6th International Conference on Earthquake Geotechnical Engineering
1-4 November 2015
Christchurch, New Zealand

Liquefaction Characteristics of Christchurch Silty Soils: Gainsborough Reserve

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ABSTRACT

The widespread liquefaction and the associated damage which occurred during the 2010 and 2011 Canterbury earthquakes has been well documented. At a number of sites throughout Christchurch, the ground performance was significantly better than what would be predicted using state of practice methods. This paper describes fieldwork and subsequent laboratory testing which was carried out at one such site and presents site characterisation information as well as preliminary results from the laboratory testing programme for liquefaction characteristics of silty soils in Christchurch.

Introduction

The Canterbury Earthquake Sequence of 2010-2011 caused widespread and repeated liquefaction in an urban setting resulting in extreme disruption to Christchurch, and the Government's offer to purchase certain parts of the city now termed the "red-zone". Despite the widespread extent of liquefaction throughout the city, there remained large areas in which heavy ground damage would be predicted using State of Practice (SOP) techniques, yet minimal to no ground damage was observed throughout the sequence of earthquakes. The inability to predict the lack of damaging liquefaction at these sites may be caused by a number of factors which includes the conservatism built into the methods, or aspects of these specific Christchurch soils which are not being adequately accounted for.

To address this issue, a research effort has begun, which will involve field sampling, "advanced" laboratory testing and numerical simulation in order to better understand the features not being captured by the SOP methods. This paper presents some of the initial results from this study to investigate if the observed behaviour at one site can be explained through the results of cyclic testing on undisturbed soil specimens. More detailed information concerning the fieldwork and laboratory testing, including descriptions of individual specimens and test data will be made available in Stringer et al. (2015).

Gainsborough Reserve

The fieldwork described in this paper took place at Gainsborough Reserve, a park in the suburb of Hoon Hay, about 5km to the South West of the Christchurch CBD, as shown in Figure 1. The work at Gainsborough Reserve is part of a larger project involving 9 sites in

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(a) Site location within Christchurch area (b) Aerial view of sampling location

Figure 1: Location of Gainsborough Reserve.

Christchurch which are shown in Figure 1 with red markers, but not discussed further in this paper. Results obtained from a second site, 85 Riccarton Rd are presented by Beyzaei et al. (2015). During the Canterbury Earthquake Sequence, no obvious surface manifestations of liquefaction were observed at Gainsborough Reserve, and the surrounding area was classed as "minor liquefaction damage" during the post-earthquake surveys (CGD 2015a).

The soils in the South-West corner of Gainsborough Reserve were characterised by a number of methods before sampling by CPT, borehole samples to 20m, cross-hole measurements of shear wave velocities and seismic dilatometer. In addition to providing a visual inspection of the soils, a number of samples from the borehole were selected for further analysis of fines content (FC) and plasticity index (PI). Three monitoring wells were installed very near to the site after the earthquakes, and the data suggests that the groundwater level at the time of the February earthquake is likely to have been between 0.8 and 1m below ground level. P-wave data (CGD 2015b) shows that full saturation (V_p approx. 1500ms^{-1}) at this site is first reached at about 2.2m below ground level, but at 2.6m below the ground surface, the p-wave velocity drops significantly, reaching a minimum of 600ms^{-1} at approximately 3.5m. Full saturation is again reached at 4.4m below the ground surface.

Selected information from these characterisation activities are shown in Figure 2 and the data suggests that the soil profile can largely be split into layers of two different types – clayey soils and silty sands. It should be noted that the distributions of fines content (FC) and plasticity index (PI) shown in the figure contain values from both the initial characterisation (blue "x"), and those which were obtained from specimens which had previously been tested in the laboratory triaxial device (red "+"). Fines content has been defined as the proportion of soil finer than 0.075mm, and was determined by laser diffraction for specimens obtained by undisturbed sampling. It should be noted that the particle size distributions for a number of specimens were compared with results from sieve and hydrometer methods and the PSDs by laser diffraction were typically found to be coarser.

In New Zealand, simplified methodologies are commonly used to predict the likelihood of liquefaction at a given site. When the CPT information obtained at Gainsborough Reserve is tested against the simplified method of Boulanger and Idriss (2014), large proportions of the soil profile would be expected to "liquefy" or experience cyclic softening during the February 2011 earthquake, as shown in Figure 2(e). For this analysis, the earthquake moment magnitude was 6.2, and the peak ground acceleration (PGA) experienced at the site was taken

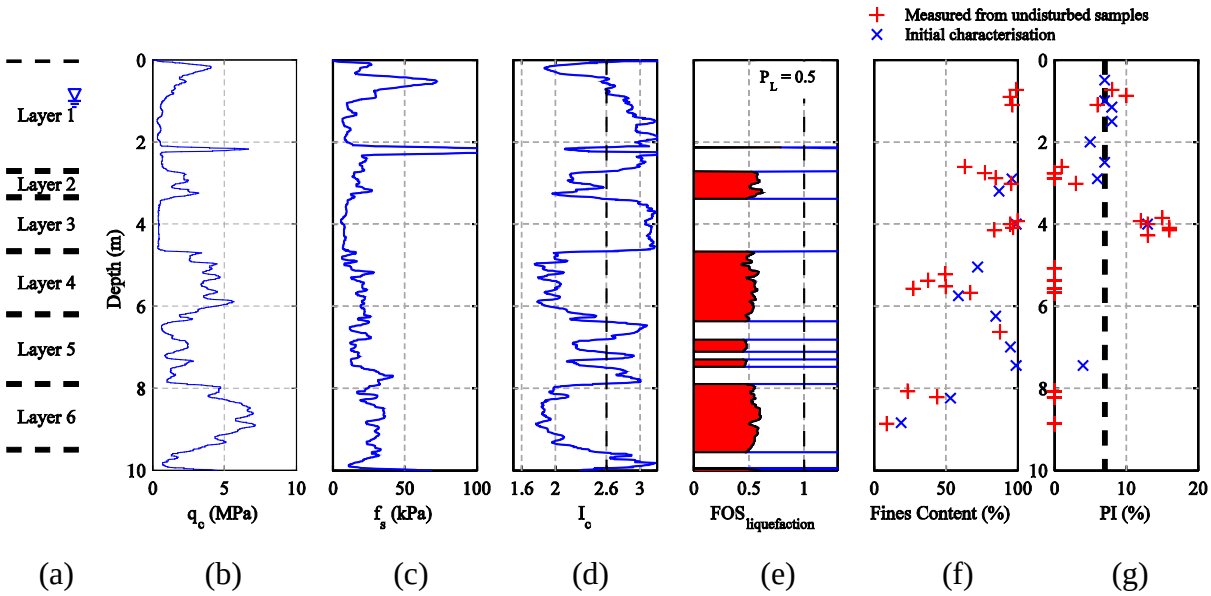


Figure 2: Site properties.

as 0.34g, based on the contours of ground motion estimated by O'Rourke et al. (2012). Bradley & Hughes (2012) estimated significantly higher PGA (approx. 0.43g) for this site and if this PGA was used in the analysis, the factor of safety against liquefaction triggering would be reduced significantly.

In the simplified method of Idriss and Boulanger (2008) a distinction is made that soils with a PI up to 4 are likely to exhibit "sand-like" liquefaction behaviour, those with a PI greater than 7 (shown as a dashed line in Figure 2(g)) might exhibit "clay-like" cyclic softening behaviour, and "transitional behaviour" observed for soils with PI in the range 4 to 7. The distribution of PI suggests that the soils in Layers 2, 4 and 6 may be susceptible to liquefaction, while the soils in the other layers would be expected to display cyclic softening behaviour.

The simplified method of Boulanger and Idriss (2014) incorporates a probabilistic framework for estimating the factor of safety against liquefaction triggering and for design purposes, the 15th percentile is commonly used to obtain a conservative estimate of liquefaction triggering. In this study, the actual response of the site is being compared to the predictions, so the curve corresponding to the 50th percentile is shown in Figure 2(e). At this level of probability, the factors of safety against liquefaction triggering range between 0.45 and 0.6.

The discrepancy between the simplified analysis and the actual observations could be due to many factors. The demand of the soils in critical layers may not be as high as expected, for example due to early softening of deeper layers. Given that the cyclic resistance has been based on the 50th percentile, the actual resistances of the soils may be higher than that predicted. The clayey/silty layer between the ground surface and the first sandy layer (and interbedded between deeper sandy layers) may be sufficiently robust to prevent surface manifestations from being observed.

Sampling at Gainsborough Reserve

A number of "undisturbed" soil samples were obtained using both "Gel-Push" (GP) and Dames & Moore (DM) samplers. Boreholes were located approximately 2-3 meters from the holes made during the initial site characterisation. When carrying out gel-push sampling, a cutting shoe is advanced into the soil and the sample is retained within an inner plastic core-liner barrel. Soil samples are protected from side-wall friction by coating the sample with a lubricating polymer as it passes through the cutting shoe, and a core catcher is used to retain the soil as the sampler is lifted from the hole. Successful GP sampling yields soil samples which are 92cm in length, and 70mm in diameter. The DM hydraulic fixed-piston sampler advances thin-walled brass tubes (to reduce sidewall friction) with a bevelled cutting edge, directly into the ground. DM soil samples are typically 45cm long and 61mm in diameter.

The DM sample recovery was typically very good at this site with recoveries mostly greater than 85% of the theoretical maximum except one sample which was noted to be quite sandy. Technical issues with the GP sampler (a missing pressure seal) affected a number of the samples, but 4 samples with greater than 75% recovery (considered sufficient to warrant triaxial testing) were obtained.

Samples were carefully transported by car to the laboratory at the University of Canterbury, where they were stored vertically until extruded for testing. Beyzaei et al. (2015) describes procedures for handling DM specimens, while Taylor et al. (2012) describes the processes for handling the GP specimens.

Advanced Laboratory Testing

Cyclic triaxial testing was performed on soil specimens using two separate triaxial test devices. To remove the outer layers of soil which may have absorbed the lubricating gel, GP specimens were trimmed to 50mm diameter before testing in a Seiken triaxial device. DM specimens were tested at their nominal diameter of 61mm on a CKC triaxial device. Specimens were isotropically consolidated to 110% of the estimated in-situ vertical effective stresses, assuming a unit weight of 18kN/m^3, to ensure that specimens are both close to in-situ stresses and in a normally consolidated stress state at the time of testing. Cyclic loading was applied at a nominal loading frequency of either 0.05Hz or 0.1Hz.

A particular difficulty of sampling silty and sandy soils is the disturbance which can occur between sampling and the point of testing. If severe, the results obtained from a testing programme can be significantly altered. In order to establish the level of disturbance during sampling, the shear wave velocity of specimens in the field were compared with those measured in the laboratory on consolidated specimens. At the time of testing, triaxial platens with bender elements were available for testing with 50mm diameter specimens only, hence shear wave velocity measurements were obtained for GP specimens tested. Shear wave velocities in the field (cross-hole measurements; CGD 2015b) and lab (bender elements) were corrected to a mean reference stress level of 100kPa, and lab measurements were conducted with excitation frequencies between 2kHz and 8kHz. The comparison of shear wave velocities are shown in Figure 3 and in the majority of cases appear to indicate that the disturbance to the specimens may not be severe. However, the shear wave velocity measured on the shallowest specimen, taken close to the water table, is significantly lower than the field measurement. This sample had good recovery (89%) but a lot of horizontal cracking was visible on the specimen when it was extruded.

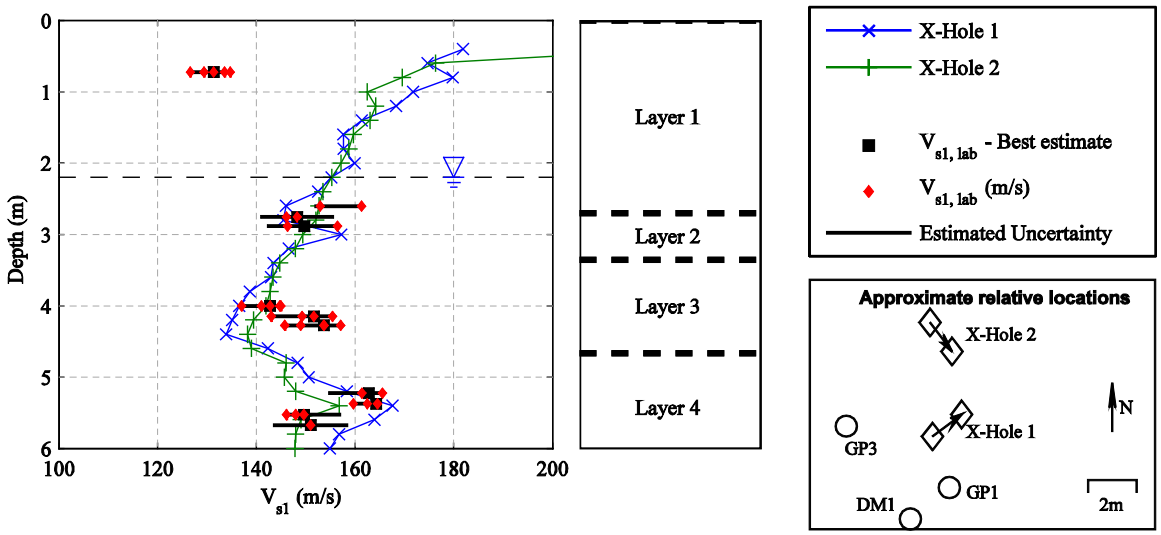
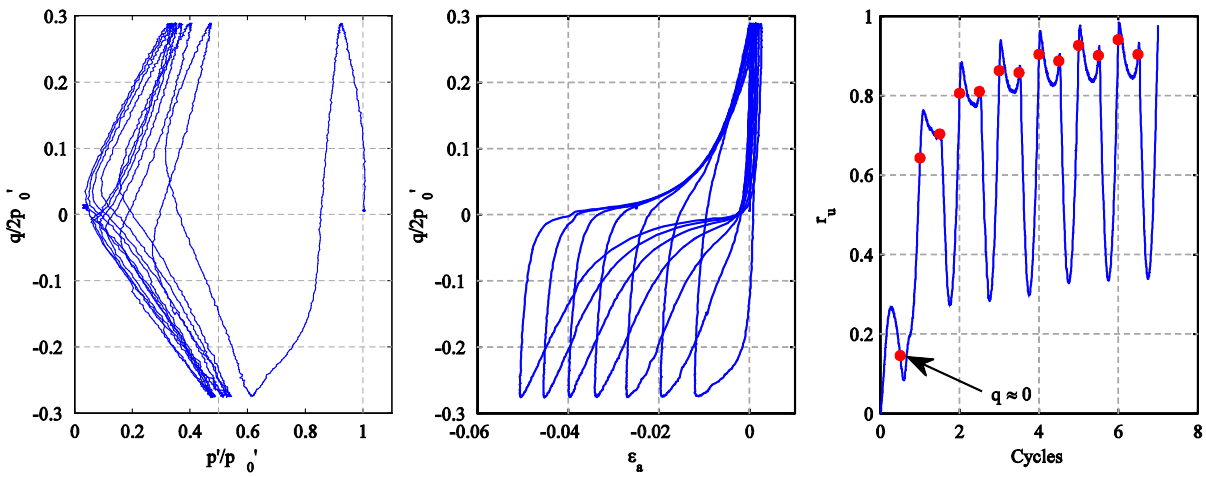


Figure 3: Comparison of corrected shear wave velocities measured in the field and laboratory.

Typical results from one of the tests (from Layer 4) are shown in Figure 4 to illustrate some of the key features observed in most tests: strains developed gradually, even after large pore pressures had been generated, and the development of axial strains was highly biased towards the extensional loading side. It should be noted that the pore water pressures measured during cyclic testing on samples from layers 2 and 3 were thought to be unrepresentative of the whole specimen due to a lower hydraulic conductivity in these soils.

Using 5 percent double amplitude axial strain, curves of cyclic resistance have been estimated for a number of different soil layers where three or more data points were obtained. These results are shown in Figure 5, split into soils which are likely to exhibit strain softening (5a) and those which are potentially liquefiable (5b). Common soil parameters are summarised in Table 1. It should be noted that the soils in Layer 2 have I_c values in the range of 2.0-2.6, which is slightly higher than those in Layer 4. These soils are also noted to have finer grain size distributions and were observed to be inhomogeneous, with clear lenses and pockets of finer/coarser material.



(a) (b) (c)

Figure 4: Test results from S2-GP1-4U C (Layer 4).

Table 1: Specimen properties by layer.

Layer	q_{c1}	I_c	PI (%)	$FC_{75\mu m}$ (%)	D_{50} (mm)	$e_{consolidated}$
1	10-29	2.13-2.90	5	63-99	0.012-0.049	0.69-0.87
2	15-35	2.08-2.61	NP-3	77-85	0.025-0.04	0.74-0.80
3	5-6	3.04-3.13	12-17	84-100	0.008-0.013	0.82-1.01
4	38-53	1.84-2.13	NP-10	27-67	0.059-0.106	0.75-0.89
6	38-67	1.85-2.23	NP	9-44	0.082-0.189	0.71-0.79

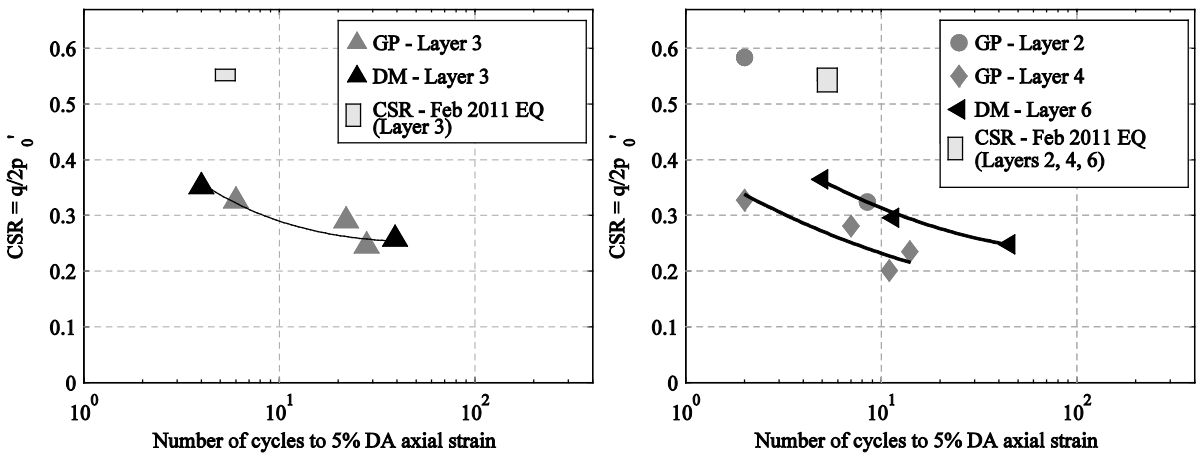
Examination of the data presented in Figure 5 reveals a number of interesting features. In Layer 3 the individual values of I_c, normalised cone resistance (q_{c1}), PI, FC and average grain size (D50) are very similar so direct comparisons of sampler performance can be made. The data from this layer is shown in Figure 5a, and suggests that the points from the GP and DM samplers would fall on coincident curves of cyclic resistance. Further, as shown in Figure 3, the shear wave velocity measured in the laboratory for specimens obtained by Gel-Push compares favourably with those obtained by cross-hole measurements in the field. This suggests that the specimens from this layer can be considered of good quality or "undisturbed" for both samplers.

Similar comparisons of the cyclic resistance relationships from the sandier soils obtained with the different samplers were anticipated. However, the difficulties encountered during both the sampling and the testing programme meant that cyclic triaxial tests on comparable silty sand specimens (i.e. from similar depths in the soil profile) are not available for this site.

While insufficient data points existed from Layer 2 to define its own CRR curve, it appears that the relationship would be significantly steeper (or potentially higher) than those from Layers 4 and 6, and that the resistance is much higher at low numbers of cycles. Additional scrutiny revealed that the loops of cyclic mobility in a q-p' plot (noting that pore pressure measurements were poor in these tests) showed that the shape of the effective stress path and residual effective stresses that are not typical for liquefiable sandy soils. While the q-ε_a plots showed similar degradation of stiffness at low deviatoric stress, the previous two observations point away from these soils displaying classic liquefaction behaviour. It is possible that the interbedding of the soils within this layer, and the finer grained nature of the soil may play an important role in the response, with any "liquefaction" being constrained to very thin lenses within the layer. Additionally, the saturation in this layer was noted to drop below 100%. The effect of the partial saturation on these silty soils is not well quantified, though studies on clean sands (e.g. Tsukamoto et al. 2002) have indicated that the cyclic resistance would be expected to increase in these circumstances.

The shaded boxes in Figure 5 indicate the estimated demand during the February 2011 earthquake, corrected for the isotropic stress conditions and 1-D loading during the triaxial testing. While the estimated demands are much greater than the strengths of the soils in Layers 3, 4 and 6, it is likely that the cyclic resistance of Layer 2 is close to the applied demands, especially if the effects of partial saturation are considered.

The lack of visible expressions of liquefaction at Site 2 may be explained by a number of different interpretations. The first is that the cyclic strength of the soils in Layer 2 was exceeded during the February 2011 earthquake, but the behaviour of these soils was closer to softening than "liquefaction". Alternatively the increased cyclic resistance of the soil in Layer 2 as a result of partial saturation was sufficient to resist development of 'damaging' liquefaction. Finally, even though liquefaction may have been triggered in the deeper layers



(a) High PI soil (cyclic softening) (b) Low PI soil (liquefaction)

Figure 5: Cyclic resistance of soils by layer.

(Layer 4), the non-liquefied crust in the top 4.5 m would have supressed liquefaction manifestation on the ground surface. Additional effects may have arisen from the interaction between different layers in the seismic response, including reduced shear stresses due to deeper liquefaction or softening, which may have reduced further generation of excess pore water pressures.

In each of these interpretations, the "critical layer" for surface manifestations of liquefaction becomes Layer 4, which is approximately 4.5m below the ground surface. While it is expected that liquefaction would be triggered in this layer (or a deeper one), it is likely that the thickness of overlying soil was sufficient to prevent liquefied soil from reaching the surface at Gainsborough Reserve. It may also be the case that a mixture of these interpretations was responsible for the better than anticipated site response based on simplified triggering analysis, and the authors aim to confirm these hypotheses in the future through effective stress analyses.

Conclusions

- V_s measurements of the soils obtained at Gainsborough Reserve suggest that a wide range of soil types were successfully sampled by the Gel-Push methodology.
- Very similar cyclic resistance relationships were obtained in a high I_c soil using the Gel-Push and Dames & Moore samplers.
- Liquefaction of the deeper sandier layers would be expected on the basis of the triaxial testing and estimated demand during the February 2011 earthquake.
- The shallowest potentially "liquefiable" layer was relatively fine grained and displayed key differences in the behaviour, which included a residual effective stress being maintained during cyclic loading, and suggested that the soil may not be susceptible to classic sand-type liquefaction.
- The cyclic resistance of the shallowest potentially liquefiable layer is expected to be larger than the laboratory results suggest due to partial saturation of the layer.
- The lack of visible liquefaction evidence during the February 2011 earthquake may be a result of the greater depth to the first significant zone of liquefied soil.

Acknowledgments

This work was funded jointly by the Ministry for Business, Innovation and Environment and the National Science Foundation (under grants CMMI-1407364, EAPSI-1414671, and CMMI-0825734). Additional funding was provided by Building Research NZ (BRANZ), the Earthquake Commission and the Pacific Earthquake Engineering Research Center. Any findings or conclusions in this work do not necessarily reflect the views of the sponsoring bodies. The authors wish to make particular thanks to Christchurch City Council for access to Gainsborough Reserve, Mr Iain Haycock and McMillan Drilling for the drilling and sampling work, the University of Texas at Austin research team led by Profs. Ken Stokoe and Brady Cox for P and S wave velocity measurements, Ms Nicole Van de Weerd and Mr Jonathan Doak for carrying out conventional and triaxial testing, and the technical and administrative staff of the University of Canterbury.

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