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Liquefaction Resistance of Silty Soils at the Riccarton Road Site, Christchurch, New Zealand

C.Z. Beyzaei[1], J.D. Bray[2], M. Cubrinovski [3], M. Riemer[4], M.E. Stringer [5], M. Jacka[6], and F.J. Wentz[7]

ABSTRACT

During field studies following the 2010-2011 Canterbury Earthquake Sequence, investigators observed negligible or minimal liquefaction effects at many predominantly silty soil sites in Christchurch, despite cone penetration test (CPT)-based state-of-the-practice methods indicating that liquefaction would occur at these sites during the Christchurch earthquake. Several silty soil sites were selected for investigation based on discrepancies between field observations and liquefaction triggering calculations. A comprehensive field sampling and laboratory testing program was carried out at each site to evaluate liquefaction resistance of silty soils and the potential levels of conservatism in current liquefaction triggering procedures. This paper presents the case history for one such site along Riccarton Road. Subsurface conditions in the top 10 m at this site consist of shallow surficial fill, underlain by silts and silty fine sands. “Undisturbed” soil samples were obtained using a Dames & Moore hydraulic fixed-piston sampler and cyclic triaxial testing was conducted at the University of Canterbury using a CKC testing device. Results of these tests and insights regarding the cyclic response of silty soils are shared in this paper.

Introduction and Motivation

During the 2010-2011 Canterbury Earthquake Sequence, multiple earthquake events triggered widespread damaging liquefaction throughout Christchurch, New Zealand. This degree of extensive repeated liquefaction was virtually unprecedented in a modern urban setting. Several earthquakes damaged buildings, infrastructure networks, and critical lifeline systems. However, there were also countless examples of cases where soil deposits previously thought to be potentially liquefiable did not express surface manifestations of liquefaction. For some sites, especially sites with silty soils, conventional cone penetration test (CPT)-based state-of-the-practice methods over-estimated the occurrence and severity of liquefaction. Current liquefaction triggering procedures are largely based on observations following earthquakes at sites containing deposits of relatively clean sands. There remains considerable debate around liquefaction resistance of fine-grained soils, such as silts, including how liquefaction of silty soils might manifest damage, and the appropriate assessment procedure to employ. This paper presents a

[1]Ph.D. Researcher, Dept. of Civil & Environmental Engineering, UC Berkeley, USA, zbeyzaei@berkeley.edu
[2]Professor, Dept. of Civil & Environmental Engineering, UC Berkeley, USA, jonbray@berkeley.edu
[3]Professor, Civil Engineering, U. of Canterbury, Christchurch, New Zealand, misko.cubrinovski@canterbury.ac.nz
[4]Adjunct Professor, Dept. of Civil & Environmental Engineering, UC Berkeley, USA, riemer@ce.berkeley.edu
[5]Post-Doctoral Fellow, Civil Engineering, U. of Canterbury, Christchurch, NZ, mark.stringer@canterbury.ac.nz
[6]Geotechnical Engineer, Tonkin & Taylor, Ltd., Christchurch, New Zealand, MJacka@tonkin.co.nz
[7]Principal, Wentz-Pacific Ltd., Napier, New Zealand, rwentz@wp-geo.co.nz

case history for a silty soil site along Riccarton Road, herein called Site 23. The results of a laboratory testing program will be compared to the results of CPT-based liquefaction triggering procedures to identify consistencies and discrepancies between the two approaches.

Site 23 – Riccarton Road

Site 23 is a privately-owned commercial lot, located in the Riccarton suburb of Christchurch (Figure 1), approximately 2 km west of the Central Business District, in a block along Riccarton Road. A commercial structure currently occupies the northern half of Site 23, while a parking lot occupies the southern half; the remaining area of the block is comprised of low-rise commercial structures and parking lots.

During the post-earthquake field observations in 2010 and 2011, investigators noted no surface manifestations of liquefaction at Site 23; however, current state-of-the-practice liquefaction triggering procedures indicate that significant liquefaction should have occurred at Site 23 when strongly shaken by the 4 September 2010 Darfield and 22 February 2011 Christchurch earthquakes. The poor comparison of post-earthquake liquefaction observations and state-of-practice liquefaction triggering assessments provided the motivation for selecting Site 23 for detailed investigations and will be discussed in the following sections.

Figure 1. Site location (-43.530°, 172.604°) in greater Christchurch (from Google Earth^{TM})

Subsurface Characterization

Tonkin & Taylor performed a preliminary field investigation at Site 23 from December 2013 to February 2014. One exploratory CPT sounding (CPT_36420), one crosshole seismic test (VsVp_38170), and one sonic boring (BH_38195) were completed. Crosshole seismic velocity testing was performed by a team from The University of Texas at Austin led by Professors Ken Stokoe and Brady Cox. Additional CPT data were available from a previous investigation (CPT_26958, 27469, 27471, and 27473) and depict a consistent subsurface profile across the site. The site investigation layout is presented in Figure 2. All subsurface characterization data are available in the Canterbury Geotechnical Database (CGD).

Subsurface conditions in the top 10 m at Site 23 consist of shallow surficial fill, underlain by silts and silty fine sands. Figure 3 presents profiles for CPT tip resistance (q_c), sleeve resistance (f_s), soil behavior type index (I_c), crosshole seismic measured shear wave velocity (V_s) and compression wave velocity (V_p), and a simplified profile based on the sonic boring. Current groundwater conditions were established based on piezometer readings and the transition depth from moist to wet soil in the sonic boring core boxes, ranging from approximately 1.5 m to 2 m below ground surface (BGS), with 1.8 m BGS used for effective stress calculations; depth to groundwater during each earthquake is from the CGD. Following examination of the sonic boring samples, several soil samples were tested by Geotechnics Ltd. to obtain particle size distributions, fines contents (FC), and plasticity indices (PI) at various depths. These data helped guide development of the high-quality sampling plans and provide useful site characterization data.

Figure 2. Subsurface investigation layout, modified from CGD (from Google Earth™)

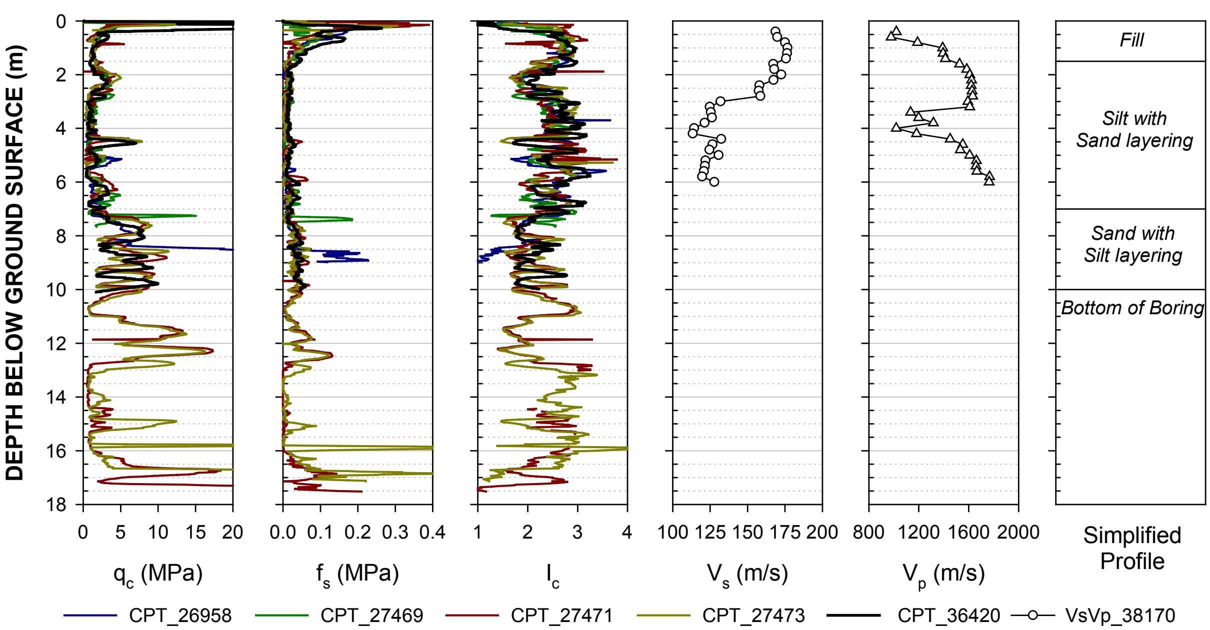

Figure 3. Subsurface Investigation Summary (CPT, Crosshole Seismic, and Sonic Boring Data)

Seismic Performance

One of the goals of this research is to obtain silty soil liquefaction resistance data that is relevant for practicing engineers and applicable to typical design projects. The majority of structures in the greater Christchurch area are residential and low-rise commercial structures whose design will be governed by state-of-the-practice methods. Considering this, liquefaction triggering was assessed using state-of-the-practice methods for comparison with post-earthquake liquefaction observations.

The Boulanger & Idriss (2014) CPT method was used to evaluate liquefaction triggering for Site 23. As presented in Figure 4, a significant portion of the profile is susceptible to liquefaction based on this state-of-the-practice method, but in both post-earthquake investigations and later examination of post-earthquake liquefaction observation maps publically available from the CGD, surface evidence of liquefaction was not observed at Site 23.

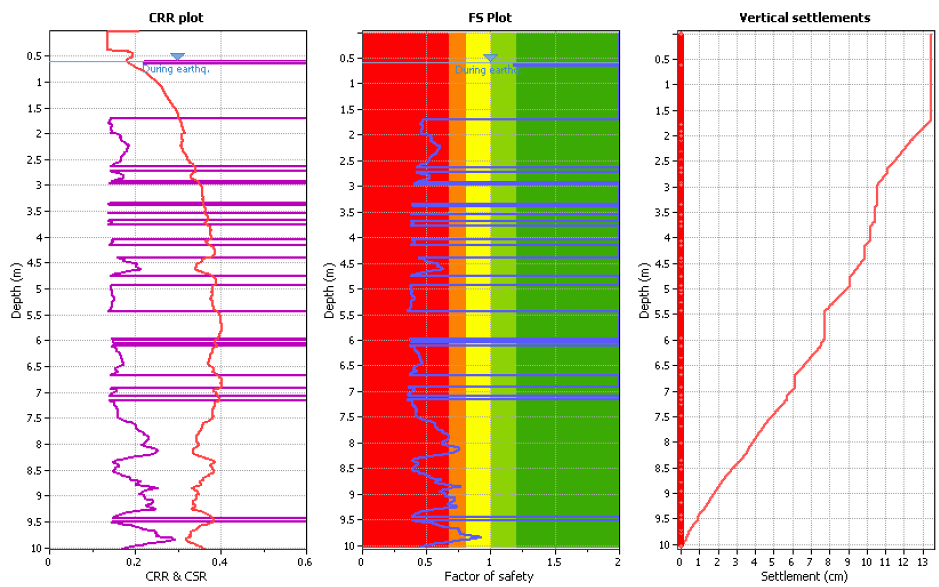

Figure 4. State-of-the-practice liquefaction triggering assessment for the 22 February 2011 event: PGA = 0.37 g, GWT = 0.6 m BGS, P_L=50%, LPI = 19, CPT_36420 (plots exported from CLiq)

Advanced Investigation

The current research was conducted during June and July 2014, with an advanced site investigation at Site 23 consisting of a) high-quality field sampling, b) conventional laboratory testing, and c) cyclic triaxial testing.

High-Quality Field Sampling

Fieldwork to retrieve high-quality ("undisturbed") soil samples was performed by McMillan Drilling using a cased mud-rotary wash boring advanced with a side-discharge tri-cone bit on 21 June 2014 under the direction of the lead author. Sampling plans were developed based on data from the exploratory CPT sounding at the site. The mud-rotary boring (DM-1) was located approximately 2.5 m from the exploratory CPT sounding (see Figure 2). High-quality samples were obtained in the exploratory boring utilizing a Dames & Moore (D&M) hydraulic fixed-piston sampling device with thin-walled brass sample tubes of constant inner diameter, ID = 61.2 mm, and outer diameter, OD = 63.5 mm. The cutting edge of the sample tubes is beveled at about 60° with the bevel on the outside. The smooth-finished brass tubes provide minimal friction between the soil and the tube during sampling. The area ratio, defined by Hvorslev (1949) as $C_a = 100*(OD^2-ID^2)/ID^2$, was 7.6% for the D&M brass tubes. The tubes were advanced into the soil a length of 45 cm, when possible. Following sampling, the tubes were sealed, placed upright, and then carefully transported by car from the site to the geotechnical laboratory at the Univ. of Canterbury (UC) for testing. Eight samples were obtained with the D&M sampler below the groundwater table; the retrieved soil samples did not show visual evidence of disturbance and had sample recoveries of 98% to 102%.

Conventional Laboratory Testing

Conventional laboratory testing was conducted by the UC staff to evaluate the physical characteristics of soil specimens used for advanced laboratory testing. Particle size analysis, Atterberg Limits, and specific gravity tests were conducted in accordance with NZ and US standards, as listed in the references section.

Table 1. Physical characteristics of Site 23 specimens

Specimen	Mid-Depth (m)	$q_{c1N, avg}$	$I_{c, avg}$	$FC_{63\mu m}$ (%)	$FC_{75\mu m}$ (%)	D10 (μm)	D50 (μm)	D60 (μm)	C_u	PI	G_s
S23-DM1-3U-A	3.39	16.6	2.66	88.0	93.3	9	33	39	4.2	5	2.72
S23-DM1-3U-B	3.55	15.1	2.71	89.6	94.0	10	32	37	3.9	7	---
S23-DM1-4U-B	4.13	21.5	2.52	92.9	95.3	7	19	26	4.0	4	2.71*
S23-DM1-5U-A	5.07	27.2	2.32	91.9	93.6	5	14	19	3.9	9	2.72
S23-DM1-7U-A	6.27	44.4	2.04	59.2	63.5	8	44	65	8.4	6	---
S23-DM1-7U-B	6.43	44.4	2.12	65.7	72.7	7	38	54	8.0	7	2.69
S23-DM1-8Ub-A	8.37	35.4	2.25	49.9	62.6	15	63	72	5.0	-4	2.67

Note: Specific gravity listed for specimen S23-DM1-4U-B is from specimen S23-DM1-4U-A which was not tested due to equipment malfunction during consolidation.

Particle size distributions for each specimen were obtained by the laser diffraction method using a Horiba LA-950 machine and assuming refractive indices for quartz. Particle size distribution parameters for fines content (FC63 and FC75), particle diameter (D10, D50, and D60), and plasticity index (PI) data are given in Table 1.

Cyclic Triaxial Testing

Advanced laboratory testing of D&M soil specimens was conducted using an updated 2014 model of the CKC triaxial testing device (Li et al. 1988). Prior to testing, the D&M sample tubes were cut at the top and bottom of each specimen, so that the specimen was extruded immediately prior to testing. Cuts were made with a manually-rotated tube cutter at the locations corresponding to the top and bottom of the specimen, with approximately 1-2 cm additional height at the top of the specimen to allow for pipe deburring. The portion of the specimen affected by deburring was trimmed with a wire saw and used to obtain an initial water content. Stiffening rings were placed above and below each cut location during tube-cutting, to prevent ovaling of the brass sample tube. To ensure a smooth cut at the top and bottom surface of each specimen, a wire saw was passed through the tube at each cut location before separating the tube portions. Specimens were extruded from the cut brass tube portion by pushing the soil out of the tube in the same direction as it entered the tube using an electric extruder. Specimens are tested as-extruded, with approximately 1-2 cm trimmed from the top of the specimen where the top cut location was made while the specimen is still in the brass tube. Nominal height and diameter of the test specimens were approximately 140 mm and 61 mm, respectively. The specimen was then transferred to the CKC triaxial testing cell and encased in a latex membrane without further trimming. Vacuum extraction was used to remove air from the specimen prior to application of cell pressure and back pressure, maintaining the initial effective stress level with a constant differential between the applied internal specimen vacuum and chamber vacuum.

Each test specimen was fully saturated under back-pressure saturation (i.e., the back pressure was raised until the B-value exceeded 0.96, except in one case) and isotropically consolidated to 1.1 times the estimated in-situ vertical effective stress before performing the cyclic triaxial test. Cyclic loading was applied at a period of T=1.0 seconds. After completion of the cyclic test, specimens were reconsolidated to measure post-liquefaction volumetric strain. Table 1 and Table 2 summarize physical characteristics and laboratory testing parameters of the specimens tested for Site 23. Consistent with expectations for shallow alluvial Christchurch soil, every specimen tested contained visible organic content, varying from trace amounts to roots and pieces of wood.

Table 2. Laboratory testing parameters for Site 23 specimens

Specimen	B_{consol}	p'_{consol} (kPa)	ρ_{consol} (kg/m^3)	e_{consol}	CSR	$N_{5\%,DA}$	$N_{3\%,SA}$	$\varepsilon_{a,max}$ (%)	$\varepsilon_{v,recon}$ (%)
S23-DM1-3U-A	0.965	51.5	1362	0.95	0.40	8	5	-7.1	2.2
S23-DM1-3U-B	0.973	52.6	1310	1.02	0.30	16	12	-6.1	2.5
S23-DM1-4U-B	0.986	58.5	1366	0.94	0.35	10	6	-8.1	2.8
S23-DM1-5U-A	0.985	67.9	1289	1.06	0.25	43	36	-7.7	3.1
S23-DM1-7U-A	0.917	78.7	1574	0.68	0.32	7	5	-8.0	3.4
S23-DM1-7U-B	0.972	80.5	1597	0.66	0.40	6	4	-8.0	3.1
S23-DM1-8Ub-A	0.967	99.4	1518	0.75	0.25	25	18	-6.9	2.6

Note: Subscript "consol" denotes value at end of consolidation, prior to CTX test. $N_{5\%, DA}$ = number of cycles to 5% double amplitude axial strain; $N_{3\%, SA}$ = number of cycles to 3% single amplitude axial strain; $\varepsilon_{a,max}$ = maximum axial strain during CTX test; $\varepsilon_{v,recon}$ = volumetric strain during post-liquefaction reconsolidation test

Liquefaction Resistance

In this study, liquefaction resistance is evaluated for each specimen by the number of cycles to reach a specified condition of liquefaction at a given cyclic stress ratio (CSR). Unique liquefaction criteria do not exist, but common definitions of liquefaction include 100% excess pore water pressure generation, 5% double amplitude strain, and 3% single amplitude strain. In these results, the number of cycles to liquefaction is presented for the 5% and 3% strain criteria to provide context on the possible variation in data interpretation. The number of cycles to 100% excess pore water pressure is not applicable for interpretation of these results given the fine-grained nature of the soils and the T=1.0 second loading period, which preclude accurate measurement of pore water pressures during testing.

Representative cyclic triaxial test results are shown for one specimen (S23-DM1-3U-A) in Figure 5. Cyclic triaxial test (CTX) data are provided for similar soil types, grouping specimens by soil behavior type index (I_c), CPT tip resistance (q_{c1N}), and depth, as well as visual classification of individual specimens. The strata for interpreting CTX results are shown in Figure 6, with results presented by soil layer. Individual specimen descriptions, photographs, and detailed test data are provided in a data report prepared by the Univ. of California, Berkeley & Univ. of Canterbury (2015).

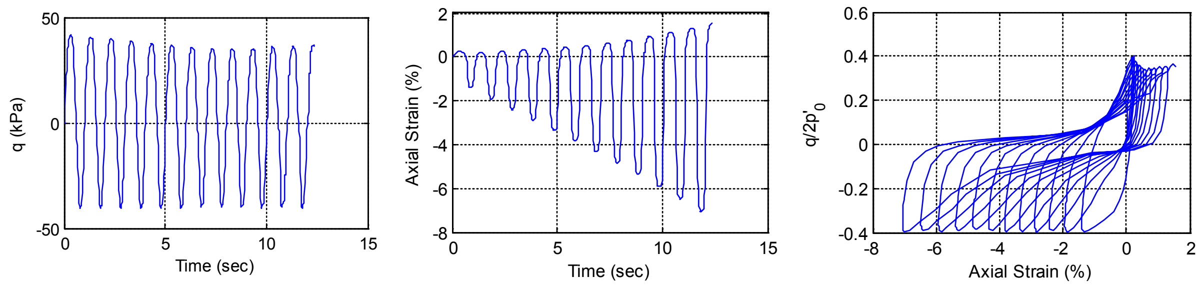

Figure 5. Representative cyclic triaxial test results (S23-DM1-3U-A)

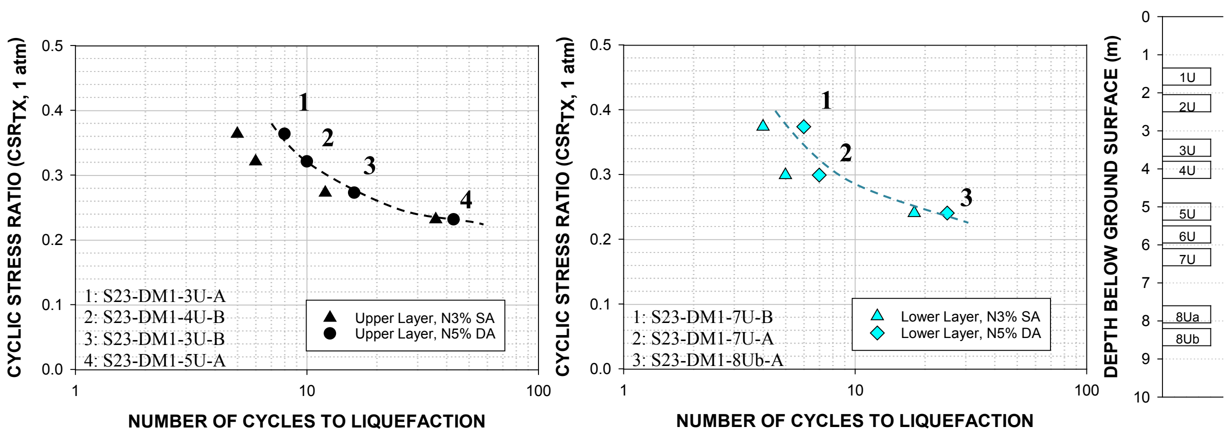

Figure 6. CTX liquefaction resistance results for Upper and Lower layers

Discussion and Conclusion

To compare the laboratory CTX test results shown in Figure 6 with the state-of-the-practice CPT analysis shown in Figure 4, the cyclic resistance ratios (CRR) are compared for an equivalent M7.5 event (i.e., 15 cycles of loading). The laboratory CTX testing curves, overburden corrected to 1 atm, indicate that CRR ≈ 0.26-0.28 for 5% double-amplitude strain at 15 cycles. Considering a CTX-to-field loading factor of 0.68, the CTX testing field-equivalent CRR for an M7.5 earthquake ($CRR_{7.5}$) is 0.18-0.19. Conversely, CPT-based procedures estimate $CRR_{7.5}$ ≈ 0.11-0.18 for the liquefiable portions of the soil profile and probability of liquefaction (P_L) range of 15% to 85%. The CTX-based estimate of $CRR_{7.5}$ is near the upper end of the P_L = 15-85% range of $CRR_{7.5}$ estimated with state-of-the-practice CPT procedures. Considering the uncertainties involved in both laboratory testing and state-of-the-practice methods, the two approaches are reasonably consistent. However, the cyclic resistance estimates from both methods remain far below the cyclic demand imposed by the 22 February 2011 event. Thus, liquefaction would be expected to have triggered in the critical layer.

There are several other factors that may have contributed to the suppression of observable liquefaction effects in this case. Site 23 is characterized by a highly stratified profile of liquefiable and non-liquefiable soils in its top 10 m, which would limit pore water pressure redistribution and the formation of sediment ejecta. Its relatively higher I_c values (i.e., $I_c > 2.05$) indicate fine sand/silt-size particle distributions and previous observations indicate subangular-shaped particles, which may have suppressed the observations of surface manifestations of liquefaction. Ongoing nonlinear effective stress analyses will provide insights regarding these and other factors.

This research provides a dataset that can be used as a reference point for further understanding the liquefaction potential of silty soils in alluvial environments such as Christchurch and a preliminary comparison of laboratory test data with CPT-based liquefaction triggering analysis. The results will be incorporated with results from additional select silty soil sites to provide guidance for liquefaction assessment of Christchurch silty soils by comparing laboratory test results with the observed liquefaction performance during the 2010-2011 Canterbury Earthquake Sequence and CPT-based liquefaction triggering procedures. Additionally, the procedures developed for field sampling and laboratory testing can be used to guide field investigations and discern the need for advanced testing on critical projects. The research is ongoing, and additional laboratory results and numerical results will be forthcoming and provide insight on the seismic response of silty soils.

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