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Liquefaction Assessments in the Central Business District of Christchurch, New Zealand

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ABSTRACT

The 2010-2011 Canterbury earthquake sequence provides an exceptional opportunity to investigate the effects of varying degrees of liquefaction on the built environment. Over 18,000 CPTs have been performed in the Christchurch area. When combined with over 4,000 exploratory soil borings and hundreds of geophysical tests, these site investigations provide an unparalleled dataset to evaluate state-of-the-practice liquefaction assessment procedures. The in situ tests performed in the Central Business District (CBD) are combined with cyclic triaxial testing (CTX) to characterize the soil deposits at selected buildings sites in Christchurch. Significant ground settlements and building damage in the CBD were observed for the Christchurch earthquake. Liquefaction-induced ground settlement procedures do not capture shear-induced deformation mechanisms and the effects of ground loss due to sediment ejecta. Improved procedures are thus required. The CTX tests estimate generally consistent cyclic resistances as the CPT-based methods for medium dense sands and silty sands. Correlations and CTX tests performed on loose clean sands indicate that these specimens were disturbed by the sampling process. Overall, the CTX test results provide the data required to calibrate soil constitutive models and perform nonlinear effective stress analyses, which are ongoing. These results will provide greater insights.

Introduction

Christchurch, New Zealand (NZ) has been affected by a series of earthquakes that include the 4 September 2010 (M_w 7.1) Darfield earthquake, 26 December 2010 (M_w 4.8) aftershock, 22 February 2011 (M_w 6.2) Christchurch earthquake, 13 June 2011 (M_w 6.0/M_w 5.3) earthquakes, and 23 December 2011 (M_w 5.9/M_w 5.8) earthquakes. The earthquake shaking from these events triggered localized-to-widespread minor-to-severe liquefaction in the Christchurch area. Details of the geotechnical effects of these earthquakes may be found in publications such as Green and Cubrinovski (2010), Cubrinovski et al. (2011a,b,c), Bray et al. (2014a,b), and van Ballegooy et al. (2014). Ground movements resulting from liquefaction-induced sediment ejecta, liquefaction-induced differential settlement, and lateral spreading damaged buildings and infrastructure in areas throughout Christchurch, including its Central Business District (CBD).

Comprehensive geotechnical site investigations were performed following the Canterbury earthquake sequence (CES). Over 18,000 cone penetration tests (CPTs), 4,000 exploratory soil borings, 1,000 shallow piezometers, and hundreds of geophysical surveys were carried out to

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characterize subsurface conditions as part of this liquefaction-induced land damage assessment (e.g., van Ballegooy et al. 2014). Recognizing the value of this extensive set of field investigations, the Canterbury Earthquake Recovery Authority (CERA) and the Ministry of Business Innovation and Employment (MBIE) established and maintain the Canterbury Geotechnical Database (CGD), which provides geotechnical and groundwater data to the public (https://canterburygeotechnicaldatabase.projectorbit.com). The CPT soundings, in conjunction with state-of-the-practice liquefaction assessment procedures, have been used as the primary tools to assess the depth of the critical layer for liquefaction triggering and to derive parameters representing liquefaction vulnerability.

The objective of this paper is to describe some of the damage from the 2010-2011 Canterbury earthquake sequence within the CBD with emphasis on the performance of multi-story buildings during the Christchurch earthquake. The important role of the CPT in characterizing the subsurface conditions and in providing data for evaluating liquefaction hazard is discussed. Cyclic triaxial testing of relatively “undisturbed” soil specimens complement the CPT data and provide useful insights. Some important findings from the ongoing research study are presented.

Subsurface Conditions and Observed Damage

The Christchurch earthquake caused 185 fatalities and many serious injuries. Much of the damage of multi-story buildings was within the CBD. Nearly half of the buildings inspected within the CBD were marked as restricted access due to potential safety issues, and most of the CBD was cordoned off for over two years after the Christchurch earthquake. A majority of the 4,000 buildings within the CBD have been demolished, including most of the city’s high-rise buildings. The seismic performance of modern multi-story buildings and their supporting buried utilities in the CBD were often significantly impacted by soil liquefaction. Study areas and liquefaction zones within the CBD are depicted in Figure 1.

The Canterbury Plains are composed of complex alluvial fans deposited by eastward-flowing rivers draining the Southern Alps and discharging into Pegasus Bay on the Pacific Coast. Christchurch lies along the eastern extent of the Canterbury Plains, just north of the Banks Peninsula, the eroded remnant of the extinct Lyttelton Volcano, comprised of weathered basalt and Pleistocene loess (Cubrinovski et al. 2011a). The city was built on a historic floodplain of the Waimakiriri River, a large braided river that is now channelized approximately 25 km north of the CBD. The Waimakiriri River regularly flooded Christchurch prior to the construction of levees and river realignment carried out shortly after the city was established in the 1850s (Brown & Weeber 1992 & Brown et al. 1995). The “Black” Maps depict several buried stream channels through the CBD, which intersect the study zones (Cubrinovski et al. 2011a). Several of these buried stream channels are indicated on Figure 2. The subsurface conditions in the CBD are highly variable with alternating layers of sands and gravels with overbank deposits of silty soils and some peat pockets, which is indicative of the ephemeral nature of floodplains.

There are three geological formations of primary interest in foundation engineering within the CBD: the Springston Formation, Christchurch Formation, and Riccarton Gravels. The Springston Formation was deposited during the last 3000 years and is the shallowest of the three formations. It consists of three lithologic units (Brown & Weeber 1992): 1) gravels deposited in old flood channels of the Waimakariri River; 2) overbank alluvial silt and sandy silt; and 3) peat deposits formed in marshland. The Christchurch Formation consists of beach, estuarine, lagoon, dune, and coastal swamp deposits composed of gravel, sand, silt, clay, shells, and peat, and its top is found at a depth of typically 7 to 10 m within the CBD. Brown & Weeber (1992) describe its age as post-glacial and likely less than 6500 years old near the maximum inland extent of the post-glacial marine transgression, which likely extended across the CBD based on the presence of shells observed in soil samples (Tonkin & Taylor 2011). The Riccarton Gravels are beneath the Christchurch Formation and consist of well-graded brown or blue-gray gravels up to cobble size. This 10 to 20 m thick formation is the uppermost confined gravel aquifer in coastal northern Canterbury and is typically about 18 to 30 m below the ground surface in the CBD (Brown & Weeber 1992 & Tonkin & Taylor 2011).

Two spring fed rivers, the Avon and Heathcote, meander through Christchurch and discharge into an estuary east of Christchurch. The Avon River, labeled in Figure 2, meanders through the CBD, while the Heathcote River flows south of the CBD. Much of the observed moderate-to-severe liquefaction within and to the east of the CBD occurred near the Avon River during the Canterbury earthquakes. The groundwater table is generally within 1 to 3 meters of the ground surface within the CBD.

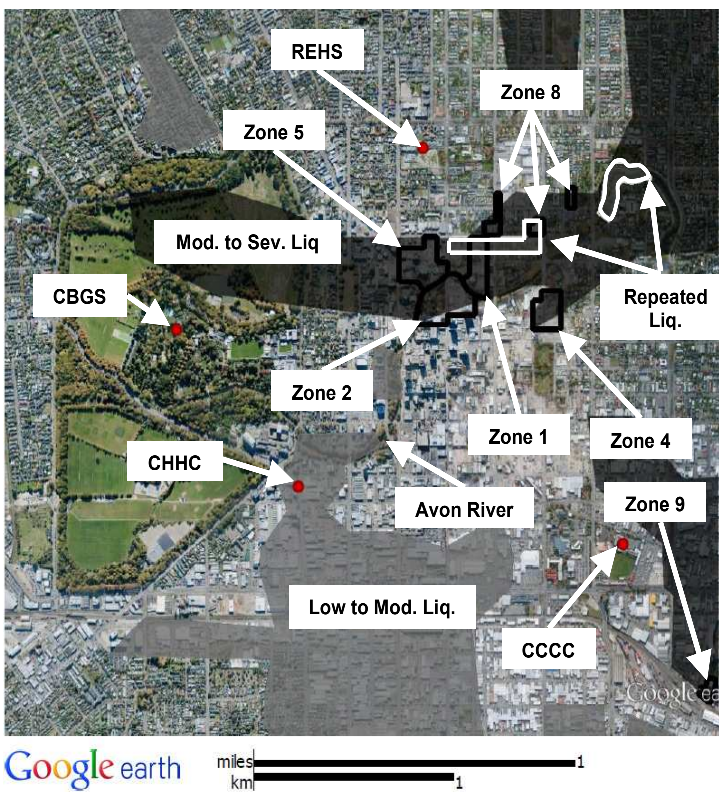

Figure 1. CBD location plan with strong ground motion recording stations and study zones. Shaded zones indicate areas with surficial evidence of soil liquefaction during the 22 FEB 11 event: dark shading indicates moderate-to-severe liquefaction and gray shading indicates low-to-moderate liquefaction (as defined by Cubrinovski et al. 2011b); white outlining indicates areas that also liquefied during the 4 SEP 10 and 13 JUN 11 events (from Bray et al. 2014a).

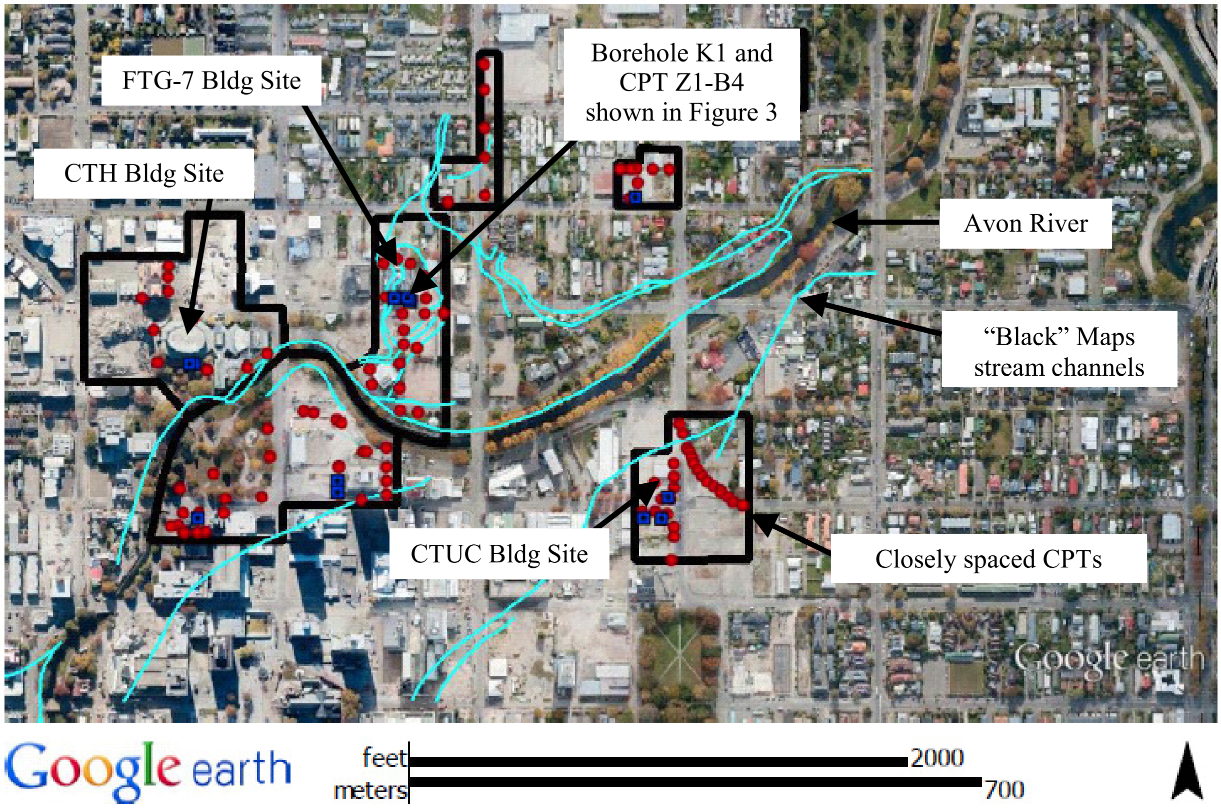

Figure 2. UCB-UC site investigation overview. CPTs are indicated with red dots, and boreholes are indicated with blue squares. Locations of buried stream channels from the “Black” Maps are shown with aqua lines. The locations of the CTUC, CTH and FTG-7 building sites, the closely spaced CPTs, borehole K1, and CPT Z1-B4 are shown for reference. Zone 9 is not shown.

Ground shaking was recorded at four strong motion stations (SMS) within the CBD during the Canterbury earthquakes, the locations of which are shown in Figure 1. The geometric mean horizontal peak ground accelerations (PGA) recorded at the stations are provided in Table 1 for five earthquakes. Median (i.e., PGA_{50}), 16% (PGA_{16}), and 84% (PGA_{84}) values of PGA estimated by Bradley & Hughes (2012) are used in this study to evaluate liquefaction triggering and its effects within the CBD. The 22 FEB 11 Christchurch M_w 6.2 earthquake produced the most intense ground shaking in the CBD, because the source-to-site distances (R) were only 3-6 km. Its PGA values were twice those recorded during the larger magnitude, but more distant (R = 18-20 km), 4 SEP 10 Darfield M_w 7.1 earthquake. The PGAs recorded in the CBD during the Darfield event are similar to those recorded during the 26 DEC 10 M_w 4.8, 13 JUN 11 M_w 6.0 (which occurred 80 minutes after a 5.3 event), and 23 DEC 11 M_w 5.9 events (which occurred 80 minutes after a 5.8 event). The PGA values of the dozens of other M_w 5+ events are lower than those recorded during these events.

The 22 FEB 11 Christchurch earthquake produced liquefaction over large areas in the CBD as shown in Figure 1. The 4 SEP 10 Darfield and 13 JUN 11 earthquakes also produced localized areas of liquefaction in some areas as also shown in Figure 1. Significant liquefaction during the 23 DEC 11 earthquake was not observed, and no liquefaction was observed in the CBD from the 26 DEC 10 earthquake.

Table 1. Recorded PGAs in the CBD during five 2010-11 Canterbury earthquake events

Date	M_w	Recorded Geometric Mean PGA (g) in CBD				Median PGA (g)	PGA_{50} (g) Bradley & Hughes (2012)
		CBGS	CCCC	CHHC	REHS		
4 SEP 10	7.1	0.17	0.21	0.18	0.25	0.20	0.22
26 DEC 10	4.8	0.25	0.22	0.16	0.24	0.21	-
22 FEB 11	6.2	0.48	0.42	0.35	0.51	0.44	0.45
13 JUN 11	6.0*	0.16	-	0.21	0.29	0.21	0.23
23 DEC 11	5.9*	0.20	0.18	0.21	0.30	0.22	0.20

* These events were preceded by smaller earthquakes that likely generated excess pore water pressure.

Site Characterization

Following the Christchurch earthquake, Tonkin & Taylor, Ltd. (T+T) organized a general site characterization study to assess the subsurface conditions within the CBD. This effort included 151 CPTs, 48 soil exploratory boreholes, 45 km of geophysical surveys, installation of piezometers, and laboratory testing of collected soil samples. With a focused aim to characterize 23 building sites within six study zones to enable detailed subsurface characterizations and simplified seismic evaluations of representative buildings within the CBD, the UC Berkeley (UCB) – Univ. of Canterbury (UC) research team performed 107 CPTs and 13 exploratory boreholes as shown in Figure 2. These structures consisted of multi-story buildings on shallow and deep foundations and displayed interesting engineering performance characteristics. This subset included buildings that performed well, in addition to buildings that were severely damaged during the Christchurch earthquake.

Although subsurface conditions within the CBD are variable (Brown & Weeber 1992, Tonkin & Taylor 2011, Taylor et al. 2012a), each building study zone has one or two representative soil profiles that reflect the subsurface conditions within the zone. Figure 3 illustrates representative subsurface conditions within Zone 1 near the FTG-7 Building with the log of borehole K1 and the adjacent CPT Z1-B4 (locations indicated on Figure 2) with its normalized cone tip resistance (q_{c1N}) and soil behavior type index (I_c) profiles calculated using the procedure described by Robertson & Wride (1998). The soil layer that lies at a depth of about 2 to 8 m has relatively low q_{c1N} values (< 50) and consists of predominantly silty sand (SM), but also sandy silt (ML), with a median fines content (FC) of approximately 37% (range of 15% to 82% with a standard deviation of 21%) based on 19 laboratory tests performed on retrieved samples. The fines were non-plastic. Below it, a denser poorly graded sand (SP) extends down to about 20 m. The base silt/clay (ML/MH) layer is the oldest and deepest material of the Christchurch Formation and is typically observed just above the Riccarton Gravels (Tonkin & Taylor 2011).

In addition to advancing CPTs adjacent to buildings, a line of 15 CPTs spaced about 10 m apart were advanced in a parking lot at the northeast corner of the intersection of Armagh and Madras streets to characterize the variability in soil conditions over relatively short distances (Bray et al. 2014a,b). At this location within Zone 4, along the southern boundary of the moderate-to-severe liquefaction zone illustrated in Figure 1, Cubrinovski et al. (2011a) observed that liquefaction was manifested by a "*well-defined, narrow zone of surface cracks, fissures, and depression of the ground surface about 50 m wide, as well as water and sand ejecta*" following the 22 FEB 11 earthquake. The shallowest layer within this narrow zone of observed liquefaction is composed of silty sand and sandy silt (SM/ ML) with a q_t generally less than 5 MPa and I_c between 2.0 and 2.5. Samples from a nearby borehole indicated a FC of about 50% for this layer. The next layer was a clean sand to gravelly sand (SP) with q_t greater than 20 MPa and often greater than 30 MPa and I_c between 1.0 and 1.5. Clean to silty sands of varying penetration resistance, but typically with q_t greater than 10 MPa followed the dense SP layer.

The buried SM/ML layer was only about 50 m wide in the parking lot. Outside of this this zone, the denser SP layer was found below the groundwater. The portion of the SM/ML layer that was below the groundwater table should have liquefied based on the median estimated PGA from Bradley & Hughes (2012) during the 22 FEB 11 earthquake using state-of-the-practice liquefaction triggering procedures, which are discussed later. The shallow liquefiable SM/ML layer, when present in the soil profile, was often the critical layer in the observed liquefaction in the CBD. As illustrated in the closely spaced CPTs performed by Bray et al. (2014a), its thickness below the groundwater table could vary considerably over relatively short distances.

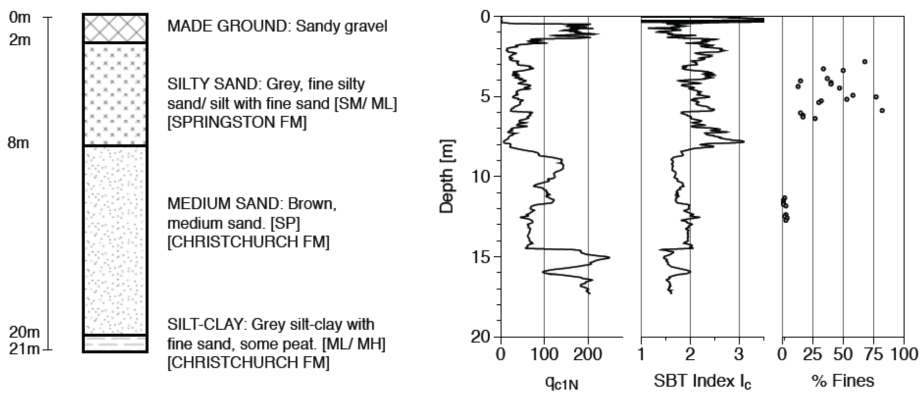

Figure 3. Log of borehole K1 and adjacent CPT Z1-B4 profile in Zone 1. The locations of borehole K1 and CPT Z1-B4 are indicated on Figure 2 (from Bray et al. 2014a).

Simplified Liquefaction Assessment Procedures

Simplified liquefaction evaluations were performed utilizing the CPT-based liquefaction triggering correlations of Robertson & Wride (1998), henceforth RW98; Moss et al. (2006), henceforth MS06; Idriss & Boulanger (2008), henceforth IB08; and Boulanger & Idriss (2014), henceforth BI14. The CPT-based procedure developed by Zhang et al. (2002), henceforth ZR02, was applied to estimate free-field, level ground settlements due to post-liquefaction volumetric strains. For the purposes of applying the ZR02 procedure, the clean sand equivalent, normalized, CPT tip resistance ($q_{c1N\text{-}CS}$) and the factor of safety for liquefaction triggering (FS_l) were calculated in accordance with RW98.

"Undisturbed" Sampling and Cyclic Triaxial Testing Procedures

High quality samples of silts and clays have been obtained previously using the Dames & Moore (D&M) hydraulic piston sampler as described by Bray and Sancio (2006). Sample quality was evaluated in this earlier study using the ratio of the change in void ratio to the soil's initial void ratio upon recompression to the in situ effective stress state criteria developed by Lunne et al. (1997) (i.e., $\Delta e/e_o < 0.04$ indicates very good to excellent specimen quality for OCR < 2). The Lunne et al. criteria were developed for plastic clays, so its applicability to low plasticity silts can be questioned, and it cannot be applied to nonplastic silty sands. In this study, three specimen quality evaluation approaches were employed: 1) drilling and sampling notes and visual inspection, 2) comparison of laboratory relative density (D_r) with estimates from CPT correlations, and 3) comparison of field measured and lab measured shear wave velocity (V_s). Lab measured cyclic resistance ratios (CRR) were also compared with those estimated using CPT liquefaction triggering correlations (e.g., BI14) to discern if the cyclic responses of the lab specimens were consistent with those expected based on established CPT-based procedures.

Block sampling or ground freezing sampling techniques have been shown to provide high quality laboratory specimens (e.g., Yoshimi et al. 1978, and Yoshimi et al. 1994). However, the use of block sampling on the soils well below the shallow groundwater table in Christchurch was impractical, and ground freezing sampling technology was not available in New Zealand. In the UCB-UC collaborative study, the D&M hydraulic piston sampler and the Gel-Push (G-P) sampler (Taylor et al. 2012b) were employed. In this paper, testing conducted on specimens retrieved in the CBD using the D&M sampler is discussed.

Soil exploratory borings were advanced using a tri-cone side-discharge drill bit in a cased borehole with drilling mud. The D&M is an Osterberg-type hydraulic piston sampler that has been shown to retrieve relatively "undisturbed" soil samples (Figure 4). The D&M sampler uses thin walled brass tubes of constant internal diameter, ID = 61.2 mm, and outside diameter, OD = 63.5 mm. The tubes are pushed into the soil a length of 45 cm using the pressure provided by the circulation mud. Friction between the soil and the tube was minimized by using smooth brass tubes. The area ratio, defined by Hvorslev (1949) as $C_a = 100\cdot(OD^2\text{-}ID^2)/ID^2$, is 7.6% for the D&M brass tubes. Following sampling, tubes were sealed, placed upright in a specially designed box, and then carefully transported to the University of Canterbury laboratory for testing.

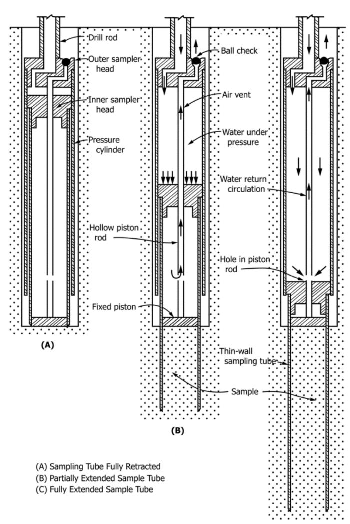

Figure 4. Schematic of hydraulic piston sampler operation using a thin-walled sampling tube (from ASTM D6519-08)

Test specimens were prepared from sample tubes with high recovery (typically ≥ 95%). The bottom 50 mm and upper 100 mm of each tube sample were assumed to be too disturbed for testing, so that a maximum of two 135-140-mm-high test specimens were obtained from each tube. Extrusion length was minimized by cutting tubes to the desired height. After installing stiffening rings on the tube above and below the location of the intended cut, a large-diameter pipe cutter was slowly rotated around the sample tube to cut it (Figure 5). The test specimen was extruded from the tube in the same direction as the soil first entered the tube using a hydraulic jack. It was visually inspected to ensure it was relatively undisturbed. Following the placement and securement of a flexible latex membrane around each specimen, an internal vacuum of 10-15 kPa was applied to each specimen to allow for set-up of the triaxial chamber. Flushing of de-aired water through each specimen was attempted using differential vacuum, which was necessary for the saturation of the coarser-grained sand specimens. Subsequent specimen saturation was achieved through vacuum saturation (extraction) followed by back pressure saturation so that B-values larger than 0.95 were achieved (most B-values were ≥ 0.97).

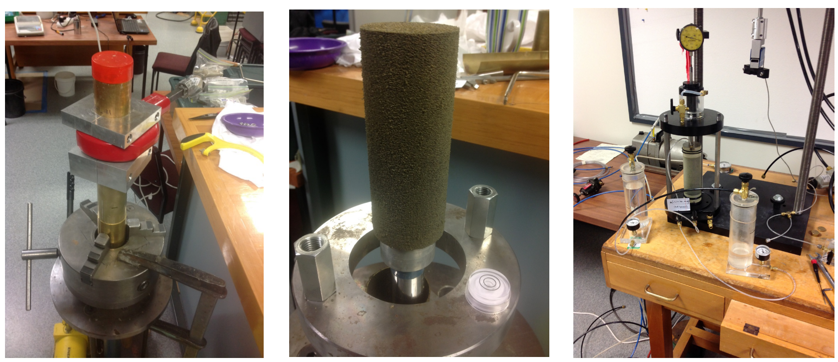

Figure 5. Specimen extrusion and set-up

Estimated field and laboratory relative density (D_r) values of test specimens were compared when FC were less than 50% to gain insights into possible volumetric strains induced during the sampling and specimen preparation process. The minimum and maximum void ratio (e_{min} and e_{max}) for soil specimens with FC < 50% were found using the Japanese Standard method (JIS A 1224:2000) so that $D_{r\text{-Lab}}$ could be calculated. $D_{r\text{-Field}}$ was estimated from various CPT-D_r correlations (e.g., Salgado et al. 1997 and Jamiolkowski et al. 2001). These correlations require the use of a normalized equivalent clean-sand penetration resistance (e.g., q_{c1Ncs}—see Boulanger and Idriss 2014), which were estimated based on the CPT data obtained near each borehole. The estimated values of $D_{r\text{-Lab}}$ tended to be higher than values of $D_{r\text{-Field}}$ for specimens with low equivalent penetration resistance (i.e., $q_{c1Ncs} < 60$). This discrepancy suggests that the sampling and specimen preparation procedures potentially densified the loose sands prior to testing.

In addition, V_s was measured in five of the laboratory specimens using bender elements. Direct field measurements of V_s were not available at the CBD sites. However, a robust Christchurch-specific CPT-V_s correlation developed by McGann et al. (2014) was used to obtain an estimate of $V_{s\text{-Field}}$. The difference between lab and field Vs values was about 10% (i.e., $V_{s\text{-Lab}}/V_{s\text{-Field}}$ values were between 1.09 and 1.11). According to the Chiara and Stokoe (2006) criteria, this indicates "medium to low" levels of sample disturbance. A more comprehensive companion study by Beyzaei et al. (2015 & personnel communication) and Stringer et al. (2015) that focused on soil samples retrieved from the suburbs of Christchurch using the same sampling and testing procedures as this study also found good agreement between $V_{s\text{-}Lab}$ and $V_{s\text{-}Field}$ measurements in samples retrieved below the water table, where $V_{s\text{-}Field}$ was measured using cross-hole seismic testing (which was completed by the University of Texas at Austin research team led by K. Stokoe and B. Cox, personnel communication).

Cyclic and monotonic triaxial testing was carried out on retrieved D&M test specimens using the CKC electropneumatic triaxial device (Li et al. 1988). Specimens were isotropically consolidated to conservative estimates of the field vertical effective stresses, which included estimates of geostatic overburden stresses, pore water pressure, and net pressure increases due to building loads. Stress-controlled cyclic triaxial tests were performed using a sinusoidal loading pattern at a frequency of 0.1 Hz. Static triaxial tests were also stress-controlled with loading rates determined so that time to failure was greater than $4*t_{50}$, where t_{50} was the time to 50% consolidation estimated from the consolidation phase.

Buildings Affected by Shallow Liquefaction

CTH Auditorium

Structure and Damage

The footprint of the auditorium structure of the CTH complex (S43.5269, S172.635) is approximately 63 m (E-W) by 47 m (N-S) (see Figure 6; Zupan 2014). It consists of a basement, ground floor, gallery, and roof. Its foundation system is an outer ring of rectangular shallow RC spread footings that are all 0.46-m thick and either 2.2 m by 3.2 m or 3.15 m by 3.2 m in plan and an inner ring of square shallow RC spread footings that are all 0.61-m thick and 3.65 m by 3.65 m in plan. The base depths of the outer ring of footings are 3.6 m, 2.7 m, or 1.9 m below the ground surface, and the base depths of the inner ring of footings are 3.8 m or 2.9 m below the ground surface. The outer footings are connected by RC tie beams that are 0.46-m wide by 0.46-m deep, and the inner footings are connected by RC tie beams that are 1.8 m wide and 0.61 m deep. Additionally, 0.91 m wide by 0.46 m deep strip footings support wall units around the outer ring at the corners. The thickness of the concrete basement slab is 0.20 m, and its base is 3.2 m below the ground surface. The ground floor slab is 0.10 - 0.13 m thick, and the gallery flooring consists of either 0.13-m thick slab concrete or precast double tee units. The ground flooring above the basement and the gallery flooring are supported by RC beams and RC columns of varying dimensions. The roof is made up of mostly 0.075 - 0.10-m thick pre-cast concrete units that support a 0.05-m thick lightweight concrete topping. It is supported by the outer ring and inner ring RC columns. Struts provide additional support between the outer column ring and inner column ring, and a series of north-south trending trusses provide additional support across the span of the inner column ring. Holmes Consulting Group (2011) estimated the natural period of the CTH building to be approximately 0.5 s in both directions.

The auditorium sustained severe structural damage during the 22 FEB 2011 Christchurch earthquake. Differential settlement of shallow foundations that supported the main load bearing columns of the structure led to angular distortion and subsequent damage to structural elements throughout the structure (Zupan 2014). Liquefaction-induced sediment ejecta was observed in many areas south of the auditorium, which is towards the Avon River. Only a small amount of liquefaction-induced sediment ejecta was observed on the north side of the auditorium. At the south side of the auditorium, column settlements relative to the floor slab varied from 2 cm to 14 cm, and the concrete slab was cracked in this area. The cracked beam spanning the walkway at the south end of the auditorium was likely damaged by differential settlement of the inner column ring relative to the outer column ring with a measured angular distortion of 1/70. Non-uniform settlements and distortions of the paving blocks were observed on the terrace to the south of the auditorium.

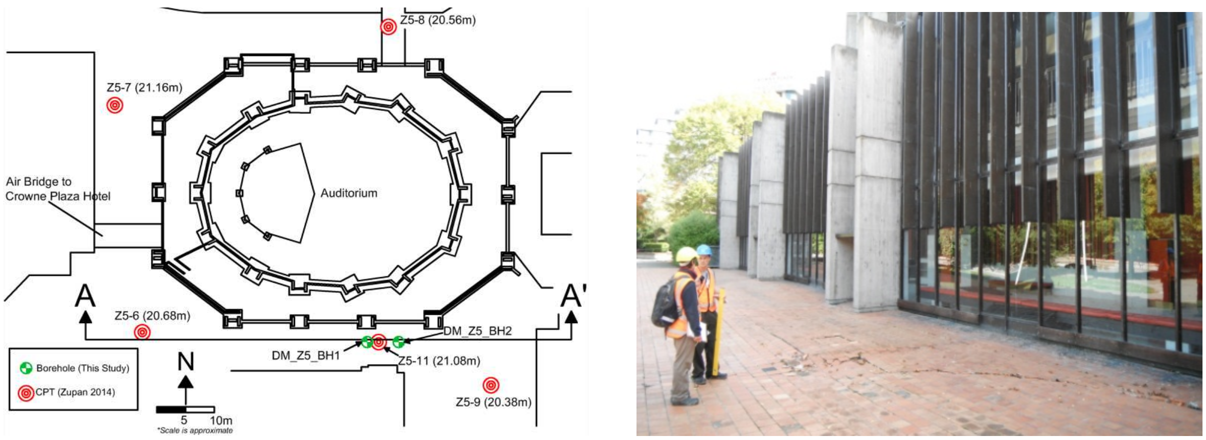

Figure 6. Plan view of CTH auditorium (modified from Zupan 2014) and view of south side

Site Characterization and Simplified Analyses

Based on the CPT data provided by Zupan (2014) as shown in Figure 7, a shallow layer of silty sand and silt (I_c is 1.75 to 2.6) is underlain by a layer of gravelly and sandy material (I_c from 1 to 1.7), and then layer of clean to slightly silty sand ($I_c < 2.05$). The shallow silty sand and silt layer and deeper clean to slightly silty sand layer generally had $FS_l < 1$ for the Christchurch earthquake level of shaking (using the RW98 procedure, Zupan 2014). The intermediate depth gravelly and sandy soil (z = 5 m to 12 m) was dense and had $FS_l > 1$. Testing of retrieved samples from the soil borings indicates that the upper portion of the subsurface comprises a layer of ML soil (to a depth of just under 4 m), which overlies a layer of SP-SM material (to a depth of about 4.5 m). Below the layer of gravelly soil is another layer of SP-SM soil. Figure 8 shows the grain size distribution plots for the CTX test specimens that were obtained at the CTH site (see Figure 7 for sample depths).

CTX Results and Findings

Two boreholes were drilled along the southern side of the main auditorium building at the CTH site to obtain “undisturbed” samples of the critical soil layers (Figs. 6 & 7). A total of 10 D&M samples were retrieved. Three CTX tests were conducted on the upper silt (specimen depths of 3.0 to 3.74 m), 2 CTX tests were conducted on the upper silty sand (specimen depths of 4.2 to 4.5 m), and 4 CTX tests were conducted on the deeper silty sand material (specimen depths of 14.2 to 15.6m). Figure 9 provides the results of a representative CTX test performed on a specimen from a depth of approximately 4.27 m (SP-SM material) at the CTH site. A strain-based liquefaction triggering criterion was used that was consistent with several previous studies (e.g., Bray and Sancio 2006). This criterion uses the number of cycles to 3% single amplitude axial strain, which typically occurred in extension.

Figure 10 provides a summary of applied cyclic stress ratio ($CSR_{CTX,1atm}$) versus the number of cycles to 3% single amplitude axial strain ($N_{c\text{-}3\%\ S.A.\ \varepsilon ax}$) for all CTX specimens tested from the CTH site. The interpreted cyclic resistance ratio (CRR) curves for these tests indicate that the upper silt is the weakest of the three layers studied, followed by the upper SP-SM material. The deeper silty sand represents the strongest of the material tested. This trend of the upper portion of the subsurface consisting of a slightly weaker, finer material overlying a stronger coarser material was also observed at the sites discussed below.

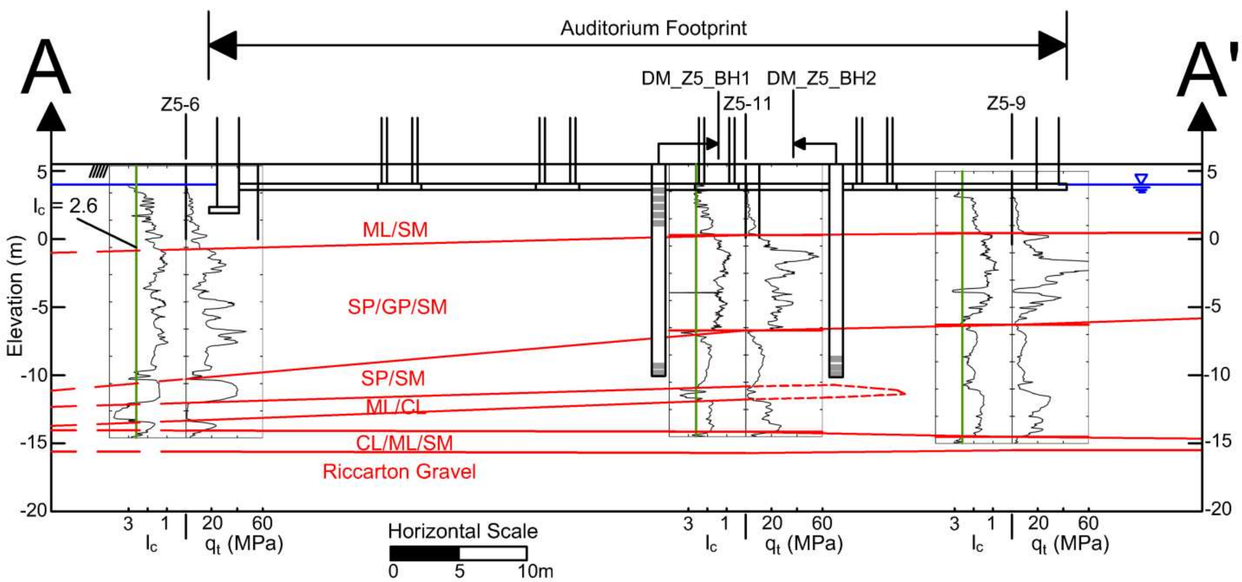

Figure 7. Representative E-W subsurface profile at the CTH site; note that “undisturbed” sample depths are shown in grey (modified from Zupan 2014)

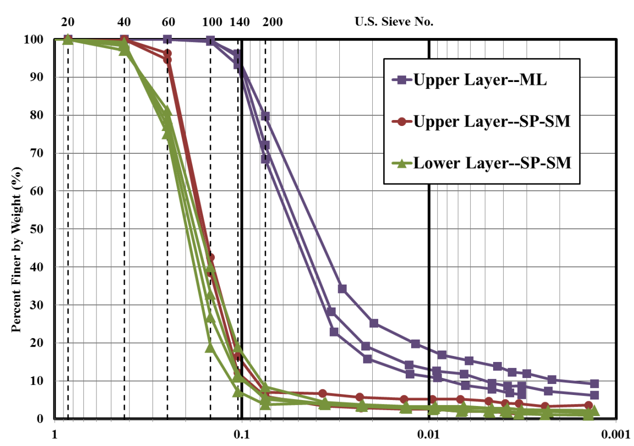

Figure 8. Grain size distribution of CTH triaxial silt and sand specimens

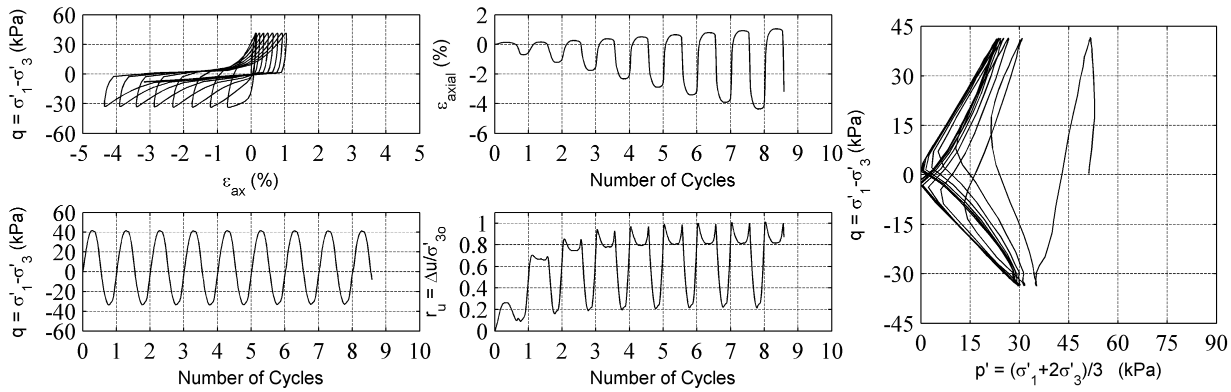

Figure 9. CTX results; CTH site, DM_BH1, 4.27 m depth, USCS: SP-SM, PI=Non-Plastic (NP), e_o=0.77, σ'_{3o}=51.0 kPa, Specimen Height=140.4 mm, Specimen Diameter=60.9 mm, G_s=2.69, Load Frequency=0.1 Hz, CSR=0.365, N to S.A. ε_{ax}=3%

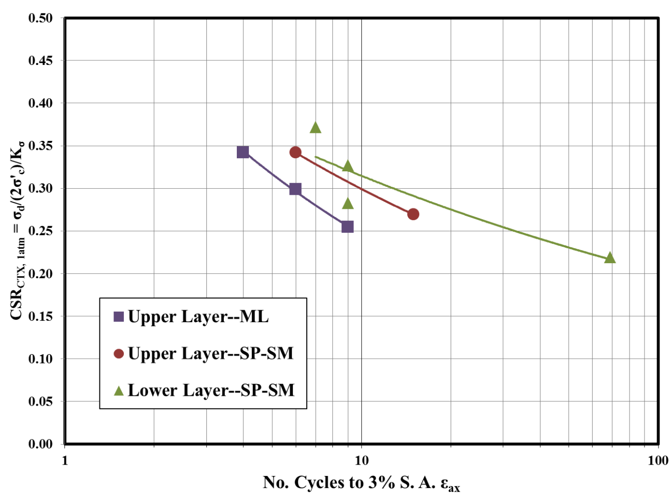

Figure 10. CTX test results at the CTH site

FTG-7 Building

Structure and Damage

The FTG-7 building (S43.5263, E172.6384) was a 7-floor 23.9-m high steel frame structure supported on RC strip footings which is 29.1 m wide (E-W) and 31.8 m long (N-S) (Zupan, 2014). The foundation consists of 0.6-m thick RC perimeter strip footings embedded 1.2-m deep and widths of 2.4 m and 2.0 m in the N-S and E-W directions, respectively, and lengths of 29 m and 34 m in the N-S and E-W directions, respectively. Four interior N-S 0.6-m thick RC strip footings are embedded 0.6-m deep with a width of 3.3 m and a length of 25 m. The centerline distances between the N-S strip footings are between 5.5 and 6.3 m. Strip footings are connected with each other with 0.6 m by 0.6 m RC tie beams. The ground floor consist of 0.1-m thick unreinforced concrete slab and floors 2 through 7 consisted of 0.12-m thick RC slab over 0.75-mm thick galvanized steel decking. The columns are wide-flange steel sections (Zupan, 2014).
Minor liquefaction was observed 25-m northeast of the N-E corner of FTG-7 building after the

Darfield earthquake. No significant building movements were noted. After the Christchurch earthquake, significant liquefaction was observed throughout the area, including along the side of the FTG-7 building. The S-E and S-W corners of the structure settled 10 cm and 7 cm, respectively, relative to the structure's N-W corner. The structure tilted about 12 cm towards the S-E (Bray et al. 2014). Minor liquefaction was observed in the vicinity of several of the exposed strip foundations following the 13 JUN 11 earthquakes with additional differential settlement of 3.5 cm of the S-E corner of the structure relative to its N-W corner.

Site Characterization and Simplified Analyses

The site was originally characterized with 5 CPTs, which were located near each corner of the building and in the center of the northern perimeter of the building (Zupan 2014). Three boreholes were then advanced near the southern corners of the building. The site shows fairly uniform conditions (Figure 11). From the ground surface to 1-1.5 m depth there is a fill material with $D_r \approx 60$ to 80%. The next stratum goes to a depth of 7 to 8.5 m and is sandy silt – silty sand with variable FC and I_c generally between 2.0–2.5. The "clean sand" equivalent relative density for this deposit varies between 35 and 55%. The next stratum goes to a depth of around 14 to 16.5 m and consists of medium dense sand ($I_c \approx 1.5$–2.0) with $D_r \approx 55$ – 70%. After the medium dense sand, very dense sand ($D_r \approx 90\%$) is encountered, and the CPT usually reaches refusal within this stratum. SASW data performed in the initial reconnaissance efforts detects the boundary of the Riccarton Gravel at a depth of 19 m. Some of the CPTs show a 1-m thick layer of clayey soil between the Riccarton Gravel and the dense sand. The water table depth was about 2.0 m throughout the earthquake sequence (Canterbury Geotechnical Database, 2013).

CTX Results and Findings

As mentioned above, three boreholes have been advanced at the FTG-7 site following the Canterbury earthquakes. The first of these boreholes (borehole K1—see Figure 2) employed the Gel-Push sampler to retrieve "undisturbed" samples. These samples were tested as part of a study described in Taylor et al. (2012a). The remaining two boreholes were performed in 2014 and employed the D&M sampler to retrieve "undisturbed" soil samples. Figure 11 provides the location of these boreholes (*DM_Z1_BH1* and *DM_Z1_BH2*), which were located close to the S-E and S-W corners of the FTG-7 footprint, respectively.

Figure 12 provides the grain size distribution plots of triaxial specimens tested from the FTG-7 site. In general the upper portion of the subsurface profile consists of both silts and silty sands (ML and SM material), which is consistent with the CPT data provided in Figure 11. Specimens tested from a depth of 10 to 11 m classified as sand to sand with silt (SP and SP-SM). Figure 13 provides a summary of applied cyclic stress ratio ($CSR_{CTX,1atm}$) versus the number of cycles to 3% single amplitude axial strain ($N_{c\text{-}3\%\ S.A.\ \varepsilon ax}$) for CTX specimens tested from the FTG-7 site. CRR curves were interpreted based on similar grain size distribution of the tested specimens, as well as depth grouping (i.e., specimens close to each other with regards to depth).

Similar to the CTH site, the upper finer silt material (depth of 3.5 to 3.79 m) was slightly weaker than the mid-depth coarser silty sand (depth of 4.7 to 5.64 m) below it. The deeper sand (depth of 10.24 to 11.18 m) was slightly stronger than the mid-depth silty sand material. Results from a few soil specimens from shallow and intermediate depths (2.57 m to 6.72 m) were not entirely consistent with the interpreted CRR curves for their respective soil layers (Figure 13). These soil specimens had slightly different grain size distributions (i.e., they were classified as silt and sand with silt; see Figure 12). These soils generally plotted below or close to the CRR curve for the upper ML material, which indicates they were among the lowest strength materials at the site. Variability in natural soil deposits should be expected, and the variation in CRR from the lab tests are no more than one might expect based on the variation in grain size, potential variability in soil of a fluvial deposit, and the CPT results. Lastly, the results of all these cyclic tests were used to calibrate numerical analyses performed by Luque and Bray (2015), which captured well the measured liquefaction-induced settlement of the FTG-7 building during the CES.

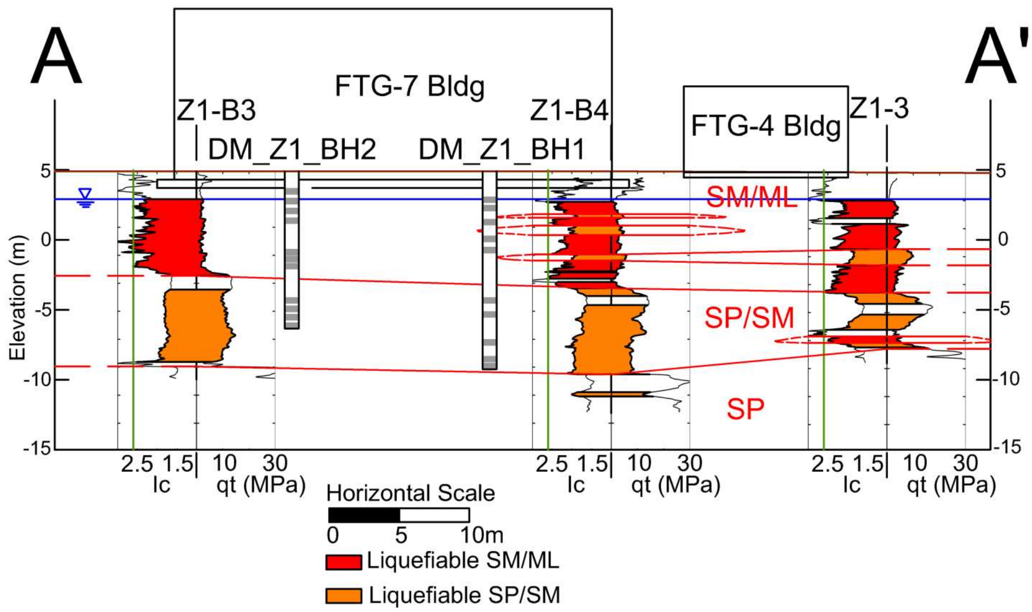

Figure 11. Representative E-W subsurface profile (southern edge) of FTG-7 building; note that "undisturbed" sample locations are shown in grey (modified from Zupan 2014)

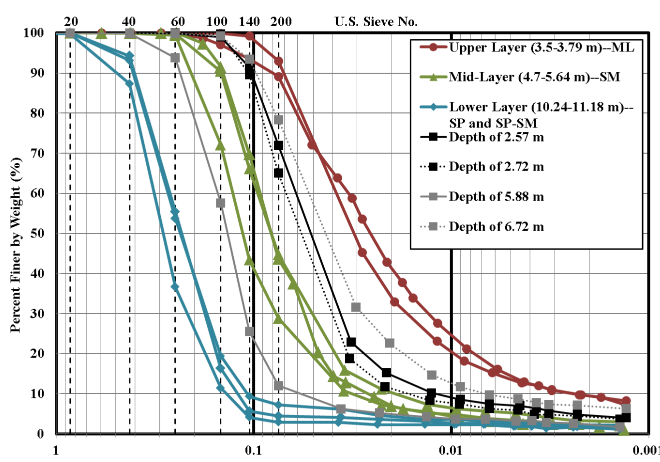

Figure 12. Grain size distribution of FTG-7 soils

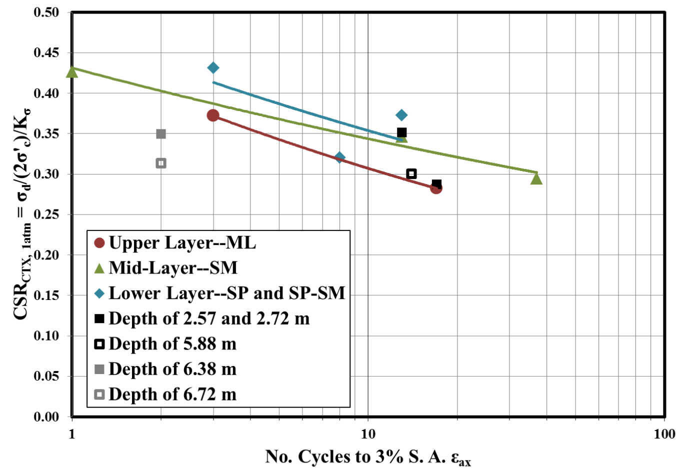

Figure 13. CTX test results at the FTG-7 site

CTUC Building

Structure and Damage

The CTUC building (S43.5286 E172.6425) was approximately 20 m west of the Armagh-Madras parking lot discussed previously. It was a six-story RC frame structure with RC core walls and block in-fill walls with its roof supported by steel framing (Figure 14). The structure was largely supported on shallow footings interconnected with tie beams The CTUC building and its performance during the Canterbury earthquake sequence is described in detail in Bray et al. (2014a). Only additional insights from recent CTX testing and some of the previously noted key observations are described herein.

Severe liquefaction of the foundation soils during the 22 FEB 11 Christchurch earthquake induced significant total and differential settlements of the building, leading to structural distortions and cracking (Cubrinovski et al. 2011a). The building tilted to the east 0.4-0.5 degrees. Differential settlement of the southeast corner of the building produced most of the structural damage. Several of the beams on the south side of the building were cracked near the beam-column connections. The building settled more on its south side than on its north side and more on its east side than its west side. Approximately 20 cm of the 25 cm of differential building settlement along the eastern side of the building was measured across its two southernmost spans (angular distortion ≈ 1/50), which is consistent with the observed cracking of structural beams. Damage to the building was negligible during the other CES events.

Site Characterization and Simplified Analyses

Six CPTs and one exploratory boring were performed at the CTUC building site (15 CPTs were also performed at the nearby Armagh-Madras parking lot site described previously). The generalized subsurface conditions along the east side of the CTUC building are depicted in Figure 15. The groundwater depth was estimated to be 2.5 m for the 4 SEP 10, 26 DEC 10, and 22 FEB 11 earthquakes, and 2.0 m for the 13 JUN 11 earthquake based on the Tonkin & Taylor

(2012) groundwater model. The shallow SM/ML layer observed at CPT Z4-5, which is at the southeast corner of the building to a depth of 4-5 m, is similar to the upper unit described previously at the nearby Armagh-Madras parking lot. It had $q_t < 3$ MPa, $2 < I_c < 2.5$, and thus was likely to liquefy under strong ground shaking. The shallow SM/ML unit was not observed at CPT Z4-28 near the center of the east side of the building, or below the groundwater table north of CPT Z4-28.

There are liquefiable soils at each of the CPT locations depicted in Figure 15. However, the distinguishing difference between them are the shallow liquefiable soils just beneath the building foundation at CPT Z4-5 whereas the liquefiable soils at CPTs Z4-28, Z4-7 and Z4-10 are located primarily at depths below 8 m. The dramatic change in the shallow soil conditions from the building's north end, which did not contain shallow liquefiable soils, to its south end, which contained shallow liquefiable soils, led to significant differential settlement over the southernmost spans of the building frame. Thus, the buried shallow SM/ML deposit that produced the well-defined liquefaction feature in the nearby Armagh-Madras parking lot just cut across the southeast corner of the CTUC Building. This unit likely caused much of the observed damage.

Low FS_l values were calculated in the shallow SM/ML layer for the intense 22 FEB 11 Christchurch earthquake (Bray et al. 2014a). FS_l values just below one are also calculated for the 4 SEP 10 Darfield and 13 JUN 11 earthquakes. Although there were no reports of liquefaction at this location after these events, it is possible that a minor amount of liquefaction was unreported or that marginal liquefaction occurred and surface manifestations were not observed.

Figure 14. View of south side of CTUC building and close-up view of its SE corner after the Christchurch earthquake (from Bray et al. 2014a).

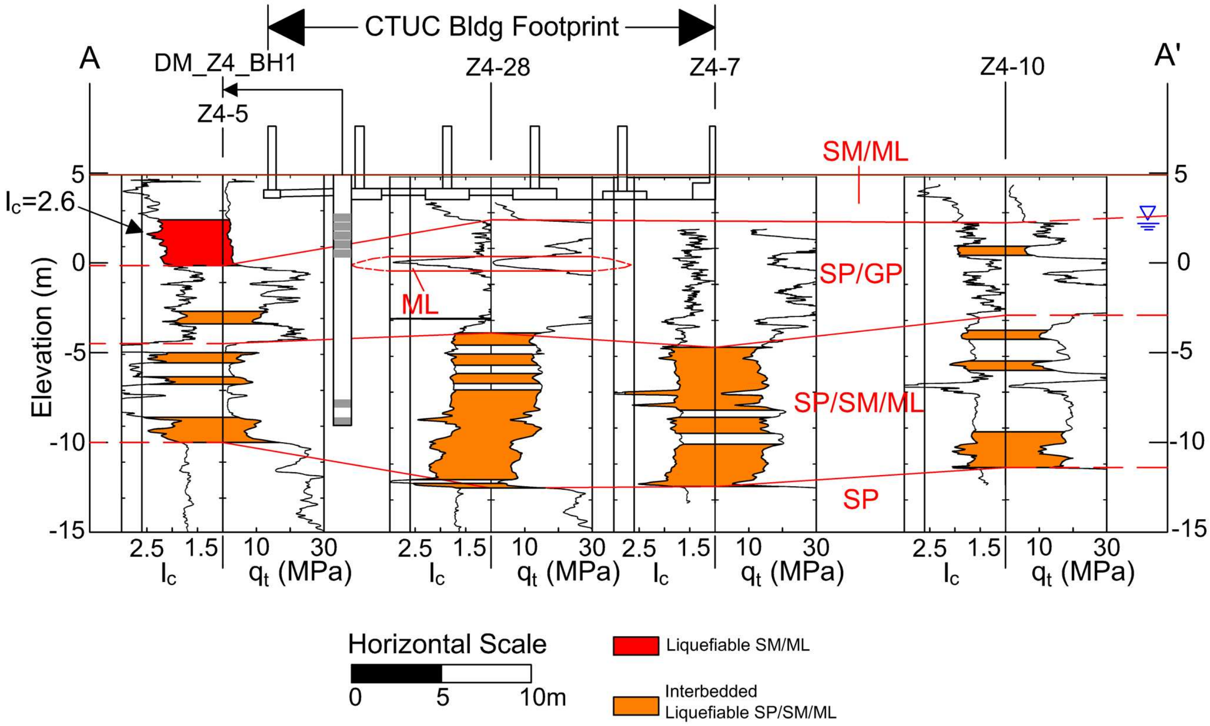

Figure 15. Subsurface conditions at CTUC site showing zones of materials with $FS_l < 1.0$ based on the RW98 procedure using $PGA_{50} = 0.45$ *g* from Bradley & Hughes (2012) for the 22 FEB 11 Christchurch earthquake (modified from Bray et al. 2014a).

The post-liquefaction residual shear strength of the shallow SM/ML layer was estimated to be 6 - 10 kPa using the Olson & Stark (2002) and IB08 procedures. The static FS against a bearing capacity failure based on a two-layer cohesive soil deposit solution provided in NAVFACDM 7.02 (1986) is 2.1 to 2.3 at the location of the southeast corner footing, using the residual shear strength of the shallow liquefiable SM/ML materials and an equivalent undrained shear strength of the SM/ML materials above the groundwater. If the materials above the groundwater lost strength due to the upward migration of liquefied soil, then the FS is below one. The southeast footing may have underwent a partial bearing capacity failure, but its differential settlement was largely the result of ground loss due to sediment ejecta, which was observed at the site (Bray et al. 2014a), and due to some contribution of the other settlement mechanisms described in Bray & Dashti (2014), because bulging of the ground surface was not observed at this site. Conventional 1D liquefaction-induced ground settlement procedures, which do not capture important shear-induced deformation mechanisms, such as SSI ratcheting and partial bearing failure, and the effects of ground loss due to sediment ejecta, underestimated the differential settlement measured after the Christchurch earthquake. Conversely, 1D procedures overestimated the ground settlement observed after the Darfield and 13 JUN 11 earthquakes (Bray et al. 2014a,b).

CTX Results and Findings

Figure 15 shows that a borehole was drilled in the southeast corner of the CTUC building site to obtain “undisturbed” samples of soils in the area where the largest building settlement occurred. Three samples were obtained of the upper liquefiable layer (depth of 2.75 m to 4.05 m) and one sample of the lower sand was obtained (at a depth of about 13.67 m). Index testing on the test specimens revealed that the upper layer of silty soil comprises a layer of silty sand (SM) from a depth of 2.75 to 3.40 m and a layer of sand and sand with silt (SP and SP-SM) from a depth of 3.41 to 4.05 m. The grain size distributions of the retrieved soils are shown in Figure 16. A summary of the applied stress normalized cyclic stress ratio ($CSR_{CTX,1atm}$) versus the number of cycles to 3% single amplitude axial strain ($N_{c\text{-}3\%\ S.A.\ \varepsilon ax}$) for CTX specimens tested from the CTUC site is shown in Figure 17. The resulting cyclic resistance ratio (CRR) curves indicate that the finer sub-layer of SM material is slightly weaker than the coarser SP and SP-SM material.

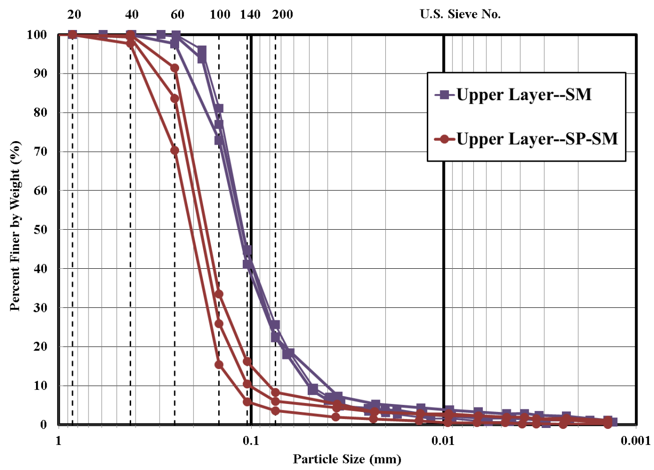

Figure 16. Grain size distribution of CTUC building site soils

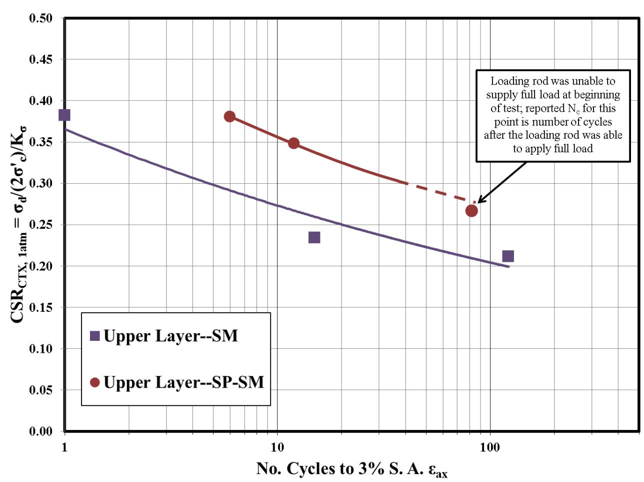

Figure 17. Results of CTX testing at CTUC site; mid-depths of specimens are reported.

One of the key findings of the CTX testing is the presence of two distinct layers of soil that have slightly different liquefaction resistances in the upper critical layer of soil in the SE corner of the site. This was not as clearly evident based on the CPT results (i.e., the q_t and I_c profiles shown in Figure 15 are relatively constant for the upper layer of liquefiable material). The laboratory testing results coupled with the CPT interpretations provide data and insights for researchers investigating the seismic performance of the CTUC building during the Canterbury earthquakes.

Summary

The Canterbury earthquake sequence produced varying degrees of liquefaction with differing effects on buildings with different structural and foundation systems. The CPT proved to be a useful site characterization tool in Christchurch. Its results enabled liquefaction triggering evaluations using established simplified procedures that were generally conservative. Extensive liquefaction was estimated in areas along the Avon River in the CBD for the intense Christchurch event, but significant liquefaction was also estimated at the CBD sites for the Darfield and 13 JUN 11 events (Bray et al. 2014a). The conservatism in the liquefaction triggering assessments led to 1D post-liquefaction ground settlement estimates that were generally similar for the large events in the Canterbury earthquake sequence, whereas significant building settlements and damage in the CBD were observed for the Christchurch earthquake but not for the Darfield and 13 JUN 11 earthquakes (Bray et al. 2014a). Thus, while the CPT-based liquefaction procedures provide valuable insights, it was difficult to capture the actual range of ground and building performances in the CBD as a result of the Canterbury earthquake sequence.

Moreover, conventional CPT-based 1D post-liquefaction reconsolidation ground settlement procedures do not capture important shear-induced deformation mechanisms and the effects of ground loss due to sediment ejecta. Performance-based earthquake engineering requires improved analytical procedures to discern between the differing levels of performance observed in Christchurch during the Canterbury earthquake sequence. Two dimensional (2D) nonlinear effective stress analyses can capture shear-induced deformation mechanisms in addition to volumetric-induced settlement mechanisms. However, robust soil constitutive models are required. Given the brittle nature of the liquefaction phenomenon as soil transforms rapidly from a stiff to a soft response as the excess pore water pressure rises beyond a threshold value, the development of robust analytical procedures to evaluate the effects of liquefaction on buildings will be challenging. However, the case histories from the Canterbury earthquake sequence provide a comprehensive set of ground and building performance data for developing such methods.

To supplement the numerous CPTs that were performed in the CBD of Christchurch, “undisturbed” sampling was carried out using the Dames & Moore hydraulic piston sampler. In addition to index testing, static and cyclic triaxial tests were performed on soil specimens from these samples. In total, eight building sites were studied to better characterize some of the key soil layers in the CBD with regards to nonlinear soil response and liquefaction resistance. One of the key findings of the sampling and testing program was that loose sands were most likely densified during the sampling and specimen preparation procedures prior to testing. However, reasonable trends were observed with regards to the liquefaction resistance of medium dense sands, silts, and silty sands. The data provided by the advanced laboratory testing of these soils is important for conducting ongoing and future nonlinear effective stress analyses, whose constitutive models can be calibrated using the CTX test results. The results from these advanced numerical simulations will provide even greater insights.

The seismic performances of most of the office buildings in the CBD that were investigated in this study were dictated by a shallow layer of silty sand or sandy silt that liquefied. The shallow liquefiable SM/ML layer, when present within the soil profile, was often the critical layer in the observed liquefaction in the CBD. Its thickness below the groundwater table could vary considerably over relatively short distances. When this layer was present and supported shallow foundations, the seismic performance of the structure was greatly affected by the response of the shallow liquefiable soil layer. The importance of shallow liquefaction for structures supported on shallow foundations cannot be overemphasized. Thus, the use of the CPT to identify when a shallow layer of potentially liquefiable soil is present is critically important. For those cases wherein the facility already exists at a site not previously well characterized or at a site wherein a recent study has significantly increased the seismic hazard, advanced numerical analyses supported by cyclic testing of carefully retrieved "undisturbed" soil specimens can provide important insights. These analyses can provide an enhanced level of understanding of the cyclic response of the key soil deposits and their potential effects on the overlying structure. Developing and calibrating these analytical procedures are primary goals of this ongoing study.

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