

Subdivision of 254-256 Fitzgerald Avenue Richmond Christchurch

Geotechnical Assessment Report

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	PREPARED FOR:	Ms R Harwood
	PREPARED BY:	Geotech Consulting Ltd
	ENQUIRIES TO:	Andrew Hurley
		ahurley@geotech.co.nz 027 479 1516
Issue	Date	Author
01	23 February 2021	A Hurley

Dr. Mark Yetton Nick Traylen Ian McCahon

email myetton@geotech.co.nz email ntraylen@geotech.co.nz email mccahon@geotech.co.nz Tel (03) 982 2538 PO Box 130 122 4 / 6 Raycroft Street Christchurch New Zealand

GEOLOGICAL & ENGINEERING SERVICES

Summary

-	-					
tions	Terrain	Near flat site but with Avon River passing 30 m to the west and approximately 4 m below site level.				
Site & Sub-surface Conditions	Soil profile	A surface layer of historic fill and topsoil up to 0.8 m deep, over interbedded silts and sands to about 5 m depth, over medium dense sands to ≈11 m and very soft silts and clays to ≈14 m. This is underlain by ≈ 9 m thickness of dense sands, then 0.5 m of clayeys silts, capping the Riccarton Gravel aquifer at 23 m deep.				
k Su	Soil classification	Class D, deep soil site to NZS1170.5:2004				
Site 8	Groundwater depth	3 m median depth on east side of site with fall to the river of 0.3 m over the site length.				
	Earthquake performance	Well tested to > SLS shaking in the September 2010 and February 2011 earthquakes, with moderate to severe liquefaction effects recorded.				
ects	Liquefaction	Significant liquefaction throughout the soil profile at ULS but in isolated layers and typically below 5 m depth at SLS.				
Aspe	Liquefaction 'index'	SLS: 20 - 40 mm				
nic	settlement	ULS: 80 – 150 mm (for top 10m of soil profile)				
Seismic Aspects	Lateral spread	Minor to moderate spread is predicted, based on construction o the CCC palisade wall protecting the Avon River-bank along Fitzgerald Ave, following the Christchurch earthquake.				
	Foundation	Red-zone by MBIE classification				
	technical category	Hybrid TC2/TC3 (SLS/ULS) by assessment				
S	Slippage	Low risk, except under liquefaction conditions when lateral spread may be an issue. The Avon River palisade wall has mitigated this risk.				
Natural hazards	Subsidence	Liquefaction settlement is expected in major earthquakes. Risk can be minimised by following MBIE Guidance and recommendations of this report.				
Natu	Inundation	The site level is well above the Avon River and the site is outside the CCC Flood Management Area. Normal Building Code provisions for floor levels above finished ground will mitigate this risk.				
	Proposal	New two-storey apartment blocks on Lots 2 and 3.				
pment	Suitable foundation	TC2 Enhanced slab foundations are suitable, with shallow ground improvement.				
Site Development	Bearing capacity	200 kPa ultimate bearing capacity is available in the natural ground. 300 kPa can be assumed for design of foundations on top of reinforced gravel rafts.				
N	Suitability for subdivision	Suitable for subdivision in terms of RMA section 106 requirements				

Subdivision

254-256 Fitzgerald Avenue, Christchurch

Geotechnical Assessment Report

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Appendix

- Site Investigation Plan
- Hand-auger logs, 10 pages
- CPT plots, 2020 investigation, 4 pages
- CPT plots from NZGD, 5 pages
- Borehole log from NZGD. 6 pages
- Liquefaction Analysis, 14 pages
- Extract from MBIE Guidance method specification for type G1d ground improvement

1 Introduction

1.1 Purpose

This geotechnical report evaluates the ground conditions, assesses the geotechnical hazards and recommends a suitable foundation system for the proposed development of 254-256 Fitzgerald Avenue, Richmond, Christchurch. It is intended to be used in support of foundation design and for building and resource consent applications.

The report includes:

- A summary of investigations and ground conditions on and around the site
- a liquefaction & lateral spread assessment
- a geo-hazard assessment against RMA Section 106
- Site ground improvement and foundation recommendations for new buildings

Any issues of ground contamination have not been considered and are outside our scope of work.

1.2 Site

This 2408 m² site is on the corner of Fitzgerald Ave and Harvey Terrace, and has established residential properties on the east and north-east sides. It is 44 m wide on Fitzgerald Ave and 48 m long on Harvey Terrace.

The site appears flat but there is about 0.5 m fall from north to south and the entire site is elevated above Fitzgerald Ave which is in turn elevated above the adjacent Avon River. The Avon River bed is estimated to be 4 m below site level.

This section of Fitzgerald Ave was closed following the February 2011 earthquake because of lateral spreading and slumping of the northbound lanes into the river. A substantial ground improvement project has created a palisade wall along the river-bank and under the edge of the north-bound lanes which allowed the road to be re-opened.

This site has been classified red-zone by MBIE as have all sites to the south of Harvey Terrace, and all sites along Fitzgerald Ave up to Heywood Terrace. Properties one back from the Fitzgerald Ave frontage are classified as Foundation Technical Category TC3.

1.3 Proposed Development

A subdivision is proposed for 254-256 Fitzgerald Ave where a single large site that has previously been occupied by three residential apartment buildings is intended to be subdivided into three titles and developed with two new apartment buildings to complement the one remaining block of four apartments on the site.

The subdivision proposal is still under development, but an early version of the plan shows Lot 1 holding the existing block of four apartments at 256 Fitzgerald Ave, with drive-on access from Harvey Terrace. Lot 2 occupies the south-west corner of the site at 254 Fitzgerald Ave and Lot 3 will be an 18m wide strip on the east side of the property corresponding to the apartments that were previously at No 5 Harvey Terrace.

Building details are not yet known but they are expected to be similar to the existing, that is, two storeyed but typically of lightweight construction.

2 Ground Information

2.1 Regional Geology

GNS Geological Map 3 (Begg, Jones, & Barrell, 2015) shows the site as being located on a fluvial interchannel trough or flat, part of the Yaldhurst member of the Springston Formation with a surface geology typically of alluvial sand and silt and an estimated maximum age of 3,000 years. To the south of Harvey Terrace is a 'recent river plain' with an estimated maximum age of 500 years.

This surface material is underlain with alluvial sands and gravels, transported by the Waimakariri River. Underlying the entire site (as it does for all of Christchurch) is the dense gravel layer known as the Riccarton Gravel. The regional geological model (Begg, Jones, & Barrell, 2015) predicts the Riccarton Gravels to be at 27 m depth and about 18 m thick in this location. The Riccarton gravel is underlain with further layers of silt, sand and gravel for another 500 – 600m before volcanic rock from the Lyttelton volcano is encountered.

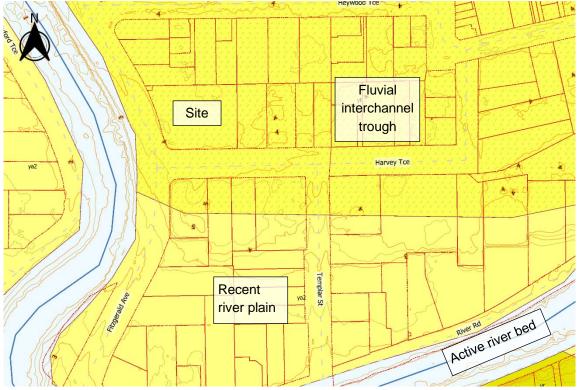


Figure 2-1 Geomorphic map data (ref GNS Geological Map 3)

2.2 Existing geotechnical records

The New Zealand Geotechnical Database (NZGD) holds data close to the site. The most relevant is listed in Table 2-1. The locations of the closest tests are shown on the appended site plan 5595/1.

NZGD Test	Location	Depth of test (m)
CPT_564	12 m west, on Fitzgerald Ave in front of site	23.1
CPT_404	8 m south on Harvey Tce outside No 5	22.9
BH_1740	BH_1740 8 m south on Harvey Tce adjacent to CPT_404	
CPT_46985	25 m north on 20 Heywood Tce	18.1

Table 2-1	NZGD deep soil test information
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2.3 Site Investigation

A site investigation was arranged in December 2020 with shallow testing by hand-auger and Scala Penetrometer with four tests around a likely building footprint on Lot 2 and six tests around a likely building footprint on Lot 3.

Deep testing by CPT was also carried out with four tests, two each on Lot 2 and Lot 3. The CPT testing was arranged to form a Tee shape in plan with the existing CPT's forming the extreme ends of the Tee. CPT_564, CPT001, 2 & 3 are aligned perpendicular to the river to test continuity of any liquefiable layers, whilst CPT003 & 4 align with the existing testing to the north and south to form a line parallel to the river under the site on Lot 3.

Test locations are shown on the appended site plan. Test data are also appended.

3 Subsurface Conditions

3.1 General soil profile

The hand auger boreholes show fill and sometimes buried topsoil from 0.4 to 0.8m depth over silts and sands on Lot 2 and sands on Lot 3 to the maximum 2.1m depth tested. HA07 on Lot 3 was unable to get past an obstruction at 0.5m depth.

An interpretation of the CPT tests are plotted together on the following page (Figure 3-1).

Depth to top	Thickness	Description	
surface (m)	(m)		
0	0.4 to 0.8	Historic fill, buried topsoil in places.	
0.4 to 0.8	≈ 5	Interbedded silts and sands - generally loose and soft	
	(up to 9m in CPT04)	with some very soft clayey layers	
≈ 5	≈ 6	Medium dense sands and silty sands. With some siltier	
		lenses (eg at -4m RL in CPT002)	
10 to 12	1.5 to 3	Very soft silts and clayey silts	
13 to 15	8 to 10 m	Dense to very dense sands – becoming silty with depth	
22.5	0.5	Clayey silts – aquifer capping layer	
23	≈ 18	Riccarton gravels aquifer (from Borehole_1740)	

A general description of the ground conditions is:

Table 3-1Generalised soil profile

The table and figure indicate substantial variability in ground conditions which is not uncommon in Christchurch alluvial deposits.

CPT_564	CPT001	CPT002		CPT003	CPT004	
5.0 4.0 - Sitty sand & sandy si			Sand & silty sand 4 Sand & silty sand	5.0 4.0 - Very densektiff soil Sand & sitty sand	5.0 4.0 - Sand & sinty sand Sinty sand & sand	
3.0 - Sitty sand & sandy si	1 3.0 - Silty sand & sandy silt	3.0 -	Sitty sand & sandy sitt 3	3.0 - Silty sand & sandy silt	3.0 - Sing said essaid	
2.0 - Clay & sitty clay Clay	2.0 - Clay & sitty clay	2.0 -		2.0 - Silty sand & sandy silt	2.0 - Sand & sitty sand	
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0.0 - Sitty sand & sandy si	Clay & sitty clay		Sand & sitty sand	Sand & sitty sand	Cial cost units	
Clay	0.0 - Clay & sitty clay Clay & sitty clay		Sand & sifty sand C Clay & sifty clay	0.0 - Silty sand & sandy silt	0.0 - Sitty sand & sand	
-1.0 - Sitty sand & sandy si Sand & sitty sand	-1.0 - Clay	- 1.0 -	Silty sand & sandy silt - 1	1.0 - Sand & sitty sand	-1.0 - Sand & sitty sand Sitty sand & sand	
-2.0 - Sitty sand & sandy si	5itty sand & sandy sitt	-20-		2.0 - Sand & silty sand	-2.0 - Silty sand & sand	
-3.0 - Siny sand & sandy si			sand e sitty sand	Sitty sand & sandy sitt	Sand & sitty sand	
	- 3.0 - Sitty sand & sandy sitt	-3.0 -	Sand & silty sand	3.0 - Sitty sand & sandy sitt Sitty sand & sandy sitt	-3.0 - Sand & sitty sand Clay & sitty clay	
- 4.0 - Sand & sitty sand	- 4.0 -	-4.0 -	Clay & sitty clay	4.0 -	-4.0 - Sitty cary	
-5.0 - Sitty sand & sandy si	-5.0 -		Sitty sand & sandy sitt	5.0 - Sand & silty sand	-5.0 - Sitty sand & sand	
-6.0 - Sand & silty sand	-6.0 - Sand & silty sand		sand a siny sand	Sitty cand & candy citt	only sand or sand	
		-6.0 -	-	6.0 - Olig Said Cosaid, Site	-6.0 - Sand & sitty sand	
- 7.0 - Clay & sitty clay Sitty sand & sandy si - 8.0 - Sitty sand & sandy si	-7.0 -	-7.0 -		7.0 -	-7.0 - Sitty sand & sand	
- 8.0 - Siny sand & sandy si	-8.0 - Clay & sitty clay	-8.0		8.0 -	-8.0 -	
-9.0 -	Clay & sitty clay				Sitty said & said	
-9.0 -	-9.0 - Sitty sand & sandy sitt	-9.0 -		9.0 -	Sand & sifty sand	
- 10.0 - Sand & sitty sand	-10.0 -	-10.0 -	-10	0.0 -	10.0 - Sand & sitty sand	
-11.0 -	-11.0 -	-11.0	-11	1.0	11.0 - Sitty sand & sand	
-12.0 -	12.0 - Sand & silty sand	100				
Sitty sand & sandy si	1210	-12.0 -	-12	2.0 -	12.0 - Sand & silty sand	
- 13.0 - Sand & sitty sand	-13.0 -	-13.0 -	-13	3.0 -	13.0 -	
-14.0 -	-14.0 -	-14.0	-14	4.0	14.0 - Sand & sitty sand	
-15.0 - Sitty sand & sandy si Sitty sand & sandy si		15.0				
Sand & sitty sand		-15.0 -		5.0 -	15.0 -	
-16.0 - Silty sand & sandy si	16.0 -	-16.0 -	-16	6.0 -	16.0 -	
-17.0 -	17.0 -	-17.0	-17	7.0	17.0 -	
-18.0 -	-18.0 -	-18.0 -		8.0		
Sand & sitty sand		-18.0 -	-18	8.0]	18.0 -	
-19.0 - Clay Sand	·19.0 -	-19.0 -	-19	9.0 -	19.0 -	
-20.0 +++++++++++++++++++++++++++++++++++	20.0 + + + + + + + + + + + + + + + + + +	20.0 +		0.0 + + + + + + + + + + + + + + + + + +	20.0	
0 2 4 6 8 10 12 14 SBTn (Robertson, 1990)	0 2 4 6 8 10 12 14		10 12 14	0 2 4 6 8 10 12 14 1	0 2 4 6 8 10 12 1	

Figure 3-1 Interpretation of soil properties from CPT data

3.2 Groundwater

The Groundwater Surface Elevation studies (GNS Science, 2014) suggests a median groundwater elevation¹ of about 1.2 m on the east side of the site falling toward the river at a grade of 1 in 120 m. The $85\%_{ile}$ water level is 0.2 m higher.

With existing ground levels of 4.2 m this gives water depths of 3 to 3.3 m (accounting for the groundwater gradient across the site).

Groundwater was observed at 3 m and 3.1 m in the recent investigations. This is consistent with the GNS model and with the water level in the river.

A groundwater depth of 3 m has been adopted for the purpose of liquefaction assessment.

4 Seismic Considerations

4.1 Seismic Category

The deep alluvial soils that underlie most of Christchurch makes this a Class D, deep or soft soil site, in terms of the seismic design requirements of NZS 1170.5:2004.

4.2 Seismic Hazard

Design of buildings must consider at least two loading situations – the serviceability limit state (SLS) and the ultimate limit state (ULS). At the SLS level of earthquake shaking a building should perform such that damage is easily repairable and does not affect the function of the structure. At ULS the structure can suffer severe damage but should not collapse.

Following the Canterbury Earthquakes a review of the regional seismic hazard has resulted in peak ground accelerations (PGA) for liquefaction assessment, recommended by MBIE (MBIE, 2012), (MBIE, 2014) for **Class D** sites and Importance Level 2 (IL2), normal occupancy, structures as shown in Table 4-1.

Design Case	PGA	Magnitude	Return period
SLSA	0.13g	M7.5	25 yr
SLSB	0.19g	M6	25 yr
ULS	0.35g	M7.5	500 yr

 Table 4-1
 Seismic design cases for liquefaction assessment

¹ to NZ Vertical Datum 2016 (or approximately 21 m to Christchurch Drainage Datum)

4.3 Recent Earthquakes

The site has been subject to repeated shaking in the Canterbury Earthquakes. Estimates of peak ground accelerations (Bradley & Hughes, 2012) show that the site is likely to have experienced shaking exceeding a Serviceability Limit State (SLS) event in each of the four main earthquakes (see Table 4-2).

Earthquake	Mag.	Peak Ground Acceleration, PGA					
		Mean	Equivalent M7.5	PGA _{10_7.5}			
4 Sep 2010	7.1	0.21	0.19	0.13			
22 February 2011	6.2	0.45	0.32	0.21			
13 June 2011	6.0	0.26	0.18	0.11			
23 Dec 2011	5.9	0.23	0.15	0.10			

Table 4-2 Estimated PGA for the main Canterbury earthquakes (green fill indicates 'sufficiently tested')

The estimated mean PGA for each earthquake has been converted to an equivalent PGA for a magnitude M7.5 earthquake (allowing direct comparison with the M7.5 MBIE design PGA's in *Table 4-1*), plus the PGA with 90%, probability of being exceeded (PGA_{10_7.5}). The 90% exceedance PGA is the level at which the MBIE guidance accepts a site as being "sufficiently tested".

At this site the September 2010 and February 2011 earthquakes almost certainly (90%) exceeded SLS shaking and are likely to have exceeded SLS in all four main earthquakes. The February 2011 earthquake is likely to have been very close to a ULS event.

4.4 Site Performance

4.4.1 Ground damage records

Ground damage reports from EQC records (EQC, 2013), following the significant earthquake events are as follows:

Event	Ground observation	Aerial photo inspection
September 2010	no records	No observed liquefaction
February 2011	severe lateral spreading ejected material often observed. "moderate" recorded on the road	Moderate-Severe
June 2011	no records (road observations only)	Moderate-Severe (in our experience interpretation for this event often overstates actual liquefaction)
December 2011	no records	Minor observed liquefaction

 Table 4-3
 EQC records of liquefaction on site

Our own examination of aerial photographs taken after the February 2011 earthquake confirms the "Moderate to Severe" assessment from the aerial photos. Significant ground cracking is visible along Fitzgerald Avenue and this may have influenced the ground-based observation.

4.4.2 Ground Cracking

Ground cracks as recorded by consultants for EQC (EQC, 2012) are shown running along the river side of Fitzgerald Ave and out to the median strip opposite Harvey Terrace (Figure 4-1) Some relatively minor cracks (green are under 10 mm and blue are under 50 mm) are seen along Harvey Terrace adjacent to the site.

Only one crack is recorded on the site itself, an 'unclassified crack crossing the north-east edge of the site. Unclassified cracks are generally minor in nature and the orientation of this crack is not consistent with lateral spread.



Figure 4-1 Ground cracks as recorded for EQC (from NZGD)

4.4.3 Change in ground surface levels

Interpretation of LiDAR surveys (EQC, 2012) suggests a total vertical elevation change of 0.4 m at the site with 0.16 m estimated as movement of the bedrock. The liquefaction induced settlement is thus 0.24 m over all of the main earthquake events.

Settlements (as estimated from LiDAR) were variable across the site with the least settlement seen in the south west corner and the most on the east side where up to 0.5 m is indicated (Figure 4-2). The settlement associated with slope failure along the river edge is seen in pink to the left of this image.

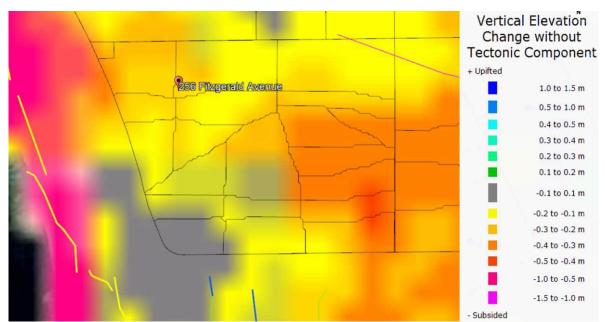


Figure 4-2 Liquefaction settlements - all events

4.4.4 Site performance summary

The site clearly suffered significant liquefaction damage in the Canterbury earthquakes. However, this appears to be mainly in terms of liquefaction ejecta and on-site settlement. There was a known lateral spread and/or slope failure along Fitzgerald Avenue, but this doesn't appear to have had a significant effect on the site itself.

4.5 Liquefaction potential

4.5.1 Analysis

Analysis of the on-site CPT's has been carried out using the methods recommended by MBIE². The peak ground accelerations used for analysis are as shown in *Table 4-1* and, for comparison, the February 2011 event was modelled with peak ground acceleration of 0.45g and Magnitude 6.2.

Standard parameters of 0.15 for Probability of Liquefaction (P_L) and a fines fitting factor $C_{FC} = 0.0$, this was found to give reasonable agreement with the observed settlements discussed in Section 4.4.3 above.

Detailed liquefaction profiles are shown on the appended output sheets. Cumulative thicknesses of liquefaction and liquefaction induced settlements for the upper 10m and for the full profile, where available, are shown in Table 4-4.

² Liquefaction assessment method by Boulanger & Idriss (2014) and settlement method by Zhang (2002)

		Lique	faction In	duced Se	ettlement	Sum c	of liquefia	ble layer t	hickness
СРТ	٩	(mm)				(m)			
GFT	Depth (m)	ULS	SLSA	SLSB	Feb '11	ULS	SLSA	SLS₿	Feb '11
	D L	M7.5	M7.5	M6	M6.2	M7.5	M7.5	M6	M6.2
CPT001	10	80	10	20	70	3.6	0.0	0.5	3.6
CPT002	10	150	30	50	150	6.3	0.8	1.5	6.3
CPT003	10	130	20	30	130	5.8	0	1.1	5.7
CPT004	10	130	20	40	130	5.4	0.2	2.0	5.4
CPT_564	10	70	10	20	70	3.3	0.0	0.5	3.2
CPT_404	10	50	0	10	50	2.4	0	0.5	2.2
CPT_46985	10	100	20	40	100	4.5	0.3	1.6	4.5
Tests deeper t	han 10m	n (full pr	ofile)						
CPT001	19.3	160	20	30	150	8.3	0.3	1.1	7.7
CPT004	18.3	210	40	70	210	9.7	0.6	2.8	9.4
CPT_564	23.1	100	10	20	90	4.7	0.1	0.5	4.5
CPT_404	22.9	100	10	20	100	5.5	0.2	0.7	5.0
CPT_46985	18.1	170	30	60	170	8	0.7	2.2	7.5

 Table 4-4
 Cumulative thickness and Liquefaction Induced Settlement

Estimated liquefaction induced settlements on the site are 20 to 40 mm at SLS and 80 to 150 mm at ULS for the upper 10m, increasing to 30-70 mm SLS and 160 to 210 mm ULS for the full soil profile. At the estimated mean level of shaking the February 2011 earthquake would be expected to result in liquefaction induced ground settlement close to a ULS event.

The settlement analysis method is empirical and approximate only, with perhaps a $\pm 50\%$ margin to the numbers given. It also applies to a 'free field'³ situation and additional large settlements may occur associated with sand ejection, lateral spread and movement under foundation loads.

4.5.2 Lateral Spread

Lateral spread and lateral stretch are the most damaging aspect of liquefaction, in Christchurch lateral spreading was mostly seen along the banks of the Avon River and was worse downstream of Barbadoes Street. Conditions that allow for lateral spread include:

- a sudden change in ground elevation, referred to as a free-face, such as a river bank,
- a significant thickness of liquefiable soils and
- continuity of liquefiable layers away from the free face to under the site in question

The standard methods for estimating lateral spread can give widely varying answers (between methods) and are known for poor accuracy. In many cases the extent of lateral spreading may be constrained by geology and will not occur as estimated by models that are usually limited by the amount of geological data available.

For this site we can see that there are liquefiable layers of reasonable thickness that appear to be near continuous between the site and the river, although the two CPT's closer to the river have more

³ 'free field' is level open ground away from any influence of foundation loads or slopes.

broken layers in the critical depths (between 3 and 8 m). We also know from observation that in a significant earthquake such as February 11 there was no significant ground cracking recorded on the site and that since then there has been a major repair of the river-bank along Fitzgerald Ave with deep ground improvement by stone columns that have the specific intention of disrupting the continuity of the liquefiable layers and holding back the ground behind the palisade wall.

We have not been able to obtain information on the design standard for this retaining wall from Council, but we assume it will be not less than a 1 in 100 year event and is more likely to be a 1 in 500 year.

Taking account of the presence of this wall and the reasonable performance during the February 2011 earthquake we assess the residual lateral spread and lateral stretch risk as **minor** or TC2 equivalent at SLS and **Minor to Moderate** (less than 200 mm) at ULS.

4.6 Liquefaction Summary

The site has been 'sufficiently tested' at SLS and the February 2011 earthquake is likely to have produced liquefaction approaching that of a ULS event. Accordingly, the observations of performance during the Canterbury Earthquakes can be relied upon to predict future performance.

The MBIE 'index' limits for liquefaction induced settlements in TC2 areas, are 50mm at SLS and 100mm at ULS over the upper 10m. At 20 - 40 mm SLS and 80-150 mm ULS the site fits into a hybrid category of TC2/TC3.

Lateral stretch risk is assessed as **minor** at SLS and **minor to moderate** at ULS, based on records of site performance in the Canterbury Earthquakes and the expectation of improved performance due to the stone column palisade wall built along Fitzgerald Ave in front of the site.

5 Geotechnical Hazards

5.1 Section 106 Assessment

Section 106 of the RMA identifies a range of hazards that may provide justification for a consent authority to refuse subdivision consent. Section 106 also requires consideration of those same hazards following any likely development.

An assessment of the site against those hazards is provided in Table 5-1. The property is assessed as being either free of particular hazards, or, the hazard can be satisfactorily mitigated, such that there is no reason from a geotechnical perspective that the subdivision cannot proceed.

Hazard	Current assessment	Post development assessment				
Erosion	The site is close to the Avon River					
	but is separated from the main					
	channel by Fitzgerald Ave.					
	As a major city thoroughfare we					
	anticipate that Council will ensure	No change in risk.				
	that the river bank does not erode in					
	this location					
Falling debris	The site is flat with no source area	No change				
	for falling debris.					
Subsidence	There is a liquefaction risk at the	Building in accordance with the				
	site which is likely to result in some	recommendations of MBIE for				
	subsidence in a future earthquake.	liquefaction prone sites will mitigate this				
		risk.				
Slippage	There is a risk of lateral spread	Development does not change this risk				
	associated with liquefaction and	but building in accordance with the				
	proximity to the Avon River, in a	recommendations of MBIE for				
	ULS earthquake	liquefaction prone sites will protect life in				
		the event that some slippage takes				
		place.				
Inundation	The site is not in the CCC Flood	No change in risk				
	Management Area					
Table 5-1 Assessment against RMA S 106						

Table 5-1 Assessment against RMA S.106

The only significant risks that affect the site are both associated with liquefaction. This has been discussed in Section 4 above.

6 Foundations

6.1 Shallow Bearing Capacity

The shallow soils testing shows uncontrolled fill at the ground surface over most of the site, with buried topsoil encountered in two of the ten holes. The depth of fill and topsoil is from 500 to 800 mm below current ground level. For shallow foundation systems this fill and any underlying topsoil must first be removed to expose natural silts and sands.

Scala penetrometer testing shows a Geotechnical Ultimate Bearing Capacity (GUBC) of 200 kPa in the natural subsoils. HA1 shows thin loose layer from 1.35 to 1.5 m. This layer has an indicative Ultimate bearing capacity of 150 kPa and is sufficiently deep that it should not affect bearing capacity for shallow foundations.

6.2 Foundation Recommendations

The relevant parameters for selecting a foundation system are:

Technical Category	TC2/TC3 hybrid
GUBC	≈200 kPa from 800 mm deep
SLS liquefaction settlement	20 to 50 mm, Lot 2
•	30 to 40 mm, Lot 3
ULS liquefaction settlement	80 to 150 mm, Lot 2
·	130 mm, Lot 3
ULS lateral spread	Assessed as minor to moderate
Proposed construction	Two storey apartment buildings, still to be designed, but assumed to be light timber framed structures with light roofing and medium weight cladding, on concrete
	foundations.

There is sufficient distinction between Lot 2 and Lot 3 to recommend different foundation systems for the structures on each. The CPT's on Lot 2 show greater differential settlement at both SLS and ULS (30 mm and 70 mm), and proximity to the river is expected to mean more significant lateral spread effects if the design capacity of the riverside palisade wall is exceeded. There is also the soft layer identified in HA01 at 1.35m depth.

6.2.1 Lot 2 – shallow ground improvement

For Lot 2, shallow ground improvement is recommended in the form of a 1.2m thick reinforced crushed gravel raft with two layers of geogrid reinforcement (Tensar Triax 160, or similar approved) (Type G1d Section 15.3.10.1b, MBIE Guide).

A method statement for construction of the gravel raft is contained in Appendix C4 of the guidance (extract appended to this report). At a depth of 1.2 m below the foundations the surface fills will be removed and the soft layer in HA01 will be improved by compaction of the base of the excavation.

6.2.2 Lot 3 - shallow ground improvement

Shallow ground improvement for Lot 3 can be as described for hybrid TC2/3 sites in Clause 15.4.6 of the MBIE Guidance, but with an additional layer of geogrid. This system includes:

- Excavate to 600 mm below foundation level (minimum 800 below ground) and to 1m outside the footprint.
- Thoroughly compact the base of the excavation.
- Place geotextile (Bidim A19 or similar) and Geogrid (Triax TX160 or equivalent) in the bottom of the excavation. Wrap the geotextile up the sides of the excavation.
- Place and compact a layer of AP40 on top of the geogrid and then a second geogrid layer.
- Place and compact layers of AP65 gravel back into the excavation up to foundation level

6.2.3 Further recommendations for shallow ground improvement

The following recommendations are common to both Lots:

- Follow all manufacturers instructions for lapping of geotextile and geogrid
- Geogrid should be laid in strips, full length across the excavation, in an east-west direction, toward the river.
- Place and compact layers of imported gravel (200 mm loose thickness) back into the excavation up to foundation level
- All layers of hardfill should receive the same compactive effort, that is, the same number of passes with the same heavy compactor (eg vibrating plate compactor of 350 kg or greater).
- ND testing should be arranged by the contractor for the second layer placed and every second layer after that as well as the finished surface
- A target of 92% of maximum dry density as determined by vibrating hammer test (NZS 4402:1988 Test 4.1.3) is to be achieved,

Following completion of the gravel rafts the sites can be considered equivalent of a TC2 site.

6.2.4 Enhanced foundations slabs

For each building construct an enhanced foundation slab on top of the hardfill raft. Option 2 or Option 4, as described in Clause 5.3.1 of the MBIE Guidance are considered suitable.

Structural design of the raft must consider standard 'loss of support' criteria as recommended by MBIE of 2 m at slab edges and 4 m in the interior.

Foundations can be designed for an ultimate bearing capacity of 300 kPa on top of the gravel raft. A capacity reduction factor of 0.5 should be applied to the GUBC to derive the design bearing strength of 150 kPa for comparison with ULS load cases.

The CPT's on Lot 3 show consistent settlements at ULS (130 mm in the upper 10m) but the adjacent CPT on Harvey Terrace is only predicting 50 mm in the upper 10 m. This suggests the possibility of dishing in the foundation slab of a long apartment block. We recommend this effect be assessed during structural design and consideration be given to a structural separation between a north and south apartment block on Lot 3.

7 Construction Monitoring

Construction monitoring inspections are recommended for:

- a) the base of the excavation, to confirm subgrade suitability.
- b) placement of geotextile and geogrid.
- c) placement and compaction of gravel hardfill early in placement.
- d) further inspections of hardfill during placement and again on completion.

8 Conclusions

Liquefaction assessment indicates that a hybrid TC2/TC3 classification is appropriate for the site, based on:

- a) Reasonable performance during the Canterbury Earthquakes where the site was 'well tested' at SLS.
- b) Subsequent construction of a major palisade wall along the Avon River bank, involving interruption of the critical liquifiable layers by deep ground improvement.
- c) Analysis of on-site CPT's.

For Lot 2 our foundation recommendation is to treat as for a TC3 site with a type G1d gravel raft and an enhanced concrete slab foundation. This is because of greater differential settlement calculated across Lot 2 and because of proximity to the Avon River with some uncertainty over the design standard used for the Fitzgerald Avenue palisade wall.

For Lot 3 a hybrid TC2/TC3 foundation system, comprising a geogrid reinforced gravel raft to 600 mm below foundation level, with two layers of geogrid, and a TC2 Enhanced foundation slab system (waffle slab or equivalent) is recommended.

A subgrade bearing capacity of 200 kPa is expected and foundations can be designed for 300 kPa (Ultimate Bearing) on top of the gravel raft.

Given that the residual liquefaction risk can be addressed by shallow ground improvement as described above we conclude that there is no geotechnical reason to prevent the subdivision of the land and construction of new apartment blocks.

Our recommendations are based on assumptions about the form of construction of the apartment blocks given that no details are available. As the design proceeds we recommend that a suitably qualified geotechnical engineer be engaged to confirm that the proposed buildings and foundations are consistent with this geotechnical assessment.

9 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.

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.

This report has been prepared for the purpose as outlined in the introduction and the information and interpretation may not be relevant for other purposes. Geotech Consulting Ltd can review the report and the sufficiency of the investigation and appropriateness of the recommendations for other purposes as needed.

This report has been prepared solely for the benefit of Ms R Harwood, and the Christchurch City Council. No liability is accepted by this Company or any employee or sub-consultant of this company with respect to its use by any other person. This disclaimer shall apply notwithstanding that the report may be made available to other persons for an application for permission or approval or to fulfil a legal requirement.

10 References

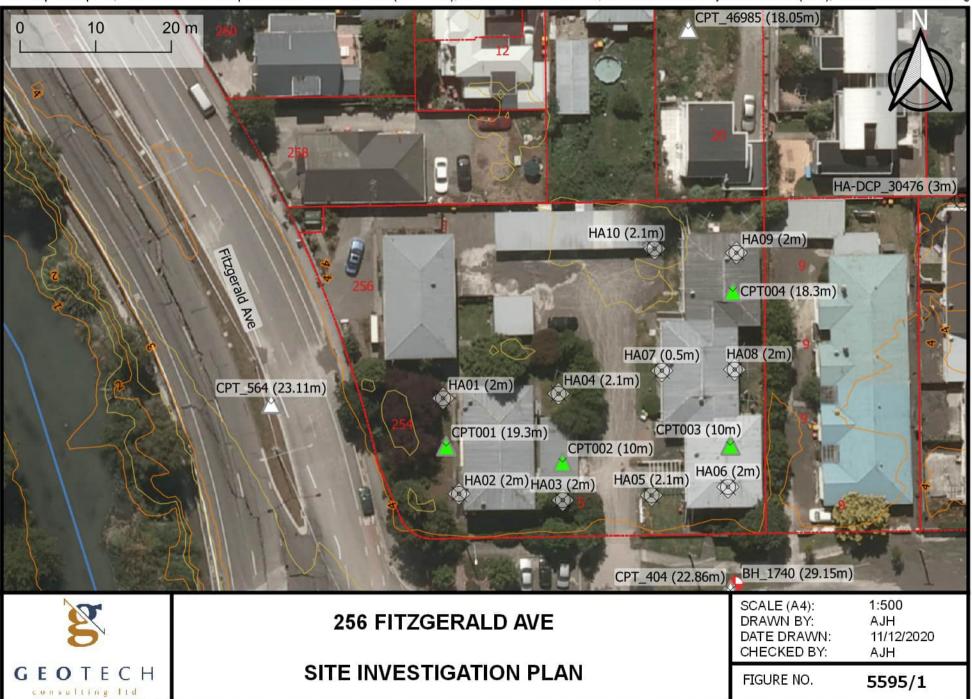
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Important notice

Some information in this report was obtained from maps and/or data extracted from the New Zealand Geotechnical Database, which were prepared and/or compiled for the Earthquake Commission (EQC) to assist in assessing insurance claims made under the Earthquake Commission Act 1993. The source maps and data were not intended for any other purpose. EQC and its engineers, Tonkin & Taylor, have no liability for any use of the maps and data or for the consequences of any person relying on them in any way. This "Important notice" must be reproduced wherever this EQC information or any derivatives are reproduced.

Appendix

- Site Investigation Plan
- Hand-auger logs, 10 pages
- CPT plots, 2020 investigation, 4 pages
- CPT plots from NZGD, 5 pages
- Borehole log from NZGD. 6 pages
- Liquefaction Analysis, 14 pages
- Extract from MBIE Guidance method specification for type G1d ground improvement



Basemap: Aerial photo, Christchurch Post Earthquake 0.1m Urban Aerial Photos (24/02/2011); Contours to NZVD 2016 from, Christchurch and Ashley River 1m DEM (2018); both accessed at data.linz.govt.nz

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1.5 Sandy Silt: Yellow-Brown, mottled grey and orange. dry, non-plastic, firm. X X Image: Constraint of the second				- No					2	Ů	
1.5 dry, non-plastic, firm. 1.5 2.0					-						
1.5 dry, non-plastic, firm. 1.5 2.0		Sandy SILT: Vollow, Bro	win mottled grow and erange		Xx			_		:	
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		BOR	ΕH	OLE	LO	G			Hole No:	HA05	
	2					•			Job No	: 5595	
	0	Project: 256 Fitzgerald Ave,	Christ	ahurah					Logged by	: WH/RBW	
6	EOTECH	Client: R Harwood	Christ	church					Date drilled Checked by	· RBW	,
G	EUTECH	Hole location: Refer to Site Plan.							Date checked)
	Driller: WH	Contractor: E	Equipn	nent: S	C+HA		R.L:		Max depth		
	Notes:			-							
	STRA	TA DESCRIPTION	nscs	Graphi c Log	Water Table	Samples	S.P.T N uncorrected		ALA PENETRO (mm/blow)		
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		w-Brown, fine, moist, loose to		_					i j		
	medium, homogeneoi	us, varies minor to silty.						ĵ_	••••••••••••••••••••••••••••••••••••••		
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	2										Job No:	5595	
	0	Brojosti	256 Fitzgerald Ave, (Chrict	ahurah						ogged by: te drilled:	WH/RBW	<u>v</u>
6	ΕΟΤΕΟΗ		R Harwood	chinst	church						te arillea: hecked by:	2/12/2020	0
G	EUTECH		Refer to Site Plan.								e checked:		0
	Driller: WH	Contractor:	E	quipn	nent: S	C+HA		R.L:			Max depth:		
	Notes:												
	STRAT	TA DESCRIP	TION	USCS	Graphi c Log	Water Table	Samples	S.P.T N uncorrected	SC		PENETRON (mm/blow)		
0.0				ŝ	55	∋ ⊑	Sa	50 100	р з	34 50	10	0	150 0.0
	SILT FILL: Grey-brown,	dry, non-plast	ic, stiff, trace to		K /					:			
	some gravel.				\vee				βΩ Ŭ.	•			
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	SAND: Some silt, Yellov	w-Brown, fine	, moist, loose to						<u></u>	•			
	medium, homogeneou								8	:			
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	No Further Progress du	ue to reaching	target depth						0	l			_
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G	EOTECH Driller: WH	Client:	BORE 256 Fitzgerald Ave, Cl R Harwood Refer to Site Plan. Eq	nrist			G	 		Hole No: Job No: Logged by: Date drilled: Checked by: Date checked: Max depth:	5595 WH/RBW 2/12/2020 RBW 7/12/2020
	Notes: Hand cleared to 40		ed scala penetrometer. R		al on larg		ete at C	r	sc	ALA PENETROM (mm/blow)	
0.0	SILT FILL: Grey-brown, gravel.			FILL	Gr	Та	Sa	100) <u>1</u> \$0 0.0
0.5	No Further Progress du	ie to refusal o	n large concrete.						89 99 1		0.5
1.0											1.0
1.5											1.5
2.0											2.0
2.5											2.5
3.0											3.0
3.5											3.5
4.0											4.0
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5.0											5.0

]	BORE	EH	OLE	LO	G			Hole N	o :	HA08	
										Job N Logged I	lo: {	5595	
	0	Project: 2	56 Fitzgerald Ave, C	hrist	church					Date drille	y: v 24- 2	2/12/2020	/ D
C	EOTECH		R Harwood							Checked	by: F	RBW	
U		Hole location: F	Refer to Site Plan.							Date check	ed: 7	7/12/2020)
	Driller: WH	Contractor:	Ec	quipn	nent: S	C+HA		R.L:		Max dep	th:	2.00	
	Notes:				I	1	ω	0.0.7		ALA PENETR	0.45		
	STRAT	TA DESCRIPT	ION	USCS	Graphi c Log	Water Table	Samples	S.P.T N uncorrected		(mm/blov	/)	TER	
0.0				S	0.	≥≞	s S) 50 100		4 50	100		150 1 0.0
	SILT FILL: Grey-brown,			_	F/					: 			-
	trace gravel, transitior	is to silty, saliu	Y GRAVEL		V				8				-
	with depth.								₿ ©				-
		<u>.</u>		_	<u> </u>				8-0	l			-
0.5	SAND: Some silt, grey,								o				0.5
	medium dense, homog	geneous, varies	minor to silty.	_	_				p	İ			_
					_					Ь			_
				SW						j			_
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1.0										<u>7</u>			1.0
				╇					⊨ĭ				1
	SAND: Trace silt, yellow				_					<u> </u>			1
	loose to medium dens			_	_				ļ				
	- 1.3m, less silt and ligh	hter grey with c	lepth	_	_				4	}			
1.5										5			1.5
				SW					ſ	b			
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									8				
										2			
2.0	- 1.9m, wetter (wet) w	ith depth							Ĵ				2.0
2.0	E.O.H target depth								8				2.0
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			BORE	ЕН	OLF	LO	G		[Н	ole No:	HA0	9
	2		_ • • • •				-				Job No:	5595	
	0	Project:	256 Fitzgerald Ave, 0	hrist	church						ogged by: te drilled:	2/12/2	3W 020
C	EOTECH		R Harwood								hecked by:		
2		Hole location:	Refer to Site Plan.		i					Date	e checked:	7/12/2	020
	Driller: WH	Contractor:	E	quipn	nent: S	C+HA		R.L:		1	Max depth:	2.00	
	Notes:			10	-=	Ι.	se	S.P.T	sc		PENETRON		
	STRA	TA DESCRIP	TION	USCS	Graphi c Log	Water Table	Samples	N uncorrected			(mm/blow)		150
0.0	SILT FILL: Grey-browr	day pop p st	iff trace gravel		\overline{V}		o o						150
	transitions to silty, sa	indv GRAVEL w	ith depth.	_									
					Y /	1							
1									H				
				_	r /								
0.5					/				8		+ - +		0.5
1	SAND: Some silt, Yell	ow-Brown. fine	. moist. loose to	_									
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1	mediam, nomogeneo		or to sitty.	SW					b				
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	SAND: Trace silt, grey	, fine to mediu	m. moist.]							
1	loose to medium de			_									
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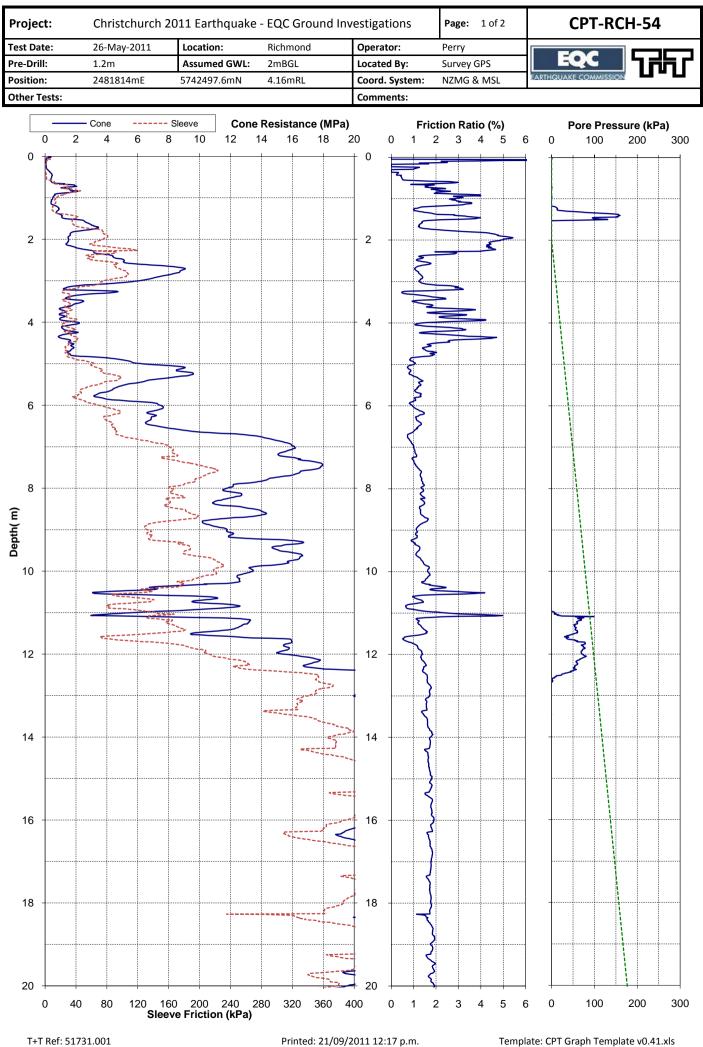
G	EOTECH Driller: WH	Project: 256 Fitzgerald Ave, 0 Client: R Harwood Hole location: Refer to Site Plan. Contractor: E	Christ			G	R.L:	Lo Dat Cl Dat	te drilled: hecked by:	5595 WH/RBW 2/12/2020 RBW 7/12/2020
	Notes: Location hand clea	red to 0.25m, then scala penetrometer re	ecomm	enced af	ter refu		urface fill. S.P.T	SCALA I	PENETRON	
0.0		A DESCRIPTION dry, non-p, stiff, trace gravel,	nscs	Graphi c Log	Water Table	Samples	N uncorrected	34 50	(mm/blow) 10	0 150 0.
	transitions to silty, sand	dy GRAVEL with depth.	EIL –							
		brown, moist, soft to firm.	Р	× آ م			8	b !		
0.5	medium, homogeneou	v-Brown, fine, moist, loose to s, varies minor to silty.	SW							
1.0										1.
1.5	SAND: Trace silt, yellow loose to medium dens Less silt and lighter gre									1.
			SW						8	
2.0	E.O.H target depth									2.
2.5										2.
3.0										3.
3.5										3.
4.0										4.
4.5										4.
5.0										5.

McMILLAN Drilling	Client: Geot	tech Cons	sulting L	td	Bore No.:	СРТ001	
	Project: 256 Fitzger	rald Aven	ue, Chris		Job No.:	19425	
Site Location: 256 Fitzgerald Avenue, Chris Grid Reference: 1571833.44m E, 5180877.93r Elevation: 0.00m Datum:	m N (NZTM) - Map or aerial pho	tograph	-	Date: 2/12/20 perator: R. Wylli iipment: 14t truo	e		
RAW DA	ТА			EHAVIOUR TYPE NORMALISED)	EESTIN	IATED PARA	METERS
Tip Friction Resistance Ratio (MPa) (%)	Pore Pressure (kPa)	Scale	SBT	SBT Descriptio (filtered)		Su (kPa)	N ₆₀
	9 111 111 111 111 111 111 111 1		₩4₩Φ►∞0		20		4 30 10 4 30 10
A A A A A A A A A A A A A A A A A A A				Sand mixtures: silty s to sandy silt Silt mixtures: clayey s silty clay Sands: clean sands to sands	silt &		South States and South States and South States and
EOH: 19.33m				Sands: clean sands to sands Sands: clean sands to sands			
Cone Type: I-CFXY-10 - Compression	Predrill: -	Term	nination	Soil Behavi	our Type (SBT)	- Robertson	et al. 1986
Cone Reference: 110542 Cone Area Ratio: -	Water Level: 3m Collapse: 3.2m	Target	_	0 Undefine		5 Sand mixtu sand to sar	res: silty Idy silt
Standards: ISO 22476-1:2012		Effective	· <u> </u>		e fine-grained	6 Sands: clea silty sands Dense sanc	n sands to I to gravelly
	After test -0.1442 -0.0291 -		Tip: Gauge: 🖌 ometer:	Clays: cla	ganic soil ay to silty clay [ures: clayey silt [lay	 7 Dense sand sand 8 Stiff sand to sand 9 Stiff fine-gr 	o clayey
Notes & Limitations Data shown on this report has been assessed to p geotechnical soil and design parameters using methor Testing for Geotechnical Engineering, 4th Edition. The carefully reviewed by the user. No warranty is provid design parameters shown and does not assume any aware of the techniques and limitations of any methor	ods published in P. K. Robertson an interpretations are presented only ded as to the correctness or the ap liability for any use of the results in	nd K.L. Cabal as a guide fo pplicability o n any design	(2010), Gui or geotechn f any of the	de to Cone Penetr ical use, and shou e geotechnical soi	ation Id be I and	Sheet 1 of 1	

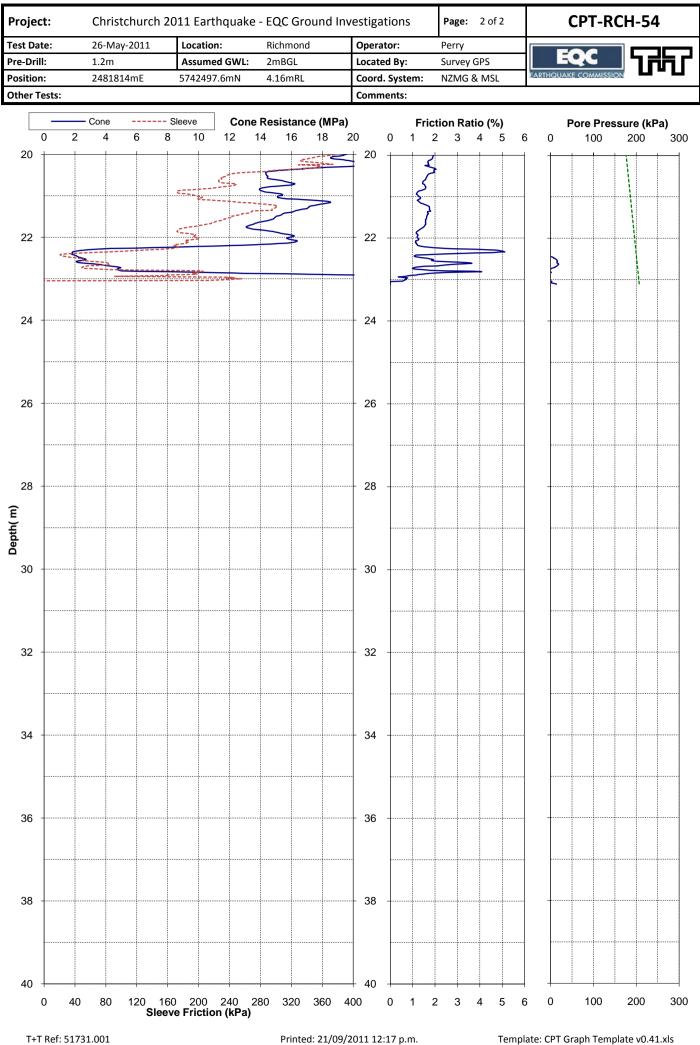
	MCMILLAN	Control Consulting Ltd					Bore No.:	re No.: CPT002			
		Project: 256 Fitzgerald Avenue, Christchurch					Job No.:	^{b No.:} 19425			
	Site Location: 256 Fitzgera	ld Avenue, Christchu	irch				Date: 2/12/20)20			
Grid Reference: 1571851.88m E, 5180876.9m N (NZTM) - Map or aerial photograph Rig Operator: R. Wyllie											
	Elevation: 0.00m	Datum: Gro									
		RAW DATA	A			SOIL BEHAVIOUR TYPE (NON-NORMALISED)		E	ESTIMATED PARAMETERS		
Predrill	Tip Resistance (MPa)	Friction Ratio (%)	Pore Pressure (kPa)	Inclination (Degrees)	Scale	SBT	SBT Descriptio (filtered)	Dr n (%)	Su (kPa)	N ₆₀	
			- 200 - 400 - 800	- 5 - 10 - 15		-0w4n0r@0			- 50 - 250 - 350 -	- 10 - 20 - 30 + 40	
		Marin Marine Marin					Sand mixtures: silty s to sandy silt Sand mixtures: silty s to sandy silt Sands: clean sands to sands Sand mixtures: silty s to sandy silt Sands: clean sands to sands	and silty			
c	Cone Type: I-CFXY-10 - Cone Reference: 110542 Cone Area Ratio: - Standards: ISO 22476- Zero load outputs (MPa)		Water Le Colla	frill: - evel: - pse: 2.7m	Targ	rmination et Depth:	O Undefine Sensitive	our Type (SBT) ed e fine-grained ganic soil	5 Sand mixtu sand to sar 6 Sands: clea silty sands	ires: silty ndy silt	
	Tip Resistance Local Friction Pore Pressure	-0.1747 -0.14 0.0293 0.028 	27		Incl	Gauge: inometer:		ay to silty clay ures: clayey silt lay	8 Stiff sand to sand 9 Stiff fine-gr		
Dat geo Test	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. No warranty is provided as to the correctness or the applicability of any of the geotechnical soil and								Remarks		
design parameters shown and does not assume any liability for any use of th aware of the techniques and limitations of any method used to derive data sh					n any desig				Sheet 1 of 1		
		,									

			Client: Geotech Consulting Ltd					Bore No.: CPT003			
McMILLAN Drilling			Project: 256 Fitzgerald Avenue, Christchurch					Job No.: 19425			
\vdash	Site Location: 256 Fitzgerald	Avonuo Christch	urch				Date: 2/12/20	120	13423		
	-										
	Elevation: 0.00m	Datum: Gro	N (NZTM) - Map or aerial photograph Rig Operator: R. Wyllie ound Equipment: 14t truck mounted rig								
		A				EHAVIOUR TYPE NORMALISED)	E EST	ESTIMATED PARAMETERS			
drill	Tip Resistance (MPa)	Friction Ratio (%)	Pore Pressure (kPa)	Inclination (Degrees)	Scale	SBT	SBT Descriptio	Dr n (%)	Su (kPa)	N60	
Predrill				_	Š		(filtered)				
		F ∩ m 4 ∩ 0 ∩ m 0	0 200 400 600			-0w4r06r00			80 100 100 100 100 100 100 100 1	- 10 - 20 - 40	
		AL MAL			mpunpunpunpunpunpunpunpunpunpunpunpunpunp		Sand mixtures: silty s to sandy silt Sands: clean sands to sands Sand mixtures: silty s to sandy silt Sands: clean sands to sands	o silty		Land and the second	
	Cone Type: I-CFXY-10 - C Cone Reference: 110542 Cone Area Ratio: -		Water Le	drill: - evel: - pse: 2.2m		rmination et Depth:		ed	T) - Robertson Sand mixtu sand to sar Sands: clea	res: silty Idy silt	
	Tip Resistance	2012 Before test Aft 0.1551 -0.1 0.0292 0.02 -	357			ve Refusal Tip: Gauge: inometer:	2 Clay - or 3 Clays: cla	e fine-grained ganic soil ay to silty clay ures: clayey silt lay	silty sands	l to gravelly o clayey	
Da [:] geo Tes	ta shown on this report has beer otechnical soil and design paramet sting for Geotechnical Engineering, refully reviewed by the user. No w	ers using methods 4th Edition. The inte	published in P. K. erpretations are p	Robertson ar resented only	nd K.L. Cal as a guide	oal (2010), Gui e for geotechn	de to Cone Penetr ical use, and shou	ation Id be	:5		
des	sign parameters shown and does r are of the techniques and limitation	not assume any liab	oility for any use o	f the results ir	n any desig	•	-		Sheet 1 of 1		

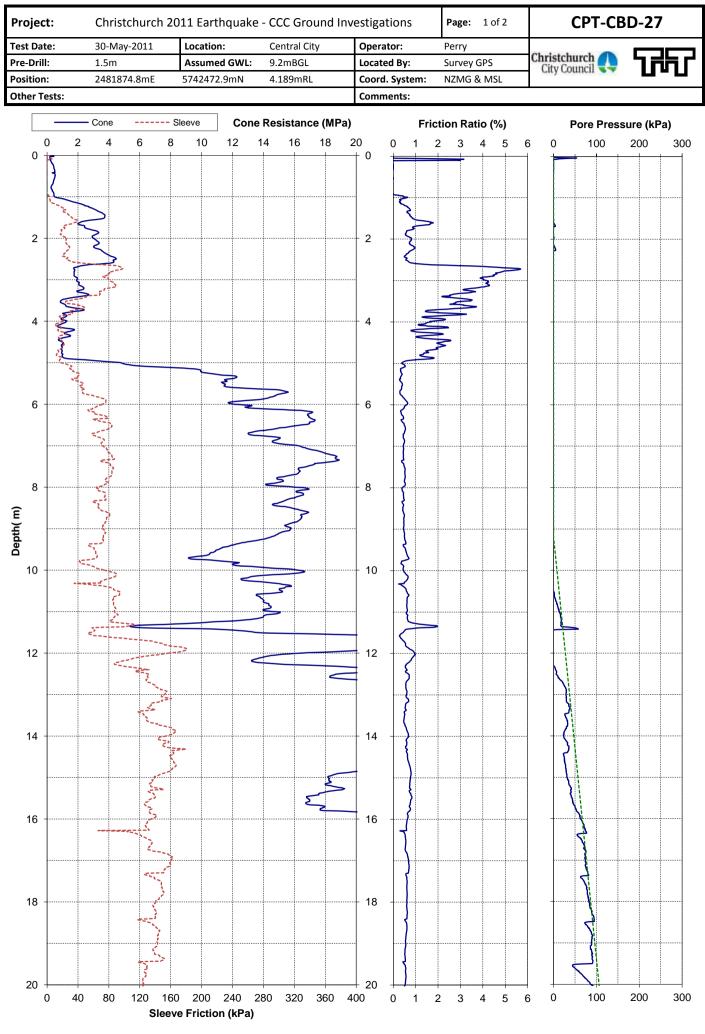
MCMILLAN Drilling	Client:	Bore No.: CPT004							
	Project: 256	stchurch	Job No.: 19425						
Site Location: 256 Fitzgerald Avenue, Christe Grid Reference: 1571873.4m E, 5180900.92m Elevation: 0.00m Datum: 0	N (NZTM) - Map or aeri	Date: 2/12/2 - Map or aerial photograph Rig Operator: R. Wyl Equipment: 14t tru				lie			
RAW DAT	A		EHAVIOUR TYPE -NORMALISED)						
Tip Friction Resistance Ratio (MPa) (%)	Pressure	nation grees) a S S S	SBT	SBT Description (filtered)	Dr 1 (%)	Su (kPa)	N ₆₀		
0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	200 200 200 200 200 200 200 200 200	2 21		Sand mixtures: silty si to sandy silt Sand mixtures: silty si to sandy silt					
Monton Anna A				Sands: clean sands to sands	silty Mary	U VIV V			
M. M. M.				Sands: clean sands to sands Sands: clean sands to sands			WWW.		
EOH: 18.26m Cone Type: I-CFXY-10 - Compression Cone Reference: 110542 Cone Area Ratio: - Standards: ISO 22476-1:2012 Zero load outputs (MPa) Before test A	Predrill: Water Level: Collapse: 5	3.1m 3.3m Targ	ermination et Depth: ive Refusal Tip:	0 Undefine	fine-grained	5 Sand mixtu sand to san Sands: clea silty sands	res: silty Idy silt		
Tip Resistance -0.1968 -0	0.1469 0287	Incl	Gauge: 🖌 🖪 Clays: clay		y to silty clay 8 Stiff sand to clayey sand				
Notes & Limitations Data shown on this report has been assessed to pro- geotechnical soil and design parameters using method Testing for Geotechnical Engineering, 4th Edition. The i carefully reviewed by the user. No warranty is provid- design parameters shown and does not assume any li aware of the techniques and limitations of any method	ation d be and	Sheet 1 of 1							



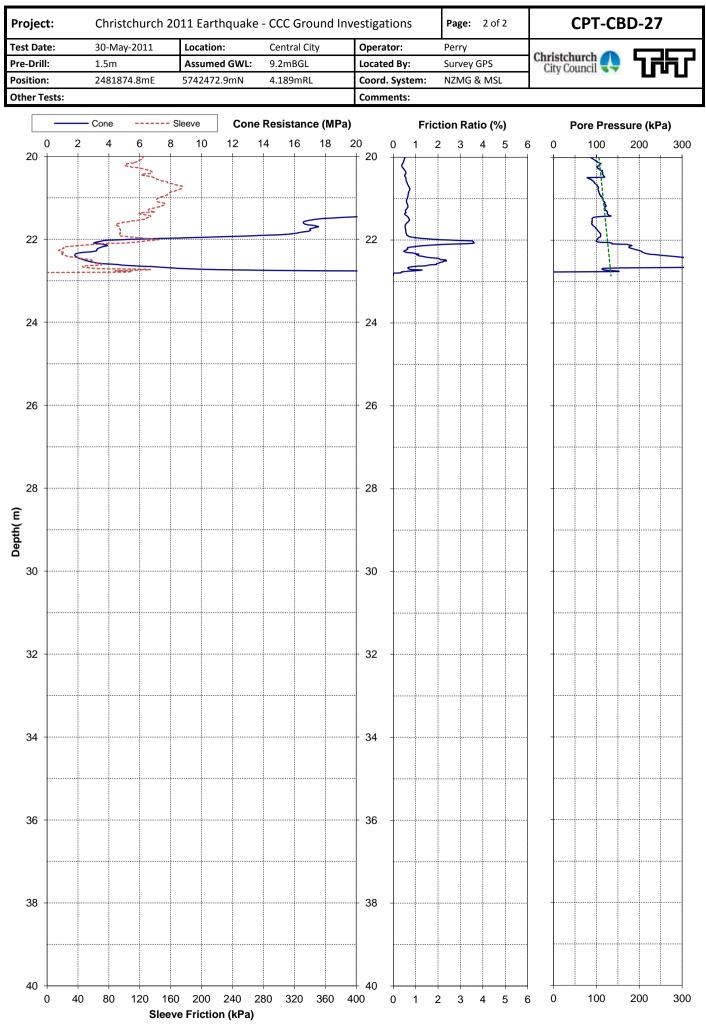
NZGD ID: CPT_564



NZGD ID: CPT_564



T+T Ref: 52000.3400



T+T Ref: 52000.3400

M	MILLAN Drilling	COI		IFTR		FST	Job:		133	02
							CPT No.:	NI		
Lo	Name: 20 Heywoo Client: Geotech C cation: 20 Heywoo	onsulting Ltd				Grid: Datum:	NZTM	E		5180935.17 1571868.35 -
		,-				Termination:	-	Hole De	pth (m):	18.05
		RAW DATA				IAVIOUR TYPE ORMALISED)	E	STIMATED	PARAM	ETERS
Predrill	Tip Resistance (MPa)	Friction Ratio (%)	Pore Pressure (kPa)	Scale	SBT	SBT Descriptio (filtered)	Dr n (%)		Su Pa)	N ₆₀
	- 10 - 20 - 40	- N M 4 15 10 - 80	- 0 - 200 - 400 - 600		− 0 m 4 m m − m m		60	150 80	-200 -250 -300 -350	
		WWW AM	•				~ ~ ~	× K		
						Sands: clean sands f silty sands Sands: clean sands				Mrand
٤	$\sum_{i=1}^{n}$			9		silty sands Sands: clean sands silty sands	10			
2						Sands: clean sands f silty sands Sands: clean sands f				
E						silty sands Sands: clean sands i silty sands	10	-		
				17		Sands: clean sands f silty sands				
EOH: 1	18,05m									
	Operator: S. Card	lona	Date:	: 03/03/201	4 Effect	ive Refusal	Soil Behaviour		Cand	
	ne Reference: 110542 ne Area Ratio: 0.75 Cone Type: -	PT	Predrill: Water Level: Collapse:	2.70	Incl	Tip: Gauge: ✔ inometer:	0 Undefined 1 Sensitive fir grained	ie-	sand f Sands to silty	mixtures: silty to sandy silt s: clean sands / sands
	ip Resistance (MPa) li			-5.0874		Other:	2 Clay - orgai	L	grave grave	e sand to Ily sand and to clayey
	Local Friction (MPa) li Pore Pressure (KPa) li		Final: Final:		Targ	et Depth:	Clay Silt mixtures silt & silty cl	s: clayey	• sand	ne-grained
Da	s & Limitations						BT) and various	Remarks Effective R		
Testin	chnical soil and design par g for Geotechnical Engine ully reviewed by the user.	ering, 4th Edition. T	he interpretations	s are preser	ited only as a guid	e for geotechnical u	use, and should be	Hole De	oth (m):	18.05
any o	f the geotechnical soil and		shown and does	s not assum	e any liability for a	iny use of the result	is in any design or		Sheet 1	



BOREHOLE LOG

BOREHOLE No: CBD 07 Hole Location: Harvey Tce

SHEET 1 OF 6

PROJECT: CHRIS	тсни	JRO	СН	CI	ΓY 2	2011	REMEDIAT	ION		LOC	ATIO	N: CEI	NTRAL	CIT	Υ		JOB No: 52000.3400
CO-ORDINATES	5742 2481				۱E					DRII	LTY	PE: R	otary				DLE STARTED: 11/7/11 DLE FINISHED: 12/7/11
R.L.	4.21 r									DRII	L ME	THOE	: OB/	Tripl	e Tube		ILLED BY: Pro-Drill
DATUM	NZM	IG								DRII	L FL	JID: N	lud				GGED BY: RKH/CP CHECKED: BMcD
GEOLOGICAL												(1)			ENGINE	ERING	DESCRIPTION
SEOLOGICAL UNIT, SENERIC NAME, DRIGIN, MINERAL COMPOSITION.		FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE WEATHERING	SSIF	T 10 SHEAR STRENGTH	200	250 DEFECT SPACING 250 DEFECT SPACING 2000 (mm)	SOIL DESCRIPTION Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filing.
HAND DIG FILL. (Potholed for servic									-	\bigotimes							Fill: Borehole drilled through pre-dug and backfilled pothole.
check and backfille	d.)			0	PRE-DUG			-4.0	0.5								
YALDHURST MEMBER OF THI SPRINGSTON FORMATION (ALLUVIAL)	3		-		SPT		1/1/0/1/0/1 N=2	-2.5	2.0		ML	S	S				Sandy SILT, brown. Soft, wet, non plastic. Sand is fine. 1.6m to 1.95m no recovery
				100	OB	*	¢ FC	B = -1.5	2.5-	* * * * * * *	SP	W	L				Silty, fine SAND, grey. Loose, wet.
									-	×	51		Ľ				
			-		SPT		2/1/2/2/2/2 N=8	-1.0	3.0-	* * * * *							3.45m to 3.85m no recovery
				62	OB	2	¢ FC	B = -0.0	4.0	× * * *							
			-		SPT		0/0/2/1/2/2 N=7	0.5	4.5-	× × × ×	SP	М	L				Fine SAND with some silt, grey. Loose, moist.

NZGD ID: BH_1740



BOREHOLE LOG

BOREHOLE No: CBD 07 Hole Location: Harvey Tce

SHEET 2 OF 6

PROJECT: CHRIS	тсн	UR	СН	CI	ΓY 2	201	1 REMEDIAT	101	N		LOC	ATIO	N: CEI	NTRAL	_ Cl	TY		JOB No: 52000.3400
CO-ORDINATES	574				. –						DRII	L TY	PE: R	otary			HC	LE STARTED: 11/7/11
R.L.	248 4.21		5.2	:5 m	ιĿ						DRIL	L ME	THOD	: OB/	Trip	le Tube		ILE FINISHED: 12/7/11 ILLED BY: Pro-Drill
R.L. DATUM	4.21 NZN										DRIL	L FL	JID: N	/lud				GGED BY: RKH/CP CHECKED: BMcD
GEOLOGICAL																ENGINE	RING	DESCRIPTION
geological Unit, generic Name, origin, Mineral composition.		FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE WEATHERING	STRENG CLASSIF	SHEAR	200 (KP3) 100 (KP3) 1 COMPRESSIVE 200 STRENGTH 100 (MPa)	50 DEFECT SPACING 1000 (mm)	SOIL DESCRIPTION Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.
YALDHURST MEMBER OF TH SPRINGSTON FORMATION (ALLUVIAL)	Е			100	SPT OB		* FC 1/1/2/2/5/6 N=15	B	-1.0	5.5- 6.0- 6.5-		SP	М	MD				Fine SAND with some silt, grey. Loose, moist. 5 - becoming medium dense 6.45m to 6.85m no recovery 6
				62	SPT OB		⊁ FC 1/1/2/2/2/4 N=10	В	3.0	7.0-								7
				100	OB				4.0	8.0-								 SAND becoming fine to medium 8 with extremely closely spaced laminated silt beds
CHRISTCHURCH FORMATION (MARINE & ESTUARINE)	[SPT		1/1/2/2/5/6 N=15		5.0	9.0-		SP	M	MD				Fine SAND with some silt, grey. Medium dense, moist. 9.45m to 9.9m no recovery 9

NZGD ID: BH_1740



BOREHOLE LOG

BOREHOLE No: CBD 07 Hole Location: Harvey Tce

SHEET 3 OF 6

PROJECT: CHRIS	тсні	IR	сн	CIT	Y	2011		TIC	N					NTRAL	CIJ	ΓY		JOB No: 52000.3400
CO-ORDINATES	5742	47	3 m	ηΝ		201			// 1				PE: R		. 011		НС	LE STARTED: 11/7/11
	2481		5.2	5 m	ιE						DRII	_L ME	THOD	: OB/	Tripl	e Tube		LE FINISHED: 12/7/11
R.L. DATUM	4.21 n NZM												JID: N		•			ILLED BY: Pro-Drill GGED BY: RKH/CP CHECKED: BMcD
GEOLOGICAL																ENGINE		DESCRIPTION
geological Unit, generic Name, origin, Mineral Composition.		FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL		SSIF	 25 26 26 26 26 27 26 27 26 27 26 27 26 26 27 26 26 27 26 26 27 26 26 27 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 26 2	200 200 50 50 50 50 50 50 50 50 50	⁵⁰ ²⁵⁰ DEFECT SPACING ¹⁰⁰⁰ ²⁰⁰⁰ (mm)	SOIL DESCRIPTION Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.
CHRISTCHURCH FORMATION				55					-	-	×	SW	W	MD				Fine to medium SAND with trace silt, grey. Medium dense, wet.
(MARINE & ESTUARINE)					HQTT				-6.0	10.5		ML	W	S				- 10.5m to 10.75m some shells SILT with some sand, blue grey. Soft, wet, low plasticity.
					SPT		1/1/2/2/3/4		-7.0	11.0	× × × × ×	SW	W	MD				1 Fine to medium SAND with some silt interbedded, grey. Medium dense, wet.
							N=11		7.5	11.5-	××××							11.45m to 11.7m no recovery 1
				76	HQTT					12.0	× , , , , , , , , , , , , , , , , , , ,							1
					SPT		3/1/3/5/9/7 N=24			-								1 12.75m to 13.25m no recovery
				71	HQTT				9.0	13.0-								1
									9.5	-	×							 extremely closely spaced thinly laminated silt bed
					SPT		3/4/4/4/5/9 N=22			14.0	* * * * *	SW	W	MD				Silty, fine to medium SAND, grey. Medium dense, wet. Silt is interbedded.
										14.5								14.35m to 14.75m no recovery 1
					HQTT				-10.	5 -								- contains some shells

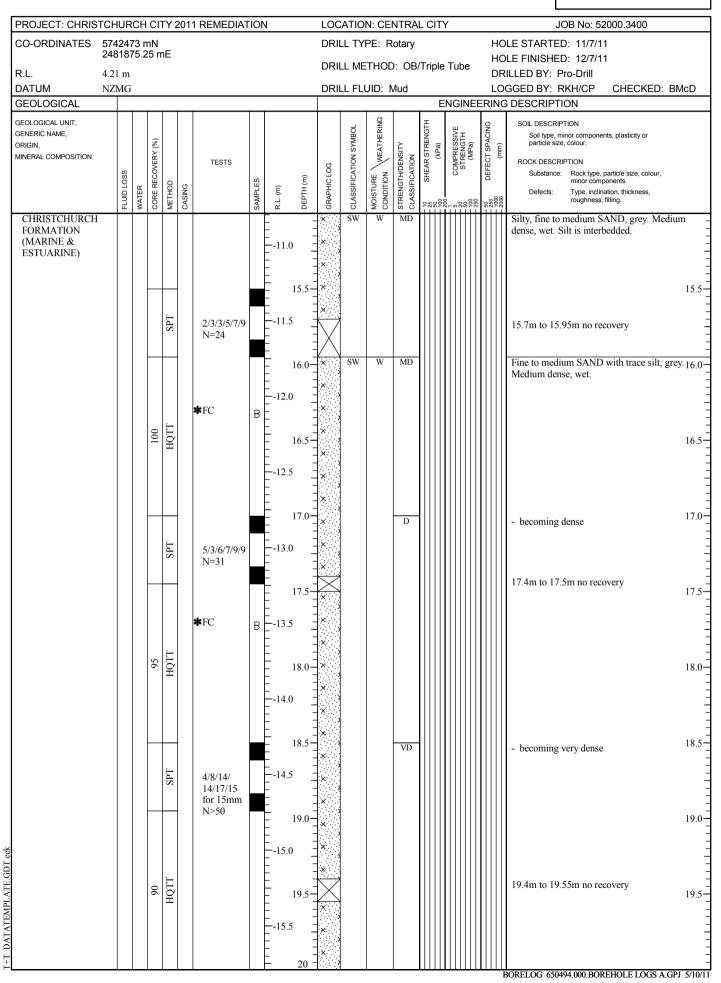
NZGD ID: BH_1740



BOREHOLE LOG

BOREHOLE No: CBD 07 Hole Location: Harvey Tce

SHEET 4 OF 6



NZGD ID: BH_1740



BOREHOLE LOG

BOREHOLE No: CBD 07 Hole Location: Harvey Tce

SHEET 5 OF 6

PROJECT: CHRIS	тсни	IRC	СН	СІЛ	Y 2	201	1 REMEDIA	TIO	N		LOC	ATIO	N: CEI	NTRAI	L CI	TY		JOB No: 52000.3400
	5742	47	3 m	۱N									PE: R				но	DLE STARTED: 11/7/11
- .	2481		5.2	5 m	ιE						DRIL	L ME	THOE): OB/	Trip	le Tube		LE FINISHED: 12/7/11
R.L. DATUM	4.21 r NZM												JID: N		•			ILLED BY: Pro-Drill GGED BY: RKH/CP CHECKED: BMcD
GEOLOGICAL			_	_							- 1 11					ENGINEER		DESCRIPTION
Geological Unit, Generic Name, Origin, Viineral composition.		FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR	100 (KF43) 100 (KF43) 100 (KF43) 100 STRENGTH 100 (MPa) 100 DEFECT SDACING	- 250 - 1000 - 2000 (mm)	SOIL DESCRIPTION Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.
CHRISTCHURCH FORMATION	[×	SW	W	MD				Fine to medium SAND with trace silt, grey. Medium dense, wet.
(MARINE & ESTUARINE)				97	HQTT				16.0 		× × X × × × × ×							20.4m to 20.45m no recovery 20
			-						-17.0		×							2
			-		SPT		4/9/12/12/ 16/10 for 35 mm N>50		-17.5	<u> </u>	××××							2
				100	HQTT				18.0		× >	ML	М	F				2 SILT with some sand, bluish grey. Firm,
									18.5	ייריין קיירייקיייי								moist, low plasticity. Sand is fine.
					SPT		1/9/16/26/8 for 25mm N>50	3	- 	<u>, , </u>		GW	W	VD				Sandy, fine to coarse GRAVEL, grey. Very ² dense, wet. Gravel is subangular to subrounded. Sand is medium to coarse. 23.15m to 23.45m no recovery
RICCARTON GRAVELS					_				-19.5		00000	GW	D	VD				Fine to coarse GRAVEL, grey. Very dense, 2 dry. Gravel is subangular to subrounded.
				43	HQTT				-20.0									23.9m to 24.5m no recovery 2
					SPT		4/4/5/5/6/7 N=23			᠄ ᠄	00000	SW	W	MD				Gravelly, medium to coarse SAND, yellowish brown. Medium dense, wet. Gravel is fine to coarse, subangular to subrounded.
					SPT				-	ז'ן ז' ו 5 י								

NZGD ID: BH_1740



BOREHOLE LOG

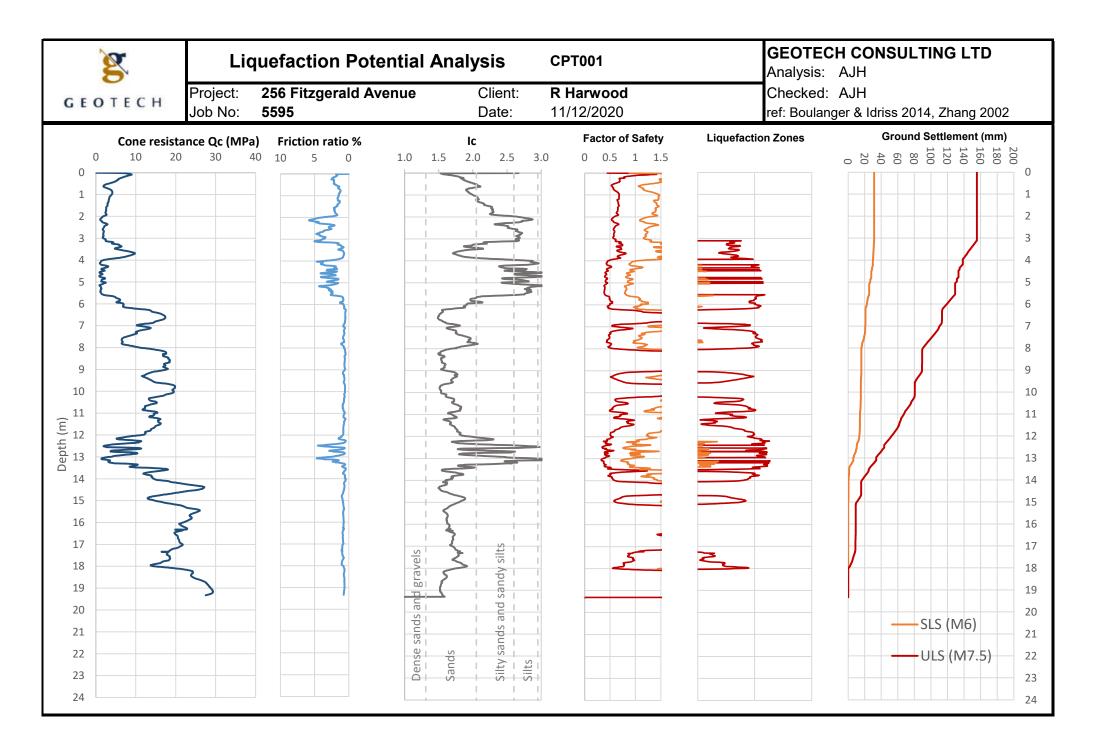
BOREHOLE No: CBD 07 Hole Location: Harvey Tce

SHEET 6 OF 6

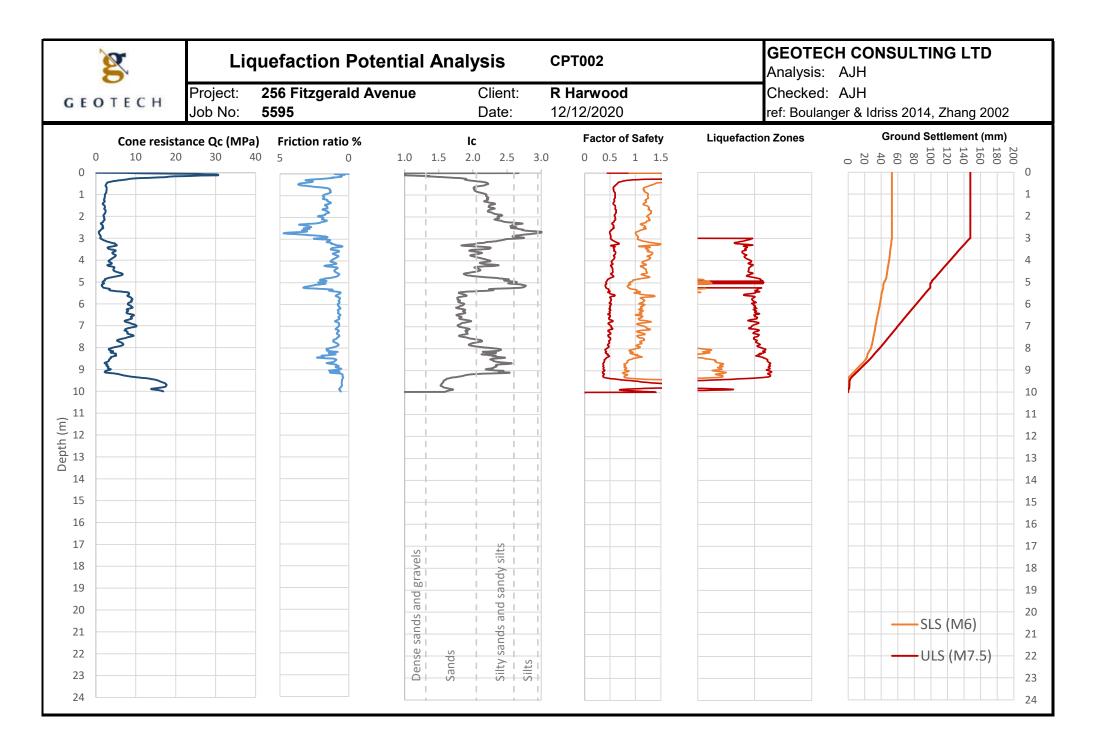
PROJECT: CHRIS	тсн	UR	RCH	I CI	TY	2011		ГЮ	N	LOC	ATIO	N: CEI	ITRAL	LC	ITY				JOB No: 52000.3400
CO-ORDINATES	574	24	73 r	nΝ						DRII	L TY	PE: R	otary					НС	DLE STARTED: 11/7/11
R.L.	248 4.21			25 N	nΕ					DRII	_L ME	THOD	: OB/	Trip	ole [.]	Tube	e		DLE FINISHED: 12/7/11 RILLED BY: Pro-Drill
DATUM	NZN									DRII	L FL	JID: N	1ud						GGED BY: RKH/CP CHECKED: BMcD
GEOLOGICAL															EN	IGIN	IEE	RIN	DESCRIPTION
geological Unit, generic Name, origin, Mineral Composition.		FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m) DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAF	± 100 (kPa)	COMPRESSIVE 20 STRENGTH		250 DEFECT SPACING 2000 (mm)	SOIL DESCRIPTION Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.
RICCARTON GRAVELS											GW	D	VD						Fine to coarse GRAVEL, grey. Very dense, dry. Gravel is subangular to subrounded. 25.15m to 26.0m no recovery
				19	HQTT				-21.5										25
					SPT		25/25 for 95mm N>50		26.0-	°0 °									26.1m to 26.45m no recovery
							14- 50		26.5-	000									- becoming very dense 20
				19	HQTT				-22.5										26.65m to 27.5m no recovery 2
					SPT		6/8/11/ 14/26 for 75mm				GW	W	VD						Sandy, fine to coarse GRAVEL, grey. Very ² dense, wet. Gravel is subangular to subrounded. Sand is medium to coarse. 27.55m to 27.95m no recovery
							N>50		-24.0		GW	D	VD						Fine to coarse GRAVEL, grey. Very dense, 2 dry. Gravel is subangular to subrounded. 28.2m to 29.0m no recovery
				24	HQTT				-24.5										Note: fines only recovered in SPT
					SPT		20/30 for 75mm		29.0-		GW	w	VD						Sandy, fine to coarse GRAVEL, grey. Very ^{2:} dense, wet. Gravel is subangular to Isubrounded. Sand is medium to coarse.
							N>50		25.0 29.5-										Subrounded. Sand is medium to coarse. 29.05m to 29.15m no recovery End of borehole at 29.15mbgl. Open standpipe piezometer installed. Please see attached diagram in Appendix F. 29
									E							$\left \right \left \right $			

NZGD ID: BH_1740

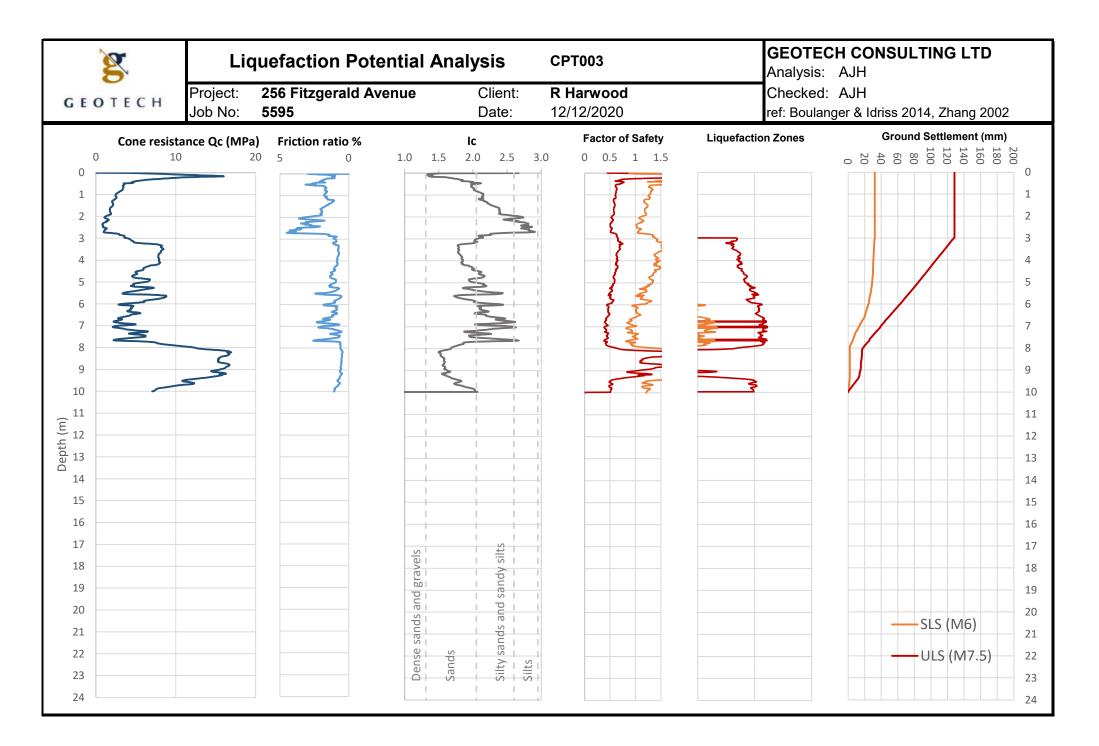
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	<u>x</u>	Liquefaction Potential Ar	nalysis	CPT001		GEOTECH CONSL Analysis: AJH	JLTING LTD
Job No: 5595 Date: 11/12/2020 ref: Boulanger & Idriss 201 Input Parameters Groundwater depth = 3 m Soil density γ = 17 kN/m ³ Fines fitting parameter C_{fc} = 0 Probability of Liquefaction = 0.15 (0.15 is standard deterministic model) Seismic Load Cases Case 1 Case 2 Case 3 Seismic Load Cases ULS at M7.5 SLS at M7.5 SLS at M6 Peak Ground Acceleration (PGA) = 0.35 0.13 0.19 Magnitude M = 7.50 7.50 6.00 Summary Results Overall settlement (Zhang) (mm): 155 20 31 Overall liquefiable thickness (m): 8.31 0.33 1.1 Settlement in top 10m (mm): 75 8 16	Proj	ect: 256 Fitzgerald Avenue	Clien	it: R Harwoo	bd	Checked: AJH	
$\begin{array}{cccc} Soil density \gamma = & 17 & kN/m^{3} \\ Fines fitting parameter C_{fc} = & 0 \\ Probability of Liquefaction = & 0.15 \\ sigma(lnR) = & 0.2 \end{array} (0.15 is standard deterministic model) \\ sigma(lnR) = & 0.2 \end{array}$	Job	No: 5595	Date	: 11/12/2020	0	ref: Boulanger & Idriss 2	2014, Zhang 2002
Fines fitting parameter $C_{fc} = 0$ Probability of Liquefaction = 0.15 (0.15 is standard deterministic model) sigma(lnR) = 0.2Seismic Load CasesCase 1 ULS at M7.5Case 2 	rameters	Groundwater depth = 3	m				
Seismic Load Cases Case 1 Case 2 Case 3 ULS at M7.5 SLS at M7.5 SLS at M6 Peak Ground Acceleration (PGA) = 0.35 0.13 0.19 Magnitude M = 7.50 6.00 6.00 representative M = 6.80 7.50 6.00 Overall settlement (Zhang) (mm): 155 20 31 Total liquefiable thickness (m): 8.31 0.33 1.1 Settlement in top 10m (mm): 75 8 16		Soil density $\gamma = 17$	kN/m	1 ³			
Seismic Load Cases Case 1 Case 2 Case 3 ULS at M7.5 SLS at M7.5 SLS at M6 Peak Ground Acceleration (PGA) = 0.35 0.13 0.19 Magnitude M = 7.50 6.00 6.00 Summary Results 0 0.2 31 Overall settlement (Zhang) (mm): 155 20 31 Total liquefiable thickness (m): 8.31 0.33 1.1 Settlement in top 10m (mm): 75 8 16		Fines fitting parameter $C_{fc} = 0$					
ULS at M7.5 SLS at M7.5 SLS at M6 Peak Ground Acceleration (PGA) = Magnitude M = representative M = 0.35 0.13 0.19 Magnitude M = representative M = 7.50 7.50 6.00 Summary Results 6.80 7.50 6.00 Overall settlement (Zhang) (mm): 155 20 31 Total liquefiable thickness (m): 8.31 0.33 1.1 Settlement in top 10m (mm): 75 8 16		•	``	is standard dete	erministic model)		
Magnitude M = 7.50 7.50 6.00 representative M = 6.80 7.50 6.00 Summary Results	Seis			ULS at M7.5	SLS at M7.5	SLS at M6	Case 4 22 Feb 2011
Magnitude M = representative M = 7.50 7.50 6.00 Summary Results 6.80 7.50 6.00 Overall settlement (Zhang) (mm): 155 20 31 Total liquefiable thickness (m): 8.31 0.33 1.1 Settlement in top 10m (mm): 75 8 16		Peak Ground Acceleration (PG/					0.45
representative M = 6.80 7.50 6.00 Summary Results Coverall settlement (Zhang) (mm): 155 20 31 Overall settlement (Zhang) (mm): 155 20 31 1.1 Total liquefiable thickness (m): 8.31 0.33 1.1 1.1 Settlement in top 10m (mm): 75 8 16 16		•					6.20
Overall settlement (Zhang) (mm):1552031Total liquefiable thickness (m):8.310.331.1Settlement in top 10m (mm):75816		8		6.80	7.50	6.00	6.20
Total liquefiable thickness (m):8.310.331.1Settlement in top 10m (mm):75816	Sun	nmary Results					
Settlement in top 10m (mm): 75 8 16		Overall settlement (Zhang) (m	וm):	155		31	149
		· ·	• •				7.70
Liquefiable thickness in top $10m (m)$: 3.62 0.00 0.47					-		74
		Liquefiable thickness in top 10m (• •	3.62	0.00	0.47	3.56
Average MSF = 1.000 1.000 1.372		•			1.000		1.314
LSN ('mm') 14 1 3			,		1		14
LDI (m) 1.59 0.09 0.21 For free face of 4 m, LDI = 0.77 0.02 0.09			· · /				1.55 0.78



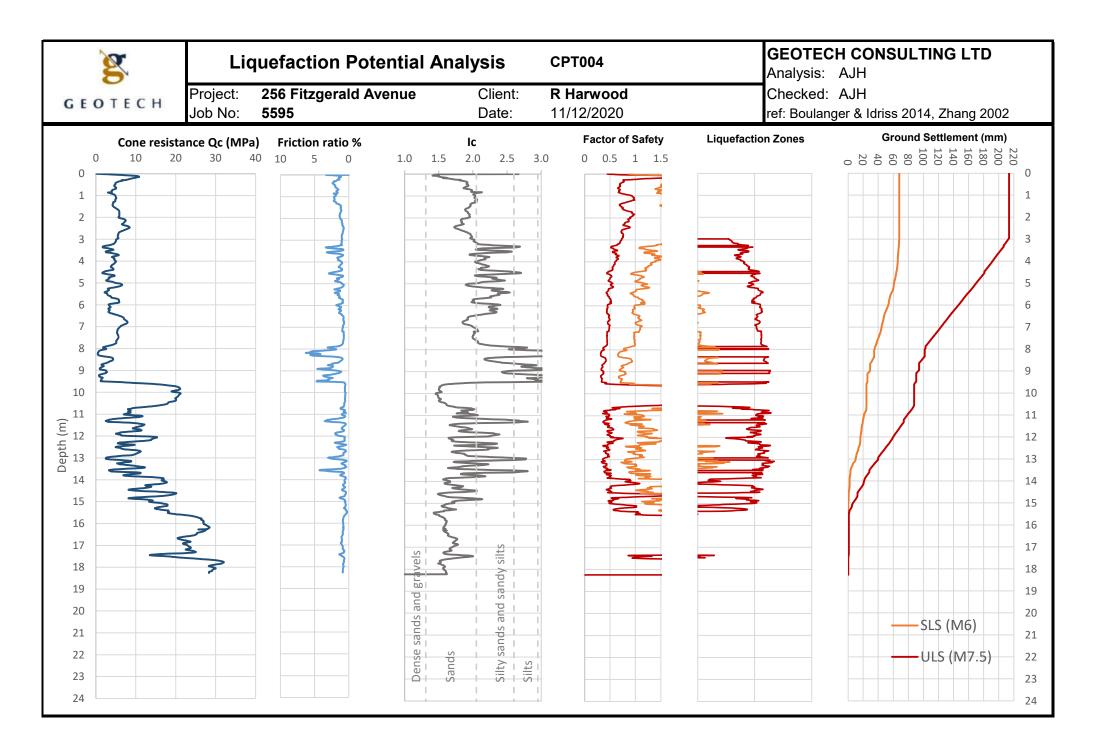
2	Liquefaction Potential Analy	ysis CPT002		GEOTECH CONSU Analysis: AJH	ILTING LTD
GEOTECH	Project: 256 Fitzgerald Avenue	Client: R Harwoo	bd	Checked: AJH	
GEOTECH	Job No: 5595	Date: 12/12/202	0	ref: Boulanger & Idriss 2	2014, Zhang 2002
Input Parameters	Groundwater depth = 3	m			
•	Soil density $\gamma = 17$	kN/m ³			
	Fines fitting parameter $C_{fc} = 0$				
	Probability of Liquefaction = 0.15 sigma(InR) = 0.2	(0.15 is standard det	erministic model)		
	Seismic Load Cases	Case 1	Case 2	Case 3	Case 4
	Seismic Load Cases	Case 1 ULS at M7.5	Case 2 SLS at M7.5	Case 3 SLS at M6	Case 4 22 Feb 2011
	Seismic Load Cases Peak Ground Acceleration (PGA) =	ULS at M7.5			
	Peak Ground Acceleration (PGA) = Magnitude M =	ULS at M7.5 0.35 7.50	SLS at M7.5 0.13 7.50	SLS at M6 0.19 6.00	22 Feb 2011 0.45 6.20
	Peak Ground Acceleration (PGA) =	ULS at M7.5 0.35 7.50	SLS at M7.5 0.13	SLS at M6 0.19	22 Feb 2011 0.45
	Peak Ground Acceleration (PGA) = Magnitude M =	ULS at M7.5 0.35 7.50	SLS at M7.5 0.13 7.50	SLS at M6 0.19 6.00	22 Feb 2011 0.45 6.20
	Peak Ground Acceleration (PGA) = Magnitude M = representative M = Summary Results Overall settlement (Zhang) (mm):	ULS at M7.5 0.35 7.50 6.80 147	SLS at M7.5 0.13 7.50 7.50 27	SLS at M6 0.19 6.00 6.00 53	22 Feb 2011 0.45 6.20 6.20 147
	Peak Ground Acceleration (PGA) = Magnitude M = representative M = Summary Results Overall settlement (Zhang) (mm): Total liquefiable thickness (m):	ULS at M7.5 0.35 7.50 6.80 147 6.33	SLS at M7.5 0.13 7.50 7.50 27 0.75	SLS at M6 0.19 6.00 6.00 53 1.5	22 Feb 2011 0.45 6.20 6.20 147 6.29
	Peak Ground Acceleration (PGA) = Magnitude M = representative M = Summary Results Overall settlement (Zhang) (mm): Total liquefiable thickness (m): Settlement in top 10m (mm):	ULS at M7.5 0.35 7.50 6.80 147 6.33 147	SLS at M7.5 0.13 7.50 7.50 27 0.75 27	SLS at M6 0.19 6.00 6.00 53 1.5 53	22 Feb 2011 0.45 6.20 6.20 147 6.29 147
	Peak Ground Acceleration (PGA) = Magnitude M = representative M = Summary Results Overall settlement (Zhang) (mm): Total liquefiable thickness (m): Settlement in top 10m (mm): Liquefiable thickness in top 10m (m):	ULS at M7.5 0.35 7.50 6.80 147 6.33 147 6.33	SLS at M7.5 0.13 7.50 7.50 27 0.75 27 0.75	SLS at M6 0.19 6.00 6.00 53 1.5 53 1.46	22 Feb 2011 0.45 6.20 6.20 147 6.29 147 6.29
	Peak Ground Acceleration (PGA) = Magnitude M = representative M = Summary Results Overall settlement (Zhang) (mm): Total liquefiable thickness (m): Settlement in top 10m (mm): Liquefiable thickness in top 10m (m): Average MSF =	ULS at M7.5 0.35 7.50 6.80 147 6.33 147 6.33 1.000	SLS at M7.5 0.13 7.50 7.50 27 0.75 27 0.75 1.000	SLS at M6 0.19 6.00 6.00 53 1.5 53 1.5 53 1.46 1.723	22 Feb 2011 0.45 6.20 6.20 147 6.29 147 6.29 1.611
	Peak Ground Acceleration (PGA) = Magnitude M = representative M = Summary Results Overall settlement (Zhang) (mm): Total liquefiable thickness (m): Settlement in top 10m (mm): Liquefiable thickness in top 10m (m):	ULS at M7.5 0.35 7.50 6.80 147 6.33 147 6.33 1.000 26	SLS at M7.5 0.13 7.50 7.50 27 0.75 27 0.75	SLS at M6 0.19 6.00 6.00 53 1.5 53 1.46	22 Feb 2011 0.45 6.20 6.20 147 6.29 147 6.29



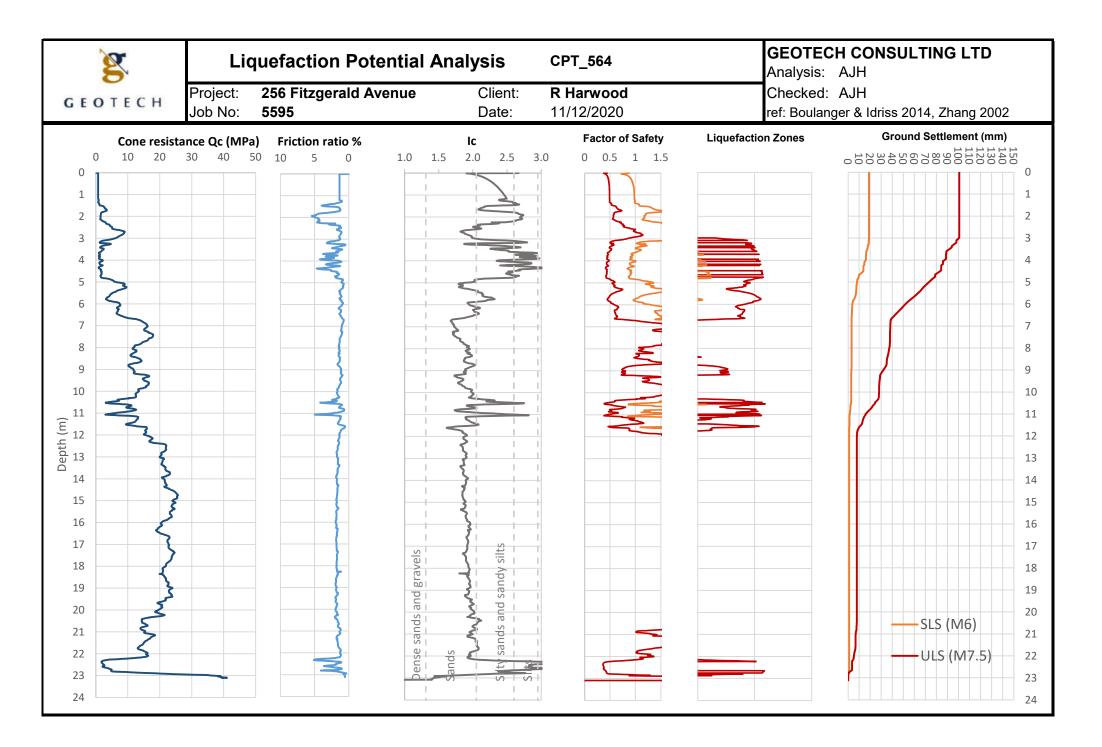
GEOTECH		ysis	CPT003		Analysis: AJH	JLTING LTD
GEOTECH	Project: 256 Fitzgerald Avenue	Client:	R Harwood		Checked: AJH	
	Job No: 5595	Date:	12/12/2020		ref: Boulanger & Idriss 2	2014, Zhang 2002
Input Parameters	Groundwater depth = 3	m				
•	Soil density $\gamma = 17$	kN/m ³				
	Fines fitting parameter $C_{fc} = 0$					
	Probability of Liquefaction = 0.15 sigma(InR) = 0.2	(0.15 is s	tandard detern	ninistic model)		
	Seismic Load Cases	ULS	ase 1 at M7.5	Case 2 SLS at M7.5	Case 3 SLS at M6	Case 4 22 Feb 2011
	Peak Ground Acceleration (PGA) =	. (0.35	0.13	0.19	0.45
	Magnitude M =		7.50	7.50	6.00	6.20
	representative M =		6.80	7.50	6.00	6.20
	Summary Results					
	Overall settlement (Zhang) (mm):		128	16	32	127
	Total liquefiable thickness (m):		5.79	0.00	1.1	5.73
	Settlement in top 10m (mm):		128	16	32	127
	Liquefiable thickness in top 10m (m):		5.79	0.00	1.14	5.73
	Average MSF =		.000	1.000	1.352	1.297
	LSN ('mm')		23	3	5	23
	LDI (m) For free face of 4 m, LDI =		1.54 1.38	0.06 0.05	0.22 0.21	1.53 1.38



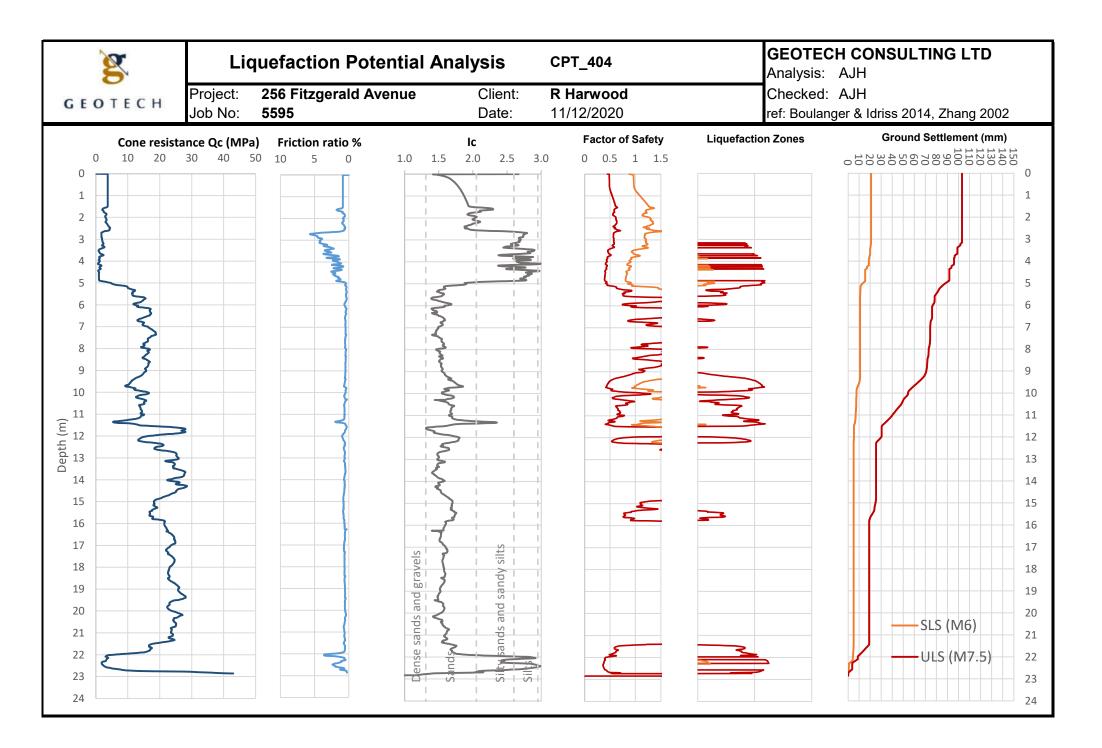
8	Liquefaction Potential Analy	ysis CPT004		GEOTECH CONSU Analysis: AJH	ILTING LTD
GEOTECH	Project: 256 Fitzgerald Avenue Job No: 5595	Client: R Harwo Date: 11/12/20		Checked: AJH ref: Boulanger & Idriss 2	2014, Zhang 2002
nput Parameters	Soil density $\gamma = 17$ Fines fitting parameter C _{fc} = 0	m kN/m ³			
	Probability of Liquefaction = 0.15 sigma(InR) = 0.2	(0.15 is standard de	terministic model)		
	Seismic Load Cases	Case 1	Case 2	Case 3	Case 4
	Seismic Load Cases	Case 1 ULS at M7.5	Case 2 SLS at M7.5	Case 3 SLS at M6	Case 4 22 Feb 2011
	Peak Ground Acceleration (PGA) =	ULS at M7.5 0.35	SLS at M7.5 0.13	SLS at M6 0.19	22 Feb 2011 0.45
	Peak Ground Acceleration (PGA) = Magnitude M =	ULS at M7.5 0.35 7.50	SLS at M7.5 0.13 7.50	SLS at M6 0.19 6.00	22 Feb 2011 0.45 6.20
	Peak Ground Acceleration (PGA) =	ULS at M7.5 0.35 7.50	SLS at M7.5 0.13	SLS at M6 0.19	22 Feb 2011 0.45
	Peak Ground Acceleration (PGA) = Magnitude M = representative M = Summary Results	ULS at M7.5 0.35 7.50 6.80	SLS at M7.5 0.13 7.50	SLS at M6 0.19 6.00	22 Feb 2011 0.45 6.20
	Peak Ground Acceleration (PGA) = Magnitude M = representative M = Summary Results Overall settlement (Zhang) (mm):	ULS at M7.5 0.35 7.50 6.80 214	SLS at M7.5 0.13 7.50 7.50 39	SLS at M6 0.19 6.00 6.00 6.00	22 Feb 2011 0.45 6.20 6.20 211
	Peak Ground Acceleration (PGA) = Magnitude M = representative M = Summary Results Overall settlement (Zhang) (mm): Total liquefiable thickness (m):	ULS at M7.5 0.35 7.50 6.80 214 9.72	SLS at M7.5 0.13 7.50 7.50 39 0.62	SLS at M6 0.19 6.00 6.00 6.00	22 Feb 2011 0.45 6.20 6.20 211 9.44
	Peak Ground Acceleration (PGA) = Magnitude M = representative M = Summary Results Overall settlement (Zhang) (mm): Total liquefiable thickness (m): Settlement in top 10m (mm):	ULS at M7.5 0.35 7.50 6.80 214 9.72 126	SLS at M7.5 0.13 7.50 7.50 0.62 20	SLS at M6 0.19 6.00 6.00 6.00 6.00 6.00	22 Feb 2011 0.45 6.20 6.20 211 9.44 126
	Peak Ground Acceleration (PGA) = Magnitude M = representative M = Summary Results Overall settlement (Zhang) (mm): Total liquefiable thickness (m): Settlement in top 10m (mm): Liquefiable thickness in top 10m (m):	ULS at M7.5 0.35 7.50 6.80 214 9.72 126 5.39	SLS at M7.5 0.13 7.50 7.50 0.62 20 0.23	SLS at M6 0.19 6.00 6.00 6.00 6.00 6.00 6.00 6.00 4.00 4	22 Feb 2011 0.45 6.20 6.20 211 9.44 126 5.38
	Peak Ground Acceleration (PGA) = Magnitude M = representative M = Summary Results Overall settlement (Zhang) (mm): Total liquefiable thickness (m): Settlement in top 10m (mm): Liquefiable thickness in top 10m (m): Average MSF =	ULS at M7.5 0.35 7.50 6.80 214 9.72 126 5.39 1.000	SLS at M7.5 0.13 7.50 7.50 0.62 20 0.23 1.000	SLS at M6 0.19 6.00 6.00 6.00 68 2.8 44 2.04 1.174	22 Feb 2011 0.45 6.20 6.20 211 9.44 126 5.38 1.147
	Peak Ground Acceleration (PGA) = Magnitude M = representative M = Summary Results Overall settlement (Zhang) (mm): Total liquefiable thickness (m): Settlement in top 10m (mm): Liquefiable thickness in top 10m (m):	ULS at M7.5 0.35 7.50 6.80 214 9.72 126 5.39 1.000 23	SLS at M7.5 0.13 7.50 7.50 0.62 20 0.23	SLS at M6 0.19 6.00 6.00 6.00 6.00 6.00 6.00 6.00 4.00 4	22 Feb 2011 0.45 6.20 6.20 211 9.44 126 5.38



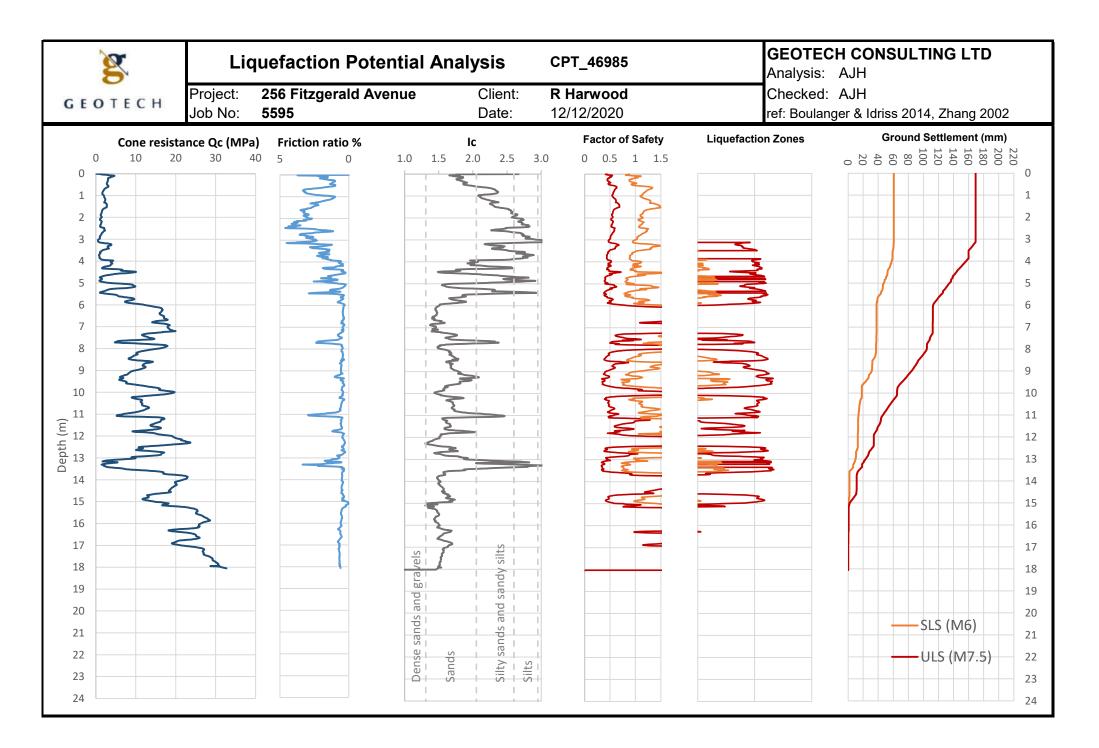
8	Liquefaction Potential Anal	ysis (CPT_564	GEOTECH CONSI Analysis: AJH	ULTING LTD
GEOTECH	Project: 256 Fitzgerald Avenue Job No: 5595	-	R Harwood 1/12/2020	Checked: AJH ref: Boulanger & Idriss	2014, Zhang 2002
Input Parameters	$\begin{array}{llllllllllllllllllllllllllllllllllll$	m kN/m ³	ndard deterministic model)		
	sigma(InR) = 0.2 Seismic Load Cases	Case	e 1 Case 2	Case 3	Case 4
		ULS at		SLS at M6	22 Feb 2011
	Peak Ground Acceleration (PGA) = Magnitude M = representative M =	7.5	0 7.50	0.19 6.00 6.00	0.45 6.20 6.20
	Magnitude M =	7.5	0 7.50	6.00	6.20
	Magnitude M = representative M =	7.5 6.8 10 4.7 73 3.2 1.00 14	0 7.50 0 7.50 1 9 3 0.06 6 8 0.00 00 1.000 1	6.00	6.20



8	Liquefaction Potential Anal	ysis	CPT_404		GEOTECH CONSU Analysis: AJH	JLTING LTD
GEOTECH	Project: 256 Fitzgerald Avenue Job No: 5595	Client: Date:	R Harwood 11/12/2020		Checked: AJH ref: Boulanger & Idriss	2014, Zhang 2002
Input Parameters	$\begin{array}{llllllllllllllllllllllllllllllllllll$	m kN/m ³ (0.15 is s	tandard detern	ninistic model)		
	Seismic Load Cases		ase 1 at M7.5	Case 2 SLS at M7.5	Case 3 SLS at M6	Case 4 22 Feb 2011
	Peak Ground Acceleration (PGA) = Magnitude M = representative M =	- 7).35 7.50 6.80	0.13 7.50 7.50	0.19 6.00 6.00	0.45 6.20 6.20
	Summary Results					
	Overall settlement (Zhang) (mm): Total liquefiable thickness (m): Settlement in top 10m (mm): Liquefiable thickness in top 10m (m): Average MSF = LSN ('mm') LDI (m) For free face of 4 m, LDI =	5 2 1	103 5.50 49 2.41 .000 8 1.03 0.37	13 0.19 5 0.00 1.000 1 0.11 0.01	21 0.7 13 0.48 1.080 3 0.25 0.15	96 4.98 46 2.20 1.067 8 0.99 0.36



8	Liquefaction Potential Anal	ysis CPT_46	985	GEOTECH CONSU Analysis: AJH	ILTING LTD
GEOTECH	Project: 256 Fitzgerald Avenue Job No: 5595	Client: R Harw Date: 12/12/2		Checked: AJH ref: Boulanger & Idriss 2	2014, Zhang 2002
Input Parameters	Groundwater depth = 3	m			
	Soil density $\gamma = 17$	kN/m ³			
	Fines fitting parameter $C_{fc} = 0$				
	Probability of Liquefaction = 0.15 sigma(InR) = 0.2	(0.15 is standard d	eterministic model)		
	Seismic Load Cases Peak Ground Acceleration (PGA) =	Case 1 ULS at M7.5 0.35	Case 2 SLS at M7.5 0.13	Case 3 SLS at M6 0.19	Case 4 22 Feb 2011 0.45
	Peak Ground Acceleration (PGA) =				
	Magnitude M =		7.50	6.00	6.20
	representative M =		7.50	6.00	6.20
	Summary Results				
	Overall settlement (Zhang) (mm):	169	33	61	165
				0.0	7.53
	Total liquefiable thickness (m):	7.96	0.69	2.2	
	Total liquefiable thickness (m): Settlement in top 10m (mm):	7.96 104	19	42	103
	Total liquefiable thickness (m): Settlement in top 10m (mm): Liquefiable thickness in top 10m (m):	7.96 104 4.53	19 0.33	42 1.61	103 4.48
	Total liquefiable thickness (m): Settlement in top 10m (mm): Liquefiable thickness in top 10m (m): Average MSF =	7.96 104 4.53 1.000	19 0.33 1.000	42 1.61 1.068	103 4.48 1.057
	Total liquefiable thickness (m): Settlement in top 10m (mm): Liquefiable thickness in top 10m (m): Average MSF = LSN ('mm')	7.96 104 4.53 1.000 18	19 0.33 1.000 3	42 1.61 1.068 7	103 4.48 1.057 18
	Total liquefiable thickness (m): Settlement in top 10m (mm): Liquefiable thickness in top 10m (m): Average MSF =	7.96 104 4.53 1.000 18 2.07	19 0.33 1.000	42 1.61 1.068	103 4.48 1.057



APPENDIX C4

C



Densified Crust Method Statement (reinforced crushed gravel raft) (Type G1d)

This method is generally suitable for most sites where the water table is at least 1.0m below ground level.

The crushed gravel raft is to be a minimum of 1.2m deep (below the underside of foundation elements) over the entire house footprint, and extend a minimum of 1.0m beyond the perimeter foundation line. The raft is to be constructed of crushed gravels comprising TNZ M/4 40mm or equivalent (eg crushed AP40 with at least 70% stone having 2 or more broken faces. Outside reinforced grid zones, crushed AP65 can be used).

Two layers of geogrid are incorporated into the raft to add resilience and improve the ability of the crust to resist differential settlement and (in the case of lateral stretch) fracturing/ pulling apart. In areas of 'major' lateral stretch as defined within these guidelines, a third layer of geogrid is incorporated.

It may be necessary to batter the sides of the excavation, and provide a drainage sump to remove ground water for the duration of the excavation, filling and compaction work. This method may have limited application where the groundwater level is high and a 'dry' and stable excavation cannot be practically formed.

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A resource consent for dewatering may be required, particularly if the site is potentially contaminated. The potential effects on settlement of neighbouring properties needs to be assessed when designing the dewatering system.

Step	Type G1d – Typical Activity Sequence for Densified Crust (reinforced crushed gravel raft)
1d.1	Set out perimeter of foundation treatment area and locate marker pegs clear of all workings. Remove all topsoil and other unsuitable materials.
1d.2	During excavation any organic material is to be removed from site and reported to the Design Engineer.
1d.3	Any physical obstructions encountered during excavation shall be reported to the Design Engineer for further direction.
1d.4	Excavation in strips or sections may be necessary due to site constraints such as adjacent properties or the physical shape of the house. In this case additional care is required at the vertical edge joins by cutting into the previous compacted zone at 2h:1v to ensure compaction integrity is attained across the joins.
1d.5	Commence excavation to 1.2m (below the underside of foundation elements) and if water is present, construct dewatering sump adjacent to work area. Install pump in the sump and pipe to sediment control.
1d.6	Level and compact the base of the excavation. Static compaction is likely to be required in wet or saturated subgrade to avoid fluidizing and/or heaving the ground.
1d.7	The base of the excavation should be stable (not yielding) prior to backfilling. In the event that soft areas are present in the base layer and the target compaction is not achieved, the soft materials should be removed and replaced with suitable material placed and compacted as described in step 1a.9. The base can also be stabilised by placing a layer of compacted rock or crushed concrete (dia. ≤ 150mm) over the soft area to create a 'working platform'. A nonwoven geotextile fabric separation layer comprising Bidim A19 or equivalent should be placed both under and over the 'platform' to prevent potential migration of soil into voids within the rock/concrete. Alternatively, cement can to be added and mixed into the first 200mm of the subgrade layer to stabilise it. The amount of cement required to stabilise moist (not saturated) soil will be in the order of 8% by weight. The mixed layer should be compacted to the extent practicable and allowed to harden prior to placing any additional fill.
1d.8	Place the first 200mm layer (loose thickness) of crushed gravel and compact as described in step 1a.9, then install two layers of geogrid (refer the preferred performance characteristics above – refer to section C4.1 for further information) separated by a 200mm thick layer of compacted fill. The grid should extend neatly to the sides of the excavation, and be lapped at joints as specified by the manufacturer Prior to placing fill on top of the geogrid, it is important that the grid is sufficiently tensioned to remove any wrinkles, bulges, etc . Note that three layers of geogrid, each separated by 200mm of compacted crushed gravel, are required in areas of 'major' lateral stretch as defined in this document.

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APF	PEN	D	X	C4

1d.9	Backfill the excavation by placing crushed gravel fill in horizontal loose layers not exceeding 200mm in thickness, moisture conditioned as necessary, and compacting to achieve a minimum of:	
	• 95% standard or 92% of vibrating hammer compaction (NZS 4402:1988 – Test 4.1.1 or Test 4.1.3); or	
	• 82% of the solid density of the fill material – (well-graded sandy gravel only, refer to section 4.1). Target density by this method is 2180 kg/m ³	
	Perform compaction testing at 600mm vertical intervals within the fill at a minimum frequency of 1 test for each 50m ² of treatment area or a minimum of 3 tests per layer.	
1d.10	Remove dewatering pump and sump once clear of the water table. Backfill and compact as for the foundation treatment work area.	
1d.11	Provide the Design Engineer with complete records of: 1) the material used to construct the raft; 2) results of laboratory MDD/moisture content or solid density tests of backfill materials; 3) results of field compaction testing of backfill; and 4) an 'as-built' plan. Documentation of other relevant details (ie stabilisation of the excavation subgrade with cement or rock) should also be provided. Field compaction test results should include depth below ground level, and horizontal locations relative to a fix point such as a corner of the excavation, and the depth below the top of the raft.	



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