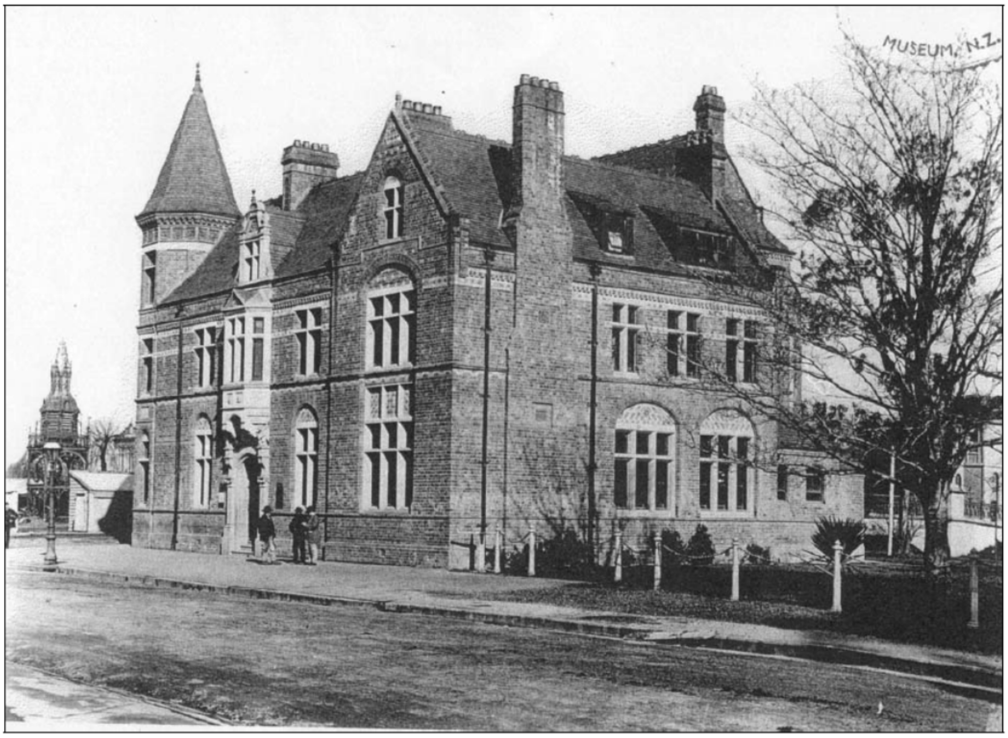
Old Municipal Chambers

159 Oxford Terrace

Christchurch



Geotechnical Report

Issue 01 Final

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GEOLOGICAL & ENGINEERING SERVICES

Old Municipal Chambers
Geotechnical Report

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Summary

This report summarises the geotechnical considerations relating to the restoration of the Old Municipal Chambers Building at 159 Oxford Terrace, on the north corner with Worcester Street..

A geotechnical site investigation was carried out on the site by Land Development and Exploration Ltd in 2011. This data has since been supplemented with a number of deep boreholes and CPT tests both on and around the site, sourced from the Canterbury Geotechnical Database (CGD), and three additional CPT tests close to the building.

The information available shows that the site is underlain with a surface soils overlying predominantly silty sand with some silt and silt lenses. These surface soils overlie a shallow gravel layer at between about 1m and 2.5m. The gravel layer is generally 4 – 7m thick and overlies sands, some of which is loose and liquefiable, but denser below 12 – 14m. Softer silt and loose sand layers below about 18m depth cap the Riccarton gravel at 20 – 22m depth. The water table is probably at about 1.9m depth under the site.

Liquefaction analysis indicates limited liquefaction above the shallow gravel, as for most of the site the top of the gravel is above the ground water table. More significant liquefaction is predicted in the looser sand layers below the gravel and above the Riccarton gravel.

The site was shaken strongly in the recent earthquakes with probably in excess of SLS levels in September 2010 with no surface ground damage reported, and approaching a ULS event (in terms of liquefaction) in February 2011. Ground damage on the site itself as well as in the immediate area appears to have been very limited.

Restoration of the building will involve some foundation work. It appears very likely that the existing foundations extend down to the gravel layer. It is recommended that any new foundations are similarly shallow spread footings bearing directly onto the gravel. Information is provided for preliminary design of shallow foundation systems.

Old Municipal Chambers
159 Oxford Terrace

Geotechnical Report

Contents

Appendices

1 Introduction

The Old Municipal Chambers Building was damaged in the February 2011 earthquake and reinstatement options are now being considered. This geotechnical report outlines the geotechnical conditions at the site, an assessment of the liquefaction hazard, considers potential foundation options, and provides parameters for foundation design to allow the reinstatement options to be properly assessed and designed with respect to the impact on the foundations.

This report collates relevant geotechnical information from around the site, together with data from tests on the site itself. Some geotechnical assessment of the site was carried out by Land Development and Exploration Ltd in 2011. There is now considerably more geotechnical investigation data for the city available on the Canterbury Geotechnical Database (CGD). There are seven boreholes and ten CPT tests within 100m of the centre of the building, and three CPTs and one borehole within 50m. This additional information justified an update of the previous reports.

This report builds on the earlier geotechnical work at the site by incorporating the more recent data into the geotechnical model, thus allowing a better understanding of the underlying soils. In addition some new testing was carried out specifically for this project. This in turn assists the consideration of how best to repair or replace damaged sections of foundation.

Therefore this report is based in part on site investigation data collated from a number of sources, all of which was carried out by others. Geotech Consulting Ltd takes no responsibility for the accuracy or otherwise of that investigation and test data.

2 Site

The building is approximately 20m square at the corner of Oxford terrace and Worcester Street, in central Christchurch. The official address is 159 Oxford Terrace. The building is constructed on the boundaries to the streets to the south and west, while there are lawn areas to the banks of the Avon river to the west and north.

The Canterbury geotechnical database (CGD) Property summary report gives a mean elevation for the site of 4.44m, with a minimum of 4.18m and maximum of 4.84m, ie 0.66m variation (Lyttelton Datum). The ground remains relatively level for about 10m to the west and 30m to the north, before sloping down to the river with the river bed about 1.8m below the site level. At the southwest corner, Worcester Street has been built up to form the approach to the road bridge, and the street is at about RL5.0. This is retained with a low wall beyond the building, but the fill abuts the base of the building wall on this part of the south side.

The building has two main floors with a third floor in the attic along the east side. It is of brick construction with a tile roof and timber floors. It was constructed in 1886. A small area at the northwest corner was not part of the original building but was infilled with a single storey structure in 1935, replacing earlier small additions in this corner. A lift was installed with associated foundation work toward the eastern side in 1989, along with a new basement storage area (approximately 4.7m by 8m in plan) under the southwest corner.

3 Geotechnical Information

Land Development and Exploration Ltd (LDE) had Brown Brothers Drilling carry out a Cone Penetration Test (CPT) off the northwest corner of the building in March 2011. Mcmillan Drilling did a further 3 CPT tests in February 2016 at the direction of Geotech Consulting Ltd. The exact location of the LDE CPT is not known, but scaling off their investigation plan in the report places it at about 5.7m from the corner. The locations of these four tests are shown on the site investigation plan, appended. The tests are summarised in table 3.1, and the test logs are appended.

Test	Date	Location	Depth to gravel	Depth of test (m)
CPT 01	LDE, March 2011	5.7m off NW cnr	1.0	3.8
CPT 02	GCL, Feb 2016	SW cnr	-	1.0m (anchor pullout)
CPT 02R	GCL, Feb 2016	1.7m off SW cnr	2.4	2.8
CPT 03	GCL, Feb 2016	4m off NE cnr	1.3	3.1
CPT 04	GCL, Feb 2016	6.3m off SE cnr	1.35	1.6

Table 3.1 CPT tests adjacent to OMC Building

At the time of the 2016 testing, the building was fenced off and several walls were propped with steel frames, restricting access to the building itself and limiting where testing could be carried out.

The CPT tests were all done with a $10cm^2$ cone. CPTs 2 & 3 were with a small track mounted rig as this was the only way to access relatively close to the building given the fencing and props around the structure. As the small rig relies on auger anchors to provide the necessary reaction, these tests were limited in depth as the dense gravel prevented the anchors from being placed to their normal depth and thus there was limited reaction to push the cone with once the gravel was reached. CPT02 suffered an anchor pullout with the tip at 1m depth, and was repeated a short distance away with refusal on tip pressure. CPT04 was performed with a truck rig in Oxford Terrace, but the dense gravel also limited penetration depth. CPT01 (2011) reached the greatest depth – details of the machine are not known.

The Canterbury Geotechnical Database (CGD) now contains considerably more geotechnical investigation data for this part of the city. There are seven boreholes and ten CPT tests within 100m of the centre of the building, and three CPTs and one borehole within 50m. These tests are summarised in Table 3.2, where they are listed in order clockwise from south of the site. These tests have all been done since 2011 and the boreholes have been drilled with sonic drillrigs.

The CGD tests provide a reasonable coverage around the site except between northeast to south. Records held by GCL have helped infill this area, with a 1970 borehole from the hotel site across Oxford Tce, and 3 boreholes in the streets by the Clarendon site diagonally across the street intersection. These are also included in Table 3.2. These older boreholes were drilled by rotary wash or cable tool methods and subtleties in the soil profile may have been lost with the sampling techniques.

Test CGD ID	Test ID	Location	Depth to gravel	Underside gravel	Depth of test
CPT 49059	Opus 03, 2013	80m S	1.9	5.5	22.5
CPT 35356	Beca CPT1, 2013	40m SW	1.6	5.2	21.2
CPT 35357	Beca CPT2, 2013	50m W	2.8	8.3	23.2
BH1748	TT CBD15, 2011	55m WNW	3.4	9.0	29.2
CPT 432	CBD-49/P, 2011	55m WNW	2.4	9.0	23.1
BH 9899	Au BH1	80m WNW	0.6	8.3	30.7
CPT 9904	CPT03-1	90m WNW	1.7	?	23.1
BH 9902	Au BH2	95m NW	2.7	8.8	30.6
BH 13571	GCL BH1	100m NNW	4.0	10.1	24.0
BH 27413	Au BH2	100m NNW	3.7	10.5	27.2
CPT 32758	11026-1-002,2013	90m NNE	1.9	7.1	22.0
BH 32760	Scirt BH02	90m NNE	2.1	7.0	25.8
BH 54462	TT BH-RES-01	95m NE	1.9	7.1	31.0
B3	Hotel 1970	50m NE	2.4	7.6	16.9
B4	Hotel 1971	65m NE	1.0	7.6	11.6
CT1	Clarendon 1986	55m ESE	2.2	9.5	25.0
CT3	Clarendon 1986	40m SE	2.0	7.2	9.0
CT5	Clarendon 1986	65m SE	2.0	7.0	10.5

Location is approximate distance in metres and direction from the centre of the OMC building

Table 3.2 Test and boreholes from around the site

The Canterbury Geotechnical Database has also been referred to for other information relevant to the site, such as depth to ground water.

4 Subsurface Conditions

4.1 Geology

The general geology of the area is of importance in terms of the geotechnical setting. The geological map of the area (Brown and Weeber, 1992) shows the site area as underlain with soils of the Springston Formation comprising dominantly alluvial sand and silt overbank deposits, and gravel deposited in channels and delta by the Waimakariri river. The formation is geologically recent; at this site none of it is older than 6,000 years old, and the soils within a metre or so of the surface may be only several hundred years old. The Springston Formation soils are inter-fingered at depth with soils of the Christchurch Formation, which is predominantly sand deposited in marine and estuarine conditions. The transition between the two is unclear and the two are probably interbedded below 8 - 10m depth. These Holocene age soils are deposited on the top of the first glacial outwash deposits known as the Riccarton gravel, dated at 14000 – 70,000 years old. This is generally a well graded gravel up to cobble size, but there is considerable grading and density variation within the unit.

The Riccarton gravel is probably about 10 to 15m thick under the site. Below this gravel layer is a deep sequence of gravel layers separated by beds of sand, silt and clay, which extends to about 500m depth. Below this depth there are volcanic rocks from the Lyttelton volcano (9 – 12 million years old) over Tertiary aged sedimentary rock overlying the greywacke basement at perhaps 1.2 – 1.5 km. The profile deeper than about 30m is of little direct consequence to building foundations except that it will modify the seismic waves as they propagate through the deep soil column, and may also be subject to some deformation in response to strong earthquake shaking.

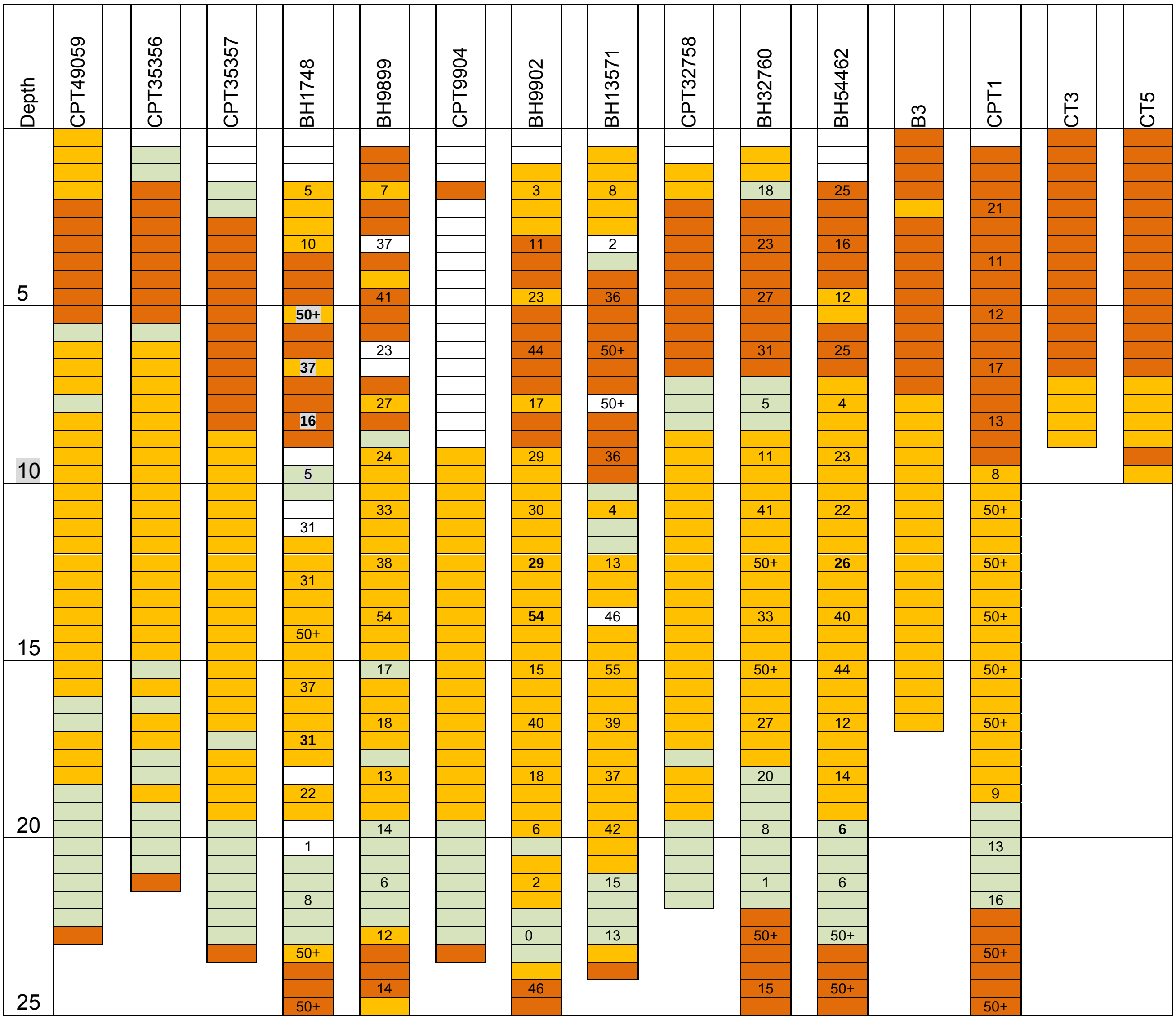
4.2 Soil Profile

The soil profile above 25m depth is summarised in Figure 4.1, which shows the simplified borehole logs for the tests around the site.

General profile of surrounding area
The site is underlain with a sequence of predominant soil types that extends over a large area of the central city between Armagh Street to south of Tuam Street, and Colombo Street west into the Botanic Gardens.

The surface soils (fill, basecourse or topsoil) overly predominantly silty sand with some silt and silt lenses, which in turn overlies sandy gravel. The depth to the gravel varies between about 0.6m and about 4m in the tests around the site. The gravel is closer to the ground surface on the east side of the river where the ground elevation is about 1m lower than Cambridge Terrace on the west side. The thickness of this gravel also varies considerably from less than 4m to over 7m.

The gravel extends to between about 5m and 10m depth, overlying predominantly sand soils which are often loose between the gravel and about 12m depth and then dense to very dense. Below about 18m there is a sequence of interbedded silt, sandy silt and sand. The Riccarton gravel was contacted at between 21m and 23m depth in the tests around the site; some of this variation will be due to changes in ground level.



Tests are in clockwise order starting from south of the site; refer Table 3.2

Numbers in cells are SPT 'N' values (blows per 300mm penetration) uncorrected;

KEY		gravel		sand		silty sand		silt

Figure 4.1 Summary of Boreholes and CPT tests around the Old Municipal Chambers

Shallow Soil Profile at the site

The four CPTs close to the building all show silty sand / sandy silt soil overlying what is inferred to be sandy gravel. The depth to the gravel is reasonably consistent at 1.0 – 1.3m in three of the tests, but lower at about 2.4m in CPT02R, close the southwest corner. The ground level is higher here, by an estimated 0.3m, which accounts for some of this difference. The top of the gravel is dense with 20 – 30 MPa tip resistance in 3 of the tests, but CPT01 records lower tip resistances of 8 – 18 MPa between 1m and 3.5m depth.

4.3 Shallow Gravel

The shallow gravel is part of a formation which extends from a little north of Armagh St in the north to south of Tuam St and from well west of Rolleston Avenue to east of Colombo St. There is some grading variation within the gravel layer at this and other sites, with grading varying from gravely sand to gravel with only a trace of sand.

Grading curves from a borehole in the area indicate a poorly graded gravel with a deficit of soil in the 0.5 to 5mm range (see 4.3). The density also varies from loose to dense conditions as indicated by both the CPT tests and SPT results at different depths.

The tests around the site provide data on the gravel thickness, as shown in Table 4.1.

Test	thickness	Test	thickness
CPT 49059	3.6	CPT 32758	5.2
CPT 35356	3.6	BH 32760	4.9
CPT 35357	5.5	BH 54462	5.2
BH1748	5.6	B3	5.2
CPT 432	6.6	B4	6.6
BH 9899	7.7	CT1	7.3
BH 9902	6.1	CT3	5.2
BH 13571	6.1	CT5	5.0
BH 27413	6.8		

Table 4.1 Thickness of shallow gravel unit (m)

It varies between 3.6m and 7.7m in these tests, with an average of 5.6m, with the layer being thinner to the southwest and thickest to the north.

4.3 Soil Properties

Particle size distribution data is available for five soil samples from a borehole about 250m north of the site. The results are summarized in Table 4.2. Only one sample has a high proportion of fines; the remaining soils are all clean sand and gravel. A number of samples from another site about 200m to the south on a similar soil profile were tested to determine fines content and plasticity index, and are relevant in defining the range of gradings and soil types present within the soil profile. The fines contents are summarised in Table 4.2, grouped according to the major soil strata.

Sample depth	description	% gravel	% sand	% fines
Site to North				
3.0 – 3.5	Sandy Silt	0	33	67
4.5 – 5.0	Gravelly Sand (poorly graded)	37	58	5
6.75 – 7.25	Sandy Gravel (poorly graded)	78	21	1
8.5 – 9.0	Sandy Gravel (poorly graded)	74	25	1
10.5 – 11.0	sand	2	96	2
Site to south				
2.3 – 2.8	Sandy silt			69
3.5 – 9.0	Sandy gravel			1 - 4
6.3 – 17.0	sand			3 - 70
18.5 – 22.5	Silt and sand			59 - 99

Table 4.2 Grading fractions for soils

The particle size distribution curves for the site to the south are plotted together in Figure 4.2, grouped for the shallow gravel, sands and deeper silty soils above the Riccarton gravel, to show the typical grading and range for each of the main strata. Figure 4.2(c) also shows the curve for a sample of the surface sandy silt. The gravel samples all show a surprisingly similar grading with a noticeable gap in particles between 0.5mm and 5mm size.

There is a wider range of grading in the sand soils. There is a trend in increasing fineness with depth. One sample (13m depth) has an appreciable (30%) gravel content, while a sample from 16m is similar to the soils below 18m depth.

***(a) Shallow Gravel** (samples 3.5m – 9.0m depth)*

***(b) Sand below gravel** (samples 6.5m – 16.5m)*

Sample BH03, 2.3 – 2.8m

***(c) silt and sand layer capping Riccarton gravel** (samples 18.5m – 22.5m)*

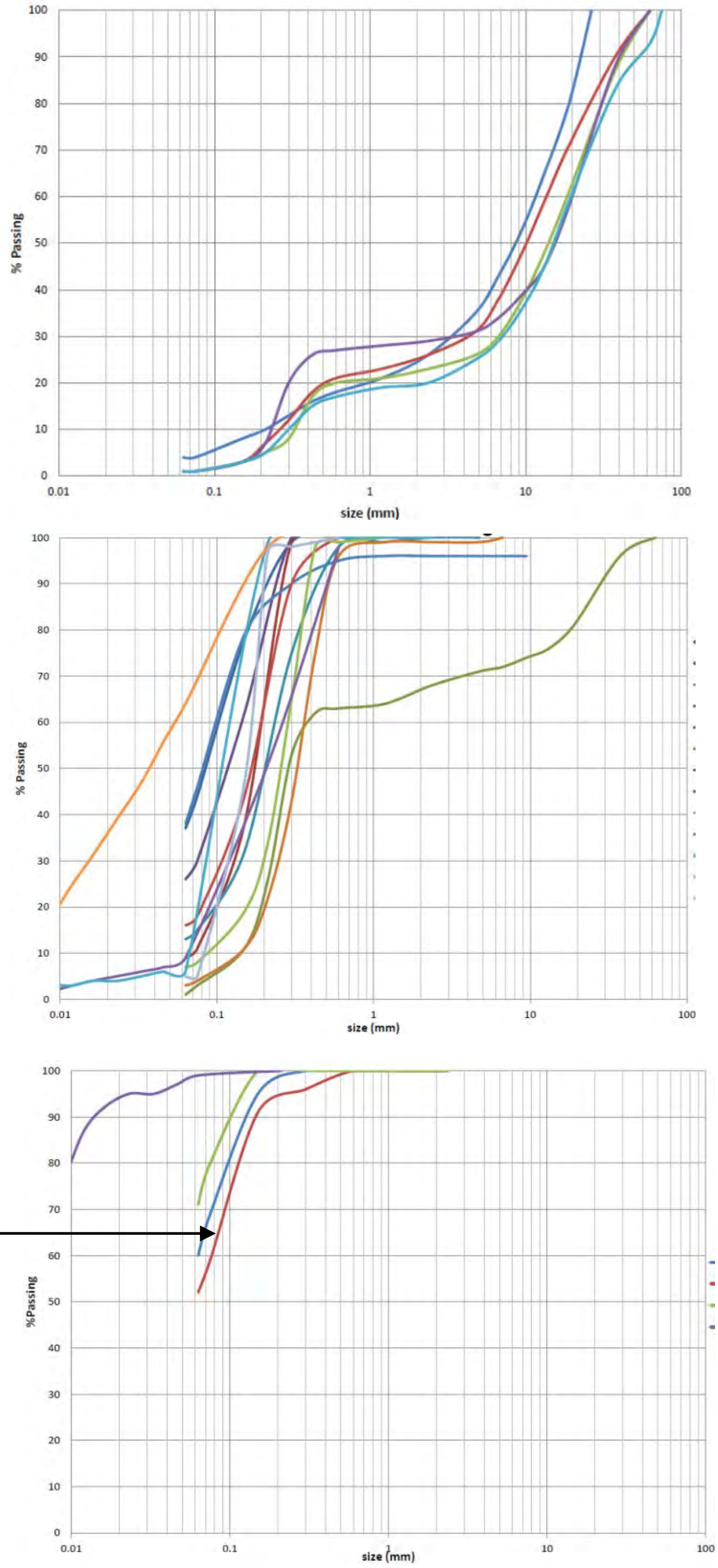


Figure 4.2 Grading curves for samples

The soils are essentially cohesionless sands and gravels. Soil properties suitable for design are shown below in Table 4.4. These values are likely to be conservative.

Unit (in order of depth)	Unit weight (kN/m3)	Angle of internal friction (degrees)	Cohesion (kPa)
Shallow sand and silt	18	30	0
Shallow gravel	20	35	0
Sand	18	30	0
Sand 14 – 17m	18	35	0
Silt 2	18	30	0
Riccarton gravel	21	35	0

Table 4.4 Soil Properties

4.4 Ground water levels

There are limited measurements of the depth to groundwater in the bores around the site, as listed in Table 4.5. Water level measurements in boreholes are always uncertain as the drilling procedure involves the addition and removal of considerable volumes of water from the casing, and often the time available is insufficient for water levels to stabilise. Ground levels are unknown but the first four bores listed in Table 4.5 are all on the higher west side of the river whereas the remaining four are from the east side where ground levels are lower.

Borehole	Depth to water		Borehole	Depth to water
BH 9899	2.3		BH 32760	1.85
BH 9902	2.3 – 2.5		B3	1.8
BH 13571	2.3		B4	1.4
BH 27413	3.0		CT1	1.5

Table 4.5 Depth (m) to water table as measured in test holes

The recent GNS median groundwater depth study, as available on the CGD, uses all the available and reliable data from all the monitoring wells in the city which have a reasonable length of time of recording. The data is of a better quality than from investigation boreholes, and while there is clearly considerable uncertainty as interpolation between monitoring locations, it is likely to give considerably more reliable levels than the boreholes. The GNS study indicates the water table elevations as shown in Table 4.6.

	CGD 735 55m WNW	CGD 736 210m E	river 45m SW	interpolated contours at site
Median	2.56	2.62		2.7
15 percentile	2.45	2.23	2.6	2.4
85 percentile	2.8	2.72		2.8

Reduced levels are to Lyttelton Datum

Table 4.6 Water table elevations and depths (m) from GNS study

The contours in the model are somewhat irregular curving around the well CGD735 and the river monitoring point on the upstream side of the Worcester St bridge. The most reliable estimate of water table elevation is therefore likely to the levels as recorded in the monitoring well CGD 735, located at the top of the bank immediately across the river from the site. The variation in the data recorded between November 2011 and November 2015 is shown in Figure 4.3

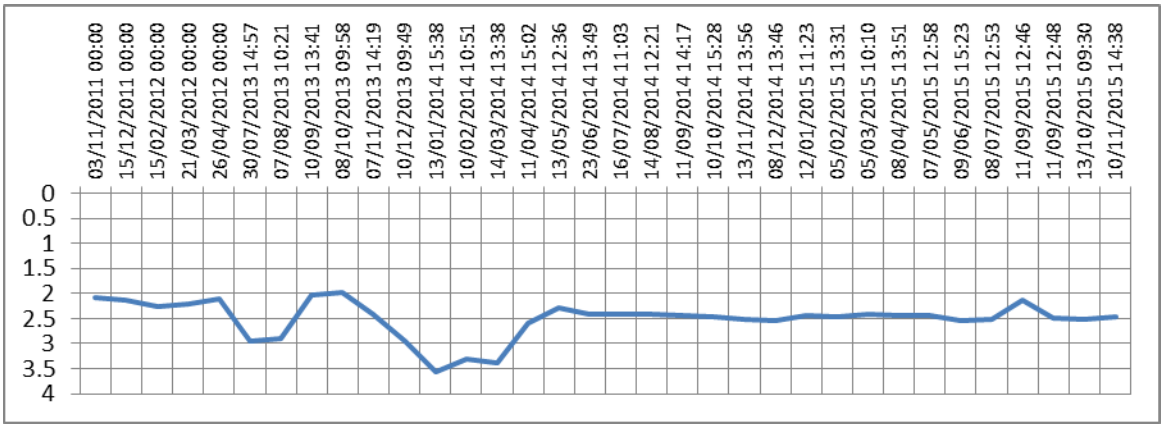


Figure 4.3 depths to water in monitoring well CGD 735

It is considered reasonable to take this well data as representative of the ground water at the site. With a mean ground level of RL4.44, this gives the median depth to groundwater as 1.9m in a 15% - 85% range of 1.6 – 2.0m.

When these depths are compared with the depths to gravel from the CPT tests around the building, we can estimate water level in relation to top of gravel as follows

Test	Location	Approx GL	Depth to gravel	RL gravel	Depth water above gravel
CPT 01	5.7m off NW cnr	4.2	1.0	3.2	- 0.6
CPT 02R	1.7m off SW cnr	4.8	2.4	2.4	+ 0.2
CPT 03	4m off NE cnr	4.4	1.3	3.1	- 0.5
CPT 04	6.3m off SE cnr	4.5	1.35	3.1	- 0.5

Assumes median water table elevation RL2.6

Table 4.7 Comparison of water table with top of gravel

Thus is appears that for most of the site the water table will be below the top of the gravel and the surface silt / sand soils will be unsaturated, with only CPT02R indicating saturation of aboiut 0.2m of sand above the gravel.

5 Seismic Considerations

5.1 Seismic Category

The deep alluvial formations underlying this site defines this site as Class D – deep or soft soil site - in terms of the seismic design requirements of NZS 1170:2004.

5.2 Seismic Hazard

A probabilistic seismic hazard assessment for Canterbury (Stirling, 2007) gives long term probabilities for shaking intensities and peak ground accelerations for Christchurch. As a result of the recent earthquakes, ongoing aftershocks and new recognition of the seismic setting around Christchurch and Canterbury, the probabilities have been changed to reflect the short to medium term increased seismic hazard in the region, with a resultant increase in peak ground accelerations (pga) to be used for design. The accelerations in Table 5.1 are for class D (deep soil) sites. Row (2) shows the PGA as current prior to the recent earthquakes, whereas the last row (3) shows now current PGA recommended for liquefaction assessment by MBIE in April 2012 (TC3 guidelines).

	Return Period (years)	25	50	200	475	1000
1	Stirling, 2007	0.07	0.1	0.18	0.26	0.34
2	NZS 1170.5: 2004*	0.07	0.1	0.18	0.25	0.34
3	PGA for liquefaction, April 2012	0.13	0.185	0.28	0.35	0.42

**derived from PGA = Z.R.C*

Table 5.1 Peak Ground Acceleration (PGA) in %g for Christchurch

Design of buildings uses two loading situations – the serviceability limit state 9SLS) and the ultimate limit state (ULS). At the SLS level of earthquake shaking the building should perform such that damage is minimal and easily repairable and does not affect the function or operation of the structure. At the ULS level of shaking, the structure is permitted to suffer significant damage, but the structure or any parts of the building should not collapse to safeguard life safety. For an importance level 2 building, such as this house, the SLS earthquake is set at that expected, on average, once every 25 years, and the ULS is set at a 500 year return period. Table 5.1 shows that the PGA at SLS level shaking has effectively doubled and at ULS it has increased by about 40% since before the start of the recent earthquakes.

5.3 Recent Earthquakes

The site has been subjected to repeated shaking from the recent Canterbury earthquake sequence. A method by Bradley of the University of Canterbury which combines the empirical strong motion attenuation with distance model with the actual observation to produce conditional peak ground accelerations, gives the data in Table 5.2.

Earthquake	Mag.	PGA			Equivalent M7.5 PGA		
		Mean - σ	Mean	Mean + σ	Mean - σ	Mean	Mean + σ
4 Sep 2010	7.1	0.16	**0.21**	0.26	0.15	**0.18**	0.23
22 February 2011	6.2	0.34	**0.44**	0.57	0.24	**0.31**	0.40
13 June 2011	6.0	0.17	**0.23**	0.30	0.11	**0.15**	0.20
23 Dec 2011	5.9	0.16	**0.21**	0.27	0.10	**0.13**	0.17

Table 5.2 Conditional PGA for recent earthquakes (Bradley model)

The predicted mean PGA for each earthquake plus the 16th and 84th percentiles have also been converted to an equivalent PGA for a magnitude M7.5 earthquake, to allow direct comparison with Table 5.1, and approximate return periods for the recent earthquake shaking are shown in Table 5.3. These are all for liquefaction analysis purposes.

Earthquake	Sep 2010	Feb 2011	June 2011	Dec 2011
Pre EQ seismicity	200	800	150	100
Current seismicity	50	270	35	25

Table 5.3 Approximate return periods (years) for PGA in recent earthquakes

The site has experienced shaking equivalent to or well in excess of a SLS event (with respect to the current upgraded probabilities) on two occasions, and approaching a ULS event on one. In terms of the pre-2010 seismicity, all four events were well in excess of the SLS event and the February 2011 earthquake was well in excess of the previous ULS level of shaking.

5.4 Liquefaction

There is a liquefaction hazard under this site with strong earthquake shaking, mainly at depth in the looser parts of the sandy soils below the upper gravel and above the Riccarton gravel at 22m depth. There is limited potential liquefaction of the sandy surface soils where the top of the shallow gravel layer dips below the water table. At the site, only CPT02 at the southwest corner suggests that this occurs under the building, as elsewhere the water table appears to be well below the top of the gravel.

The extent of shallow liquefaction that might occur at CPT02R has been estimated by liquefaction analysis. Analysis is with the method by Boulanger & Idriss 2014 with the settlement estimation is by the method of Zhang et al (2002). It suggests no liquefaction at SLS, and a layer about 40mm thick at ULS resulting in 1mm settlement. This is with the water table at RL 2.6 (2.2m depth below the somewhat higher ground level in this corner). If the water level is higher there is no additional liquefaction until the water table rises to about RL2.7, and then the soils above this level are analysed as liquefiable. With a water table at RL 3.0 (ie depth 1.8m) then 2mm settlement is predicted at SLS and a total thickness of 360mm and settlement of 9mm at ULS. There is therefore minimal shallow liquefaction predicted at the site.

Analysis of CPT data in the area suggests liquefaction induced settlements in the looser sand at depth can range from 5mm to 60mm at SLS and 10mm to 160mm at ULS, as summarized for multiple CPT tests at three sites in Table 5.4. Analysis is with the method by Boulanger & Idriss 2014 with the settlement estimation is by the method of Zhang et al (2002). This method is empirical and approximate only, with perhaps a +100% / -50% margin to the numbers given

		SLS M7.5 0.13g pga	SLS M6 0.19g pga	ULS 0.35g pga	Chch
Site A, 200m SW	Average	15	30	105	100
	range	5 - 35	5 - 60	30-160	25-145
Site B, 100m W	Average	10		45	50
	range	0 - 10		10 - 70	10 - 80
Site B, 150m N	Average	10	20	60	60
	range	5 - 15	5 - 40	40 - 100	35 - 95

Table 5.4 Liquefaction induced settlement (mm) from CPTs (Boulanger)

The effects of the liquefaction at depth will be muted by the overlying denser gravel, with little surface manifestation expected, other than global settlement.

It is to be also noted that most of the liquefiable soils liquefy with peak ground accelerations of about 0.2g (return period of about 100 years). There is only a limited increase in liquefaction and liquefaction induced settlement at higher accelerations. The February 20-11 earthquake should have produced similar liquefaction as for a ULS earthquake and a 2,500 year return period earthquake is predicted to produce only a little more again.

5.5 Lateral spread

The building being located close to the Avon River bank could be at risk of lateral spread should there be extensive liquefaction at a depth where the soils above the liquefied layer at can affected by the lack of restraint along the river bank. At this site the liquefaction below the gravel is at a depth where it should not initiate any lateral spreading, as the gravels above it will be continuous across the river, and thus there is no free face impacted by this deeper liquefaction. The shallow liquefaction could potentially initiate lateral spreading into the river, but only if the liquefaction is continuous and of a thickness where irregularities will not interfere with lateral movement. The evidence suggests that shallow liquefaction is limited in extent both in terms of thickness and in area and there does not appear to be any continuous layer. There is also no evidence of lateral movement reported or observed on the site or along the river bank.

5.6 Earthquake Ground damage observations

The recent earthquakes have subjected the site to strong seismic shaking. Observations of damage to the land and foundations provide very useful information as to what performance might be expected in future large earthquakes.

(a) Canterbury Geotechnical Database

Information collated mainly from EQC inspections[(1)] and held on the Canterbury Geotechnical Database summarises the ground damage as follows:

Sep 2010	no observation – (no ground damage reported to warrant inspection)
Feb 2011	"no observed ground cracking or ejected liquefied material" on site and surrounding area except minor to moderate quantities of ejected material on sites 100m north and 100m southwest (both on far side of river).

Interpretation from aerial photographs by EQC suggest

Sep 2010	no observed liquefaction on site & surrounding area
Feb 2011	minor observed liquefaction on site & surrounding area
June 2011	moderate to severe observed liquefaction on site & all surrounding area (interpretation for this event very often indicates much greater damage than any other record, and should be treated with considerable caution)
Dec 2011	no observed liquefaction on site & surrounding area

---

(1) **CGD Important Notice**

This map and data was prepared and/or compiled for the earthquake Commission (EQC) to assist in assessing insurance claims made under the Earthquake Commission Act 1993 and/or for the Canterbury Geotechnical Database on behalf of the Canterbury Earthquake recovery Authority (CERA). It was not intended for any other purpose. EQC, CERA, and their data suppliers and their engineers, Tonkin and Taylor, have no liability to any used of this map and data or for the consequences of any person relying on them in any way. Each Canterbury Geotechnical Database map and data is made available solely on the basis that any Database user has read and agrees to the terms of use for the Database, any database user has read any explanatory text accompanying this map, and the "Important notice" accompanying the map and data must be reproduced wherever the map and data are reproduced.

There were no ground cracks recorded on the site and in the immediate area by EQC (CGD), but two unclassified cracks are recorded across Worcester Street east of Oxford Tce and one 10 – 50mm crack about 40m long in Oxford Tce south of Worcester Street. While these cracks are all parallel to the river, the sparsity and locations do not suggest any liquefaction related lateral spread.

(b) Aerial photography

An aerial photograph of the site (sourced from Ecan GIS) taken 2 days after the 22 February 2011 earthquake does not show very much that could be interpreted as liquefaction. There is a small area of stain on the footpath about 20m north of the northwest corner of the building that could conceivably be from liquefaction, but nothing resembling the silt and sand deposits evident elsewhere, such as north of Armagh St or south along part of Hereford St..

(c) Site Observations

The site was not inspected following any of the earthquakes, but Worcester Street was walked through a week after the February 2-11 earthquake. A photograph of part of the building was taken but there was nothing of note in the ground that was seen to warrant recording.

The site was not specifically visited until late January 2015, nearly 5 years after the most damaging 22 February 2011 earthquake, and thus much evidence of any ground damage would have been removed or weathered to make its cause uncertain. In the intervening period work has also been done on the site securing the building with props and fencing. No ground damage was noted.

The LDE geotechnical report of July 2011 includes the following:

> *Earthquake damage appears to be almost entirely due to shaking. However, it is noted that there is a very slight tilt of the northwestern corner of the building1 towards the Avon River. A monument some 8m to the north of the building also shows tilting towards the same direction, although to a greater degree. Hairline cracks in the footpath also exist along the northern side of the building indicating that very minor lateral movement has occurred. No ground deformation was observed around the southwestern area of the property.*

(d) Foundations to nearby buildings

Most of the buildings in the vicinity were constructed on shallow foundations. We have not seen specific details of damage sustained in any of these structures but we note that the 12 storey hotel directly opposite the site is still standing although not in use, as is the two storey heavy masonry building across the river; if significant differential settlement had occurred they may well have been demolished by now. The 18 level Clarendon Tower was demolished because of structural issues; to our knowledge foundation performance was not a factor. Although this is of only peripheral relevance to the Municipal Chambers, it does indicate that shallow foundations in the area, some under much larger buildings, have performed adequately with strong seismic shaking.

5.5 Vertical ground movements with recent earthquakes

The Canterbury geotechnical Database (CGD) contains information on vertical ground movement derived from LiDAR surveys of the city before the earthquake sequence and following each of the major events. The cumulative movement from before the first earthquake to April 2012, is shown in Figure 5.1. The yellow shading on the west side and to the northeast of the site indicates 0.1 – 0.2m of vertical settlement; the light orange to the north and south indicates 0.2 – 0.3m of settlement. Larger settlement is indicated of the fill to the abutments on both sides of the Worcester St bridge.

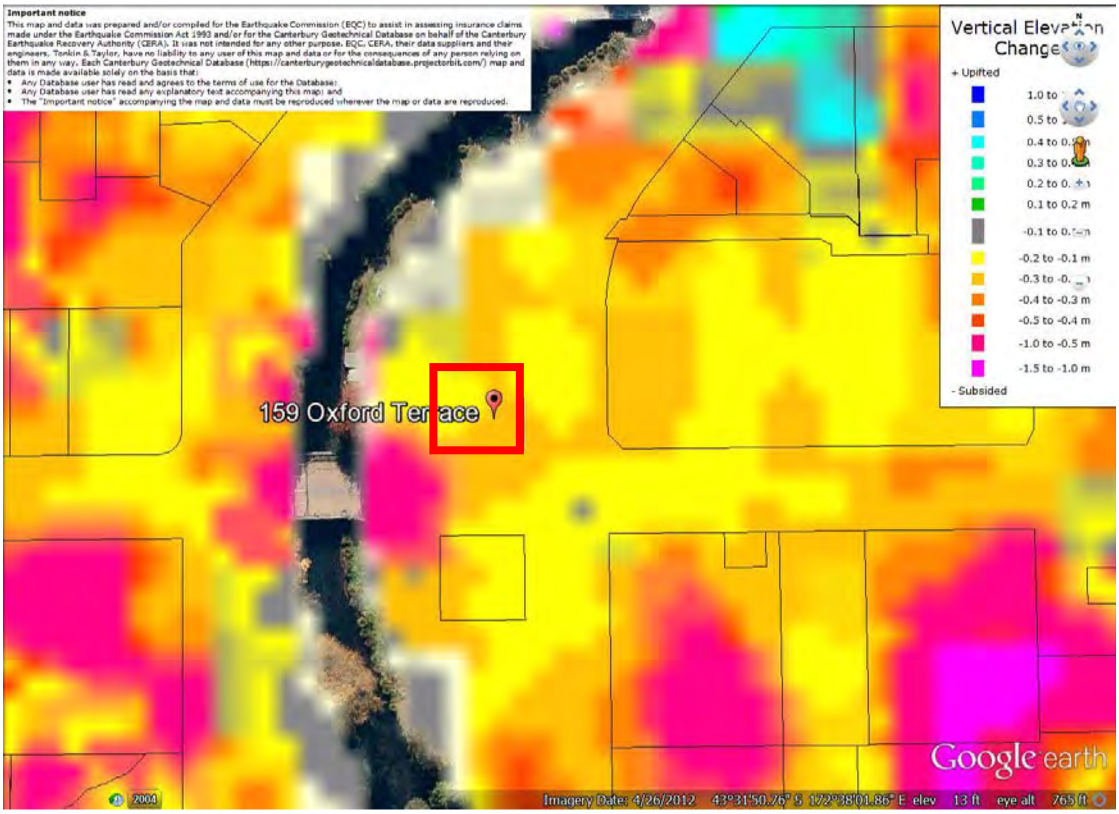


Figure 5.1 Cumulative vertical movement from all earthquakes, from LiDAR surveys
CGD Important Notice – see footnote p20

It must be noted that the LiDAR surveys have some error associated with them, and that the elevations as used to calculate the changes are from manipulated data to remove buildings, vehicles etc to give a ground surface. Some of the areas which appear to show high settlements are probably related to this manipulation and are actually associated with the removal of buildings and the like. There is no marked trend visible however, with settlements in the area appearing to be relatively uniform.

Property surface elevation data from the LiDAR surveys as held on the CGD Property summary report shows the level data in Figure 5.5. What is apparent is that the site data indicates a rise in level, although even the tectonic change in bedrock level suggests 130mm subsidence. It appears that the site has not undergone any significant vertical ground deformation.

Survey Dates:	Pre earthquake	5 Sep 2010	8 Mar-30 May 2011	18 Jul-3 Sep 2011	17-18 Feb 2012
95 percentile		4.52	4.99	4.90	4.84
Mean elevation		4.38	4.45	4.60	4.44
5 percentile		4.23	4.14	4.15	4.18
Cumulative Tectonic change		-0.01	-0.11	-0.13	-0.13

Table 5.5 Surface elevation changes (CGD property summary report)

6 Foundations

The soil profile is relatively good for Christchurch. Under normal static conditions, shallow foundations would be appropriate for even quite large new structures. However, seismic conditions present some hazard with the presence of liquefiable sands between the underside of the gravel at 7 - 10m and the top of the Riccarton gravels at 22m.

6.1 Existing Foundations

Drawings of the building include a plan of the 1886 foundations, reproduced here as Figure 6.1. All the brick walls are supported on concrete strip footings. The majority of these are understood to be 0.6m wide, but some internal footings appear to be somewhat narrower. Cross sections of the building show these foundations with shallow piles to the timber floors internal to the rooms.

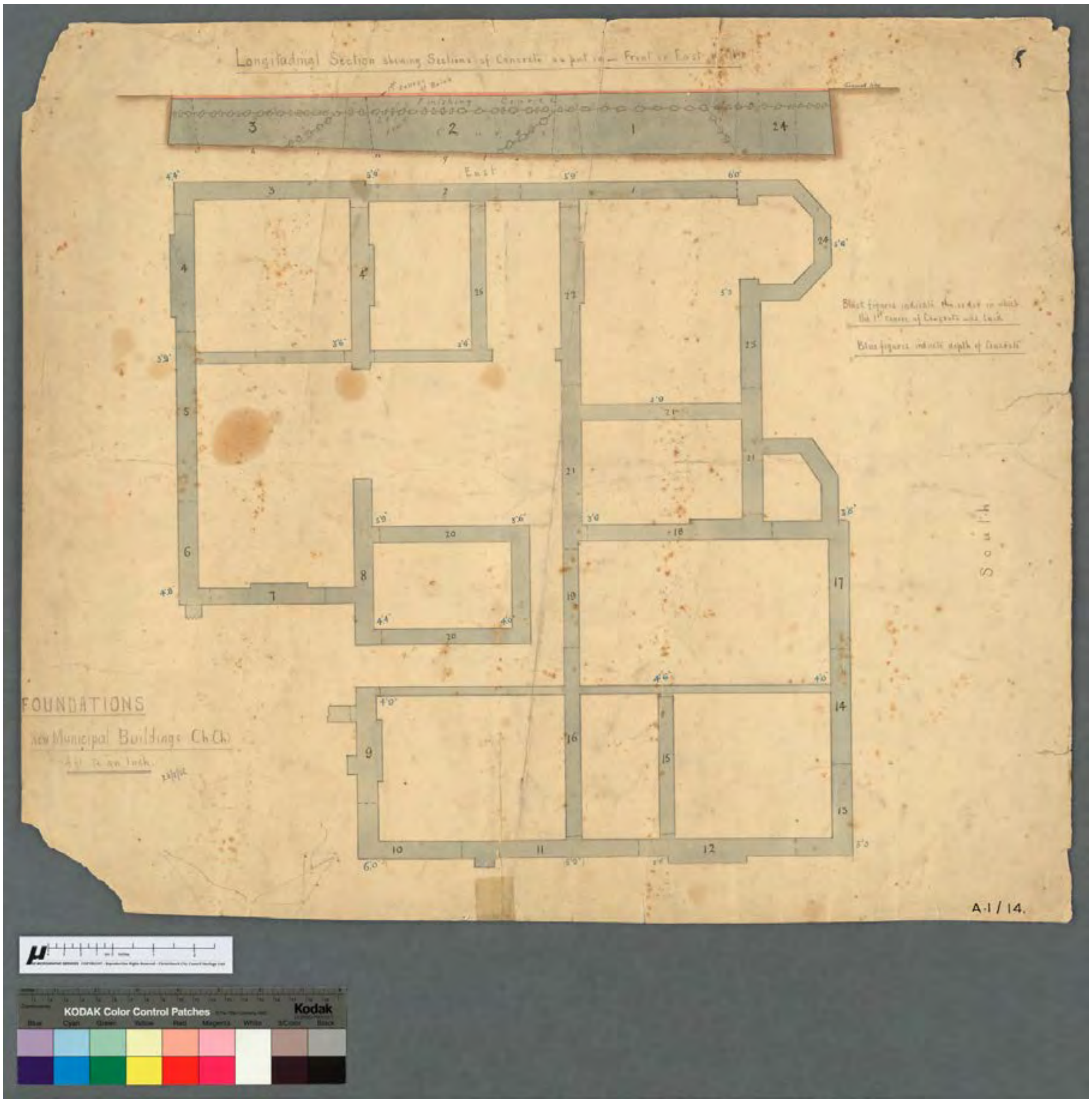


Figure 6.1 – Plan of 1886 Foundations

The Conservation Report of 2000 reports that "continuous mass concrete foundations approximately 600mm wide by 600mm deep support the walls of the building", but this is likely to be a layer of "finishing" concrete placed over the top of a concrete base.

The section of the eastern footing along the Oxford Terrace side (at the top of Figure 6.1) details 3 pours of concrete below the "finishing" concrete. Numbers on the plan indicate the foundation was poured in 24 sections below the finishing layer, which is presumably the 0.6m by 0.6m foundation referred to in the conservation report. The drawings also suggest a taper to the foundations, so that the base may well be wider than 0.6m.

The plan has dimensions noted at intervals and these appear to be the depth of footing. Depths through the middle of the building (north to south) are consistently shallowest at 1.05 – 1.15m (3 ft-6inch to 3ft-9 inch), but increase to both the east and west walls. The east wall varies from 1.3m at the north end to 1.8m at the south and the west wall varies from 1.6m at the south to 1.8m at the north. The depths are presumably from the top of concrete which is generally close to ground level. The depth of footings can be compared with the depth to gravel as determined from the CPT tests, as shown in Table 6.1.

Test	Location	Depth to gravel	Depth of footing
CPT 01	5.7m off NW cnr	1.0	1.8
CPT 02R	1.7m off SW cnr	2.4	1.6
CPT 03	4m off NE cnr	1.3	1.3
CPT 04	6.3m off SE cnr	1.35	1.8

Table 6.1 CPT tests adjacent to OMC Building

The depths do not match particularly well, but do suggest that the foundations were taken to found directly on the gravel. This was the practice on other important heavy masonry buildings at the time, including what is now the Art Centre and the stone Provincial Chamber.

There is also a small basement under the southwest part of the building. Details of this structure have not been seen, other than a plan, but if it provides standing room, then the floor must be founded at a level similar to the original foundations. This is the only part of the building where the CPOT tests suggests that the foundation may not be bearing directly on gravel, but the basement, if ties into the adjacent foundation, provides a much larger effective footing in this area.

6.2 Observations of Existing Foundation Performance

Safety hazards and the amount of propping around the building has limited access, and only a limited inspection of the building has been made, particularly of the east and south sides which were viewed through the fence only. No cracking in the brickwork suggestive of differential settlement has been observed. There is no indication of any differential movement between the building and the adjacent ground. We note that LDE concluded that "*Earthquake damage appears to be almost entirely due to shaking.*"

Opus have provided some levels taken off post-earthquake 3D scans of the building. Analysis of horizontal architectural lines on parts of the external walls indicate the following differences in level.

Building wall	Level differential (mm)	Length over differential (m)
East elevation	9 (down to south)	14.3
West elevation	6 (down to north)	13.2
South elevation	26 (down to east	9.4
North elevation	12mm (down to east)	12.0

Table 6.2 Level differences from 3D scan (Opus)

Scans of the building floors and ceilings are somewhat ambiguous but do suggest some fall from west to east with as much as 50 – 60mm on the southern side. Floor levels in the Chamber indicate some fall from southwest to northeast, of perhaps a similar magnitude.

Wall inclination surveys by 43below show the external walls all out of plumb, but the results do not appear to form a coherent pattern or consistent tilt in any one direction. 3D scans show a tilt of the west wall out towards the river, which is inconsistent with the level trends of the building being down on the east side. Levels on the Kate Shepherd Memorial Wall, a short distance to the north of the building shows that there is about a 60mm difference in level along its length, with the east end low, ie a slight tilt away from the river, consistent with the trend on the building.

The 1987 Building Survey report found no evidence of settlement and concluded that “the foundations were adequate for supporting the design loads of the building.”

It appears that the foundations have not been greatly damaged by the earthquake with little obvious settlement or stretching. There does seem to be some settlement of the building with movement down on the east side, as indicated by the 3D scans. The vertical ground deformations derived from LiDAR surveys (Figure 5.1) does suggest a small increase in vertical change to the east. We have clear evidence on three other sites underlain with the same general soil profile and all within 300m of the Municipal Chambers, of general ground deformations unrelated to building foundations. On this site the apparent trend to the building and differential on the Kate shepherd memorial Wall suggest that this has also occurred here. This deeper seated deformation in the ground is unrelated to the building foundations which have simply followed the underlying ground movement.

6.3 Re-levelling

If the building is determined to be out of level, and it is out of level to a degree that necessitates re-levelling, techniques are available that can achieve this. Although mechanical lifting could be possible, the most likely approach would be to use grouting techniques. Multiple grouting points to under the load bearing foundations can be used and with suitable control can lift the structure evenly. The gravel under the foundations provides a good bearing to provide the reaction for lifting. No remedial work to prevent future settlement is needed, if our conclusion that any settlement that has occurred is the result of deep seated ground deformation and not problems with the foundations or the soils immediately below them.

7 Restoration Foundation Options

7.1 Considerations for foundation work

Options to reinstate the Old Municipal chambers are likely to involve strengthening work to the superstructure that will impose increased loads on the foundations, and thus some new foundation construction is likely to be needed. Some general principles for the design are:

(a) Compatibility of foundation systems

As a general principal, it is good practice to use only one foundation system on any one building. Mixing piles and shallow footings, or even piles of significantly different lengths has resulted in differential movement and damage to many buildings. If there is a move away from shallow foundations for some parts of the building, the implications for movement between the parts and the risks entailed should be carefully considered.

Given the apparently good performance of the ground and foundations to date, there is little reason to move away from the continued use of shallow foundations. In addition, any new foundations should be founded on the gravel layer underlying the site at a similar depth to the existing foundations to provide similar stiffnesses and bearing.

If the strengthening requires large uplift loads to be resisted, the first preference is probably to use concrete mass, but failing that, uplift anchors with a link that will take only limited compression load, rather than a pile.

(b) Future risk

The evidence from the recent earthquakes and the building history indicates that the site and the existing foundations have performed well with little differential settlement or lateral movement. A large earthquake in the future could result in somewhat greater ground movement and thus it would be desirable to restore the buildings on foundations that are less susceptible to damage from ground movement, but this needs to be balanced against cost and maintenance of heritage values.

Given the performance of the foundations and the site no special consideration need be given for lateral spread. However, it is standard good practice to tie any foundation system together, and the addition of elements with tension capacity across the building would clearly be desirable, if this is practical.

(c) Heritage

The Old Municipal Chambers Building is a category 1 historic structure and therefore maintenance of the heritage values is of great importance. This will have to be balanced with the engineering requirements to restore the buildings with sufficient structural strength and the practicalities of construction.

(d) Pile Foundations

A piled foundation has been suggested in a previous report. We have reservations about this as an option. It appears to have been assessed from a degree of shallow liquefaction which in our opinion is quite unlikely to occur, and which shows little evidence of having occurred with shaking approaching ULS conditions. In addition:

- The whole building would have to be piled. Installing piles under the existing masonry structure would clearly entail the use of piles on each side of each wall, with pile caps between them and under new reinforced concrete footings able to span between the piles. This would clearly be a major undertaking with great disruption to the whole of the ground floor.
- Piles into the shallow gravel would only mitigate any liquefaction / settlement in the shallow sands and in liquefiable lenses within the top of the gravel. The structure would still be subject to settlement from deeper liquefaction (as appears to have happened with the apparent settlement of the eastern side relative to the west). If any relevelleing was ever needed, it is a lot more difficult with piles unless they are set into sockets to allow the foundation beams to be lifted off the piles.
- Piles concentrate load into small discrete volumes within the soil. In some ways this increases the risk of differential settlement if a pile of piles ends up in or just above a liquefiable sand lens or loose gravel that then causes the pile tip to settle. A good quality control of additional soil testing and monitoring of the pile installation would be needed.

Given the above, we have not considered piles further in this report and the following sections refer only to shallow foundation design.

Design should consider two criteria: bearing failure capacity (ULS) and bearing pressures to limit settlement (SLS).

7.2 Shallow Foundation bearing pressures

Shallow footing design for STATIC conditions can be based on the following. Ultimate bearing pressures should be limited to those values shown in Table 7.1 for bearing on the existing natural gravel soils. These ultimate stresses should be reduced by a capacity reduction factor to give values of "allowable ultimate" bearing stresses to be used with fully factored loads in accordance with NZS 4203:1992.

Footing Shape	Footing width	Depth of footing	
		1.0	1.5
Square Pad	Less than 0.8m	q = 770 + 190B	q = 1200 + 80B
	Greater than 1m	q = 860 + 80B	q = 1200 + 80B
Strip footing	Less than 0.8m	q = 460 + 310B	q = 770 + 140B
	Greater than 1m	q = 620 +140B	q = 770 + 140B

Notes:
- B = width of footing
- Depth of footing is depth below lowest adjacent ground level.
- Water table is assumed to be below 1.9m depth.
- Soil parameters $\Phi = 33^0$, c = 0
- Use capacity reduction factor of 0.5, including seismic overstrength.

Table 7.1 Ultimate bearing pressures for footings at Ground Level (kPa)

A capacity reduction factor of 0.5 should be used for all load combinations. Previously a value of 0.8 was allowed for earthquake overstrength conditions, but the behaviour of soils in the Christchurch earthquake has brought this into question, as it is likely that at least some soils will be softened by high pore pressures and actually be weaker at the time of highest demand. A value of 0.5 is recommended for all load combinations.

7.3 Settlement

Application of load will lead to settlement under footings. Bearing stresses calculated from SLS load combinations should generally be limited to less than the values shown in Figure 7.1, assuming the footings are bearing directly onto gravel soils.

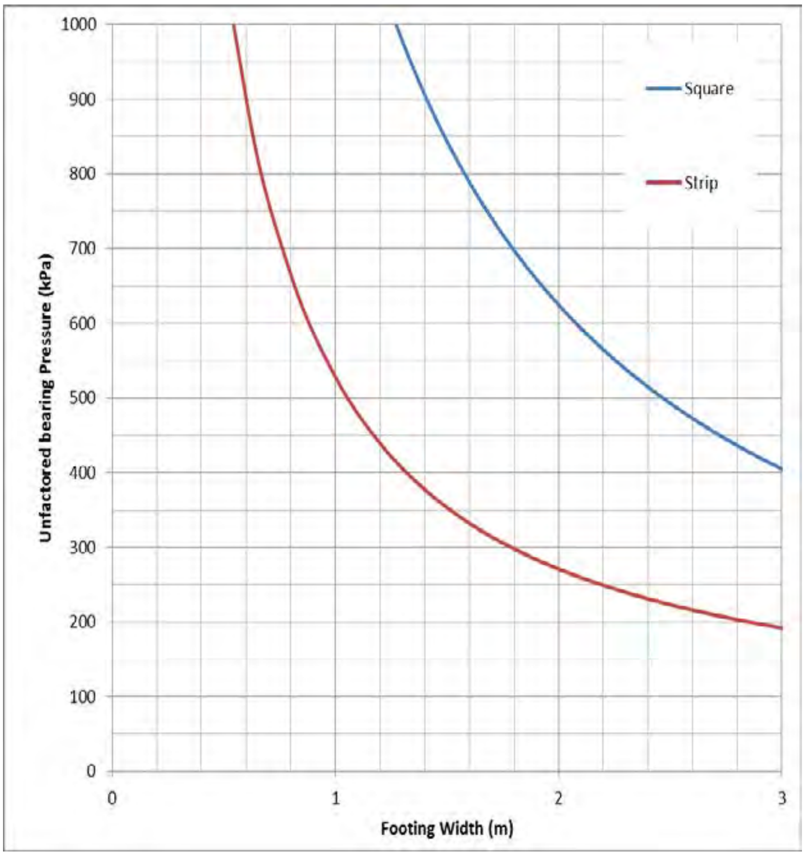


Figure 7.1
SLS bearing pressures (kPa) under shallow footings at 1.0m depth to limit settlement to 25mm

The bearing values should limit settlement to less than 25mm for strip and square footings. Stresses to limit settlement to values other than 25mm can be estimated by multiplying the values from the figure by the ratio of the settlement to 25mm.

The bearing stresses in Figure 7.1 are very high for smaller width footings. We recommend that an upper limit of 300 kPa be applied for unfactored loads.

The cohesionless nature of most of the soil column under the site means that settlement should occur within a short time of the load being applied.

The values in Figure 7.1 are for normal static conditions. Greater and unpredictable settlements must be expected with strong earthquake shaking sufficient to cause liquefaction of susceptible layers or consolidation of loose gravel lenses under the site

The existing footings will have settled under the weight of the building. Any new footings will undergo some settlement as the load is applied. Given the stiff nature of the sugbgrade soils under static conditions, such movements should be able to be kept small, but some settlement is probably inevitable under earthquake conditions when large seismic loads will be imposed on footings for the first time.

7.4 Subgrade stiffness

Modulus of subgrade reaction has been assessed for the site. This parameter is hard to determine and various methods give a range of between 4 and 35 MPa/m for a 2m square footing. It is recommended that a sensitivity analysis be done when using the modulus of subgrade reaction to provide stiffnesses for springs in a foundation model.

Footing width (m)	0.5	1	2	3	4
Square - lower	50	20	11	8.5	7
Square - upper	80	50	25	20	18
Strip – lower	20	11	8.5	7	6
Strip - upper	50	25	20	18	16

Table 7.2 Modulus of Subgrade Reaction (MPa/m)

8 Limitations

The subsurface conditions and the interpretations reported are those identified at the locations of the investigations at the time of the investigation and are subject to the limitations of the investigation methods. The borelogs are an engineering/geological interpretation of the subsurface conditions dependent on the method and frequency of sampling and testing. The boreholes represent only a very small sample of the total subsurface soils. The interpretation of the information and its application must take into account the spacing of the boreholes, the frequency of sampling and testing and the possibility of undetected variations in soils.

The Geotechnical model and analysis for this report is based in part on testing carried out by other consultants without any input by Geotech Consulting Ltd. Geotech Consulting Ltd does not accept any liability for the accuracy or otherwise of the data used.

While care has been taken with the report as it relates to interpretation of subsurface conditions, and recommendations or suggestions for design and construction, Geotech Consulting Ltd cannot anticipate or assume responsibility for unexpected variations in ground conditions or the actions of contractors. If conditions encountered on site during construction appear to vary from those, which can be expected from the information, contained in this report, Geotech Consulting Ltd requests that it be notified immediately.

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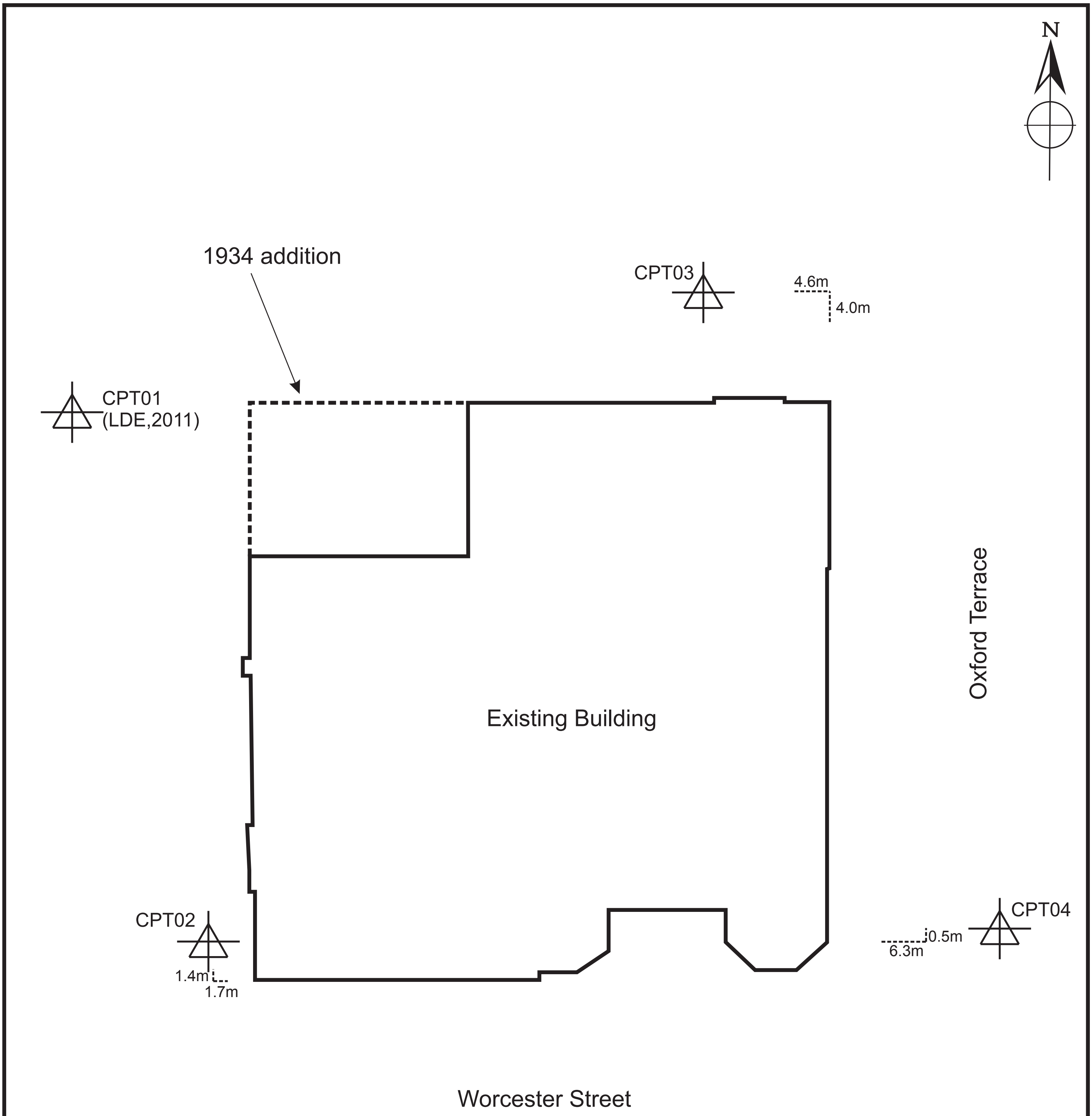
Appendix

Figure 1 Site Investigation Plan

Cone Penetration Tests

CPT01 (LDE, 2011)

CPT02, 03, 04 (GCL, 2016)

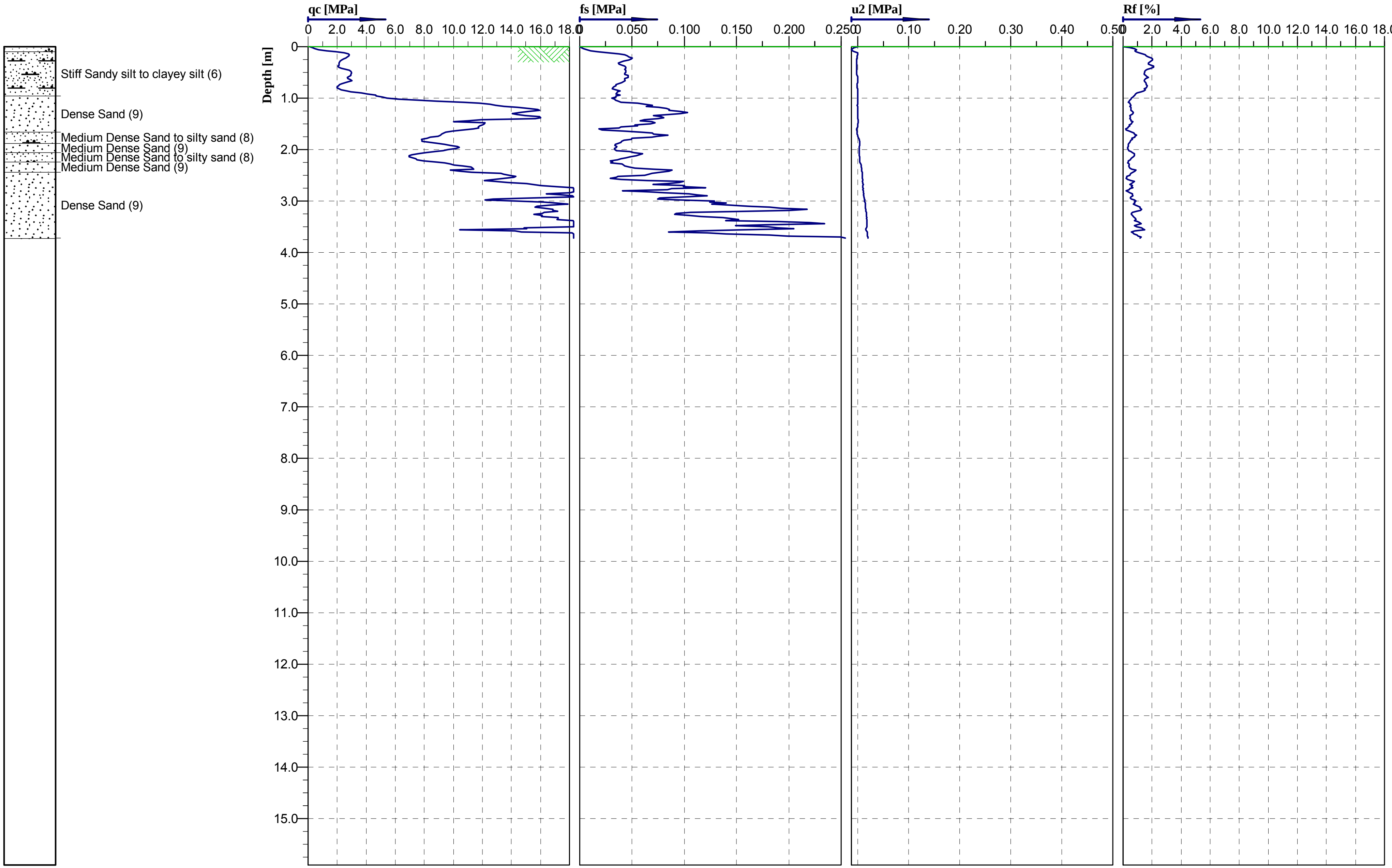


GEOTECH

SITE INVESTIGATION PLAN

Old Municipal Chambers

SCALE:	**1:200 (Aprox.)**
DRAWN BY:	**YUY**
DATE DRAWN:	**12/02/2016**
CHECKED BY:	**IMC**
FIGURE No .:	**4840**

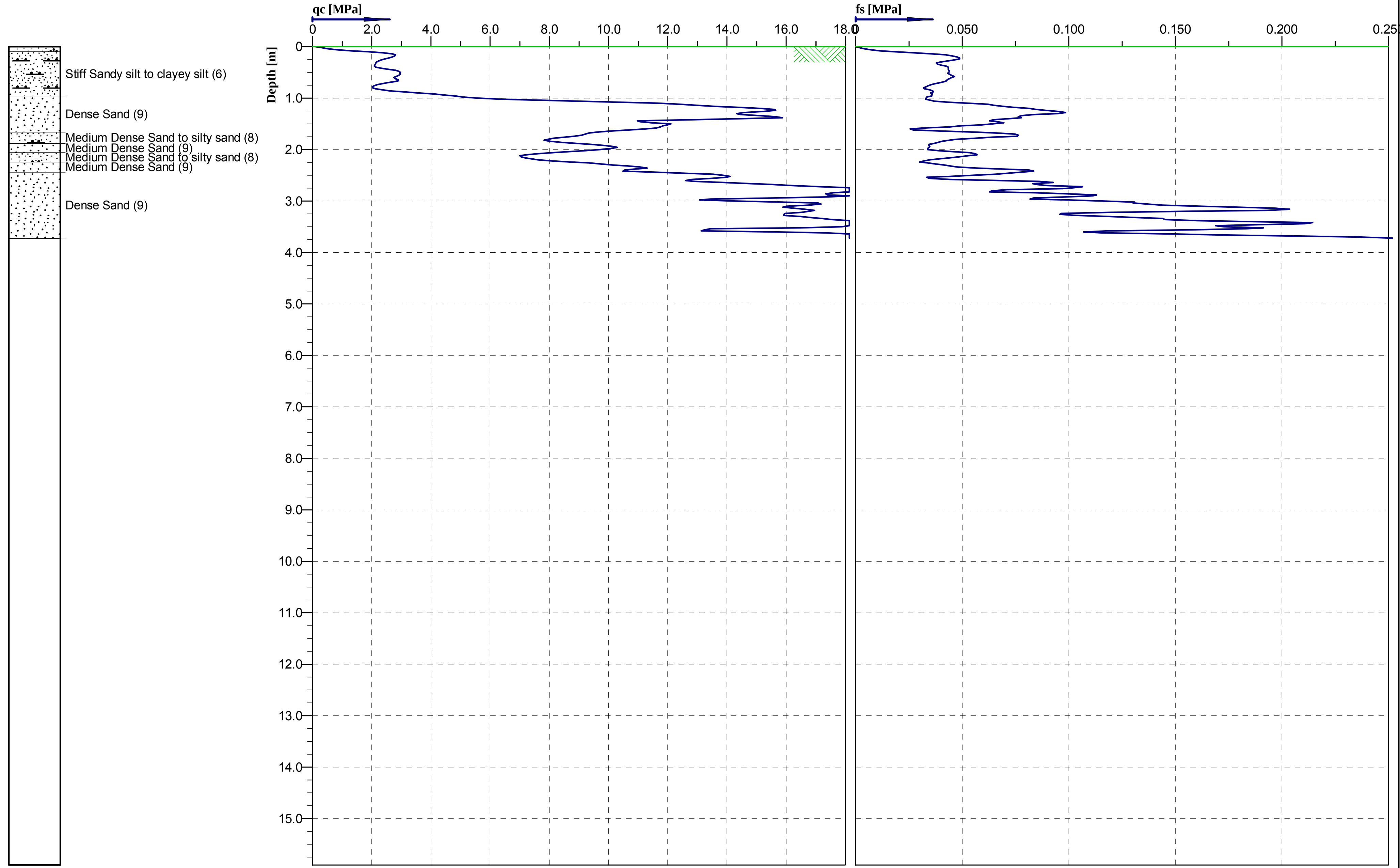


Location: 159 OXFORD TCE, CHRISTCHURCH CITY	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no:
Project ID: 10048	Client: LAND DEVELOPMENT & EXPLORATION LTD	Date: 8/3/2011	Scale: 1 : 100
Project: OLD MUNICIPAL CHAMBERS		Page: 1/1	Fig:
		File: cpt01.001	

BROWN BROS DRILLING

U2

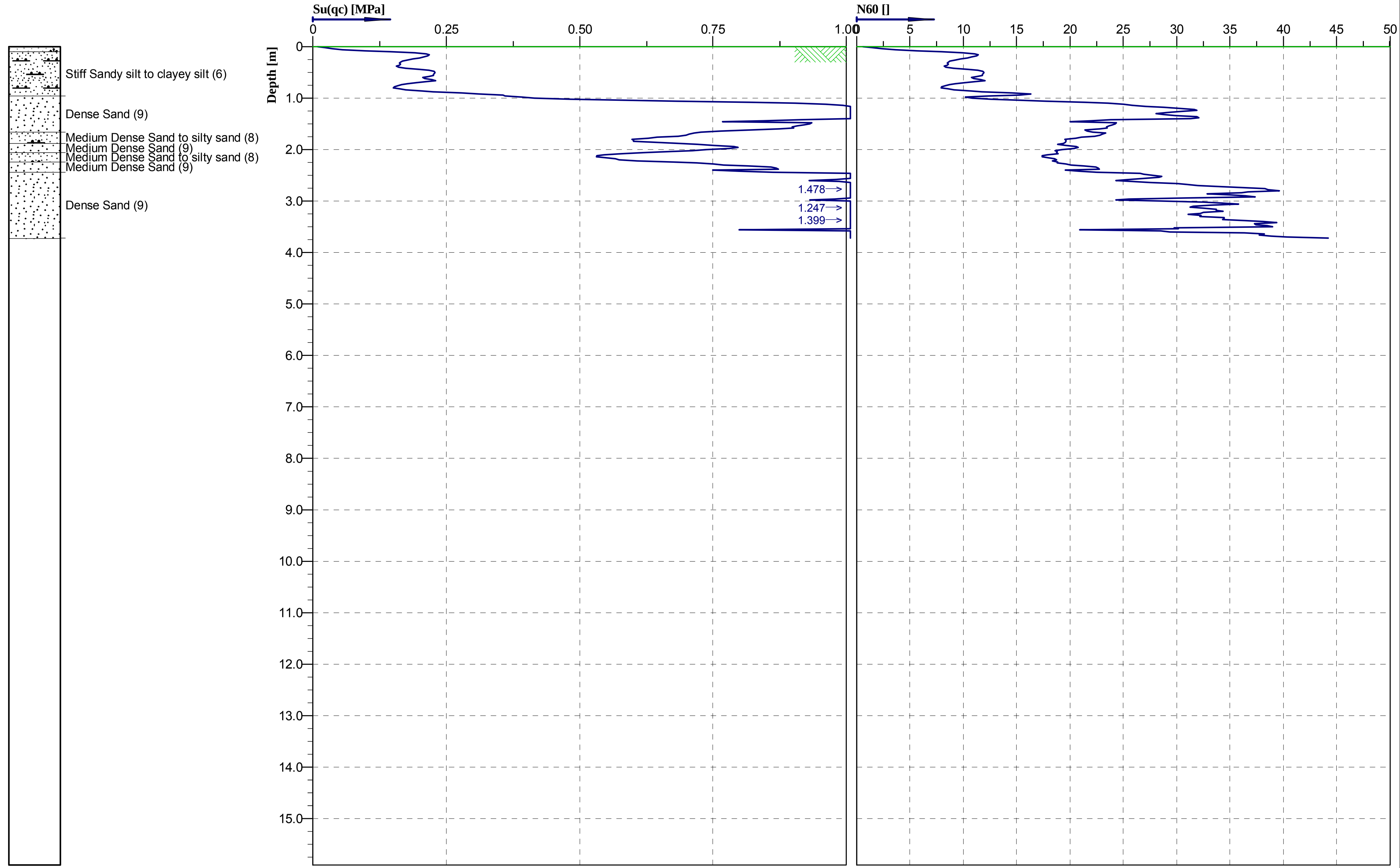
Cone No: 4355
Tip area [cm2]: 10
Sleeve area [cm2]: 150



U2

Cone No: 4355
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: 159 OXFORD TCE, CHRISTCHURCH CITY	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no:
Project ID: 10048	Client: LAND DEVELOPMENT & EXPLORATION LTD	Date: 8/3/2011	Scale: 1 : 100
Project: OLD MUNICIPAL CHAMBERS		Page: 1/1	Fig:
		File: cpt01.001	



Cone No: 4355
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: 159 OXFORD TCE, CHRISTCHURCH CITY	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no:
Project ID: 10048	Client: LAND DEVELOPMENT & EXPLORATION LTD	Date: 8/3/2011	Scale: 1 : 100
Project: OLD MUNICIPAL CHAMBERS		Page: 1/1	Fig:
		File: cpt01.001	

CONE PENETRATION TEST

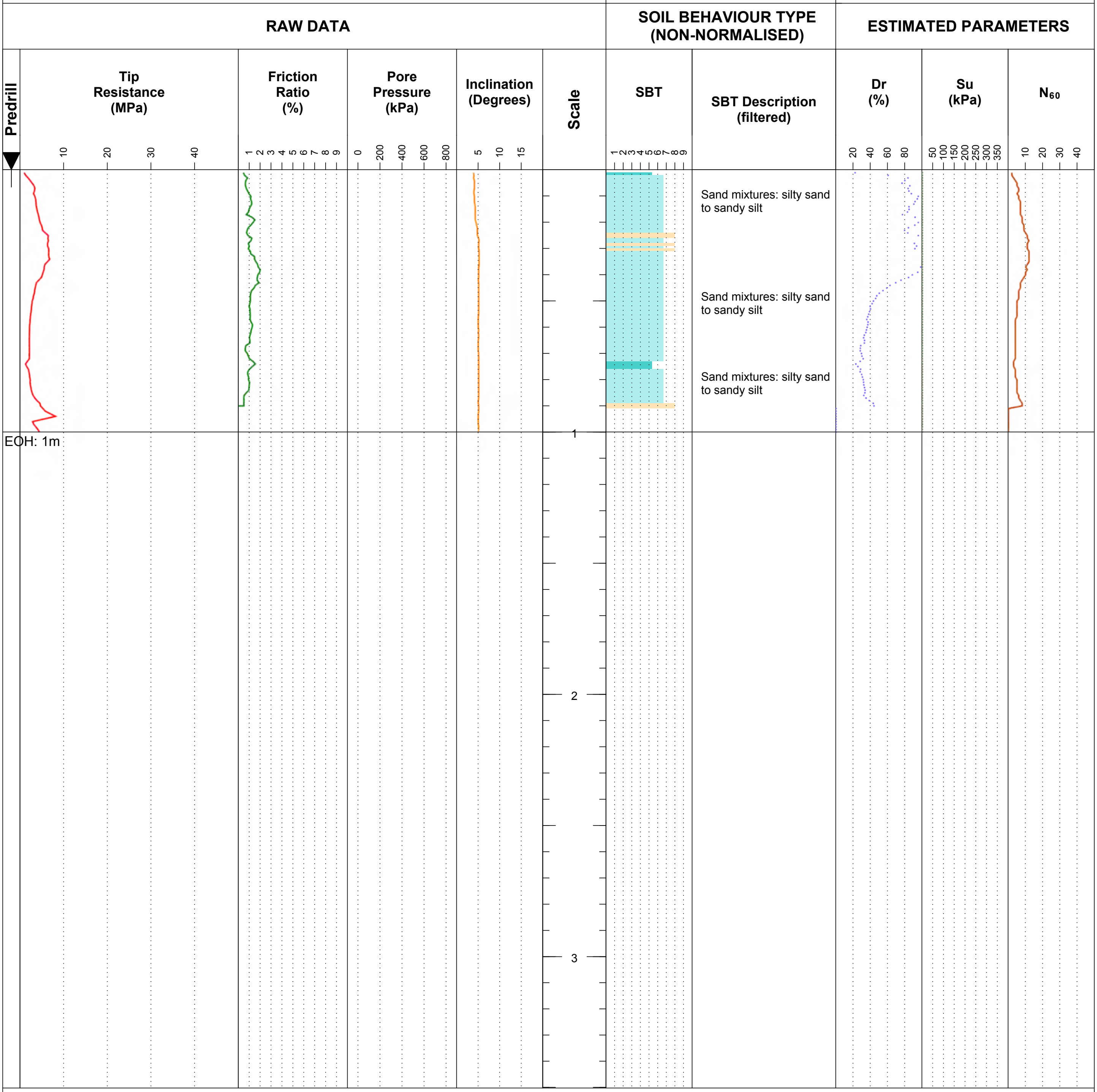
McMILLAN Drilling GROUP

Job: 15640
CPT No.: CPT002

Name: Old Municipal Chambers
Client: Geotech Consulting Ltd
Location: 159 Oxford Terrace, Christchurch

Grid: NZTM
Datum: Ground
Termination: -

North (m): 5180169.46
East (m): 1570382.11
Elevation (m): -
Hole Depth (m): 1.00



Operator: S. Cardona
Rig: AMS VTR9100 - track
Cone Reference: 110542T
Cone Area Ratio: 0.75
Cone Type: I-CFXY-10
Tip Resistance (MPa) Initial: -0.1327 **Final:** -0.1312
Local Friction (MPa) Initial: -0.0079 **Final:** -0.008
Pore Pressure (KPa) Initial: - **Final:** -

Date: 09/02/2016
Predrill: 0.00
Water Level: -
Collapse: 0.70

Effective Refusal
Tip:
Gauge:
Inclinometer:
Other: Anchor
Target Depth:

Soil Behaviour Type (SBT) - Robertson et al. 1986

0	Undefined	5	Sand mixtures: silty sand to sandy silt
1	Sensitive fine-grained	6	Sands: clean sands to silty sands
2	Clay - organic soil	7	Dense sand to gravelly sand
3	Clays: clay to silty clay	8	Stiff sand to clayey sand
4	Silt mixtures: clayey silt & silty clay	9	Stiff fine-grained

Notes & Limitations

Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P. K. Robertson and K.L. Cabal (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 4th Edition. The interpretations are presented only as a guide for geotechnical use, and should be carefully reviewed by the user. Both McMillan Drilling Ltd & Geroc Solutions Ltd do not warrant the correctness or the applicability of any of the geotechnical soil and design parameters shown and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used to derive data shown in this report.

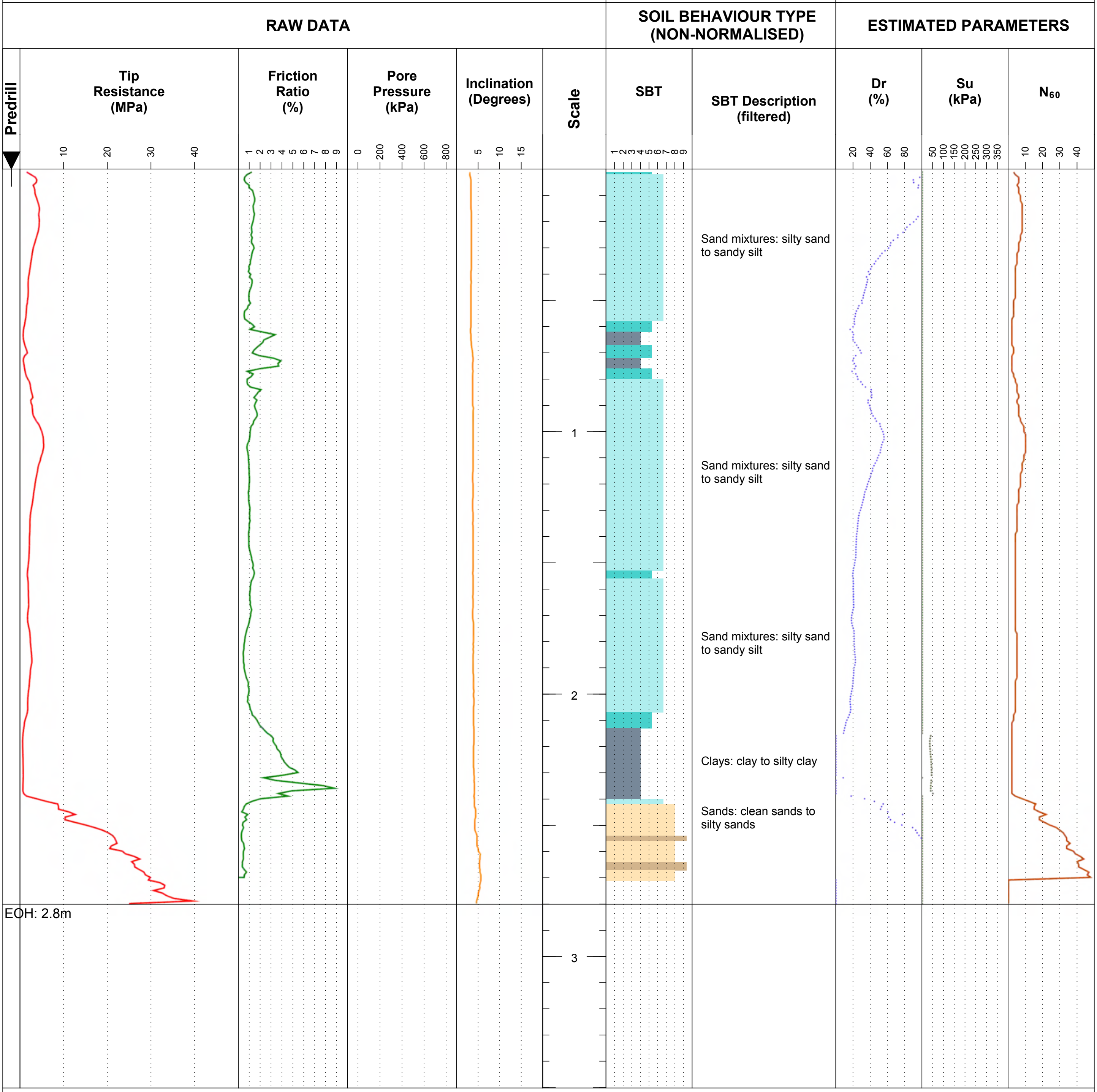
Remarks

Effective Refusal

Hole Depth (m): 1.00

McMILLAN Drilling	# CONE PENETRATION TEST	Job: 15640
		CPT No.: CPT002R

Name: Old Municipal Chambers **Client:** Geotech Consulting Ltd **Location:** 159 Oxford Terrace, Christchurch	**Grid:** NZTM **Datum:** Ground **Termination:** -	**North (m):** 5180169.91 **East (m):** 1570381.80 **Elevation (m):** - **Hole Depth (m):** 2.80



Operator: S. Cardona	**Date:** 09/02/2016	**Effective Refusal**	
Rig: AMS VTR9100 - track	**Predrill:** 0.00	**Tip:** ✓	
Cone Reference: 110542T	**Water Level:** -	**Gauge:**	
Cone Area Ratio: 0.75	**Collapse:** 1.60	**Inclinometer:**	
Cone Type: I-CFXY-10		**Other:**	
Tip Resistance (MPa) Initial: -0.1102	**Final:** -0.0992		
Local Friction (MPa) Initial: -0.0098	**Final:** -0.0103		
Pore Pressure (KPa) Initial: -	**Final:** -	**Target Depth:**	

Soil Behaviour Type (SBT) - Robertson et al. 1986

0 Undefined	5 Sand mixtures: silty sand to sandy silt
1 Sensitive fine-grained	6 Sands: clean sands to silty sands
2 Clay - organic soil	7 Dense sand to gravelly sand
3 Clays: clay to silty clay	8 Stiff sand to clayey sand
4 Silt mixtures: clayey silt & silty clay	9 Stiff fine-grained

Notes & Limitations

Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P. K. Robertson and K.L. Cabal (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 4th Edition. The interpretations are presented only as a guide for geotechnical use, and should be carefully reviewed by the user. Both McMillan Drilling Ltd & Geroc Solutions Ltd do not warranty the correctness or the applicability of any of the geotechnical soil and design parameters shown and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used to derive data shown in this report.

Remarks

Effective Refusal

Hole Depth (m): 2.80

CONE PENETRATION TEST

McMILLAN Drilling GROUP

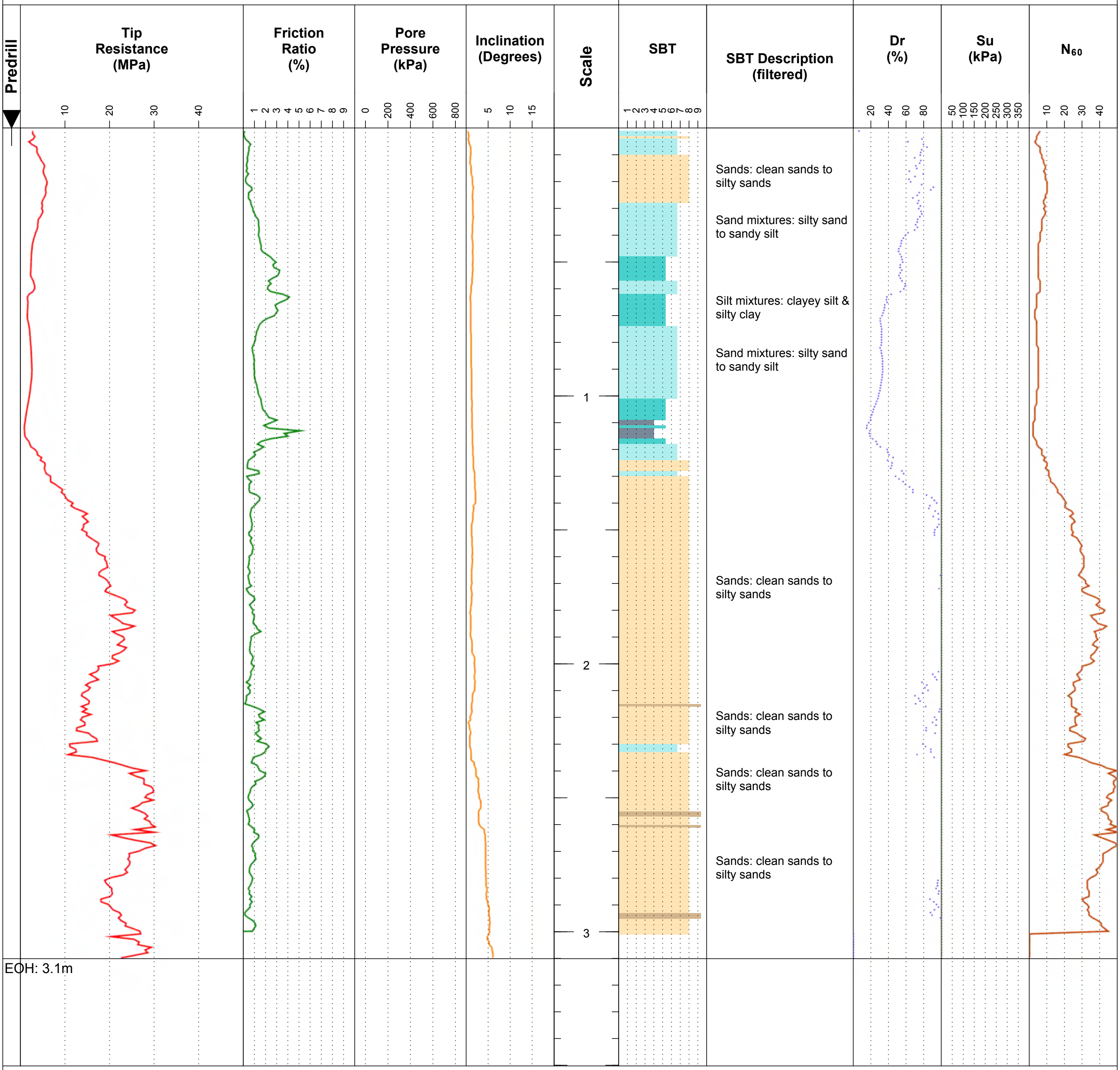
Job: 15640
CPT No.: CPT003

Name: Old Municipal Chambers
Client: Geotech Consulting Ltd
Location: 159 Oxford Terrace, Christchurch

Grid: NZTM
Datum: Ground
Termination: -

North (m): 5180190.39
East (m): 1570399.96
Elevation (m): -
Hole Depth (m): 3.10

RAW DATA	SOIL BEHAVIOUR TYPE (NON-NORMALISED)	ESTIMATED PARAMETERS



Operator: S. Cardona
Rig: AMS VTR9100 - track
Cone Reference: 110542T
Cone Area Ratio: 0.75
Cone Type: I-CFXY-10
Tip Resistance (MPa) Initial: -0.096 **Final:** -0.0915
Local Friction (MPa) Initial: -0.0102 **Final:** -0.0103
Pore Pressure (KPa) Initial: - **Final:** -

Date: 09/02/2016
Predrill: 0.00
Water Level: -
Collapse: 1.70

Effective Refusal
Tip: ✔
Gauge:
Inclinometer:
Other: Anchor
Target Depth:

Soil Behaviour Type (SBT) - Robertson et al. 1986

0	Undefined	5	Sand mixtures: silty sand to sandy silt
1	Sensitive fine-grained	6	Sands: clean sands to silty sands
2	Clay - organic soil	7	Dense sand to gravelly sand
3	Clays: clay to silty clay	8	Stiff sand to clayey sand
4	Silt mixtures: clayey silt & silty clay	9	Stiff fine-grained

Notes & Limitations

Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P. K. Robertson and K.L. Cabal (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 4th Edition. The interpretations are presented only as a guide for geotechnical use, and should be carefully reviewed by the user. Both McMillan Drilling Ltd & Geroc Solutions Ltd do not warranty the correctness or the applicability of any of the geotechnical soil and design parameters shown and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used to derive data shown in this report.

Remarks

Effective Refusal

Hole Depth (m): 3.10

McMILLAN Drilling GROUP

CONE PENETRATION TEST

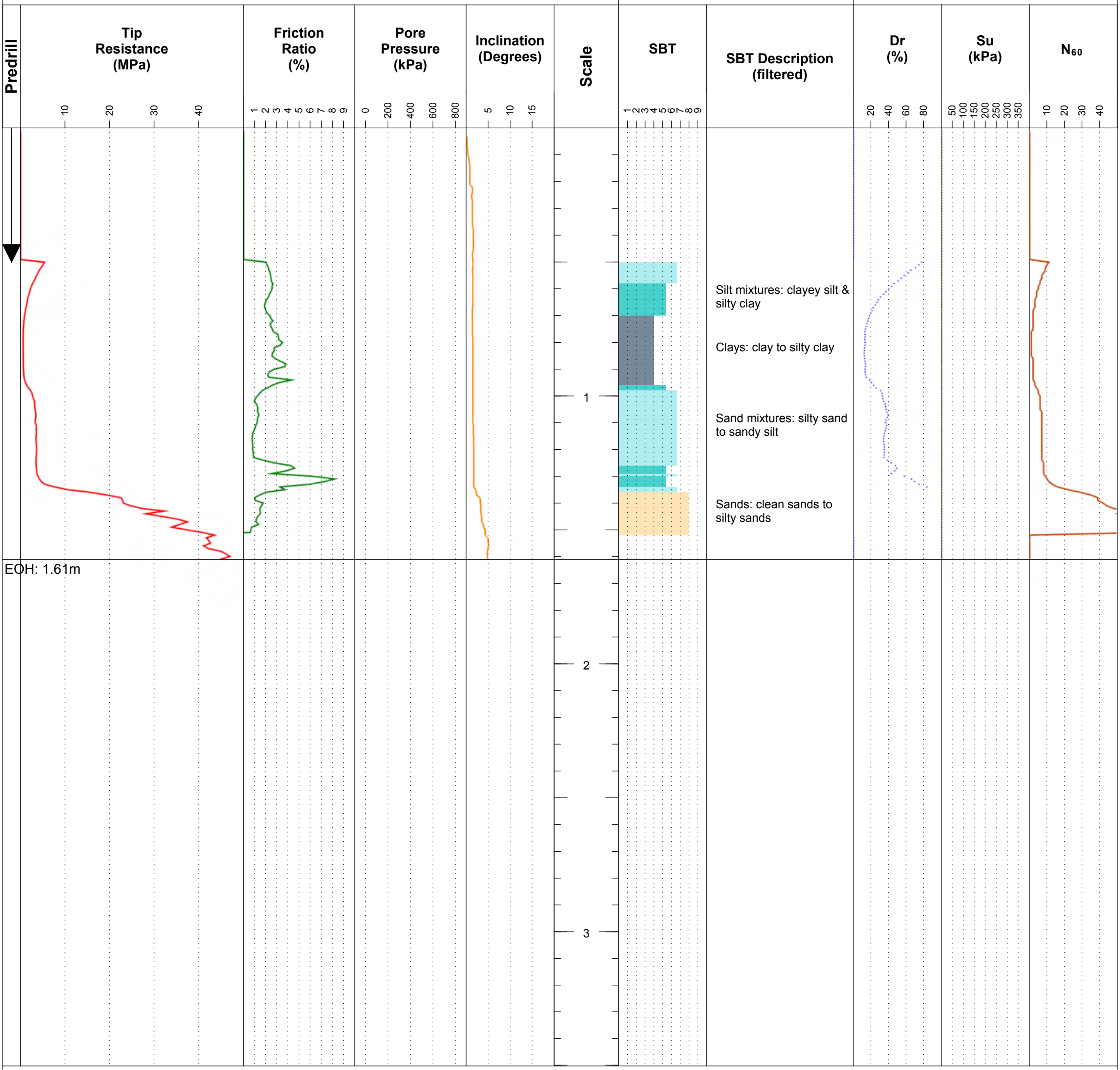
Job: 15640
CPT No.: CPT004

Name: Old Municipal Chambers
Client: Geotech Consulting Ltd
Location: 159 Oxford Terrace, Christchurch

Grid: NZTM
Datum: Ground
Termination: -

North (m): 5180171.39
East (m): 1570409.36
Elevation (m): -
Hole Depth (m): 1.61

RAW DATA	SOIL BEHAVIOUR TYPE (NON-NORMALISED)	ESTIMATED PARAMETERS



Operator: R. Wyllie
Rig: 14t truck mounted rig
Cone Reference: 110542T
Cone Area Ratio: 0.75
Cone Type: I-CFXY-10
Tip Resistance (MPa) Initial: 0.0259 **Final:** -0.0847
Local Friction (MPa) Initial: -0.0104 **Final:** -0.0101
Pore Pressure (KPa) Initial: - **Final:** -

Date: 09/02/2016
Predrill: 0.50
Water Level: -
Collapse: 1.50

Effective Refusal
Tip:
Gauge: ✔
Inclinometer:
Other:
Target Depth:

Soil Behaviour Type (SBT) - Robertson et al. 1986

0	Undefined	5	Sand mixtures: silty sand to sandy silt
1	Sensitive fine-grained	6	Sands: clean sands to silty sands
2	Clay - organic soil	7	Dense sand to gravelly sand
3	Clays: clay to silty clay	8	Stiff sand to clayey sand
4	Silt mixtures: clayey silt & silty clay	9	Stiff fine-grained

Notes & Limitations

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Remarks
Effective Refusal

Hole Depth (m): 1.61