Appendix J

Geotechnical Review and Options Report

PAK'n SAVE Papanui

PAK'n SAVE Papanui Geotechnical Review and Options Report

Foodstuffs South Island Limited

Reference: 243354

Revision: 2

2018-08-09

Document control record

Document prepared by:

Aurecon New Zealand Limited
Level 2, Iwikau Building
93 Cambridge Terrace
Christchurch 8013
New Zealand

T +64 3 366 0821
F +64 3 379 6955
E christchurch@aurecongroup.com
W aurecongroup.com

Document control						aurecon
Report title		PAK'n SAVE Papanui Geotechnical Review and Options Report				
Document code			**Project number**		243354	
File path		P:\243354\Papanui PnS\Geotech\Report\243354 PAKn SAVE Papanui Geotech Rev1.docx				
Client		Foodstuffs South Island Limited				
Client contact		Rebecca Parish	**Client reference**			
Rev	**Date**	**Revision details/status**	**Author**	**Reviewer**	**Verifier** (if required)	**Approver**
0	2018-05-30	Draft for internal review	D. Mahoney	I. McPherson	-	-
1	2018-06-05	Draft for Client comment	D. Mahoney	I. McPherson	-	J. Kupec
2	2018-08-09	Issue Copy	D. Mahoney	I. McPherson	-	J. Kupec
Current revision		**2**				

Approval			
Author signature		Approver signature	
Name	Dominic Mahoney	Name	Dr Jan Kupec elect. sign
Title	Senior Geotechnical Engineer	Title	Technical Director – Ground Engineering

Contents

Appendices

Tables

Executive Summary

Introduction

Foodstuffs South Island Limited (Foodstuffs) is proposing to develop a new PAK'n SAVE Supermarket at their site located at 171 Main North Road, Papanui in Christchurch. A key component of the proposed development is to provide a high level of post disaster resilience and it is proposed to have sufficient resilience built into the building to allow it to be used it for Civil Defence purposes and as a post natural disaster Community Hub, in addition to normal retail activities. Therefore, the building is to be assessed as an Importance Level 4 structure in terms of the New Zealand Loadings Standard NZS1170. Foodstuffs have engaged Aurecon NZ Limited (Aurecon) to provide geotechnical engineering services, amongst other services, to support a land use consent application.

The project will consist of a 6,000m^2 supermarket building with a underground car parking basement approximately 4m deep. The development will have 364 car parks underground and surface car parks On the eastern side of the car park there will be a vehicle fuel facility. The site has a total area of 1.56ha.

Geotechnical Investigations

A review of the existing available geotechnical information has been undertaken using published information in both the New Zealand Geotechnical Database and Environmental Canterbury GIS System.

This review indicated that the site is underlain by interbedded layers and lenses of Silts, Sandy-Silts and Sand to approximately 18m depth. Below this depth is the 'Riccarton Gravel' layer. The long-term groundwater level at the site is in the order of 1m below ground level.

Liquefaction Assessment

A liquefaction hazard assessment has been undertaken at the site based on the methodology outlined in the MBIE (2012) and MBIE/NZGS (2016) guidelines. Based on this assessment:

- Under the 1 in 25 year SLS design earthquake event the calculated reconsolidation settlements are expected to range from 65mm to 75mm, with minor softening in the sandy material throughout the upper 18m of the soil profile. Minor surface expression of liquefaction is possible with some sand boils expected. The extent of liquefaction and associated ground movement is expected to be less than that which occurred during either the 2010 Darfield or 2011 Christchurch Earthquakes
- Under the 1 in 500 year SLS-2 design earthquake the calculated reconsolidation settlements are expected to range from 135mm to 150mm, with liquefaction throughout the sandy material in the upper 18m of the soil profile. Moderate to severe expression of liquefaction and settlement that could cause structural damage is expected. Liquefaction triggering and associated ground damage expected to be greater than which occurred during the 2010-2011 earthquakes.
- Under the 1 in 2,500 year ULS design earthquake event the calculated reconsolidation settlements are expected to range from 135mm to 150mm, with liquefaction throughout the sandy material in the upper 18m of the soil profile. Moderate to severe expression of liquefaction and settlement causing structural damage is expected. Liquefaction triggering and associated ground damage expected to be greater than which occurred during the 2010-2011 earthquakes and slightly larger than that the SLS-2 design earthquake event.
- The site is inferred to have minimal risk of lateral spreading.

Site Flexibility

We currently consider that the site subsoil category in terms of NZS 1170:5:2004 Clause 3.1.3 is Class D (deep or soft soil sites).

Foundation Recommendations

Given the underlying ground conditions; the identified liquefaction risk; the size and form of the proposed supermarket building including the underground car park basement; and Foodstuff's desire for post-earthquake resilience, the preferred foundation system is steel screw piles founded into the underlying 'Riccarton Gravels' with a fully suspended basement floor slab. These piles would likely be founded at a depth in the order of 23m (a nominal 5m into the underlying 'Riccarton Gravels').

Preliminary foundation design parameters are presented in Section 4.5.4 of this Report

Preliminary Development Recommendations

Preliminary geotechnical recommendations around the basement structure (including the walls, floor slab and waterproofing), the re-use of the existing artesian bores, the fuel facility and underground fuel tanks, and basement construction constraints are presented in Section 4.5.5 to 4.5.9 of this Report.

Recommendations

Due to the limited geotechnical information currently available, as part of the detailed design process we recommend that a program of additional geotechnical testing is undertaken. The scope of this testing is detailed in Section 4.6.1 of this Report.

A review of any detailed environmental site assessments should be undertaken to determine what effect, if any, the outcome of this assessment may have on the geotechnical component of the detailed design.

Due to the specialist nature of screw pile design and considering to the complexities of pile design and the feedback loops that pile design can have on superstructure and floor slab design, we recommend that a specialist screw pile contractor is engaged early in the design process to undertake the detailed design of screw piles. This design would be undertaken in conjunction with the project structural engineer using the geotechnical design parameters presented above and confirmed following the additional geotechnical testing recommended above.

Due to the potential technical difficulties in excavating such a large basement in the confined urban setting of the site, and the associated disposal of extracted groundwater, we recommend early engagement with potential specialist earthworks contractors to develop an appropriate dewatering methodology as groundwater this has the potential to affect the construction sequencing and design of the basement.

Safety in Design

A preliminary Safety in Design Assessment is presented in Section 4.7 of this report.

Limitations

This report presents preliminary foundations design recommendations based upon readily available published and unpublished geotechnical information for the site. This report will need to be updated and reissued once additional geotechnical investigation data becomes available and will confirm detailed foundation design parameters. This report shall be read as a whole and our limitations are in Section 6 of this report.

1 Introduction

Foodstuffs is proposing to develop a new PAK'n SAVE Supermarket at their site located at 171 Main North Road, Papanui in Christchurch. A key component of the proposed development is to provide a high level of post disaster resilience and it is proposed to have sufficient resilience built into the building to allow it to be used it for Civil Defence purposes and as a post natural disaster Community Hub, in addition to normal retail activities. Therefore, the building is to be assessed as an Importance Level 4 structure in terms of the New Zealand Loadings Standard NZS1170. Foodstuffs have engaged Aurecon to provide geotechnical engineering services, amongst other services, to support a land use consent application.

The project will consist of a 6,000m^2 supermarket building with a underground car parking basement approximately 4m deep. The development will have 364 car parks in the underground car park below the store and using on-grade car parking at the front of the store. On the eastern side of the car park there will be a vehicle fuel facility. The site has a total area of 1.56ha.

As part of the initial development assessment, and to support a land use consent application, a geotechnical desktop review and preparation of a geotechnical options report is to be undertaken. This initial review and optioneering scope of works comprises:

- Reviewing the existing geotechnical information for the site from readily available published and unpublished reports.
- Undertaking a liquefaction hazard assessment specific to an IL4 building.
- Preparing concept level design options for the site including the building, fuel facility etc.
- Commenting on the technical pros and cons of options.
- Commenting on basement construction.
- Identifying additional geotechnical testing requirements going forward.
- Preparing indicative hand sketches, as required.
- Liaising with design team members, especially the structural and civil engineers.
- Preparing this report detailing the above.

This report presents the results of our geotechnical review, and our preliminary recommended foundation system and foundation design parameters. Our work was carried out as a variation to our existing agreement with Foodstuffs as per our fee proposal email to Rebecca Parish of Foodstuffs on 22 June 2018. Approval to proceed was given a short time thereafter.

Our limitations are outlined in Section 6 and this report shall be read as a whole.

2 Site Conditions

2.1 Site Description

The site is located at 171 Main North Road, Papanui in Christchurch and is rectangular in shape. See Figures 1 and 2 in Appendix A for further details

- The site has the legal description of Lot 1 DP212074 and has an area of approximately 1.56ha.
- The site is located in the northeast corner of the wider Foodstuff's Papanui site. It is bounded to the east by Main North Road, to the south by car parking and office buildings associated with the wider Foodstuff's facility, to the west by a warehouse building that was previously used as Foodstuffs Distribution Centre, and to the north by residential properties, a motor garage and a set of shops.
- The former Murdoch Manufacturing Plant is located in the centre of the site and comprises a number of separate buildings of various shapes, sizes and uses.
- The site is covered in a mixture of buildings, paved car parks, driveways and garden areas with mature trees.
- Site drainage is inferred to be via a reticulated drainage system on site.
- The site is effectively flat and level with less than 1m change in ground level across the site.

2.2 Regional Geology

The regional geology of the site is described by Brown and Weeber (1992) as straddling a river terrace and is underlain by *"Dominantly alluvial sand and silt overbank deposits (spy)."*

2.3 Seismicity

The site lies close to the epicentres of recent significant earthquakes as summarised in Table 1 below.

Table 1 Recent Earthquake Activity

Earthquake	Distance from Epicentre(1)	Moment Magnitude	Median PGA on Site(2)	Standard Deviation(2)	Equivalent Median PGA for M_w7.5 Event(3)
4 September 2010 – Darfield Earthquake	37km east	M_w7.1	0.21g	0.300	0.19g
22 February 2011 – Christchurch Earthquake	12km north-northwest	M_w6.2	0.23g	0.325	0.16g
13 June 2011 – Major Aftershock	17km northwest	M_w6.0	0.13g	0.350	0.09g
23 December 2011 – Major Aftershock	15km west-northwest	M_w5.9	0.17g	0.350	0.11g

(1) Institute of Geological and Nuclear Sciences (GNS, 2018).
(2) Peak Ground Accelerations (PGA) on site based on the median values of the study by Bradley Seismic Limited as published in the NZGD (Bradley and Hughes, 2012).
(3) Calculated using the magnitude scaling factor based on the method of Idriss and Boulanger (2008).

2.4 Earthquake Induced Ground Damage

Based on our discussions with both Foodstuffs' staff and Powell Fenwick (who carried out post-earthquake inspections and a Detailed Engineering Evaluation of the existing buildings on site) no ground damage, foundation movement or settlement was observed on site following the major seismic events of the 2010-2011 Canterbury earthquake sequence.

A review of the New Zealand Geotechnical Database (NZGD, 2018) indicated the following as summarised in Table 2 on below.

2.5 Technical Category Classification

According to the Ministry of Business Innovation and Employment (MBIE) the site is currently classified as *N/A Urban Non-residential.* However, the neighbouring residential sites directly adjacent to the site are all classified Technical Category 2 (TC2). A TC2 zoned site indicates that *"Minor to moderate land damage from liquefaction is possible in future large earthquakes."* We note that although a Technical Category Classification is not directly applicable to a commercial building it does provide some insight into expected future site behaviour in a major earthquake event.

Table 2 **New Zealand Geotechnical Database Review**

Parameter	4 Sep 2010	22 Feb 2011	13 Jun 2011	23 Dec 2011
Review of Aerial Photographs	No observed liquefaction	No observed liquefaction	No observed liquefaction	No observed liquefaction
Liquefaction and Lateral Spreading Observations	N/A	No observed ground cracking or ejected liquefied material in residential properties on south side of wider Foodstuffs site. (Main North Road) No observed liquefied material	(Main North Road) No observed ground cracking or ejected liquefied material	N/A
Ground Cracking	Not Mapped		None mapped	
Vertical Ground Movement, LiDAR (±0.1m)	-0.2m to +0.1m	-0.1m to +0.1m	-0.1m to +0.1m	Not mapped
Horizontal Ground Movement, LiDAR (±0.4m)	0.26m to west	0.3m to west-southwest	<0.05m	Not mapped
EQC Ground Water Levels	2 to 3mbgl	2 to 3mbgl	2 to 3mbgl	2mbgl

3 Geotechnical Investigations

3.1 General

The objective of the geotechnical review was to obtain information on the site ground and groundwater conditions. From this information likely geotechnical risks at the site can be assessed and recommendations provided on the foundation requirements and geotechnical design parameters for concept design of the new building. The investigation results would also allow a preferred foundation solution to be determined. At this stage no physical testing has taken place and the geotechnical investigations comprise a review of readily available published information adjacent to the site. Further geotechnical testing will be carried out as part of the detailed design stage of the project.

This process comprised a review of readily available information on the Environment Canterbury (ECan) GIS system (ECan, 2018) and New Zealand Geotechnical Database (NZGD, 2018). This section of the report presents the results of this review.

For this review we have only reported on the logs that are adjacent to the site (i.e. within 250m). The locations of the logs are presented in Figure 2 in Appendix A

3.2 ECan Borehole Logs

Two Environment Canterbury (ECan) boreholes are located at the site and the stratigraphy encountered in the two boreholes is summarised in Table 3 below. The locations of the ECan borehole logs are presented in Figure 2 in Appendix A and the borehole logs are presented in Appendix B. In addition to these two deep boreholes, numerous shallow (typically less than 3m depth) borehole were located around the site. These borehole logs typically recorded silts and sands in the upper soil profile. We understand that these two borehole logs correspond to existing water wells located on the site.

Table 3 ECan Borehole Logs Summary

Borehole	Location	Depth	Summary of Stratigraphy
M35/1348	On site	24.4mbgl	• 0 to 10.7m – Sand and clay • 10.7 to 11.6m – Blue clay • 11.6 to 15.8m – Blue sand and gravel • 15.8 to 18m – Clay and peat • 18 to +24.4m – Brown shingle
M35/1472	On site	18.3mbgl	• 0 to 1.5m – Clay silt • 1.5 to 6.1m – Lenses of clay, silt and organic matter • 6.1 to 7.3m – Organic clay and peat • 7.3 to 12.2m – Silty clay • 12.2 to +18.3m - Sand

3.3 NZGD Review

A review of the NZGD identified four deep tests with useable logs (boreholes and CPTs) within approximately 350mof the site. These logs are summarised in Table 4 below and the logs are presented in Appendix C. In addition to these four deep tests there are numerous shallow tests (hand auger boreholes and Dynamic Cone Penetrometer tests) typically less than 3m deep in the vicinity of the site. These logs typically indicated silts and sands in the upper soil profile.

Table 4 NZGD Log Summary

Test	Location	Depth	Summary
CPT_57000 (KGA CPT 15203-A)	200m west on western side of old distribution centre warehouse	18mbgl	• Surface to 0.5 – Sandy-Gravel (Fill) • 0.5m to 2.4m – Sandy Silt • 2.4m to 3m – Sand • 3m to 4.4m – Sandy-Silt and Silt • 4.4m to 5m – Silty-Sand • 5m 10.4m – Clayey-Silt to Silty-Clay • 10.4m to 11.4m – Silty-Sand and Sandy-Silt • 11.4m to 13m – Sand • 13m to 14.2m - Clayey-Silt to Silty-Clay • 14.2m to 17.4m Interbedded layers of Clay and Sand • 17.4m onwards - Gravel
CPT_57002 (KGA CPT 15203-B)	200m west on western side of old distribution centre warehouse	18mbgl	• Surface to 0.5 – Sandy-Gravel (Fill) • 0.5m to 2.2m – Silty-Sand and Sandy-Silt • 2.2m to 2.8m – Sand • 2.8m to 4.4m – Sandy-Silt and Silt • 4.4m to 5m – Silty-Sand • 5m 10.4m – Clayey-Silt to Silty-Clay • 10.4m to 11.6m – Silty-Sand and Sandy-Silt • 11.6m to 13m – Sand • 13m to 14.2m - Clayey-Silt to Silty-Clay • 14.2m to 17.8m Interbedded layers of Clay and Sand • 17.8m onwards - Gravel
BH_62411 (TT BH01)	350m northeast in St Bede's College	20.25mbgl	• Surface to 2.2m – Firm Silt • 2.2m to 9.1m – Very soft Silt with some organics • 9.1m to 12.1m – Medium dense Sand with some Silt • 12.1m to 13.4m – Very soft Silt • 13.4m to 17.3m – Medium dense to dense Sand with minor silt • 17.3m to 20m onwards - Dense to very dense Gravel in a Sand and Silt matrix
BH_62413 (TT BH03)	350m northeast in St Bede's College	20.25mbgl	• Surface to 2.2m – Firm Silt • 2.2m to 2.5m – Loose Sand • 2.5m to 10.1m – Very soft to soft Silt with some organics • 10.1m to 12.7m – Medium dense Sand with some Silt and lenses of soft Silt • 12.7m to 14.1m –Soft Silt • 14.1m to 18.9m - Medium dense to dense Sand with lenses of Silty-Sand and Organics • 18.9m to 19.4m – Stiff Silt • 19.4m to 20m onwards- Very dense Gravel in a Sand and Silt matrix

3.4 Ground Water

Groundwater levels have been recorded from the following sources:

- The NZGD indicates a long-term estimate of groundwater levels across the site at approximately 18mRL (Christchurch Drainage Datum) corresponding to a depth of approximately 1mbgl.
- ECan borehole log M35/1348 recorded a minimum water level of 0.4mbgl.
- Various other ECan borehole logs around the site record groundwater levels between 1.4m and 0.8mbgl.
- NZGD boreholes BH_62411 and BH_62413 do not record shallow groundwater levels. They do however record artesian groundwater flow in the 'Riccarton Gravels' at 20m depth with artesian head estimated to be several metres above ground level.

Groundwater levels will vary seasonally or with periods of prolonged precipitation or drought.

4 Engineering Considerations

4.1 General

Foodstuffs is proposing to develop a new PAK'n SAVE Supermarket at their site located at 171 Main North Road, Papanui in Christchurch. A key component of the proposed development is to provide a high level of post disaster resilience and is proposed to have sufficient resilience built into the building to allow it to be used it for Civil Defence purposes and as a post natural disaster Community Hub, in addition to normal retail activities. Therefore, the building is to be assessed as an Importance Level 4 structure in terms of the New Zealand loadings standard NZS1170.

The project will consist of a 6,000m^2 supermarket building with a underground car parking basement approximately 4m deep. The development will have 364 car parks in the underground car park below the store and using on-grade car parking at the front of the store. On the eastern side of the car park there will be a vehicle fuel facility. The site has a total area of 1.56ha.

Two existing artesian water wells are located on site which are to be retained for re-use if possible. We understand that the wells are located within the footprint of the proposed new PAK'n SAVE and will have to be detailed in such a way that the well shaft can penetrate through the basement structure.

Due to the likely ground and groundwater conditions, the presence of the basement, and the high-level of resilience required for this store, a standard shallow type foundation system is unlikely to viable for this development. Therefore, a geotechnical optioneering exercise has been undertaken using the available geotechnical information. From this exercise key geotechnical risk have been identified and suitable foundation options have been determined to address these risks.

This section of the report presents our preliminary ground model underlying the site, the site subsoil classification, our liquefaction assessment, and our recommendations for foundations in-ground mechanical services and civil engineering activities.

4.2 Ground Model

Based upon the results of our geotechnical review we infer the ground model at the site as detailed in Table 5 below.

Table 5 Inferred Ground Model

Geotechnical Unit	Material	Depth to top of layer	Thickness of layer	Typical q_c value	Typical N_{SPT} Value
Unit 1	Soft to firm Silt and Sandy-Silt interbedded with loose to medium dense Sand	Surface	4m to 5m	2 to 5MPa	4 to 5
Unit 2	Very soft Silt with some Organics	4m to 5mbgl	7m	0.5 to 1MPa	0 to 2
Unit 3	Loose to medium dense Sand with lenses of very soft Silt	11.5m to 12mbgl	4m	10 to 15MPa	20 to 40
Unit 4	Interbedded medium dense Sand and soft Silt	16mbgl	2m	2 or 8MPa	N/A
Unit 5	Dense to very dense Gravel in a Silt and Sand Matrix	18m to 18.5mbgl	Proven to over 6m	>40MPa	41 (29 – 50+)

Based upon the result of the geotechnical site investigation for design purposes we have assumed a groundwater level of 1mbgl. Groundwater levels will however vary seasonally and with periods of prolonged precipitation or drought. Flowing artesian groundwater pressures, possibly with several metres of head are assumed to be present in the 'Riccarton Gravel' layer below the site.

4.3 Site Classification

In the site's normal non-liquefied condition, we have assessed the site flexibility based on the following:

- Brown and Weeber (1992) indicate that the depth to rock in northern Christchurch area is hundreds of metres deep.
- Site stratigraphy comprises over 18m of sands, silts with some organics overlying gravel of an unknown thickness.
- Clause 3.1.3 and Table 3.2 of NZS 1170.5:2004.

We consider that based upon current information the site subsoil category in terms of NZS 1170.5:2004 Clause 3.1.3 is Class D (Deep or soft soil sites).

We note that based upon an inferred SPT 'N' profile generated from CPT_57000 and CPT_57002 using published correlations, the thickness of soil with SPT 'N' less than 6 is close to the accumulated thickness of 10m which is the boundary between a D or E subsoil classification. Therefore, the potential exists that the site could possibly classified as a Class E with further geotechnical testing and the site subsoil category will need to be assessed during detailed design (See Section 4.6.1 for further comment). We recommend using a site shear wave velocity testing to confirm the site classification.

4.4 Liquefaction Assessment

4.4.1 Introduction

Under cyclic loading (i.e. during an earthquake) loose, non-cohesive materials such as gravels, sands, silty-sands, tend to decrease in volume. This tendency to decrease in volume is much greater in loose than in dense soils. When loose non-cohesive soils are saturated and rapid loading occurs under undrained conditions, the soils densification causes pore water pressure to increase. The increase in pore water pressure results in a loss of soil strength due to a decrease in effective stress and eventually liquefaction occurs when the effective stress drops to zero. Liquefaction can lead to large displacements of foundations, flow failures of slopes and ground surface settlement, sand boils, and post-earthquake stability failures.

In determining the liquefaction potential at the site, the main factors to be considered are:

- Which layers have liquefied?
- What is the likelihood of further liquefaction in the future?
- How the potential liquefaction affects the development?

Each of these is considered below.

4.4.2 Potential for Liquefaction

Three primary factors contribute to liquefaction potential:

- Soil grading and density
- Groundwater
- Earthquake intensity and level of ground shaking

Each of these is discussed below

Soil Grading and Density

The CPT and borehole logs show layers of loose to medium dense sands and silty-sands in the upper 18m of the soil profile. These layers are considered to be potentially susceptible to liquefaction from a soil grading and density perspective.

Some layers of the upper soils were logged as clayey-silt and these have been assumed to be non-liquefiable. For the CPT profiles this non-liquefiable cut-off is assumed to be where the Soil Character Index, I_c, is greater than 2.6. The underlying gravel is also considered to be non-liquefiable.

Groundwater

Based upon measured groundwater levels, and accounting for likely seasonal variation, we have adopted a groundwater level of 1m below ground level. Therefore, soils are potentially liquefiable below 1m depth from a saturation criterion. It should be noted that groundwater levels are subject to seasonal changes.

Earthquake Intensity and Level of Shaking

The level of ground shaking is one of the key factors in determining whether liquefaction will or will not occur. For this study, we have assessed three design levels of shaking. We understand that the building is likely to be classified as an Importance Level 4 (IL4) structure in accordance with Table 3.2 of the New Zealand structural loadings standard (NZS 1170.0.2004) and the building will have a nominal 50 year design life. To determining the design level so earthquake shaking we have adopted the MBIE/NZGS (2016) recommendations. For our analysis we have also undertaken a back analysis of both the Darfield and Christchurch Earthquakes at the site. The back analysis and design level earthquake events as follows:

- Darfield Earthquake M_w7.1 with 0.21g PGA
- Christchurch Earthquake M_w6.2 with 0.23g PGA
- SLS-1 – 1 in 25 year earthquake M_w6.0 with 0.19g PGA
- SLS-2 - 1 in 500 year earthquake M_w7.5 with 0.35g PGA
- ULS – 1 in 2,500 year earthquake Mw6.2 with 0.47g PGA

4.4.3 Liquefaction Assessment

Methodology

The ability of subsoils to resist the effect of ground shaking associated with the design level earthquakes has been assessed from the subsoil information obtained from the CPTs and boreholes. Liquefaction can have a number of effects on buildings and land and in our assessment we have considered the following effects:

- Liquefiable layers
- Liquefaction induced reconsolidation settlement
- Liquefaction induced ground damage

The liquefaction assessments have been carried out using the references in Table 6 below. Due to the lower test resolution of borehole and the distance of the NZGD boreholes from the site, the preliminary liquefaction assessment has only been based upon the two CPT logs from the tests to the west of the site.:

Table 6 Liquefaction Assessment Methodology Summary

Test	Liquefaction Assessment Methodology	Fine Content	Liquefaction Cut Off	Liquefaction Settlement Method	Liquefaction Ground Damage Method
CPT	Boulanger and Idriss (2014) with a 15% probability of liquefaction	Based on I_c with C_{fc}=0.2	Based on a 2.6 I_c cut off	Zhang et al (2002)	Ishihara (1985) and Tonkin & Taylor (2013)

Liquefaction Results

The results of the liquefaction assessment are summarised in Table 7 and the results are presented in Appendix D.

Table 7 Liquefaction Hazard Assessment Summary

Earthquake Event	Earthquake Effects	Results
Darfield EQ (M_w7.1, 0.21g)	Potentially Liquefiable Layers(1)	Some softening and minor liquefaction in sandy material in Geotechnical Units 1, 3 and 4
	Settlement(2)	105mm to 110mm / 50mm to 65mm
	Ground Damage	Minor expression of liquefaction is possible with some sand boils is calculated.
	Comments	No surface expression observed and up to 100mm of settlement recorded with LiDAR. Analysis may be over predicting surface expression.
Christchurch EQ (M_w6.2, 0.23g)	Potentially Liquefiable Layers(1)	Some softening and minor liquefaction in sandy material in Geotechnical Units 1, 3 and 4
	Settlement(2)	95mm to 105mm / 50mm to 65mm
	Ground Damage	Minor expression of liquefaction is possible with some sand boils is calculated.
	Comments	No surface expression observed, so analysis may be over estimating surface expression of liquefaction.
SLS-1 EQ (M_w6.0, 0.19g 1/25 year)	Potentially Liquefiable Layers(1)	Some minor softening in sandy material in Geotechnical Units 1, 3 and 4
	Settlement(2)	65mm to 75mm / 35mm to 45mm
	Ground Damage	Minor expression of liquefaction is possible with some sand boils is calculated
	Comments	The extent of liquefaction and associated ground movement is expected to be less than that which occurred during either the Darfield or Christchurch Earthquakes.
SLS-2 EQ (M_w7.5, 0.35g 1/500 year)	Potentially Liquefiable Layers(1)	Liquefaction throughout the sandy material in Geotechnical Units 1, 3 and 4
	Settlement(2)	135mm to 150mm / 75mm
	Ground Damage	Moderate to severe expression of liquefaction, settlement can cause structural damage
	Comments	Liquefaction triggering and associated ground damage expected to be greater than which occurred during the Darfield or Christchurch Earthquakes.
ULS EQ (M_w6.2, 0.48g 1/2,500 year	Potentially Liquefiable Layers(1)	Liquefaction throughout the sandy material in Geotechnical Units 1, 3 and 4
	Settlement(2)	135mm to 150mm / 75mm to 80mm
	Ground Damage	Moderate to severe expression of liquefaction, settlement can cause structural damage
	Comments	Liquefaction triggering and associated ground damage expected to be greater than which occurred during the Darfield or Christchurch Earthquakes and slightly larger than that the SLS-2 design earthquake event.

(1) Due to the inherent uncertainty in calculating liquefiable layers, the calculated layers are indicative only. Actual positions and thickness of liquefiable layers could vary from those above.

(2) Settlements are calculated from the full CPT profiles / the upper 10m of the soil profile. Settlements are presented to the nearest 5mm. Due to the inherent uncertainty in calculating liquefaction induced settlements, the calculated settlements are indicative only and actual settlements will vary from those above.

4.4.4 Lateral Spreading

Lateral spreading occurs in the surface soils move downslope or towards a free edge, such as a river or basin. Lateral spreading can occur during an earthquake under seismic loading and following the earthquake until the excess pore water pressure caused by ground shaking dissipate and the soil regains strength.

When assessing liquefaction induced lateral spreading we considered the following:

- There are no streams, rivers or significant changes in height in close proximity to the site
- The site and surrounding area is relatively level
- No lateral spreading damage was observed or recorded at or around the site after any major earthquake in the 2010 to 2011 Canterbury Earthquake Sequence

Based on the flat site topography with no obvious ‘free edges’, and the lack of observed damage, we consider that the global lateral and lateral stretch potentials across the site are low and will not govern the building design. As such no further assessment of lateral spreading has been undertaken.

4.5 Foundation Options

4.5.1 General

Based upon the available geotechnical information we consider the key geotechnical risks for the site, in particular the building structures, is ground damage and movement caused by:

- The presence of weak and potentially compressible silty-clayey soils under long term static and short term seismic loads.
- Liquefiable silty-sandy soils in the upper 18m of the subsoil profile.
- Buoyancy forces on the basement and fuel tanks due to static/flood groundwater and liquefied soils during an earthquake.

Due to the shallow groundwater table, and the size and depth of the car parking basement, under long term static conditions the basement structure will want to float due to hydrostatic buoyancy forces. Therefore, some form of tie down anchors will be required to prevent floatation. Under static conditions with tiedown anchors shallow foundation would be a viable option. However, under a moderate to major earthquake event, e.g. SLS-2 loading, shallow foundations are unlikely to be viable due to the expected ground deformations, the loss of soil strength, and surface expression of liquefaction. Additionally, due to liquefaction within the upper 4m of the subsoil profile the buoyancy forces acting on the basement will increase significantly.

Significant liquefaction induced building damage, not dissimilar to that from a ULS event, is likely during an earthquake with a return period much less than a ULS level event, i.e. less than even a SLS-2 earthquake event. Therefore, in order to meet the level of resilience an IL4 structure requires a significantly more robust foundation system that addresses both liquefaction and buoyancy issues. As such, we consider either deep piles or some form of liquefaction mitigation in combination with shallow footings and tie down anchors will be required as part of the site development. Mitigation options are detailed below.

4.5.2 Liquefaction Mitigation

It is considered that the site in its current assessed state is susceptible to seismically induced liquefaction in a future moderate to major seismic event. In terms of liquefaction hazard mitigation there are four basic approaches as follows:

Accept the Liquefaction Risk

Essentially design a structure with no regards to the liquefaction risk. Following a moderate to major seismic event the structures on this site design expected to perform badly, be excessively damaged, and/or lose significant amenity. This solution would leave Foodstuffs with limited post-earthquake resilience as significant damage could be expected even at medium shaking levels and this option does not achieve the resilience requirements of an IL4 structure. As discussed with the Client, a 'Do Nothing' approach is not recommended and has not been considered as it does not meet the Client's business objectives.

Building Strengthening

Structurally design the building to accommodate the effects of basement buoyancy and seismically induced liquefaction. Examples of this include using piled foundations founded in non-liquefiable soil layers. Building strengthening does not remove the liquefaction hazard but reinforces the structure in such a way that it maintains stability during a liquefaction event.

Ground Improvement

Improve the soil at the site so that it is less susceptible to seismically induced liquefaction. This general approach can be divided into three categories:

1. Densify the soil so that soil grain skeleton will not collapse under earthquake loading. Examples of this include stone columns, vibro-floatation, dynamic compaction, and cut and replacement (refilling with material which will not liquefy).
2. Soil reinforcement. Examples include stone columns, driven piles to densify and stiffen the soil, deep soil mixing, soil cement columns etc.
3. Allow dissipation of excess pore water pressure so that liquefaction hazard is reduced. Examples of this include installation of drains, drainage blankets, and/or stone columns. Drainage limits the liquefaction potential but significant settlement, both total and differential, can still occur.

Alternative Land Use

Move to another less susceptible site. At this stage we do not believe this to be a practicable solution.

4.5.3 Preferred Foundation Options

We have assessed the potential foundation options for the site when accounting for the identified geotechnical risks and likely construction issues. Based upon our current understanding of the site we consider two potential resilient foundations options for the site:

- Deep steel screw piles with a fully suspended basement floor slab.
- Ground Improvement with shallow footings and tie down anchors

When comparing the two foundation options, the ground improvement option is considered the least practicable as the ground improvement will address the seismic ground performance but will not address the basement floatation issues. Due to the ground conditions, the shallow groundwater level, the relatively lightweight structural form of the building, and the extent of the basement, tension anchors will likely need to be founded into the underlying 'Riccarton Gravel' layer from 18m depth onwards. Depending on the anchor spacing, anchors will have significant uplift loads on them particularly during an earthquake when buoyancy forces from liquefied soil are in the order of 80% greater than static conditions. The anchors are likely to be of similar size and form as the 'pile' foundation option but will still require the ground improvement component. This approach also has the potential for significant technical issues associated with strain compatibility between the anchors and surrounding ground following a liquefaction inducing earthquake and the ground settlement it causes. Additionally, depending upon the method of ground improvement there exists the risk that dewatering and construction of the basement will be made harder by the presence of highly permeable ground improvement elements, e.g. deep artesian groundwater pressures flowing up the stone columns, or similar.

Therefore, we consider steel screw piles founded into the 'Riccarton Gravels' to be geotechnically the preferred solution at this stage, as they can deal with static floatation, seismic structural actions, and carry geotechnical loads under normal operational conditions. It is considered the most resilient foundation option for the project.

4.5.4 Concept Pile Design

Based upon the current level of geotechnical knowledge, the preferred foundation system has been identified as steel screw piles founded into the Riccarton Gravels. This foundation system would involve supporting the building on deep steel screw piles founded in the lower Riccarton Gravel layer, with the pile founded at a nominal 23m depth (approximately 5m or so into the 'Riccarton Gravels') depending upon detailed ground investigation, final pile loads and the magnitude tension loads are required to be resisted. As the piles will not suppress the effects of liquefaction, the basement must be designed to be fully suspended to minimise damage.

Due to the specialist nature of screw pile design, where installation requirements and technique are important factors in achieving pile performance we recommend that a specialist screw pile contractor is engaged as soon as possible in the design process to undertake the detailed design of screw piles in conjunction with the project structural engineer using the geotechnical design parameters. Concept design level geotechnical parameters and comments are presented below.

In addition to axial loading the piles will need to be designed to account for lateral loading from both building inertia base shear and kinematic soil drift. These load cases are discussed below.

Lateral Loading

For the preliminary assessment of resisting lateral loading the foundations system will need to resist both inertia building loading (building base shear) and kinematic ground movement (ground lurch) as follows:

- Due to the size and depth of the car parking basement, Inertia building loading should readily be resisted by the basement walls acting in passive loading. Retaining wall design parameters to resist these loads can be addressed at the detailed design stage of the project.
- As per the recommendations of the Royal Commission of Enquiry into the Canterbury Earthquakes, the base friction component of the pile supported basement floor slab should not be used to resist base shear due to the presence of liquefiable soils and the likelihood of ground settlement leaving a void under the floor slab.
- To account for kinematic ground movement the piles should be designed to accommodate a soil drift of 50mm, 150mm and 200mm each way over the approximately 18m length of pile shaft during a SLS-1, SLS-2 and ULS design earthquake, respectively.
- In terms of assessing kinematic ground lurch and building base shear inertia load combinations we recommend using the method detailed in Tokamastu and Asaka (1998). This method recommends the following combinations of inertia and kinematic loads based upon the relationship between the site (T_g) and building (T_b) periods:
  - If $T_b < T_g$ inertial and kinematic forces tend to be in phase and both effects should be considered as the same time.
  - If $T_b \approx T_g$ inertia and kinematic forces tend to be out of phase by 90˚ and thus each effect may be considered separately.
  - If $T_b > T_g$ inertia forces decrease and thus only kinematic effects may be considered.
- Based upon on site stratigraphy, inferred depth to basement rock, the seismograph records of the major event of the Canterbury Earthquake sequence, the Christchurch area the natural period the soil column (T_g) is in the order of 3s.
- Considering the size and form of the proposed supermarket building the building period (T_b) is likely to be significantly less than the ground period (T_g), therefore both kinematic (ground lurch) and Inertia loading (base shear) effects should be considered both acting in phase together for pile design.

Vertical Loading

The piles should be designed to accommodate the effects of liquefaction induced soil down drag. For preliminary design purposes this should include:

- SLS-1 EQ – Up to 75mm of liquefaction induced soil down-drag along the upper 10m of the pile shaft
- SLS-2 EQ – Up to 150mm of liquefaction induced soil down-drag along the upper 18m of the pile shaft.
- ULS EQ - Up to 150mm of liquefaction induced soil down-drag along the upper 18m of the pile shaft.

Pile Capacity

In terms of determining a preliminary pile capacity we have spoken to Piletech, a specialist screw pile design-build contractor with significant experience working in Christchurch. Piletech have indicated that they anticipate an individual screw pile founded into the Riccarton Gravel in northern Christchurch would have a Rupture Bearing Capacity of 3,300kN to 4,200kN, with a tension capacity approximately 70% of these values. ULS bearing capacities would be approximately 60% of the Rupture Capacity and SLS bearing capacities 45% of the Rupture Capacity.

At this stage based upon experience with steel screw piles at nearby sites Piletech have indicated piles would likely be sized to target the following pile head deflections:

- 10-15mm pile top displacement at SLS compression or tension
- 30-40mm pile top displacement at ULS compression or tension

The suggested preliminary vertical stiffness for preliminary design are as follows:

- Compression – approximately 20 to 50kN/mm in compression. The lower range is applicable to more lightly loaded piles (<750kN) and the higher stiffness is considered appropriate for more heavily loaded piles (>2000kN).
- Tension - approximately 10 to 40kN/mm. The lower stiffness is applicable to more lightly loaded piles (<500kN) and higher stiffness appropriate for more heavily loaded piles (>1500kN).

Pile Load Test

During the costing and detailed design process consideration should be made to undertaking a pile load test on site. The pile load test would provide more confidence in the assumed pile capacity and deflection behaviour. As such a higher strength reduction factor could potentially be adopted.

In determining the potential benefit of a pile load test, consideration will need to be given to what governs the overall foundation design, the floor slab capacity to span between piles and to resist the uplift buoyancy pressure, or pile axial capacity to carry compression and tension loads. Therefore, despite being potentially being able to get more usable axial capacity in design by using a higher ULS strength reduction factor there may not be a significant saving in terms of pile numbers or sizes to warrant the additional cost and time associated with the pile load testing.

Underfloor Services

The proposed pile foundation system will not suppress liquefaction (or associated ground settlement) but will transfer structural loads to competent deep soil layers. Therefore, in order to provide the level of resilience an IL4 structure requires that no services are routed below the basement slab. Underfloor services from the market floor should be hung off the underside of the ground floor slab in the ceiling of the car parking basement.

We recommend that flexible connections are used where the services exit the structural system. These flexible connections should be designed to accommodate the expected ground movements (both vertical settlement and horizontal ground lurch) detailed above, so that they can be readily repairable following a major seismic event.

4.5.5 Basement

Lateral Wall Loading

The basement will need to be deigned to withstand loading from:

- Hydrostatic uplift from groundwater.
- Hydrostatic uplift from liquefied soils in the upper 4m of the soil profile.
- Earthquake loading from active pressures pushing in on the building, and passive pressures pushing outwards that are generated resisting building base shear loading.
- External loadings from vehicle traffic etc. under static load conditions.

These load combination and magnitudes of loading should be assessed at the detailed building design stage of the project.

Basement Floor Slab

Due to the likelihood of post-earthquake liquefaction induced reconsolidation settlement (potentially in the in the order of 150mm following a moderate sized earthquake) it is recommended that the floor slab is designed to be fully suspended between pile/ground beam elements. When designing floors consideration should also be given to the potential uplift pressure from groundwater and liquefied soils.

Basement Waterproofing

Due to the likelihood of post-earthquake liquefaction induced reconsolidation settlement all basement tanking will need to be designed to accommodate the anticipated extent of ground movement (approximately 150mm of ground settlement following a major earthquake) without effecting the water proofing capacity of the tanking.

4.5.6 Artesian Bores

We understand that Foodstuffs intend to reuse the existing artesian wells that are currently on site and that these wells are located in what will be the footprint of the new PAK’n SAVE. Therefore, the well shafts will need to penetrate up through the basement.

As part of the detailed assessment of the wells and detailed design we recommend:

- The steel pipe forming the well shaft is assessed to confirm if it can withstand the expected ground lurch values presented in the *Lateral Loading* section above and the expected ground settlement and down drag loading presented in *Vertical* Loading section above.
- Where the well physically penetrates through the basement slab it will need to be detailed with flexible connections with suitable tanking to accommodate the expected vertical ground movements.
- Appropriate emergency procedures are in place to cap the well etc. if damaged after a major earthquake event so that it does not flood the basement.

4.5.7 Fuel Facility and Fuel Tanks

Due to the geotechnical conditions at the site we consider that there is a risk of tank floatation from both hydrostatic ground water pressures and liquefied soil pressures following a major earthquake event. The tanks will need to be detailed in such a way to withstand these uplift pressures using tiedown anchors/piles to prevent floatation and possibly tank strengthening to withstand the increased buoyancy pressures, particularly when the tank is empty.

During detailed design consideration will need to be given foundations capacity of the fuel facility canopies etc. with particular regard to bearing capacity under liquefied ground conditions

Depending upon the finalised detailing flexible connections and underground piping will be needed to withstand the expected liquefaction induced ground movement and settlement.

4.5.8 External Tanks

As part of the resilience requirements for the new PAK'n SAVE several large potable and wastewater tanks will be required. During the detailed design stage once tank sizes and locations are confirmed we recommend the following is checked:

- Bearing capacity under liquefied ground conditions. It is likely that a geogrid reinforced raft, say 600mm thick and extending 1m beyond the tank perimeter, will be required under all tanks to prevent a shallow foundation failure.
- Flexible piping/couplings will be required with sufficient head built into the piping network to accommodate differential ground movement between the piled building and the non-piled tanks.

4.5.9 Basement Excavation

The geotechnical aspects associated with the car parking basement excavation include providing temporary support for the sides of the excavation during basement construction and dewatering of the ground to allow the basement floor and walls to be constructed considering:

- The depth and size of the basement excavation (approximately 4m deep with an area of approximately 5,000m^2)
- The shallow groundwater level
- Proximity to neighbours (in particular along the northern boundary)

It is considered not appropriate for dewatering with unsupported slopes, therefore the excavation will need to be temporarily supported. Dewatering options are discussed below.

Sheet Piling

Considering the area of the basement excavation, and our understanding of the ground conditions at this stage, sheet piles are considered to be the likely most appropriate form of shoring when accounting for technical performance, and likely dewatering requirements and construction cost.

Sheet piling may require driven installation, which may cause significant vibration issues, especially when working close to adjacent buildings depending upon the final depth of excavation, the length of sheet piles, and final ground conditions encountered, the sheet piles may need propping or anchors. Sheet piles would be removed once the basement has been completed.

The toe of the sheet pile wall would be keyed into the softer silt material between 8m and 12m depth to control groundwater inflows. Depending upon the basement construction methodology anchors may be needed to support the sheet pile wall. If anchors are to be considered these may need to cross legal property boundaries and which could require permission from the neighbours. Instead of sacrificial anchors, screw piles could be used and recovered at the end of the construction.

Dewatering

Due to the size of the basement excavation, the high groundwater table, and the physically constrained site careful consideration will need to be given to developing appropriate dewatering and groundwater disposal methodologies. Groundwater control will need to be done in conjunction with the contractor and the project design team.

It is noted that the soft silty soils (Geotechnical Unit 2) that underlay the site are considered potentially vulnerable to consolidation/settlements induced by changes in groundwater level/pressures associated with dewatering activities. This settlement potential will need to be checked as part of the detailed geotechnical site investigation, and during the sheet piling and dewatering design.

4.6 Recommendations

4.6.1 Additional Geotechnical Testing

As part of the detailed design of the building, additional geotechnical testing is required. As a minimum this investigation should include the following testing:

- A nominal six exploratory boreholes to a minimum of 30m depth with SPT testing at 1.5m centres. The information would be used for an updated liquefaction assessment and for use in the detailed design of the screw piles.
- Installation of four stand pipe piezometers in two of the boreholes to measure both the shallow groundwater table and the potential artesian pressures in the 'Riccarton Gravels (i.e. two piezometers per borehole).'
- Cone Penetrometer Testing across the site to help better define the liquefaction risk and general ground stratigraphy.
- Laboratory testing (fines content, Atterberg Limits etc.) from soil samples help refine the liquefaction hazard assessment and the settlement potential of the soft silty soils to dewatering induced consolidation effects.
- Shear Wave Velocity profiling across the site for use in assessing the site period and NZS1170.5:2004 subsoil classification assessment.
- Potentially groundwater drawdown testing for dewatering design.

4.6.2 Environmental Review

We recommend undertaking a review of any detailed environmental site assessment to determine what effect, if any, this may have on the geotechnical component of the detailed design.

4.6.3 Early Contractor Engagement

Screw Piles

Due to the specialist nature of screw pile design, and considering to the complexities of pile design and the feedback loops that pile design can have on superstructure and floor slab design, we recommend that a specialist screw pile contractor is engaged early in the design process to undertake detailed screw pile design. This design would be undertaken in conjunction with the project structural engineer using the geotechnical design parameters presented above and confirmed following the additional geotechnical testing recommended above.

Basement Excavation

Due to the potential technical difficulties in excavating a large basement in the confined urban setting of the site, and the associated disposal of extracted groundwater, we recommend early engagement with potential specialist earthworks contractors to develop an appropriate dewatering methodology. The objective of early involvement is to identify potential problems which could affect cost, programme, construction sequencing and impact on design.

4.7 Safety in Design

Safety in design is an important consideration during design and construction of the new house. The geotechnical hazards that will need to be considered are uncertain at this stage of the project but could include:

- Inground geotechnical testing and soil sampling process
- Bulk earthworks and site preparation
- Open excavation and stability of excavations for the car parking basement
- Steel screw pile installation
- Confined spaces for people working behind retaining/basement walls say installing drainage or water proofing
- Falls into excavations

A detailed hazard assessment will be required as part of final design.

5 References

Boulanger and Idriss, 2014. *CPT and SPT Based Liquefaction Triggering Procedures*. Department of Civil and Environmental Engineering. Report No. UCD/CGM-14/01

Bradley and Hughes, 2012. Conditional Peak Ground Accelerations in the Canterbury Earthquakes for Conventional Liquefaction Assessment Part 1 and 2. Technical Report for the Ministry of Business, Innovation and Employment.

Brown and Weeber, 1992. *Geology of the Christchurch urban area.* Institute of Geological and Nuclear Sciences, 1:25,000 Geological Map 1. 1 sheet + 104p. Lower Hutt, New Zealand.

ECan, 2018. Retrieved 23 May 2018 from https://mapviewer.canterburymaps.govt.nz/

Idriss and Boulanger, 2008. *Soil Liquefaction During Earthquakes. EERI monograph MNO12.*

Institute of Geological and Nuclear Sciences (GNS, 2018). Retrieved 23 May 2018 from http://www.gns.cri.nz/Home/Our-Science/Natural-Hazards/Recent-Events/Canterbury-quake/Recent-aftershock-map)

Ishihara, 1985. *Stability of Natural Deposits During Earthquakes*.

MBIE, 2012. *Revised issue of Repairing and rebuilding houses affected by the Canterbury earthquakes.* December 2012. Ministry of Business, Innovation and Employment, Wellington, New Zealand.

MBIE/NZGS. 2016. Earthquake Geotechnical Engineering Practice – Module 1: Overview of the Guidelines. Revision 0, dated March 2016.

New Zealand Geotechnical Database (NZGD), 2018. Retrieved 23 May 2018 from https://www.nzgd.org.nz

NZS1170.0:2004. *Australia/New Zealand Standard, Structural Design Actions, Part 0: General Principles*. Standards New Zealand, Wellington, New Zealand. Wellington, New Zealand.

NZS1170.5:2002. *Australia/New Zealand Standard, Structural Design Actions, Part 5: Earthquake Actions – New Zealand*. Standards New Zealand, Wellington, New Zealand.

Tokimatus and Asaka, 1998. *Effects of Liquefaction-Induced Ground Displacement on Pile Performance in the 1995 Hyogoken-Nambu Earthquake.* Japanese Geotechnical Society Special Issue of Soils and Foundations, pp.163-177, September 1998.

Tonkin & Taylor Ltd, 2013. *Liquefaction Vulnerability Study*, February 2013. Christchurch, New Zealand.

Zhang, Robertson and Brachman, 2002. *Estimating Liquefaction-Induced Ground Settlements from CPT for Level Ground*. NRC Research Press

6 Limitations

We have prepared this report in accordance with the brief as provided. The contents of the report are for the sole use of the Client and no responsibility or liability will be accepted to any third party. Data or opinions contained within the report may not be used in other contexts or for any other purposes without our prior review and agreement.

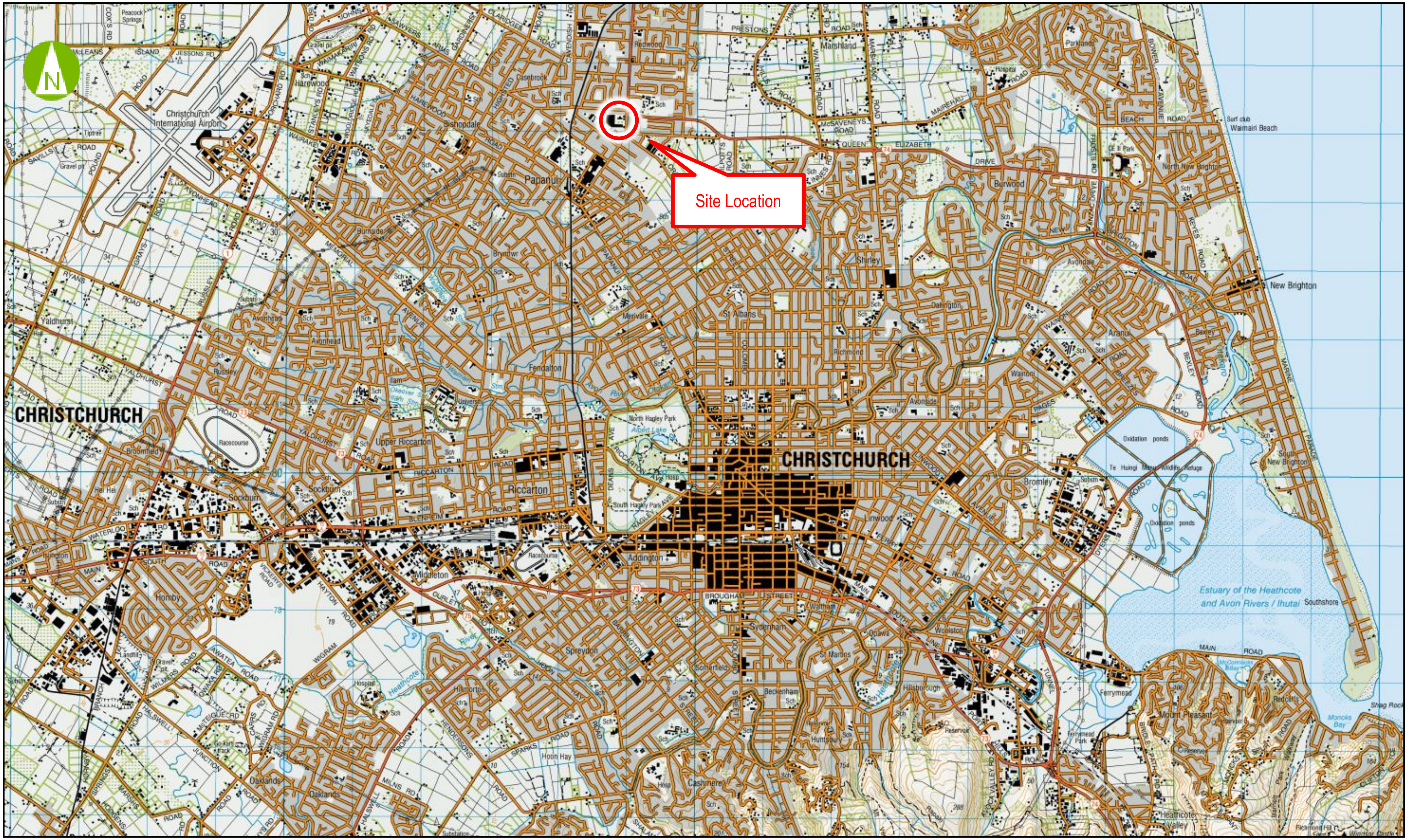
The recommendations in this report are based on data collected at specific locations and by using appropriate investigation methods with limited site coverage. Only a finite amount of information has been collected to meet the specific financial and technical requirements of the Client's brief and this report does not purport to completely describe all the site characteristics and properties. The nature and continuity of the ground between test locations has been inferred using experience and judgment and it must be appreciated that actual conditions could vary from the assumed model.

Subsurface conditions relevant to construction works should be assessed by contractors who can make their own interpretation of the factual data provided. They should perform any additional tests as necessary for their own purposes.

Subsurface conditions, such as groundwater levels, can change over time. This should be borne in mind, particularly if the report is used after a protracted delay.

This report is not to be reproduced either wholly or in part without our prior written permission.

Appendix A
Figures



CLIENT	PRELIMINARY NOT FOR CONSTRUCTION	ALL DIMENSIONS APPROXIMATE ONLY	SCALE	SIZE	TITLE	SITE LOCATION
aurecon www.aurecongroup.com FOODSTUFFS South Island Limited	FIGURE	FIGURE 1	NTS	A4		
			BY: D. MAHONEY		REFERENCE	Images sourced from: LINZ Topographic Maps. Crown Copyright Reserved
	PROJECT	PAK'N SAVE PAPANUI	APPROVED: J. KUPEC			
			DATE: 30 MAY 2018		FIGURE No.	PROJECT 243354 · WBS 010 · TYPE FIG · DISC INF · NUMBER 01 · REV A

aurecon
www.aurecongroup.com

CLIENT	FOODSTUFFS South Island Limited	PRELIMINARY NOT FOR CONSTRUCTION	ALL DIMENSIONS APPROXIMATE ONLY
FIGURE	FIGURE 2		
PROJECT	PAK'N SAVE PAPANUI		

SCALE	SIZE
NTS	A4

BY	APPROVED	DATE
D. MAHONEY	J. KUPEC	30 MAY 2018

TITLE	EXISTING GEOTECHNICAL TEST LOCATIONS
REFERENCE	Background image from: New Zealand Geotechnical Database

FIGURE No.	PROJECT	WBS	TYPE	DISC	NUMBER	REV
	243354	010	FIG	INF	02	A

Appendix B
ECan Borehole Logs

Borelog for well M35/1348

Gridref: M35:7885-4678 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 10.1 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Cable Tool
Drill Depth : -24.4m Drill Date : 26/01/1945

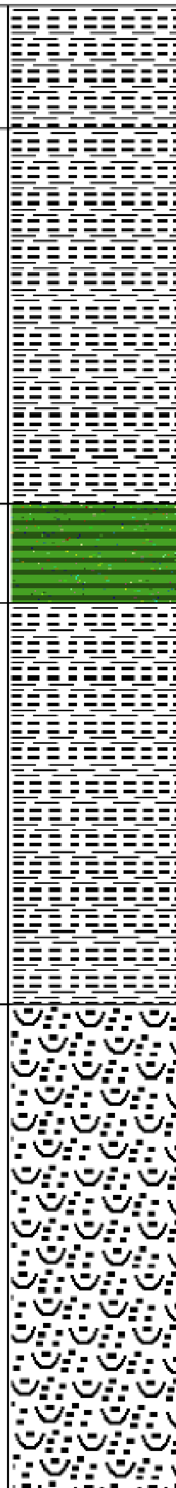
Water Level: -0.4CalcMin

Depth(m)	Full Drillers Description	Formation Code
- 10.7m	Sand & clay	sp?
- 11.6m	Blue clay	sp?
- 15.8m	Blue sand & gravel	sp?
- 18.0m	Clay & peat	ch
- 24.4m	Brown shingle	ri

Scale(m): -5, -10, -15, -20

Borelog for well M35/1472

Gridref: M35:78851-46821 Accuracy : 2 (1=high, 5=low)
Ground Level Altitude : 10.2 +MSD
Driller : not known
Drill Method : Unknown
Drill Depth : -18.29m Drill Date :



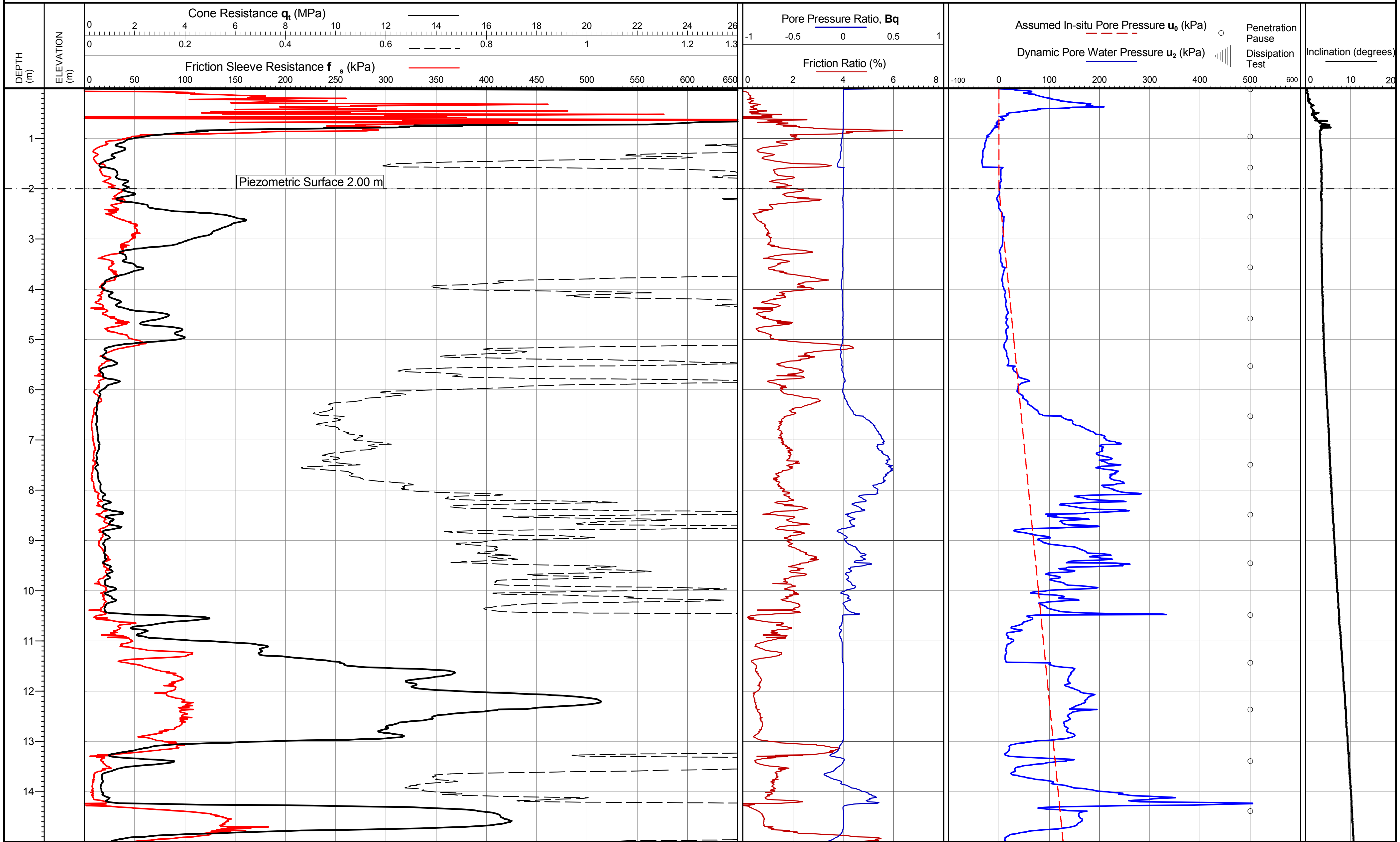
Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
	-0.4CalcMin	-1.50m	Clay silt	sp?
-5		-6.09m	Lense of clay silt,silt & organic matter	sp?
		-7.30m	Organic clay & peat	sp?
-10		- 12.2m	Silty clay	sp?
-15		- 18.3m	Sand,shells @ 17m	ch

Appendix C
NZGD Logs

Client: KGA

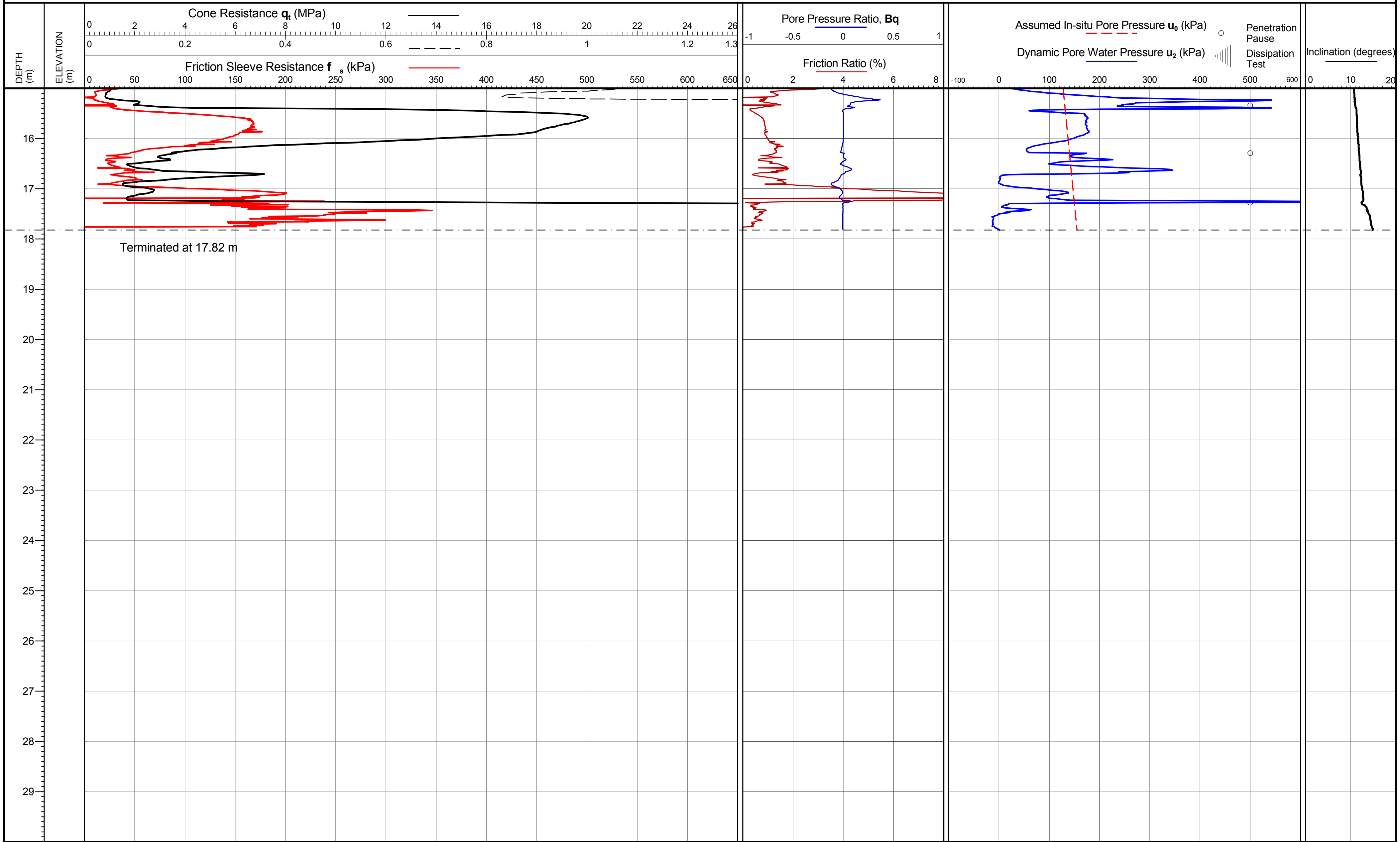
Project: 155 MAIN NORTH RD

LANKELMA



Cone area (mm2):1500 Cone ID: S15CFIIP.803 Operator: Javan Cassidy Rig Used: NZ1 Date of test: 06/07/2015 14:22:10	Zero drift (Pre/post test) q_c (MPa): -0.0183 f_s (kPa): 0.2 U_2 (kPa): -1.5	Location: Christchurch Coordinates: , Elevation: Coordinate system:	Remarks: *Piezometric surface origin: Est. from u2 piezo data Termination Remark: Tip load	Date of plot: 07-07-15 Checked by: Emma Stickland	Lankelma Project Ref: PNZ2397	**TEST ID: CPT 15203-A** Page 1 of 2

Client: KGA **Project: 155 MAIN NORTH RD**

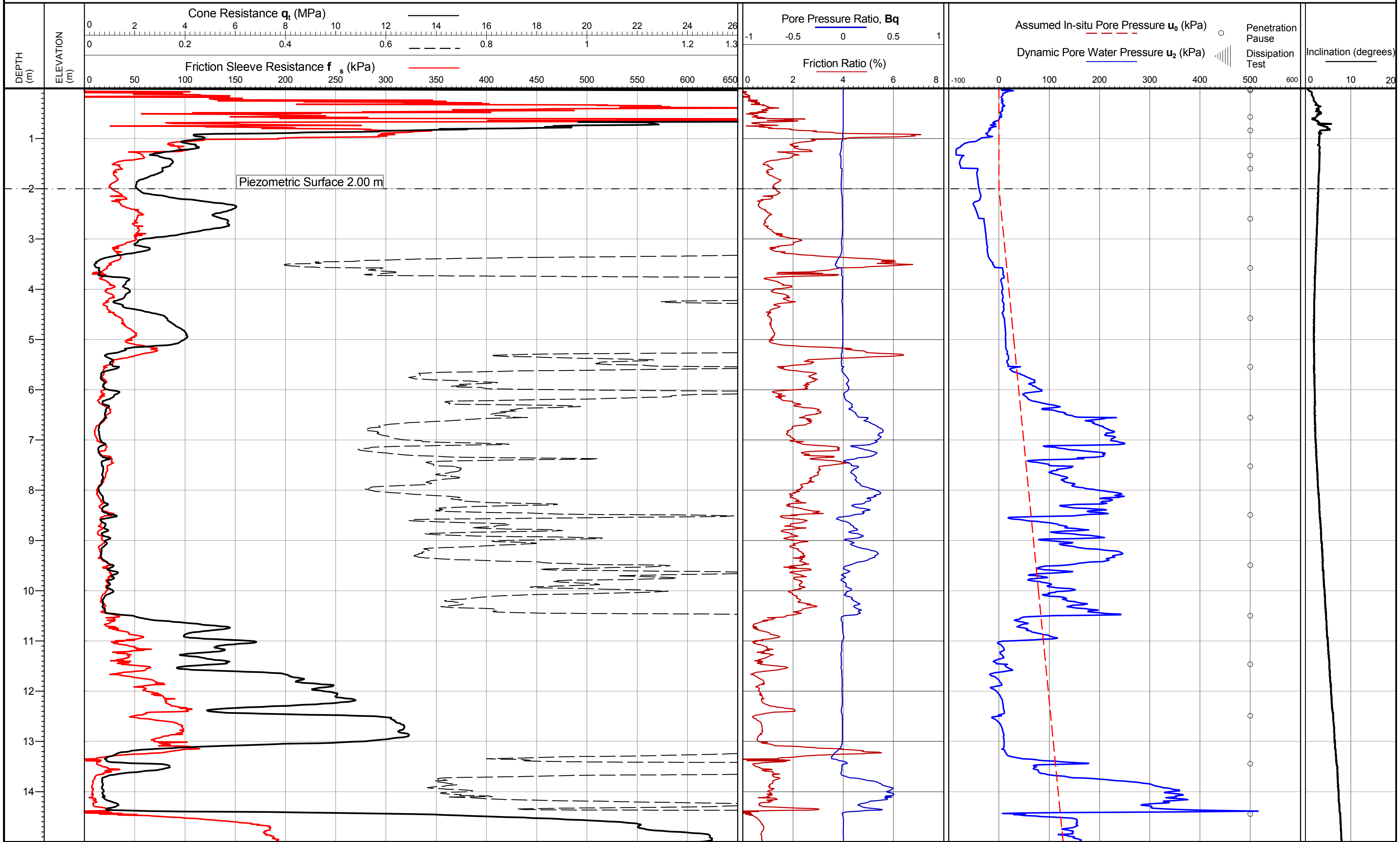


Cone area (mm2):1500 Cone ID: S15CFIIP.803 Operator: Javan Cassidy Rig Used: NZ1 Date of test: 06/07/2015 14:22:10	Zero drift (Pre/post test) q_c (MPa): -0.0183 f_s (kPa): 0.2 U_2 (kPa): -1.5	Location: Christchurch Coordinates: , Elevation: Coordinate system:	Remarks: *Piezometric surface origin: Est. from u2 piezo data Termination Remark: Tip load	Date of plot: 07-07-15 Checked by: Emma Stickland	Lankelma Project Ref: PNZ2397	**TEST ID: CPT 15203-A** Page 2 of 2

Client: KGA

Project: 155 MAIN NORTH RD

LANKELMA



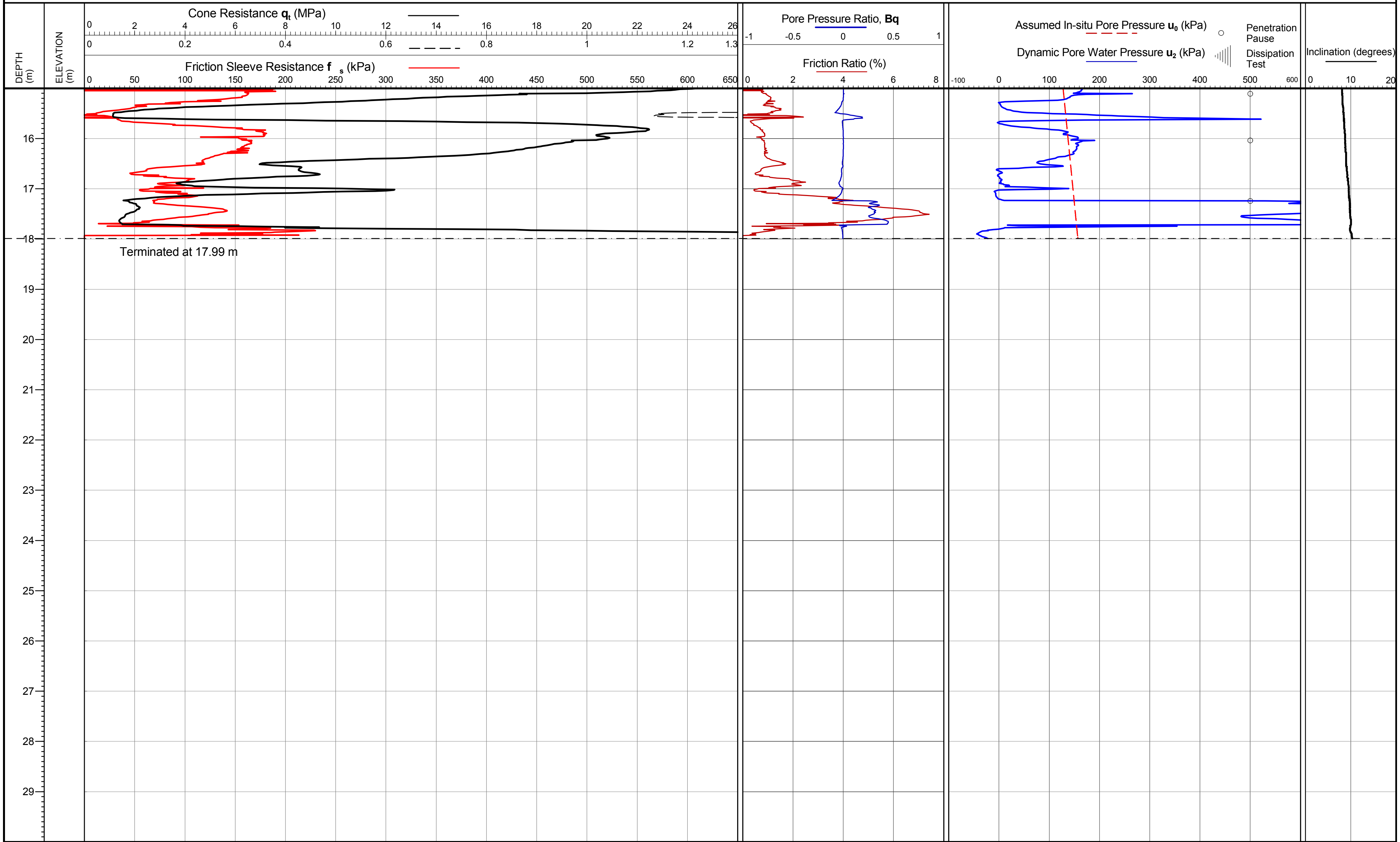
Cone area (mm2):1500 Cone ID: S15CFIIP.803 Operator: Javan Cassidy Rig Used: NZ1 Date of test: 06/07/2015 13:29:58	Zero drift (Pre/post test) q_c (MPa): 0.0000 f_s (kPa): -0.5 U_2 (kPa): -5.6	Location: Christchurch Coordinates: , Elevation: Coordinate system:	Remarks: *Piezometric surface origin: Est. from u2 piezo data Termination Remark: Tip load	Date of plot: 07-07-15 Checked by: Emma Stickland	Lankelma Project Ref: PNZ2397

TEST ID: CPT 15203-B

Client: KGA

Project: 155 MAIN NORTH RD

LANKELMA



Cone area (mm2):1500 Cone ID: S15CFIIP.803 Operator: Javan Cassidy Rig Used: NZ1 Date of test: 06/07/2015 13:29:58	Zero drift (Pre/post test) q_c (MPa): 0.0000 f_s (kPa): -0.5 U_2 (kPa): -5.6	Location: Christchurch Coordinates: , Elevation: Coordinate system:	Remarks: *Piezometric surface origin: Est. from u2 piezo data Termination Remark: Tip load	Date of plot: 07-07-15 Checked by: Emma Stickland	Lankelma Project Ref: PNZ2397

TEST ID: CPT 15203-B

TONKIN & TAYLOR LTD

BORE CONSTRUCTION LOG

BOREHOLE No: BH01
Instrument:

PROJECT: St Bede's College	LOCATION: 210 Main North Road	JOB No: 53139
CO-ORDINATES 5747121.4 mN 2479097.4 mE	DRILL TYPE:	HOLE STARTED: 22/8/12
	DRILL METHOD: Rotosonic	HOLE FINISHED: 22/8/12
R.L.		DRILLED BY: ProDrill (Ray)
DATUM N/A	COLLAR RL:	LOGGED BY: adw CHECKED:

INTERPRETIVE GEOLOGICAL LOG — USCS DESCRIPTION

OBSERVATION and INTERPRETATION	CASING	WELL GRAPHIC LOG	WATER LEVEL	R.L. (m)	DEPTH (m)	SAMPLES	TEST RESULTS	GRAPHIC LOG	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.)
Pre dig.					0.0				No Recovery.
					0.5				
					1.0				
Yaldhurst Member of the Springston Formation.					1.5		1/0/1/1/1/1 N=4		SILT, brown with iron staining. Wet, firm. Moderate plasticity, non-dilatant.
					2.0				
					2.2				SILT, grey. Wet, very soft. Low plasticity, rapidly dilatant.
					2.5				
					3.0		0/0/0/0/0/0 N=0		
					3.5				
					3.7				- moderately dilatant. Trace organics.
					4.0				
					4.5		0/0/0/0/0/0 N=0		
					5				

TONKIN & TAYLOR LTD

BORE CONSTRUCTION LOG

BOREHOLE No: BH01
Instrument:

PROJECT: St Bede's College	LOCATION: 210 Main North Road	JOB No: 53139
CO-ORDINATES 5747121.4 mN 2479097.4 mE	DRILL TYPE:	HOLE STARTED: 22/8/12
	DRILL METHOD: Rotosonic	HOLE FINISHED: 22/8/12
R.L.		DRILLED BY: ProDrill (Ray)
DATUM N/A	COLLAR RL:	LOGGED BY: adw CHECKED:

INTERPRETIVE GEOLOGICAL LOG — USCS DESCRIPTION

OBSERVATION and INTERPRETATION	CASING	WELL GRAPHIC LOG	WATER LEVEL	R.L. (m)	DEPTH (m)	SAMPLES	TEST RESULTS	GRAPHIC LOG	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.									
					5.5				
									- fibrous organics. 100mm.
									- minor organics. Organic staining.
					6.0				
									- organic staining absent.
							0/0/0/0/0/0 N=0		
					6.5				
					7.0				
									- fibrous organics.
					7.5				
							0/0/0/0/1/0 N=1		
					8.0				
					8.5				
					9.0				- wood fragment.
									Fine to coarse SAND, grey. Wet, medium dense.
							0/0/3/6/5/6 N=20		
					9.5				
									No Recovery.
Christchurch Formation.									Fine to coarse SAND, grey. Wet, medium dense.
					10				

TONKIN & TAYLOR LTD

BORE CONSTRUCTION LOG

BOREHOLE No: BH01
Instrument:

PROJECT: St Bede's College | LOCATION: 210 Main North Road | JOB No: 53139

CO-ORDINATES 5747121.4 mN
2479097.4 mE

R.L.

DATUM N/A

DRILL TYPE:

DRILL METHOD: Rotosonic

COLLAR RL:

HOLE STARTED: 22/8/12

HOLE FINISHED: 22/8/12

DRILLED BY: ProDrill (Ray)

LOGGED BY: adw CHECKED:

INTERPRETIVE GEOLOGICAL LOG | USCS DESCRIPTION

OBSERVATION and INTERPRETATION	CASING	WELL GRAPHIC LOG	WATER LEVEL	R.L. (m)	DEPTH (m)	SAMPLES	TEST RESULTS	GRAPHIC LOG	SOIL DESCRIPTION: Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION: Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.
Christchurch Formation.									- silt lense. 100mm.
					10.5				
					11.0		1/1/2/0/1/0 N=3		
									- some silt. Trace shell fragments.
					11.5				
					12.0				
									SILT, grey. Wet, very soft. Non-plastic, rapidly dilatant.
									No Recovery.
									SILT, grey. Wet, very soft. Non-plastic, rapidly dilatant.
					12.5		0/1/0/0/1/0 N=1		
									- minor sand.
					13.0				
									Fine to medium SAND with trace shell fragments, grey. Wet, medium dense.
					13.5				
									- shells absent.
					14.0		1/1/2/5/7/9 N=23		
					14.5				
					15				

PROJECT: St Bede's College	LOCATION: 210 Main North Road	JOB No: 53139
CO-ORDINATES 5747121.4 mN 2479097.4 mE	DRILL TYPE:	HOLE STARTED: 22/8/12
R.L.	DRILL METHOD: Rotosonic	HOLE FINISHED: 22/8/12
		DRILLED BY: ProDrill (Ray)
DATUM N/A	COLLAR RL:	LOGGED BY: adw CHECKED:

INTERPRETIVE GEOLOGICAL LOG / USCS DESCRIPTION

OBSERVATION and INTERPRETATION	CASING	WELL GRAPHIC LOG	WATER LEVEL	R.L. (m)	DEPTH (m)	SAMPLES	TEST RESULTS	GRAPHIC LOG	SOIL DESCRIPTION: Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION: Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.	
Christchurch Formation.									- sand is fine to coarse.	
					15.5		4/5/8/9 /10/10 N=37			15.5
					16.0					16.0
					16.5				- minor silt.	16.5
									- organic lense. 150mm.	
					17.0		1/2/2/2/3/4 N=11		Fine to medium SAND with trace shells, grey. Wet, medium dense.	17.0
Riccarton Gravel.									Fine to coarse GRAVEL in a sand and silt matrix, grey. Wet, very dense. Gravel is rounded to sub rounded.	
					17.5				- brown.	17.5
					18.0				- matrix absent. Some sand.	18.0
					18.5		3/5/10/10 /11/14 N=45 SOLID			18.5
					19.0					19.0
					19.5					19.5
					20					

TONKIN & TAYLOR LTD

BORE CONSTRUCTION LOG

BOREHOLE No: BH01
Instrument:

PROJECT: St Bede's College	LOCATION: 210 Main North Road	JOB No: 53139
CO-ORDINATES 5747121.4 mN 2479097.4 mE	DRILL TYPE:	HOLE STARTED: 22/8/12
	DRILL METHOD: Rotosonic	HOLE FINISHED: 22/8/12
R.L.		DRILLED BY: ProDrill (Ray)
DATUM N/A	COLLAR RL:	LOGGED BY: adw CHECKED:

INTERPRETIVE GEOLOGICAL LOG									USCS DESCRIPTION
OBSERVATION and INTERPRETATION	CASING	WELL GRAPHIC LOG	WATER LEVEL	R.L. (m)	DEPTH (m)	SAMPLES	TEST RESULTS	GRAPHIC LOG	SOIL DESCRIPTION Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.
Riccarton Gravel.							2/5/9/7/6/7 N=29 SPT effected by artesian water flow		
					20.5				EOH at 20.25 m bgl due to artesian ground water flow. Artesian head estimated to be several meters above ground level.
					21.0				
					21.5				
					22.0				
					22.5				
					23.0				
					23.5				
					24.0				
					24.5				

TONKIN & TAYLOR LTD

BORE CONSTRUCTION LOG

BOREHOLE No: BH03
Instrument:

PROJECT: St Bede's College	LOCATION: 210 Main North Road	JOB No: 53139
CO-ORDINATES 5747025.6 mN 2479277.4 mE	DRILL TYPE:	HOLE STARTED: 22/8/12
R.L.	DRILL METHOD: Rotosonic	HOLE FINISHED: 22/8/12
		DRILLED BY: ProDrill (Ray)
DATUM N/A	COLLAR RL:	LOGGED BY: adw CHECKED:

INTERPRETIVE GEOLOGICAL LOG — USCS DESCRIPTION

OBSERVATION and INTERPRETATION	CASING	WELL GRAPHIC LOG	WATER LEVEL	R.L. (m)	DEPTH (m)	SAMPLES	TEST RESULTS	GRAPHIC LOG	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.
					0.0–1.5				No Recovery.
					1.5–1.95		1/0/1/1/1/2 N=5		SILT with trace sand, brownish grey. Wet, firm. Sand is fine.
					1.95–2.1				No Recovery.
					2.1–2.25				SILT with trace sand, brownish grey. Wet, firm. Sand is fine.
					2.25–2.5				Fine SAND with trace silt, grey. Wet, loose.
					2.5–3.0				SILT with trace organics, grey. Wet, firm. Moderate plasticity, non-dilatent. - organic lense. 100mm.
					3.0–3.15				No Recovery.
					3.15–4.5		0/1/0/1/0/1 N=2		SILT with trace organics, grey. Wet, soft. Moderate plasticity, non-dilatent.
					4.5–4.65				No Recovery.
					4.65–4.95		0/0/0/0/0/0 N=0		SILT, grey. Wet, very soft. Moderate plasticity, non-dilatent.

TONKIN & TAYLOR LTD

BORE CONSTRUCTION LOG

BOREHOLE No: BH03
Instrument:

PROJECT: St Bede's College	LOCATION: 210 Main North Road	JOB No: 53139
CO-ORDINATES 5747025.6 mN 2479277.4 mE	DRILL TYPE:	HOLE STARTED: 22/8/12
	DRILL METHOD: Rotosonic	HOLE FINISHED: 22/8/12
R.L.		DRILLED BY: ProDrill (Ray)
DATUM N/A	COLLAR RL:	LOGGED BY: adw CHECKED:

INTERPRETIVE GEOLOGICAL LOG — USCS DESCRIPTION

OBSERVATION and INTERPRETATION	CASING	WELL GRAPHIC LOG	WATER LEVEL	R.L. (m)	DEPTH (m)	SAMPLES	TEST RESULTS	GRAPHIC LOG	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.
									No Recovery.
									SILT, grey. Wet, very soft. Moderate plasticity, non-dilatent.
									- trace organics.
					5.5				
									- some organics.
					6.0				
									- trace organics.
							0/0/1/0/0/0 N=1		
									- organic staining.
					6.5				
					7.0				
					7.5				
							0/0/0/1/0/0 N=1		
					8.0				
									- organic staining absent.
					8.5				
					9.0				
							0/1/0/1/1/0 N=2		
					9.5				
					10				

TONKIN & TAYLOR LTD

BORE CONSTRUCTION LOG

PROJECT: St Bede's College	LOCATION: 210 Main North Road	JOB No: 53139
CO-ORDINATES 5747025.6 mN 2479277.4 mE	DRILL TYPE:	HOLE STARTED: 22/8/12
	DRILL METHOD: Rotosonic	HOLE FINISHED: 22/8/12
R.L.		DRILLED BY: ProDrill (Ray)
DATUM N/A	COLLAR RL:	LOGGED BY: adw CHECKED:

INTERPRETIVE GEOLOGICAL LOG — USCS DESCRIPTION

OBSERVATION and INTERPRETATION | CASING | WELL GRAPHIC LOG | WATER LEVEL | R.L. (m) | DEPTH (m) | SAMPLES | TEST RESULTS | GRAPHIC LOG

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)	TEST RESULTS	DESCRIPTION
		Fine to medium SAND with trace silt, grey. Wet, medium dense.
		- silt absent. Sand is medium to coarse.
10.5		
11.0	1/1/3/4/7/7 N=21	
11.5		
		- shell fragments.
		SILT with minor shells and minor sand, grey. Wet, soft. Low plasticity, slowly dilatent.
12.0		Fine to medium SAND with some silt, grey. Wet, medium dense.
		- trace shells.
12.5	2/3/3/3/4/4 N=14	
		SILT with some sand and trace shells, grey. Wet, soft. Low plasticity, rapidly dilatent.
13.0		- non-plastic.
13.5		
14.0	1/1/0/0/0/2 N=2	
		Fine SAND with trace silt, grey. Wet, medium dense.
		Fine to medium SAND, light brown. Wet, dense.
14.5		
15		

TONKIN & TAYLOR LTD

BORE CONSTRUCTION LOG

BOREHOLE No: BH03
Instrument:

PROJECT: St Bede's College | LOCATION: 210 Main North Road | JOB No: 53139

CO-ORDINATES 5747025.6 mN
2479277.4 mE
R.L.
DATUM N/A

DRILL TYPE:
DRILL METHOD: Rotosonic
COLLAR RL:

HOLE STARTED: 22/8/12
HOLE FINISHED: 22/8/12
DRILLED BY: ProDrill (Ray)
LOGGED BY: adw CHECKED:

INTERPRETIVE GEOLOGICAL LOG | USCS DESCRIPTION

OBSERVATION and INTERPRETATION | CASING | WELL GRAPHIC LOG | WATER LEVEL | R.L. (m) | DEPTH (m) | SAMPLES | TEST RESULTS | GRAPHIC LOG | 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)	Test results	Description
15.5	3/6/8/9 /10/10 N=37	
16.0		
16.5		
17.0	3/8/10/9 /9/12 N=40	Fine to coarse SAND, grey. Wet, dense.
		Silty fine to coarse SAND, grey. Wet, dense.
17.5		- trace silt, trace organics.
18.0		
		- wood fragment. 50mm.
		- sand is fine to coarse. Silt absent.
18.5	3/3/5 /3/4/4 N=16	- wood fragment. 100mm.
		- wood fragment. 100mm.
		- fibrous organics.
19.0		SILT with trace sand, mottled grey and brown. Wet, stiff. Non-plastic, non-dilatent.
19.5		Fine to coarse GRAVEL in a sand and silt matrix, brown. Wet, very dense.
20		

TONKIN & TAYLOR LTD

BORE CONSTRUCTION LOG

BOREHOLE No: BH03
Instrument:

PROJECT: St Bede's College	LOCATION: 210 Main North Road	JOB No: 53139
CO-ORDINATES 5747025.6 mN 2479277.4 mE	DRILL TYPE:	HOLE STARTED: 22/8/12
	DRILL METHOD: Rotosonic	HOLE FINISHED: 22/8/12
R.L.		DRILLED BY: ProDrill (Ray)
DATUM N/A	COLLAR RL:	LOGGED BY: adw CHECKED:

INTERPRETIVE GEOLOGICAL LOG | USCS DESCRIPTION

OBSERVATION and INTERPRETATION	CASING	WELL GRAPHIC LOG	WATER LEVEL	R.L. (m)	DEPTH (m)	SAMPLES	TEST RESULTS	GRAPHIC LOG	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.
							5/11/14 /14/12/10 for 55mm SOLID N>50		
									EOH at 20.25 m bgl due to artesian ground water flow. Artesian head estimated to be several meters above ground level.
					20.5				
					21.0				
					21.5				
					22.0				
					22.5				
					23.0				
					23.5				
					24.0				
					24.5				

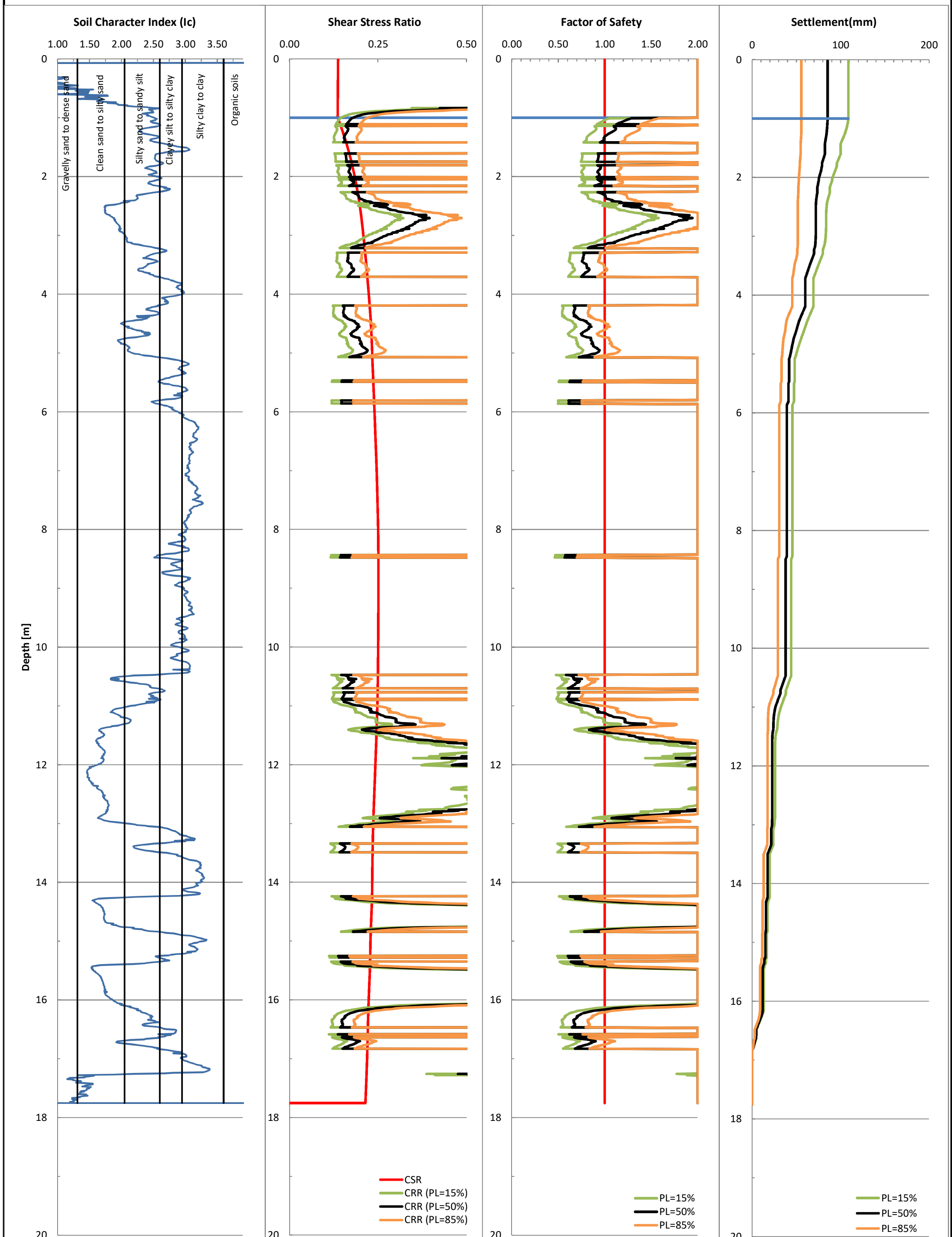
Appendix D
Liquefaction Assessment Outputs

Based upon the methods specified in MBIE Guidelines "Repairing and rebuilding houses affected by the Canterbury earthquakes" Rev 3, Dec 2012) and clarifications and updates to the guidedance Issue 7, October 2014

LIQUEFACTION ANALYSIS

Ic calcuated from Robertson and Cabal (Robertson) (2014). CSR, CRR and FS calcuated From Boulanger and Idriss (2014).
Settlement calcuated from Zhang, Robertson and Brachman (2004).

I_c cut off:	2.60	Water Table [m]	1.00
C_{FC}:	0.20	Magnitude	7.10
		Acceleration [g]	0.21



Indexed' Settlement (PL=15%) [mm]:	65	**Total Settlement (PL=15%) [mm]:**	**109**
Indexed' Settlement (PL=50%) [mm]:	48	**Total Settlement (PL=50%)[mm]:**	**85**
Indexed' Settlement (PL=85%) [mm]:	26	**Total Settlement (PL=85%)[mm]:**	**56**
Indexed' LSN:	25	**LSN:**	**28**

aurecon

Aurecon New Zealand Limited
Unit 1, 150 Cavendish Road
Casebrook
PO Box 1061
Christchurch - New Zealand

Telephone:+64 3 366 0821
Facsimile:+64 3 379 6955
Email:christchurch@ap.aurecongroup.com
Website: www.aurecongroup.com

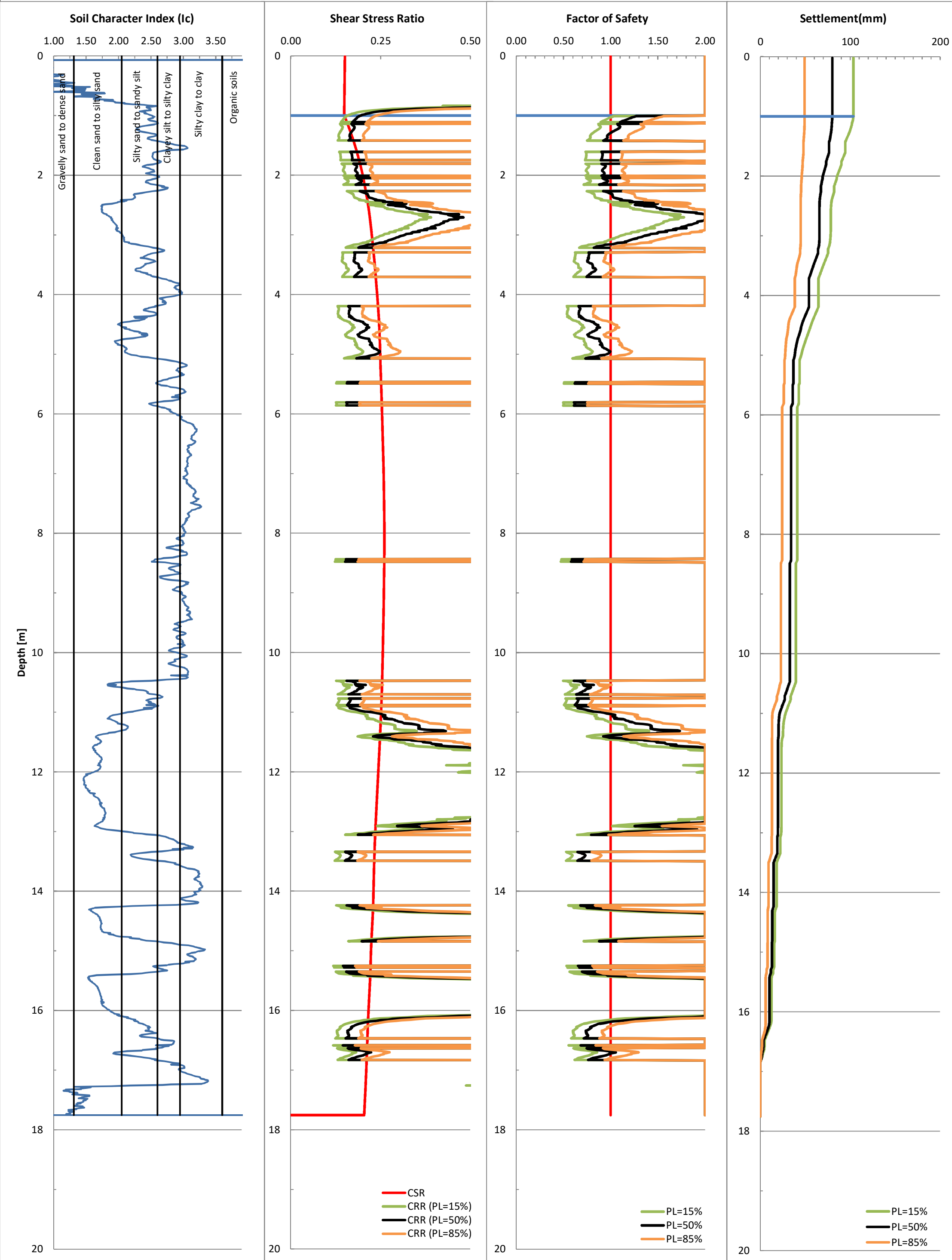
Client **Foodstuffs SI Ltd**	Location **Pak'n Save Papanui**
Project No. **243354**	Test No. **CPT_57000**
Design Event **DAR EQ**	Date **22 May 2018**

Based upon the methods specified in MBIE Guidelines "Repairing and rebuilding houses affected by the Canterbury earthquakes" Rev 3, Dec 2012) and clarifications and updates to the guidedance Issue 7, October 2014

LIQUEFACTION ANALYSIS

Ic calcuated from Robertson and Cabal (Robertson) (2014). CSR, CRR and FS calcuated From Boulanger and Idriss (2014).
Settlement calcuated from Zhang, Robertson and Brachman (2004).

I_c cut off:	2.60	Water Table [m]	1.00
C_{FC}:	0.20	Magnitude	6.20
		Acceleration [g]	0.23



Indexed' Settlement (PL=15%) [mm]:	64	**Total Settlement (PL=15%) [mm]:**	**103**
Indexed' Settlement (PL=50%) [mm]:	47	**Total Settlement (PL=50%)[mm]:**	**80**
Indexed' Settlement (PL=85%) [mm]:	26	**Total Settlement (PL=85%)[mm]:**	**49**
Indexed' LSN:	25	**LSN:**	**28**

aurecon
Aurecon New Zealand Limited
Unit 1, 150 Cavendish Road
Casebrook
PO Box 1061
Christchurch - New Zealand
Telephone:+64 3 366 0821
Facsimile:+64 3 379 6955
Email:christchurch@ap.aurecongroup.com
Website: www.aurecongroup.com

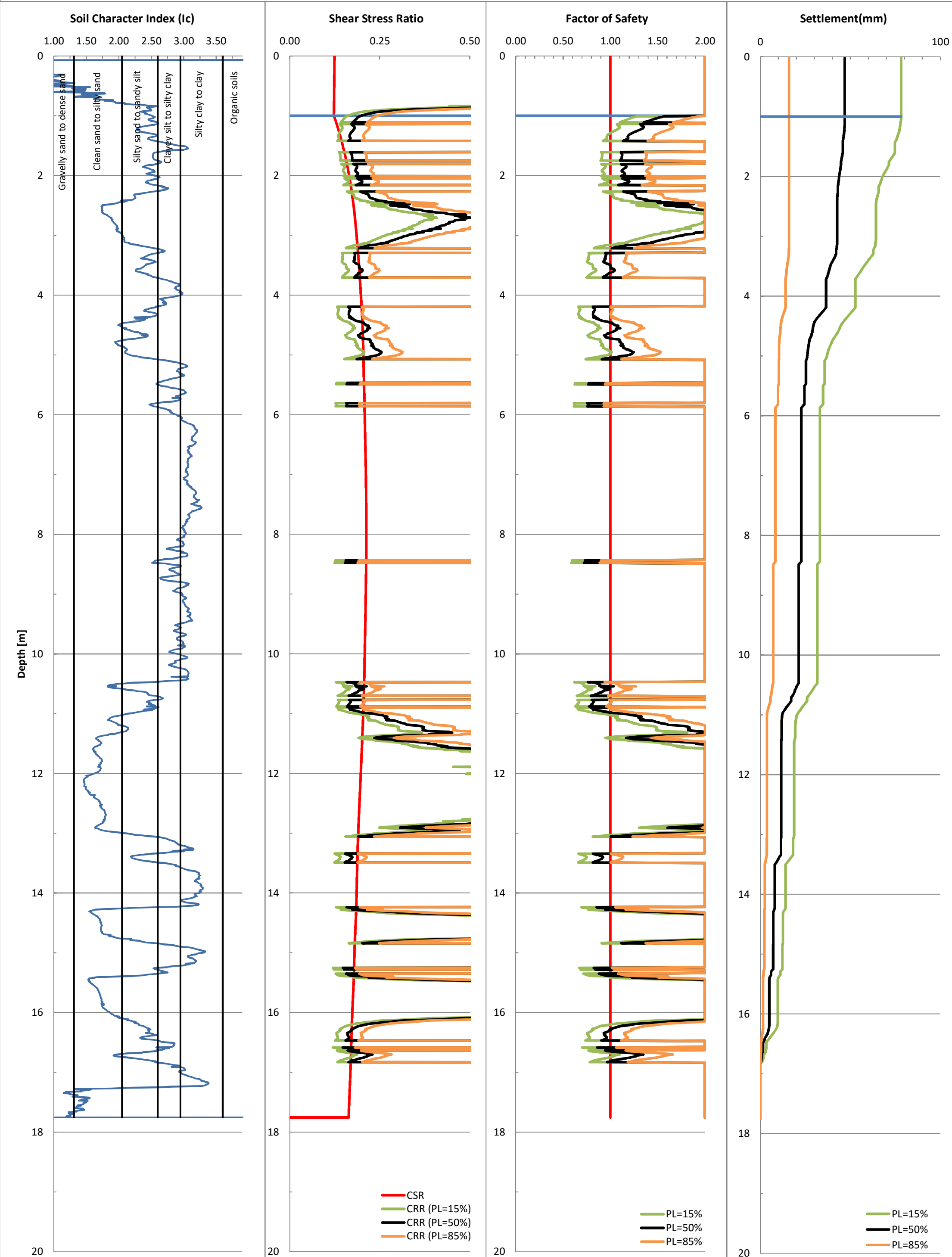
Client **Foodstuffs SI Ltd**	Location **Pak'n Save Papanui**
Project No. **243354**	Test No. **CPT_57000**
Design Event **CHC EQ**	Date **22 May 2018**

LIQUEFACTION ANALYSIS

Ic calcuated from Robertson and Cabal (Robertson) (2014). CSR, CRR and FS calcuated From Boulanger and Idriss (2014).
Settlement calcuated from Zhang, Robertson and Brachman (2004).

I_c cut off: 2.60
C_{FC}: 0.20

Water Table [m] 1.00
Magnitude 6.00
Acceleration [g] 0.19



Indexed' Settlement (PL=15%) [mm]: 47
Indexed' Settlement (PL=50%) [mm]: 26
Indexed' Settlement (PL=85%) [mm]: 9
Indexed' LSN: 16

Total Settlement (PL=15%) [mm]: 78
Total Settlement (PL=50%)[mm]: 47
Total Settlement (PL=85%)[mm]: 16
LSN: 19

aurecon
Aurecon New Zealand Limited
Unit 1, 150 Cavendish Road
Casebrook
PO Box 1061
Christchurch - New Zealand
Telephone:+64 3 366 0821
Facsimile:+64 3 379 6955
Email:christchurch@ap.aurecongroup.com
Website: www.aurecongroup.com

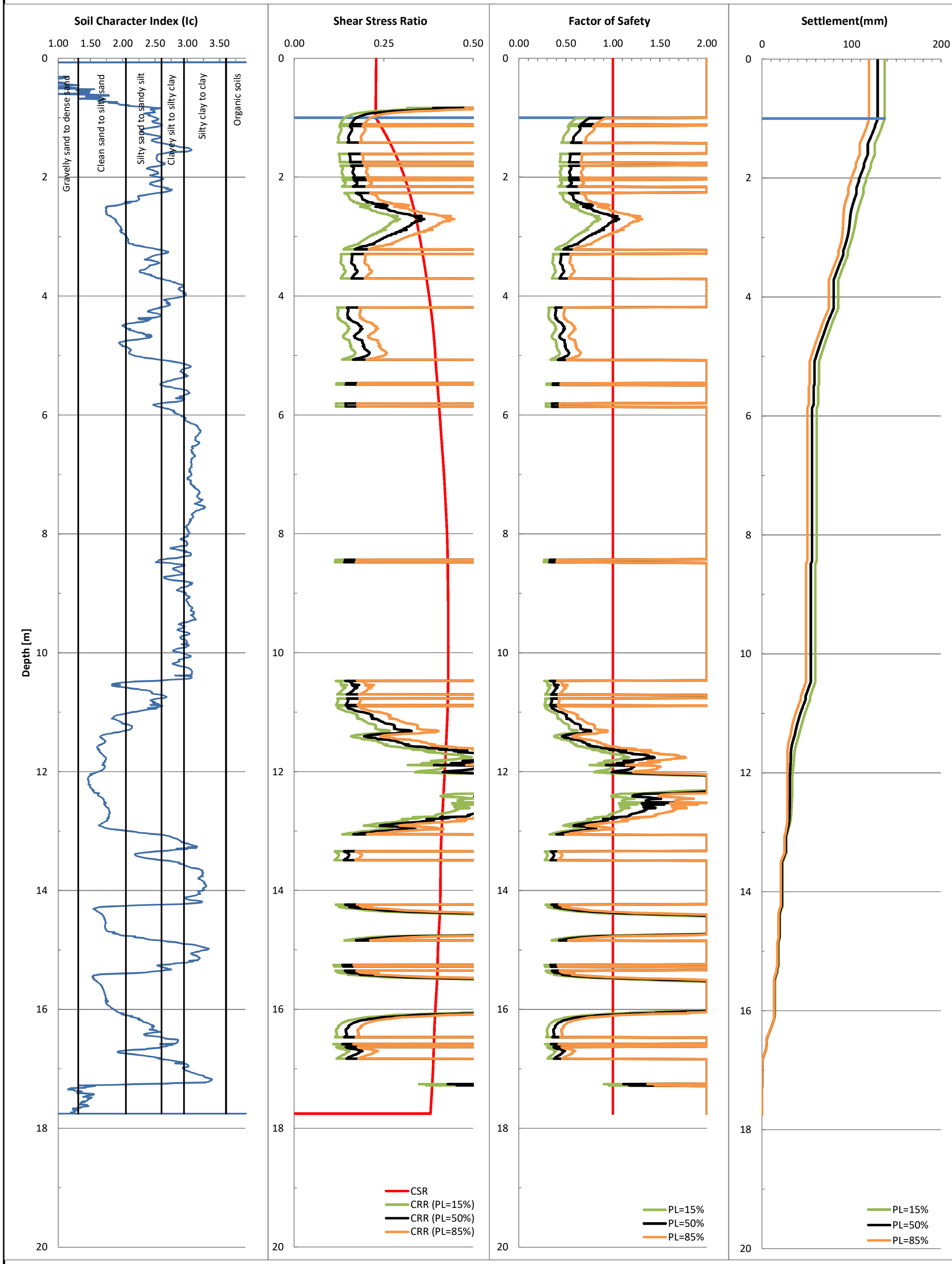
Client **Foodstuffs SI Ltd**	Location **Pak'n Save Papanui**
Project No. **243354**	Test No. **CPT_57000**
Design Event **SLS-1**	Date **22 May 2018**

Based upon the methods specified in MBIE Guidelines "Repairing and rebuilding houses affected by the Canterbury earthquakes" Rev 3, Dec 2012) and clarifications and updates to the guidedance Issue 7, October 2014

LIQUEFACTION ANALYSIS

Ic calcuated from Robertson and Cabal (Robertson) (2014). CSR, CRR and FS calcuated From Boulanger and Idriss (2014).
Settlement calcuated from Zhang, Robertson and Brachman (2004).

I_c cut off:	**2.60**	**Water Table [m]**	**1.00**
C_{FC}:	**0.20**	**Magnitude**	**7.50**
		Acceleration [g]	**0.35**



Indexed' Settlement (PL=15%) [mm]:	77	**Total Settlement (PL=15%) [mm]:**	**137**
Indexed' Settlement (PL=50%) [mm]:	75	**Total Settlement (PL=50%)[mm]:**	**129**
Indexed' Settlement (PL=85%) [mm]:	70	**Total Settlement (PL=85%)[mm]:**	**120**
Indexed' LSN:	31	**LSN:**	**35**

Aurecon New Zealand Limited
Unit 1, 150 Cavendish Road
Casebrook
PO Box 1061
Christchurch - New Zealand
Telephone:+64 3 366 0821
Facsimile:+64 3 379 6955
Email:christchurch@ap.aurecongroup.com
Website: www.aurecongroup.com

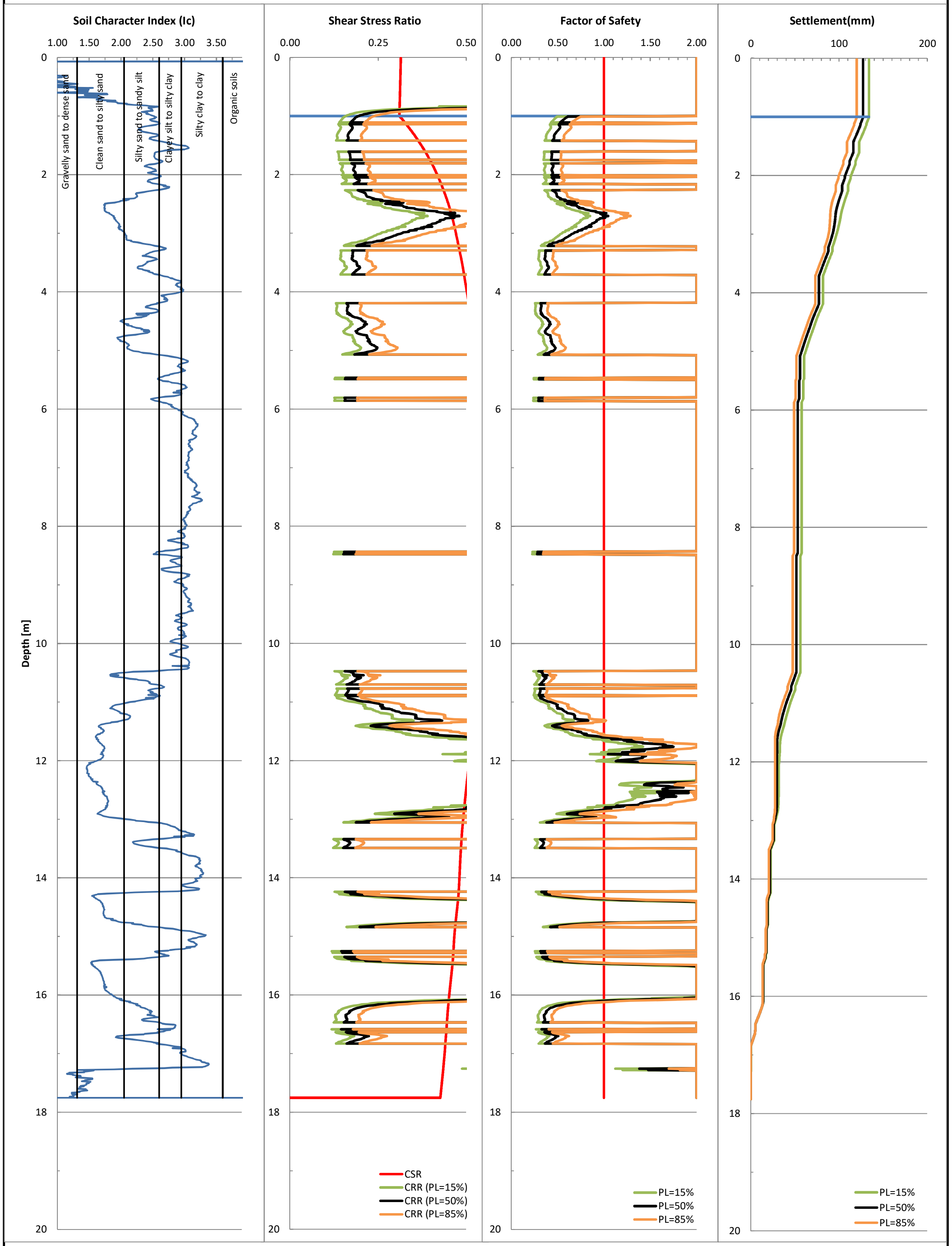
Client **Foodstuffs SI Ltd**	Location **Pak'n Save Papanui**
Project No. **243354**	Test No. **CPT_57000**
Design Event **SLS-2**	Date **22 May 2018**

Based upon the methods specified in MBIE Guidelines "Repairing and rebuilding houses affected by the Canterbury earthquakes" Rev 3, Dec 2012) and clarifications and updates to the guidedance Issue 7, October 2014

LIQUEFACTION ANALYSIS

Ic calcuated from Robertson and Cabal (Robertson) (2014). CSR, CRR and FS calcuated From Boulanger and Idriss (2014).
Settlement calcuated from Zhang, Robertson and Brachman (2004).

I_c cut off:	2.60	Water Table [m]	1.00
C_{FC}:	0.20	Magnitude	6.20
		Acceleration [g]	0.48



Indexed' Settlement (PL=15%) [mm]:	78	**Total Settlement (PL=15%) [mm]:**	**134**
Indexed' Settlement (PL=50%) [mm]:	76	**Total Settlement (PL=50%)[mm]:**	**127**
Indexed' Settlement (PL=85%) [mm]:	72	**Total Settlement (PL=85%)[mm]:**	**120**
Indexed' LSN:	31	**LSN:**	**35**

aurecon

Aurecon New Zealand Limited
Unit 1, 150 Cavendish Road
Casebrook
PO Box 1061
Christchurch - New Zealand
Telephone:+64 3 366 0821
Facsimile:+64 3 379 6955
Email:christchurch@ap.aurecongroup.com
Website: www.aurecongroup.com

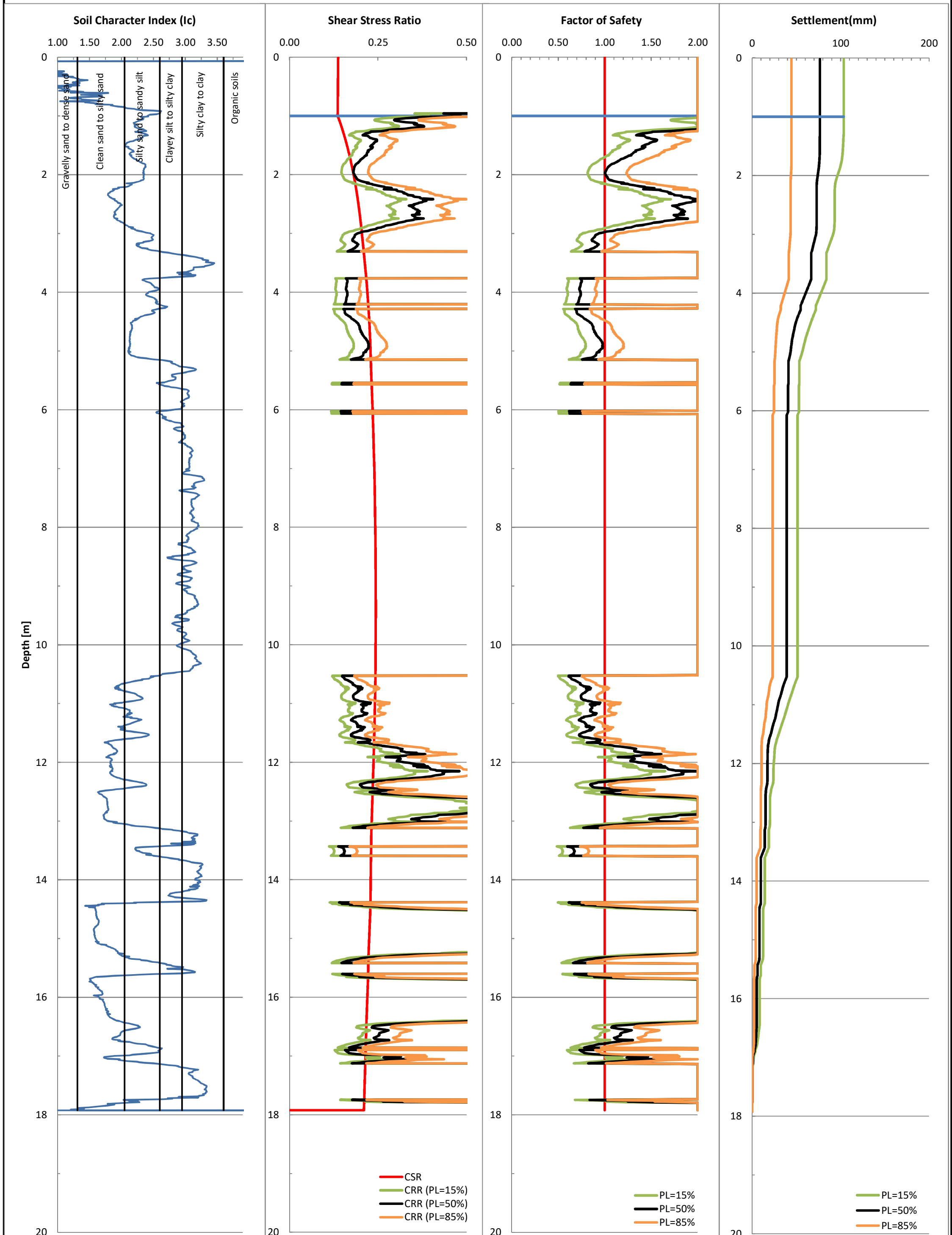
Client **Foodstuffs SI Ltd**	Location **Pak'n Save Papanui**
Project No. **243354**	Test No. **CPT_57000**
Design Event **ULS**	Date **22 May 2018**

Based upon the methods specified in MBIE Guidelines "Repairing and rebuilding houses affected by the Canterbury earthquakes" Rev 3, Dec 2012) and clarifications and updates to the guidedance Issue 7, October 2014

LIQUEFACTION ANALYSIS

Ic calcuated from Robertson and Cabal (Robertson) (2014). CSR, CRR and FS calcuated From Boulanger and Idriss (2014).
Settlement calcuated from Zhang, Robertson and Brachman (2004).

I_c cut off:	2.60	Water Table [m]	1.00
C_{FC}:	0.20	Magnitude	7.10
		Acceleration [g]	0.21



Indexed' Settlement (PL=15%) [mm]:	52	**Total Settlement (PL=15%) [mm]:**	**104**
Indexed' Settlement (PL=50%) [mm]:	37	**Total Settlement (PL=50%)[mm]:**	**76**
Indexed' Settlement (PL=85%) [mm]:	21	**Total Settlement (PL=85%)[mm]:**	**44**
Indexed' LSN:	16	**LSN:**	**20**

aurecon

Aurecon New Zealand Limited
Unit 1, 150 Cavendish Road
Casebrook
PO Box 1061
Christchurch - New Zealand

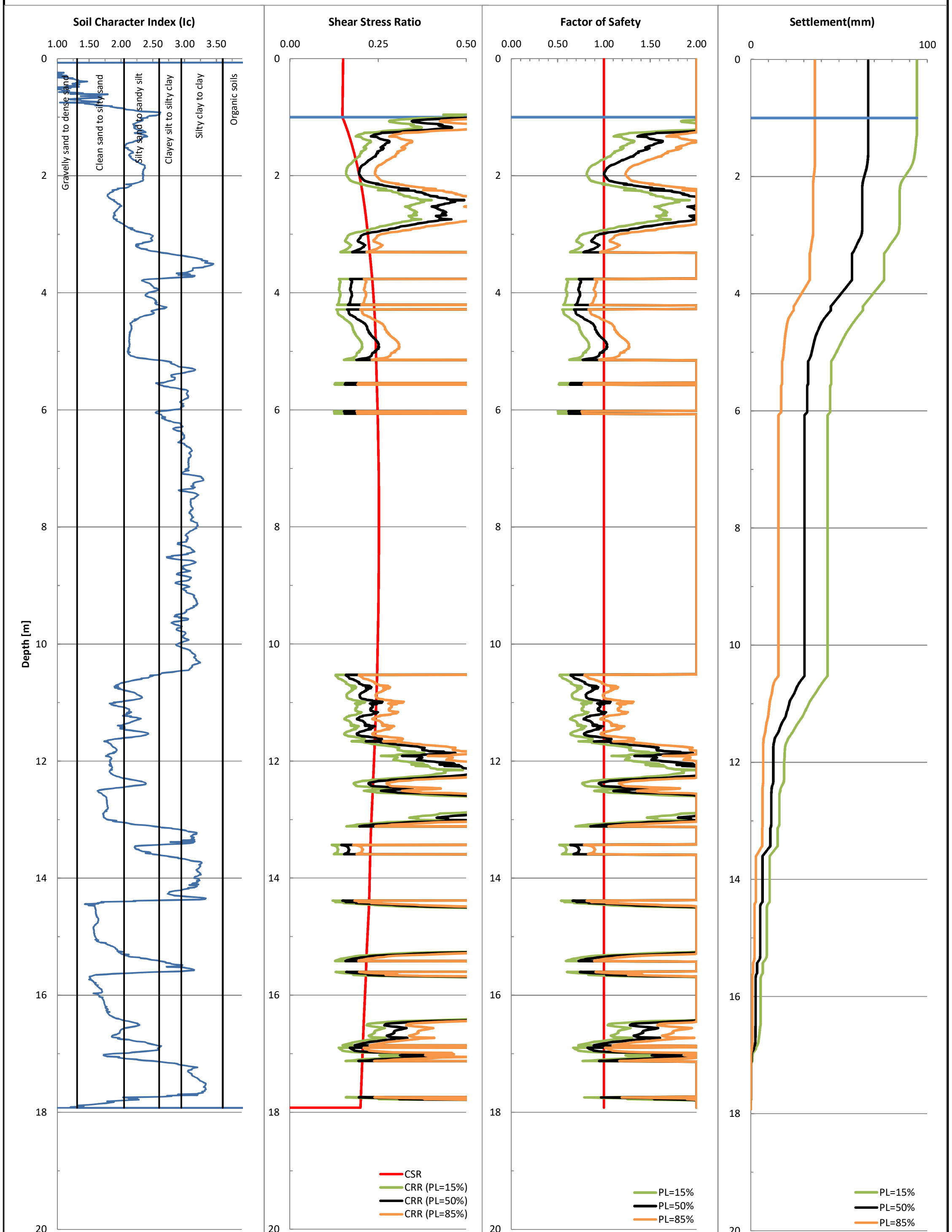
Telephone:+64 3 366 0821
Facsimile:+64 3 379 6955
Email:christchurch@ap.aurecongroup.com
Website: www.aurecongroup.com

Client **Foodstuffs SI Ltd**	Location **Pak'n Save Papanui**
Project No. **243354**	Test No. **CPT_57002**
Design Event **DAR EQ**	Date **22 May 2018**

LIQUEFACTION ANALYSIS

Ic calcuated from Robertson and Cabal (Robertson) (2014). CSR, CRR and FS calcuated From Boulanger and Idriss (2014).
Settlement calcuated from Zhang, Robertson and Brachman (2004).

I_c cut off:	**2.60**	**Water Table [m]**	**1.00**
C_{FC}:	**0.20**	**Magnitude**	**6.20**
		Acceleration [g]	**0.23**



Indexed' Settlement (PL=15%) [mm]:	51	**Total Settlement (PL=15%) [mm]:**	**94**
Indexed' Settlement (PL=50%) [mm]:	36	**Total Settlement (PL=50%)[mm]:**	**67**
Indexed' Settlement (PL=85%) [mm]:	21	**Total Settlement (PL=85%)[mm]:**	**36**
Indexed' LSN:	15	**LSN:**	**19**

aurecon

Aurecon New Zealand Limited
Unit 1, 150 Cavendish Road
Casebrook
PO Box 1061
Christchurch - New Zealand

Telephone:+64 3 366 0821
Facsimile:+64 3 379 6955
Email:christchurch@ap.aurecongroup.com
Website: www.aurecongroup.com

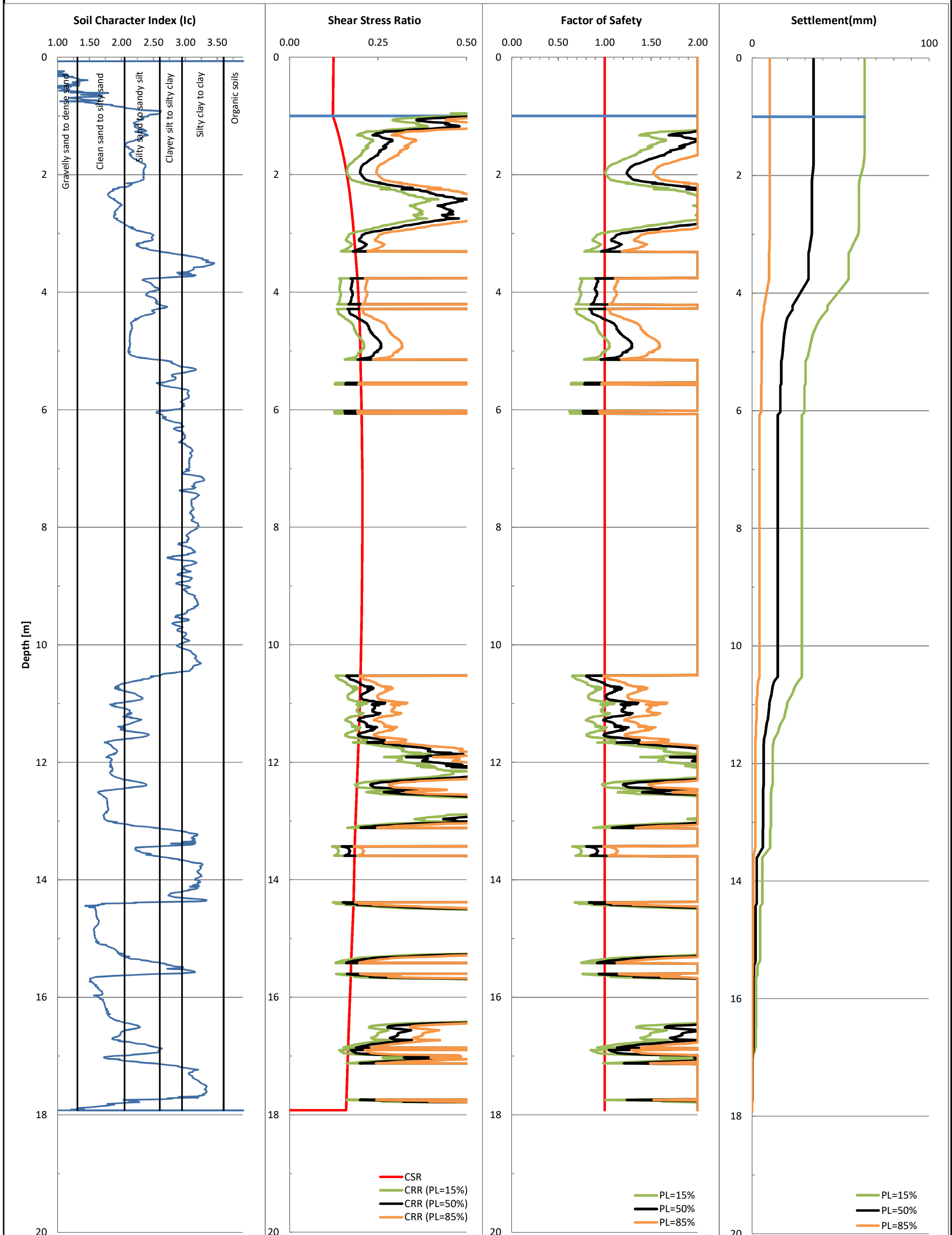
Client **Foodstuffs SI Ltd**	Location **Pak'n Save Papanui**
Project No. **243354**	Test No. **CPT_57002**
Design Event **CHC EQ**	Date **22 May 2018**

LIQUEFACTION ANALYSIS

Ic calcuated from Robertson and Cabal (Robertson) (2014). CSR, CRR and FS calcuated From Boulanger and Idriss (2014).
Settlement calcuated from Zhang, Robertson and Brachman (2004).

I_c cut off: 2.60
C_{FC}: 0.20

Water Table [m] 1.00
Magnitude 6.00
Acceleration [g] 0.19



Indexed' Settlement (PL=15%) [mm]: 35
Indexed' Settlement (PL=50%) [mm]: 20
Indexed' Settlement (PL=85%) [mm]: 6
Indexed' LSN: 10

Total Settlement (PL=15%) [mm]: 64
Total Settlement (PL=50%)[mm]: 35
Total Settlement (PL=85%)[mm]: 10
LSN: 12

aurecon
Aurecon New Zealand Limited
Unit 1, 150 Cavendish Road
Casebrook
PO Box 1061
Christchurch - New Zealand
Telephone:+64 3 366 0821
Facsimile:+64 3 379 6955
Email:christchurch@ap.aurecongroup.com
Website: www.aurecongroup.com

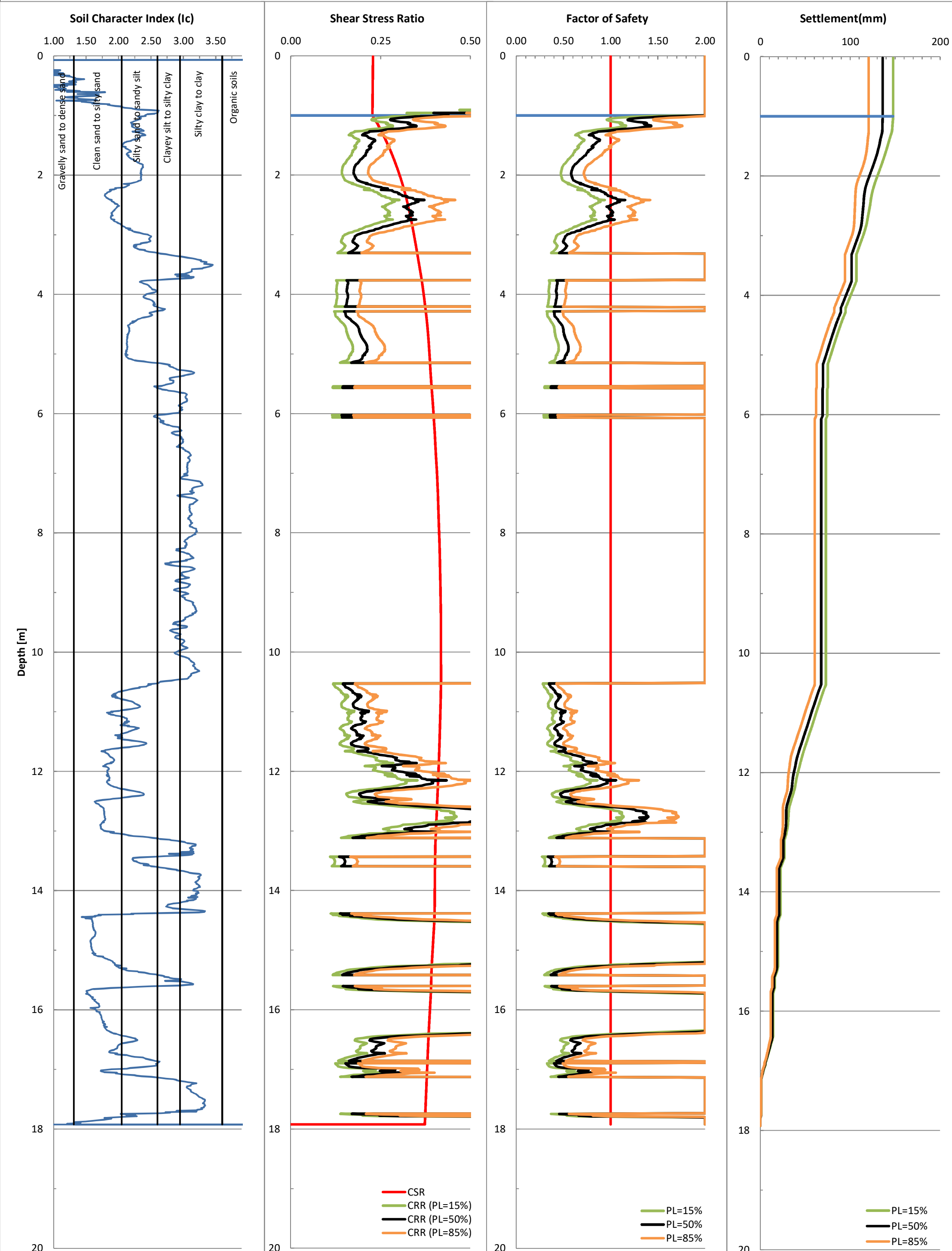
Client **Foodstuffs SI Ltd**	Location **Pak'n Save Papanui**
Project No. **243354**	Test No. **CPT_57002**
Design Event **SLS-1**	Date **22 May 2018**

LIQUEFACTION ANALYSIS

Ic calcuated from Robertson and Cabal (Robertson) (2014). CSR, CRR and FS calcuated From Boulanger and Idriss (2014).
Settlement calcuated from Zhang, Robertson and Brachman (2004).

I_c cut off: 2.60
C_{FC}: 0.20

Water Table [m] 1.00
Magnitude 7.50
Acceleration [g] 0.35



Indexed' Settlement (PL=15%) [mm]: 75
Indexed' Settlement (PL=50%) [mm]: 68
Indexed' Settlement (PL=85%) [mm]: 60
Indexed' LSN: 28

Total Settlement (PL=15%) [mm]: 148
Total Settlement (PL=50%)[mm]: 136
Total Settlement (PL=85%)[mm]: 121
LSN: 33

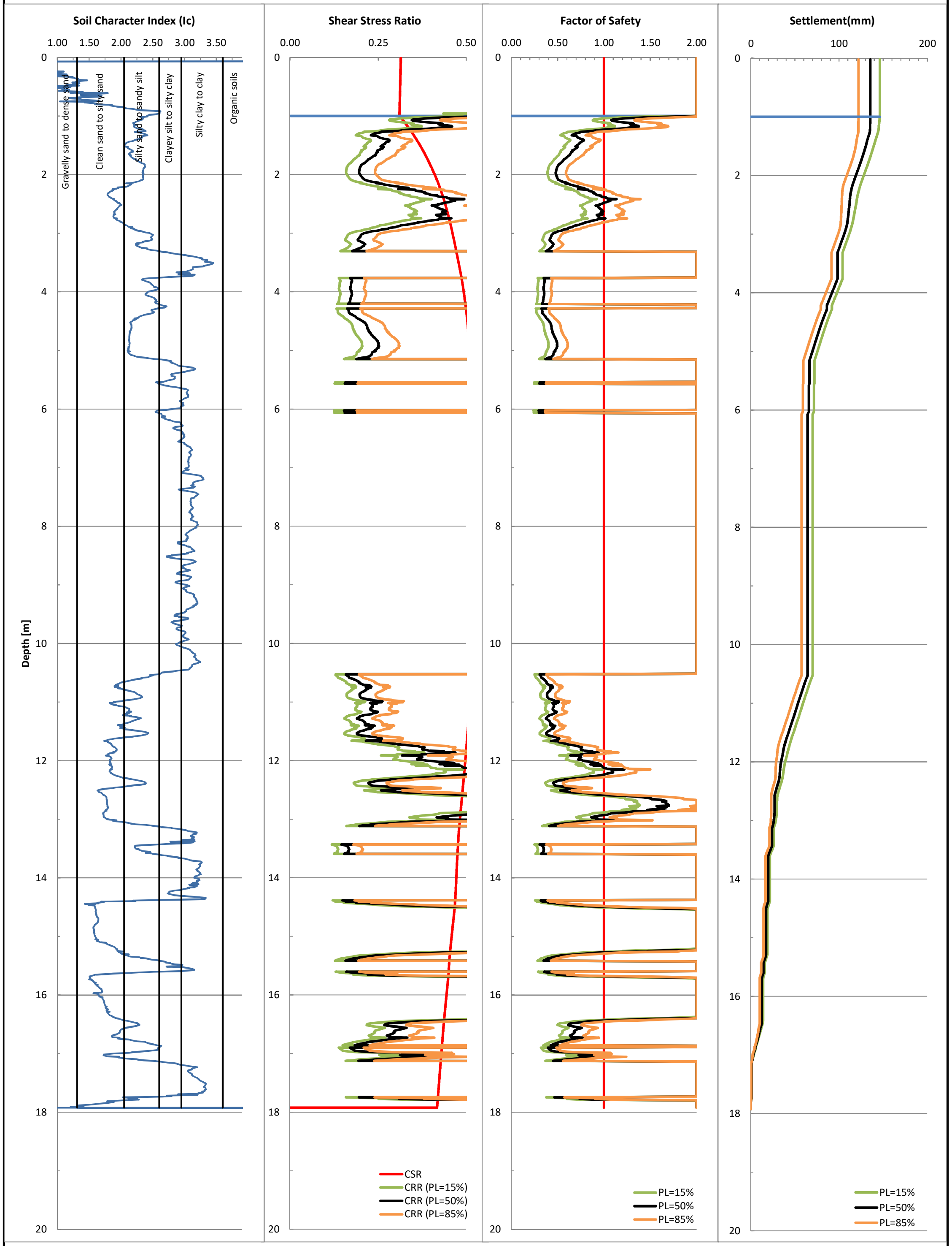
aurecon
Aurecon New Zealand Limited
Unit 1, 150 Cavendish Road
Casebrook
PO Box 1061
Christchurch - New Zealand
Telephone:+64 3 366 0821
Facsimile:+64 3 379 6955
Email:christchurch@ap.aurecongroup.com
Website: www.aurecongroup.com

Client Foodstuffs SI Ltd	Location Pak'n Save Papanui
Project No. 243354	Test No. CPT_57002
Design Event SLS-2	Date 22 May 2018

LIQUEFACTION ANALYSIS

Ic calcuated from Robertson and Cabal (Robertson) (2014). CSR, CRR and FS calcuated From Boulanger and Idriss (2014).
Settlement calcuated from Zhang, Robertson and Brachman (2004).

I_c cut off:	2.60	Water Table [m]	1.00
C_{FC}:	0.20	Magnitude	6.20
		Acceleration [g]	0.48



Indexed' Settlement (PL=15%) [mm]:	76	Total Settlement (PL=15%) [mm]:	146
Indexed' Settlement (PL=50%) [mm]:	71	Total Settlement (PL=50%)[mm]:	136
Indexed' Settlement (PL=85%) [mm]:	64	Total Settlement (PL=85%)[mm]:	122
Indexed' LSN:	29	LSN:	34

aurecon
Aurecon New Zealand Limited
Unit 1, 150 Cavendish Road
Casebrook
PO Box 1061
Christchurch - New Zealand
Telephone:+64 3 366 0821
Facsimile:+64 3 379 6955
Email:christchurch@ap.aurecongroup.com
Website: www.aurecongroup.com

Client: Foodstuffs SI Ltd	Location: Pak'n Save Papanui
Project No.: 243354	Test No.: CPT_57002
Design Event: ULS	Date: 22 May 2018

Document prepared by

Aurecon New Zealand Limited
Level 2, Iwikau Building
93 Cambridge Terrace
Christchurch 8013
New Zealand

T +64 3 366 0821
F +64 3 379 6955
E christchurch@aurecongroup.com
W aurecongroup.com

aurecon

Bringing ideas to life

Appendix K
Minimum Floor Level Certificate

MINIMUM FLOOR LEVEL CERTIFICATE UNDER THE CHRISTCHURCH DISTRICT PLAN

REFERENCE NUMBER: RMA/2018/1163

Pursuant to Rule 5.4.1.2 in Chapter 5 Natural Hazards of the Christchurch District Plan, the minimum floor level for new buildings, and additions to existing buildings that increase the ground floor area of the building, is certified as:

Property address:	155, 159, 161, 165 & 171 Main North Road and 3-7 Northcote Road
Legal description:	Pt Lot 1 DP 21207, Lot 1 DP 479583, Lot 1 DP 76152, Lot 1 DP 14400, Lot 7 14400, Lot 9 DP 14400
Minimum floor level:	19.49m above the Christchurch City Datum
Date of issue:	21 May 2018

This is the minimum floor level required for a building or addition to be a permitted activity under P3 (new buildings) and P4 (additions to existing buildings) in Rule 5.4.1.1 of the Christchurch District Plan.

This certificate is valid for two years from the date of issue.

Advice notes:

- For a building or addition to be a permitted activity under the Christchurch District Plan as a whole, all other relevant rules must be complied with.
- The minimum floor level certified under the District Plan may be different to the floor level required by the Building Act 2004 which must be met in order to obtain a building consent.
- Reference to this certificate when applying for a building consent will assist with the processing of your application.

Signed for and on behalf of the Christchurch City Council:

John Higgins
Head of Resource Consents

Civic Offices, 53 Hereford Street, Christchurch 8011
PO Box 73014, Christchurch 8154
Phone: 03 941 8999, Fax: 03 941 8792
www.ccc.govt.nz