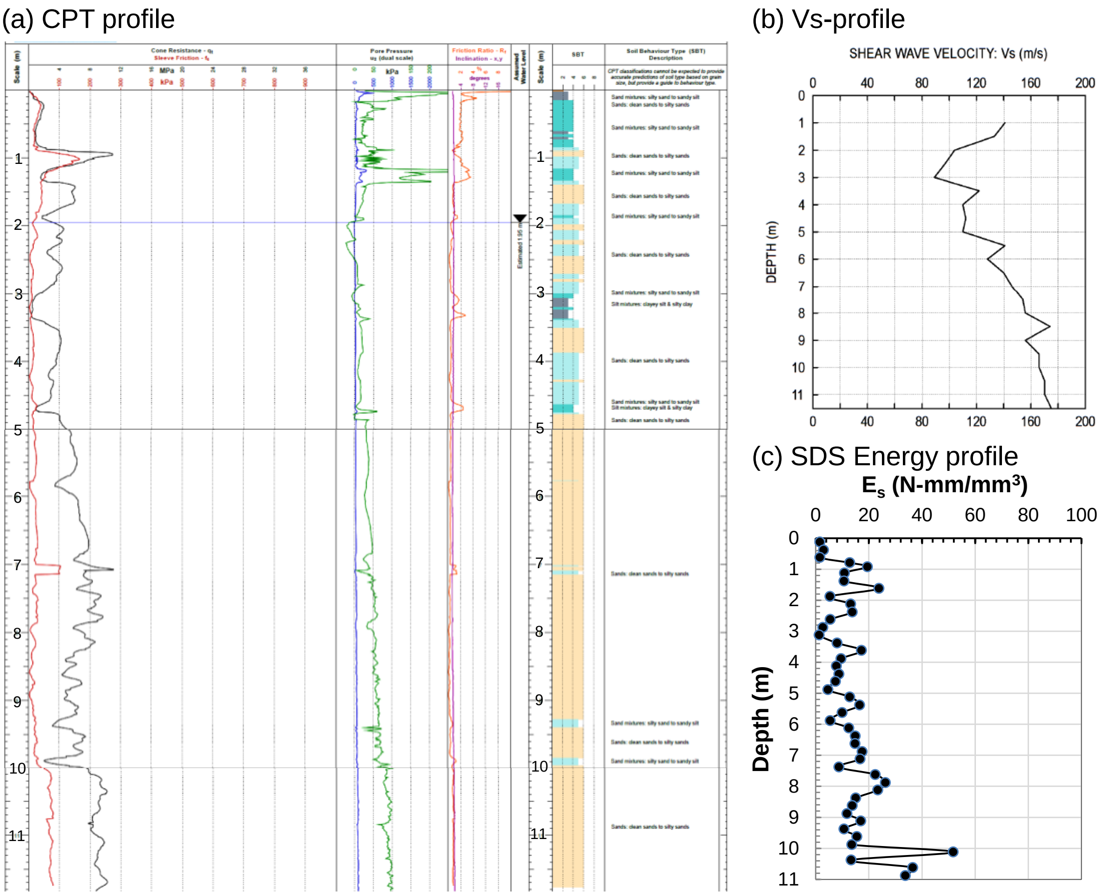

Figure 3: Results of field testing at site #1.

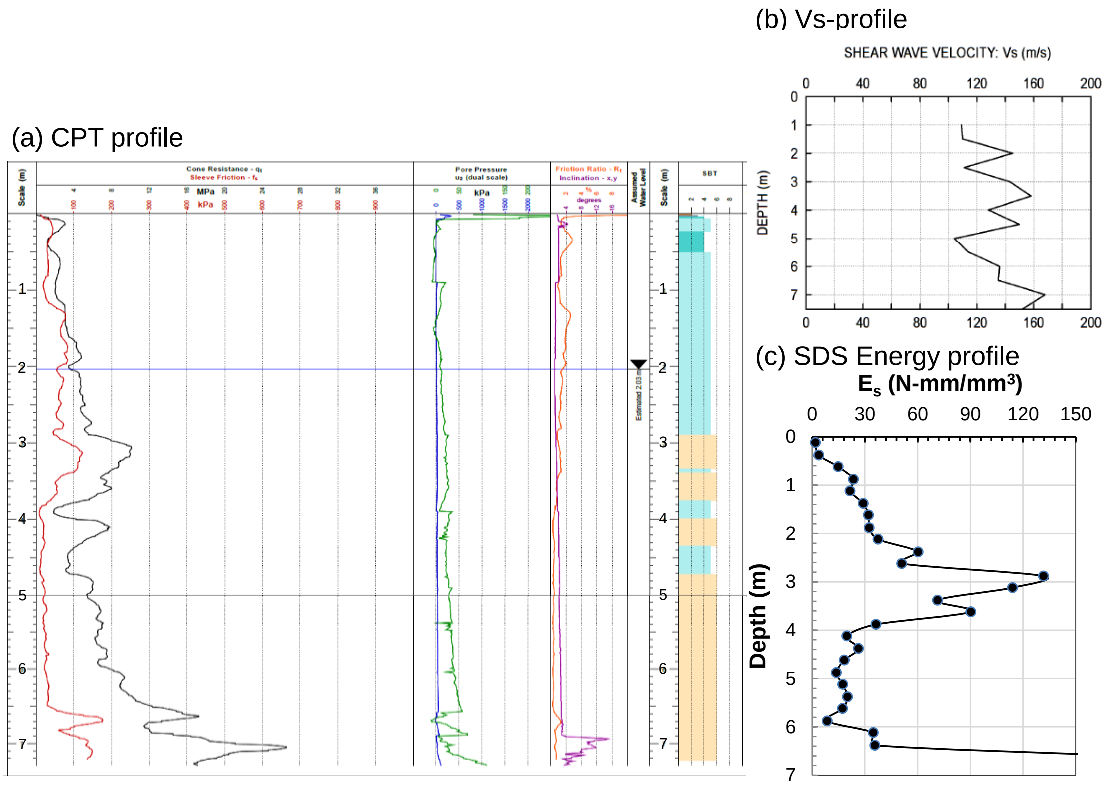

Figure 4: Results of field testing at site #2.

layer over much of the plains." Thus, for the purpose of the analyses, the GWT location was assumed at: 1m, 2m and 3m from the ground surface for site #1, and 2m, 3m and 4m for site #2.

Considering the input parameters mentioned above, liquefaction assessment was conducted using the 6 simplified methods. Results for site #1 considering GWT=2m are shown in Figure 5, while Figure 6 illustrates the results for site #2 with GWT=3m. In the figures, the depth profile of the cyclic stress ratio (*CSR*) and cyclic resistance ratio (*CRR*) are plotted, and the shaded regions represent the pumiceous zones which are deemed to have liquefied (i.e. $CRR \leq CSR$).

Based on the results, it is clear that at site #1 where field testing has been done up to a depth of 11.5m, all the methods considered would predict liquefaction of pumiceous deposits (between the depth ranging from the location of the water table up to 7m depth). Similarly, at site #2, all methods would predict pumice liquefaction between GWT up to 7m depth. Similar results were obtained for the other GWT locations. Thus, it would appear that for the sites selected, all empirical methods were able to predict the occurrence of liquefaction of the pumice layers. Note that at both sites, it would appear that the *CSR* induced by the earthquake was almost twice that of the liquefaction resistance of the soil, *CRR* (i.e. factor of safety, $F_L \approx 0.5$).

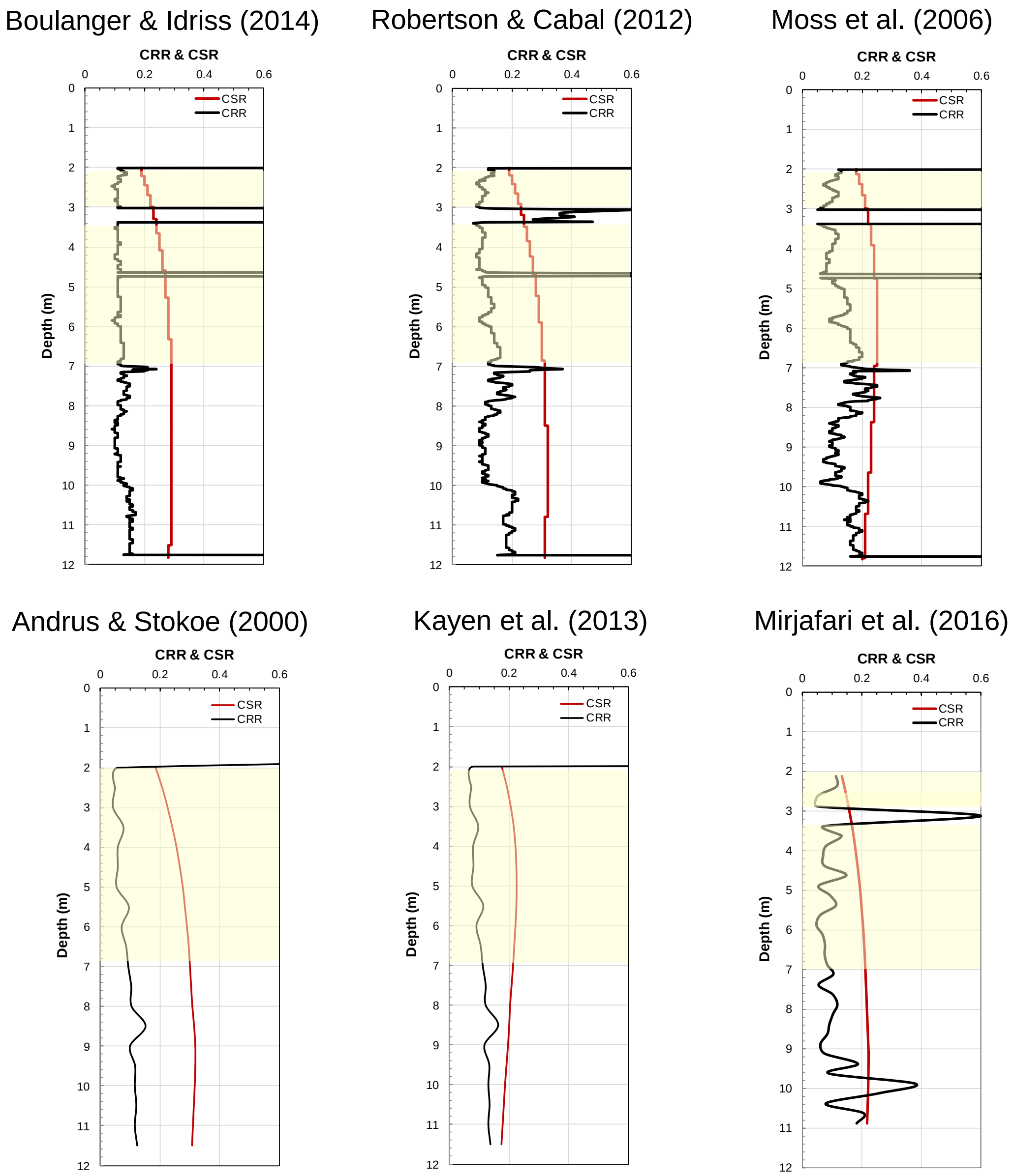

Figure 5: Liquefaction triggering results for site #1, with GWT=2.0m.

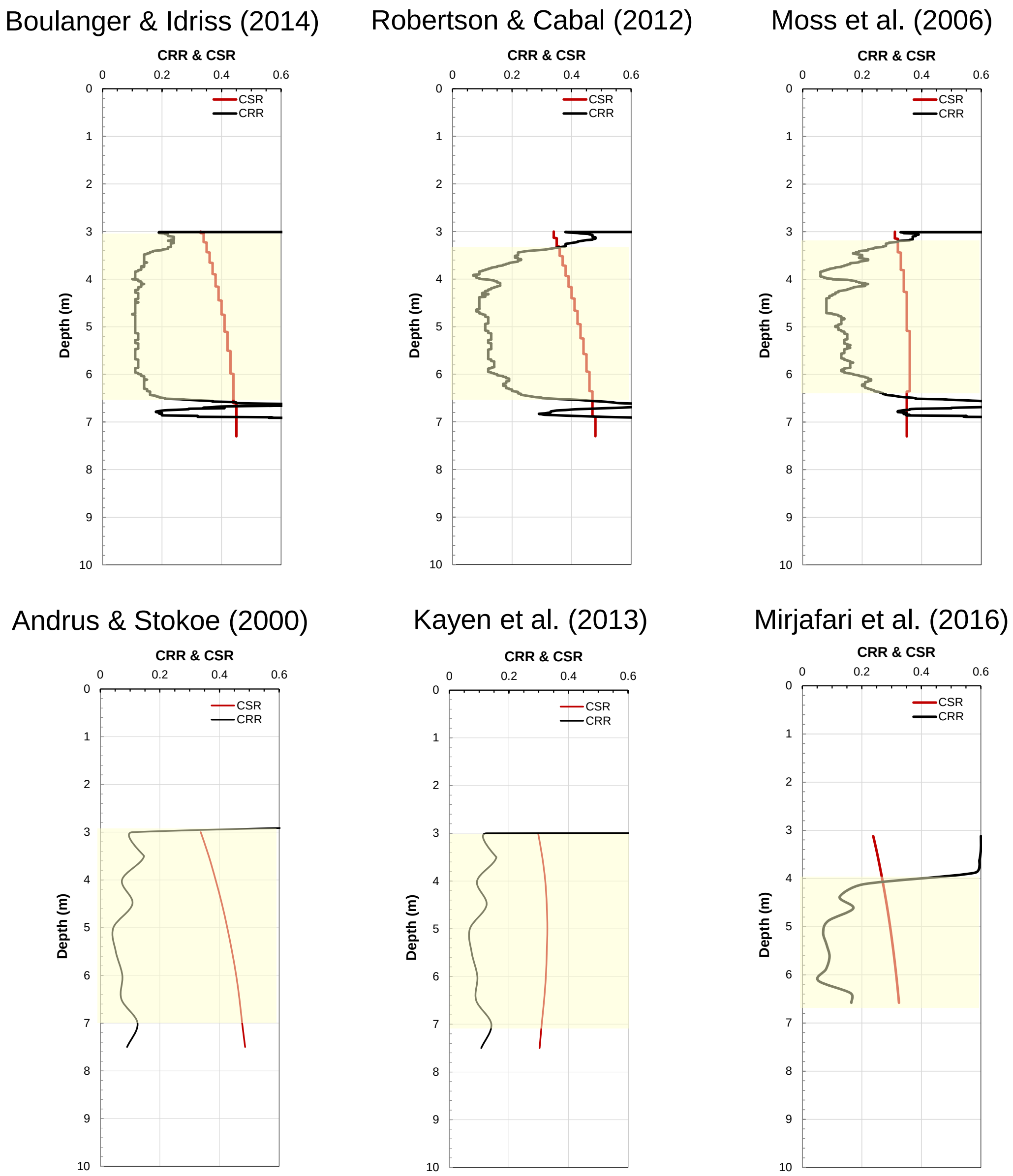

Figure 6: Liquefaction triggering results for site #2, with GWT=3.0m.

As mentioned earlier, borehole logs were taken at both sites and, in addition, undisturbed samples were obtained at site #1. Careful inspection of the obtained samples seems to indicate that the pumice contents at these sites were not as significant as initially reported in the literature (e.g. Pender et al. 2007). Figure 7 shows some of the samples obtained, where the pumiceous layers, depicted as white lenses, are not that many. Unfortunately, there are no reliable methods developed to date to quantify the pumice contents of soil deposits, so the authors cannot provide exact values except for visual inspection. As such, the current empirical methods developed for hard-grained sands seem to work well for these two sites.

4 CONCLUDING REMARKS

In order to investigate whether existing empirical field-based methods to evaluate liquefaction potential of sands, which were originally developed for hard-grained soils, are applicable to crushable pumice deposits, two sites were selected where liquefaction had been observed following the Edgecumbe earthquake. Manifestations of soil liquefaction, such as sand boils and

Figure 7: Samples of soils taken at site #1. White-coloured layers are pumiceous, while dark-coloured ones are hard-grained sands.

ejected materials, have been reported at both sites. Field tests, including cone penetration tests (CPT), shear-wave velocity profiling through seismic CPT, and screw driving sounding (SDS) tests were performed at the sites. Then, considering estimated peak ground accelerations (PGAs) at the site based on recorded motions following the 1987 earthquake and possible range of ground water table locations, liquefaction evaluation was conducted at the sites using available empirical chart-based approaches.

Using empirical methods, site #1 showed that all methods considered would predict liquefaction of pumice at depth ranging from the location of the water table up to the maximum depth. Similar results were obtained site #2. These results were consistent with the observed manifestation of liquefaction at the sites. Subsequent sampling showed that both sites have less pumice contents and, considering the large *CSR* at the sites, the current empirical methods appeared to predict well the liquefaction triggering at these deposits. Another reason may be the significant PGAs estimated at both sites.

From the results of this research, future work will focus on comparison of the liquefaction resistance of pumiceous sites in the Waikato region obtained in the laboratory (through undrained cyclic tests on high-quality samples) with those estimated from field-based parameters.

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