BEFORE INDEPENDENT HEARING COMMISSIONERS IN CHRISTCHURCH

TE MAHERE Ā-ROHE I TŪTOHUA MŌ TE TĀONE O ŌTAUTAHI

IN THE MATTER of the Resource Management Act 1991

AND

IN THE MATTER of the hearing of submissions on Plan Change 14 (Housing and Business Choice) to the Christchurch District Plan

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STATEMENT OF PRIMARY EVIDENCE OF ANDREW JAMES HURLEY ON BEHALF OF THE GLENARA FAMILY TRUST

(Submitter 91 / Further Submitter 2070)

QUALIFYING MATTER - SPECIFIC PURPOSE ŌTĀKARO AVON RIVER CORRIDOR ZONE

ALTERNATIVE ZONING PROVISIONS FOR PRE-EXISTING RESIDENTIAL ACTIVITIES IN THE ŌTĀKARO AVON RIVER CORRIDOR ZONE

Dated 20 September 2023

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Introduction

1. My full name is Andrew James Cornelius Hurley. I am a director of Geotech Consulting (NZ) Ltd.

2. I am a chartered professional engineer with over thirty-five years' experience.

3. For the past seven years I have primarily practiced in earthquake geotechnical engineering in the Canterbury region.

4. I hold a bachelor's degree in civil engineering from Canterbury University (1983) and have a certificate of proficiency in Advanced Geotechnical Earthquake Engineering (2017).

5. I carried out a geotechnical assessment on the Glenara site, at 254- 256 Fitzgerald Avenue and 5 Harvey Terrace, in February 2021. A copy of that report is attached as **Appendix 1**.

6. I am very familiar with the geotechnical provisions of the *Guidance on repairing and rebuilding houses affected by the Canterbury earthquakes* issued by MBIE in 2012, with subsequent updates. This guidance established the Technical Classification system for Christchurch land (TC1 to TC3) and provides advice on methods for assessment of sites and design of foundations.

7. I have read and agree with, the evidence of Marie-Claude Hébert, geotechnical expert for the Christchurch City Council, and I have read submissions #54 from Shirley Van Essen and #794 from Greg Partridge. My opinion on geotechnical aspects of these submissions is provided from paragraph 30 below.

Code of Conduct

8. While this is a Council hearing, I have read the Code of Conduct for Expert Witnesses (contained in the Environment Court Practice Note 2023) and agree to comply with it. Except where I state I rely on the evidence of another person, I confirm that the issues addressed in this statement of evidence are within my area of expertise, and I have not omitted to consider material facts known to me that might alter or detract from my expressed opinions.

Executive Summary

9. My 2021 report concludes that the site is subject to liquefaction effects including TC2-like settlements in a 1 in 25 year Serviceability Limit State (SLS) earthquake and TC3-like settlements in a 1 in 500 year year Ultimate Limit State (ULS) earthquake. There is some potential for lateral spread, assessed as **minor** at SLS and **minor to moderate** at ULS, with the

risk being reduced by the presence of a large palisade wall constructed by the City Council along the Avon River bank following the Christchurch earthquakes.

10. I have recommended foundation systems that can be generically described as shallow ground improvement, by means of a geogrid reinforced gravel raft, with a TC2 enhanced foundation slab system on top. Systems of this type are recommended in the MBIE Guidance for TC3 and TC2/TC3 hybrid sites as they are tolerant of ground movement, in the range assessed, and the gravel raft provides a firm base from which the concrete foundations can be relevelled in future.

11. My report (Appendix 1) was limited to proposed buildings up to two storeys high, but this limitation is for compatibility with the MBIE guidance. In my opinion there are no geotechnical issues which would prevent development of the site with three storey structures provided that there is appropriate specific foundation design and consideration of the ground conditions.

12. Ms Hebert addresses the possibility of 4-6 storey development.[1] Ms Hebert considers that development of up to 6 storeys at the site may be feasible so long as detailed geotechnical analysis and design information is provided at the consent stage. I agree that development of up to 6 storeys is likely to be technically feasible with appropriate additional detailed geotechnical analysis and design.

13. Ms Van Essen's submission appears to be based on assumptions about the nature of TC3 classified ground and that certain prescriptive solutions are the only possible responses. This view fails to recognise that the *Guidance* is simply that - guidance. It conveniently provides some agreed processes and solutions that can be adopted to give a reasonably economic way of continuing development in most liquefaction prone areas. It is not prescriptive and it is not intended to prevent development. For most TC3 sites the proposed methods of ground improvement in the guidance can be expected to bring a site back to TC2-like levels of performance and in some cases it would be practical to improve a site to TC1.

14. Mr Partridge's submission raises concerns of increased seismic hazard primarily from the Alpine and Christchurch Faults. My understanding of the most recent seismicity modelling is that the liquefaction load cases, recommended for assessment by MBIE, adequately represent the hazard from these faults.

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[1] Statement of Primary Evidence of Marie-Claude Hebert 11 August 2023 at [38]-[41].

Summary of my 2021 report

15. Field investigations, on which I based my assessment, were carried out in December 2020. The on-site testing was supplemented by information from the NZ Geotechnical Database (NZGD), which provides a repository of records from the Canterbury Earthquakes and subsequent field test data.

16. I assessed the Glenara site as likely to be affected by 'index' settlements of 20-40 mm in a Serviceability Limit State (SLS[2]) earthquake and 80-150 mm in an Ultimate Limit State (ULS[3]) earthquake. This is TC2 performance at SLS and TC3 performance at ULS.

17. I note here that there is a minor error in my report, in that the actual results at SLS indicate a range of 20-50 mm. However, this error does not affect the conclusions or recommendations.

18. Fitzgerald Avenue, adjacent to the site, was badly affected by lateral spread in the February 2011 earthquake. However, the location was remediated by construction of a substantial palisade wall along the riverbank by the City Council. The method of construction for that wall will inhibit the development of liquefaction in critical soil layers and thus limit the propagation of future lateral spreads toward the Glenara site.

19. Based on the presence of this wall and the recorded performance of the site in the earthquakes, I assessed the lateral stretch risk as **minor** at SLS and **minor to moderate** at ULS.

20. The combination of assessment for 'index' settlement and lateral stretch results in an overall assessment of the Glenara site as a hybrid TC2/TC3 (SLS/ULS).

21. My recommendations for foundation systems were for shallow ground improvement by way of geogrid reinforced gravel rafts to a TC3 standard for the sites along Fitzgerald Avenue and to a TC2/TC3 hybrid (with supplementary geogrid) for 5 Harvey Terrace.

22. My report references the assessment as being for buildings up to two-storeys high. This is for compatibility with the MBIE Guidance, which provides standardised foundation solutions for buildings at that scale.

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[2] For buildings of normal importance the SLS earthquake is a 1 in 25 year event with characteristic parameters of 0.19 g peak ground acceleration and Magnitude 6 in Christchurch.

[3] The ULS earthquake is a 1 in 500 year event with characteristic parameters of 0.35 g peak ground acceleration and Magnitude 7.5 in Christchurch

Three storey development

23. Subsequent to my 2021 report the possibility of three storey development has been raised and I have been asked whether three storey development would be possible.

24. I do not consider there are any material constraints to three storey development, provided that appropriate specific design of the foundation system is carried out at the design stage.

25. In this respect I agree with the discussion by Ms Hébert at paragraphs 35 to 37 of her evidence that the potential effects of liquefaction below 10 m depth should be considered in the selection of a suitable foundation system and that aspects of serviceability and relevellability will need to be taken into account by the foundation designer.

4-6 storey development

26. The prospect of four to six storey development is raised by Ms Hébert at paragraphs 38 to 41 of her evidence. In summary she considers that such buildings may be possible with appropriate additional assessment and design, the likely foundation system is suggested as likely to include deep ground improvement.

27. Two significant concerns are discussed by Ms Hébert, firstly; a practical consideration about the potential effects of a significant civil engineering operation on neighbouring properties, and second; a caution that the combined cost of additional ground improvement and foundation works may make a development financially unviable. These are both valid issues and I agree they will need to be addressed by any future developer proposing buildings of this scale.

28. At paragraph 41 Ms Hébert provides helpful commentary on how an application for such a proposal is likely to be treated at Council.

29. I agree that a foundation system, based on deep ground improvement, may result in a site that is technically feasible for buildings of four to six storeys. This is because the deep ground improvement can effectively mitigate liquefaction effects over the depth of treatment.

Comments on submissions

30. In submission #54 Ms Van Essen makes the following comments:

- TC3 land is the *least able to support the weight of buildings* and the *most at risk of subsidence and liquefaction*.
- Buildings in TC3 areas should be as *lightweight as possible* and *at most 2 storeys high*.

- Buildings should have a TC3 Ribraft foundation, in order to level the house after the next earthquake.
- *TC3 land is absolutely unsuited to large heavy buildings covering most of the site.*

31. In my opinion these comments show a misunderstanding of the intent of the TC3 classification, as follows:

    (a) Whilst I agree that TC3 is generally more at risk of liquefaction damage than TC1 & TC2. I disagree with the statement that TC3 sites are *least able to support the weight of buildings*. There are many sites in Christchurch that are TC2 classified but have poor ground conditions such that expensive foundation systems are required to achieve acceptable building performance, even in the absence of liquefaction. The point here is that understanding of ground conditions and selection of appropriate remediation and/or foundation systems is the key to supporting the weight of buildings on any site.

    (b) A design principle expressed in the MBIE Guidance is that lightweight materials are preferred, however, the same paragraph also states that *"Heavier weight construction materials are however not precluded, and could still be used where supported by appropriate engineering advice and careful design of ground improvement or deep pile systems"*. Furthermore. more recent guidance on foundation design for liquefied ground[4] does not express any preference for lighter weight building components.

    (c) The **TC3 Ribraft** foundation is one solution of many provided in the MBIE Guidance and is certainly not mandated. At the time the guidance was written the **TC3 Ribraft** was not commercially developed and is only discussed in the Guidance as a concept for specific design. Shallow ground improvement by way of gravel raft, with a TC2 foundation on top, is another recommended method for achieving a relevellable foundation. I have recommended this latter system for the Glenara site.

    (d) There are substantial parts of central Christchurch where site performance was equivalent to TC3 and the typical building form is multi-storey, heavy construction. New buildings on these sites need to adopt foundation systems that are appropriate to the expected site performance in an earthquake.

32. In submission #794 Mr Partridge discusses the risks arising from various faults within and around Christchurch, specifically:

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[4] NZ Geotechnical Society Inc. (2021). *Earthquake geotechnical engineering practice Module 4: Earthquake resistant foundation design.* Wellington: MBIE

- The Alpine Fault and the increased activity that has been revealed by recent research.

- The Christchurch Fault and its proximity to the central city and hence the Glenara site on Fitzgerald Avenue.

33. I disagree with the concerns raised in submission #794 for the following reasons:

    (a) The Alpine Fault is capable of generating large earthquakes and recent research has shown an increased *likelihood* of occurrence but has not materially increased the *consequences*. At its closest the Alpine Fault passes 110 km from Christchurch City and the ground accelerations expected to be felt in the city are less than those experienced in the main Christchurch Earthquakes and less than that considered in the design ULS earthquake analysed in my report.

    The GNS Science, National Seismic Hazard Model[5] (NSHM) was updated in 2022 to include the latest available research on frequency and directivity effects from the Alpine Fault (amongst other earthquake sources). Figure 1 shows the estimated PGA, aggregated from all sources, over the central South Island. The region close to the Alpine Fault can be seen to have much higher ground accelerations than Christchurch.

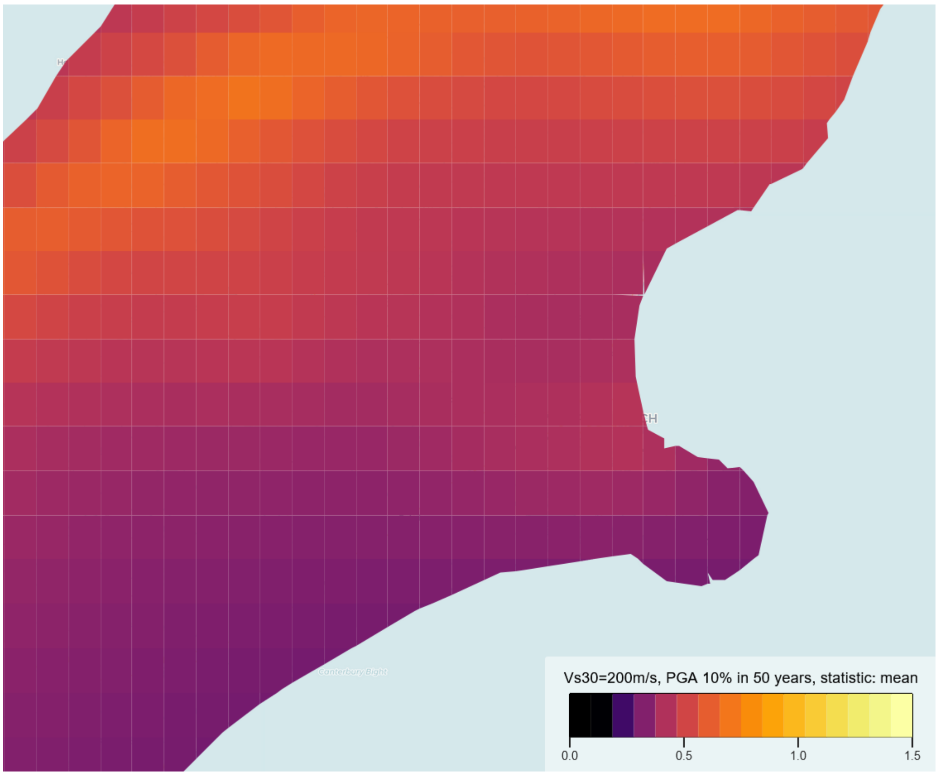

Figure 1 PGA Map from NSHM2022

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[5] https://nshm.gns.cri.nz/

Importantly, the relative contribution of the Alpine Fault can be seen in 'disaggregation plots' (Figure 2). Here the Alpine Fault can be seen to contribute approximately 2% to the Christchurch earthquake hazard, with the highest contribution coming from smaller, nearby faults.

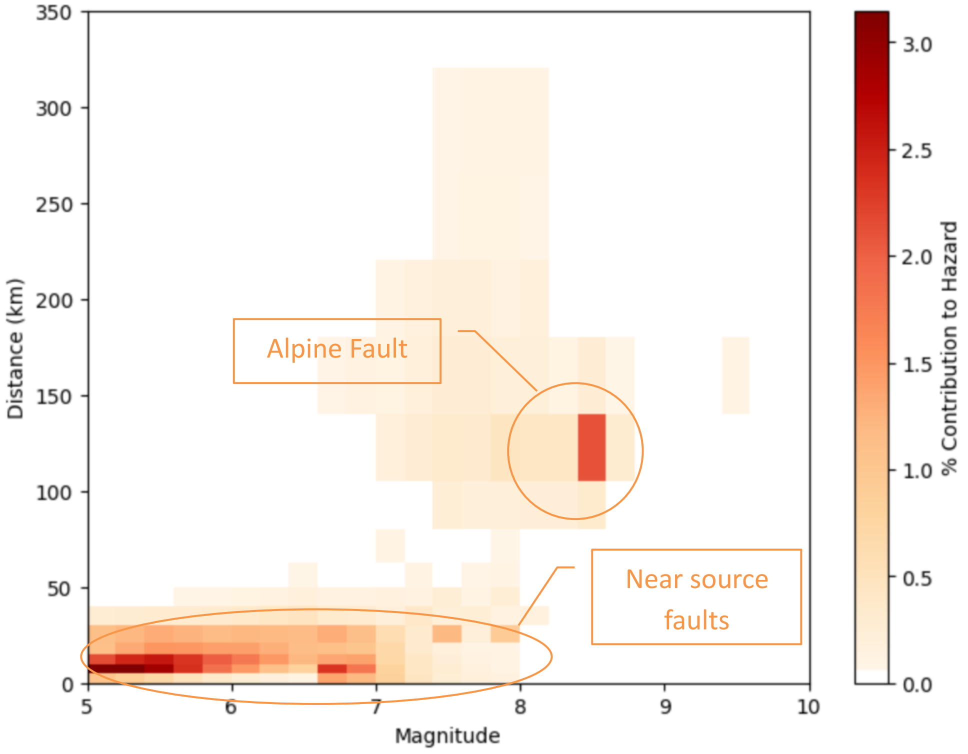

Figure 2 **Disaggragation of seismic hazard for Christchurch (source NSHM2022)**

(b) The Christchurch Fault is not documented in the GNS Active Faults database[6] and little is known about its seismicity. It is an example of one of the unknown near-source faults that are included in the NSHM. Observations of activity on this inferred fault, around Boxing Day 2010 and in the 1869 Christchurch earthquake of 1869, suggest that it tends to produce small earthquakes around Magnitude 5. There are some reports of minor liquefaction effects from the Boxing Day earthquakes and liquefaction has been inferred to have occurred in highly susceptible sites in the 1869 earthquake. However, effects will certainly have been very limited in comparison to the February 2011 Christchurch earthquake, for which the effects are well documented at the Glenara site and have been considered in my assessment.

I would anticipate that the largest expected earthquake on this fault would be less that the modelled SLS earthquake (M6, 0.19g) and is certainly less than the modelled ULS

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[6] https://data.gns.cri.nz/af/

earthquake (M7.5, 0.35g) that anticipates an unknown fault, somewhat larger than the Greendale Fault, rupturing within a 15 km distance of the site.

Conclusion

34. In conclusion I note that my 2021 report finds the site to have TC2-like liquefaction behaviour with SLS (1 in 25 year) ground shaking and TC3-like behaviour at ULS (1 in 500 year). This assessment includes consideration of the benefit of the palisade wall along the Avon River bank, adjacent to the site in restricting lateral spread.

35. I have recommended a suitable foundation system, including shallow ground improvement and a TC2 raft foundation, for buildings that fit within the scope of MBIE Guidance.

36. I agree that more significant buildings, between three and six storeys, may be feasible subject to further assessment of the ground conditions and specific design of the foundations and/or ground improvement to suit.

37. Accordingly, I disagree with the submission of Ms Van Essen, that “TC3” land is not able to be developed with heavy buildings. In fact, the purpose of ground improvement, either shallow or deep, is to mitigate liquefaction effects such that “TC3” land can be built on with effects on the buildings then being similar to TC2.

38. I also consider that the seismicity concerns of Mr Partridge are adequately covered by the liquefaction load cases prescribed by MBIE and used in my analysis.

Andrew Hurley
20 September 2023

Appendix 1

Geotechnical Assessment February 2021

GEOTECH
consulting ltd

Subdivision of
254-256 Fitzgerald Avenue
Richmond
Christchurch

Geotechnical Assessment Report

REFERENCE NUMBER: 5595

DATE: February 2021

PREPARED FOR: Ms R Harwood

PREPARED BY: Geotech Consulting Ltd

ENQUIRIES TO: Andrew Hurley

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Issue	Date	Author
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GEOLOGICAL & ENGINEERING SERVICES

Summary

Site & Sub-surface Conditions	Terrain	Near flat site but with Avon River passing 30 m to the west and approximately 4 m below site level.
	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.
	Soil classification	Class D, deep soil site to NZS1170.5:2004
	Groundwater depth	3 m median depth on east side of site with fall to the river of 0.3 m over the site length.
Seismic Aspects	Earthquake performance	Well tested to > SLS shaking in the September 2010 and February 2011 earthquakes, with moderate to severe liquefaction effects recorded.
	Liquefaction	Significant liquefaction throughout the soil profile at ULS but in isolated layers and typically below 5 m depth at SLS.
	Liquefaction ‘index’ settlement	SLS: 20 - 40 mm ULS: 80 – 150 mm (for top 10m of soil profile)
	Lateral spread	**Minor to moderate** spread is predicted, based on construction of the CCC palisade wall protecting the Avon River-bank along Fitzgerald Ave, following the Christchurch earthquake.
	Foundation technical category	Red-zone by MBIE classification Hybrid TC2/TC3 (SLS/ULS) by assessment
Natural hazards	Slippage	Low risk, except under liquefaction conditions when lateral spread may be an issue. The Avon River palisade wall has mitigated this risk.
	Subsidence	Liquefaction settlement is expected in major earthquakes. Risk can be minimised by following MBIE Guidance and recommendations of this report.
	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.
Site Development	Proposal	New two-storey apartment blocks on Lots 2 and 3.
	Suitable foundation	TC2 Enhanced slab foundations are suitable, with shallow ground improvement.
	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.
	Suitability for subdivision	Suitable for subdivision in terms of RMA section 106 requirements

Subdivision

254-256 Fitzgerald Avenue, Christchurch

Geotechnical Assessment Report

Contents

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 m^2 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.

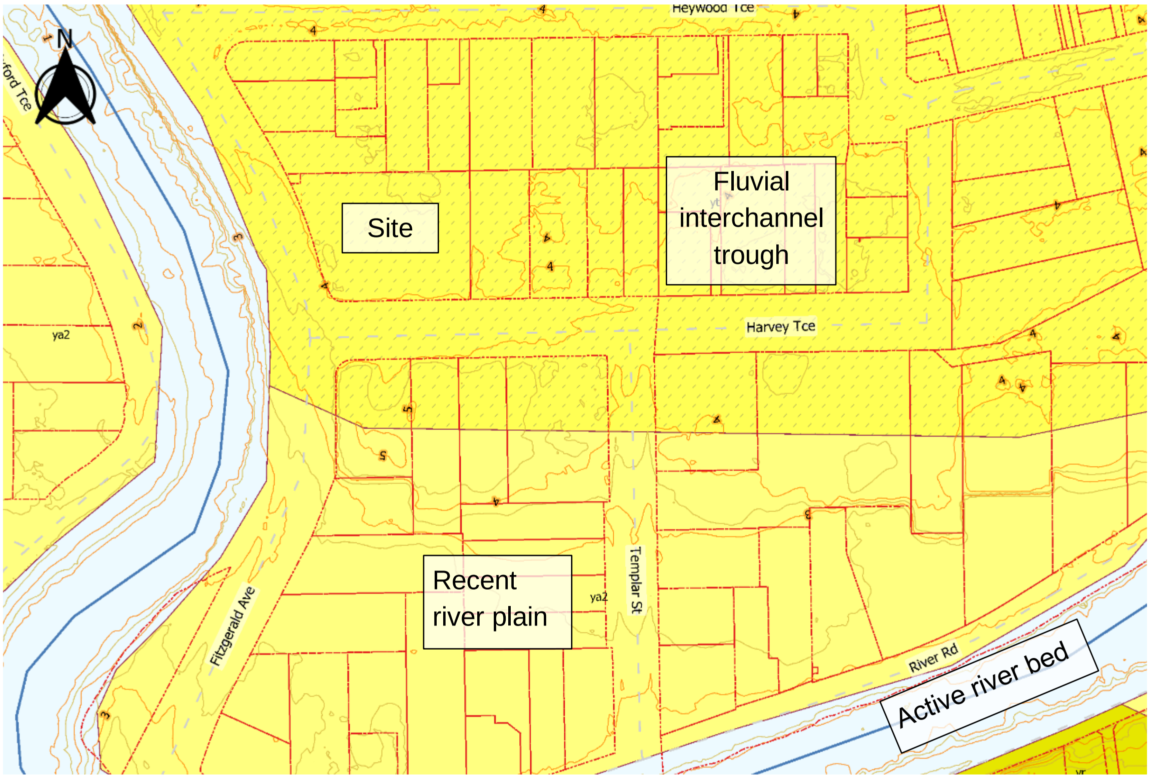

Figure 2-1 Geomorphic map data (ref GNS Geological Map 3)

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	8 m south on Harvey Tce adjacent to CPT_404	29.2
CPT_46985	25 m north on 20 Heywood Tce	18.1

Table 2-1 NZGD deep soil test information

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).

A general description of the ground conditions is:

Depth to top surface (m)	Thickness (m)	Description
0	0.4 to 0.8	Historic fill, buried topsoil in places.
0.4 to 0.8	≈ 5 (up to 9m in CPT04)	Interbedded silts and sands – generally loose and soft 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)

Table 3-1 Generalised soil profile

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

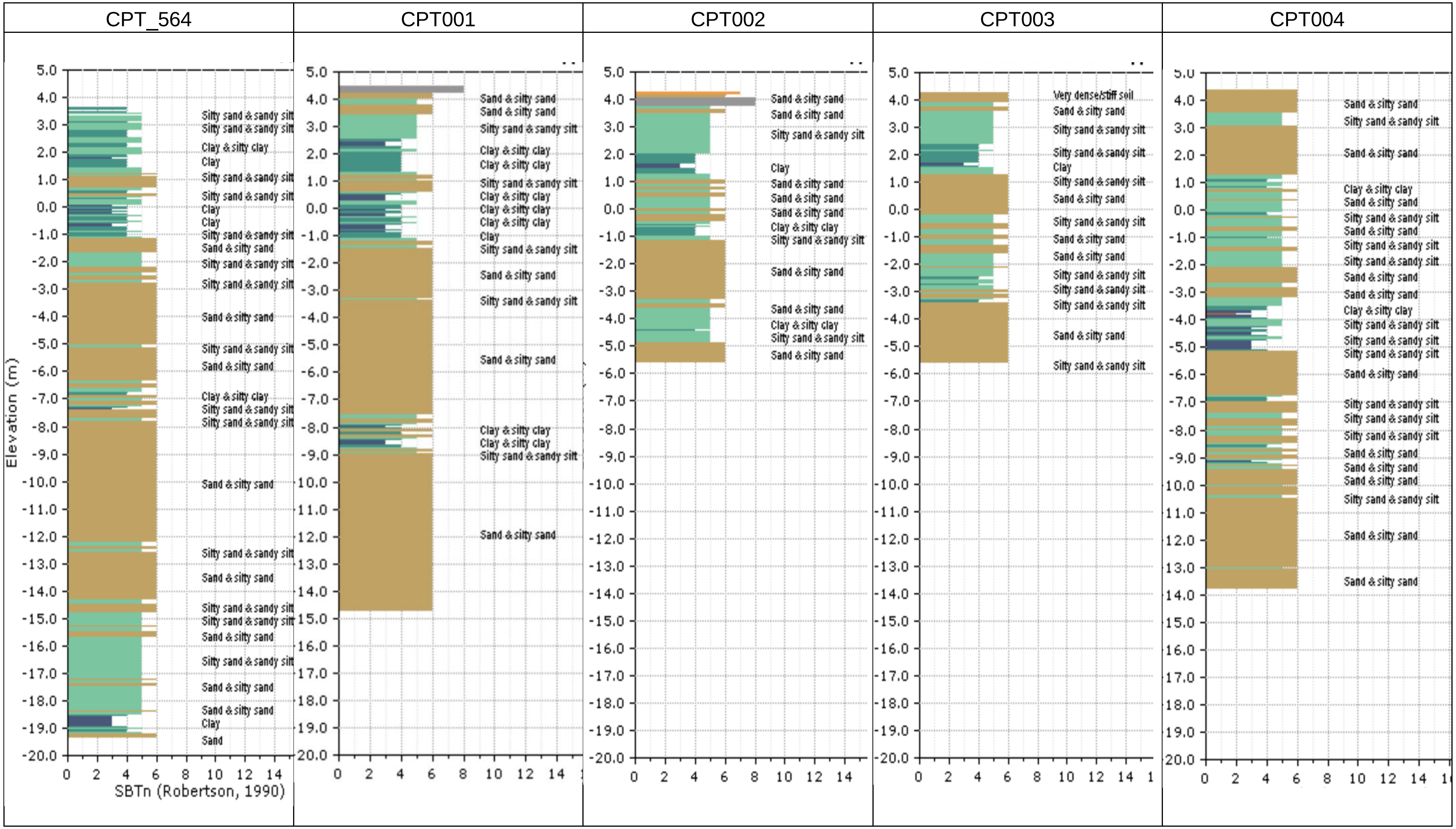

Figure 3-1 Interpretation of soil properties from CPT data

3.2 Groundwater

The Groundwater Surface Elevation studies (GNS Science, 2014) suggests a median groundwater elevation[1] 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%$_{\text{ile}}$ 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
SLS_A	0.13g	M7.5	25 yr
SLS_B	0.19g	M6	25 yr
ULS	0.35g	M7.5	500 yr

Table 4-1 Seismic design cases for liquefaction assessment

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[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.

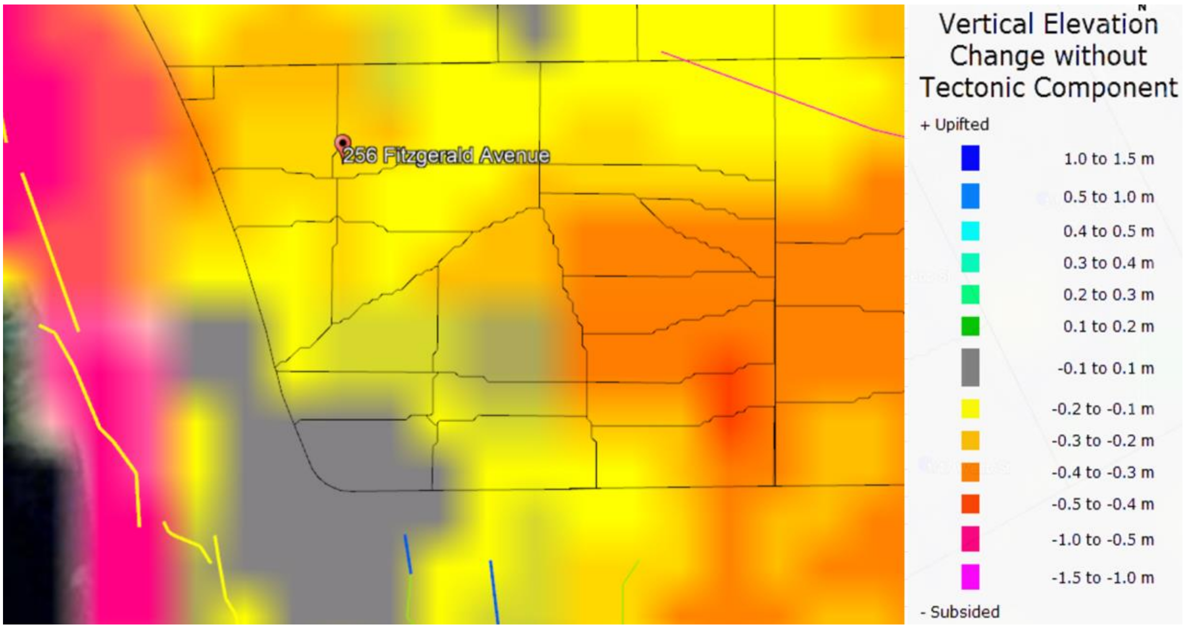

Figure 4-2 Liquefaction settlements - all events

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.

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4.5 Liquefaction potential

4.5.1 Analysis

Analysis of the on-site CPT's has been carried out using the methods recommended by MBIE[2]. 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.

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

CPT	Depth (m)	Liquefaction Induced Settlement (mm)				Sum of liquefiable layer thickness (m)			
		ULS M7.5	SLS$_A$ M7.5	SLS$_B$ M6	Feb '11 M6.2	ULS M7.5	SLS$_A$ M7.5	SLS$_B$ M6	Feb '11 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 than 10m (full profile)									
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 ±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

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[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 that the river bank does not erode in this location	No change in risk.
Falling debris	The site is flat with no source area for falling debris.	No change
Subsidence	There is a liquefaction risk at the site which is likely to result in some subsidence in a future earthquake.	Building in accordance with the recommendations of MBIE for liquefaction prone sites will mitigate this risk.
Slippage	There is a risk of lateral spread associated with liquefaction and proximity to the Avon River, in a ULS earthquake	Development does not change this risk but building in accordance with the recommendations of MBIE for liquefaction prone sites will protect life in the event that some slippage takes place.
Inundation	The site is not in the CCC Flood Management Area	No change in risk

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

MBIE. (2012, December). *Guidance on repairing and rebuilding houses affected by the Canterbury earthquakes.* Ministry of Building Innovation and Employment.

Begg, J. G., Jones, K. E., & Barrell, D. J. (2015). *Geology and geomorphology of urban Christchurch and eastern Canterbury. GNS Science Geological Map 3.* Lower Hutt: GNS Science.

Bradley, & Hughes. (2012). *"Conditional PGA for Liquefaction Assessment" Map Layer CGD5110 - 22 September 2016*. Retrieved from NZ Geotechnical Database: www.nzgd.org.nz

EQC. (2012). *"Observed Ground Crack Locations", Map Layer CGD0400 - 23 July 2012*. Retrieved from NZ Geotechnical Database: https://www.nzgd.org.nz/

EQC. (2012). *"Vertical Ground Surface Movements", Map Layer CGD0600 - 23 July 2012*. Retrieved from NZ Geotechnical Database: https://www.nzgd.org.nz/

EQC. (2013). *"Liquefaction and Lateral Spreading Observations", Map Layer CGD0300 - 22 September 2016*. Retrieved from NZ Geotechnical Database: https://www.nzgd.org.nz/

EQC. (2013). *"Liquefaction Interpreted from Aerial Photography", Map Layer CGD0200 - 11 Feb 2013*. Retrieved from NZ Geotechnical Database: https://www.nzgd.org.nz/

GNS Science. (2014). "Median Groundwater Surface Elevations". *Map Layer CGD5160 - 10 June 2014 retrieved from the New Zealand Geotechnical Database*.

MBIE. (2014, October). *Building Performance: Issue 7.*

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 NZVD2016 from, Christchurch and Ashley River 1m DEM (2018); both accessed at data.linz.govt.nz

GEOTECH
consulting ltd

256 FITZGERALD AVE

SITE INVESTIGATION PLAN

SCALE (A4):	1:500
DRAWN BY:	AJH
DATE DRAWN:	11/12/2020
CHECKED BY:	AJH
FIGURE NO.	5595/1

GEOTECH

BORE HOLE LOG

Hole No:	HA01
Job No:	5595
Logged by:	WH/RBW
Date drilled:	2/12/2020
Checked by:	RBW
Date checked:	7/12/2020
Max depth:	2.00

Project: 256 Fitzgerald Ave, Christchurch

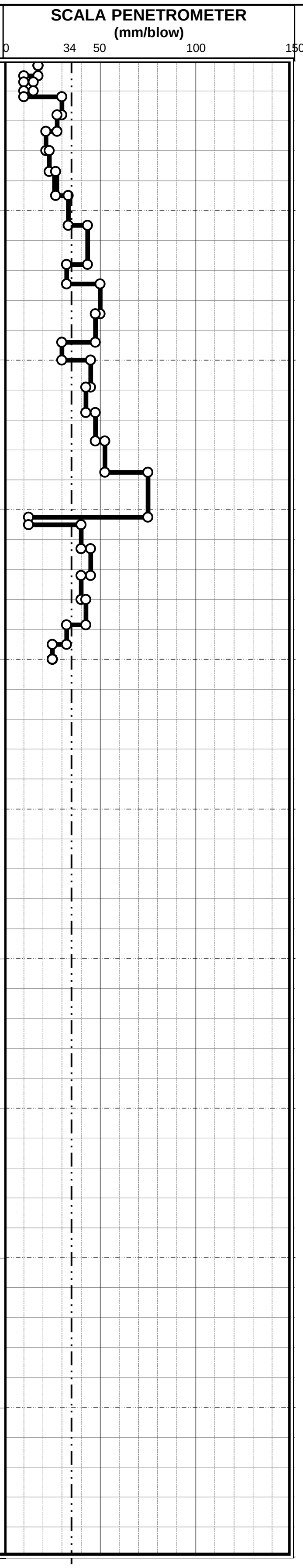
Client: R Harwood

Hole location: Refer to Site Plan.

Driller: WH **Contractor:** **Equipment:** SC+HA **R.L.:**

Notes:

Depth (m)	STRATA DESCRIPTION	USCS	Graphic Log	Water Table	Samples	S.P.T N uncorrected
0.0 – 0.5	SILT FILL: Grey-brown, dry, non-plastic, firm.	FILL				
0.5 – 0.7	Sandy SILT: Yellow brown, dry, non-plastic, firm.	SM				
0.7 – 1.5	SAND: Some silt, Yellow-Brown, fine, moist, loose to medium, homogeneous, varies minor to silty.	SW				
1.5 – 2.0	Sandy SILT: Yellow-Brown, mottled grey and orange, dry, non-plastic, firm.	SM				
2.0	E.O.H - target depth					



BORE HOLE LOG

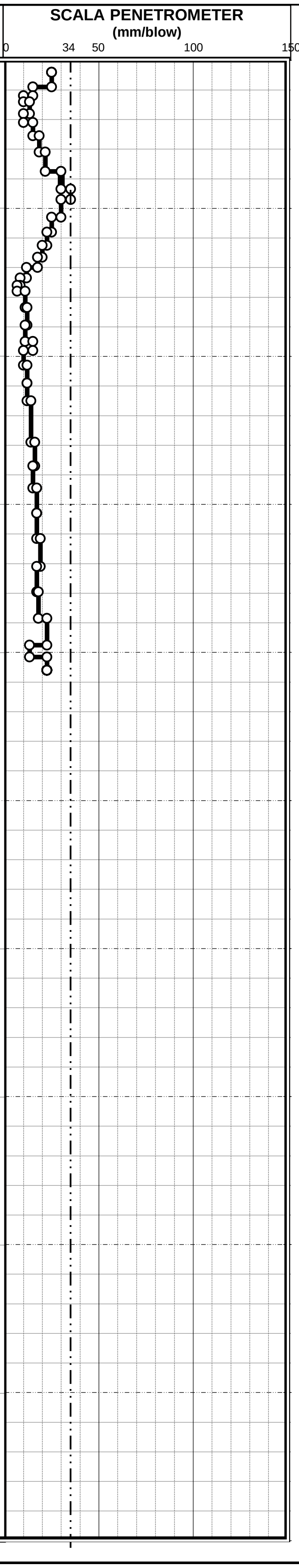
Project:	**256 Fitzgerald Ave, Christchurch**	**Hole No:**	**HA02**
Client:	**R Harwood**	**Job No:**	**5595**
Hole location:	Refer to Site Plan.	**Logged by:**	**WH/RBW**
		Date drilled:	**2/12/2020**
		Checked by:	**RBW**
		Date checked:	**7/12/2020**
Driller:	WH	**Max depth:**	2.00
Contractor:		**Equipment:**	SC+HA
R.L.:		**Notes:**	

Depth	STRATA DESCRIPTION	USCS
0.0	SILT FILL: Grey-brown, dry, non-plastic, firm.	FILL
0.5	TOPSOIL [buried]: Sandy SILT, Dark brown, dry, non-plastic, firm.	OL
0.8	SAND: Some silt, Yellow-Brown, fine, moist, loose to medium, homogeneous, varies minor to silty.	SW
2.0	E.O.H - target depth	

Water Table

Samples

S.P.T
N uncorrected



BORE HOLE LOG

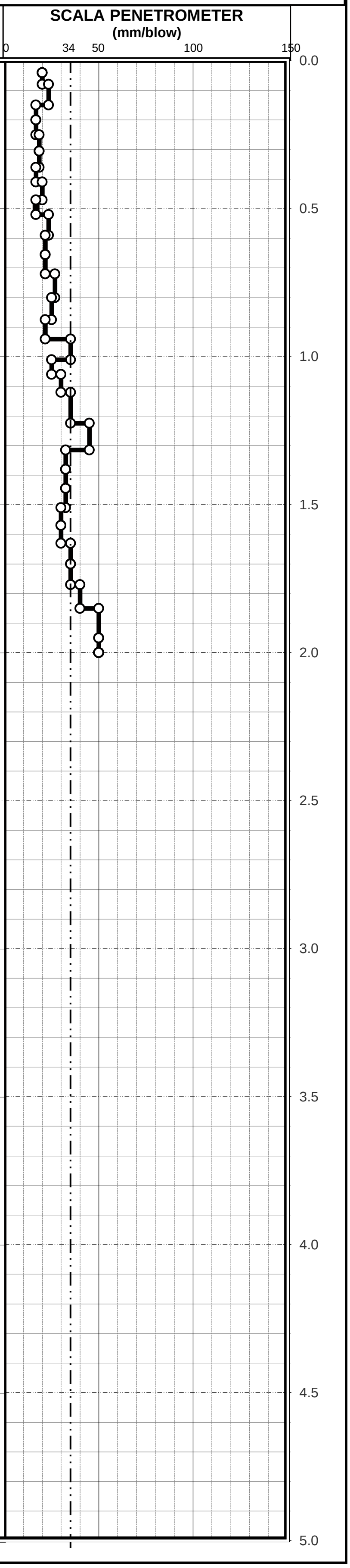
GEOTECH

Project:	**256 Fitzgerald Ave, Christchurch**	**Hole No:**	**HA03**
Client:	**R Harwood**	**Job No:**	**5595**
Hole location:	Refer to Site Plan.	**Logged by:**	**WH/RBW**
		Date drilled:	**2/12/2020**
		Checked by:	**RBW**
		Date checked:	**7/12/2020**
		Max depth:	2.00

Driller: WH
Contractor:
Equipment: SC+HA
R.L.:
Notes:

	STRATA DESCRIPTION	USCS	Graphic Log	Water Table	Samples	S.P.T N uncorrected
0.0	SILT FILL: Grey-brown, dry, non-plastic, firm.	FILL				
0.5	Sandy SILT: Yellow-Brown, dry, non-plastic, firm.	ML	x x x x			
	SAND: Some silt, Yellow-Brown, fine, moist, loose to medium dense, homogeneous, varies minor to silty.	SW				
2.0	E.O.H. - target depth					

Depth scale: 0.0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0



GEOTECH

BORE HOLE LOG

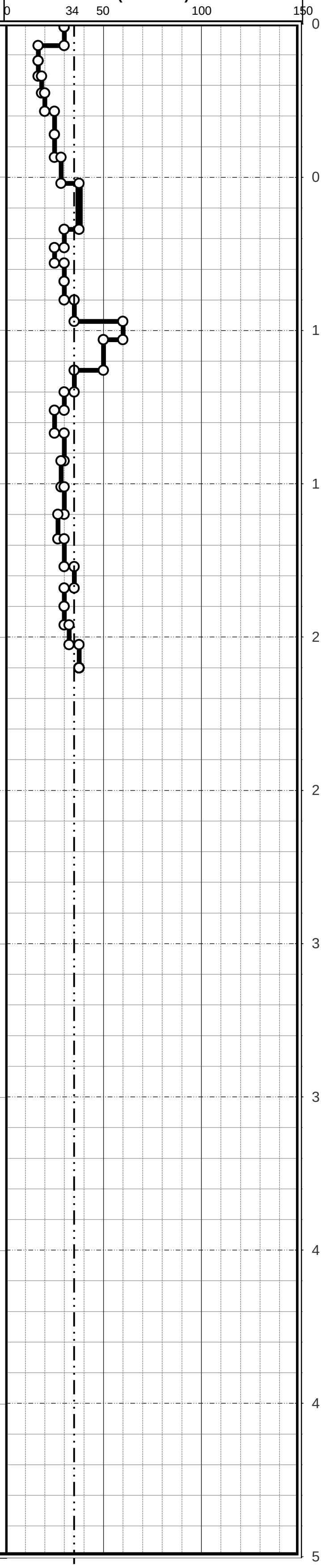
Hole No:	**HA04**	**Job No:**	**5595**
Project:	**256 Fitzgerald Ave, Christchurch**	**Logged by:**	**WH/RBW**
Client:	**R Harwood**	**Date drilled:**	**2/12/2020**
Hole location:	Refer to Site Plan.	**Checked by:**	**RBW**
Driller:	WH	**Date checked:**	**7/12/2020**
Contractor:		**Max depth:**	2.10
Equipment:	SC+HA	**R.L.:**	
Notes:			

STRATA DESCRIPTION	USCS	Graphic Log	Water Table	Samples	S.P.T N uncorrected
SILT FILL: Grey-brown, dry, non-plastic, firm.	FILL				
Sandy SILT: Yellow-Brown, dry, non-plastic, firm.	SM				
SAND: Some silt, Yellow-Brown, fine, moist, loose to medium dense, homogeneous, varies minor to silty.	SW				
Sandy SILT: Yellow-Brown, mottled grey and orange, dry, non-plastic, firm.	SM				
E.O.H. - target depth					

Depth scale: 0.0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

S.P.T N uncorrected: 0, 50, 100

SCALA PENETROMETER (mm/blow)



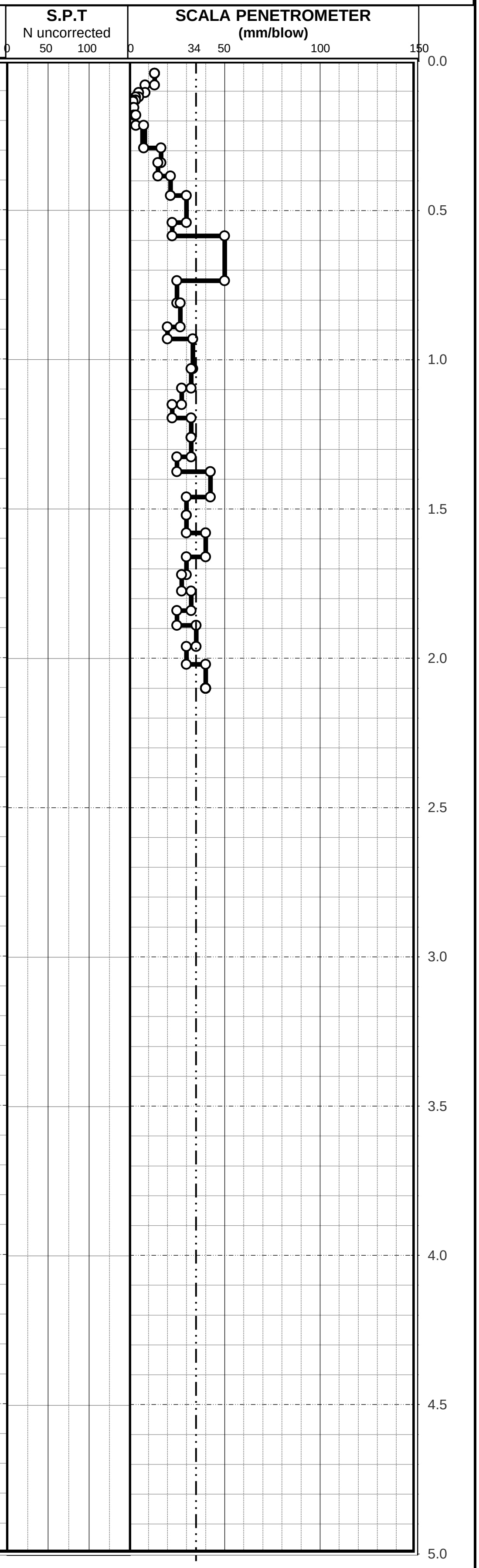
BORE HOLE LOG

GEOTECH

Hole No:	**HA05**
Job No:	**5595**
Logged by:	**WH/RBW**
Date drilled:	**2/12/2020**
Checked by:	**RBW**
Date checked:	**7/12/2020**
Max depth:	2.00
Project:	**256 Fitzgerald Ave, Christchurch**
Client:	**R Harwood**
Hole location:	Refer to Site Plan.
Driller:	WH
Contractor:	
Equipment:	SC+HA
R.L.:	
Notes:	

STRATA DESCRIPTION	USCS	Graphic Log	Water Table	Samples
SILT FILL: Grey-brown, dry, non-plastic, stiff, trace to some gravel.	FILL	▨		
SAND: Some silt, Yellow-Brown, fine, moist, loose to medium, homogeneous, varies minor to silty.	SW	⠿		
E.O.H. - target depth				

S.P.T
N uncorrected
0 50 100

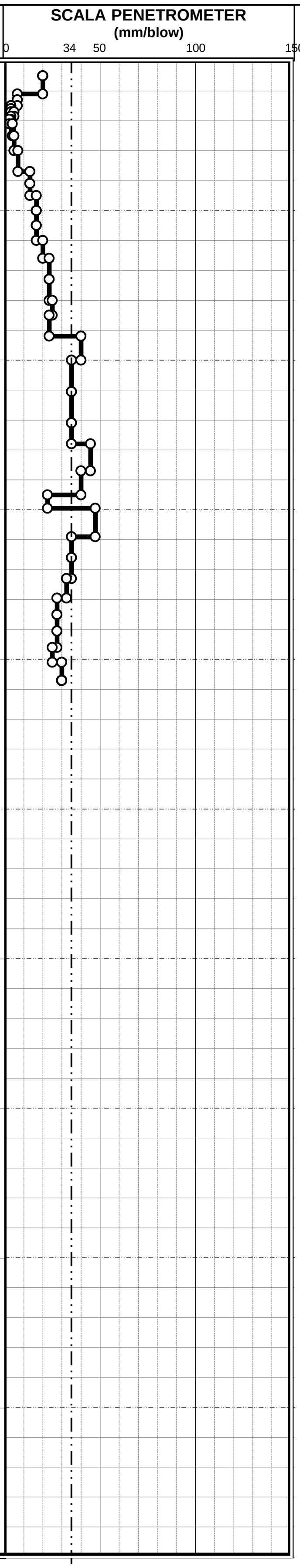


BORE HOLE LOG

GEOTECH

Hole No:	HA06		
Job No:	5595		
Logged by:	WH/RBW		
Project:	256 Fitzgerald Ave, Christchurch	**Date drilled:**	2/12/2020
Client:	R Harwood	**Checked by:**	RBW
Hole location:	Refer to Site Plan.	**Date checked:**	7/12/2020
Driller:	WH	**Max depth:**	2.00
Contractor:		**Equipment:**	SC+HA
R.L.:		**Notes:**	

Depth (m)	STRATA DESCRIPTION	USCS	Graphic Log	Water Table	Samples	S.P.T N uncorrected
0.0	SILT FILL: Grey-brown, dry, non-plastic, stiff, trace to some gravel.	FILL	▨			
0.7	SAND: Some silt, Yellow-Brown, fine, moist, loose to medium, homogeneous, varies minor to silty.	SW	⁙			
2.0	No Further Progress due to reaching target depth					



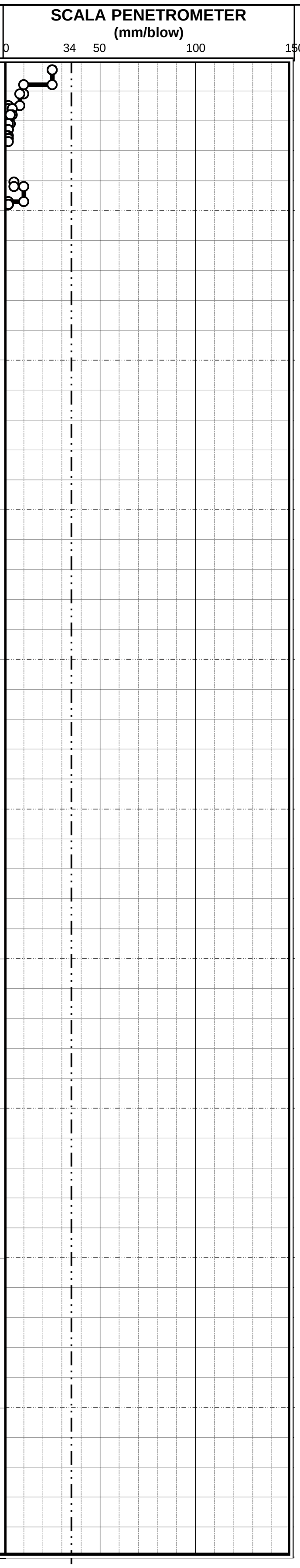
Depth scale: 0.0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

BORE HOLE LOG

Hole No:	**HA07**		
Job No:	**5595**		
Logged by:	**WH/RBW**		
Project:	**256 Fitzgerald Ave, Christchurch**	**Date drilled:**	**2/12/2020**
Client:	**R Harwood**	**Checked by:**	**RBW**
Hole location:	Refer to Site Plan.	**Date checked:**	**7/12/2020**
Driller:	WH	**Max depth:**	0.50
Contractor:		**Equipment:**	SC+HA
R.L.:			

Notes: Hand cleared to 400mm then resumed scala penetrometer. Refusal on large concrete at 0.5m

Depth	STRATA DESCRIPTION	USCS	Graphic Log	Water Table	Samples	S.P.T N uncorrected
0.0	SILT FILL: Grey-brown, dry, non-plastic, stiff, some gravel.	FILL	▨			
0.5	No Further Progress due to refusal on large concrete.					



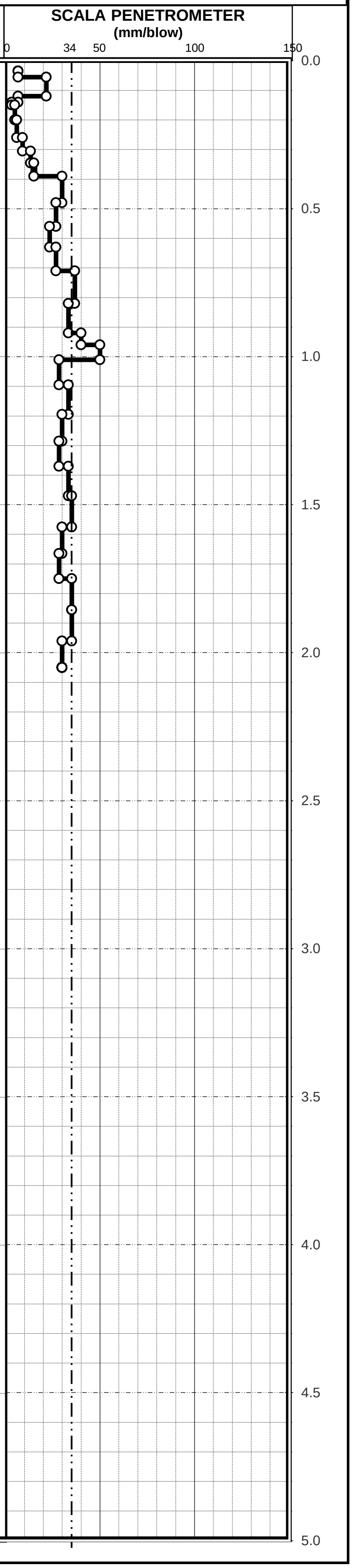
BORE HOLE LOG

GEOTECH

Hole No: HA08
Job No: 5595
Logged by: WH/RBW
Date drilled: 2/12/2020
Checked by: RBW
Date checked: 7/12/2020
Max depth: 2.00

Project: 256 Fitzgerald Ave, Christchurch
Client: R Harwood
Hole location: Refer to Site Plan.
Driller: WH
Contractor:
Equipment: SC+HA
R.L.:
Notes:

Depth (m)	STRATA DESCRIPTION	USCS	Water Table	Samples	S.P.T N uncorrected
0.0	SILT FILL: Grey-brown, dry, non-plastic, stiff, trace gravel, transitions to silty, sandy GRAVEL with depth.	FILL			
0.4	SAND: Some silt, grey, fine, moist, loose to medium dense, homogeneous, varies minor to silty.	SW			
1.1	SAND: Trace silt, yellow-brown, fine to medium, moist, loose to medium dense, fine to medium. - 1.3m, less silt and lighter grey with depth - 1.9m, wetter (wet) with depth	SW			
2.0	E.O.H. - target depth				



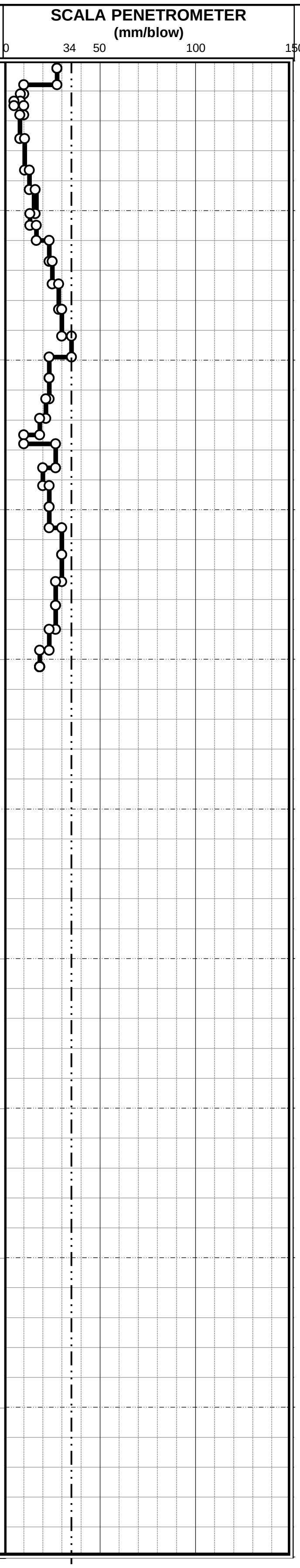
BORE HOLE LOG

Hole No:	**HA09**
Job No:	**5595**
Logged by:	**WH/RBW**
Date drilled:	**2/12/2020**
Checked by:	**RBW**
Date checked:	**7/12/2020**
Max depth:	2.00
Project:	**256 Fitzgerald Ave, Christchurch**
Client:	**R Harwood**
Hole location:	Refer to Site Plan.
Driller:	WH
Contractor:	
Equipment:	SC+HA
R.L.:	
Notes:	

STRATA DESCRIPTION	USCS	Graphic Log	Water Table	Samples	S.P.T N uncorrected
SILT FILL: Grey-brown, dry, non-p, stiff, trace gravel, transitions to silty, sandy GRAVEL with depth.	FILL				
SAND: Some silt, Yellow-Brown, fine, moist, loose to medium, homogeneous, varies minor to silty.	SW				
SAND: Trace silt, grey, fine to medium, moist, loose to medium dense, fine to medium. Less silt and becomes more yellow-brown with depth	SW				
E.O.H. - target depth					

Depth (m): 0.0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

S.P.T N uncorrected: 0, 50, 100



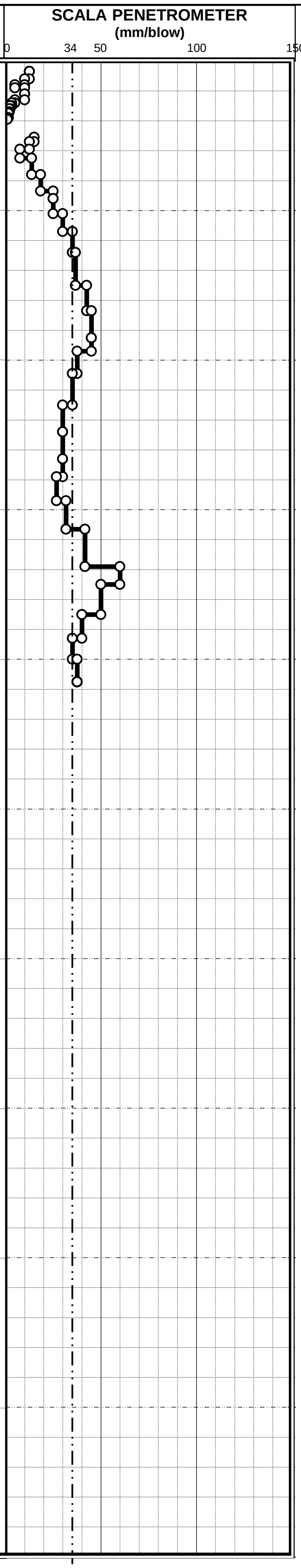
GEOTECH

BORE HOLE LOG

Hole No:	HA10
Job No:	5595
Logged by:	WH/RBW
Date drilled:	2/12/2020
Checked by:	RBW
Date checked:	7/12/2020
Max depth:	2.00

Project: 256 Fitzgerald Ave, Christchurch
Client: R Harwood
Hole location: Refer to Site Plan.
Driller: WH
Contractor:
Equipment: SC+HA
R.L.:
Notes: Location hand cleared to 0.25m, then scala penetrometer recommenced after refusal in surface fill.

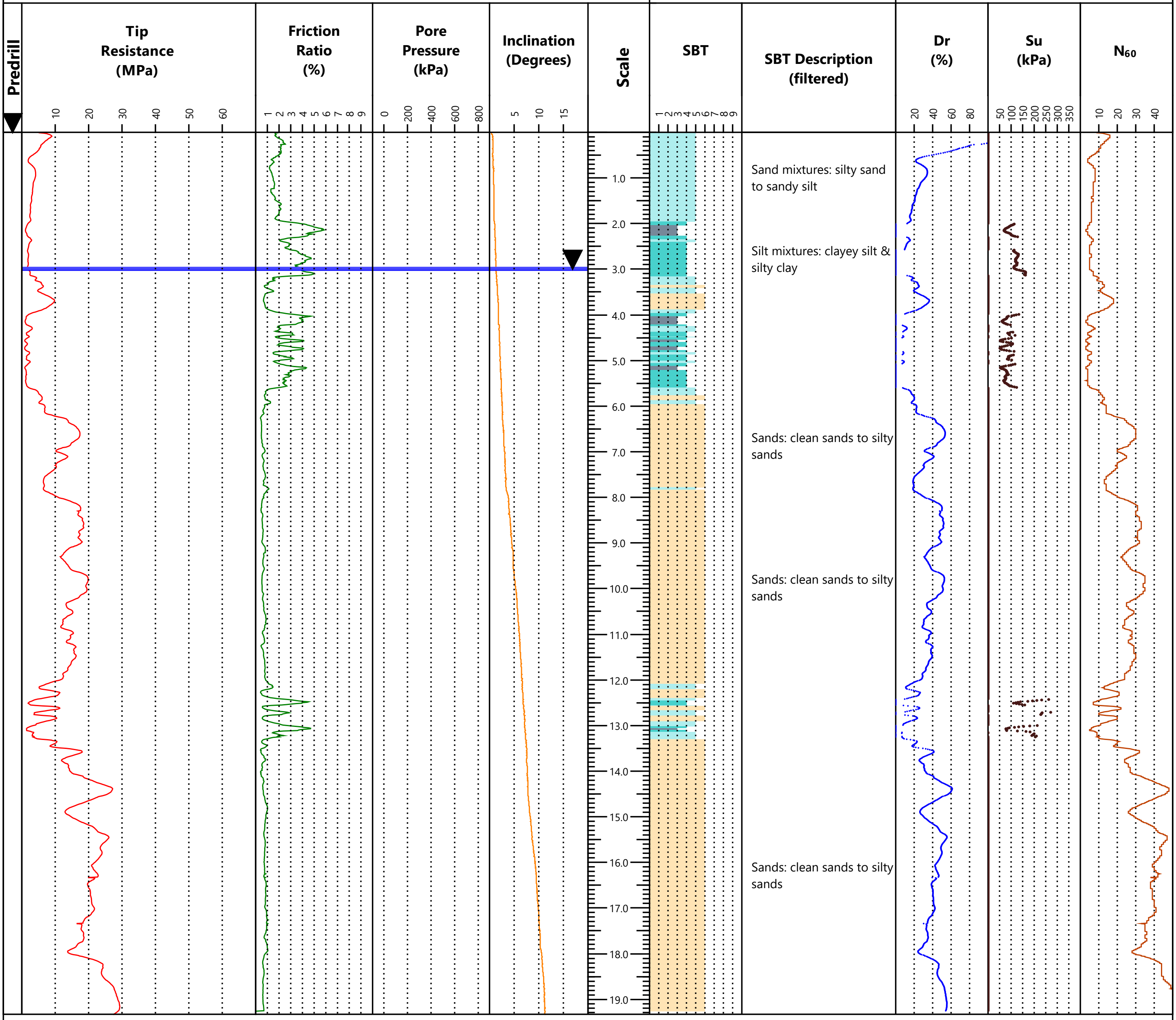
Depth	STRATA DESCRIPTION	USCS	Graphic Log	Water Table	Samples	S.P.T N uncorrected
0.0	SILT FILL: Grey-brown, dry, non-p, stiff, trace gravel, transitions to silty, sandy GRAVEL with depth.	FILL				
	TOPSOIL [buried]: Dark brown, moist, soft to firm.	OL				
	SAND: Some silt, Yellow-Brown, fine, moist, loose to medium, homogeneous, varies minor to silty.	SW				
	SAND: Trace silt, yellow-brown, fine to medium, moist, loose to medium dense, fine to medium. Less silt and lighter grey with depth	SW				
2.0	E.O.H. - target depth					



McMILLAN Drilling	**Client:** Geotech Consulting Ltd	**Bore No.:** CPT001
	Project: 256 Fitzgerald Avenue, Christchurch	**Job No.:** 19425

Site Location: 256 Fitzgerald Avenue, Christchurch
Date: 2/12/2020
Grid Reference: 1571833.44m E, 5180877.93m N (NZTM) - Map or aerial photograph
Rig Operator: R. Wyllie
Elevation: 0.00m
Datum: Ground
Equipment: 14t truck mounted rig

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



EOH: 19.33m

Cone Type: I-CFXY-10 - Compression
Cone Reference: 110542
Cone Area Ratio: -
Standards: ISO 22476-1:2012

Predrill: -
Water Level: 3m
Collapse: 3.2m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	-0.1482	-0.1442
Local Friction	0.0291	0.0291
Pore Pressure	-	-

Termination

Target Depth: ☐

Effective Refusal

Tip: ☐
Gauge: ☑
Inclinometer: ☐

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. No warranty is provided as to 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.

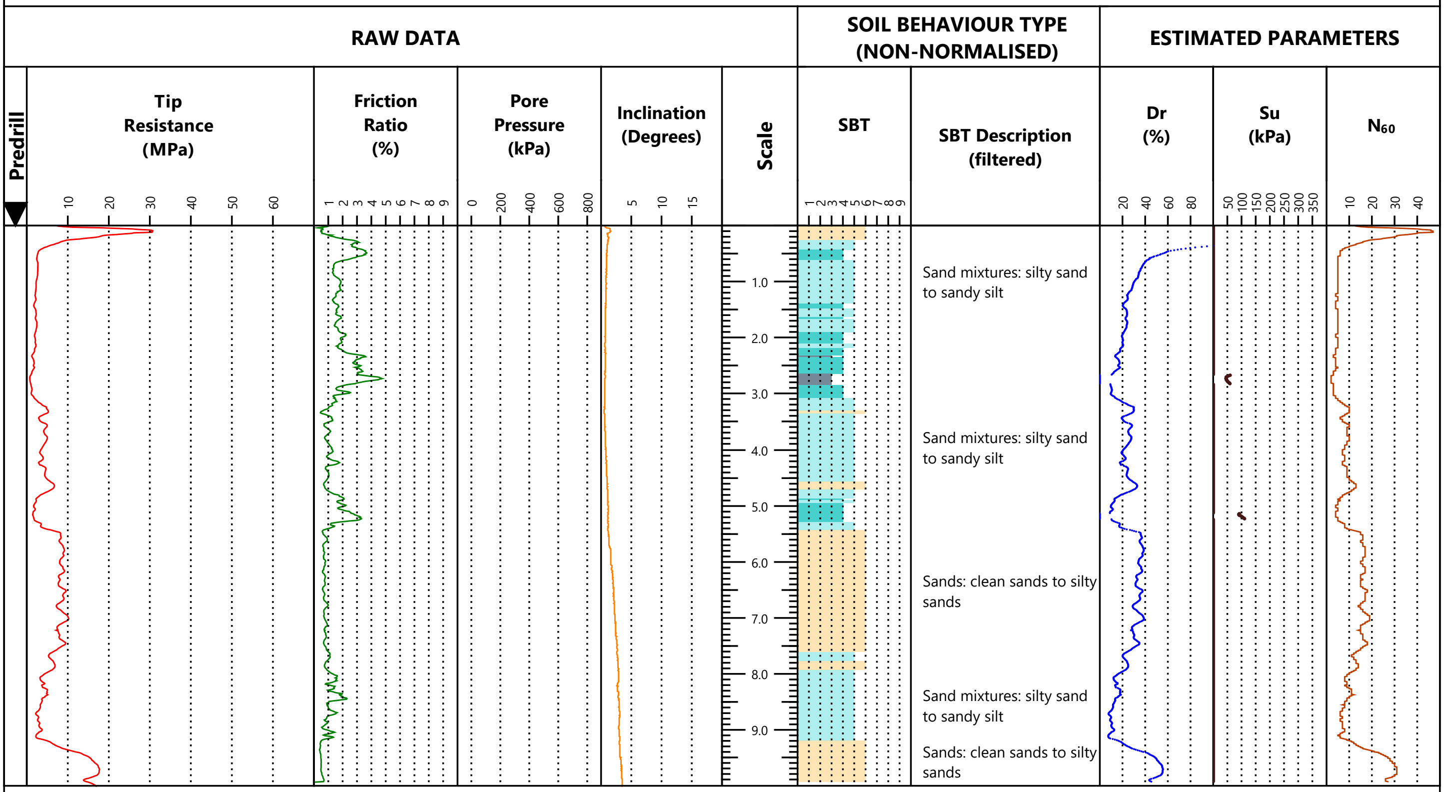
Remarks

Generated with Core-GS by Geroc

McMILLAN Drilling

Client:	Geotech Consulting Ltd	**Bore No.:**	**CPT002**
Project:	256 Fitzgerald Avenue, Christchurch	**Job No.:**	**19425**

Site Location: 256 Fitzgerald Avenue, Christchurch
Date: 2/12/2020
Grid Reference: 1571851.88m E, 5180876.9m N (NZTM) - Map or aerial photograph
Rig Operator: R. Wyllie
Elevation: 0.00m
Datum: Ground
Equipment: 14t truck mounted rig



EOH: 10m

Cone Type: I-CFXY-10 - Compression
Cone Reference: 110542
Cone Area Ratio: -
Standards: ISO 22476-1:2012

Predrill: -
Water Level: -
Collapse: 2.7m

Termination
Target Depth: ☑
Effective Refusal
Tip: ☐
Gauge: ☐
Inclinometer: ☐

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	-0.1747	-0.1427
Local Friction	0.0293	0.0288
Pore Pressure	-	-

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. No warranty is provided as to 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

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McMILLAN Drilling

Client: Geotech Consulting Ltd

Bore No.: CPT003

Project: 256 Fitzgerald Avenue, Christchurch

Job No.: 19425

Site Location: 256 Fitzgerald Avenue, Christchurch

Date: 2/12/2020

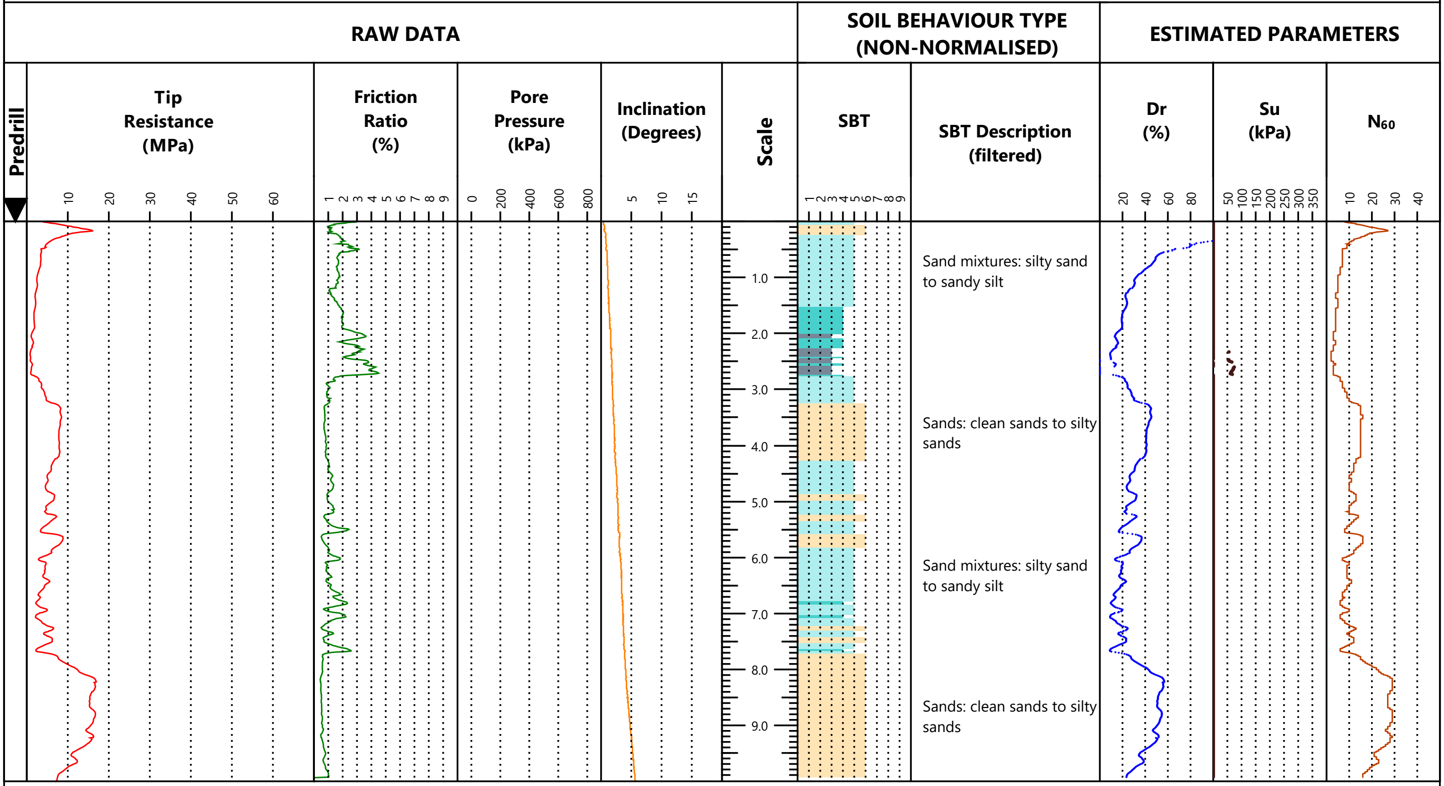
Grid Reference: 1571873.18m E, 5180876.89m N (NZTM) - Map or aerial photograph

Rig Operator: R. Wyllie

Elevation: 0.00m

Datum: Ground

Equipment: 14t truck mounted rig



EOH: 10m

Cone Type: I-CFXY-10 - Compression

Cone Reference: 110542

Cone Area Ratio: -

Standards: ISO 22476-1:2012

Predrill: -

Water Level: -

Collapse: 2.2m

Termination

Target Depth: ☑

Effective Refusal

Tip: ☐

Gauge: ☐

Inclinometer: ☐

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	-0.1551	-0.1357
Local Friction	0.0292	0.029
Pore Pressure	-	-

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. No warranty is provided as to 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

Sheet 1 of 1

McMILLAN Drilling

Client: Geotech Consulting Ltd

Bore No.: CPT004

Project: 256 Fitzgerald Avenue, Christchurch

Job No.: 19425

Site Location: 256 Fitzgerald Avenue, Christchurch

Date: 2/12/2020

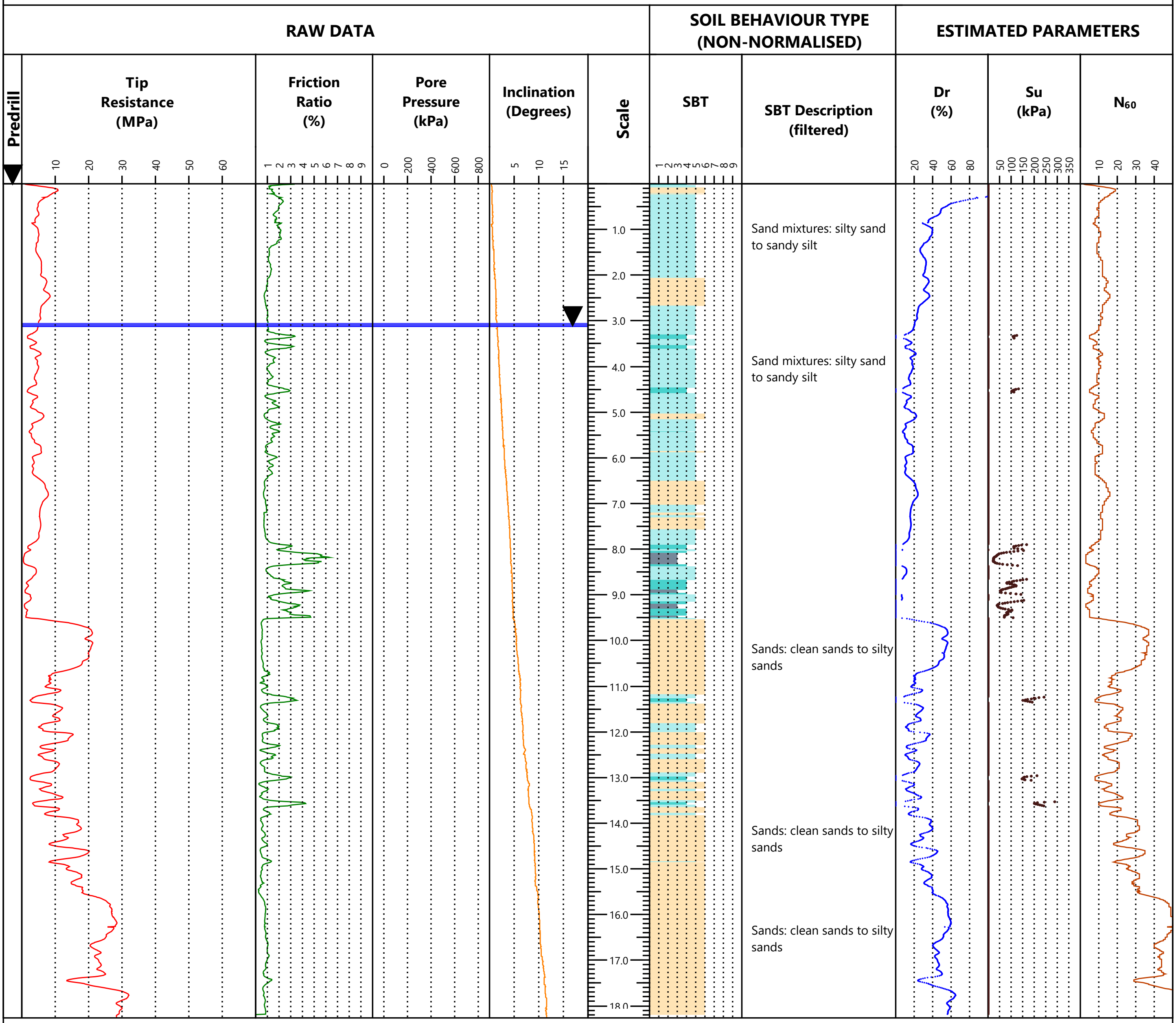
Grid Reference: 1571873.4m E, 5180900.92m N (NZTM) - Map or aerial photograph

Rig Operator: R. Wyllie

Elevation: 0.00m

Datum: Ground

Equipment: 14t truck mounted rig



EOH: 18.26m

Cone Type: I-CFXY-10 - Compression

Cone Reference: 110542

Cone Area Ratio: -

Standards: ISO 22476-1:2012

Predrill: -

Water Level: 3.1m

Collapse: 3.3m

Termination

Target Depth: ☐

Effective Refusal

Tip: ☐

Gauge: ☑

Inclinometer: ☐

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	-0.1968	-0.1469
Local Friction	0.0306	0.0287
Pore Pressure	-	-

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

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

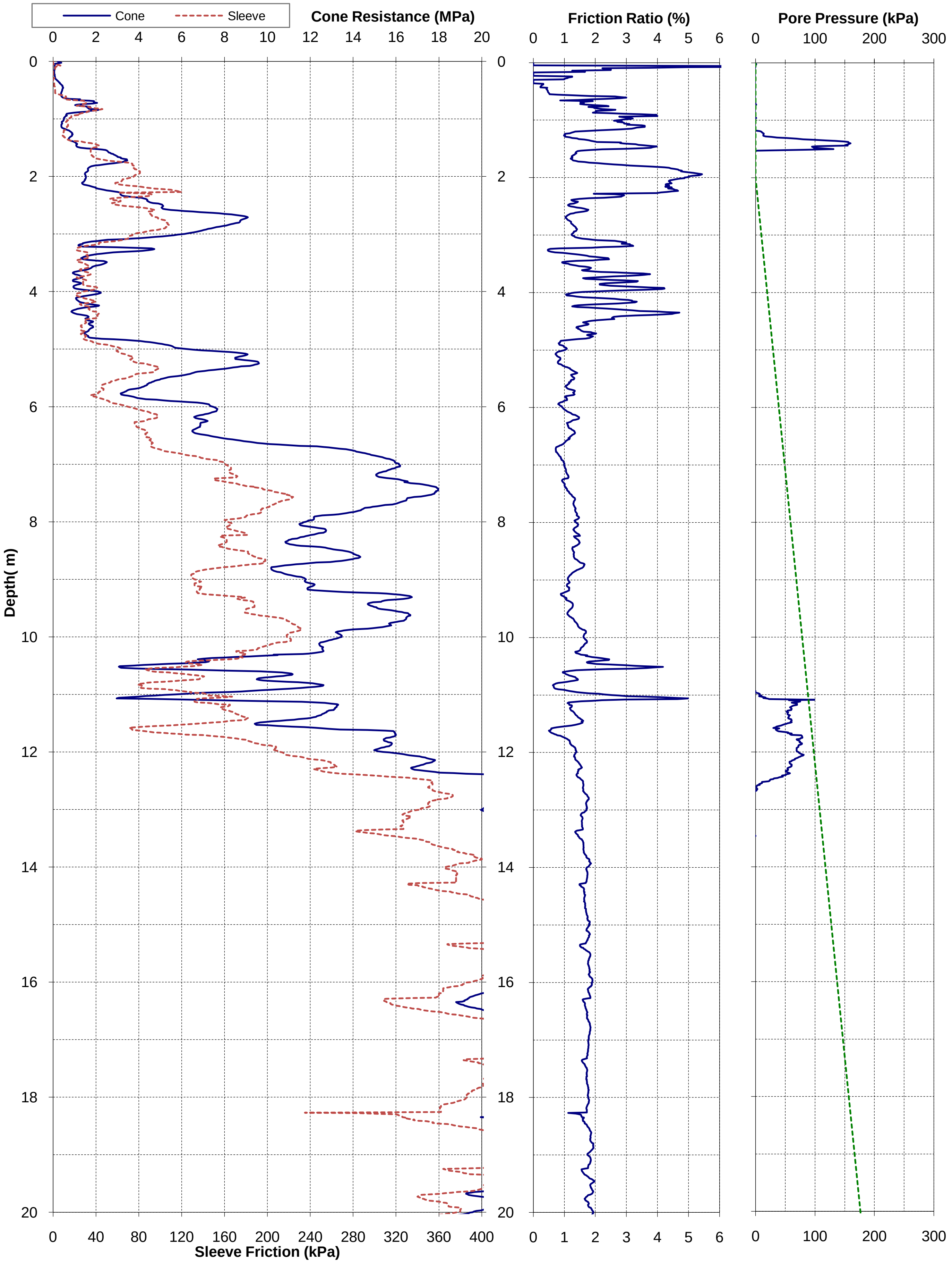
Notes & Limitations

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. No warranty is provided as to 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

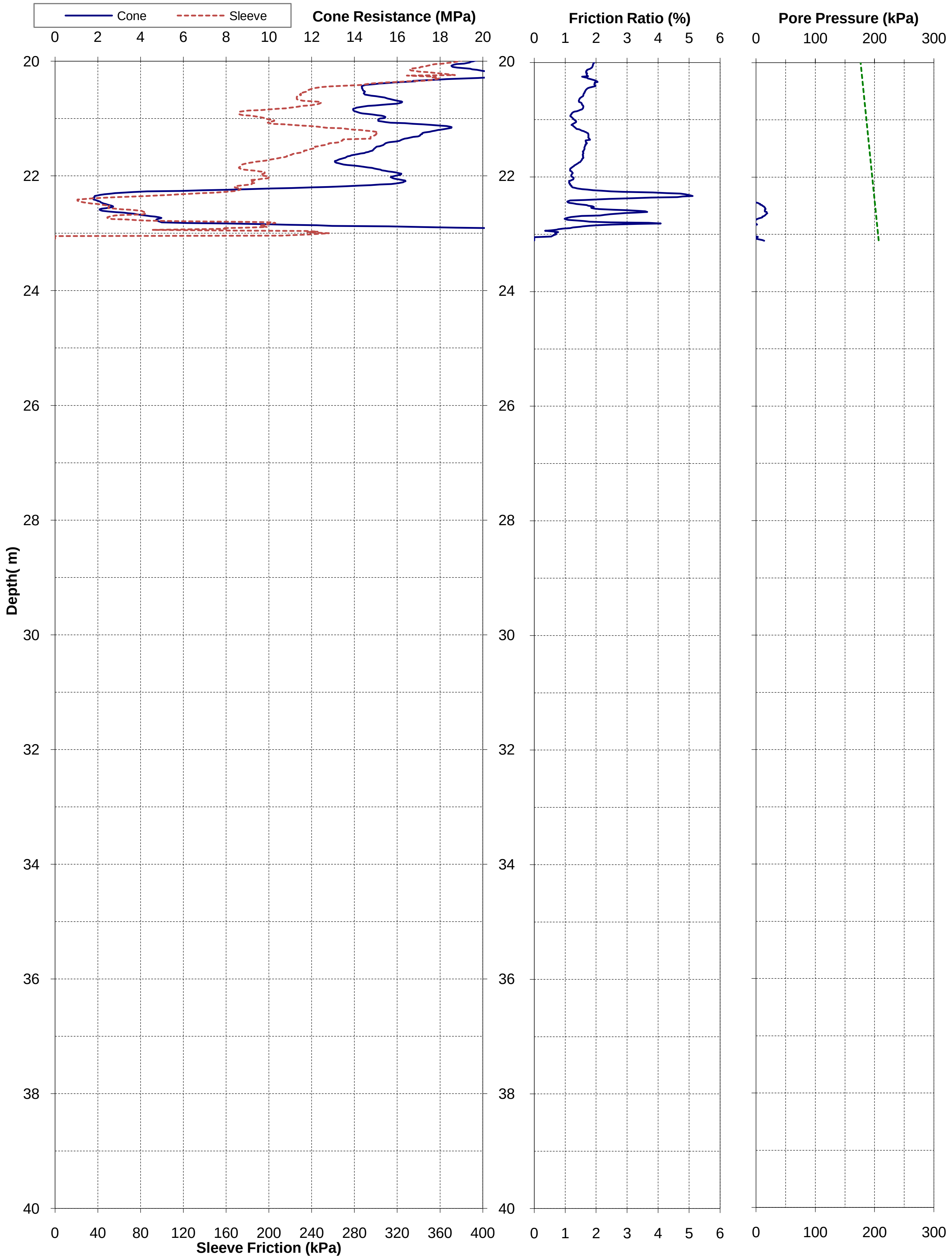
Sheet 1 of 1

Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			**Page:** 1 of 2	**CPT-RCH-54**
Test Date:	26-May-2011	**Location:**	Richmond	**Operator:**	Perry
Pre-Drill:	1.2m	**Assumed GWL:**	2mBGL	**Located By:**	Survey GPS
Position:	2481814mE	5742497.6mN	4.16mRL	**Coord. System:**	NZMG & MSL
Other Tests:				**Comments:**	



T+T Ref: 51731.001 Printed: 21/09/2011 12:17 p.m. Template: CPT Graph Template v0.41.xls

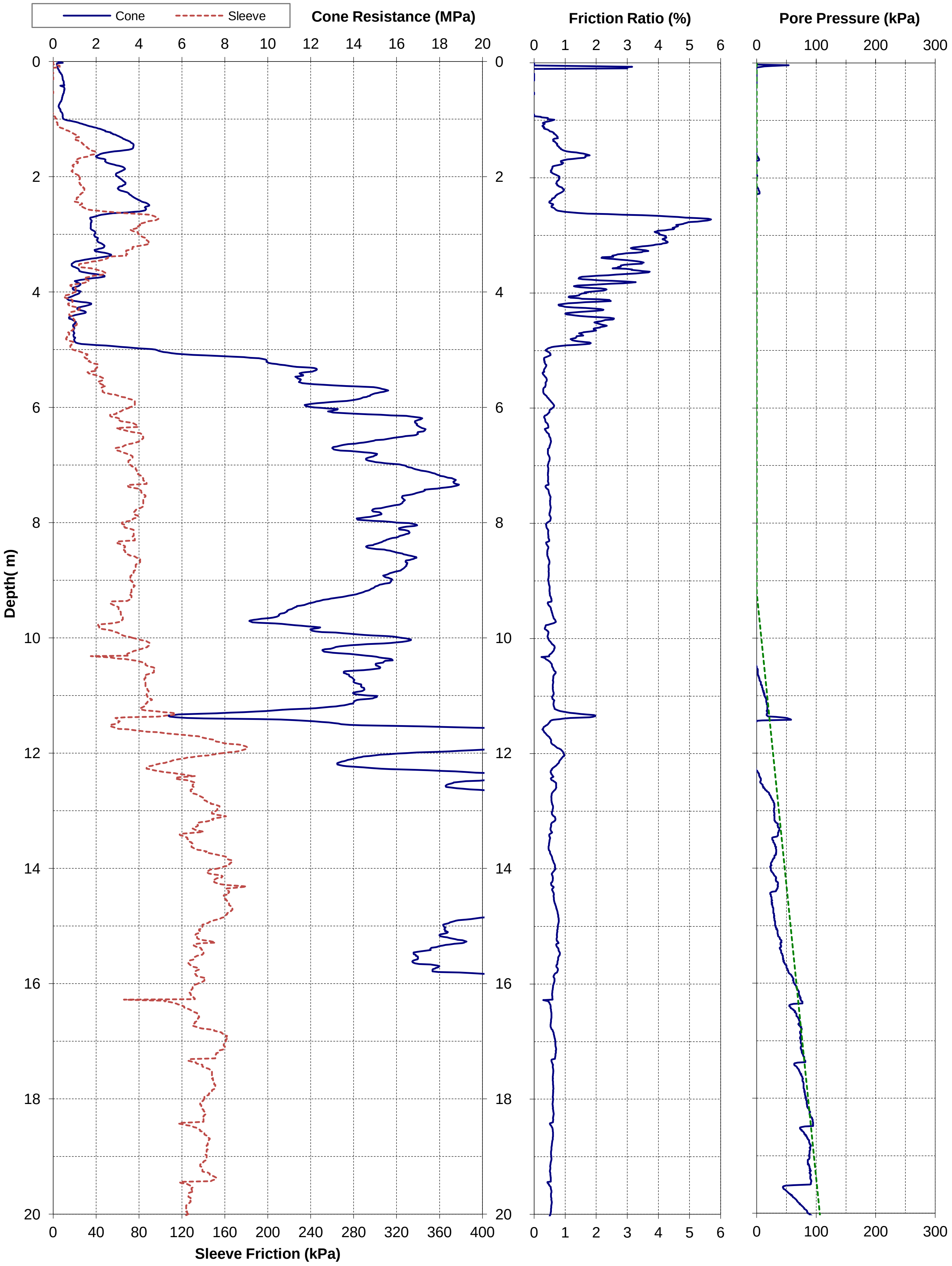
Project:	Christchurch 2011 Earthquake - EQC Ground Investigations			**Page:**	2 of 2	**CPT-RCH-54**
Test Date:	26-May-2011	**Location:**	Richmond	**Operator:**	Perry	
Pre-Drill:	1.2m	**Assumed GWL:**	2mBGL	**Located By:**	Survey GPS	
Position:	2481814mE	5742497.6mN	4.16mRL	**Coord. System:**	NZMG & MSL	
Other Tests:				**Comments:**		



T+T Ref: 51731.001 Printed: 21/09/2011 12:17 p.m. Template: CPT Graph Template v0.41.xls

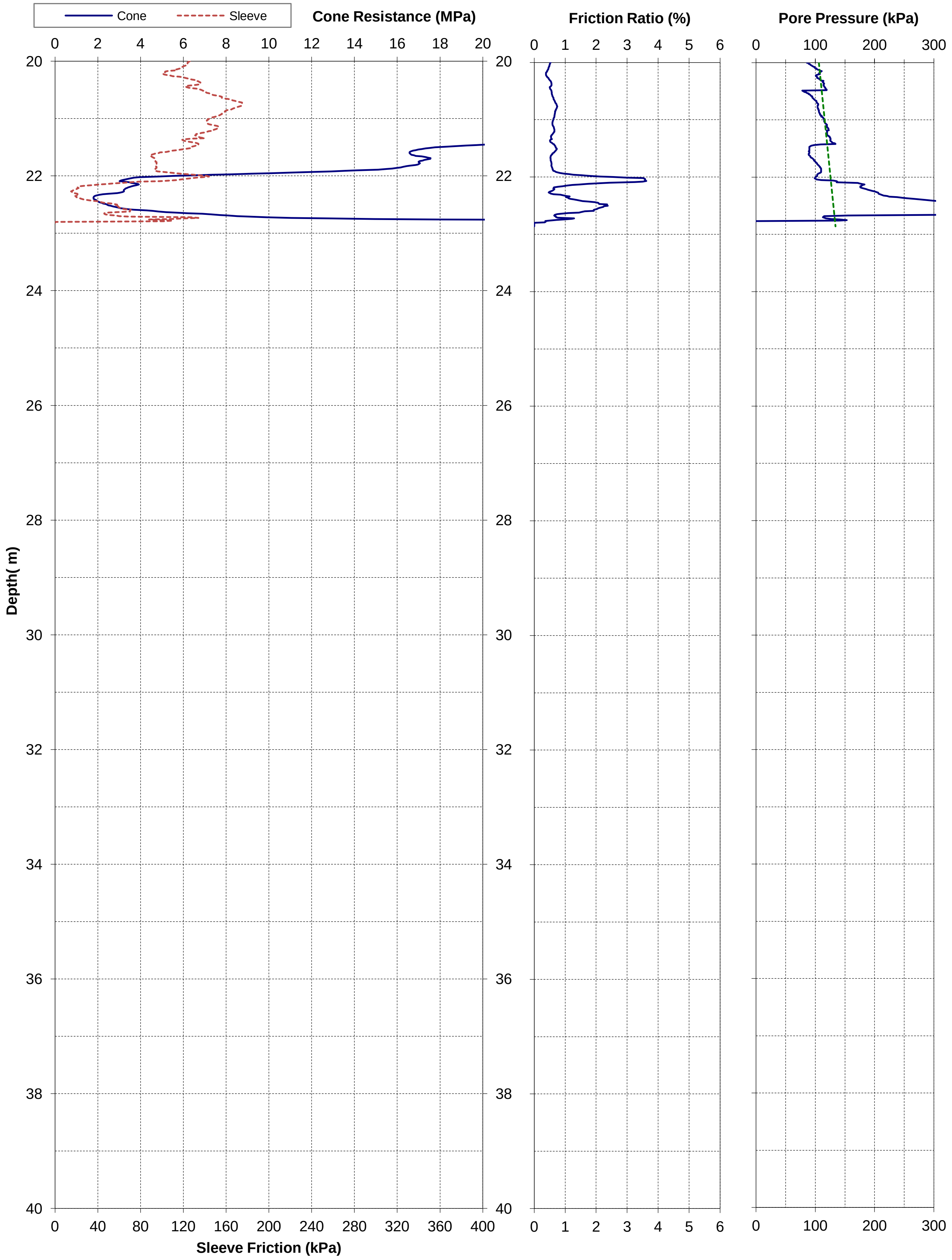
Project:	Christchurch 2011 Earthquake - CCC Ground Investigations			**Page:**	1 of 2
Test Date:	30-May-2011	**Location:**	Central City	**Operator:**	Perry
Pre-Drill:	1.5m	**Assumed GWL:**	9.2mBGL	**Located By:**	Survey GPS
Position:	2481874.8mE	5742472.9mN	4.189mRL	**Coord. System:**	NZMG & MSL
Other Tests:				**Comments:**	

CPT-CBD-27



T+T Ref: 52000.3400 Printed: 4/11/2011 11:49 a.m. Template: CPT Graph Template v0.41.xls

Project:	Christchurch 2011 Earthquake - CCC Ground Investigations		**Page:**	2 of 2	**CPT-CBD-27**
Test Date:	30-May-2011	**Location:**	Central City	**Operator:**	Perry
Pre-Drill:	1.5m	**Assumed GWL:**	9.2mBGL	**Located By:**	Survey GPS
Position:	2481874.8mE	5742472.9mN	4.189mRL	**Coord. System:**	NZMG & MSL
Other Tests:				**Comments:**	



McMILLAN Drilling GROUP

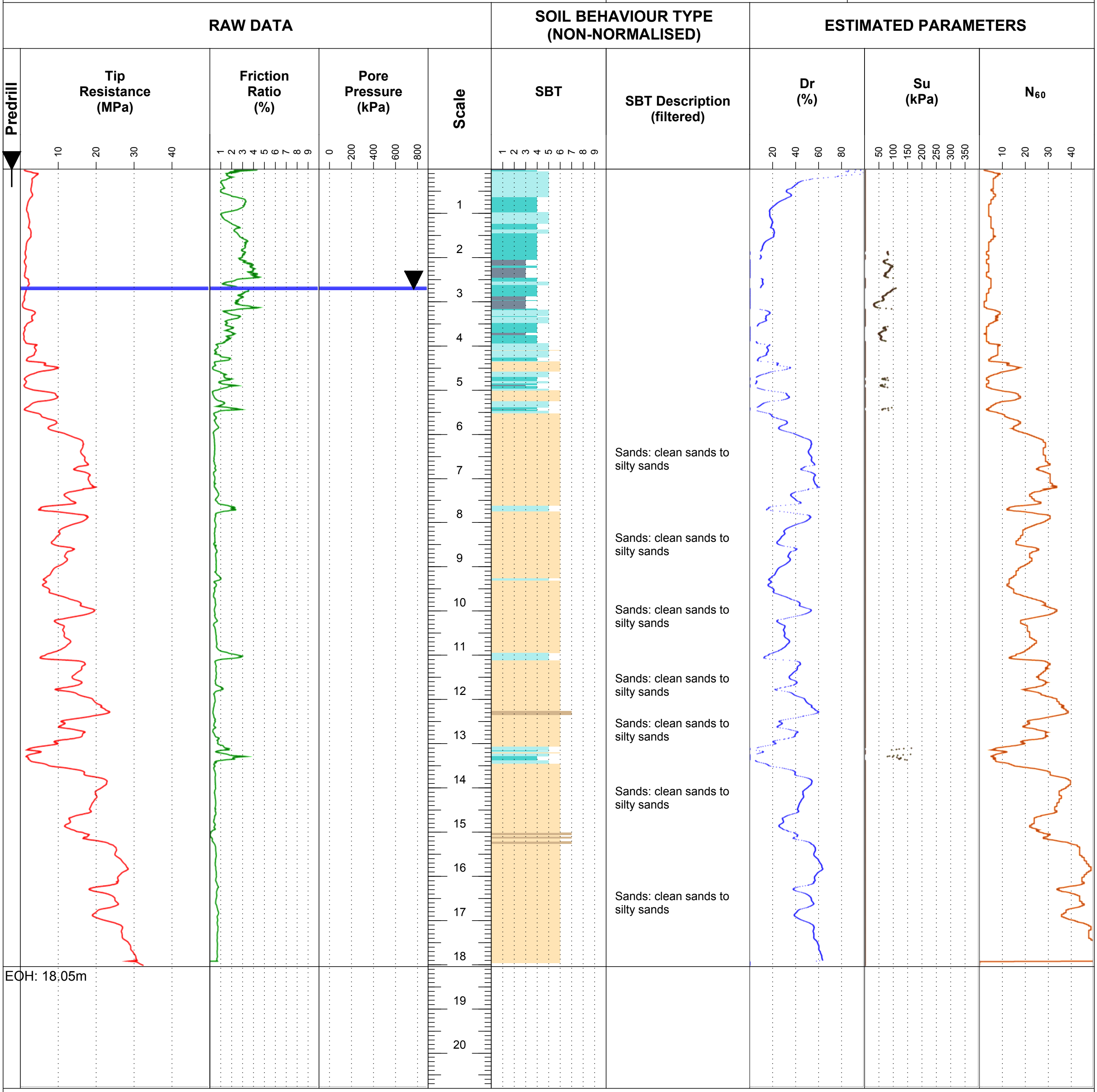
CONE PENETRATION TEST

Job: 13302
CPT No.: CPT002

Name: 20 Heywood Terrace, Christchurch
Client: Geotech Consulting Ltd
Location: 20 Heywood Terrace, Christchurch

Grid: NZTM
Datum:
Termination: -

North (m): 5180935.17
East (m): 1571868.35
Elevation (m): -
Hole Depth (m): 18.05



Operator: S. Cardona
Cone Reference: 110542T
Cone Area Ratio: 0.75
Cone Type: -

Date: 03/03/2014
Predrill: 0.00
Water Level: 2.70
Collapse:

Effective Refusal
Tip:
Gauge: ✓
Inclinometer:
Other:

Tip Resistance (MPa) Initial: -5.0007 **Final:** -5.0874
Local Friction (MPa) Initial: 0.0192 **Final:** 0.02
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

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Remarks
Effective Refusal

Hole Depth (m): 18.05

Sheet 1 of 1

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TONKIN & TAYLOR LTD

BOREHOLE LOG

BOREHOLE No: CBD 07
Hole Location: Harvey Tce

SHEET 1 OF 6

PROJECT: CHRISTCHURCH CITY 2011 REMEDIATION
LOCATION: CENTRAL CITY
JOB No: 52000.3400

CO-ORDINATES 5742473 mN
2481875.25 mE
R.L. 4.21 m
DATUM NZMG

DRILL TYPE: Rotary
DRILL METHOD: OB/Triple Tube
DRILL FLUID: Mud

HOLE STARTED: 11/7/11
HOLE FINISHED: 12/7/11
DRILLED BY: Pro-Drill
LOGGED BY: RKH/CP
CHECKED: BMcD

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	CORE RECOVERY (%)	METHOD	TESTS	SAMPLES	DEPTH (m)	CLASSIFICATION SYMBOL	MOISTURE CONDITION	STRENGTH/DENSITY CLASSIFICATION	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.
HAND DIG FILL. (Potholed for services check and backfilled.)	0	PRE-DUG			0.0–1.5				Fill: Borehole drilled through pre-dug and backfilled pothole.
YALDHURST MEMBER OF THE SPRINGSTON FORMATION (ALLUVIAL)		SPT	1/1/0/1/0/1 N=2		1.5–1.95	ML	S	S	Sandy SILT, brown. Soft, wet, non plastic. Sand is fine. 1.6m to 1.95m no recovery
	100	OB	✱FC	B	1.95–3.0				
					2.8	SP	W	L	Silty, fine SAND, grey. Loose, wet.
		SPT	2/1/2/2/2/2 N=8		3.0–3.45				
	62	OB	✱FC	B	3.45–4.5				3.45m to 3.85m no recovery
					4.5	SP	M	L	Fine SAND with some silt, grey. Loose, moist.
		SPT	0/0/2/1/2/2 N=7		4.5–4.95				

R.L. (m): 4.0, 3.5, 3.0, 2.5, 2.0, 1.5, 1.0, 0.5, 0.0, -0.5

DEPTH (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5

SHEAR STRENGTH (kPa): 10 25 50 100 200
COMPRESSIVE STRENGTH (MPa): 1 5 20 50 100 250
DEFECT SPACING (mm): 50 250 1000 2000

T+T DATATEMPLATE.GDT eek

TONKIN & TAYLOR LTD

BOREHOLE LOG

BOREHOLE No: CBD 07
Hole Location: Harvey Tce

SHEET 2 OF 6

PROJECT: CHRISTCHURCH CITY 2011 REMEDIATION
LOCATION: CENTRAL CITY
JOB No: 52000.3400

CO-ORDINATES 5742473 mN
2481875.25 mE
R.L. 4.21 m
DATUM NZMG

DRILL TYPE: Rotary
DRILL METHOD: OB/Triple Tube
DRILL FLUID: Mud

HOLE STARTED: 11/7/11
HOLE FINISHED: 12/7/11
DRILLED BY: Pro-Drill
LOGGED BY: RKH/CP
CHECKED: BMcD

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa) 10 25 50 100 200	COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100 250	DEFECT SPACING (mm) 50 250 1000 2000	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 THE SPRINGSTON FORMATION (ALLUVIAL)			100	OB		✱FC	B	-1.0 -1.5	5.5 6.0	SP	M	L				Fine SAND with some silt, grey. Loose, moist.
				SPT		1/1/2/2/5/6 N=15		-2.0				MD				- becoming medium dense
			62	OB		✱FC	B	-2.5 -3.0	6.5 7.0							6.45m to 6.85m no recovery
				SPT		1/1/2/2/2/4 N=10		-3.5	7.5							
			100	OB				-4.0 -4.5	8.0 8.5							- SAND becoming fine to medium - 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.
								-5.5	9.5 10							9.45m to 9.9m no recovery

T+T DATATEMPLATE.GDT eek

NZGD ID: BH_1740

TONKIN & TAYLOR LTD

BOREHOLE LOG

BOREHOLE No: CBD 07
Hole Location: Harvey Tce

SHEET 3 OF 6

PROJECT: CHRISTCHURCH CITY 2011 REMEDIATION		LOCATION: CENTRAL CITY	JOB No: 52000.3400
CO-ORDINATES	5742473 mN 2481875.25 mE	DRILL TYPE: Rotary	HOLE STARTED: 11/7/11
		DRILL METHOD: OB/Triple Tube	HOLE FINISHED: 12/7/11
R.L.	4.21 m		DRILLED BY: Pro-Drill
DATUM	NZMG	DRILL FLUID: Mud	LOGGED BY: RKH/CP CHECKED: BMcD

GEOLOGICAL: GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.

CHRISTCHURCH FORMATION (MARINE & ESTUARINE)

CORE RECOVERY (%)	METHOD	TESTS	DEPTH (m)
55	HQTT		10.1–11.0
	SPT	1/1/2/2/3/4 N=11	11.0–11.45
76	HQTT		11.45–12.5
	SPT	3/1/3/5/9/7 N=24	12.5–12.95
71	HQTT		12.95–14.0
	SPT	3/4/4/4/5/9 N=22	14.0–14.45
71	HQTT		14.45–15

R.L. (m): -6.0, -6.5, -7.0, -7.5, -8.0, -8.5, -9.0, -9.5, -10.0, -10.5

DEPTH (m): 10.5, 11.0, 11.5, 12.0, 12.5, 13.0, 13.5, 14.0, 14.5, 15

ENGINEERING DESCRIPTION

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.

DEPTH (m)	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	DESCRIPTION
to 10.6	SW	W	MD	Fine to medium SAND with trace silt, grey. Medium dense, wet. - 10.5m to 10.75m some shells
10.6–11.1	ML	W	S	SILT with some sand, blue grey. Soft, wet, low plasticity.
11.1–13.9	SW	W	MD	Fine to medium SAND with some silt interbedded, grey. Medium dense, wet. 11.45m to 11.7m no recovery 12.75m to 13.25m no recovery - extremely closely spaced thinly laminated silt bed
13.9–15	SW	W	MD	Silty, fine to medium SAND, grey. Medium dense, wet. Silt is interbedded. 14.35m to 14.75m no recovery - contains some shells

T+T DATATEMPLATE.GDT eek

NZGD ID: BH_1740

TONKIN & TAYLOR LTD

BOREHOLE LOG

BOREHOLE No: CBD 07
Hole Location: Harvey Tce

SHEET 4 OF 6

PROJECT: CHRISTCHURCH CITY 2011 REMEDIATION
LOCATION: CENTRAL CITY
JOB No: 52000.3400

CO-ORDINATES 5742473 mN
2481875.25 mE
R.L. 4.21 m
DATUM NZMG

DRILL TYPE: Rotary
DRILL METHOD: OB/Triple Tube
DRILL FLUID: Mud

HOLE STARTED: 11/7/11
HOLE FINISHED: 12/7/11
DRILLED BY: Pro-Drill
LOGGED BY: RKH/CP
CHECKED: BMcD

GEOLOGICAL: CHRISTCHURCH FORMATION (MARINE & ESTUARINE)

DEPTH (m)	R.L. (m)	CORE RECOVERY (%)	METHOD	TESTS	SAMPLES	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	ENGINEERING DESCRIPTION
15.0–15.7	-11.0					SW	W	MD	Silty, fine to medium SAND, grey. Medium dense, wet. Silt is interbedded.
15.5–15.95	-11.5		SPT	2/3/3/5/7/9 N=24					15.7m to 15.95m no recovery
15.95–17.0	-12.0 -12.5	100	HQTT	✱FC	B	SW	W	MD	Fine to medium SAND with trace silt, grey. Medium dense, wet.
17.0–17.45	-13.0		SPT	5/3/6/7/9/9 N=31				D	- becoming dense 17.4m to 17.5m no recovery
17.45–18.5	-13.5 -14.0	95	HQTT	✱FC	B				
18.5–18.95	-14.5		SPT	4/8/14/ 14/17/15 for 15mm N>50				VD	- becoming very dense
18.95–20	-15.0 -15.5	90	HQTT						19.4m to 19.55m no recovery

T+T DATATEMPLATE.GDT eek

TONKIN & TAYLOR LTD

BOREHOLE LOG

BOREHOLE No: CBD 07
Hole Location: Harvey Tce

SHEET 5 OF 6

PROJECT: CHRISTCHURCH CITY 2011 REMEDIATION LOCATION: CENTRAL CITY JOB No: 52000.3400

CO-ORDINATES	5742473 mN 2481875.25 mE	DRILL TYPE: Rotary	HOLE STARTED: 11/7/11
		DRILL METHOD: OB/Triple Tube	HOLE FINISHED: 12/7/11
R.L.	4.21 m		DRILLED BY: Pro-Drill
DATUM	NZMG	DRILL FLUID: Mud	LOGGED BY: RKH/CP CHECKED: BMcD

GEOLOGICAL / ENGINEERING 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 CONDITION / WEATHERING | STRENGTH/DENSITY CLASSIFICATION | SHEAR STRENGTH (kPa) 10 25 50 100 200 | COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100 250 | DEFECT SPACING (mm) 50 250 1000 2000

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.

Geological unit	Core recovery (%)	Method	Tests	Depth (m)	Classification symbol	Moisture	Strength/density	Description
CHRISTCHURCH FORMATION (MARINE & ESTUARINE)	97	HQTT		20.0–21.5	SW	W	MD	Fine to medium SAND with trace silt, grey. Medium dense, wet. 20.4m to 20.45m no recovery
		SPT	4/9/12/12/ 16/10 for 35 mm N>50	21.5–21.95				
	100	HQTT		21.95–23.0				
				22.55	ML	M	F	SILT with some sand, bluish grey. Firm, moist, low plasticity. Sand is fine.
		SPT	1/9/16/26/8 for 25mm N>50	23.0–23.45	GW	W	VD	Sandy, fine to coarse GRAVEL, grey. Very dense, wet. Gravel is subangular to subrounded. Sand is medium to coarse. 23.15m to 23.45m no recovery
RICCARTON GRAVELS	43	HQTT		23.45–24.5	GW	D	VD	Fine to coarse GRAVEL, grey. Very dense, dry. Gravel is subangular to subrounded. 23.9m to 24.5m no recovery
		SPT	4/4/5/5/6/7 N=23	24.5–24.95	SW	W	MD	Gravelly, medium to coarse SAND, yellowish brown. Medium dense, wet. Gravel is fine to coarse, subangular to subrounded. 24.85m to 24.95m no recovery

R.L. (m)	Depth (m)
-16.0	
	20.5
-16.5	
	21.0
-17.0	
	21.5
-17.5	
	22.0
-18.0	
	22.5
-18.5	
	23.0
-19.0	
	23.5
-19.5	
	24.0
-20.0	
	24.5
-20.5	
	25

T+T DATATEMPLATE.GDT eek

NZGD ID: BH_1740

TONKIN & TAYLOR LTD

BOREHOLE LOG

BOREHOLE No: CBD 07
Hole Location: Harvey Tce

SHEET 6 OF 6

PROJECT: CHRISTCHURCH CITY 2011 REMEDIATION	LOCATION: CENTRAL CITY	JOB No: 52000.3400
CO-ORDINATES 5742473 mN 2481875.25 mE	DRILL TYPE: Rotary	HOLE STARTED: 11/7/11
	DRILL METHOD: OB/Triple Tube	HOLE FINISHED: 12/7/11
R.L. 4.21 m		DRILLED BY: Pro-Drill
DATUM NZMG	DRILL FLUID: Mud	LOGGED BY: RKH/CP CHECKED: BMcD

GEOLOGICAL — GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.

ENGINEERING DESCRIPTION — 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.

Column scales: SHEAR STRENGTH (kPa) 10 25 50 100 200; COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100 250; DEFECT SPACING (mm) 50 250 1000 2000

GEOLOGICAL UNIT	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	CLASSIFICATION SYMBOL	MOISTURE CONDITION	WEATHERING / STRENGTH/DENSITY CLASSIFICATION	SOIL DESCRIPTION
RICCARTON GRAVELS			19	HQTT			GW	D	VD	Fine to coarse GRAVEL, grey. Very dense, dry. Gravel is subangular to subrounded. 25.15m to 26.0m no recovery
				SPT		25/25 for 95mm N>50				26.1m to 26.45m no recovery - becoming very dense
			19	HQTT						26.65m to 27.5m no recovery
				SPT		6/8/11/ 14/26 for 75mm N>50	GW	W	VD	Sandy, fine to coarse GRAVEL, grey. Very dense, wet. Gravel is subangular to subrounded. Sand is medium to coarse. 27.55m to 27.95m no recovery
			24	HQTT			GW	D	VD	Fine to coarse GRAVEL, grey. Very dense, dry. Gravel is subangular to subrounded. 28.2m to 29.0m no recovery Note: fines only recovered in SPT
				SPT		20/30 for 75mm N>50	GW	W	VD	Sandy, fine to coarse GRAVEL, grey. Very dense, wet. Gravel is subangular to 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.

R.L. (m): -21.0, -21.5, -22.0, -22.5, -23.0, -23.5, -24.0, -24.5, -25.0, -25.5

DEPTH (m): 25.5, 26.0, 26.5, 27.0, 27.5, 28.0, 28.5, 29.0, 29.5, 30

T+T DATATEMPLATE.GDT eek

NZGD ID: BH_1740

GEOTECH

Liquefaction Potential Analysis CPT001

Project: **256 Fitzgerald Avenue** Client: **R Harwood**
Job No: **5595** Date: 11/12/2020

GEOTECH CONSULTING LTD
Analysis: AJH
Checked: AJH
ref: Boulanger & Idriss 2014, Zhang 2002

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)
sigma(lnR) = 0.2

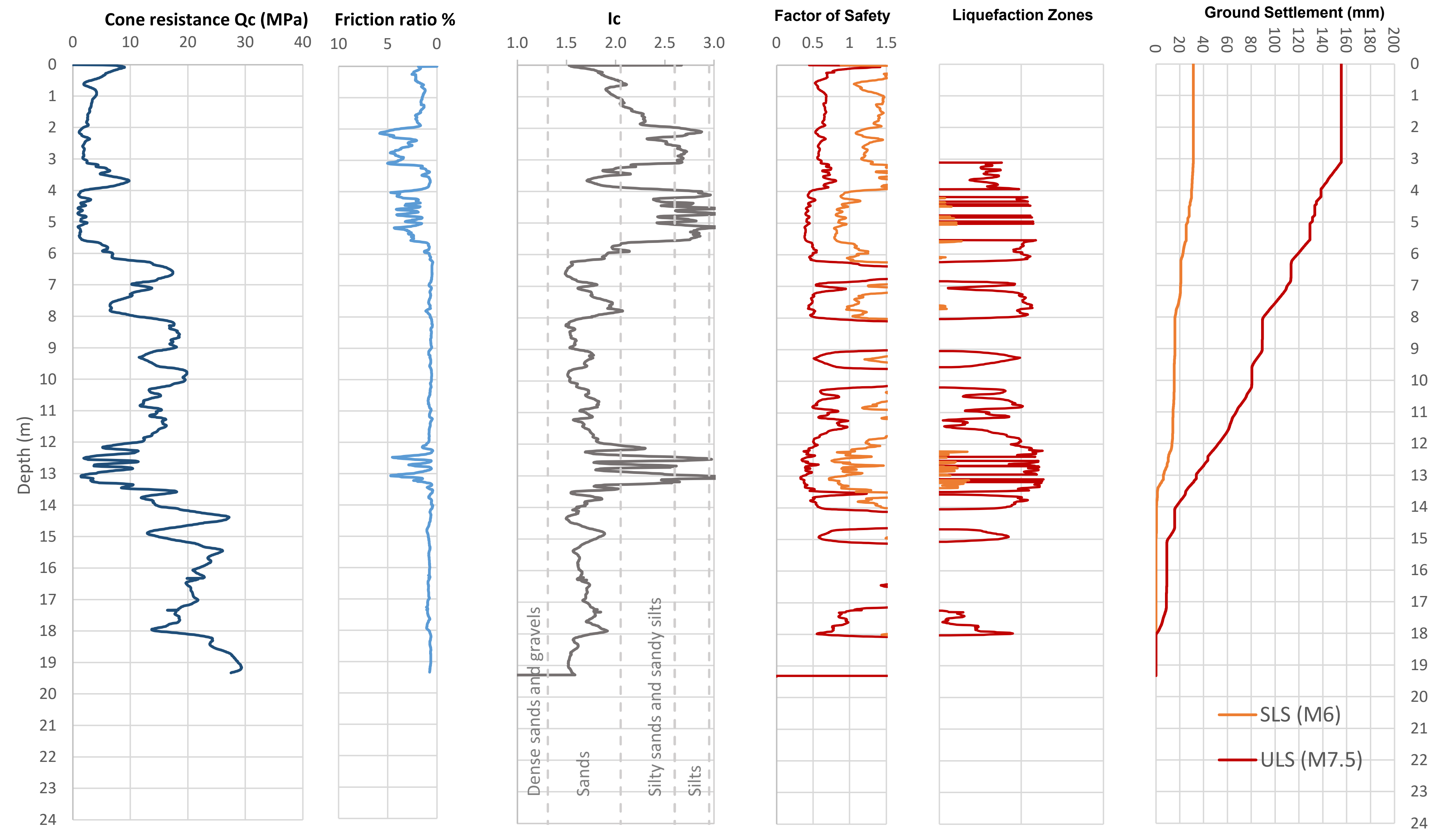
Seismic Load Cases	**Case 1** **ULS at M7.5**	**Case 2** **SLS at M7.5**	**Case 3** **SLS at M6**	**Case 4** **22 Feb 2011**
Peak Ground Acceleration (PGA) =	0.35	0.13	0.19	0.45
Magnitude M =	7.50	7.50	6.00	6.20
representative M =	6.80	7.50	6.00	6.20
Summary Results				
Overall settlement (Zhang) (mm):	155	20	31	149
Total liquefiable thickness (m):	8.31	0.33	1.1	7.70
Settlement in top 10m (mm):	75	8	16	74
Liquefiable thickness in top 10m (m):	3.62	0.00	0.47	3.56
Average MSF =	1.000	1.000	1.372	1.314
LSN ('mm')	14	1	3	14
LDI (m)	1.59	0.09	0.21	1.55
For free face of 4 m, LDI =	0.77	0.02	0.09	0.78

GEOTECH

Liquefaction Potential Analysis CPT001

Project: **256 Fitzgerald Avenue**
Job No: **5595**
Client: **R Harwood**
Date: 11/12/2020

GEOTECH CONSULTING LTD
Analysis: AJH
Checked: AJH
ref: Boulanger & Idriss 2014, Zhang 2002



GEOTECH

Liquefaction Potential Analysis CPT002

Project: **256 Fitzgerald Avenue**
Job No: **5595**
Client: **R Harwood**
Date: 12/12/2020

GEOTECH CONSULTING LTD
Analysis: AJH
Checked: AJH
ref: Boulanger & Idriss 2014, Zhang 2002

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)
sigma(lnR) = 0.2

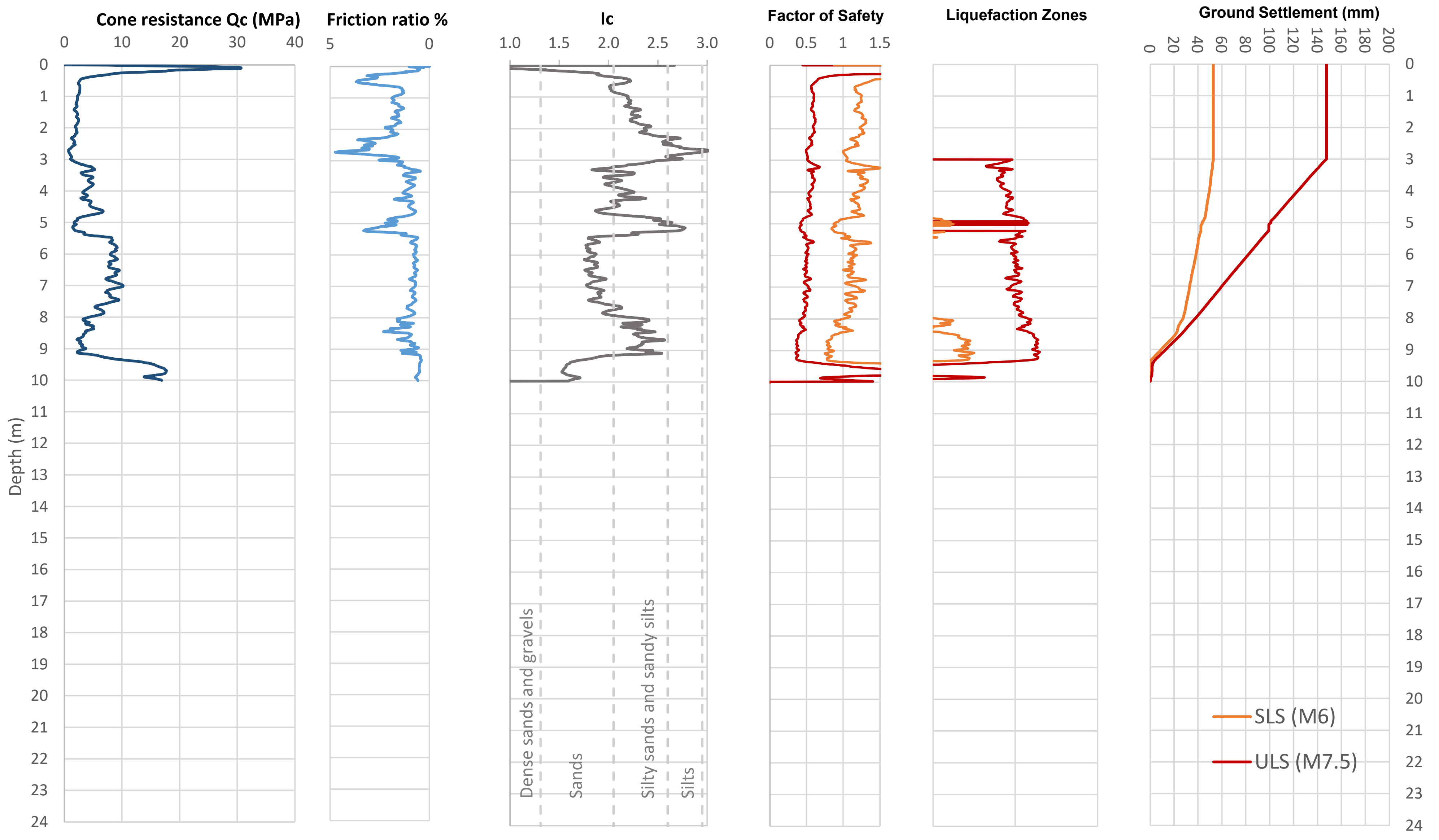
Seismic Load Cases	**Case 1** **ULS at M7.5**	**Case 2** **SLS at M7.5**	**Case 3** **SLS at M6**	**Case 4** **22 Feb 2011**
Peak Ground Acceleration (PGA) =	0.35	0.13	0.19	0.45
Magnitude M =	7.50	7.50	6.00	6.20
representative M =	6.80	7.50	6.00	6.20
Summary Results				
Overall settlement (Zhang) (mm):	147	27	53	147
Total liquefiable thickness (m):	6.33	0.75	1.5	6.29
Settlement in top 10m (mm):	147	27	53	147
Liquefiable thickness in top 10m (m):	6.33	0.75	1.46	6.29
Average MSF =	1.000	1.000	1.723	1.611
LSN ('mm')	26	4	8	26
LDI (m)	1.97	0.16	0.55	1.96
For free face of 4 m, LDI =	1.40	0.04	0.14	1.40

GEOTECH

Liquefaction Potential Analysis CPT002

Project: **256 Fitzgerald Avenue**
Job No: **5595**
Client: **R Harwood**
Date: 12/12/2020

GEOTECH CONSULTING LTD
Analysis: AJH
Checked: AJH
ref: Boulanger & Idriss 2014, Zhang 2002



GEOTECH

Liquefaction Potential Analysis CPT003

Project: **256 Fitzgerald Avenue**
Job No: **5595**
Client: **R Harwood**
Date: 12/12/2020

GEOTECH CONSULTING LTD
Analysis: AJH
Checked: AJH
ref: Boulanger & Idriss 2014, Zhang 2002

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)
sigma(lnR) = 0.2

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) =	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

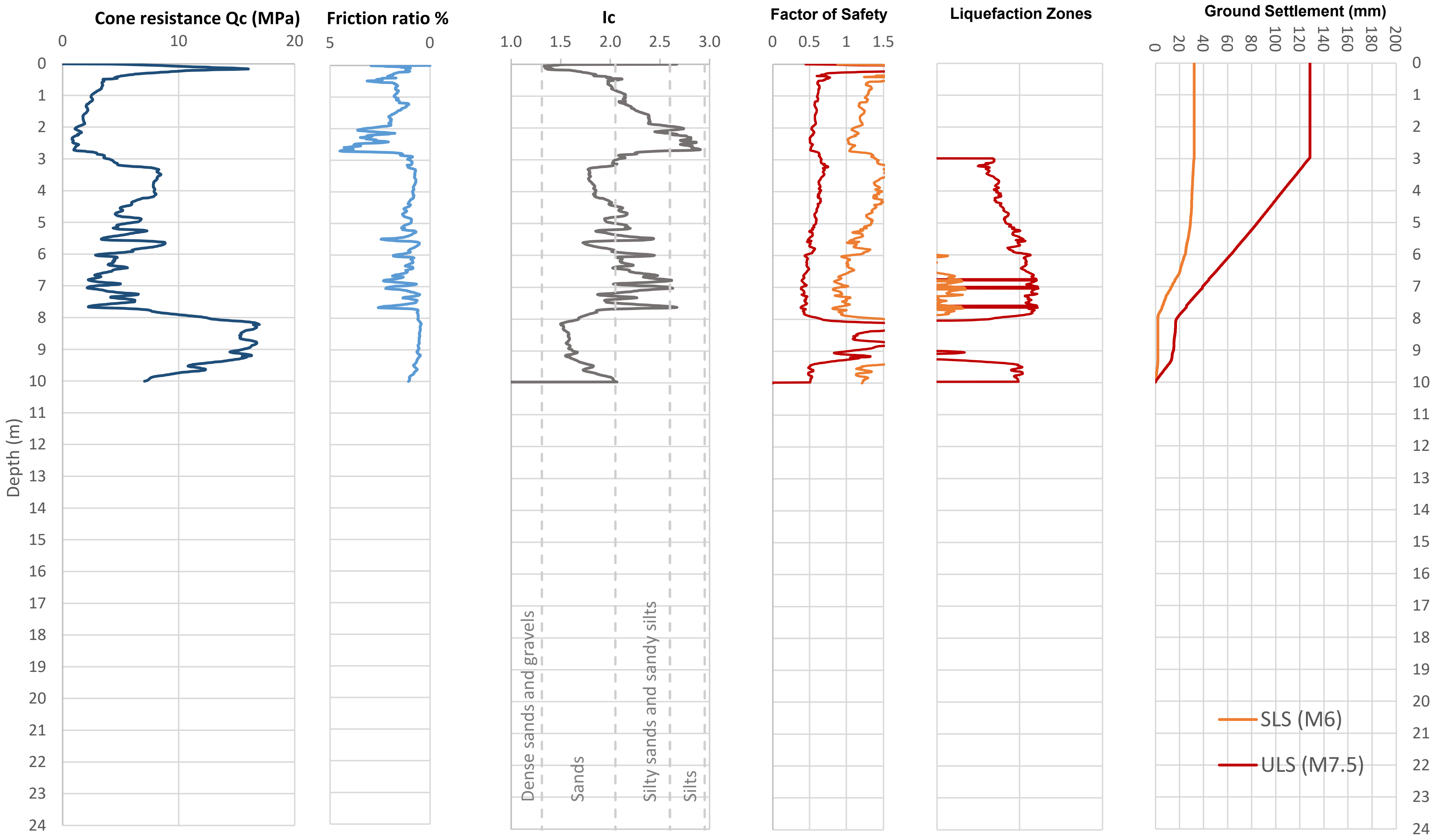
Summary Results				
Overall settlement (Zhang) (mm):	128	16	32	127
Total liquefiable thickness (m):	5.79	0.00	1.1	5.73
Settlement in top 10m (mm):	128	16	32	127
Liquefiable thickness in top 10m (m):	5.79	0.00	1.14	5.73
Average MSF =	1.000	1.000	1.352	1.297
LSN ('mm')	23	3	5	23
LDI (m)	1.54	0.06	0.22	1.53
For free face of 4 m, LDI =	1.38	0.05	0.21	1.38

GEOTECH

Liquefaction Potential Analysis CPT003

Project: **256 Fitzgerald Avenue**
Job No: **5595**
Client: **R Harwood**
Date: 12/12/2020

GEOTECH CONSULTING LTD
Analysis: AJH
Checked: AJH
ref: Boulanger & Idriss 2014, Zhang 2002



GEOTECH

Liquefaction Potential Analysis CPT004

GEOTECH CONSULTING LTD
Analysis: AJH
Checked: AJH
ref: Boulanger & Idriss 2014, Zhang 2002

Project: **256 Fitzgerald Avenue**
Client: **R Harwood**
Job No: **5595**
Date: 11/12/2020

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)
sigma(lnR) = 0.2

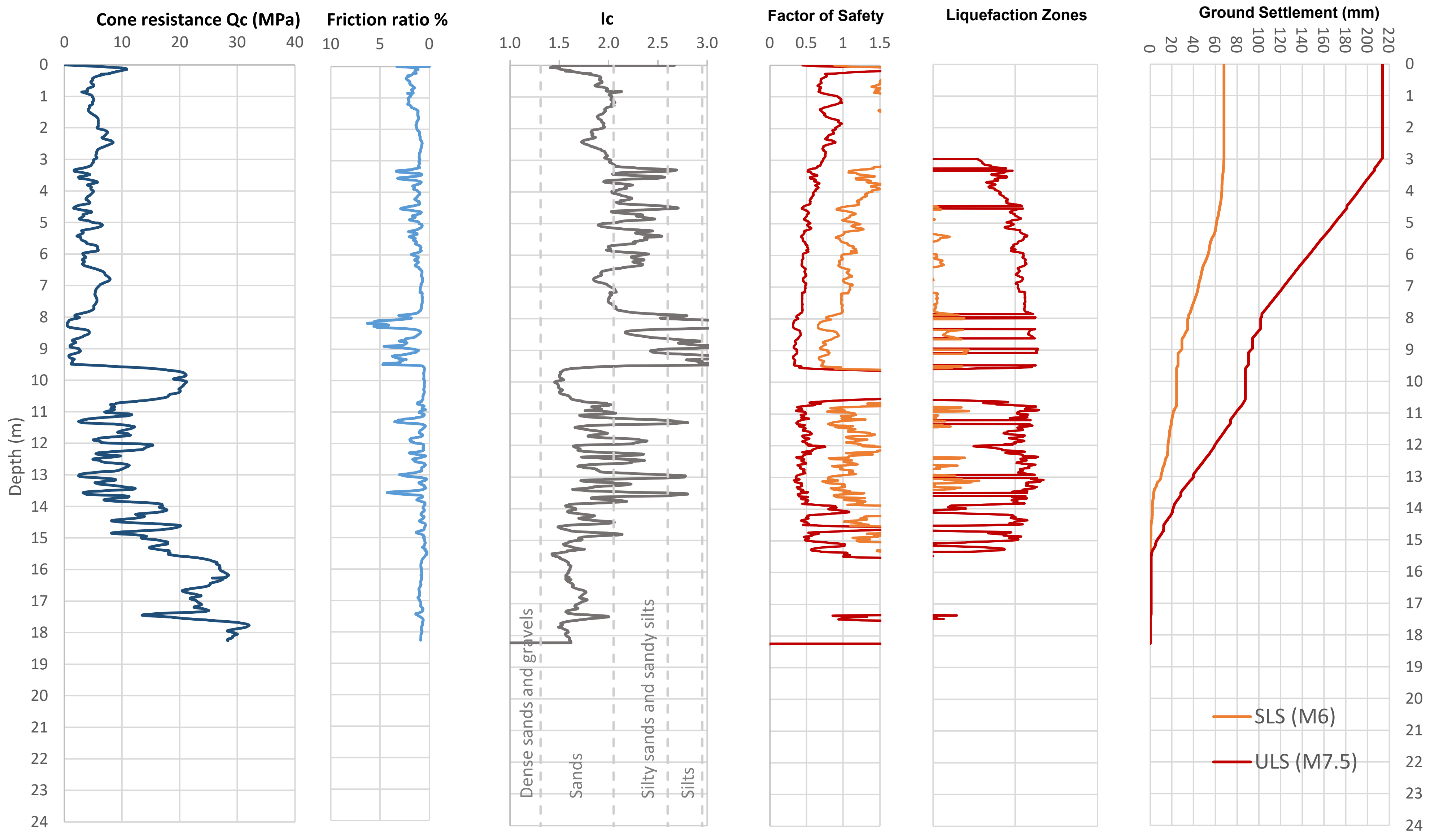
Seismic Load Cases	**Case 1** **ULS at M7.5**	**Case 2** **SLS at M7.5**	**Case 3** **SLS at M6**	**Case 4** **22 Feb 2011**
Peak Ground Acceleration (PGA) =	0.35	0.13	0.19	0.45
Magnitude M =	7.50	7.50	6.00	6.20
representative M =	6.80	7.50	6.00	6.20
Summary Results				
Overall settlement (Zhang) (mm):	214	39	68	211
Total liquefiable thickness (m):	9.72	0.62	2.8	9.44
Settlement in top 10m (mm):	126	20	44	126
Liquefiable thickness in top 10m (m):	5.39	0.23	2.04	5.38
Average MSF =	1.000	1.000	1.174	1.147
LSN ('mm')	23	3	7	23
LDI (m)	2.63	0.20	0.50	2.61
For free face of 4 m, LDI =	1.47	0.05	0.16	1.47

GEOTECH

Liquefaction Potential Analysis CPT004

Project: **256 Fitzgerald Avenue**
Job No: **5595**
Client: **R Harwood**
Date: 11/12/2020

GEOTECH CONSULTING LTD
Analysis: AJH
Checked: AJH
ref: Boulanger & Idriss 2014, Zhang 2002



GEOTECH

Liquefaction Potential Analysis

CPT_564

Project: **256 Fitzgerald Avenue**
Job No: **5595**
Client: **R Harwood**
Date: 11/12/2020

GEOTECH CONSULTING LTD
Analysis: AJH
Checked: AJH
ref: Boulanger & Idriss 2014, Zhang 2002

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)
sigma(lnR) = 0.2

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) =	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):	101	9	19	94
Total liquefiable thickness (m):	4.73	0.06	0.5	4.47
Settlement in top 10m (mm):	73	6	16	70
Liquefiable thickness in top 10m (m):	3.28	0.00	0.49	3.20
Average MSF =	1.000	1.000	1.060	1.050
LSN ('mm')	14	1	4	14
LDI (m)	1.10	0.03	0.18	1.06
For free face of 4 m, LDI =	0.80	0.02	0.16	0.80

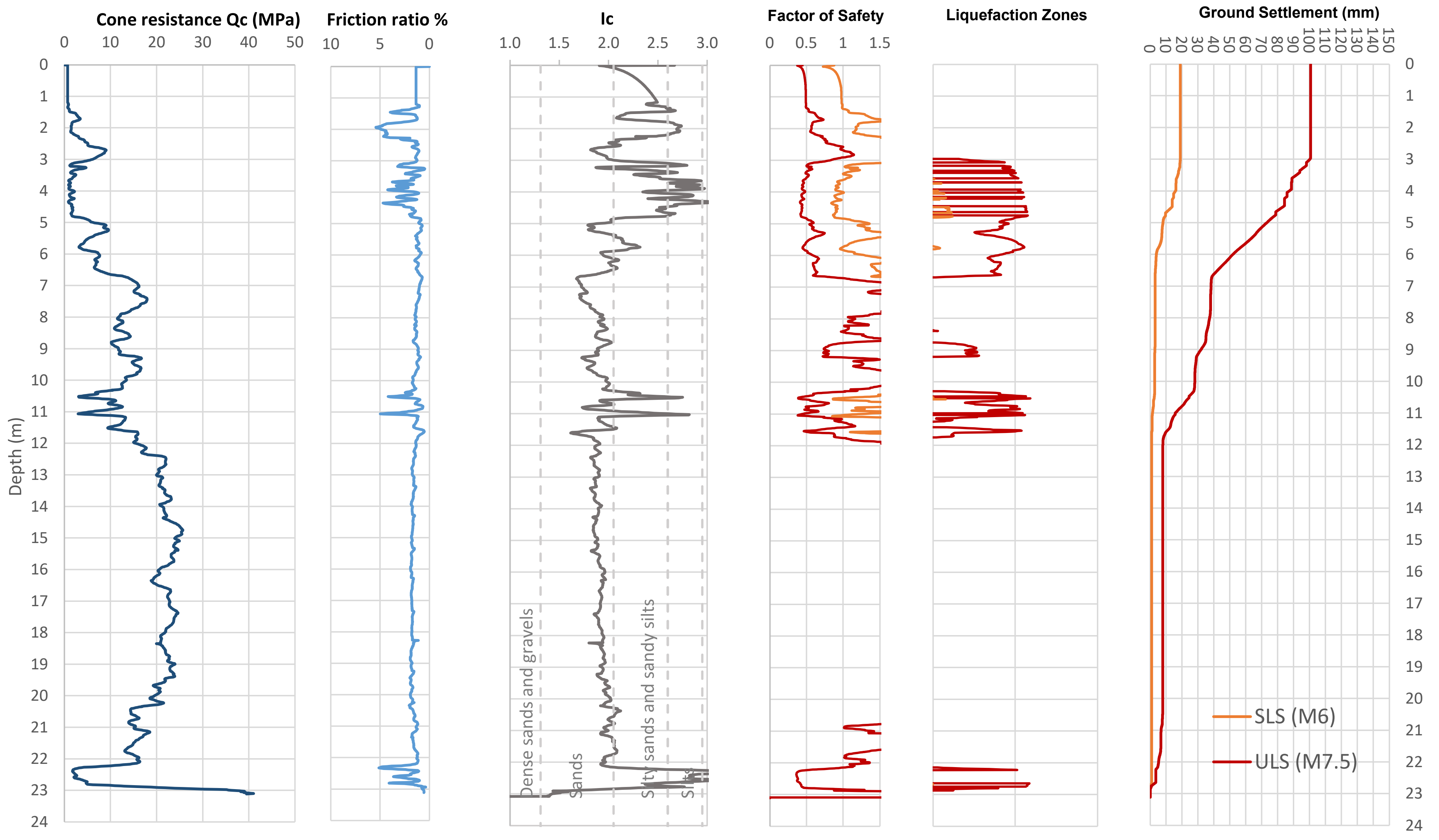
GEOTECH

Liquefaction Potential Analysis

CPT_564

Project: **256 Fitzgerald Avenue**
Job No: **5595**
Client: **R Harwood**
Date: 11/12/2020

GEOTECH CONSULTING LTD
Analysis: AJH
Checked: AJH
ref: Boulanger & Idriss 2014, Zhang 2002



GEOTECH

Liquefaction Potential Analysis CPT_404

Project: **256 Fitzgerald Avenue**
Job No: **5595**
Client: **R Harwood**
Date: 11/12/2020

GEOTECH CONSULTING LTD
Analysis: AJH
Checked: AJH
ref: Boulanger & Idriss 2014, Zhang 2002

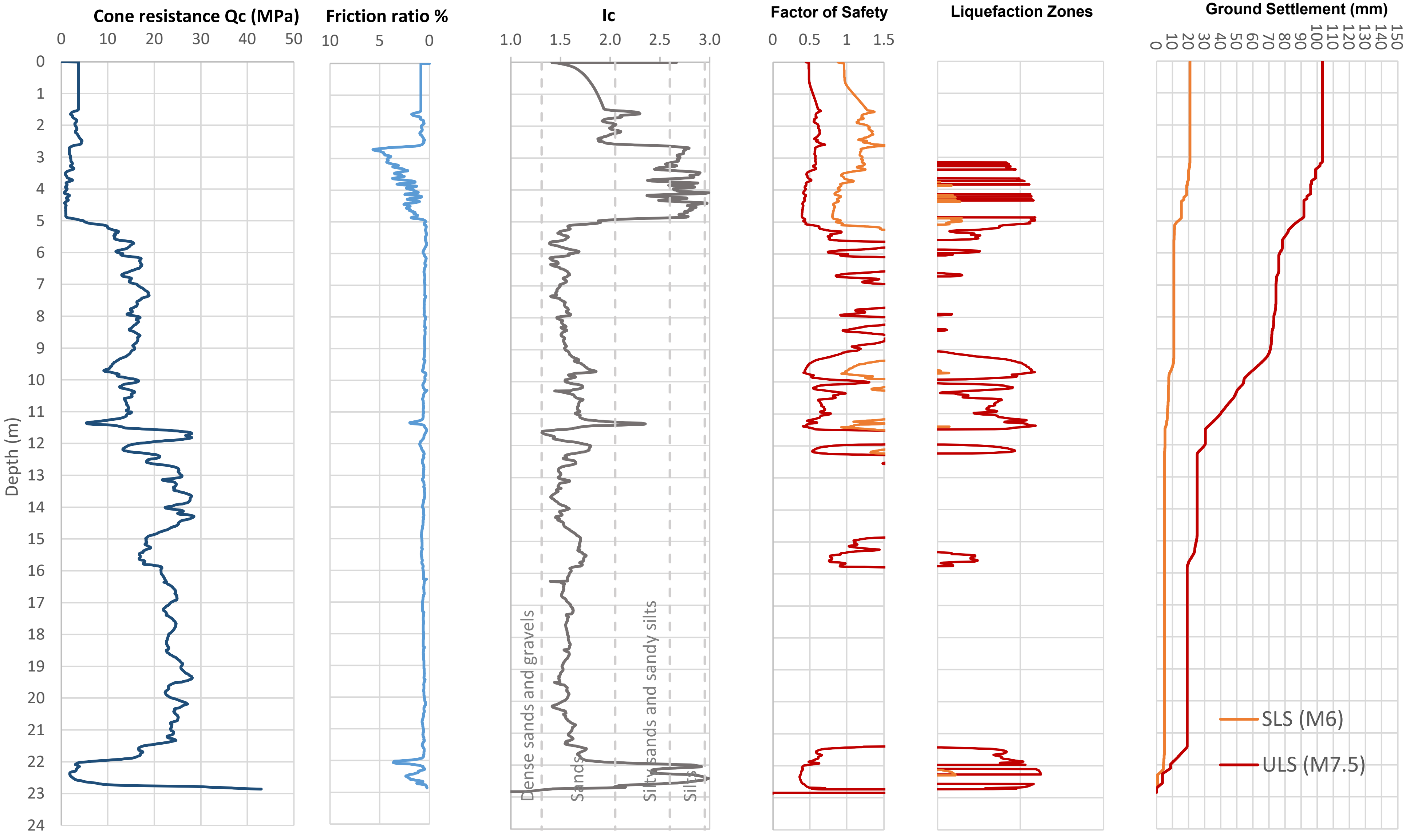
Input Parameters

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)
sigma(lnR) = 0.2

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) =	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):	103	13	21	96
Total liquefiable thickness (m):	5.50	0.19	0.7	4.98
Settlement in top 10m (mm):	49	5	13	46
Liquefiable thickness in top 10m (m):	2.41	0.00	0.48	2.20
Average MSF =	1.000	1.000	1.080	1.067
LSN ('mm')	8	1	3	8
LDI (m)	1.03	0.11	0.25	0.99
For free face of 4 m, LDI =	0.37	0.01	0.15	0.36

Liquefaction Potential Analysis	**CPT_404**		
Project:	**256 Fitzgerald Avenue**	Client:	**R Harwood**
Job No:	**5595**	Date:	11/12/2020

GEOTECH CONSULTING LTD
Analysis: AJH
Checked: AJH
ref: Boulanger & Idriss 2014, Zhang 2002



GEOTECH

Liquefaction Potential Analysis

CPT_46985

Project: **256 Fitzgerald Avenue**
Job No: **5595**
Client: **R Harwood**
Date: 12/12/2020

GEOTECH CONSULTING LTD
Analysis: AJH
Checked: AJH
ref: Boulanger & Idriss 2014, Zhang 2002

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)
sigma(lnR) = 0.2

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) =	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):	169	33	61	165
Total liquefiable thickness (m):	7.96	0.69	2.2	7.53
Settlement in top 10m (mm):	104	19	42	103
Liquefiable thickness in top 10m (m):	4.53	0.33	1.61	4.48
Average MSF =	1.000	1.000	1.068	1.057
LSN ('mm')	18	3	7	18
LDI (m)	2.07	0.26	0.70	2.04
For free face of 4 m, LDI =	0.89	0.02	0.27	0.89

GEOTECH

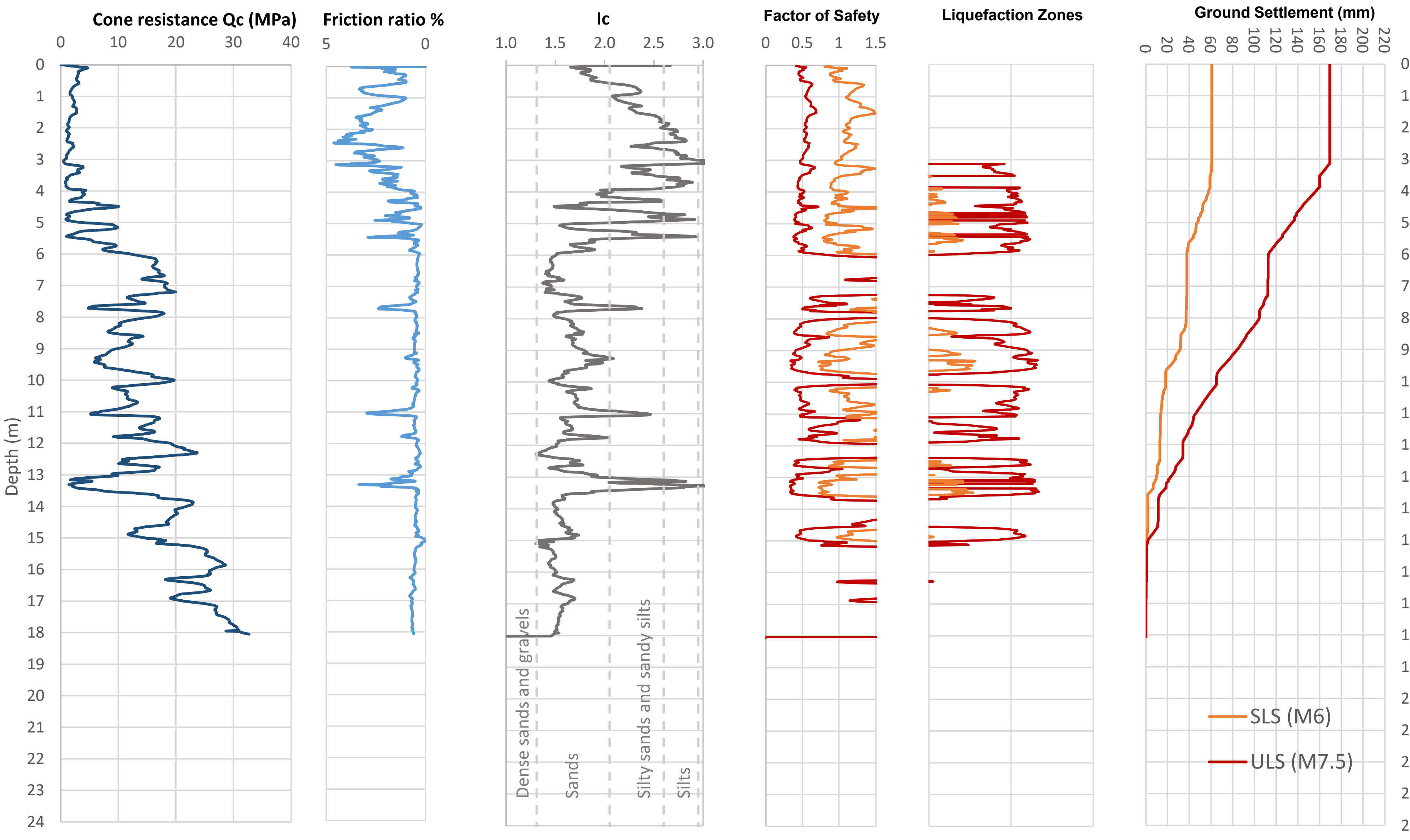
Liquefaction Potential Analysis

CPT_46985

Project: **256 Fitzgerald Avenue**
Job No: **5595**

Client: **R Harwood**
Date: 12/12/2020

GEOTECH CONSULTING LTD
Analysis: AJH
Checked: AJH
ref: Boulanger & Idriss 2014, Zhang 2002



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.

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.

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.

[redacted]