Appendix 6: Geotechnical Investigation and RFI response letter

Fraser
Thomas
ENGINEERS • RESOURCE MANAGERS • SURVEYORS

18 February 2021 CH00417

Selwyn District Council
PO Box 90
Rolleston

Attention: Ms Rachel Carruthers

Dear Madam,

PC200072: TRICES ROAD REZONING GROUP PRIVATE PLAN CHANGE- RESPONSE TO REQUEST FOR FURTHER INFORMATION

This letter has been prepared in response to a letter, sent by Selwyn District Council (SDC), dated 2 February 2021, requesting further information relating to a submission on the Proposed Selwyn District Plan, and a private plan change request to the Operative Selwyn District Plan, seeking rezoning of Trices Road properties from “Rural” to “Residential”.

Fraser Thomas previously prepared a Geotechnical Investigation Report, dated 10 November 2020, for the subject site, in support of a submission on the Proposed Selwyn District Plan, for a private plan change.

The SDC letter has requested further clarification on six geotechnical matters, identified as Items 20, 21, 22, 23, 24 and 25. This letter addresses those items.

Item 20- Matters relating to the site’s past performance

Item 20 of the SDC letter, requests:

> **“The mean peak ground accelerations from the Bradley & Hughes model are set out in Table 1. Please advise how these relate to SLS and ULS levels of shaking and if the site has been “sufficiently tested” at SLS (MBIE 13.5.1), as past performances has been used to partially justify the TC1 classification.”**

The primary justification for our determination that the subject site should be assumed to be within Foundation Technical Category 1 (TC1), as defined by the MBIE guidance documents, is summarised below:

(1) the results of the theoretical analyses, presented in the November 2020 geotechnical report, which indicates that the surficial soils are not expected to liquefy under the SLS or ULS design earthquake events,

(2) the nature of the upper soils underlying the site, i.e. generally dense to very dense gravel soils,

(3) the depth to groundwater (expected to be no shallower than 2.5 m).

The observed performance of the site in response to seismic loading imposed by the 2010/2011 Canterbury earthquake sequence provides some validation of the results of our theoretical assessment, but is not the primary justification for the TC1 classification.

Nevertheless, The NZGD indicates the following conditional median peak ground accelerations were likely experienced at the site, during the 20201/2011 Canterbury earthquake sequence:

Earthquake Event	**Likely Peak Ground Accelerations (pga) (proportion of gravity acceleration (m/s^2))**
September 2010	0.35g
February 2011	0.25g
June 2011	0.11g

When these values are adjusted, using the recommended Magnitude Scaling Factor (MSF), it is evident that the September 2010 earthquake event likely imposed an equivalent design earthquake event (pga) of approximately 0.32g at the subject site (0.35g x 0.90 (MSF)).

A pga value of 0.32g is significantly higher than the SLS design earthquake loading of 0.13g, and the subject site is therefore considered to have been "sufficiently tested" under SLS design earthquake load conditions.

It should also be noted that a pga value of 0.32g is approaching the ULS design earthquake loading of 0.35g, and is therefore considered to also provide a good predictor as to the likely performance of the site under future ULS loading conditions.

Item 21- NZGD test data

Item 21 of the SDC letter, requests:

> **"Please supply the test data from the NZGD (location and logs) used to help identify the soil profile (8.3)."**

The logs of the existing machine excavated test pits, put down by other consultants, which have been sourced from the NZGD, are appended to this letter.

The test pits are located at a site abutting the western site boundary. The approximate inferred location and extent of these test pits are shown on the appended Fraser Thomas Ltd drawing G00417-02.

Item 22- ECan water bore logs

Item 22 of the SDC letter, requests:

> **"Please supply the Ecan well logs and locations used to model the gravels as extending to 18m depth (8.3)."**

The logs of the relevant existing water bore logs, sourced from ECan records, are appended to this letter.

The approximate inferred location and extent of the relevant water bores are shown on the appended Fraser Thomas Ltd drawing G00417-02.

Item 23- Deep testing density

Item 23 of the SDC letter, requests:

> **"Please confirm that the number of tests either on site or close by, do adequately meet the intent of the MBIE Guidance (16.2) to adequately characterize the soils to at least 5m depth in terms of density and depth (MBIE 6.3)."**

The MBIE guidelines "Repairing and rebuilding houses affected by the Canterbury earthquakes", provides some suggested minimum investigation density guidelines for "deep investigations". The guidelines suggest, for a plan change, a minimum of 5 deep tests (with a suggested range of 0.2 to 0.5 tests per hectare).

The subsoil information presented in the Fraser Thomas report, dated November 2020, has been determined using the following geotechnical field investigation tests:

(1) Eight hand augered boreholes

(2) five CPT probes

(3) four water bores (18 in total within close proximity to the site)

(4) two machine excavated test pits.

CPT probes are generally considered to be "deep investigation" tests, although, due to the nature of the subsoils underlying the site, the CPT probes were unable to be progressed deeper than approximately 3.8 m below the existing ground surface.

The water bores and machine excavated test pits (approximately 4.0 m deep), however, should be considered to be "deep investigations". There are 18 water bores within, or in close proximity to, the site, which vary in depth between approximately 6 m and 36 m below the existing ground surface. We have only presented the logs for some of the deeper water bores and for the water bores spatially separated across the site, so as to provide for a good site coverage, in order to demonstrate the consistency of the gravel soils across the site.

If you include the data provided by the existing machine excavated test pits, and all of the existing water bores, a total of 20 “deep” test locations have been sourced for the determination of the subsoil conditions at the site, which exceeds the minimum suggested by the MBIE guidelines for Plan change purposes. That been said, it should be noted that the MBIE guidelines were issued as “guidance” under Section 175 of the Building Act 2004, so the suggestions/methods provided in the guidelines are not considered to be mandatory. It is our opinion that the nature and extent of geotechnical investigation works should be determined by an appropriately qualified and experienced CPEng (Geotechnical) Engineer, and should be developed by assessing the geological conditions, determining the likely geotechnical hazards affecting the subject site, and should be cognitive of the nature of the proposed development.

Given the nature of the subsoils underlying the site, i.e generally dense to very dense gravel soils encountered at shallow depths, the type and the quantum of “deep investigation” undertaken for the site, for the purposes of determining the nature and consistency of the subsoils for a Plan change, is considered to be adequate.

It should be noted, should the site be rezoned and a concept subdivision be proposed, that Fraser Thomas would be required to prepare a Geotechnical Investigation Report, in support of an application for the proposed subdivision. It is envisaged that additional field investigations would be undertaken for this “subdivision” geotechnical report, in order to provide more information relating to the nature and consistency of the subsoils and the groundwater depths, which would likely include:

(a) 2 sonic machine boreholes (with standpipe piezometers installed)- 10 m to 15 m deep

(b) 6 machine excavated test pits.

Item 24- Groundwater depth

Item 24 of the SDC letter, requests:

> **“Please supply the data from which the groundwater depth has been derived”**

No groundwater was encountered at the locations of the CPT probes, the hand augered boreholes or the machine excavated test pits (abutting the western site boundary). This would indicate that the groundwater level underlying the site is likely to be greater than 4.0 m depth. I believe that one of the water bore logs had a recorded groundwater level of 2.5 m depth. Although this depth is not consistent with the groundwater levels encountered at the locations of other test positions across the site (i.e deeper than 4.0 m), we adopted this conservatively shallow groundwater level for analyses purposes.

In reality, it is likely that the groundwater level beneath the site is likely to be deeper than 2.5 m. This will be confirmed by the installation of standpipe piezometers (proposed for the subdivision report).

Item 25- RMA Section 106

Item 25 of the SDC letter, requests:

> **"The RMA section 106 sets out natural hazards which need to be considered before granting subdivision consent. Please supply a natural hazard assessment."**

It should be noted that the Fraser Thomas Ltd report, dated 10 November 2020, has been prepared in support of a submission on the Proposed Selwyn District Plan, for a private plan change, and has not been prepared in support of an application for subdivision consent.

It is our opinion that the "opinion statement" as to the suitability of the subject site for future residential development, is well summarised in Section 19.1(a) of our report, which states:

> *"In general terms and within the limits of the investigation as outlined and reported herein, no unusual problems, from a geotechnical perspective, are anticipated with residential development at the subject site.*
>
> *The site is, in general, considered suitable for its intended use, with satisfactory conditions for future residential building development, subject to the recommendations and qualifications reported herein, and provided the design and inspection of foundations are carried out as would be done under normal circumstances in accordance with the requirements of the relevant New Zealand Standard Codes of Practice."*

Nevertheless, in order to satisfy the peer reviewer's request, we confirm that the Fraser Thomas Ltd geotechnical report, dated 10 November 2020, includes recommendations which will appropriately avoid, remedy or mitigate potential geotechnical hazards on the land subject to the application, in accordance with the provisions of Section 106 of the Resource Management Act (in particular- see Sections 14.0, 15.0, 16.0 and 17.0 of the November 2020 report).

I trust the foregoing satisfies the requirements of SDC.

Kind regards

MASON REED
Director
CPEng (Geotechnical Engineer)

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Machine Excavated Test Pit Logs

GEOSCIENCE A proud member of ENGEO

Excavation Log - TP07

100 Birches Road

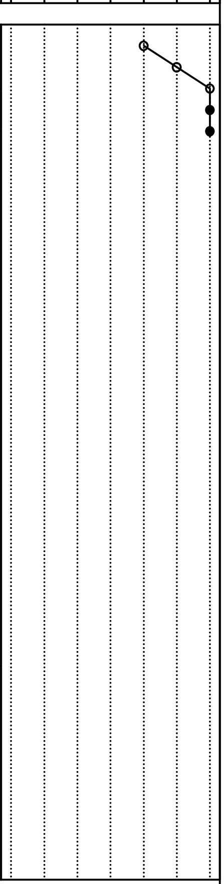
Prebbleton

Client : Conifer Grove Trustees Ltd
Project : 09875
Excavation Method : Test pit
Excavator Type : 6 T
Bucket Type : Toothed
Date : 31/1/13
Shear Vane No. : 1379
Logged/Reviewed By : RB/CL
Latitude : -43.5931
Longitude : 172.5103

Depth (m)	Method	Support	Geological Unit	USCS	DESCRIPTION	Water Level	Moisture Condition	Shear Vane (kPa) Peak/Remolded	Consistency / Density Index
0.0	EXCAVATION	UNSUPPORTED	TS	ML	SILT with trace rootlets; light brown [TOPSOIL].		D		VSt
			ALLUVIUM	SP	Fine SAND; greyish brown. Poorly graded. Moderately packed.		D-M		D
0.5				GW	Sandy fine to coarse GRAVEL with trace cobbles; greyish brown. Well graded; subrounded gravel; fine to coarse sand; trace tree roots from 0.5 m to 2.5 m depth. Moderately compacted.				MD-D
					Becomes moist at 1.6 m depth.		M		
					Trace organic silt encountered from 2.8 m depth.				
3.0					EOH: 3.0 m Termination: target depth Groundwater not encountered. Scala penetrometer terminated at practical refusal. TS = TOPSOIL				

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

Penetration: 0 1 2 3



Excavation Log - TP08

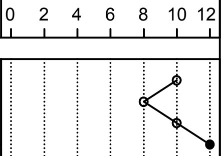
100 Birches Road

Prebbleton

Client	: Conifer Grove Trustees Ltd	**Date**	: 31/1/13
Project	: 09875	**Shear Vane No.**	: 1150
Excavation Method	: Test pit	**Logged/Reviewed By**	: CL
Excavator Type	: 6 T	**Latitude**	: -43.5938
Bucket Type	: Toothed	**Longitude**	: 172.5102

Depth (m)	Method	Support	Geological Unit	USCS	DESCRIPTION	Moisture Condition	Shear Vane (kPa) Peak/Remolded	Consistency / Density Index
0.0 – 0.3	EXCAVATION	UNSUPPORTED	TS	TS	SILT with some rootlets; brown.[TOPSOIL]	D	UTP	H
0.3 – 0.4			ALLUVIUM	ML	SILT with minor rootlets; light yellowish brown. Low plasticity.			
0.4 – 3.0				GW	Sandy fine to coarse GRAVEL; light brown. Well graded, rounded, greywacke gravel; medium to coarse sand. Tightly packed, minor undercut due to dislodging cobbles from the pit walls. Trace cobbles from 1m.	D (to 0.8) M		D
3.0					EOH 3.0m Target depth. Groundwater not encountered. Scala penetrometer met refusal, bouncing on gravel. TS = TOPSOIL			

Scala Penetrometer Blows per 100 mm

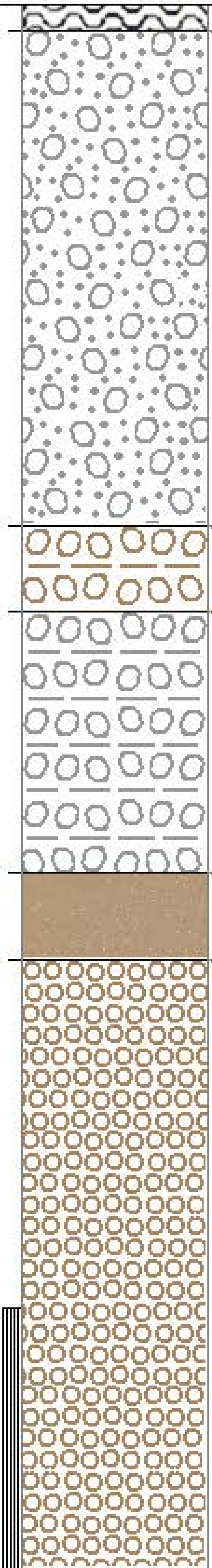


03-04-2013 Z:\Projects\2012\LYT Projects - 09801 to 09900\09875 100 Birches Road, Prebbleton\Field_Lab Testing\09875 TP08.bor

Water Bore Logs, sourced from Environment Canterbury records

Borelog for well M36/3133

Grid Reference (NZTM): 1560984 mE, 5173110 mN
Location Accuracy: 50 - 300m
Ground Level Altitude: 16.1 m +MSD Accuracy: < 0.5 m
Driller: McMillan Drilling Ltd
Drill Method: Cable Tool
Borelog Depth: 18.0 m   Drill Date: 07-Nov-1985

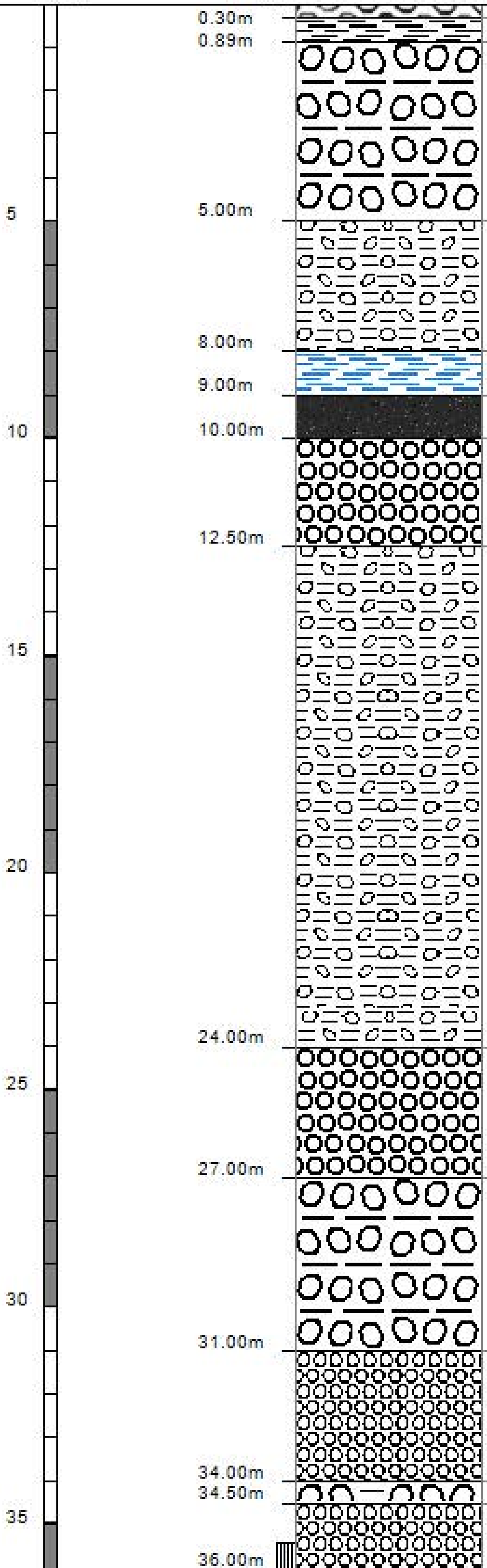


Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
		0.30m	Soil	SP
		0.30m	Soil	SP
			Grey shingle and sand	SP
5				
		6.00m		
		6.00m	Grey shingle and sand	SP
			Brown claybound gravels	SP
		7.00m		
		7.00m	Brown claybound gravels	SP
			Grey pug mixed with gravels	SP
10		10.00m		
		10.00m	Grey pug mixed with gravels	SP
			Brown peat	SP
		11.00m		
		11.00m	Brown peat	SP
			Free Brown stained gravels	RI
15				
		18.00m		

Borelog for well M36/5307

Grid Reference (NZTM): 1560645 mE, 5173256 mN
Location Accuracy: 2 - 15m
Ground Level Altitude: 18.0 m +MSD Accuracy: < 2.5 m
Driller: McMillan Drilling Ltd
Drill Method: Rotary Rig
Borelog Depth: 36.0 m Drill Date: 14-Jan-1998

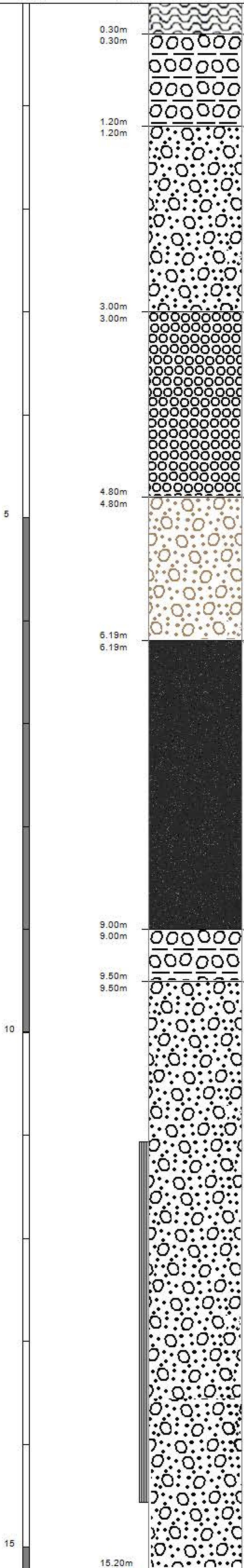
Environment Canterbury
Regional Council
Kaunihera Taiao ki Waitaha



Depth(m)	Full Drillers Description	Formation Code
0.30m	Soil.	SP
0.89m	Clay	SP
5.00m	Claybound gravel.	SP
8.00m	Claybound sandy gravel.	SP
9.00m	Blue clay.	SP
10.00m	Peat.	SP
12.50m	Free stained gravel.	RI
24.00m	Claybound sandy gravel.	RI
27.00m	Free gravel.	RI
31.00m	Claybound gravel.	RI
34.00m	Free sandy gravel.	BR
34.50m	Free gravel, clay patches.	BR
36.00m	Free stained sandy gravel.	BR

Borelog for well M36/5524

Grid Reference (NZTM): 1560843 mE, 5173323 mN
Location Accuracy: 2 - 15m
Ground Level Altitude: 17.1 m +MSD Accuracy: < 0.5 m
Driller: McMillan Drilling Ltd
Drill Method: Unknown
Borelog Depth: 15.2 m Drill Date: 02-Oct-1998

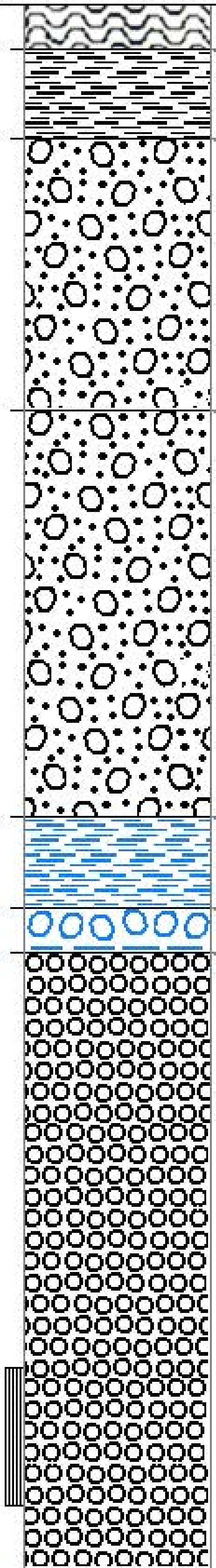


Depth(m)	Full Drillers Description	Formation Code
	Earth	SP
0.30m		
0.30m	Earth	SP
	Claybound gravel	SP
1.20m		
1.20m	Claybound gravel	SP
	Sandy gravel	SP
3.00m		
3.00m	Sandy gravel	SP
	Water-bearing free gravel	SP
4.80m		
4.80m	Water-bearing free gravel	SP
	Water-bearing free Brown stained sandy gravel	SP
6.19m		
6.19m	Water-bearing free Brown stained sandy gravel	SP
	Peat	SP
9.00m		
9.00m	Peat	SP
	Claybound gravel	RI
9.50m		
9.50m	Claybound gravel	RI
	Water-bearing free lightly stained sandy gravel	RI
15.20m		

Borelog for well M36/20546

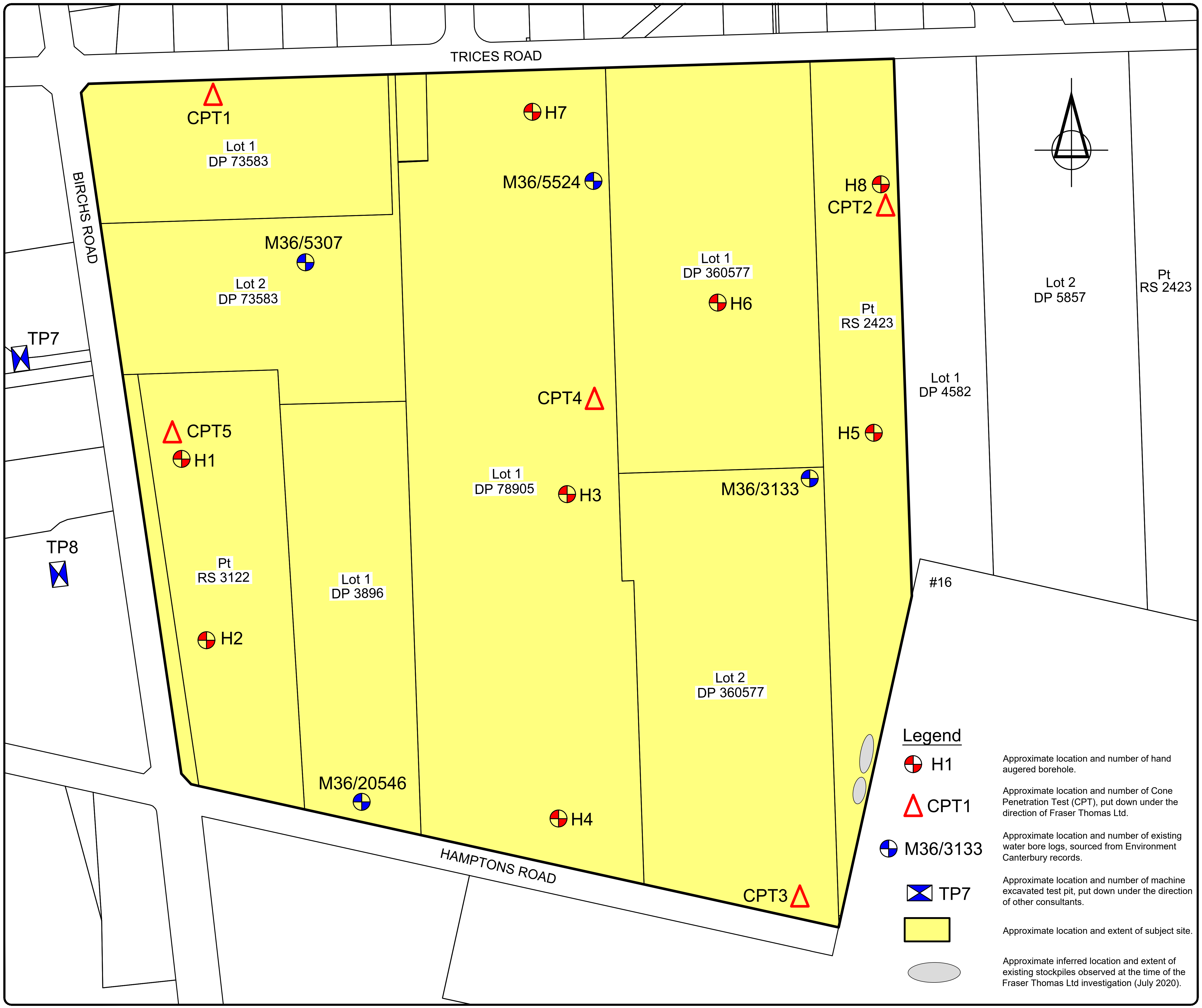
Grid Reference (NZTM): 1560681 mE, 5172885 mN
Location Accuracy: 2 - 15m
Ground Level Altitude: 16.7 m +MSD Accuracy: < 0.5 m
Driller: McMillan Drilling Ltd
Drill Method: Rotary/Percussion
Borelog Depth: 17.3 m Drill Date: 14-Dec-2010

Environment Canterbury
Regional Council
Kaunihera Taiao ki Waitaha



Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
			soil	
		0.50m		
		0.50m	soil	
			clay	
		1.50m		
		1.50m	clay	
			very sandy mixed gravel	
		4.50m		
		4.50m	very sandy mixed gravel	
5			very sandy mixed gravel	
		9.00m		
		9.00m	very sandy mixed gravel	
			blue clay	
10		10.00m		
		10.00m	blue clay	
		10.50m	blue claybound gravel	
		10.50m	blue claybound gravel	
			free water-bearing gravel	
15				
		17.30m		

Drawing G00417-02



Legend

⊕ H1	Approximate location and number of hand augered borehole.
△ CPT1	Approximate location and number of Cone Penetration Test (CPT), put down under the direction of Fraser Thomas Ltd.
⊕ M36/3133	Approximate location and number of existing water bore logs, sourced from Environment Canterbury records.
⊠ TP7	Approximate location and number of machine excavated test pit, put down under the direction of other consultants.
🟨	Approximate location and extent of subject site.
⬭	Approximate inferred location and extent of existing stockpiles observed at the time of the Fraser Thomas Ltd investigation (July 2020).

SURVEYED			APPROVED	DATE
DESIGNED			MVR	
DRAWN	JTG	17/02/2021		
CAD	GS	17/02/2021		
CHECKED				

REVISION	CHANGES	CHECKED	DATE
A	Issued for information	MVR	17/02/21

NOTES

1. This plan has been adopted from Quick map. The location and extent of the site boundaries and site features are therefore considered to be approximate only.

CLIENT

TRICES ROAD REZONING GROUP

PROJECT

TRICES ROAD AREA
PREBBLETON

TITLE

SITE PLAN

Fraser Thomas

ENGINEERS • RESOURCE MANAGERS • SURVEYORS

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SCALE 1:2500 (A3)

DRAWING No G00417-02

REVISION A

SHEET 1 of 1

Trices Road
Rezoning Group

Fraser Thomas
ENGINEERS • RESOURCE MANAGERS • SURVEYORS

PRIVATE PLAN CHANGE TO THE
OPERATIVE SELWYN DISTRICT
PLAN AND SUBMISSION ON
SELWYN DISTRICT PLAN REVIEW,
TRICES ROAD AREA,
PREBBLETON

GEOTECHNICAL INVESTIGATION REPORT

Trices Road Rezoning Group

PRIVATE PLAN CHANGE TO THE OPERATIVE SELWYN DISTRICT PLAN AND SUBMISSION ON SELWYN DISTRICT PLAN REVIEW, TRICES ROAD AREA, PREBBLETON

GEOTECHNICAL INVESTIGATION REPORT

Project No.	CH00417	Approved for Issue	
Version No.	2	Name	M V Reed
Status	Final	Signature	
Authors	J T GRAHAM		
Reviewer	M V Reed	Date	10 November 2020

Fraser Thomas Limited
Consulting Engineers, Licensed Surveyors
Planners & Resource Managers
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Riccarton 8041
PO Box 39 154, Harewood Post Centre,8545
Christchurch, New Zealand
Tel : +64 3 358-5936
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SUMMARY

This report presents the results of a geotechnical investigation and appraisal undertaken for the proposed rezoning of a site at Trices Road, Prebbleton. The subject site (approximately 28 ha) consists of eight separate existing landholdings held in ten separate Records of Title.

It is understood that it is proposed to lodge a submission on the Proposed Selwyn District Plan, and a private plan change request to the Operative Selwyn District Plan, seeking rezoning of the above properties from “Rural” to “Residential”, to enable subdivision of the site to create new lots, with an average lot size of 650m^2, but also include some areas of small lot residential development in the 400m^2 to 500m^2 size range. It is understood that the parcel of land bordering Birchs Road and Hamptons Road (approximately 2.8 ha) is also proposed to be rezoned, in order to create new lots, with an average proposed lot size of approximately 5,000m^2.

The approximate location and extent of the subject site is shown on the appended Fraser Thomas Ltd drawing G00417-01.

The results of the CPT probe and borehole investigations reported herein, in general, indicate that the surficial soils underlying the site are likely to comprise alluvial sediments of the Springston Formation of Holocene age.

It is our opinion that the subject site, for the purposes of the submission on the Selwyn District Plan Review and the private plan change request, should be assumed to be within Foundation Technical Category 1 (TC1), as defined by the MBIE guidance document, and that it is unlikely that liquefaction induced ground deformation could occur within the area in response to a large earthquake event, and that the ground settlements within the area in response to seismic loading should be considered to be “within normally accepted tolerances”, as defined by the MBIE December 2012 guidance document.

Foundation design recommendations are presented in Sections 14.0 and 15.0 of this report.

In general terms and within the limits of the investigation as outlined and reported herein, no unusual problems, from a geotechnical perspective, are anticipated with residential development at the subject site.

The site is, in general, considered suitable for its intended use, with satisfactory conditions for future residential building development, subject to the recommendations and qualifications reported herein, and provided the design and inspection of foundations are carried out as would be done under normal circumstances in accordance with the requirements of the relevant New Zealand Standard Codes of Practice.

PRIVATE PLAN CHANGE TO THE OPERATIVE SELWYN DISTRICT PLAN AND SUBMISSION ON SELWYN DISTRICT PLAN REVIEW
TRICES ROAD AREA,
PREBBLETON

TRICES ROAD REZONING GROUP

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TRICES ROAD AREA,
PREBBLETON

TRICES ROAD REZONING GROUP

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation and appraisal undertaken for the proposed rezoning of a site at Trices Road, Prebbleton. The subject site (approximately 28 ha) consists of the following existing properties:

1. Lot 1 DP 3896; approx. 24,357 m^2
2. Lot 1 DP 5284; approx. 1,279 m^2
3. Lot 1 DP 73583; approx. 20,236 m^2
4. Lot 1 DP 78905; approx. 80,000 m^2
5. Lot 1 DP 360577; approx. 40,002 m^2
6. Lot 2 DP 73583; approx. 23,868 m^2
7. Lot 2 DP 360577; approx. 41,198 m^2
8. Part RS 2423; approx. 28,327 m^2
9. Part RS 3122; approx. 24,837 m^2
10. RS 3974: approx. 3,037 m^2

It is understood that it is proposed to lodge a submission on the Proposed Selwyn District Plan, and a private plan change request to the Operative Selwyn District Plan, seeking rezoning of the above properties from “Rural” to “Residential”, to enable subdivision of the site to create new lots, with an average lot size of 650m^2, but also include some areas of small lot residential development in the 400m^2 to 500m^2 size range. It is understood that the parcel of land bordering Birchs Road and Hamptons Road (approximately 2.8 ha) is also proposed to be rezoned, in order to create new lots, with an average proposed lot size of approximately 5,000m^2.

The subject site is located on the corner of Trices Road and Birchs Road. In general, existing rural properties abut the subject site, to the east and Council reserve to the south. Existing residential properties abut the northern site boundary, and existing rural residential properties abut the western site boundary.

The approximate location and extent of the subject site is shown on the appended Fraser Thomas Ltd drawing G00417-01.

The Canterbury region has been subjected to significant seismic activity over the period September 2010 to June 2011 and beyond, which has resulted in significant land deformation and associated building damage throughout some areas of the Christchurch region.

Information obtained from the New Zealand Geotechnical Database indicates that the ejection of silt and sand, inferred to be associated with liquefaction of the underlying soil layers, was not observed within the vicinity of the site immediately following the September 2010, February 2011 and June 2011 earthquake events.

The subsurface conditions underlying the subject site have been investigated by means of Five Cone Penetration Tests (CPT) probes, eight hand augered boreholes, and associated Dynamic Cone Penetrometer (DCP) scala tests.

A visual appraisal of the site and a study of geological maps have also been undertaken.

The purpose of the geotechnical investigation reported herein was to determine the subsoil conditions beneath the subject site as they may affect future residential development, with particular regard to foundation considerations, and to determine the suitability of the subject site for the residential development, in support of the submissions discussed in Section 1.0 of this report.

2.0 SUMMARY OF 2010/2011 DAMAGING CANTERBURY EARTHQUAKE EVENTS

The Canterbury region has been subjected to significant seismic activity over the period September 2010 to June 2011 and beyond.

The significant damaging earthquake events are considered to be the following:

(a) 4 September 2010 (Moment Magnitude (M_w 7.1, epicentre depth = 11km)

(b) 22 February 2011 (M_w 6.2, epicentre depth = 5km)

(c) 13 June 2011 (M_w 6.0, epicentre depth = 6km)

(d) 23 December 2011 (M_w 5.9, epicentre depth = 6km)

The cyclic loading associated with these earthquake events has resulted in significant land deformation and associated building damage throughout some areas of the Canterbury region.

3.0 RECORDED PEAK GROUND ACCELERATIONS

Conditional median peak ground accelerations (pga) for the Canterbury region have been determined for various earthquake events (Bradley et al, 2012).

The conditional median peak ground accelerations have been determined by combining the peak ground acceleration values predicted using empirical ground motion models with the actual peak ground accelerations recorded at strong motion stations in the Christchurch region. The median peak ground accelerations predicted are therefore ‘conditional’ on the observations at distinct locations.

The conditional median peak ground accelerations inferred to have occurred at the subject site, in response to the September 2010, February, and June 2011 earthquake events are presented in Table 1.

TABLE 1: CONDITIONAL MEDIAN PEAK GROUND ACCELERATIONS EXPERIENCED AT THE SITE

Earthquake Event	Likely Peak Ground Accelerations (pga) (proportion of gravity acceleration (m/s^2))
September 2010	0.35g
February 2011	0.25g
June 2011	0.11g

4.0 OBSERVED PERFORMANCE OF THE LAND AND BUILDINGS FOLLOWING THE VARIOUS EARTHQUAKE EVENTS

Based on information obtained from the New Zealand Geotechnical Database (NZGD), the observed performance of the land and buildings immediately following the various earthquakes is summarised in Table 2.

TABLE 2: OBSERVED PERFORMANCE OF THE LAND AND BUILDINGS IMMEDIATELY FOLLOWING THE VARIOUS EARTHQUAKE EVENTS

Earthquake Event	Observed Performance of the Land	Observed Performance of the Buildings
September 2010	• Information obtained from the NZGD indicates that no obvious silt and sand ejecta (i.e. liquefaction) was observed in the vicinity the site.	• No first hand information was obtained from the property owners.
February 2011	• Information obtained from the NZGD indicates that no obvious silt and sand ejecta (i.e. liquefaction) was observed in the vicinity of the site.	• No first hand information was obtained from the property owners.
June 2011	• Information obtained from the NZGD indicates that no obvious silt and sand ejecta (i.e. liquefaction) was observed in the vicinity of the site.	• No first hand information was obtained from the property owners.

5.0 GEOLOGY

In assessing the geology of the site, reference has been made to the Institute of Geological & Nuclear Sciences Geological Map 16, scale 1:250,000, “Christchurch”.

This map indicates that the site is likely to be underlain by “grey river alluvium” of the Springston Formation of Holocene age.

The results of the CPT probe and borehole investigations reported herein, in general, indicate that the surficial soils underlying the site are likely to comprise alluvial sediments of the Springston Formation of Holocene age.

6.0 PROPOSED SUBDIVISIONAL DEVELOPMENT

As discussed in Section 1.0 of this report, the subject site (approximately 28 ha) consists of the following existing properties:

1. Lot 1 DP 3896; approx. 24,357 m^2
2. Lot 1 DP 5284; approx. 1,279 m^2
3. Lot 1 DP 73583; approx. 20,236 m^2
4. Lot 1 DP 78905; approx. 80,000 m^2
5. Lot 1 DP 360577; approx. 40,002 m^2
6. Lot 2 DP 73583; approx. 23,868 m^2
7. Lot 2 DP 360577; approx. 41,198 m^2
8. Part RS 2423; approx. 28,327 m^2
9. Part RS 3122; approx. 24,837 m^2
10. RS 3974: approx. 3,037 m^2

It is understood that it is proposed to lodge a submission on the Proposed Selwyn District Plan, and a private plan change request to the Operative Selwyn District Plan, seeking rezoning of the above properties from “Rural” to “Residential”, to enable subdivision of the site to create new lots, with an average lot size of 650m^2, but also include some areas of small lot residential development in the 400m^2 to 500m^2 size range. It is understood that the parcel of land bordering Birchs Road and Hamptons Road (approximately 2.8 ha) is also proposed to be rezoned, in order to create new lots, with an average proposed lot size of approximately 5,000m^2.

The approximate location and extent of the subject site is shown on the appended Fraser Thomas Ltd drawing G00417-01.

7.0 FIELD INVESTIGATION

7.1 GENERAL

The field investigation comprised a visual appraisal, five Cone Penetration Test (CPT) probes, and eight hand augered boreholes, and associated Dynamic Cone Penetrometer (DCP) scala tests.

The approximate locations of the investigation test positions are shown on Fraser Thomas Ltd drawing G00417-01.

7.2 RESULTS OF VISUAL APPRAISAL

A visual appraisal of the subject site was undertaken by a Fraser Thomas Ltd geotechnical engineer on 2 July 2020.

The subject site is located on the corner of Trices Road and Birchs Road. In general, existing rural properties abut the subject site, to the east and Council reserve to the south. Existing residential properties abut the northern site boundary, and existing rural residential properties abut the western site boundary.

The topography within the subject site is generally flat, and is generally vegetated with pasture.

Several existing dwellings and associated detached structures are located at the site.

Two existing stockpiles of material were observed in the south-eastern corner of the site (Pt RS 2423). The stockpiles are inferred to range between approximately 1.5 m and 3.5 m in vertical height and appear to comprise a mixture of silt, sand, gravel, and concrete block material. The origin of this material is unknown.

The approximate inferred locations and extent of the existing stockpiles are shown on drawing G00417-01.

No obvious signs of any significant ground deformation, that could be attributed to liquefaction induced ground movement, were observed within the subject site, at the time of the investigation reported herein.

7.3 HAND AUGERED BOREHOLE INVESTIGATION

Eight hand augered boreholes, numbered H1 to H8 inclusive, were put down at the site on 2 July 2020, in order to determine the nature and consistency of the subsoils underlying the site.

The boreholes were put down by a qualified Fraser Thomas Ltd geotechnical engineer. The logs of the boreholes are presented in Appendix A of this report.

The boreholes were generally terminated, when the soils became too difficult to auger, at depths ranging between approximately 0.2 m and 1.3 m below the ground surface existing at the time of the investigation reported herein (i.e. the existing ground surface).

All soils in the boreholes were carefully logged.

In situ undrained shear strength measurements were carried out, where possible, within the cohesive materials encountered in the boreholes using hand held field shear vane equipment.

A Dynamic Cone Penetrometer (DCP) scala test was undertaken in the base of Borehole H1. DCP tests were also undertaken from the existing ground surface, at the locations of Boreholes H5 and H8. A DCP test was also undertaken, when sands were encountered, at a depth of approximately 0.5 m below the existing ground surface, at the location of Borehole H4.

The results of the DCP scala tests are also presented in Appendix A of this report.

The approximate locations of Boreholes H1 to H8 inclusive are shown on drawing G00417-01.

7.4 CPT INVESTIGATION

Five Cone Penetration Test (CPT) probes, numbered CPT1 to CPT5 inclusive, were carried out at the site on 2 July 2020, under the direction of Fraser Thomas Ltd. The CPT probes were pushed in order to obtain continuous strength profiles for the subsoils, and for the purpose of determining the theoretical liquefaction potential of the soils.

The CPT probes were carried out by Ground Investigation, who conducted the CPT probes according to the American Society for Testing and Materials Standard D5778-12.

CPT3, CPT4 and CPT5 were terminated at depths of approximately 3.8 m, 2.3 m and 1.9 m respectively below the existing ground surface.

CPT1 and CPT2 refused generally at the ground surface, due to high cone resistance values encountered in shallow gravel soils.

The CPT data has been interpreted using the computer program CPeT-IT. The results of the interpretation of the relevant CPT data (i.e. CPT3, CPT4 and CPT5) are presented in Appendix A of this report.

The approximate locations of the CPT probes are shown on drawing G00417-01.

8.0 SUBSURFACE CONDITIONS

8.1 GENERAL

The subsoil information, presented in Appendix A of this report, indicates that the subject site is, in general, underlain by soils inferred to be alluvial sediments of the Springston Formation of Holocene age.

It has been assumed that even though the various subsoil strata (depths, thicknesses, and locations of groundwater levels) have been determined only at the locations and within the depths of the various test positions recorded herein, these various subsurface features can be projected between the various test positions. Even though such inference is made, no guarantee can be given as to the validity of this inference or of the nature and continuity of these various subsurface features.

8.2 TOPSOIL

A surficial layer of topsoil, generally comprising silts, was encountered to a depth of between approximately 0.2 m and 0.4 m below the existing ground surface, at the locations of the boreholes.

8.3 ALLUVIAL SEDIMENTS

8.3.1 Clayey Silts

The results of the hand augered borehole and CPT probe investigations, undertaken in the southern part of the subject site, indicate that the surficial topsoil is generally underlain by an upper layer of clayey silts, inferred to be alluvial sediments of the Springston Formation.

These sediments were generally encountered at depths of between approximately 0.3 m and 0.4 m below the existing ground surface. The sediments were encountered to depths ranging between approximately 0.6 m and 0.8 m below the existing ground surface, at the locations of Boreholes H2, H4, H6 and H7, corresponding to a layer thickness of between approximately 300 mm and 400 mm.

In situ undrained shear strength values of between approximately 45 kPa and greater than 196 kPa, were generally measured in the upper layers of cohesive soils, using hand held shear vane equipment, corresponding to a firm to hard consistency.

The CPT probes generally obtained cone resistance (q_t) values ranging between approximately 1.0 MPa and 2.5 MPa in the upper cohesive sediments, corresponding to in situ undrained shear strength values of between approximately 70 kPa and 180 kPa, corresponding to a stiff to very stiff consistency.

8.3.2 Silty Sands and Sands

The results of the hand augered borehole and CPT probe investigations, undertaken in the southern part of the subject site, indicate that the cohesive soils are generally underlain by a layer of material inferred to comprise silty sands and sands.

These cohesionless soils were encountered at depths ranging between approximately 0.7 m and 0.8 m below the existing ground surface, at the locations of Boreholes H2, H4 and H7. The sediments were encountered to depths ranging between approximately 0.9 m and 1.3 m below the existing ground surface, corresponding to a layer thickness of between approximately 100 mm and 600 mm, at the locations of Boreholes H2, H4 and H7.

The results of the DCP test undertaken in these sands and silty sands generally obtained DCP blow counts of between approximately 2 and 5 blows per 50 mm penetration, corresponding to a SPT ‘N’ value of between approximately 27 and 45, corresponding to a medium dense to dense consistency.

8.3.3 Sandy Gravels and Gravelly Sands

The results of the investigations reported herein indicate that the surficial soils at the site are generally underlain by a layer of material, inferred to comprise sandy gravels and gravelly sands. These soils were generally encountered at depths ranging between approximately 0.3 m and 1.3 m below the existing ground surface, at the locations of the test positions. The CPT probes and hand augered boreholes were not able to be progressed through these soils.

The CPT probes generally obtained cone resistance (q_t) values of generally between approximately 20 MPa and greater than 40 MPa in these sediments, corresponding to a dense to very dense consistency.

The results of the DCP test undertaken in the sandy gravels, at the locations of Boreholes H1, H4, H5 and H8 generally obtained DCP blow counts of between approximately 6 and greater than

10 blows per 50 mm penetration, corresponding to a SPT ‘N’ value of generally greater than 50, corresponding to a very dense consistency.

The logs of some existing machine excavated test pits have been sourced from the NZGD. The test pits are located at a site abutting the western site boundary. These logs indicate that gravels are encountered at shallow depth and are encountered to the extent of the test pits (i.e. 3.0 m depth). The logs indicate that cobbles were also encountered in the gravel soils.

The logs of existing water bore logs, put down within the subject site, have also been sourced from Environment Canterbury records.

The existing water bore logs indicate that sandy gravels are generally located at shallow depths, which is consistent with the subsoil conditions encountered at the subject site. The bore logs indicate that these sandy gravels generally extend to depths of greater than approximately 18 m below the ground surface. Based on the foregoing, it is, in our opinion, likely that the gravel soils underlying the site extend to significant depths below the existing ground surface.

8.4 GROUNDWATER

Based on the results of the borehole investigations undertaken at the site, and ground investigation information obtained from the NZGD, the groundwater level is inferred to be at a depth of approximately 2.5 m below the existing ground surface, for analysis purposes.

9.0 LIQUEFACTION POTENTIAL ASSESSMENT

9.1 GENERAL

Liquefaction is defined as the phenomenon that occurs when soils are subject to a sudden loss in shear stiffness and strength associated with a reduction in effective stress due to cyclic loading (i.e. ground shaking associated with an earthquake).

The two main effects of liquefaction on soils are:

(a) Consolidation of the liquefied soils

(b) Reduction in shear strength within the liquefied soils

Liquefaction is considered to occur when the soils reach a condition of “zero effective stress”. It is considered that only “sand like” soils can reach a condition of “zero effective stress” and therefore only “sand like” soils are considered to be liquefiable.

An indication that the underlying soils have been subject to liquefaction is the surface expression of ejected sand and water. This occurs as a result of the dissipation of excess pore water pressures generated within the liquefied soils as a result of the cyclic loading.

It should be noted that cohesive type materials or “clay like” soils are unlikely to be subject to liquefaction, as these soils (due to their nature) are unlikely to develop sufficient excess pore water pressures during cyclic loading to reach a condition of zero effective stress, i.e. the point of liquefaction.

However, “clay like” soils do develop some excess pore water pressures during cyclic loading which can result in consolidation settlement and a temporary reduction of the shear strength (i.e. softening) of the soils. Sensitive “clay like” soils are in particular susceptible to softening as a result of cyclic loading.

A liquefaction potential assessment has been undertaken for the soils underlying the subject site.

9.2 METHOD OF ANALYSIS

The New Zealand Geotechnical Society released Guidelines, in 2016, with the objective of summarising current best practice in earthquake geotechnical engineering with a focus on New Zealand conditions. The main purpose of the Guidelines is to promote consistency of approach to everyday engineering practice in New Zealand and, thus, improve geotechnical earthquake aspects of the performance of the built environment.

The Guidelines consists of six modules (identified as Modules 1 to 6 inclusive).

“Module 3: Identification Assessment and Mitigation of Liquefaction Hazards” of the Guidelines provides guidance on the identification of liquefaction hazards, and also provides details regarding different methodologies for determining theoretical liquefaction triggering.

The Module 3 guideline suggests a three-step process for the liquefaction assessment of sites, generally being:

(i) Step 1: Assessment of liquefaction susceptibility

(ii) Step 2: Triggering of liquefaction

(iii) Step 3: Consequences of liquefaction

The Module 3 guideline refers to the methods suggested by “Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils”, dated October 2001. The guideline, among others, also refers to papers by Youd et al; Seed; Idriss; Boulanger; Robertson and Bray.

A liquefaction potential assessment of the soils underlying the subject site has been undertaken using the methods suggested by the Module 3 guideline.

9.3 ASSESSMENT OF LIQUEFACTION SUSCEPTIBILITY

The following soils are generally considered to be susceptible to liquefaction:

(a) Young (typically Holocene age) alluvial sediments (typically fluvial deposits laid down in a low energy environment) or man-made fills

(b) Poorly consolidated/compacted sands and silty sands

(c) Areas with a high groundwater level.

As discussed in Section 5.0 of this report, the geological map for the Christchurch area indicates that the site is likely to be underlain by “grey river alluvium” of the Springston Formation of Holocene age.

As discussed in Section 8.3 of this report, the results of the CPT probe and borehole investigations indicate the site is generally underlain by an upper layer of stiff to very stiff cohesive soils, which is in turn underlain by medium dense to very dense sandy gravels and gravelly sands.

As discussed in Section 8.4 of this report, the groundwater level is inferred to be at a depth of approximately 2.5 m below the existing ground surface, for analysis purposes.

Based on the foregoing, it is our opinion that some soils underlying the site are likely to be susceptible to liquefaction.

9.4 TRIGGERING OF LIQUEFACTION

The NCEER report, dated October 2001, suggests the triggering of liquefaction within soils be assessed using the methods suggested by Seed and Idriss (1971), which states that:

FL = CRR/CSR

-where FL = Liquefaction Triggering Factor

CRR = Cyclic Resistance Ratio (ability of soils to resist liquefaction)

CSR = Cyclic Stress Ratio (seismic demand on soil caused by earthquake)

When FL $\leq$ 1.0 - Liquefaction is assumed to occur within the soil layer.

Generally the calculation of the CRR value for a certain soil is determined taking into account the soil type, density and the depth (confinement) of the soil layer.

Generally the calculation of the CSR value for a certain soil is determined taking into account the theoretical peak ground acceleration (pga) resulting from an earthquake and the depth (confinement) of the soil layer.

Computer programs are available which can compute the CRR and CSR values for soils using the data obtained from CPT probe.

The CRR and CSR values, and the theoretical triggering of liquefaction within the soils underlying the site, have been assessed using the computer program CLiq using the data obtained from the CPT probe investigation discussed in Section 7.4 of this report.

CLiq is a computer program that uses the methods suggested by the NCEER report (October 2001) and which also applies amended calibration/methodology procedures suggested by Zhang, Idriss and Boulanger, Robertson et al.

The results of the analyses to determine the theoretical liquefaction triggering potential of the site soils are presented in Section 10.4 of this report.

9.5 CONSEQUENCES OF LIQUEFACTION

The possible consequences of liquefaction of the soils beneath a site may include:

(i) Ground settlement

(ii) Ejection of sand at the surface

(iii) Differential building foundation settlement as a result of differential ground settlement

(iv) Foundation settlement as a result of bearing capacity failure of the soils (both "sand like" and "clay like")

(v) Lateral displacement of the ground as a result of "lateral spread".

Theoretical analyses have been undertaken using the computer program CLiq to determine the theoretical ground settlements expected to occur as a result of liquefaction of soil layers. The analyses have been undertaken using the CPT probe data obtained from the site.

CLiq uses the methods suggested by Zhang et al (2002 and 2004) to predict ground settlements expected to occur as a result of liquefaction of "sand like" soil layers.

The results of the analyses to determine the theoretical ground settlements as a result of liquefaction of the subsoils are presented in Section 10.4 of this report.

10.0 THEORETICAL ANALYSES OF LIQUEFACTION TRIGGERING POTENTIAL AND EXPECTED GROUND SETTLEMENTS

10.1 GENERAL

Analyses have been undertaken using the computer program CLiq to assess the theoretical liquefaction triggering potential and expected ground settlements for the soils underlying the subject site.

The analyses have been undertaken for the subsoil profiles obtained at the locations of CPT3, CPT4 and CPT5.

10.2 PEAK GROUND ACCELERATION (PGA) VALUES ASSUMED FOR ANALYSES

The following design earthquake events have been assessed for the site for the purposes of the analyses reported herein:

(a) Serviceability Limit State (SLS) - 25 year return period

(b) Intermediate Limit State (ILS) – 100 year return period

(c) Ultimate Limit State (ULS) - 500 year return period

It is noted that *"Module 1: Overview of the Guidelines"*, indicates that generally, in New Zealand, the unweighted seismic hazard factors and corresponding effective earthquake magnitude presented in the NZTA Bridge Manual (2014) should be used in liquefaction triggering analyses.

However, the guideline indicates that the seismic hazard factors provided in the Ministry of Business, Innovation & Employment (MBIE) document entitled "Repairing and rebuilding houses affected by the Canterbury earthquakes"; Version 3, dated December 2012, should be used in the Canterbury area.

The MBIE guidance provides recommendations as to the peak ground accelerations that should be used for liquefaction potential assessments within the Christchurch area, for the SLS and ULS design earthquakes.

The theoretical peak ground acceleration values and corresponding earthquake Moment Magnitudes (M_w) for liquefaction potential assessments for the SLS and ULS design conditions, recommended by the MBIE, are presented in Table 1 of this report.

As a result of further research undertaken by Boulanger & Idriss (2014) - B&I (2014), which takes into account case history data from the Christchurch area, a new formulation for the determination of the Magnitude Scaling Factor (MSF) has been developed, which takes into account the nature and density of the soils. The new formulation has negligible effect on the determination of the MSF for ULS design strength earthquake events but can have a significant effect on the MSF determined for sites under loading from an SLS design earthquake event. For this reason, the MBIE guidelines (Update No. 50, dated October 2014) recommends, when undertaking analyses using the B&I (2014) method of analyses, that an "intermediate" design strength earthquake (ILS) also be analysed when predicting the expected liquefaction triggering and associated ground settlements for the SLS design earthquake event.

It is recommended by the MBIE guidelines that the larger theoretical index settlement value calculated using the earthquake loading parameters for the SLS and ILS design earthquake events be used as the theoretical SLS index settlement value, when assessing the theoretical liquefaction potential for sites in Christchurch.

TABLE 3: DESIGN PEAK GROUND ACCELERATION (PGA) VALUES FOR ASSUMED DESIGN CONDITIONS

Design Condition	Design Peak Ground Acceleration (pga) (proportion of gravity acceleration (m/s^2))	Earthquake Moment Magnitude (Mw)
SLS	0.13g	7.5
ULS	0.35g	7.5
ILS	0.19g	6.0

10.3 METHOD OF ANALYSES

10.3.1 General

The MBIE guidance document (2012) recommends that the theoretical settlement "index number" is calculated using the following methodology:

(i) assessing liquefaction induced settlement only for the upper 10 m of subsoils under SLS seismic load conditions,

(ii) using the liquefaction potential assessment methods suggested by Idriss & Boulanger (2008),

The MBIE guidelines (Update No. 50, dated October 2014), also allows the liquefaction triggering analyses of sites in Christchurch to be undertaken using the deterministic methodology suggested by Boulanger & Idriss (2014).

The research paper prepared by B&I (2014) provides an update to the CPT database case histories and updates the CPT-based liquefaction triggering correlations, based on new information obtained from sites in the Christchurch area.

Because the deterministic methodology, suggested by B&I (2014), takes into account additional case history data obtained from sites in Christchurch, it is our opinion that the 2014 methodology will likely provide more reliable predictions of liquefaction triggering and associated ground settlements for sites in Christchurch than the 2008 methodology. For this reason, we have adopted the methodology suggested by Boulanger & Idriss (2014) for the analyses reported herein.

It should also be noted that the Module 3 guidelines also recommends using the Boulanger & Idriss (2014) methodology, for determining theoretical liquefaction triggering.

10.3.2 Fines Content Correlations

B&I (2014) states the following:

> *"The revised CPT-based liquefaction triggering procedure* [i.e. the B&I- 2014 methodology] *included a recommend relationship and approach for estimating FC and soil classification from the I_c index when site specific sampling and lab testing data are not available. For analyses in the absence of site-specific soil sampling and lab testing data, it would be prudent to perform parametric analyses to determine if reasonable variations in the FC and soil classification parameters have a significant effect on the final engineering recommendation."*

B&I (2014) goes on to recommend that a sensitivity analyses be undertaken, varying the C_{FC} (fitting parameter), for the FC-I_c correlations.

Lees, et al (2015) used the results of an extensive geotechnical investigation dataset collected following the 2010/2011 Canterbury earthquake sequence to examine the correlations of the liquefaction susceptibility and FC with I_c for the Christchurch soils.

Borehole and CPT data were used to assess the appropriateness of the FC-Ic correlations, presented in B&I (2014), as well as the I_c cut-off threshold. The results of the study indicates, for Christchurch soils, that the default C_{FC} value of 0.0 will generally over-predict liquefaction triggering, and that a C_{FC} parameter of 0.2 is appropriate for Christchurch soils.

10.3.3 Thin Sand Layer "Transition Zones"

Robertson, Idriss and Boulanger et al recognise that the reliability of CPT based theoretical liquefaction triggering analyses, can be affected by an effect known as the "thin sand layer" transition zone. This occurs because the CPT probe provides readings from a soil influence zone, which is located some distance in front of the cone tip (the influence zone varies with soil types), which can underestimate the cone resistance of sand layers (particularly when sandwiched between soft cohesive soil layers), which can consequently incorrectly estimate liquefaction triggering for some layered sandy soils.

PK Robertson (2009) provides a method for adjusting for this effect in the CLiq program. The adjustment is based on the rate of change of the Soil Behaviour Type Index (I_c).

We have used the thin sand layer adjustment methodology or “transition zone” adjustment methodology suggested by PK Robertson, in the analyses reported herein.

10.3.4 Summary

The input parameters used for the theoretical liquefaction triggering analyses reported herein are summarised in Table 4.

TABLE 4: INPUT PARAMETERS FOR LIQUEFACTION ANALYSES

Input parameter	Value adopted	Comments
Design Seismic Loading	See Table 1	See Section 9.2
I_c cut-off	2.6	Appropriate value for Christchurch (Lees, et al)
Probability of Liquefaction (P_L)	16%	Deterministic value- in accordance with B&I (2014)
FC Fitting Parameter C_{FC}	Range (0.0 to 0.2)	Sensitivity analyses undertaken

10.4 RESULTS OF ANALYSES

As discussed in Section 10.3.2 of this report, the results of the study undertaken by Lees, et al, indicates, for Christchurch soils, that the default C_{FC} value of 0.0 will generally over-predict liquefaction triggering, and that a C_{FC} parameter of 0.2 is appropriate for Christchurch soils.

For the purposes of the theoretical liquefaction analyses reported herein, a sensitivity analyses has been undertaken to more reliably determine the FC-I_c correlation. The sensitivity analyses has been undertaken assuming the following C_{FC} values: 0.0, 0.1 and 0.2.

The results of the liquefaction analyses undertaken for the site for the SLS, ILS and ULS design earthquake events are summarised in Tables 5, 6 and 7 respectively.

TABLE 5: THEORETICAL EXPECTED GROUND SETTLEMENTS FOR SERVICEABILITY LIMIT STATE (SLS) DESIGN EARTHQUAKE EVENT

CPT probe	**Range of theoretical expected ground settlement (mm)-** **(C_{FC}= 0.0 to 0.2)**	**Mean theoretical expected ground settlement (mm)-** **(C_{FC}= 0.0 to 0.2)**
3	0	0
4	0	0
5	0	0

TABLE 6: THEORETICAL EXPECTED GROUND SETTLEMENTS FOR INTERMEDIATE LIMIT STATE (ILS) DESIGN EARTHQUAKE EVENT

CPT probe	**Range of theoretical expected ground settlement (mm)-** **(C_{FC}= 0.0 to 0.2)**	**Mean theoretical expected ground settlement (mm)-** **(C_{FC}= 0.0 to 0.2)**
3	0	0
4	0	0
5	0	0

TABLE 7: THEORETICAL EXPECTED GROUND SETTLEMENTS FOR ULTIMATE LIMIT STATE (ULS) DESIGN EARTHQUAKE EVENT

CPT probe	Range of theoretical expected ground settlement (mm)- (C_{FC}= 0.0 to 0.2)	Mean theoretical expected ground settlement (mm)- (C_{FC}= 0.0 to 0.2)
3	0	0
4	0	0
5	0	0

The results of the analyses are presented in Appendix B of this report.

It should be noted that the theoretical liquefaction induced ground settlements presented in Tables 5, 6 and 7, for the soils encountered at the locations of CPT3, CPT4 and CPT5, have only been obtained for the analyses of the upper 3.8 m depth of the subsoils (maximum), due the CPT probes being unable to be progressed through the sandy gravels.

It is conventionally acceptable to analyse the upper 10 m of the subsoils when assessing the potential liquefaction induced ground settlements that could be expected to affect a site, in accordance with the MBIE guidelines. It is possible that liquefiable soils are located beneath the upper 3.8 m depth of the subsoils, which could increase the theoretical liquefaction induced ground settlements for the site. For this reason, the theoretical settlement values presented in Tables 5, 6 and 7, could be considered to be conservatively low.

That been said, it is unlikely, in our opinion, that any significant liquefaction induced ground settlement would occur as a result of liquefaction of the dense to very dense sandy gravels (i.e. the layer located below the upper 3.8 m of soils), as these soils, due to their nature, are generally not expected to be liquefiable.

11.0 LIQUEFACTION SEVERITY NUMBER (LSN)

Following the Canterbury Earthquake Sequence (CES), S. van Ballegooy, et al (2013) developed an unweighted assessment methodology, to assess the vulnerability of land to liquefaction-induced damage. The methodology suggests the use of a dimensionless number termed the Liquefaction Severity Number (LSN).

The LSN, is defined as:

$$LSN = 1000 \int \frac{\varepsilon_v}{z} dz$$

- where ε_v is the calculated post-liquefaction volumetric reconsolidation strain, and z is the depth below the ground surface in metres. The LSN is calculated over the upper 10 m depth profile of the subsoil.

The theoretical value of LSN varies from 0 (representing no liquefaction vulnerability) to more than 100 (representing very high liquefaction vulnerability).

S. van Ballegooy, et al (2013) suggest a range of LSN values, which relate to three categories of expected degree of liquefaction-induced ground damage, namely:

(i) None to minor

(ii) Minor to moderate

(iii) Moderate to severe.

The suggested range of LSN values for each ground damage category, are presented in Table 8.

TABLE 8: LSN RANGES- CORRESPONDING TO EXPECTED LIQUEFACTION-INDUCED GROUND DAMAGE

LSN	Expected liquefaction-induced ground damage category
0 - 20	None to minor
20 - 40	Minor to moderate
40+	Moderate to severe

The typical consequences at the ground surface, for the various categories presented in Table 8 are described in Table 2.2 of the MBIE guidance document, titled "Planning and Engineering Guidance for Potentially Liquefaction Prone Land", dated September 2017.

For the purposes of the theoretical liquefaction analyses reported herein, and in order to determine the LSN values for the various design earthquake events, a sensitivity analyses has been undertaken assuming the following C_{FC} values: 0.0, 0.1 and 0.2.

The mean LSN value has been calculated from the sensitivity analyses, for the SLS, ILS and ULS design earthquake events, and has been adopted for the site. The results of the analyses to determine the LSN values are presented in Tables 9, 10 and 11.

TABLE 9: MEAN LSN VALUE- FOR SERVICEABILITY LIMIT STATE (SLS) DESIGN EARTHQUAKE EVENT

CPT probe	Mean LSN value	Expected liquefaction-induced ground damage category
3	0	None to minor
4	0	None to minor
5	0	None to minor

TABLE 10: MEAN LSN VALUE- FOR INTERMEDIATE LIMIT STATE (ILS) DESIGN EARTHQUAKE EVENT

CPT probe	Mean LSN value	Expected liquefaction-induced ground damage category
3	0	None to minor
4	0	None to minor
5	0	None to minor

TABLE 11: MEAN LSN VALUE- FOR ULTIMATE LIMIT STATE (ULS) DESIGN EARTHQUAKE EVENT

CPT probe	Mean LSN value	Expected liquefaction-induced ground damage category
3	0	None to minor
4	0	None to minor
5	0	None to minor

Based on the results of the investigation and appraisal reported herein (and as indicated in Tables 9 and 10 of this report), it is our opinion that the liquefaction-induced ground damage expected to occur at the site, in response to a SLS design earthquake event, is considered to be none to minor.

Based on the results of the investigation and appraisal reported herein (and as indicated in Table 11 of this report), it is our opinion that the liquefaction-induced ground damage expected to occur at the site, in response to a ULS design earthquake event, is considered to also be none to minor.

It should be noted that the foregoing LSN values for the site are consistent with the observed performance of the site during the 2010/2011 Canterbury earthquake sequence. The September 2010 and February 2011 earthquake events are expected to have imposed seismic loading at the subject site, which approximates the ULS and SLS design loadings respectively. However, no obvious liquefaction ejecta was observed at the subject site, in response to these earthquake events, which is consistent with the behaviour, as predicted by the calculated LSN values.

12.0 FOUNDATION TECHNICAL CATEGORY FOR THE SITE

The Ministry of Business, Innovation & Employment (MBIE) released a document entitled “Repairing and rebuilding houses affected by the Canterbury earthquakes”; Version 3, dated December 2012.

It should be noted that the MBIE guidance document supersedes the following previous Department of Building and Housing (DBH) and MBIE documents:

(a) “Revised guidelines on repairing and rebuilding houses affected by the Canterbury earthquake sequence”, dated November 2011,

(b) “Interim guidance for repairing and rebuilding foundations in Technical Category 3”, dated 27 April 2012,

(c) “Guidelines for the geotechnical investigation and assessment of subdivisions in the Canterbury region”

The principal objective of the MBIE guidance document is to provide building repair and reconstruction solutions and options that:

(i) are appropriate to the level of land and building damage experienced;

(ii) take account of the likely future performance of the ground;

(iii) meet Building Act and Building Code requirements; and

(iv) are acceptable to insurers and property owners.

The document also divides the previous CERA "Green Zone" on flat land, into three technical categories that reflect both the liquefaction experienced to date, and future performance expectations. The Foundation Technical Categories are identified as TC1, TC2 and TC3.

Table 3.1 of the MBIE guidance document provides expected future land performance for the various Foundation Technical Categories. These are summarised in Table 12 of this report.

TABLE 12: EXPECTED FUTURE LAND PERFORMANCE FOR VARIOUS FOUNDATION TECHNICAL CATEGORIES

Foundation Technical Category	Future Land Performance Expectation in Response to Liquefaction	Expected Ground Settlement in Response to an SLS Strength Earthquake	Expected Ground Settlement in Response to a ULS Strength Earthquake
TC1 (where confirmed)	Liquefaction damage is unlikely in a future large earthquake	0 – 15 mm	0 - 25 mm
TC2 (where confirmed)	Liquefaction damage is possible in a future large earthquake	0 – 50 mm	0 – 100 mm
TC3 (where confirmed)	Liquefaction damage is possible in a future large earthquake	>50 mm	>100 mm

The MBIE guidance document states that:

> *"In order to characterise the potential behaviour of the site and to effectively subdivide the TC3 land into 'less' and 'more vulnerable' categories an 'index number' for TC3 properties has been developed. This index reflects the consequential effects of settlement taking into account the behaviour of the shallower soils being more influential than that of deeper soils."*

Table 12.5 of the guidance document provides categories of vertical land settlement, for the calculated "index number" theoretical ground settlement. The guidance document suggests that for sites with SLS "index number" theoretical ground settlements of less than 100 mm the land settlement should be assumed to be "minor to moderate". For sites with SLS "index number" theoretical ground settlements of greater than 100 mm the land settlement should be assumed to be "potentially significant".

Using the methodology recommended by the MBIE guidance document and described in Section 10.4 of this report, an "index number" theoretical ground settlement value of 0 mm has been calculated for the site soils under the assumed SLS seismic loading (which was the larger value determined for the SLS and ILS design earthquake events, as discussed in Section 10.2 of this report).

An "index number" theoretical ground settlement value of 0 mm has been calculated for the site soils under ULS seismic loading.

The results of the CLiq analyses, using the analyses methods suggested by the MBIE guidance document, are presented in Appendix B of this report.

The foregoing "index number" theoretical ground settlement values indicate that the site has the liquefaction potential characteristics of a Foundation Technical Category 1 (TC1) site. However, as discussed in Section 10.4 of this report, the CPT probes put down at the site were unable to penetrate to a sufficient depth in order to reliably determine the theoretical liquefaction induced ground settlements expected to occur at the site in response to the SLS and ULS design earthquake events.

However, as discussed in Section 8.3.3, there is existing ground investigation data available for the site which indicates that the gravel soils extend to significant depths beneath the subject site. It is unlikely, in our opinion, that any significant liquefaction induced ground settlement would occur as a result of liquefaction of the dense to very dense sandy gravels (i.e. the layer located below the upper 3.8 m of soils), as these soils, due to their nature, are generally not expected to be liquefiable.

Based on the foregoing, and given the nature of the upper soils underlying the site, i.e. generally dense to very dense gravel soils, the depth to groundwater (expected to be no shallower than 2.5 m), and the observed performance of the site in response to seismic loading imposed by the 2010/2011 Canterbury earthquake sequence (i.e. no obvious liquefaction ejecta observed at the site for seismic loading expected to approximate the SLS and ULS design loadings), it is our opinion that the subject site, for the purposes of the submission on the Selwyn District Plan Review and the private plan change request, should be assumed to be within Foundation Technical Category 1 (TC1), as defined by the MBIE guidance document, and that it is unlikely that liquefaction induced ground deformation could occur within the area in response to a large earthquake event, and that the ground settlements within the area in response to seismic loading should be considered to be "within normally accepted tolerances" as defined by the MBIE December 2012 guidance document.

It should be noted that Figure 3.1c of the MBIE guidelines indicates that the western part of the subject site is sited within the “Rural & Unmapped” zone, as defined by the MBOE guidelines, and that generally the eastern part of the subject site is located within Foundation Technical Category 2 (TC2). It is also noted that the area located immediately to the north-west of the subject site, is shown on Figure 3.1c, as being within a Foundation Technical Category 1 (TC1) zone, as defined by the MBIE guidelines.

It is our opinion that the TC1 site classification which has been provided for the subject site in this report, which is based on the results of site specific geotechnical investigation and appraisal works, provides a more reliable indication of the theoretical liquefaction potential characteristics of the subject site than the TC2 classifications provided by the MBIE guidelines (for the eastern part of the subject site), and that the TC1 classification is consistent with the classification provided by the MBIE guidelines for the area located immediately to the north-west of the subject site.

13.0 SUITABLE SHALLOW FOUNDATIONS FOR TC1 SITES, AS SUGGESTED BY THE MBIE GUIDANCE DOCUMENT

The MBIE guidance document provides guidance for foundation repairs and reconstruction for houses within Foundation Technical Category 1 (TC1).

The document states the following with regard to new foundation construction within the TC1 zone:

> *“In TC1, foundation Types A* [suspended timber floor supported on piles] *and B* [suspended timber floor supported on piles with a perimeter foundation wall] *can be built as per NZS 3604. Type C foundations* [concrete slab on ground flooring system] *will require reinforced concrete slabs as provided in NZS 3604 Timber Framed Buildings, as modified by B1/AS1, which requires ductile reinforcing in slabs.”*

14.0 FOUNDATION DESIGN CONSIDERATIONS

14.1 GENERAL

It is our opinion that the soils underlying the subject site will exhibit only a low compressibility under the relatively light static foundation loads associated with a residential building development constructed in accordance with the requirements of NZS 3604: 2011, New Zealand Standard, Timber Framed Buildings.

It is, therefore, our opinion that settlement should not present a problem for future proposed residential development at the site, providing the inspection and design of foundations are carried out in accordance with the requirements of the relevant New Zealand Standard Codes of Practice, and in accordance with the recommendations presented in this report.

Notwithstanding, it is anticipated that a site specific geotechnical investigation will be required to be undertaken, for any new building proposed to be constructed at the subject site, in support of an application for building consent.

14.2 AREAS INFERRED TO BE OVERLAIN BY STOCKPILE MATERIAL

As discussed in Section 7.2 of this report, two existing stockpiles of material were observed in the south-eastern corner of the site. The stockpiles are inferred to range between approximately 1.5 m and 3.5 m in vertical height and appear to comprise a mixture of silt, sand, gravel, and concrete block material. The origin of this material is unknown.

The approximate inferred locations and extent of the existing stockpiles are shown on drawing G00417-01.

There is in our opinion a risk, if the stockpile material is not appropriately removed from the site, that foundations and floors underlain by stockpile material may be subject to differential movement.

It is therefore recommended that any foundation excavations associated with any new structure be founded beneath any surficial stockpile material into the underlying natural alluvial sediments. It is also recommended that any surficial stockpile material be undercut from beneath the footprint of any proposed building.

It is recommended that Fraser Thomas Ltd be engaged to inspect, in particular, any foundation excavations and building subgrades located within the areas inferred to be underlain by stockpile material, as shown on drawing G00417-01, in order to confirm that stockpile material has been removed from beneath the footprint of any proposed new building.

14.3 THE RISK OF THE PROPOSED DEVELOPMENT BEING ADVERSELY AFFECTED BY GROUND DEFORMATIONS ASSOCIATED WITH LIQUEFACTION

As discussed in Section 12.0 of this report, it is our opinion that the subject area should be assumed to be within Foundation Technical Category 1 (TC1), as defined by the December 2012 guidance document, and that it is unlikely that liquefaction induced ground deformation could occur at the site in response to a large earthquake event, and that the ground settlements at the site in response to seismic loading should be considered to be “within normally accepted tolerances” as defined by the MBIE December 2012 guidance document.

Based on the results of the investigations and appraisal reported herein, it is our opinion that an appropriate foundation solution for the site conditions would be a shallow foundation system designed in accordance with the requirements of NZS 3604: 2011 (as modified by B1/AS1), and in accordance with the recommendations presented in this report.

It is recommended that any proposed shallow foundations be founded beneath the surficial topsoil or unsuitables into the underlying alluvial sediments.

Fraser Thomas Ltd should be engaged to inspect any foundation excavations, prior to the placement of any foundation materials, in order to confirm that the excavations are founded in competent natural ground.

15.0 ALLOWABLE FOUNDATION BEARING PRESSURES

15.1 GENERAL

In this section of the report, ultimate bearing capacity values and strength reduction factors are provided in order to allow calculation of design (dependable) foundation bearing capacities, in accordance with the limit state design methods outlined in AS/NZS 1170: 2002, Structural Design Actions, by applying the appropriate strength reduction factors, as provided in this report, and the factored load combinations required by AS/NZS 1170. Allowable foundation bearing pressures are also provided, based on conventional factors of safety, for cases where unfactored load combinations are being considered.

15.2 SHALLOW PAD OR BEAM FOUNDATIONS

A minimum ultimate static bearing capacity value for vertical loading of 300 kPa is recommended for shallow concrete pads or beam foundations, founded in the underlying alluvial sediments. It is recommended that a strength reduction factor (Φ_{bc}) of 0.5 be adopted for limit state design in accordance with the requirements of AS/NZS 1170, resulting in a design (dependable) bearing capacity value of 150 kPa.

If unfactored load combinations are to be considered, the allowable foundation bearing pressures presented in Table 13 are recommended for shallow concrete pads or beam foundations, founded in the underlying alluvial sediments.

TABLE 13: ALLOWABLE FOUNDATION BEARING PRESSURES FOR SHALLOW CONCRETE PADS OR BEAM FOUNDATIONS

Load Case	Factor of Safety	Allowable Bearing Pressure (kPa)
Dead Load and Permanent Live Load	3.0	100
Dead plus Live plus Transient Load	2.0	150

16.0 EXISTING SERVICE LINES

It is recommended that the location and depth of any buried services should be verified at the site prior to the commencement of foundation construction.

It is expected that any service line trenches would have been backfilled by conventionally acceptable means, which did not involve specific compaction. It would therefore be expected that some consolidation settlement of the service trench backfill could occur, which could result in lateral and vertical deformation of the undisturbed ground on each side of the trench backfill. The deformation is caused by the soil wedge behind the side wall of the trench moving downwards and inwards with time, towards the trench backfill as the backfill consolidates. The geometry of the soil wedge defines the theoretical zone of influence of the service trench backfill.

Due to the risk of consolidation settlement of the trench backfill occurring, it is recommended that, if any foundations of any proposed new building are located within the zone of influence of any existing service line, either the trench backfill be excavated and replaced with compacted hardfill or the foundations and floor of the proposed new building be designed to span across the trench backfill and the adjacent zone of influence.

The zone of influence is defined by a theoretical line projecting upwards in both directions from the centreline of the pipeline at the invert level of the pipeline at an angle of 45° to the vertical. The zone of influence is defined by the zone between the intersection point of the theoretical line and the ground surface on each side of the pipeline.

17.0 DEVELOPMENTAL EARTHWORKS

It is recommended that, unless the stability of any developmental earthworks (i.e. constructed for an access driveway, building platform or landscaping) is considered in detail by a chartered professional engineer experienced in geotechnical engineering, and particularly slope stability considerations, permanent fill end and cut slopes should be constructed to a maximum batter slope of 26° (1V:2H) with maximum batter heights of approximately 1.0 m. Any proposed higher permanent batter slopes should be subject to specific stability appreciation so as to determine stable limiting batter slopes.

It is recommended that any temporary excavated slopes be constructed to a maximum batter slope of 45^{o} (1V:1H), with a maximum batter height of approximately one meter. It is recommended that any temporary excavation slopes not be left unsupported for a period exceeding one month. It is also recommended that stormwater run-off be diverted away from the crest of any proposed temporary excavation slopes.

18.0 STORMWATER AND EFFLUENT DISPOSAL

It is understood that issues relating to stormwater discharge and effluent disposal will be addressed by others.

19.0 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations should be read together and not be taken in isolation.

19.1 CONCLUSIONS

Our conclusions based on the field data obtained from the site and as presented in this report, our visual appraisal of the site, our study of the geological maps relating to the area and our professional judgement and opinions, are as follows:

(a) In general terms and within the limits of the investigation as outlined and reported herein, no unusual problems, from a geotechnical perspective, are anticipated with residential development at the subject site.

The site is, in general, considered suitable for its intended use, with satisfactory conditions for future residential building development, subject to the recommendations and qualifications reported herein, and provided the design and inspection of foundations are carried out as would be done under normal circumstances in accordance with the requirements of the relevant New Zealand Standard Codes of Practice.

In arriving at this conclusion and expressing this opinion, reliance has been based on the various topographical data as discussed herein and on subsoil information which has only been obtained at the locations and within the depths of the test positions reported herein. It has been assumed that this subsoil information can be projected between the various test positions. Even though such inference is made and forms the basis of the conclusions and opinions expressed herein, no guarantee can be given as to the validity of this inference or of the nature and continuity of the subsoils underlying the proposed subdivision.

(b) The purpose of the geotechnical investigation reported herein was to determine the subsoil conditions beneath the subject site as they may affect future residential development, with particular regard to foundation considerations, and to determine the suitability of the subject site for the residential development, in support of the submissions discussed in Section 1.0 of this report.

(c) Two existing stockpiles of material were observed in the south-eastern corner of the site. The stockpiles are inferred to range between approximately 1.5 m and 3.5 m in vertical height and appear to comprise a mixture of silt, sand, gravel, and concrete block material. The origin of this material is unknown. The approximate inferred locations and extent of the existing stockpiles are shown on drawing G00417-01.

(d) The results of the hand augered borehole and CPT probe investigations, undertaken in the southern part of the subject site, indicate that the surficial topsoil is generally underlain by an upper layer of clayey silts, inferred to be alluvial sediments of the Springston Formation. The sediments were encountered to depths ranging between approximately 0.6 m and 0.8 m below the existing ground surface, at the locations of Boreholes H2, H4, H6 and H7, corresponding to a layer thickness of between approximately 300 mm and 400 mm.

(e) The results of the hand augered borehole and CPT probe investigations, undertaken in the southern part of the subject site, indicate that the cohesive soils are generally underlain by a layer of material inferred to comprise silty sands and sands. The sediments were encountered to depths ranging between approximately 0.9 m and 1.3 m below the existing ground surface, corresponding to a layer thickness of between approximately 100 mm and 600 mm, at the locations of Boreholes H2, H4 and H7.

(f) The results of the investigations reported herein indicate that the surficial soils at the site are generally underlain by a layer of material, inferred to comprise sandy gravels and gravelly sands. The CPT probes and hand augered boreholes were not able to be progressed through these soils. The CPT probes generally obtained cone resistance (q_t) values of generally between approximately 20 MPa and greater than 40 MPa in these sediments, corresponding to a dense to very dense consistency.

(g) The logs of existing water bore logs, put down within the subject site, have also been sourced from Environment Canterbury records. The existing water bore logs indicate that sandy gravels are generally located at shallow depths, which is consistent with the subsoil conditions encountered at the subject site. The bore logs indicate that these sandy gravels generally extend to depths of greater than approximately 18 m below the ground surface. Based on the foregoing, it is, in our opinion, likely that the gravel soils underlying the site extend to significant depths below the existing ground surface.

(h) Based on the results of the borehole investigations undertaken at the site, and ground investigation information obtained from the NZGD, the groundwater level is inferred to be at a depth of approximately 2.5 m below the existing ground surface, for analysis purposes.

(i) Analyses have been undertaken using the computer program CLiq to assess the theoretical liquefaction triggering potential and expected ground settlements for the soils underlying the subject site. The analyses have been undertaken for the subsoil profiles obtained at the locations of CPT3, CPT4 and CPT5.

Using the methodology recommended by the MBIE guidance document and described in Section 10.4 of this report, an “index number” theoretical ground settlement value of 0 mm has been calculated for the site soils under the assumed SLS seismic loading (which was the larger value determined for the SLS and ILS design earthquake events, as discussed in Section 10.2 of this report).

An “index number” theoretical ground settlement value of 0 mm has been calculated for the site soils under ULS seismic loading.

The results of the CLiq analyses, using the analyses methods suggested by the MBIE guidance document, are presented in Appendix B of this report.

(j) Given the nature of the upper soils underlying the site, i.e. generally dense to very dense gravel soils, the depth to groundwater (expected to be no shallower than 2.5 m), and the observed performance of the site in response to seismic loading imposed by the 2010/2011 Canterbury earthquake sequence (i.e. no obvious liquefaction ejecta observed at the site for seismic loading expected to approximate the SLS and ULS design loadings), it is our opinion that the subject site, for the purposes of the submission on the Selwyn District Plan Review and the private plan change request should be assumed to be within Foundation Technical Category 1 (TC1), as defined by the MBIE guidance document, and that it is unlikely that liquefaction induced ground deformation could occur within the area in response to a large earthquake event, and that the ground settlements within the area in response to seismic loading should be considered to be "within normally accepted tolerances" as defined by the MBIE December 2012 guidance document.

It should be noted that Figure 3.1c of the MBIE guidelines indicates that the western part of the subject site is sited within the "Rural & Unmapped" zone, as defined by the MBIE guidelines, and that generally the eastern part of the subject site is located within Foundation Technical Category 2 (TC2). It is also noted that the area located immediately to the north-west of the subject site, is shown on Figure 3.1c, as being within a Foundation Technical Category 1 (TC1) zone, as defined by the MBIE guidelines.

It is our opinion that the TC1 site classification which has been provided for the subject site in this report, which is based on the results of site specific geotechnical investigation and appraisal works, provides a more reliable indication of the theoretical liquefaction potential characteristics of the subject site than the TC2 classifications provided by the MBIE guidelines (for the eastern part of the subject site), and that the TC1 classification is consistent with the classification provided by the MBIE guidelines for the area located immediately to the north-west of the subject site.

(k) It is our opinion that the soils underlying the subject site will exhibit only a low compressibility under the relatively light static foundation loads associated with a residential building development constructed in accordance with the requirements of NZS 3604: 2011, New Zealand Standard, Timber Framed Buildings.

It is, therefore, our opinion that settlement should not present a problem for future proposed residential development at the site, providing the inspection and design of foundations are carried out in accordance with the requirements of the relevant New Zealand Standard Codes of Practice, and in accordance with the recommendations presented in this report.

Notwithstanding, it is anticipated that a site specific geotechnical investigation will be required to be undertaken, for any new building proposed to be constructed at the subject site, in support of an application for building consent.

(l) It is our opinion that an appropriate foundation solution for the site conditions would be a shallow foundation system designed in accordance with the requirements of NZS 3604: 2011 (as modified by B1/AS1), and in accordance with the recommendations presented in this report.

19.2 RECOMMENDATIONS

Our recommendations based on the field data obtained from the site and as presented in this report, our visual appraisal of the site, our study of the geological maps relating to the area and our professional judgement and opinions, are as follows:

(a) It is recommended that Fraser Thomas Ltd be engaged to inspect, in particular, any foundation excavations and building subgrades located within the areas inferred to be underlain by stockpile material, as shown on drawing G00417-01, in order to confirm that stockpile material has been removed from beneath the footprint of any proposed new building.

(b) It is recommended that any proposed shallow foundations be founded beneath the surficial topsoil or unsuitables into the underlying alluvial sediments.

(c) A minimum ultimate static bearing capacity value for vertical loading of 300 kPa is recommended for shallow concrete pads or beam foundations, founded in the underlying alluvial sediments. It is recommended that a strength reduction factor (Φ_{bc}) of 0.5 be adopted for limit state design in accordance with the requirements of AS/NZS 1170, resulting in a design (dependable) bearing capacity value of 150 kPa.

If unfactored load combinations are to be considered, the allowable foundation bearing pressures presented in Table 13 are recommended for shallow concrete pads or beam foundations, founded in the underlying alluvial sediments.

(d) It is recommended that the location and depth of any buried services should be verified at the site prior to the commencement of foundation construction.

Due to the risk of consolidation settlement of the trench backfill occurring, it is recommended that, if any foundations of any proposed new building are located within the zone of influence of any existing service line, either the trench backfill be excavated and replaced with compacted hardfill or the foundations and floor of the proposed new building be designed to span across the trench backfill and the adjacent zone of influence.

(e) It is recommended that, unless the stability of any developmental earthworks (i.e. constructed for an access driveway, building platform or landscaping) is considered in detail by a chartered professional engineer experienced in geotechnical engineering, and particularly slope stability considerations, permanent fill end and cut slopes should be constructed to a maximum batter slope of 26° (1V:2H) with maximum batter heights of approximately 1.0 m. Any proposed higher permanent batter slopes should be subject to specific stability appreciation so as to determine stable limiting batter slopes.

(f) It is recommended that any temporary excavated slopes be constructed to a maximum batter slope of 45^{o} (1V:1H), with a maximum batter height of approximately one meter. It is recommended that any temporary excavation slopes not be left unsupported for a period exceeding one month. It is also recommended that stormwater run-off be diverted away from the crest of any proposed temporary excavation slopes.

20.0 LIMITATIONS

The professional opinion expressed herein has been prepared solely for, and is furnished to our client, Trices Road Rezoning Group, and Selwyn District Council for their purposes only with respect to the particular brief given to us, on the express condition that it will not be relied upon by any other person or for any other purposes without our prior written agreement, and relates to the conditions that exist up to and at the time of this report.

No liability is accepted by this firm or by any principal, or director, or any servant or agent of this firm, in respect of the use of this report by any other person, and any other person who relies upon any matter contained in this report does so entirely at its own risk. This disclaimer shall apply notwithstanding that this report may be made available to any person by any person in connection with any application for permission or approval, or pursuant to any requirement of law.

This report does not comment on stormwater management, flooding, root effects and land uses outside the specific site, which may be required to be assessed to complete a foundation design for building consent application purposes.

Notwithstanding the foregoing, if the circumstances at the subject site change with respect to topography or the proposed development concept, or the buildings are subject to further damaging earthquakes, or if a period of more than three years has elapsed since the date of this report, this report should not be used without our prior review and written agreement.

The conclusions and recommendations expressed herein should be read in conjunction with the remainder of this report and should not be referred to out of context with the remainder of this report.

Report prepared by:
FRASER THOMAS LTD.

J T GRAHAM
Geotechnical Engineer

Report reviewed and approved by:

M V REED
Director
Chartered Professional Engineer

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Appendix A

Field Investigation Results

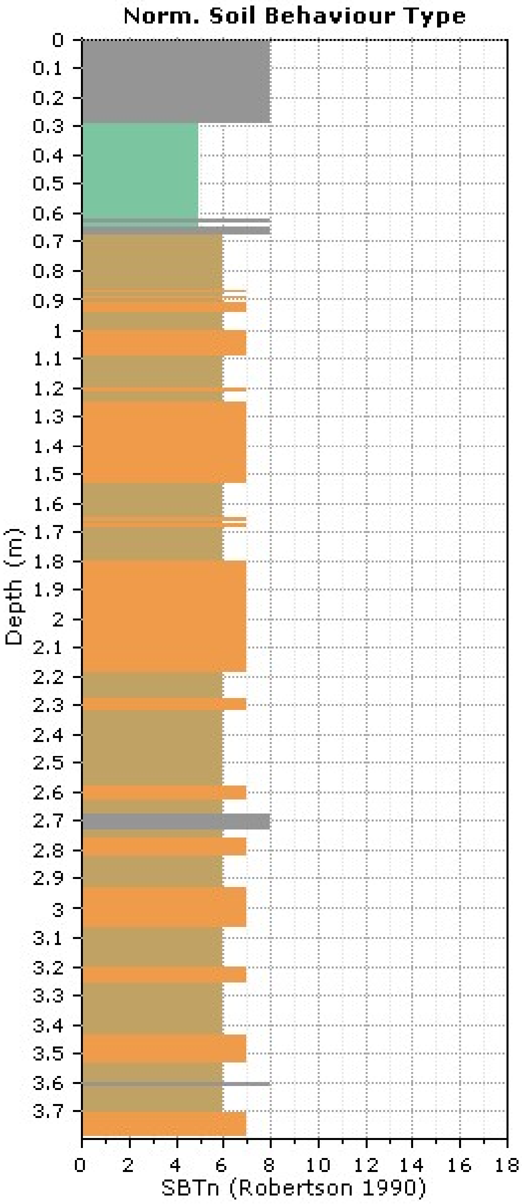
Cone Penetration Test (CPT) Results

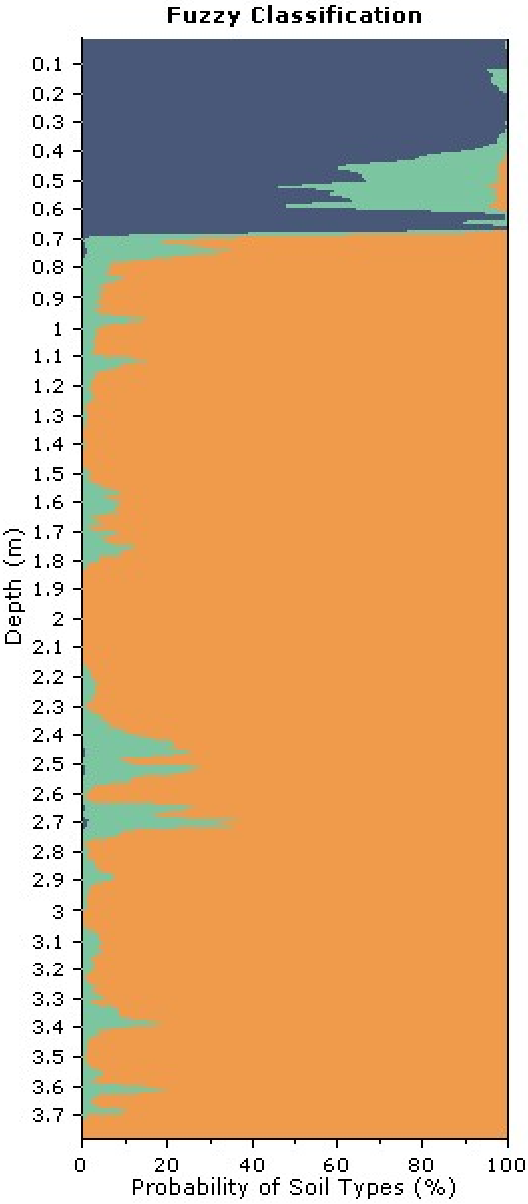
CPT 3

Unit 3a
Barry Hogan Place
Riccarton 8041

CPT: CPT3
Total depth: 3.78 m, Date: 2/07/2020
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

Project: **Trices Road Rezoning Group**
Location: **Trices Road Rezoning Area**





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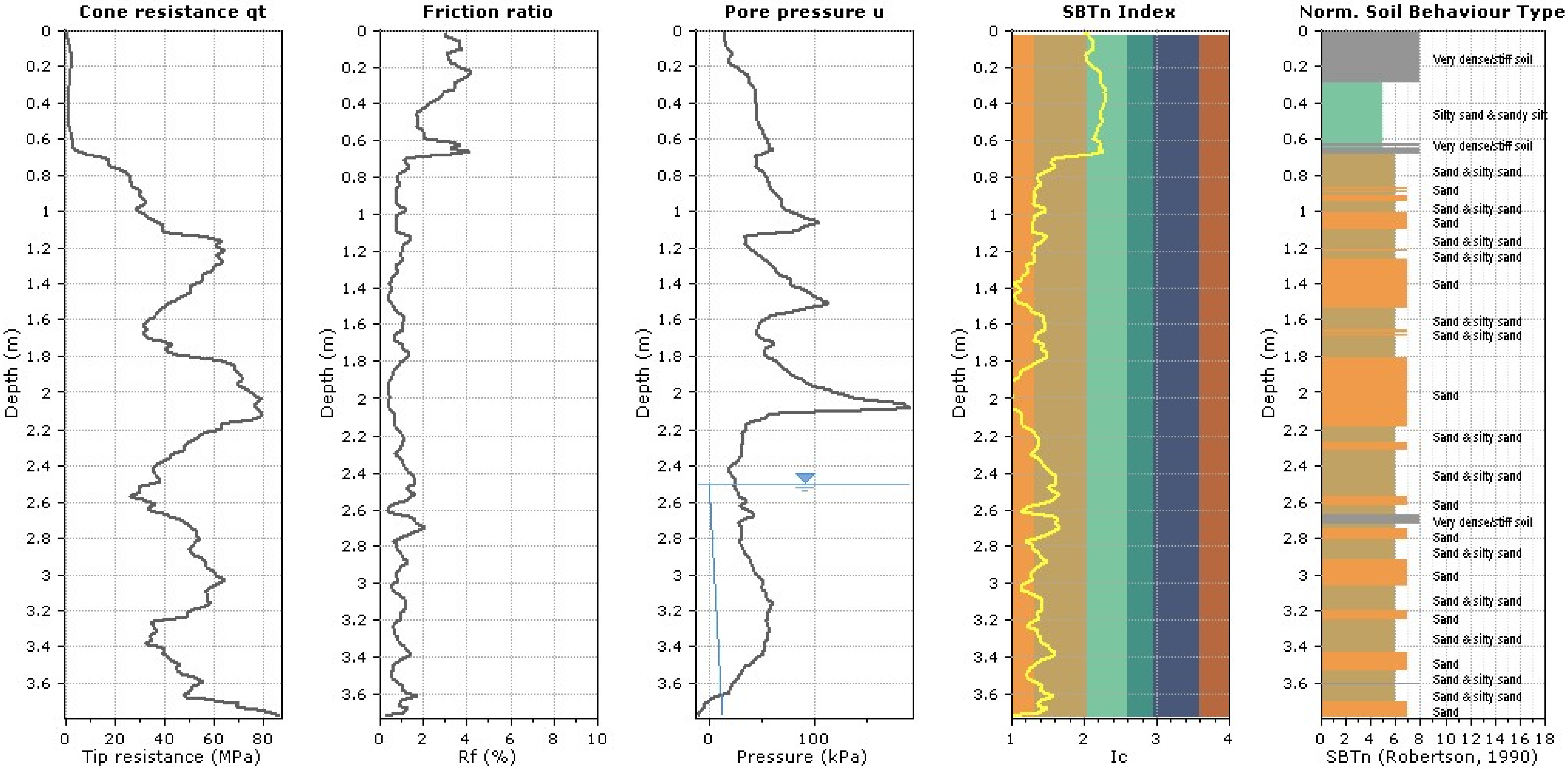
Unit 3a
Barry Hogan Place
Riccarton 8041

Project: **Trices Road Rezoning Group**
Location: **Trices Road Rezoning Area**

CPT: CPT3
Total depth: 3.78 m, Date: 2/07/2020
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown



SBTn legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravelly sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

CPT 4

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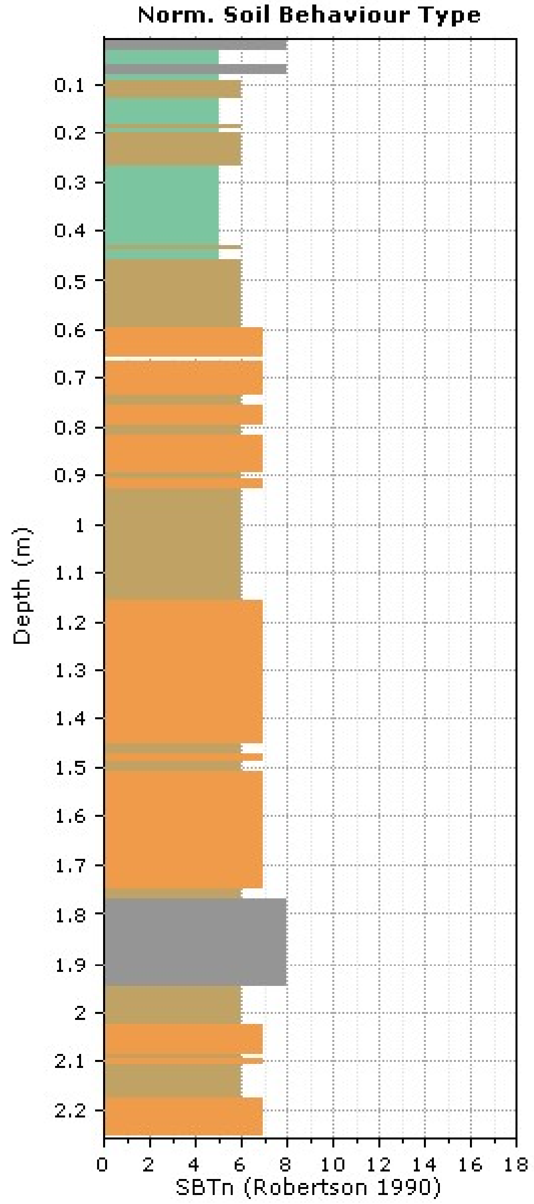
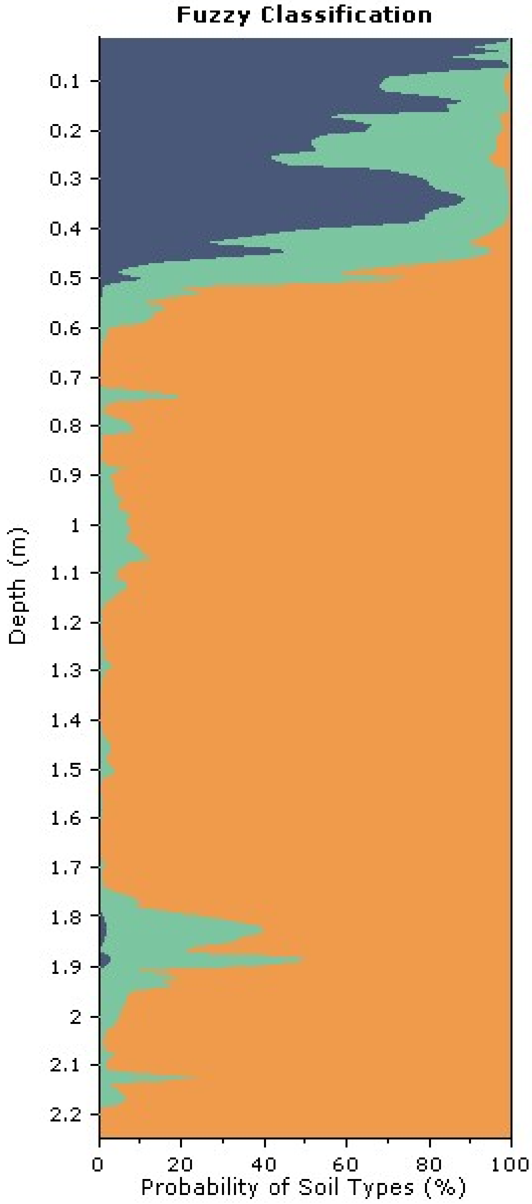
Barry Hogan Place
Riccarton 8041

Project: **Trices Road Rezoning Group**
Location: **Trices Road Rezoning Area**

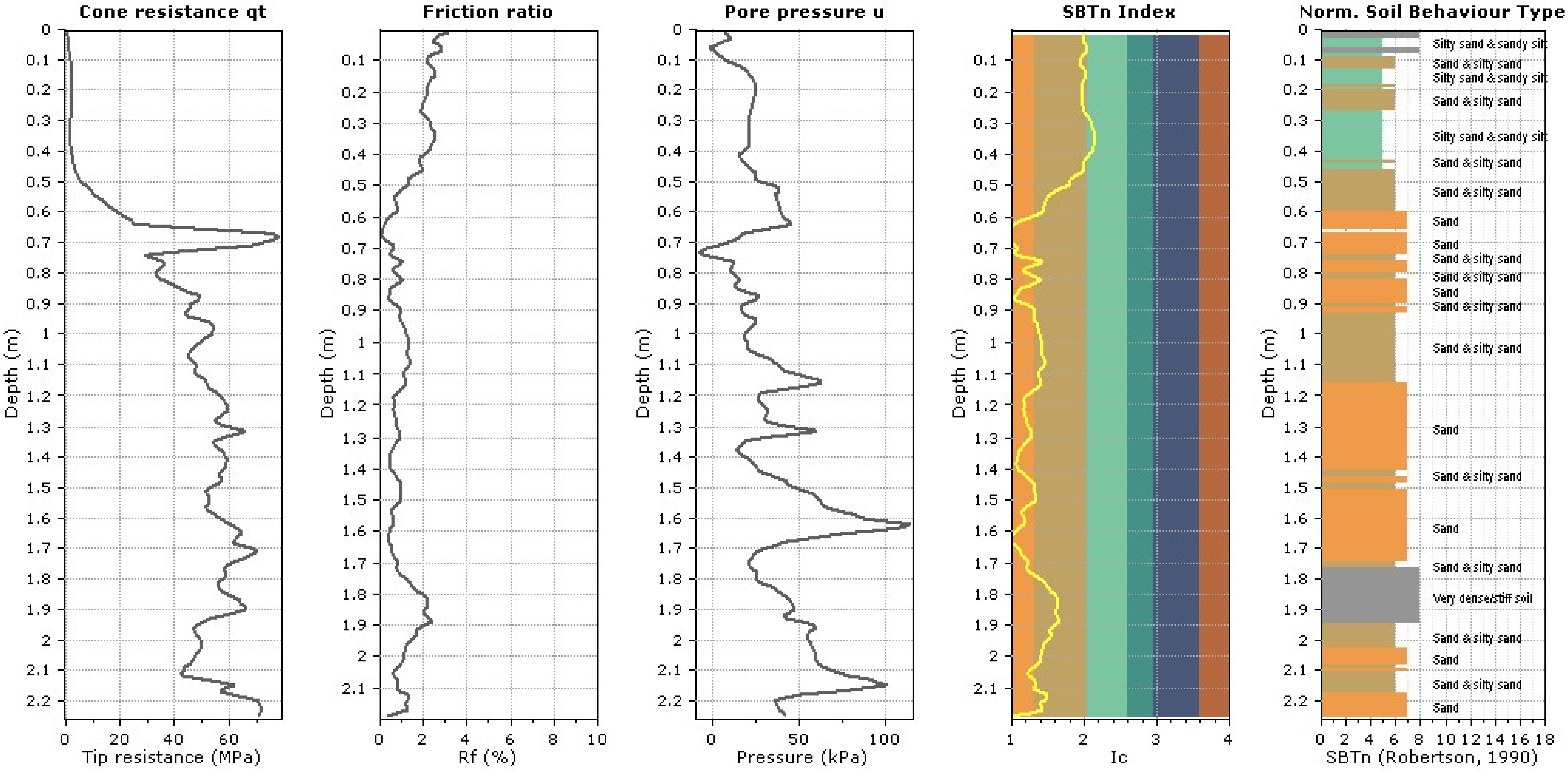
CPT: CPT4
Total depth: 2.25 m, Date: 2/07/2020
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown





Project: **Trices Road Rezoning Group**
Location: **Trices Road Rezoning Area**

CPT: CPT4
Total depth: 2.25 m, Date: 2/07/2020
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown



SBTn legend

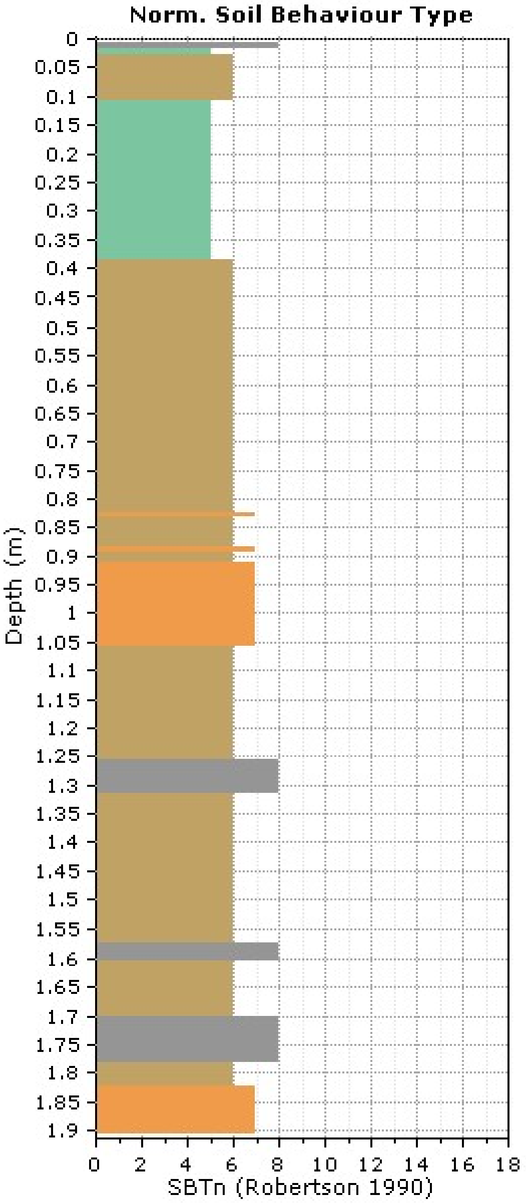
🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟨 6. Clean sand to silty sand
🟧 7. Gravelly sand to sand
⬛ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

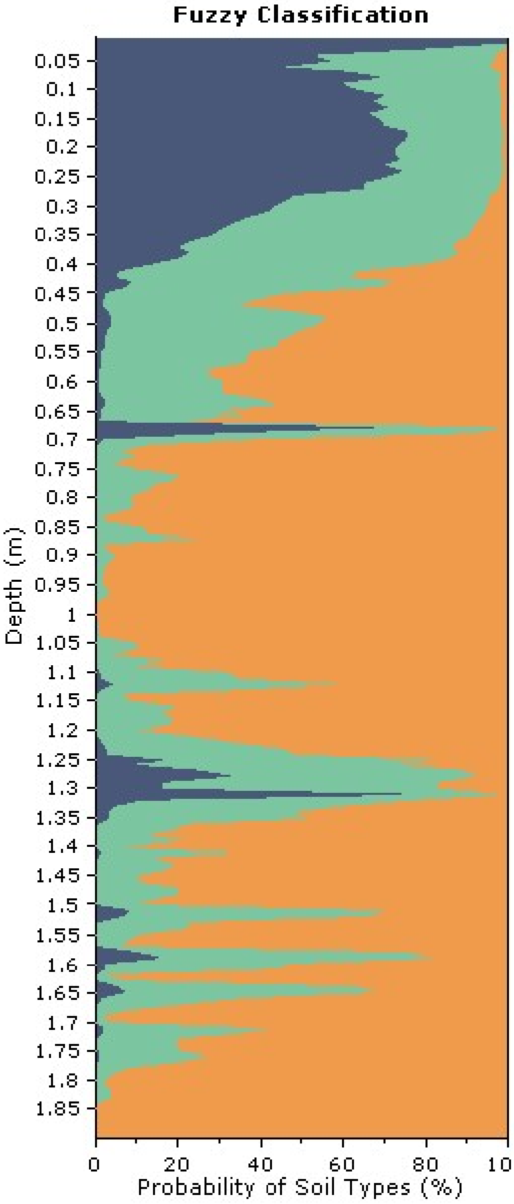
CPT 5

Unit 3a
Barry Hogan Place
Riccarton 8041

CPT: CPT5
Total depth: 1.90 m, Date: 2/07/2020
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

Project: **Trices Road Rezoning Group**
Location: **Trices Road Rezoning Area**





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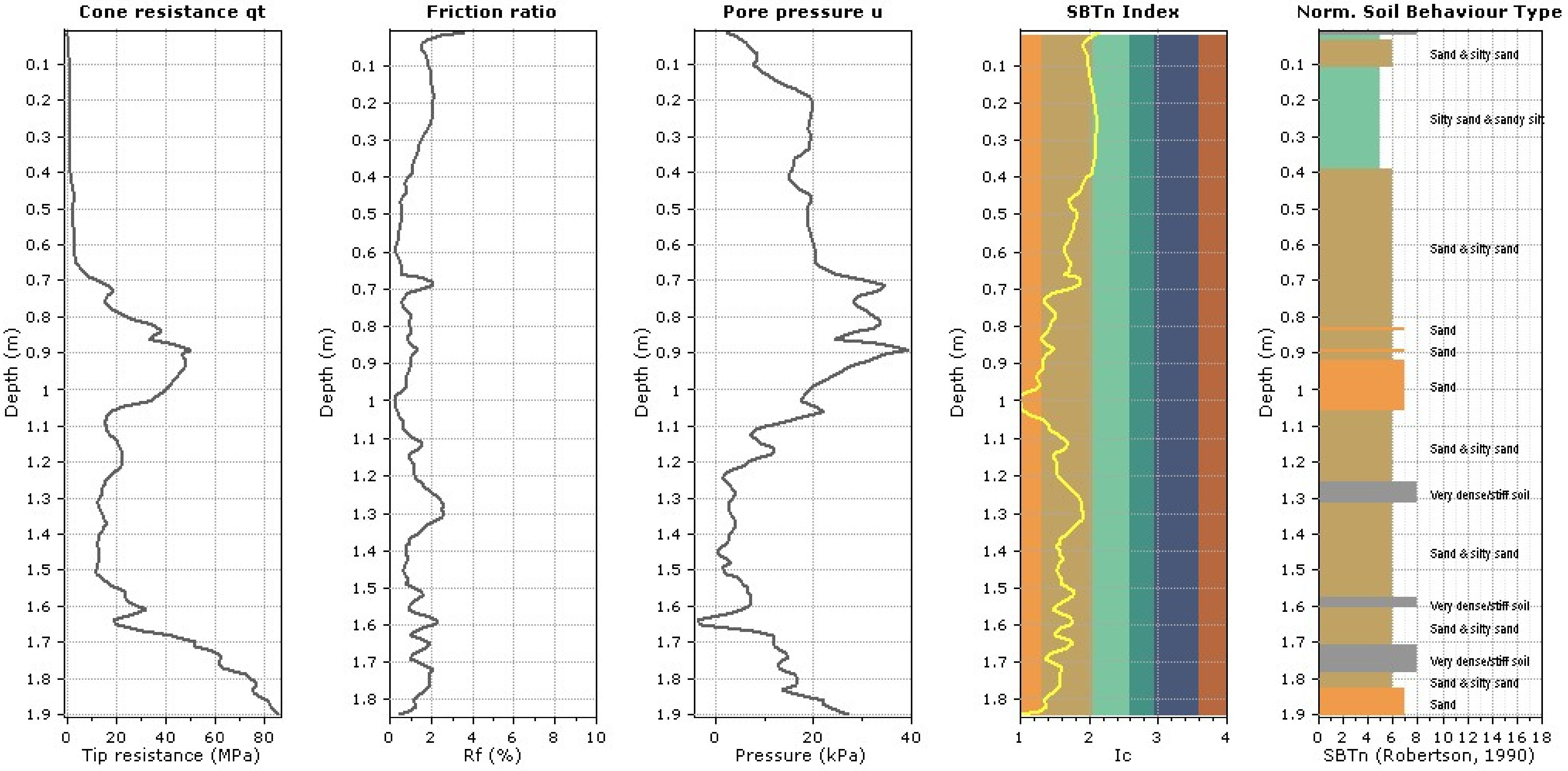
Unit 3a
Barry Hogan Place
Riccarton 8041

CPT: CPT5
Total depth: 1.90 m, Date: 2/07/2020
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

Project: **Trices Road Rezoning Group**
Location: **Trices Road Rezoning Area**



SBTn legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty clay	🟧 7. Gravelly sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to clayey sand
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

Hand Augered Boreholes

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BOREHOLE AND TEST PIT LOGS SYMBOLS AND TERMS

SYMBOLS AND ABBREVIATIONS

RL	Reduced Level
EOH	End of Hole
•	Shear vane test result
UTP	Unable to Penetrate
TDTA	Too Difficult to Auger
SPT	Standard Penetration Test
N	SPT blows per 300mm penetration
35/90	35 blows per 90mm penetration after seating for SPT
(s)	Inclusive of seating blow count for SPT
GWL	Ground Water Level

Wf	Field water content
Wp	Plastic limit (%)
WL	Liquid Limit (%)
RQD	Rock Quality Designation
SG	Specific Gravity
%F	Percentage fines (<75 microns)
PSD	Particle size distribution
CONS	Consolidation test
COMP	Compaction test
UCS	Unconfined Compressive Strength
k	Permeability coefficient (m/s)
LS	Linear Shrinkage (%)
OC	Organic Content (%)

SOIL

- ▨ TOPSOIL
- ▤ CLAY
- ▨ SILT
- ░ SAND
- ▨ GRAVEL
- ▨ COBBLES
- ▨ BOULDERS
- ▨ PEAT
- ▩ FILL

CONSISTENCY TERMS

Cohesive Description	Undrained Shear Strength (kPa)
Very Soft	<12
Soft	12 - 25
Firm	25 - 50
Stiff	50 - 100
Very Stiff	100 - 200
Hard	>200

RELATIVE DENSITY

Non-cohesive Description	SPT "N" Value
Very Loose	<4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	> 50

ROCK

- ▦ LIMESTONE
- ▤ MUDSTONE
- ░ SANDSTONE
- ▨ CONGLOMERATE
- ▲ BRECCIA
- ▨ RYHOLITE
- ▨ ANDESITE
- ▨ BASALT

STRENGTH

Description	Unconfined Compressive Strength MPa
Extremely Weak	< 1
Very Weak	1 - 5
Weak	5 - 20
Moderately Strong	20 - 50
Strong	50 - 100
Very Strong	100 - 250
Extremely Strong	> 250

WEATHERING

UW	- Unweathered (fresh rock)
SW	- Slightly Weathered
MW	- Moderately Weathered
HW	- Highly Weathered
CW	- Completely Weathered
RS	- Residual Soil

SPACING OF DISCONTINUITIES

Term	Aperture (mm)
Very widely spaced	>2000
Widely spaced	600 - 2000
Moderately widely spaced	200 - 600
Closely spaced	60 - 200
Very closely spaced	20 - 60
Extremely closely spaced	<20

Notes

1. Based on New Zealand Geotechnical Society " Field Description of Soil and Rock,Guideline for the Field Classification and Description of Soil and Rock for Engineering Purposes" December 2005
2. Composite soil types are signified by combined symbols

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HAND AUGER LOG

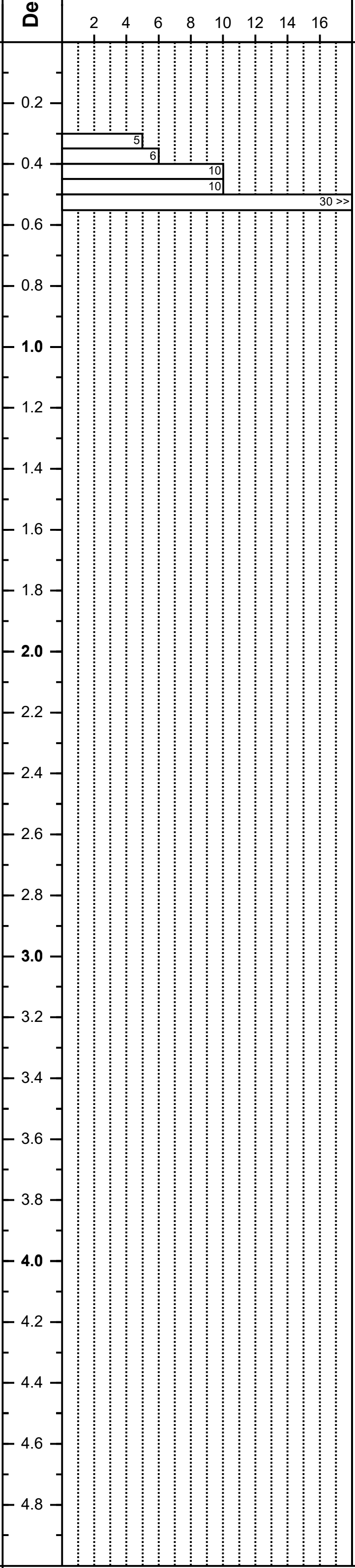
Hole No: H1

Project No: CH00417	**Project:** Trices Road Rezoning Group Trices Road Rezoning Area	**Shear Vane:**	**Date Drilled:** 02/07/2020	**Logged By:** JTG	**Checked By:** MVR

Depth (m)	Description of Strata	Geological Unit	Graphic Log	Undrained Shear Strength (kPa) Values	Groundwater
0.0–0.3	SILT, brown, moist, low plasticity [TOPSOIL]	T/S	TS		GWNE
0.3	EOH: 0.30 m TDTA - GRAVELS ENCOUNTERED				

Undrained Shear Strength (kPa): Vane readings corrected as per BS 1377; ● Shear Vane; ○ Residual Shear Vane

Dynamic Cone Penetrometer: Test Method: NZS 4402:1988, Test 6.5.2 (Blows / 50mm)



0.30–0.35	5
0.35–0.40	6
0.40–0.45	10
0.45–0.50	10
0.50–0.55	30 >>

Remarks:
1. Groundwater not encountered on 2 July 2020.

Datum:

Coordinates:

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HAND AUGER LOG

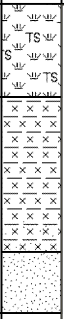
Hole No: H2

Project No:	**Project:**	**Shear Vane:**	**Date Drilled:**	**Logged By:**	**Checked By:**
CH00417	Trices Road Rezoning Group Trices Road Rezoning Area	1310	02/07/2020	JTG	MVR

Undrained Shear Strength (kPa) – Vane readings corrected as per BS 1377 – ● Shear Vane ○ Residual Shear Vane – 50, 100, 150, 200, Values

Dynamic Cone Penetrometer – Test Method: NZS 4402:1988, Test 6.5.2 (Blows / 0mm) – 2, 4, 6, 8, 10, 12, 14, 16

Depth (m)	Description of Strata	Geological Unit	Shear Strength Values	Groundwater
0.0 – 0.3	SILT, brown, moist, low plasticity. trace wood fragments [TOPSOIL]	T/S		GWNE
0.3 – 0.8	SILT, clayey, light brown streaked orange, hard, moist, low plasticity [SPRINGSTON FORMATION] 0.7 m: becomes sandy	Springston Formation	>200 (0.4 m) >200 (0.7 m)	
0.8 – 1.0	SAND, grey, medium dense, dry, well graded	Springston Formation		
1.0	EOH: 1.00 m TDTA - GRAVELS ENCOUNTERED			



Remarks:
1. Groundwater not encountered on 2 July 2020.

Datum:

Coordinates:

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HAND AUGER LOG

Hole No: **H3**

Project No:	**Project:**	**Shear Vane:**	**Date Drilled:**	**Logged By:**	**Checked By:**
CH00417	**Trices Road Rezoning Group** **Trices Road Rezoning Area**		02/07/2020	JTG	MVR

Depth (m)	**Description of Strata**	**Geological Unit**	**Graphic Log**	**Undrained Shear Strength (kPa)** Vane readings corrected as per BS 1377 ● Shear Vane ○ Residual Shear Vane 50 100 150 200	Values	**Depth (m)**	**Dynamic Cone Penetrometer** Test Method: NZS 4402:1988, Test 6.5.2 (Blows / 0mm) 2 4 6 8 10 12 14 16	**Groundwater**
0.2	SILT, brown, moist, low plasticity [TOPSOIL] EOH: 0.30 m TDTA - GRAVELS ENCOUNTERED	T/S	TS			0.2		GWNE

Remarks:
1. Groundwater not encountered on 2 July 2020.

Datum:

Coordinates:

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HAND AUGER LOG

Hole No: H4

Project No:	**Project:**	**Shear Vane:**	**Date Drilled:**	**Logged By:**	**Checked By:**
CH00417	Trices Road Rezoning Group Trices Road Rezoning Area	1310	02/07/2020	JTG	MVR

Depth (m)	Description of Strata	Geological Unit	Undrained Shear Strength (kPa) Values	Groundwater
0.0–0.3	SILT, brown, moist, low plasticity [TOPSOIL]	T/S		
0.3–0.7	SILT, clayey, light brown streaked orange, very stiff, moist, low plasticity [SPRINGSTON FORMATION]	Springston Formation	197	GWNE
0.7–1.3	SAND, silty, grey, medium dense, dry, poorly graded	Springston Formation		
1.30	EOH: 1.30 m TDTA - GRAVELS ENCOUNTERED			

Undrained Shear Strength (kPa): Vane readings corrected as per BS 1377
● Shear Vane ○ Residual Shear Vane

Dynamic Cone Penetrometer
Test Method: NZS 4402:1988, Test 6.5.2
(Blows / 50mm)

Depth (m)	Blows / 50mm
0.50–0.55	2
0.55–0.60	3
0.60–0.65	3
0.65–0.70	2
0.70–0.75	5
0.75–0.80	3
0.80–0.85	2
0.85–0.90	3
0.90–0.95	3
0.95–1.00	3
1.00–1.05	4
1.05–1.10	3
1.10–1.15	5
1.15–1.20	6
1.20–1.25	5
1.25–1.30	7

Remarks:
1. Groundwater not encountered on 2 July 2020.

Datum:

Coordinates:

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HAND AUGER LOG

Hole No: **H5**

Project No:	**Project:**	**Shear Vane:**	**Date Drilled:**	**Logged By:**	**Checked By:**
CH00417	**Trices Road Rezoning Group** **Trices Road Rezoning Area**		02/07/2020	JTG	MVR

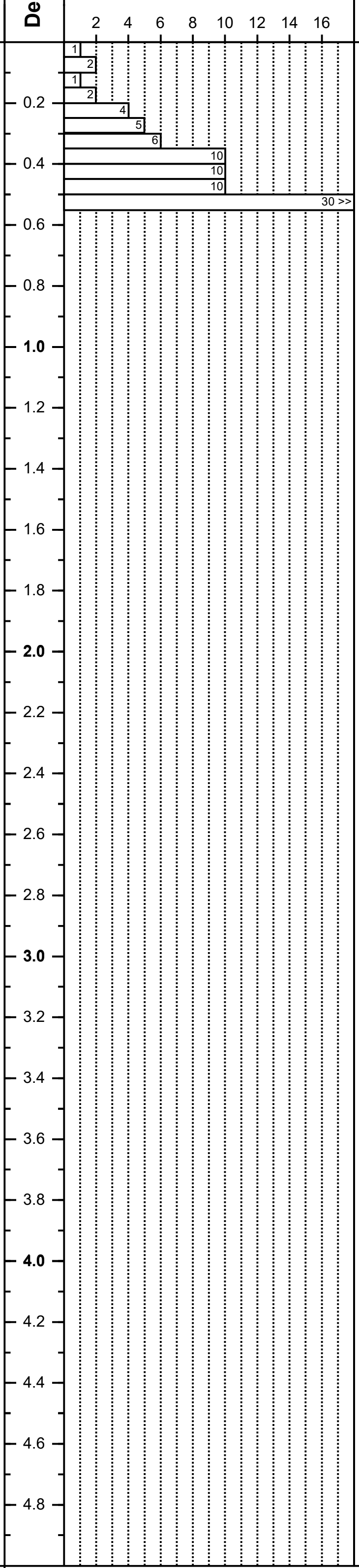
Depth (m)	**Description of Strata**	**Geological Unit**	**Graphic Log**	**Undrained Shear Strength (kPa)** Vane readings corrected as per BS 1377 ● Shear Vane ○ Residual Shear Vane — 50 100 150 200	Values	**Groundwater**
0.0 – 0.30	SILT, brown, moist, low plasticity [TOPSOIL]	T/S	TS			GWNE
0.30	EOH: 0.30 m TDTA - GRAVELS ENCOUNTERED					

Depth (m): 0.2, 0.4, 0.6, 0.8, **1.0**, 1.2, 1.4, 1.6, 1.8, **2.0**, 2.2, 2.4, 2.6, 2.8, **3.0**, 3.2, 3.4, 3.6, 3.8, **4.0**, 4.2, 4.4, 4.6, 4.8

Dynamic Cone Penetrometer

Test Method: NZS 4402:1988, Test 6.5.2

(Blows / 50mm)



Remarks:

1. Groundwater not encountered on 2 July 2020.

Datum:

Coordinates:

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HAND AUGER LOG

Hole No: H6

Project No:	**Project:**	**Shear Vane:**	**Date Drilled:**	**Logged By:**	**Checked By:**
CH00417	Trices Road Rezoning Group Trices Road Rezoning Area		02/07/2020	JTG	MVR

Undrained Shear Strength (kPa): Vane readings corrected as per BS 1377 — ● Shear Vane ○ Residual Shear Vane (scale 50, 100, 150, 200; Values)

Dynamic Cone Penetrometer: Test Method: NZS 4402:1988, Test 6.5.2 (Blows / 0mm) (scale 2, 4, 6, 8, 10, 12, 14, 16)

Depth (m)	Description of Strata	Geological Unit	Groundwater
0.0 – 0.30	SILT, brown, moist, low plasticity [TOPSOIL]	T/S	GWNE
0.30 – 0.60	SILT, clayey, light brown streaked orange, very stiff, moist, low plasticity [SPRINGSTON FORMATION]		
0.60	EOH: 0.60 m TDTA - GRAVELS ENCOUNTERED		

Depth (m): 0.2, 0.4, 0.6, 0.8, 1.0, 1.2, 1.4, 1.6, 1.8, 2.0, 2.2, 2.4, 2.6, 2.8, 3.0, 3.2, 3.4, 3.6, 3.8, 4.0, 4.2, 4.4, 4.6, 4.8

Remarks:
1. Groundwater not encountered on 2 July 2020.

Datum:

Coordinates:

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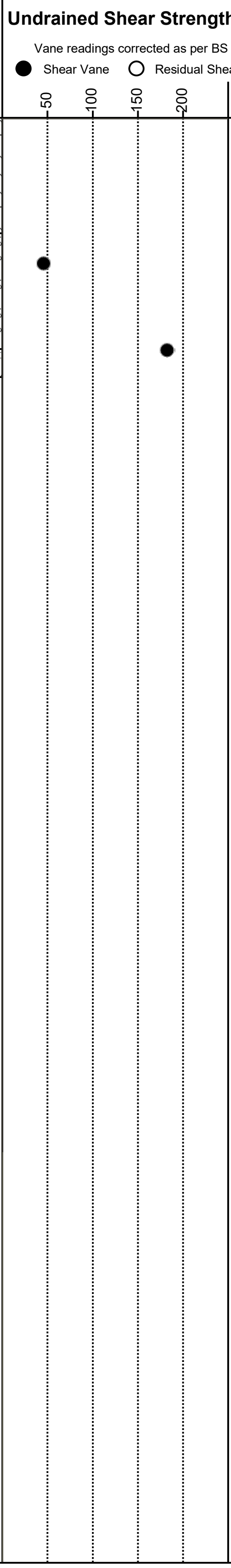
HAND AUGER LOG

Hole No: H7

Project No:	**Project:**	**Shear Vane:**	**Date Drilled:**	**Logged By:**	**Checked By:**
CH00417	Trices Road Rezoning Group Trices Road Rezoning Area	1310	02/07/2020	JTG	MVR

Depth (m)	Description of Strata	Geological Unit	Undrained Shear Strength (kPa) Values	Groundwater
0.0 – 0.4	SILT, brown, moist, low plasticity [TOPSOIL]	T/S		GWNE
0.4 – 0.8	SILT, clayey, light brown streaked orange, firm, moist, low plasticity [SPRINGSTON FORMATION]	Springston Formation	45	
0.8 – 0.9	SAND, silty, grey, dense, dry, poorly graded		182	
	EOH: 0.90 m TDTA - GRAVELS ENCOUNTERED			



Dynamic Cone Penetrometer

Test Method: NZS 4402:1988, Test 6.5.2 (Blows / 0mm)

2 4 6 8 10 12 14 16

Remarks:

1. Groundwater not encountered on 2 July 2020.

Datum:

Coordinates:

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HAND AUGER LOG

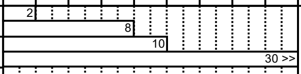
Hole No: H8

Project No:	**Project:**	**Shear Vane:**	**Date Drilled:**	**Logged By:**	**Checked By:**
CH00417	Trices Road Rezoning Group Trices Road Rezoning Area		02/07/2020	JTG	MVR

Depth (m)	Description of Strata	Geological Unit	Graphic Log	Undrained Shear Strength (kPa) Values	Depth (m)	Dynamic Cone Penetrometer (Blows / 50mm)	Groundwater
0.0–0.2	SILT, brown, moist, low plasticity [TOPSOIL]	T/S	TS		0.0–0.2	2, 8, 10, 30 >>	GWNE
0.2	EOH: 0.20 m TDTA - GRAVELS ENCOUNTERED				0.2		

Undrained Shear Strength (kPa): Vane readings corrected as per BS 1377; ● Shear Vane ○ Residual Shear Vane; 50, 100, 150, 200

Dynamic Cone Penetrometer: Test Method: NZS 4402:1988, Test 6.5.2 (Blows / 50mm)



Remarks:
1. Groundwater not encountered on 2 July 2020.

Datum:

Coordinates:

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Appendix B

CLiq Analyses Results

CPT 3

Serviceability Limit State (SLS) Design Earthquake Event

(i.e. the larger value determined for the SLS and ILS design earthquake events)

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Unit 3a
Barry Hogan Place
Riccarton 8041

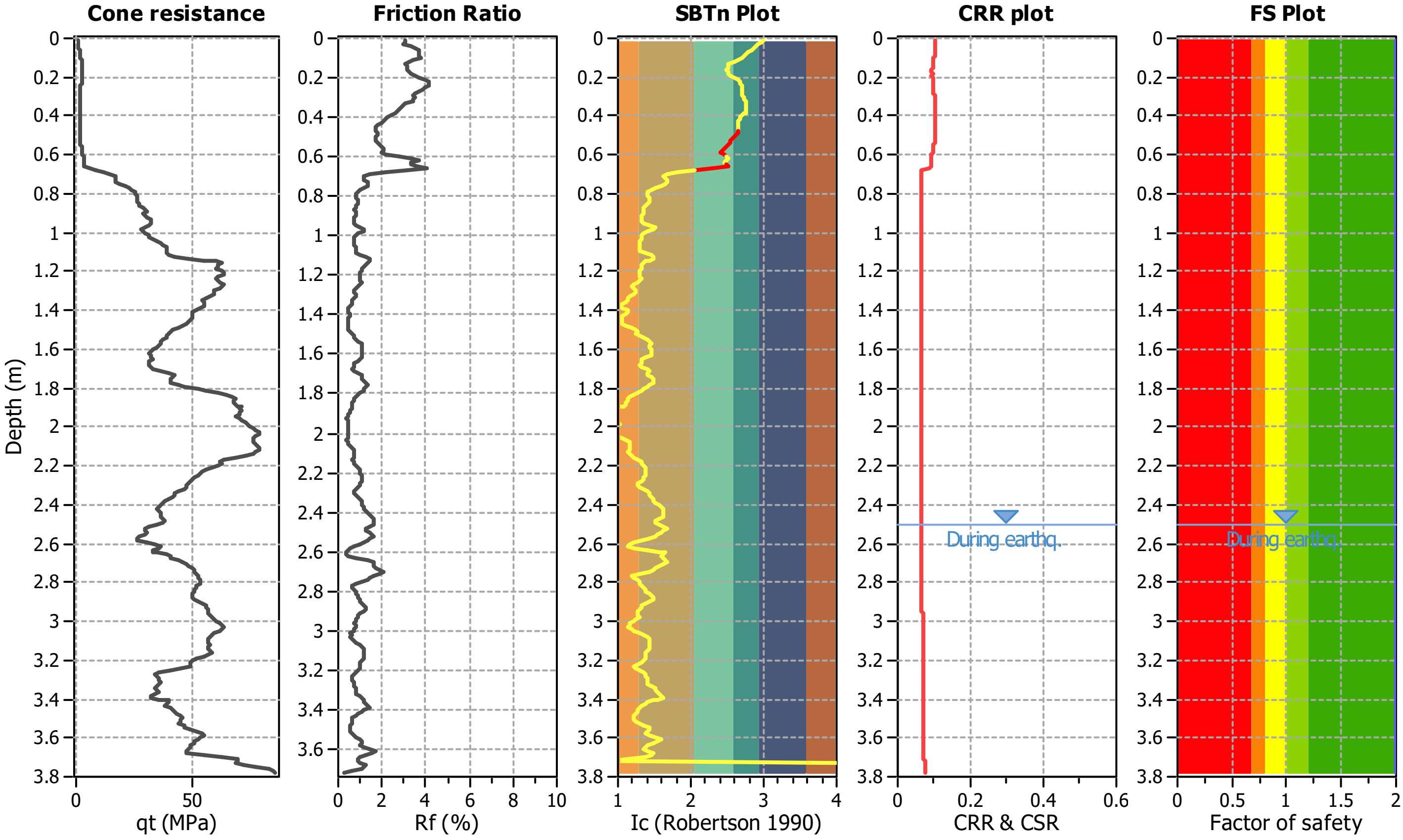
LIQUEFACTION ANALYSIS REPORT

Project title : Trices Road Rezoning Group **Location : Trices Road Rezoning Area**

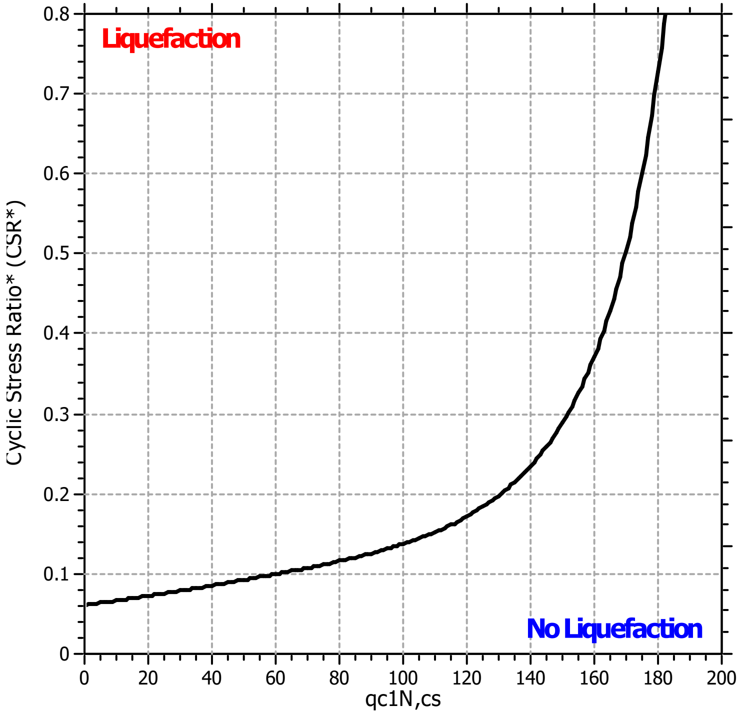
CPT file : CPT3

Input parameters and analysis data

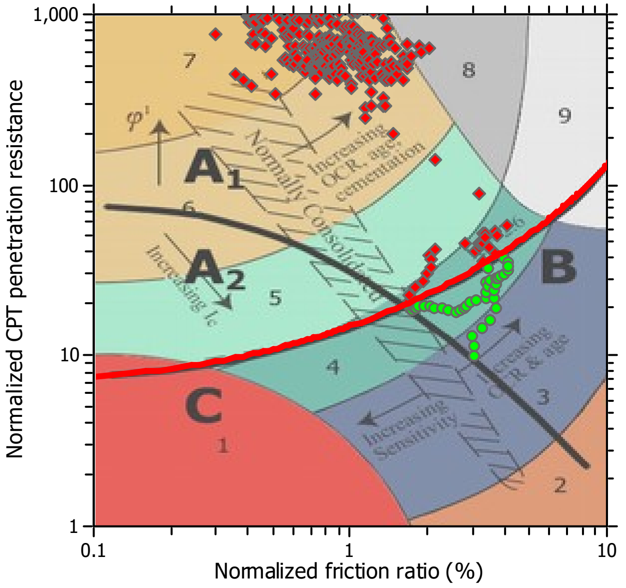
Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	10.00 m
Earthquake magnitude M_w:	6.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve

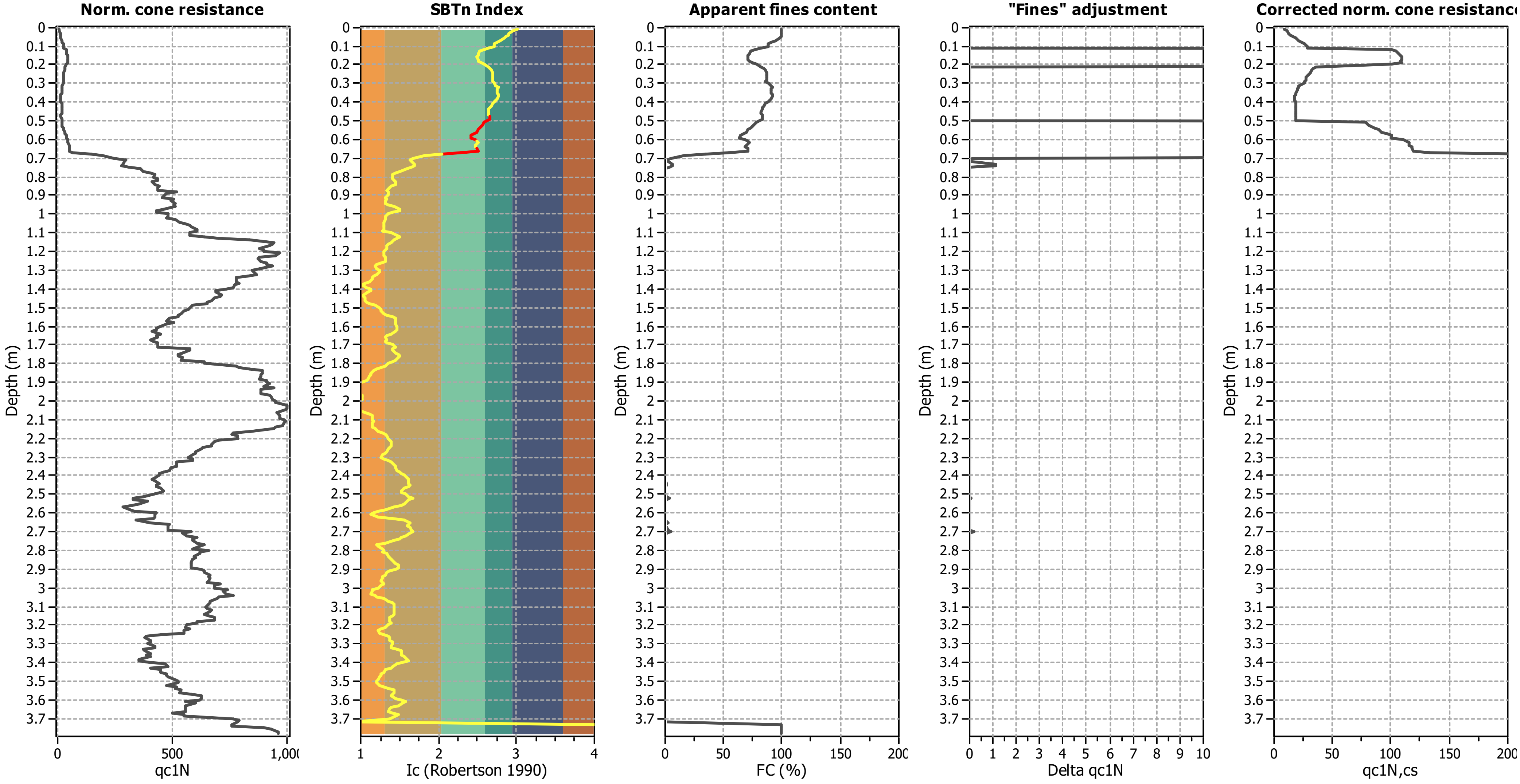


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots (intermediate results)



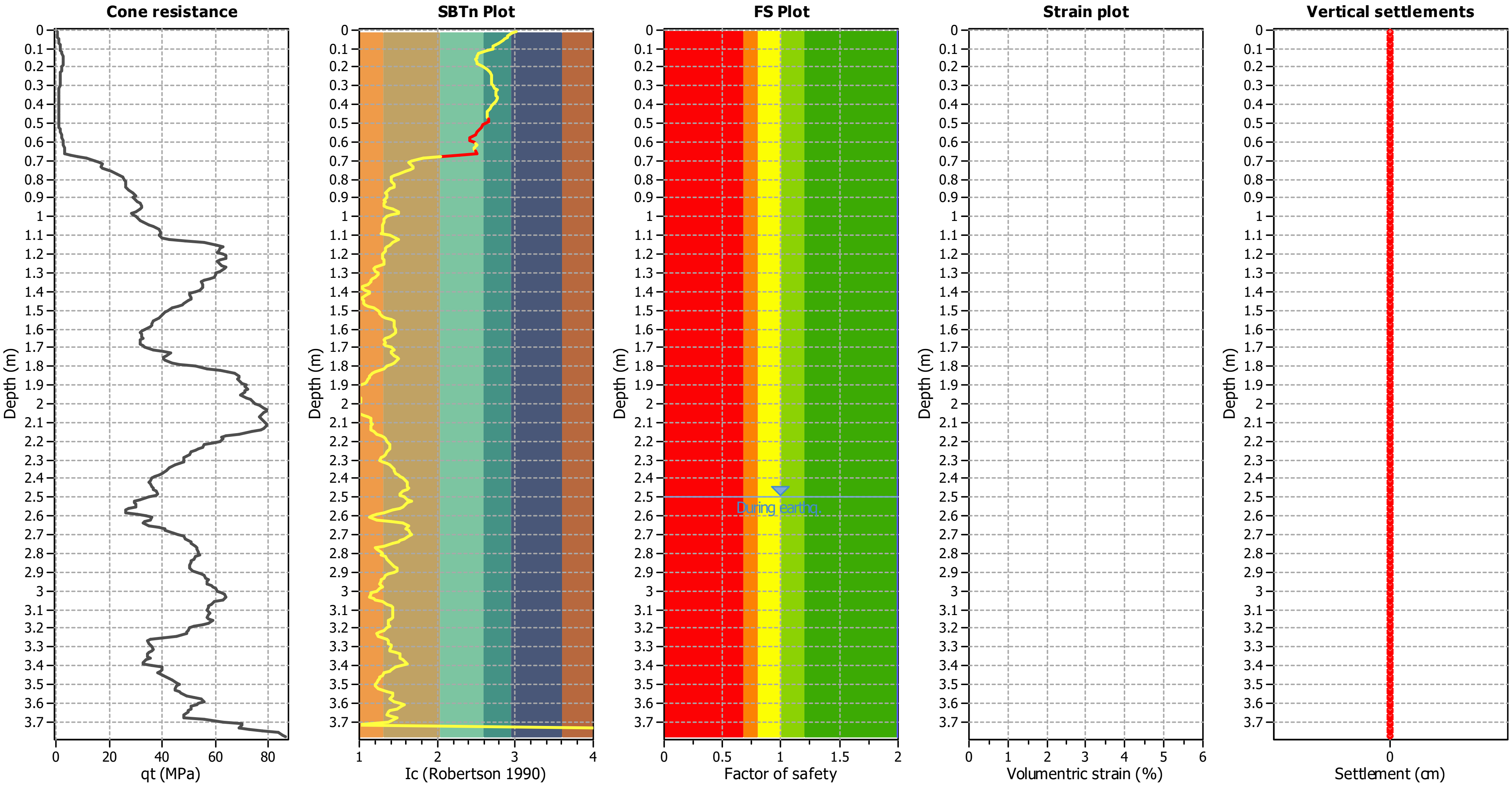
Input parameters and analysis data

Analysis method: B&I (2014)
Fines correction method: B&I (2014)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.00
Peak ground acceleration: 0.19
Depth to water table (insitu): 2.50 m

Depth to GWT (erthq.): 2.50 m
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: Yes
K_σ applied: Yes
Clay like behavior applied: Sand & Clay
Limit depth applied: Yes
Limit depth: 10.00 m

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

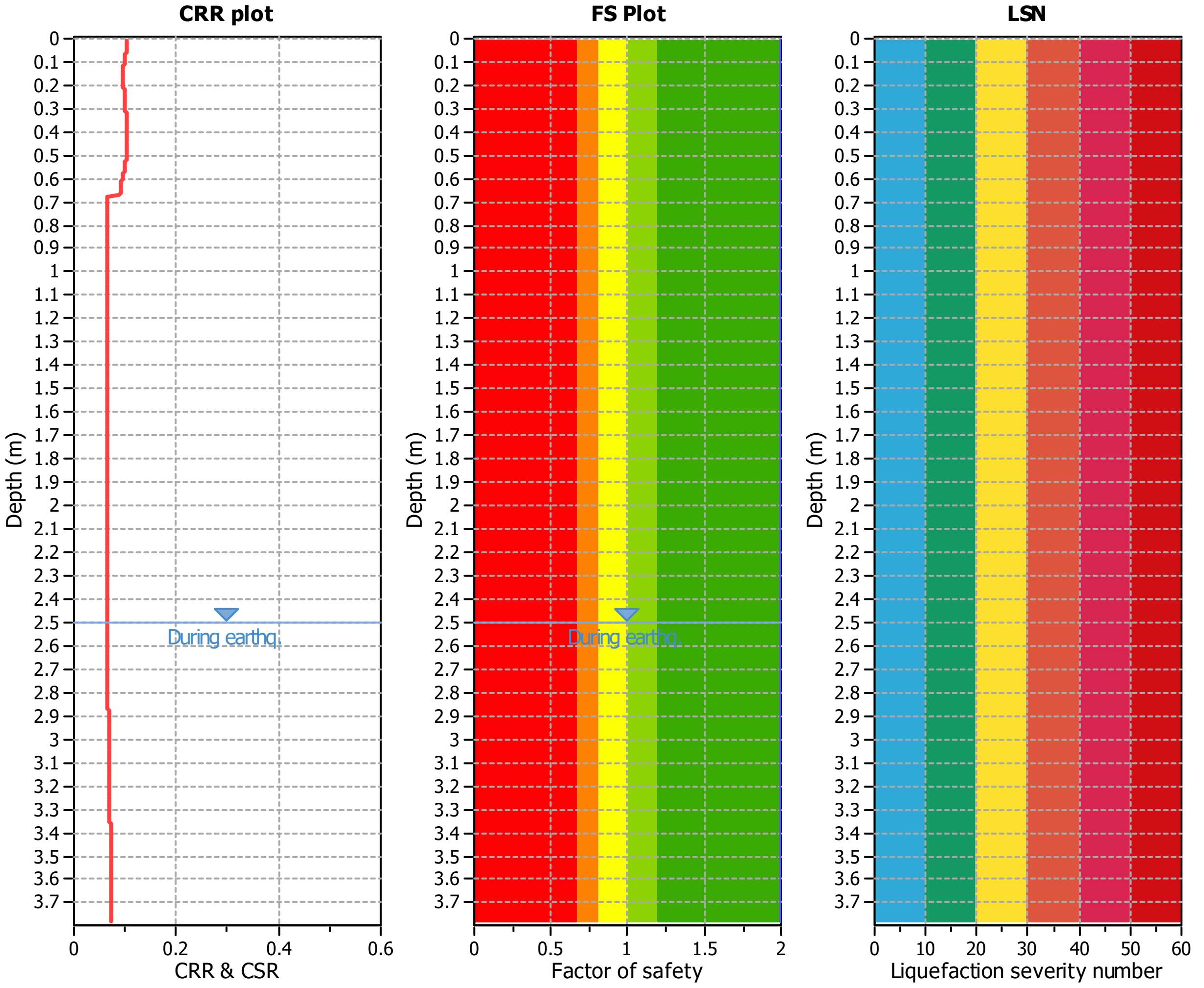
Unit 3a
Barry Hogan Place
Riccarton 8041

Project: **Trices Road Rezoning Group**

Location: **Trices Road Rezoning Area**

CPT: CPT3

Total depth: 3.78 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	10.00 m
Earthquake magnitude M_w:	6.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Ultimate Limit State (ULS) Design Earthquake Event

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Unit 3a
Barry Hogan Place
Riccarton 8041

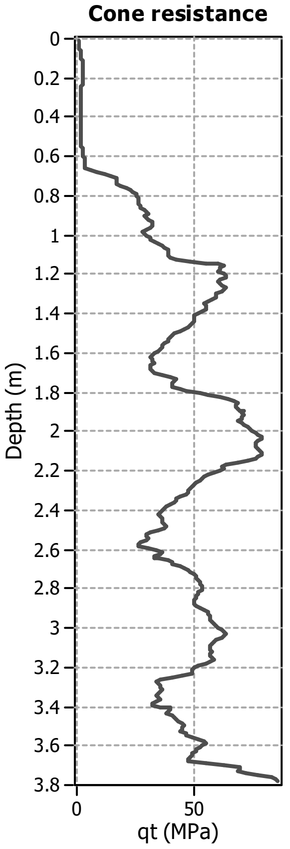
LIQUEFACTION ANALYSIS REPORT

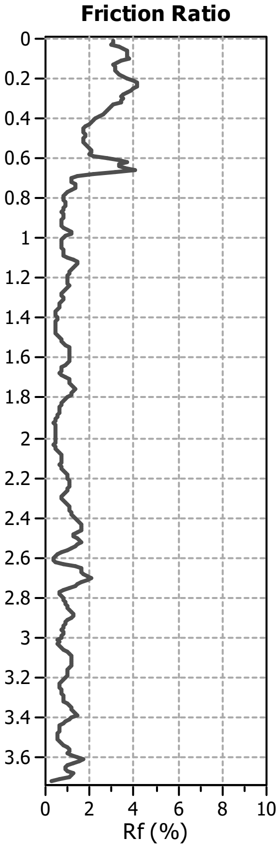
Project title : Trices Road Rezoning Group **Location : Trices Road Rezoning Area**

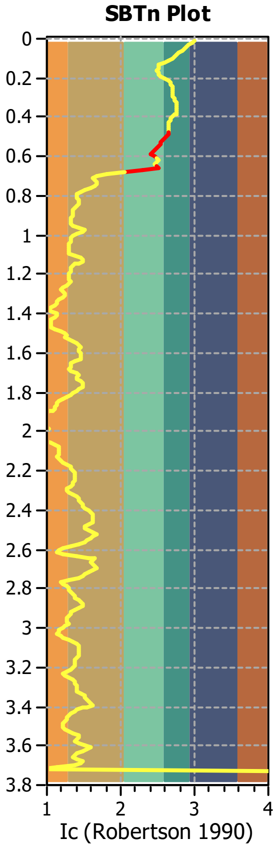
CPT file : CPT3

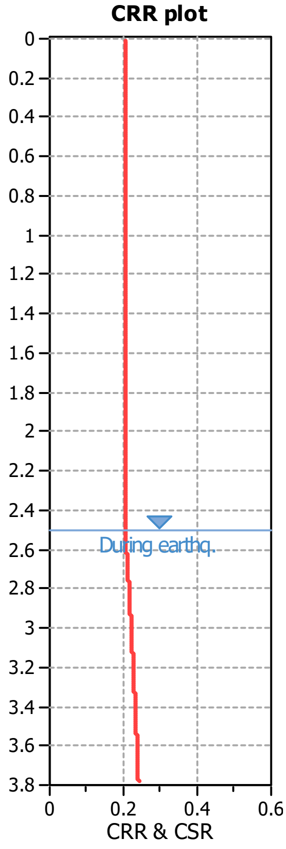
Input parameters and analysis data

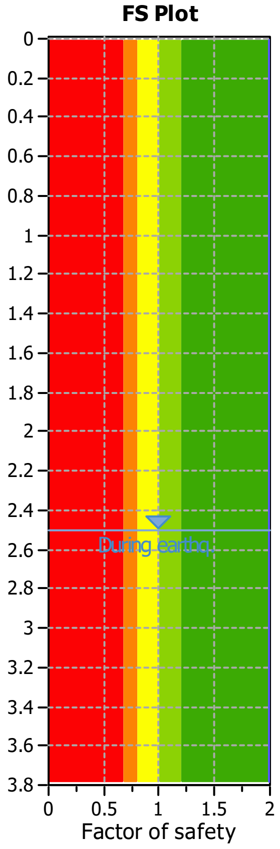
Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	10.00 m
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.35	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

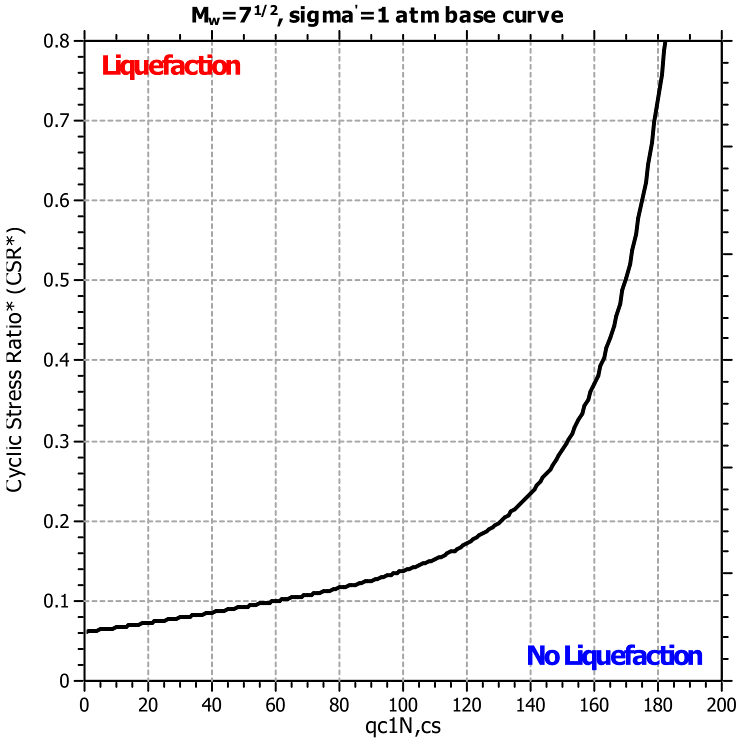

Cone resistance

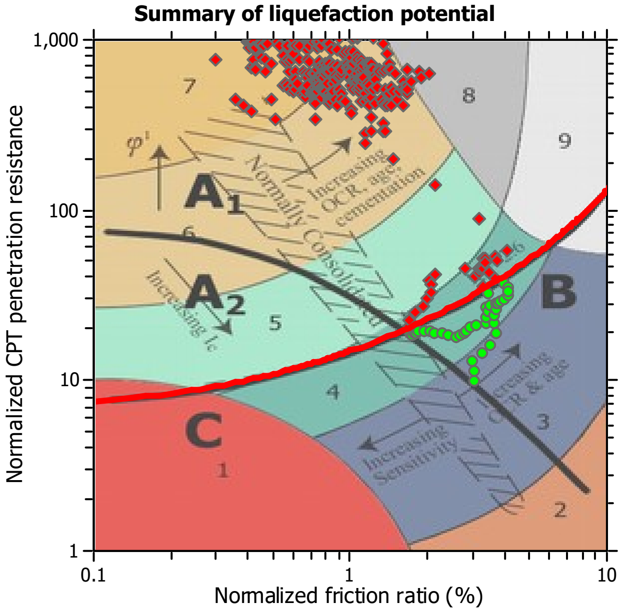

Friction Ratio


SBTn Plot


CRR plot

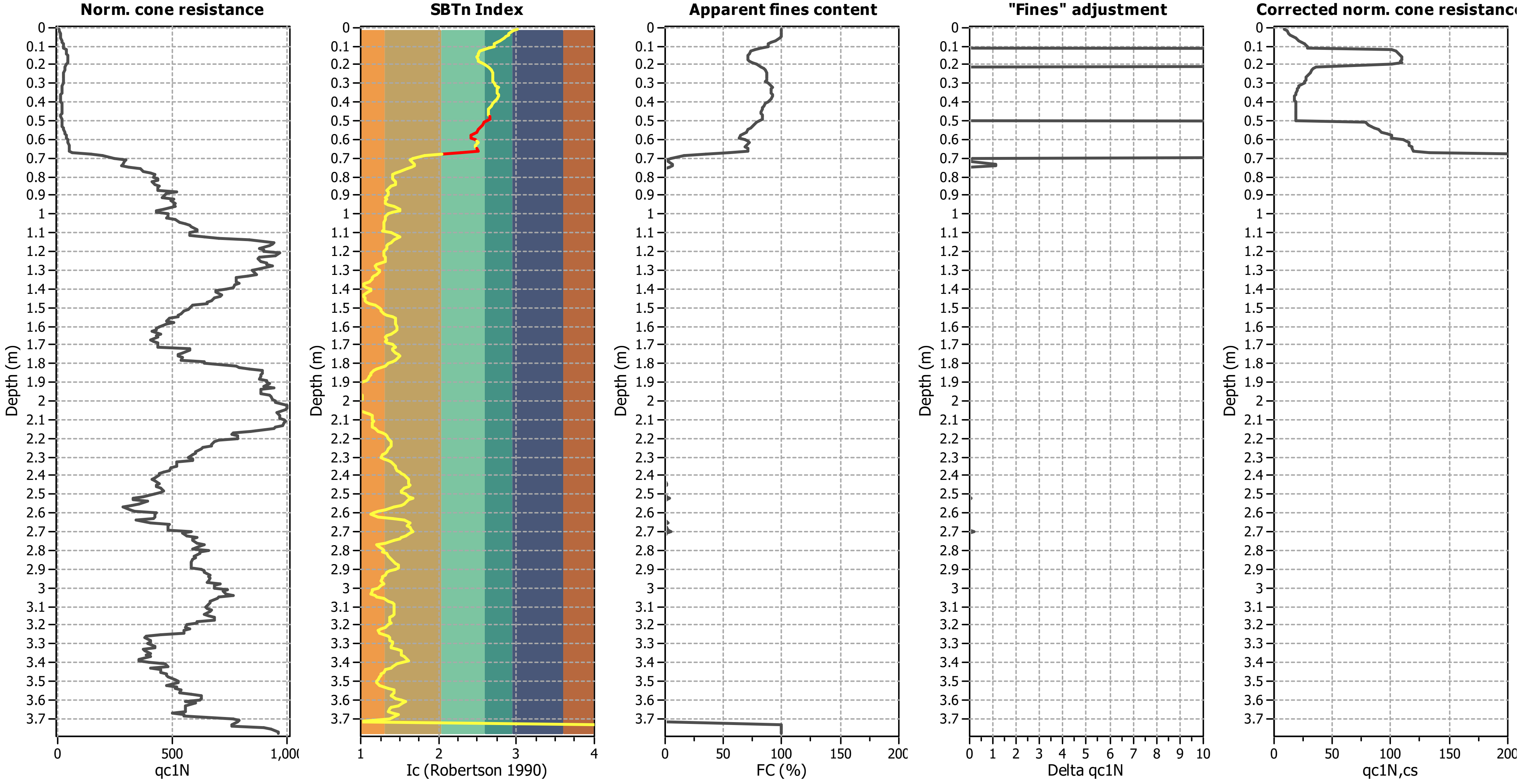

FS Plot


$M_w=7^{1/2}$, sigma'=1 atm base curve


Summary of liquefaction potential

Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

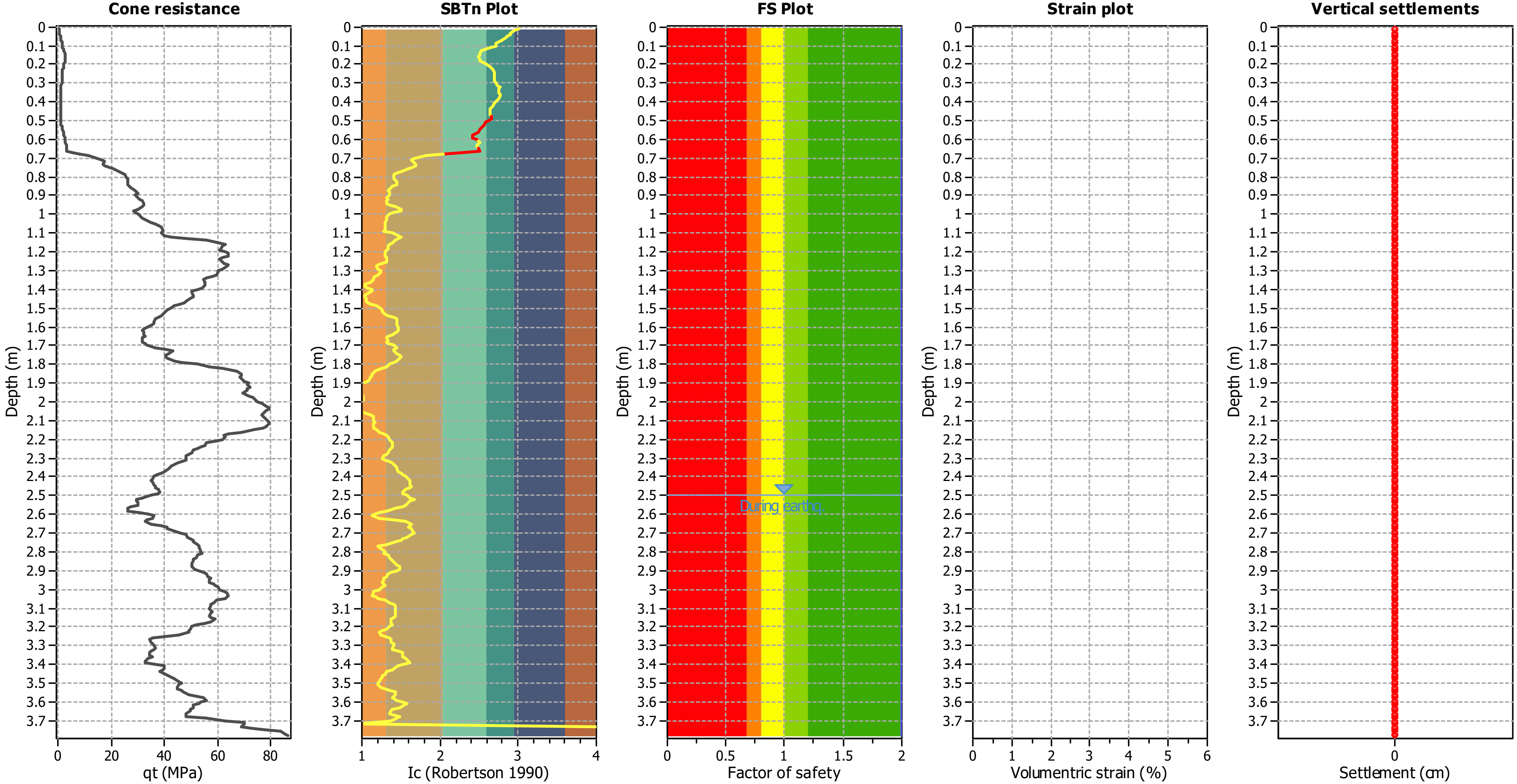
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	10.00 m

Estimation of post-earthquake settlements

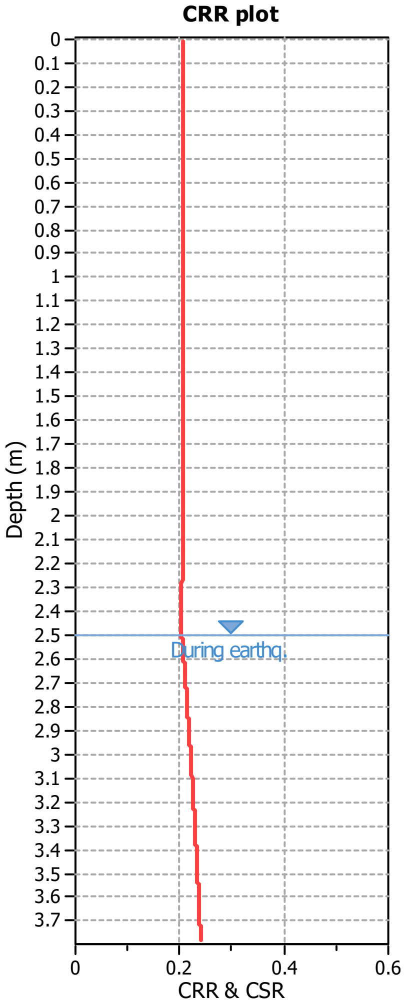


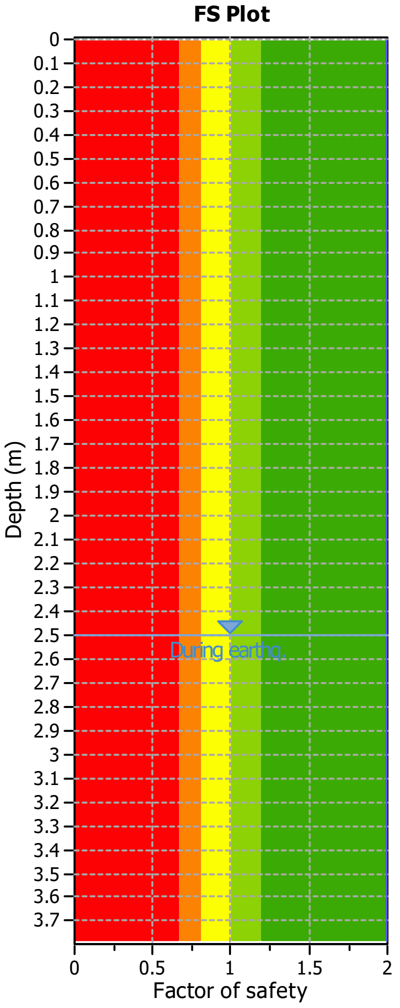
Abbreviations

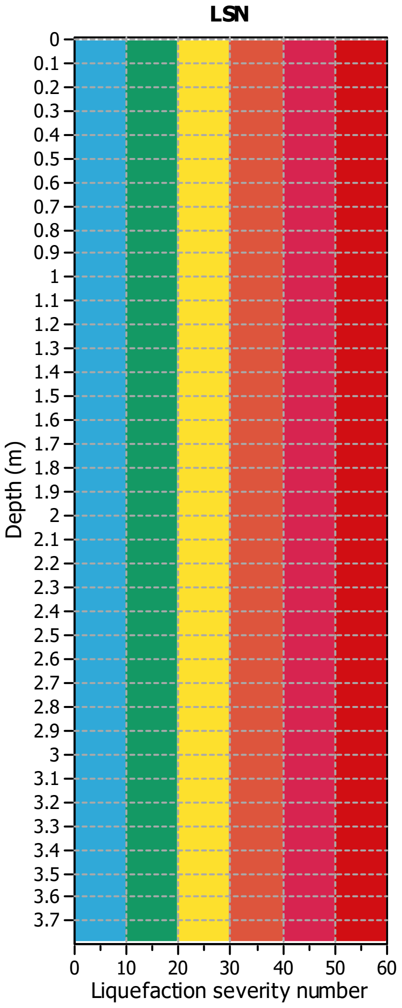
q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Project: **Trices Road Rezoning Group**
Location: **Trices Road Rezoning Area**

CPT: CPT3
Total depth: 3.78 m







Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	10.00 m
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.35	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

CPT 4

Serviceability Limit State (SLS) Design Earthquake Event

(i.e. the larger value determined for the SLS and ILS design earthquake events)

LIQUEFACTION ANALYSIS REPORT

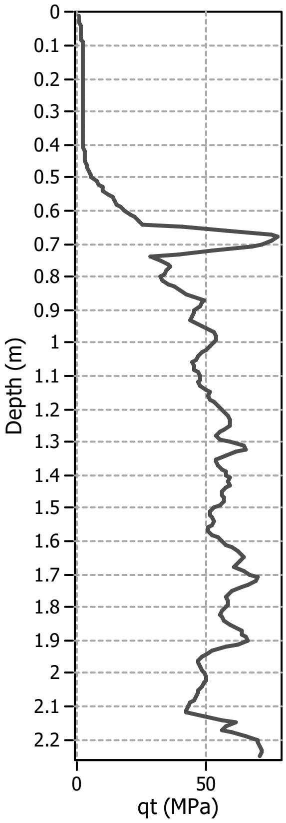
Project title : Trices Road Rezoning Group **Location : Trices Road Rezoning Area**

CPT file : CPT4

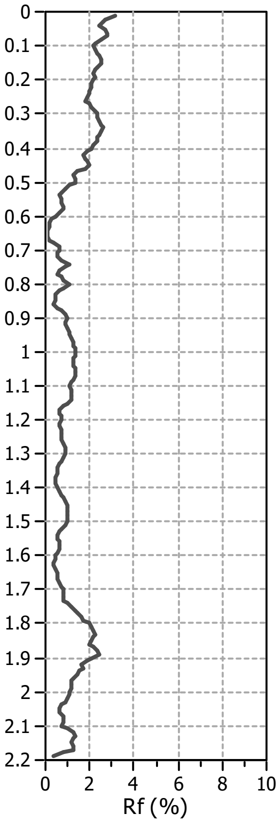
Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	10.00 m
Earthquake magnitude M_w:	6.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

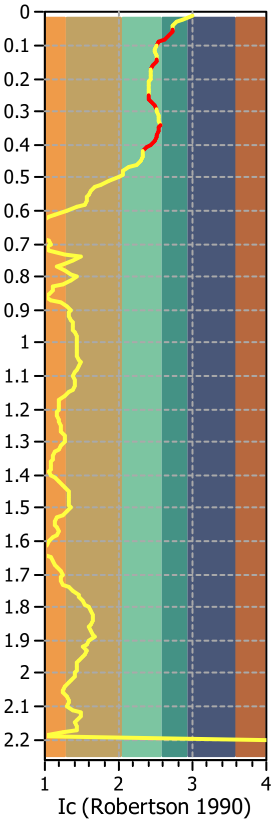
Cone resistance



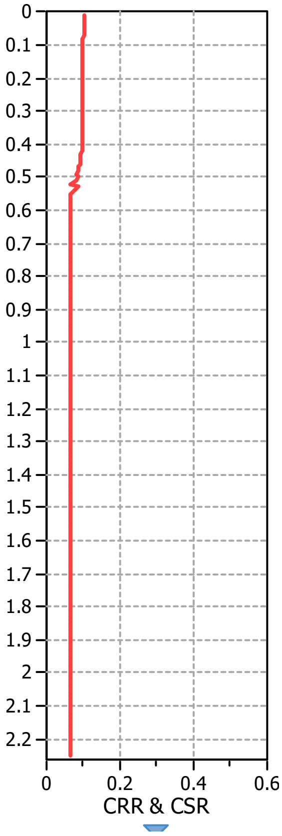
Friction Ratio



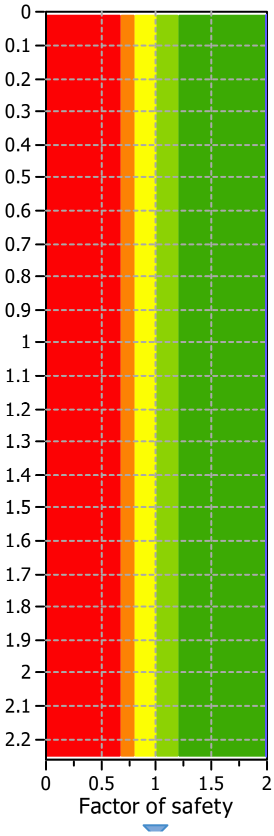
SBTn Plot



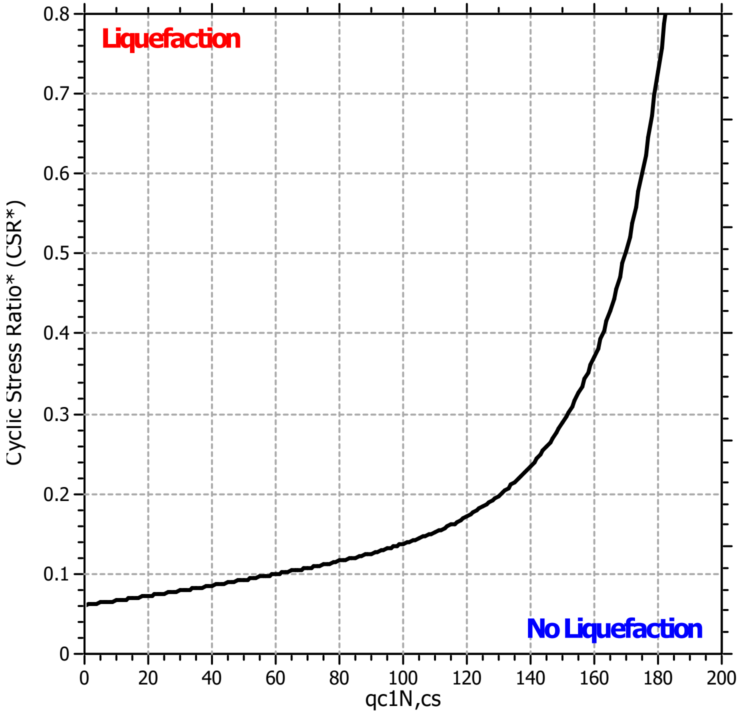
CRR plot



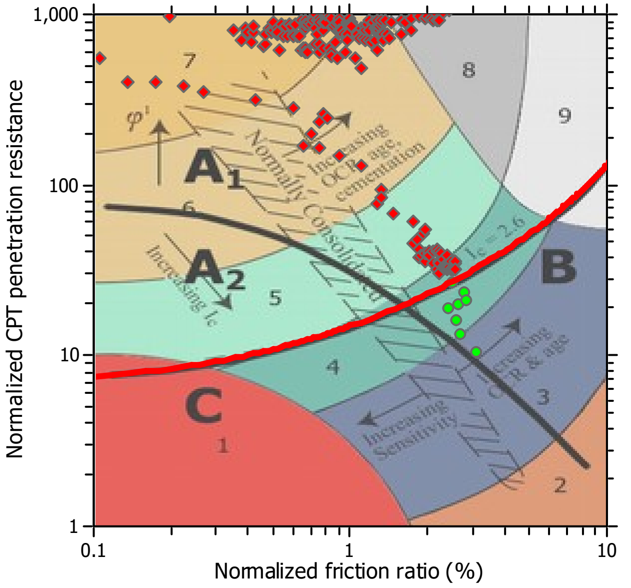
FS Plot



$M_w=7^{1/2}$, sigma'=1 atm base curve

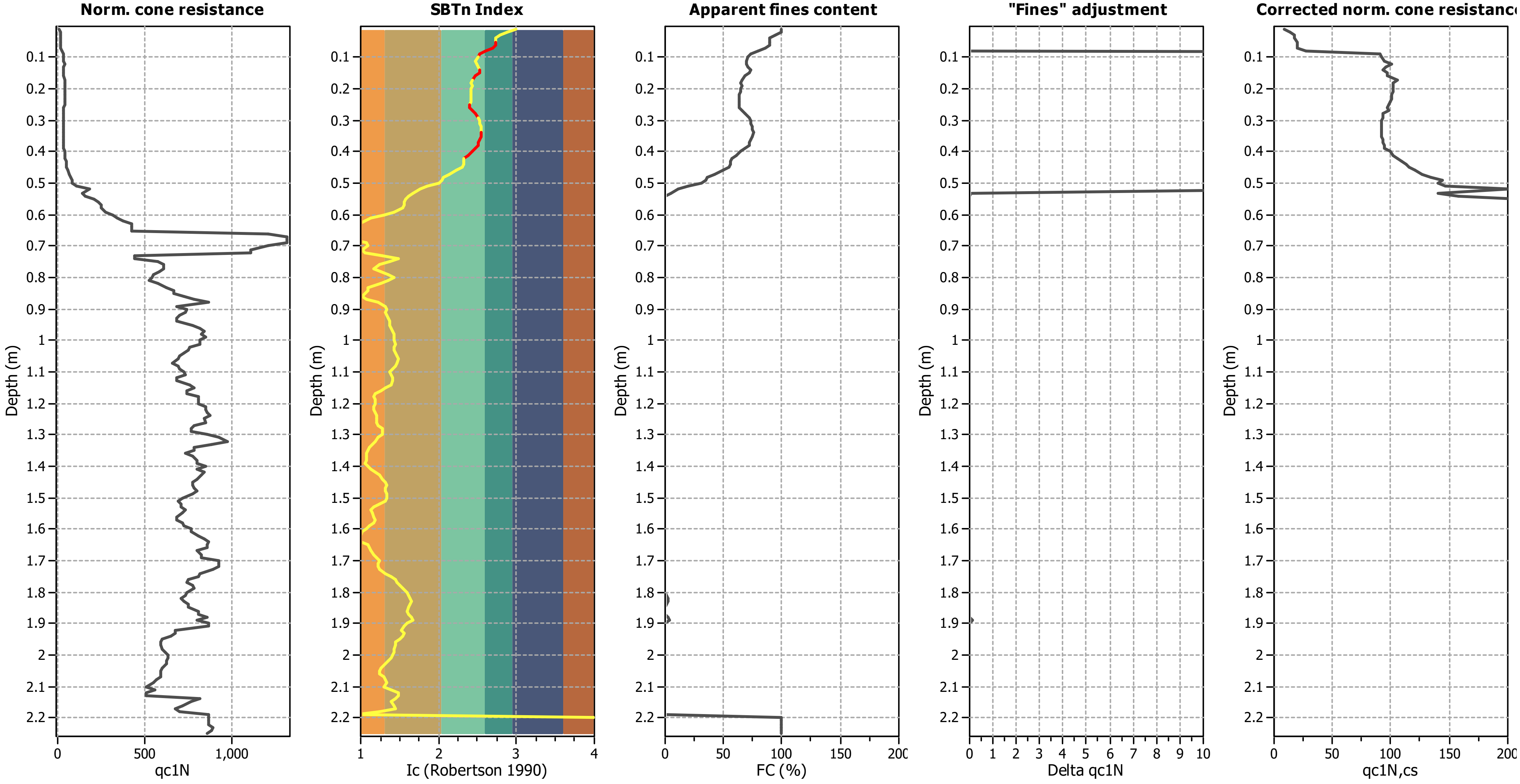


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

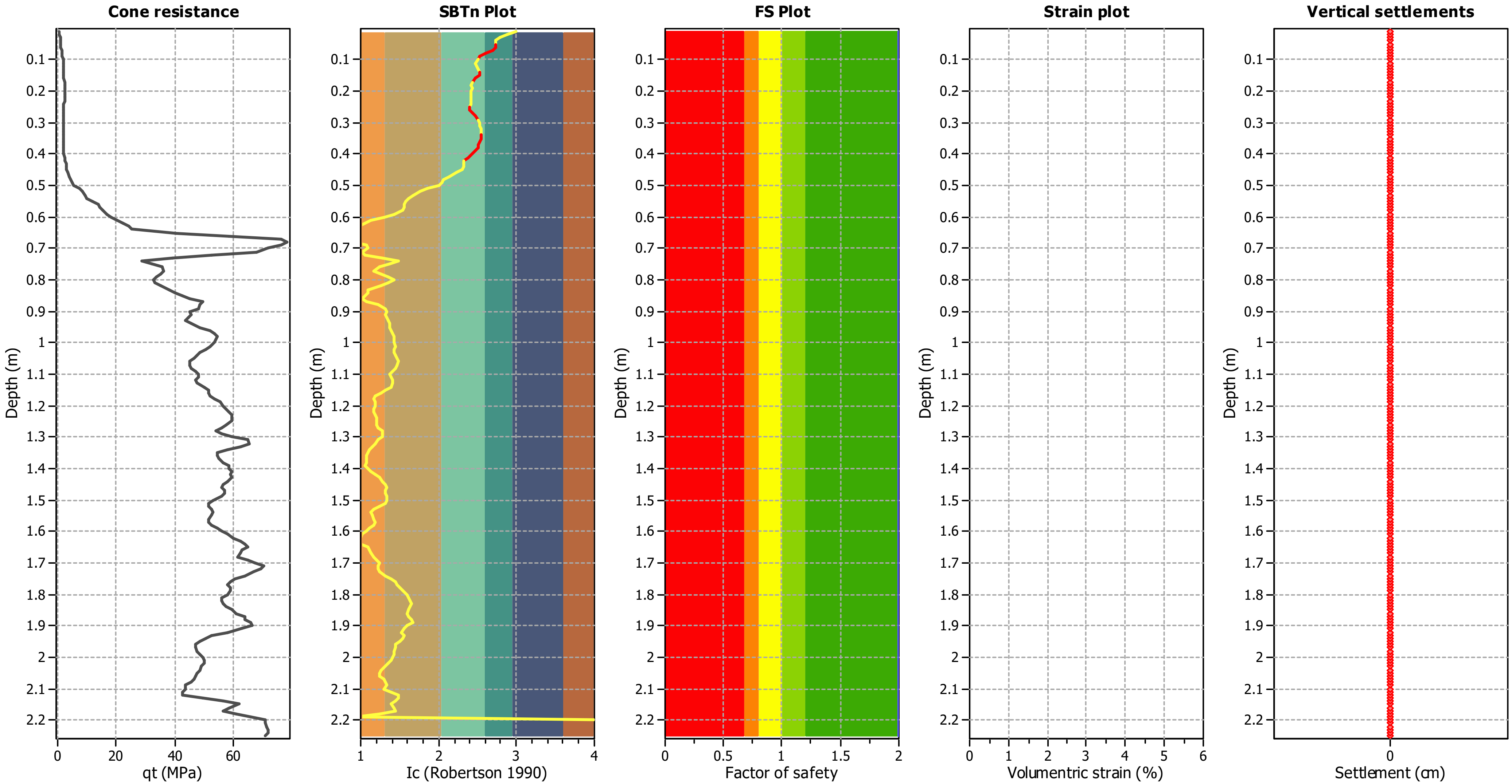
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	10.00 m

Estimation of post-earthquake settlements

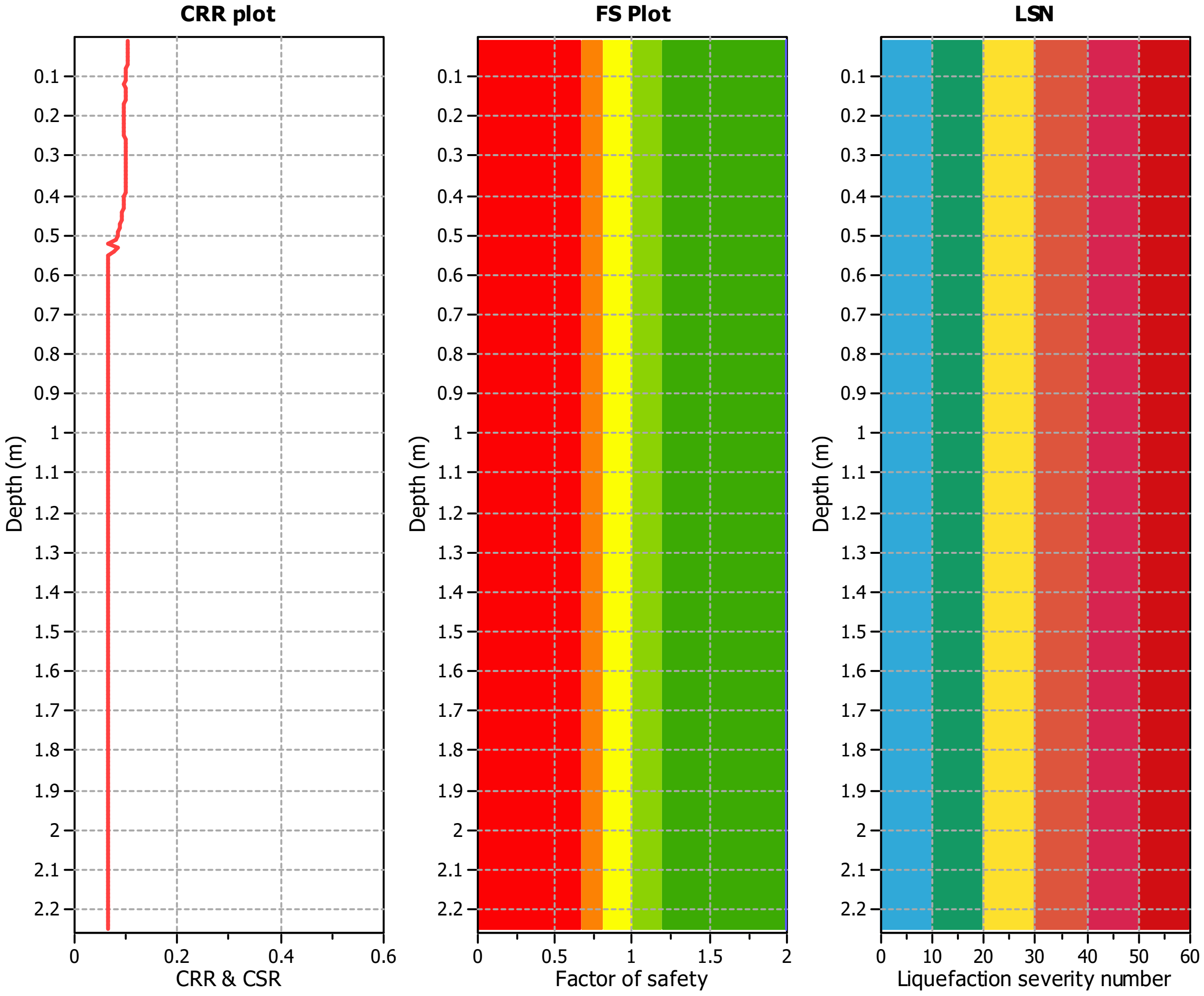


Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Project: **Trices Road Rezoning Group**
Location: **Trices Road Rezoning Area**

CPT: CPT4
Total depth: 2.25 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	10.00 m
Earthquake magnitude M_w:	6.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Ultimate Limit State (ULS) Design Earthquake Event

Unit 3a
Barry Hogan Place
Riccarton 8041

LIQUEFACTION ANALYSIS REPORT

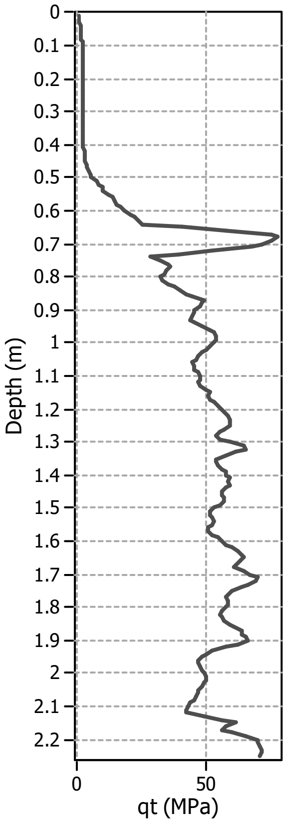
Project title : Trices Road Rezoning Group **Location : Trices Road Rezoning Area**

CPT file : CPT4

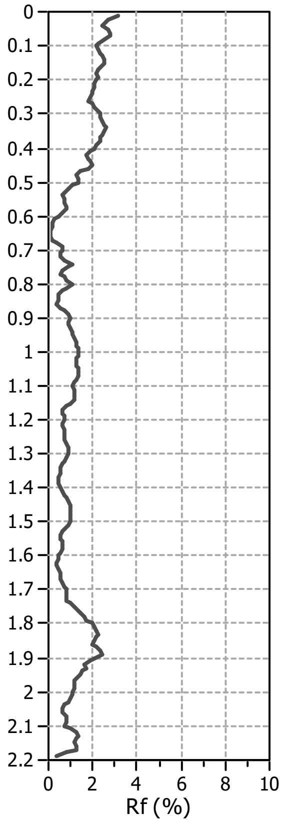
Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	10.00 m
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.35	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

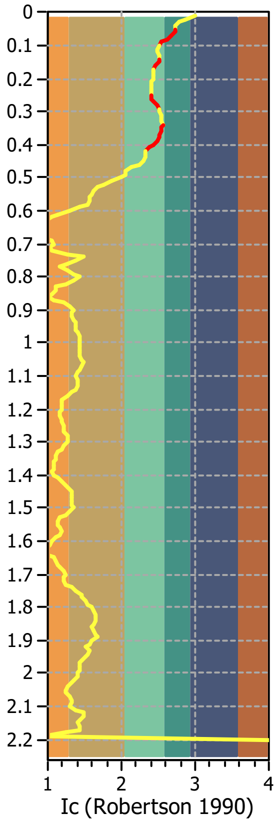
Cone resistance



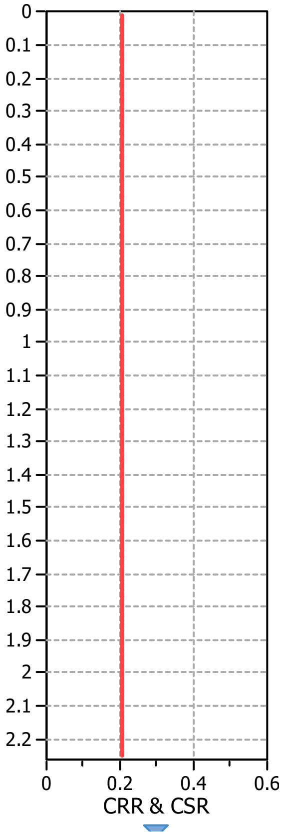
Friction Ratio



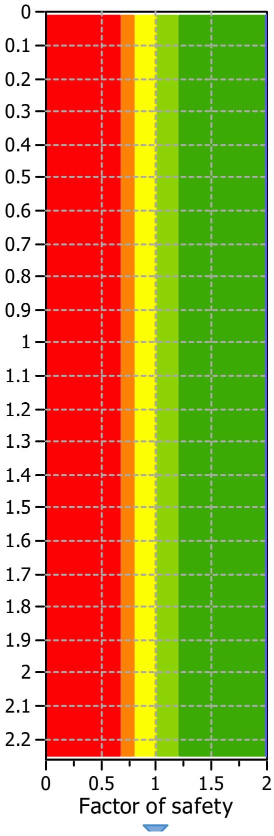
SBTn Plot



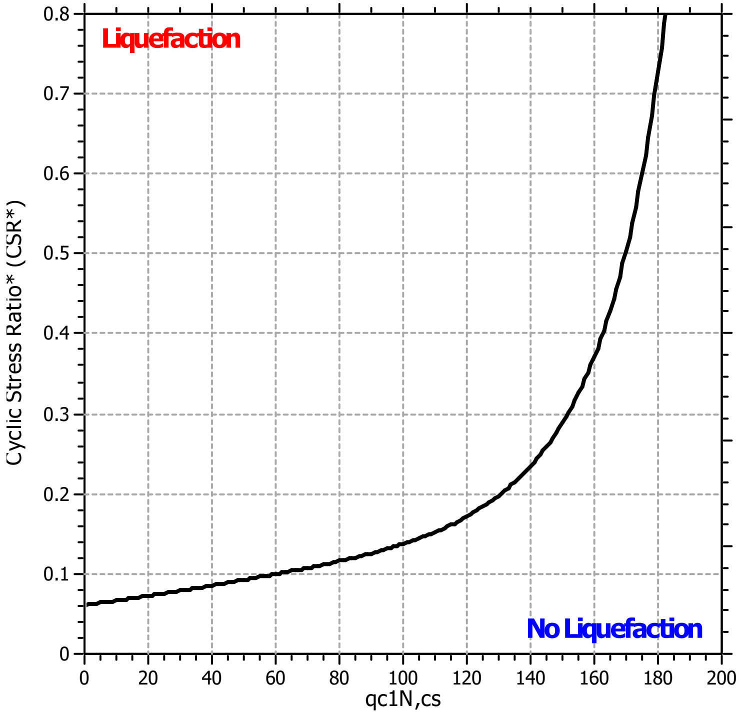
CRR plot



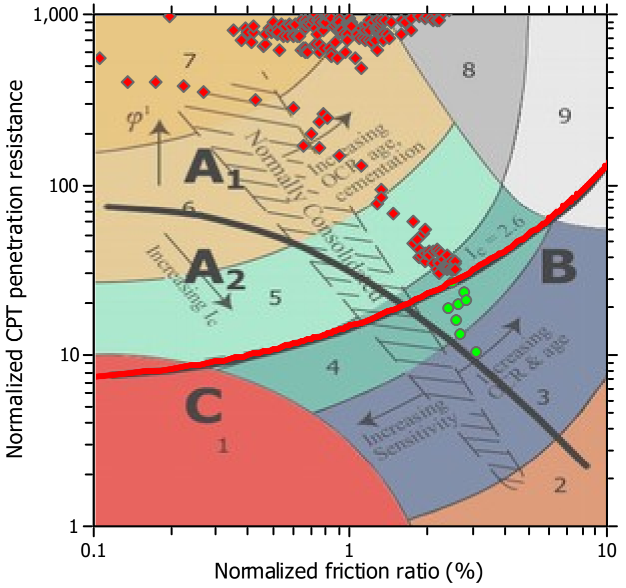
FS Plot



$M_w=7^{1/2}$, sigma'=1 atm base curve

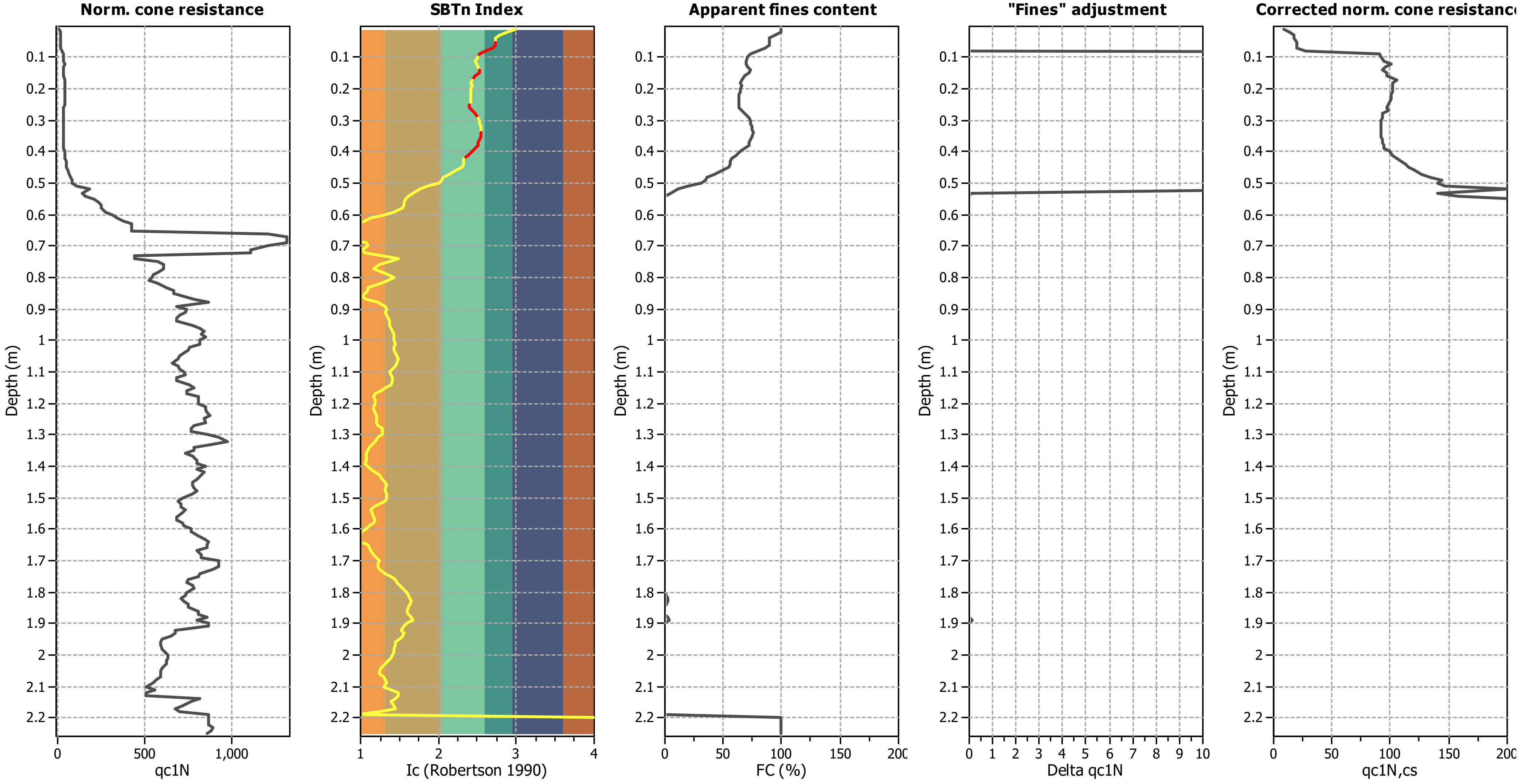


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

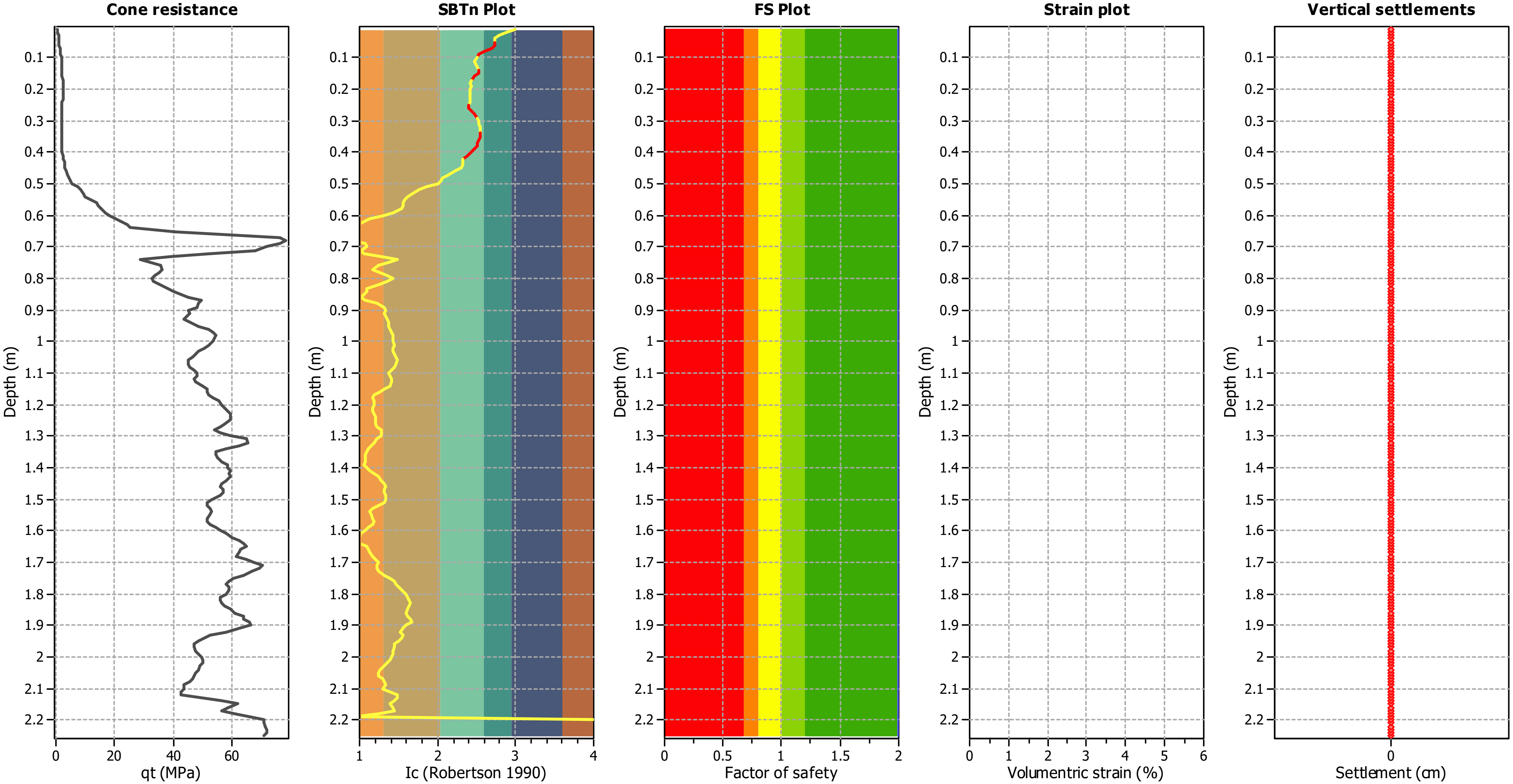
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	10.00 m

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

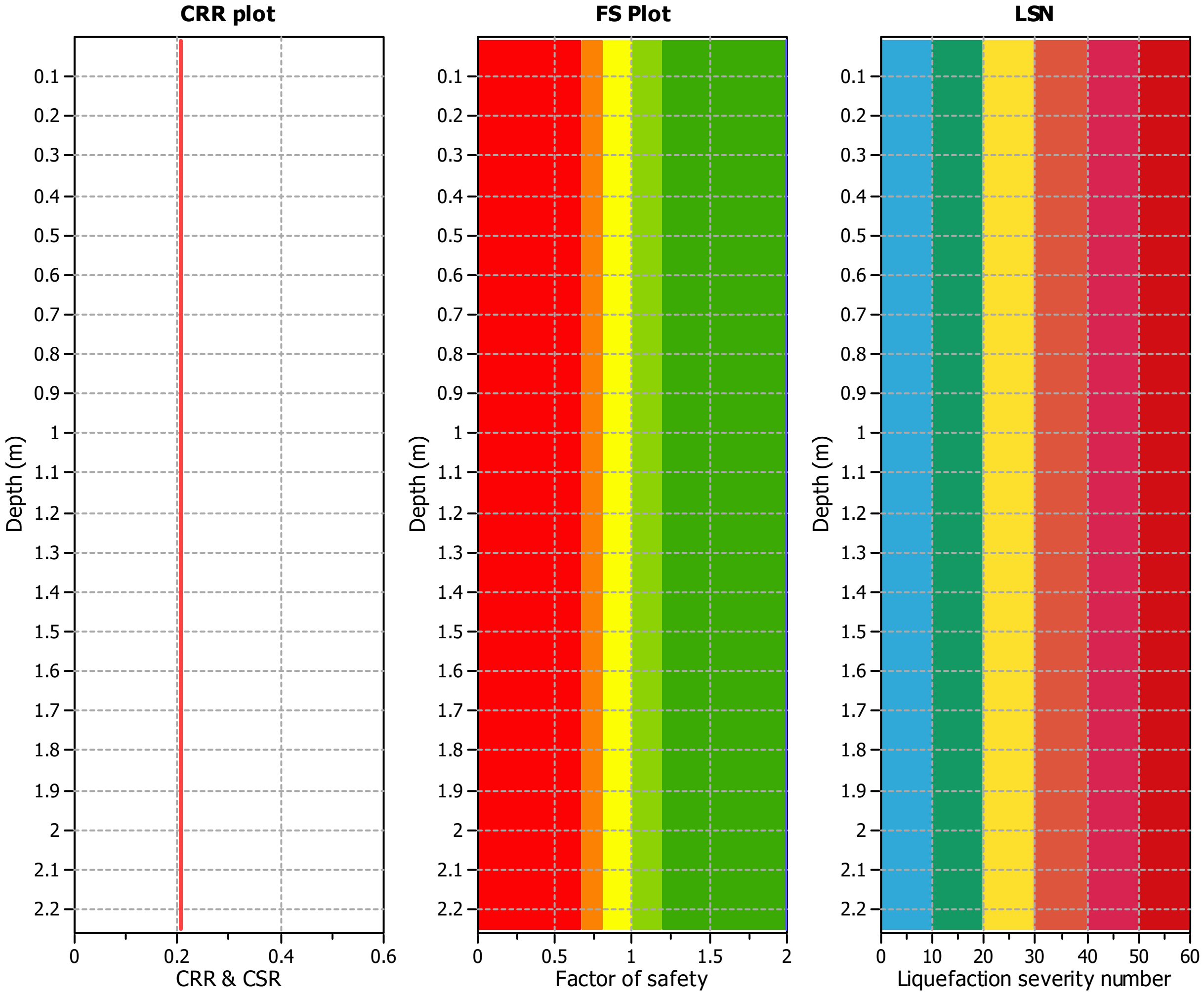
Unit 3a
Barry Hogan Place
Riccarton 8041

Project: **Trices Road Rezoning Group**

Location: **Trices Road Rezoning Area**

CPT: CPT4

Total depth: 2.25 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	10.00 m
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.35	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

CPT 5

Serviceability Limit State (SLS) Design Earthquake Event

(i.e. the larger value determined for the SLS and ILS design earthquake events)

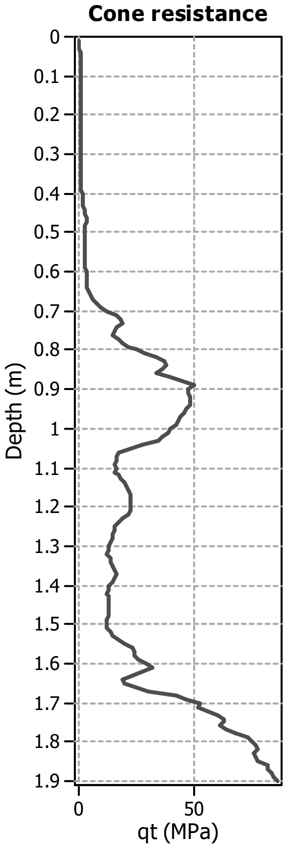
LIQUEFACTION ANALYSIS REPORT

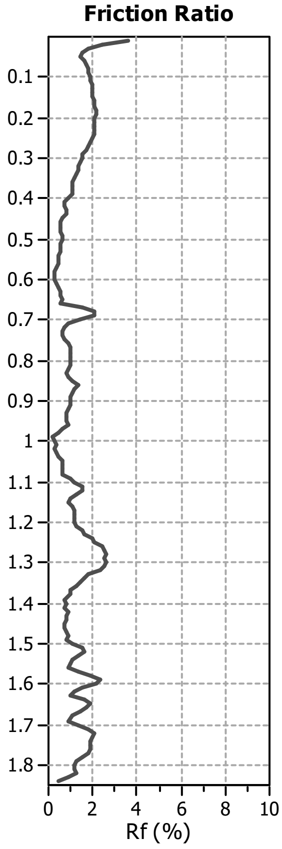
Project title : Trices Road Rezoning Group **Location : Trices Road Rezoning Area**

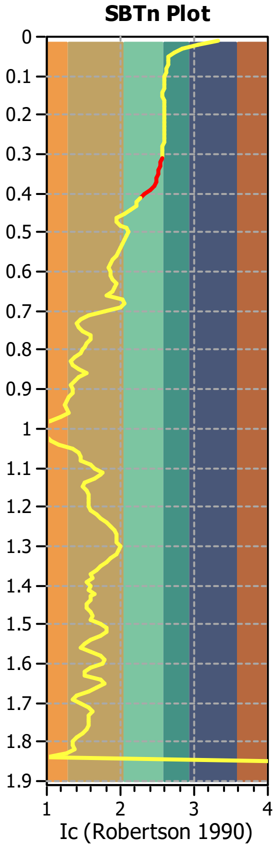
CPT file : CPT5

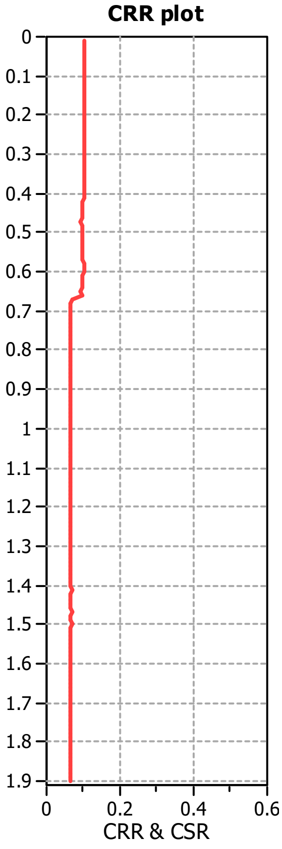
Input parameters and analysis data

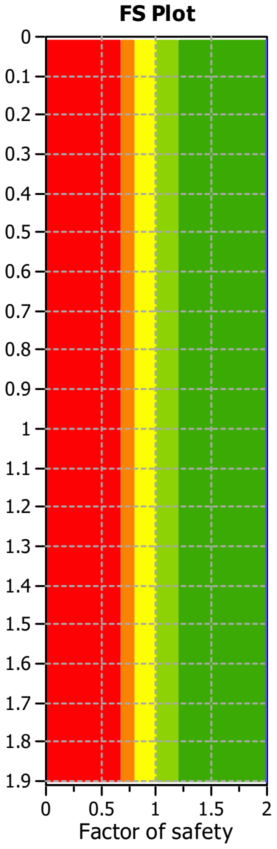
Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	10.00 m
Earthquake magnitude M_w:	6.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

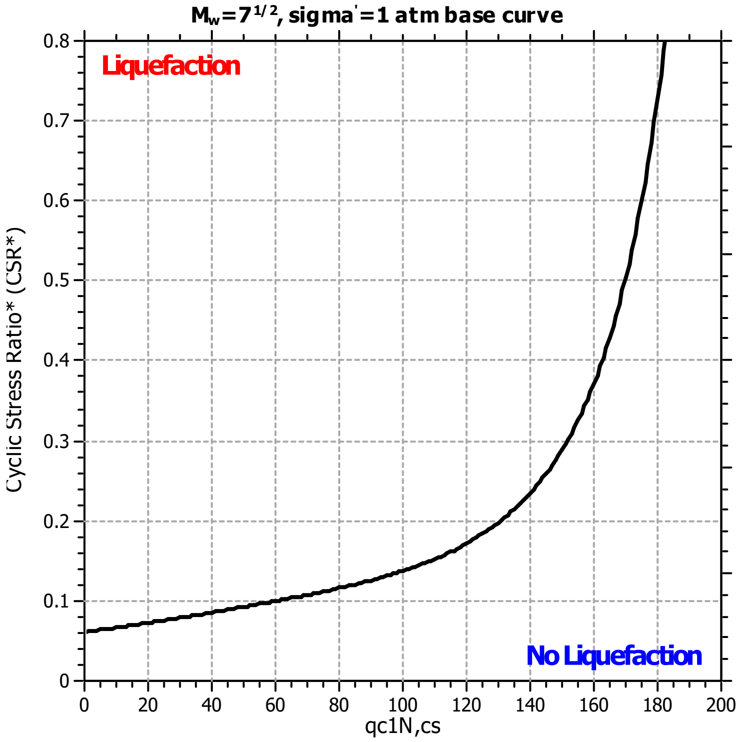


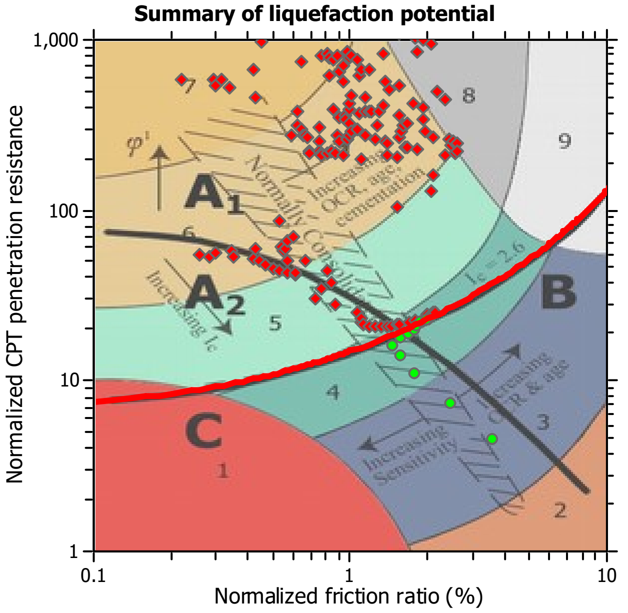






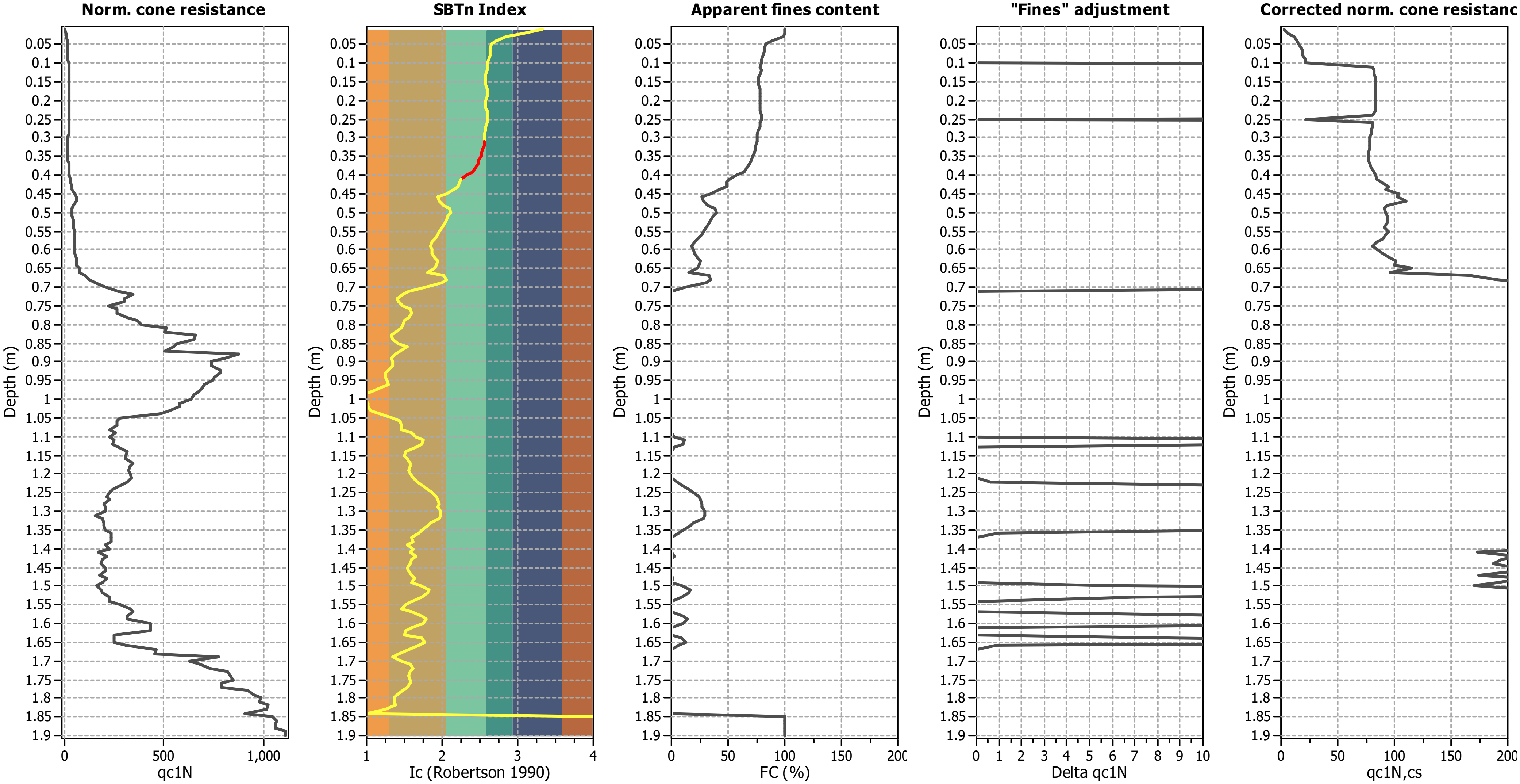






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

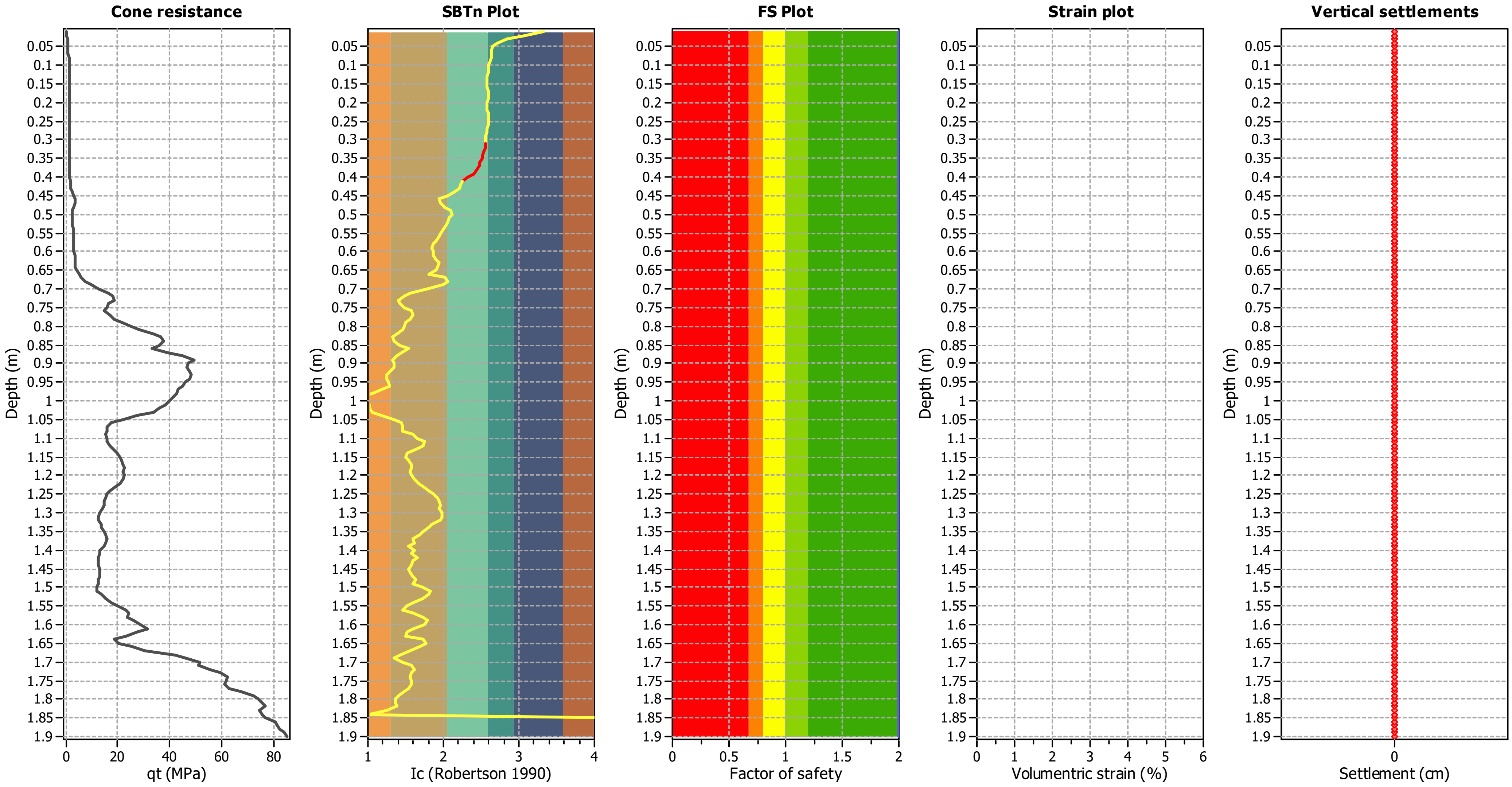
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	10.00 m

Estimation of post-earthquake settlements

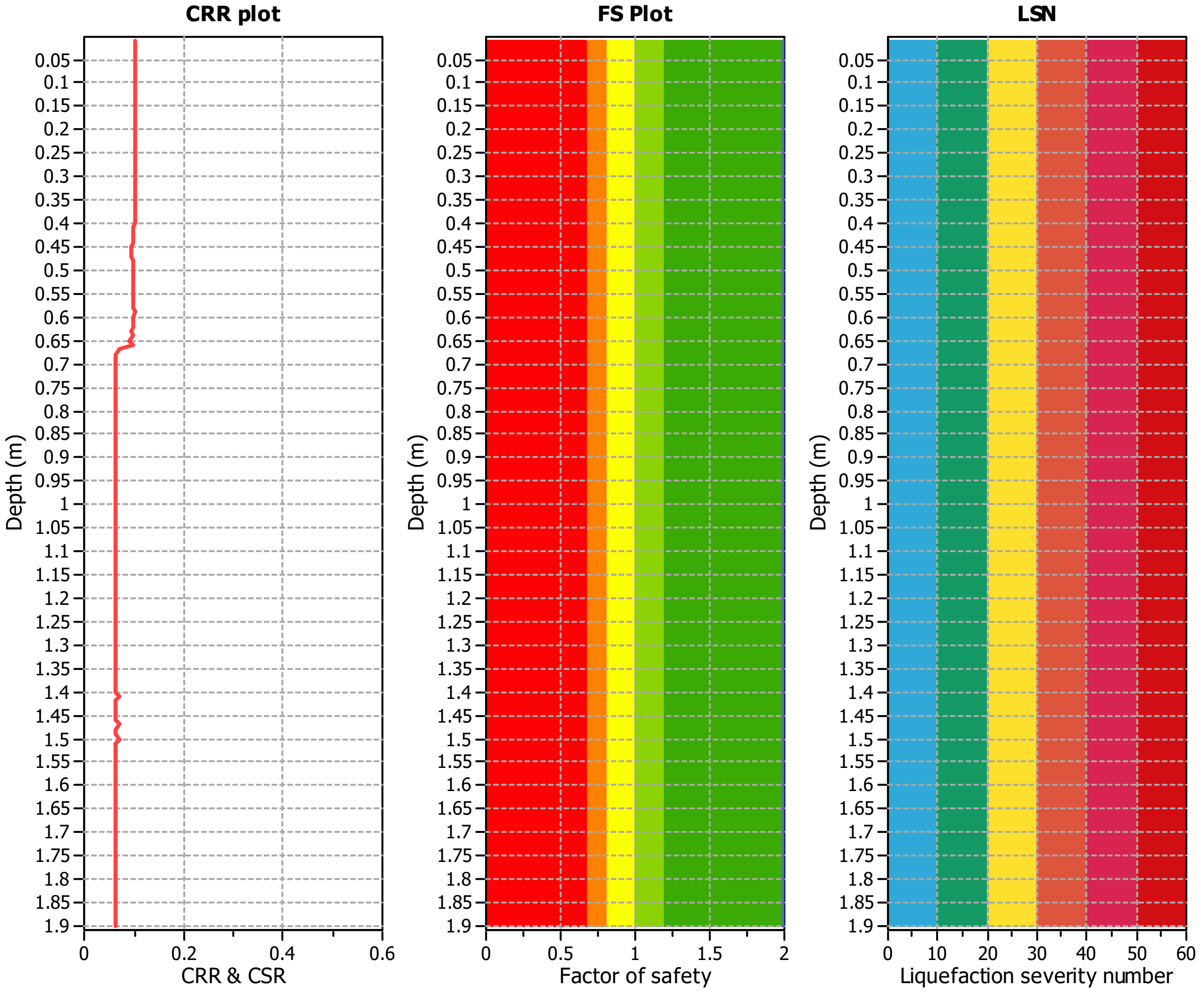


Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Project: **Trices Road Rezoning Group**
Location: **Trices Road Rezoning Area**

CPT: CPT5
Total depth: 1.90 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	10.00 m
Earthquake magnitude M_w:	6.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Ultimate Limit State (ULS) Design Earthquake Event

Fraser Thomas
ENGINEERS · RESOURCE MANAGERS · SURVEYORS

Unit 3a
Barry Hogan Place
Riccarton 8041

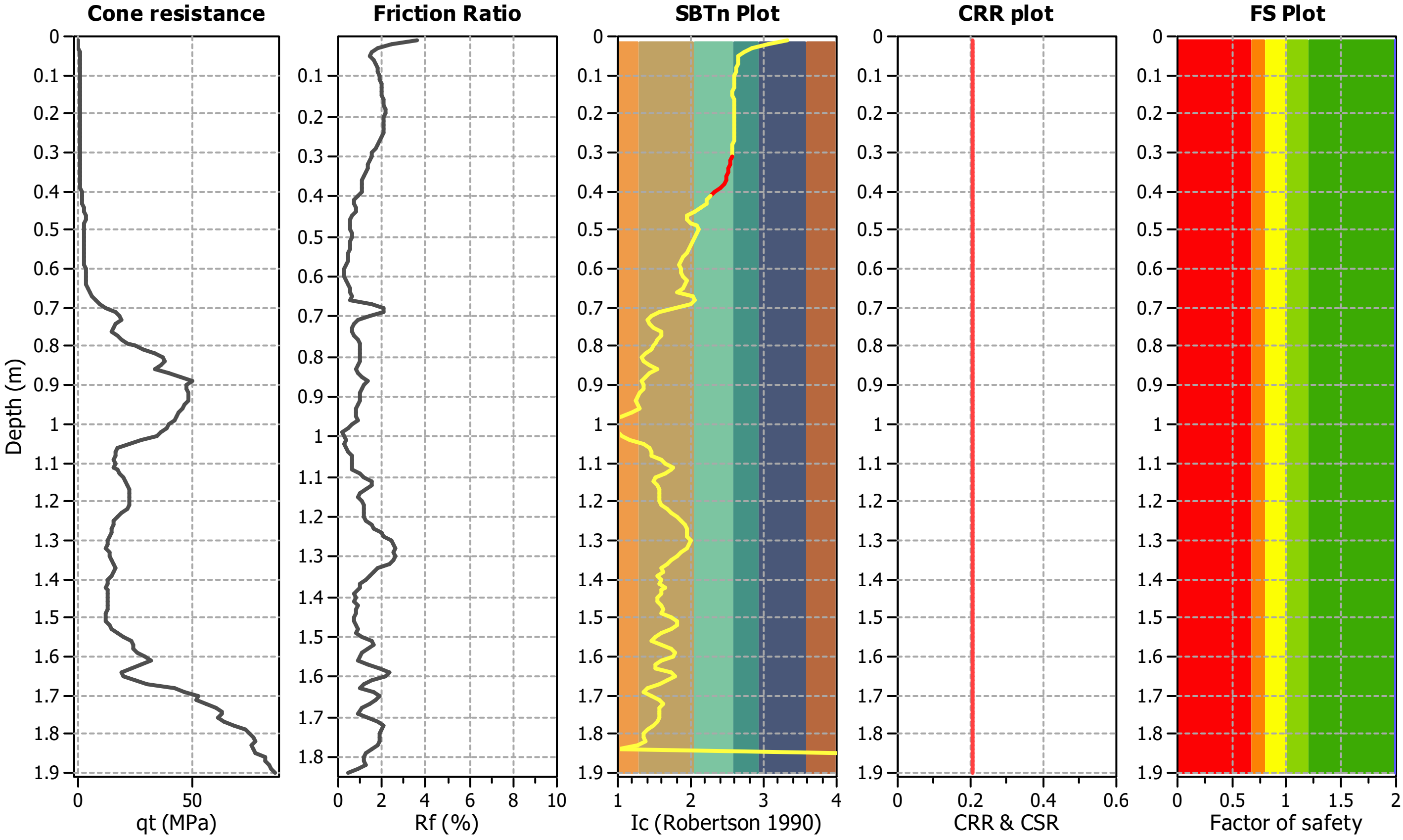
LIQUEFACTION ANALYSIS REPORT

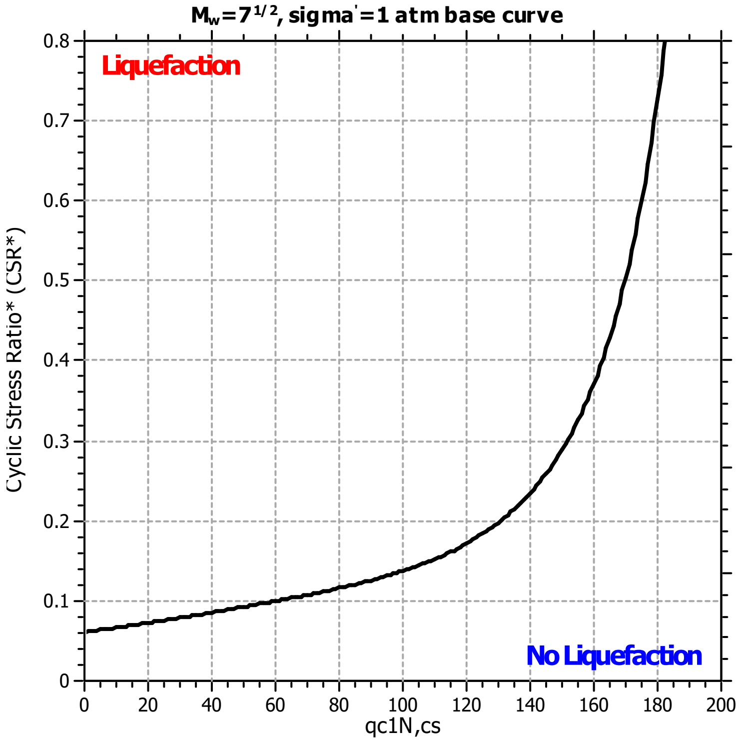
Project title : Trices Road Rezoning Group **Location : Trices Road Rezoning Area**

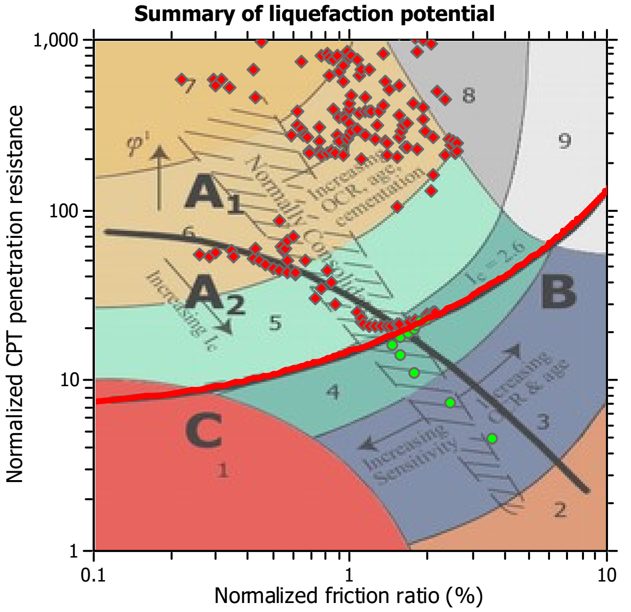
CPT file : CPT5

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	10.00 m
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.35	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

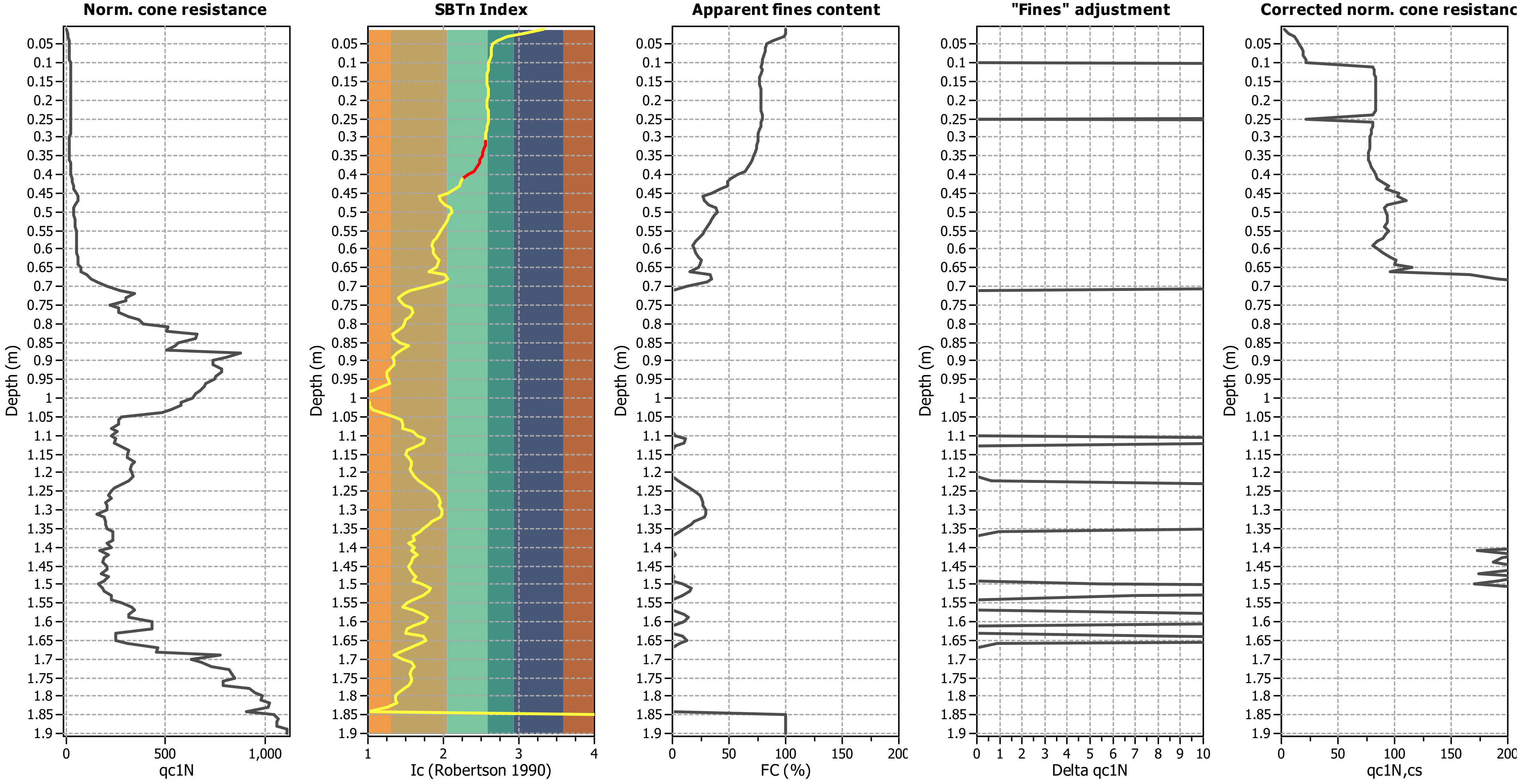






Zone A₁: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A₂: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

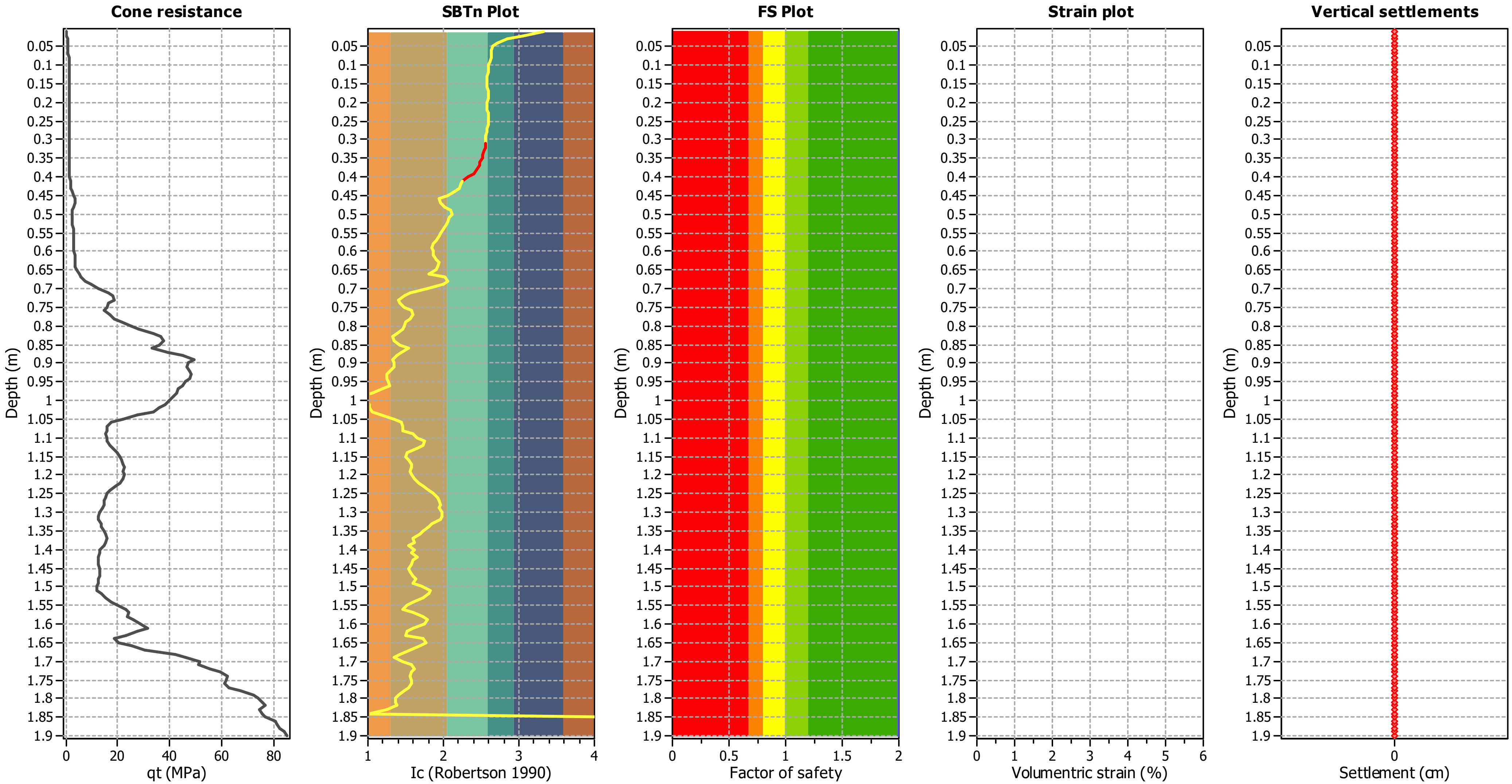
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.35	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	2.50 m	Fill height:	N/A	Limit depth:	10.00 m

Estimation of post-earthquake settlements



Abbreviations

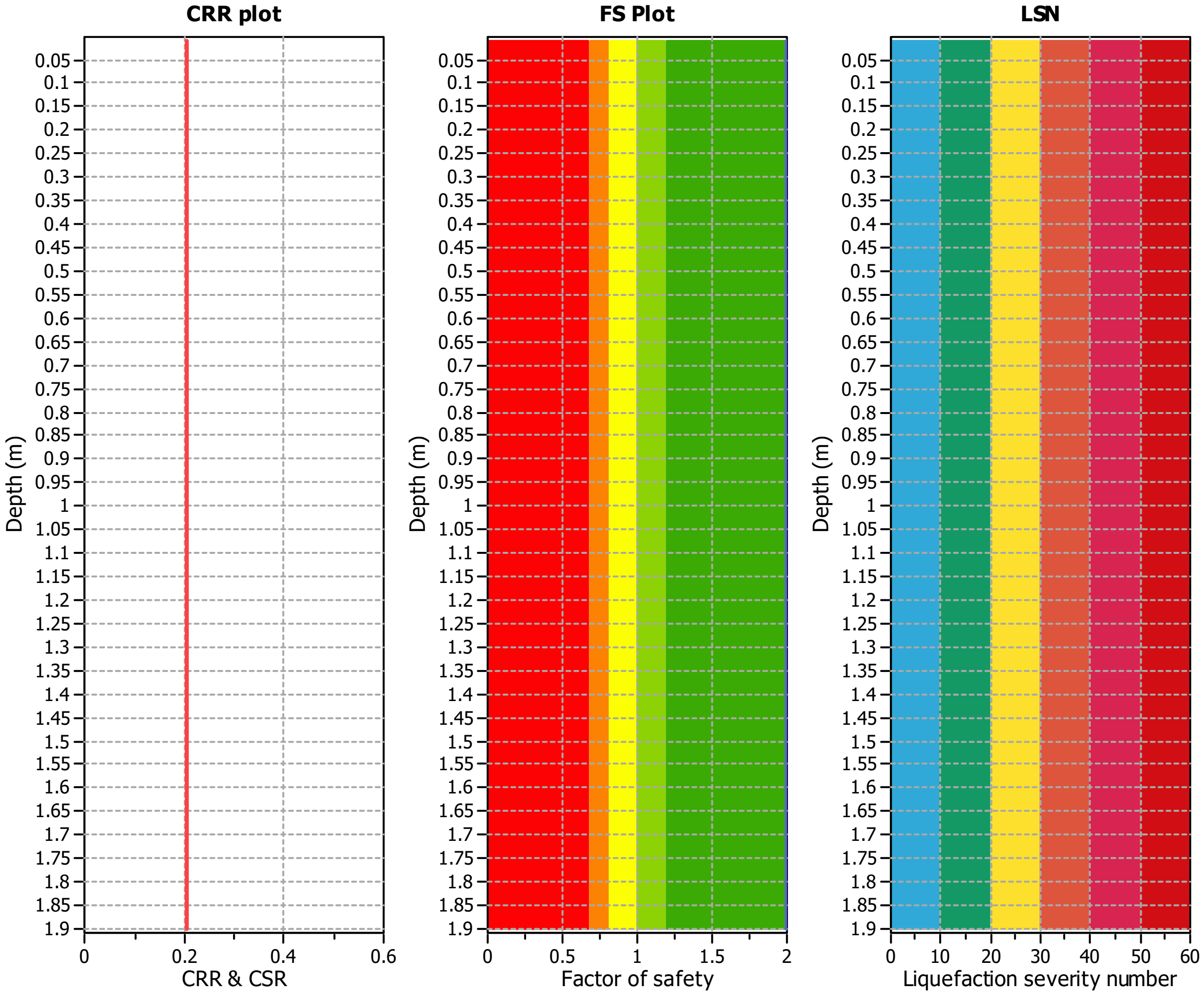
q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Project: **Trices Road Rezoning Group**

Location: **Trices Road Rezoning Area**

CPT: CPT5

Total depth: 1.90 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	.
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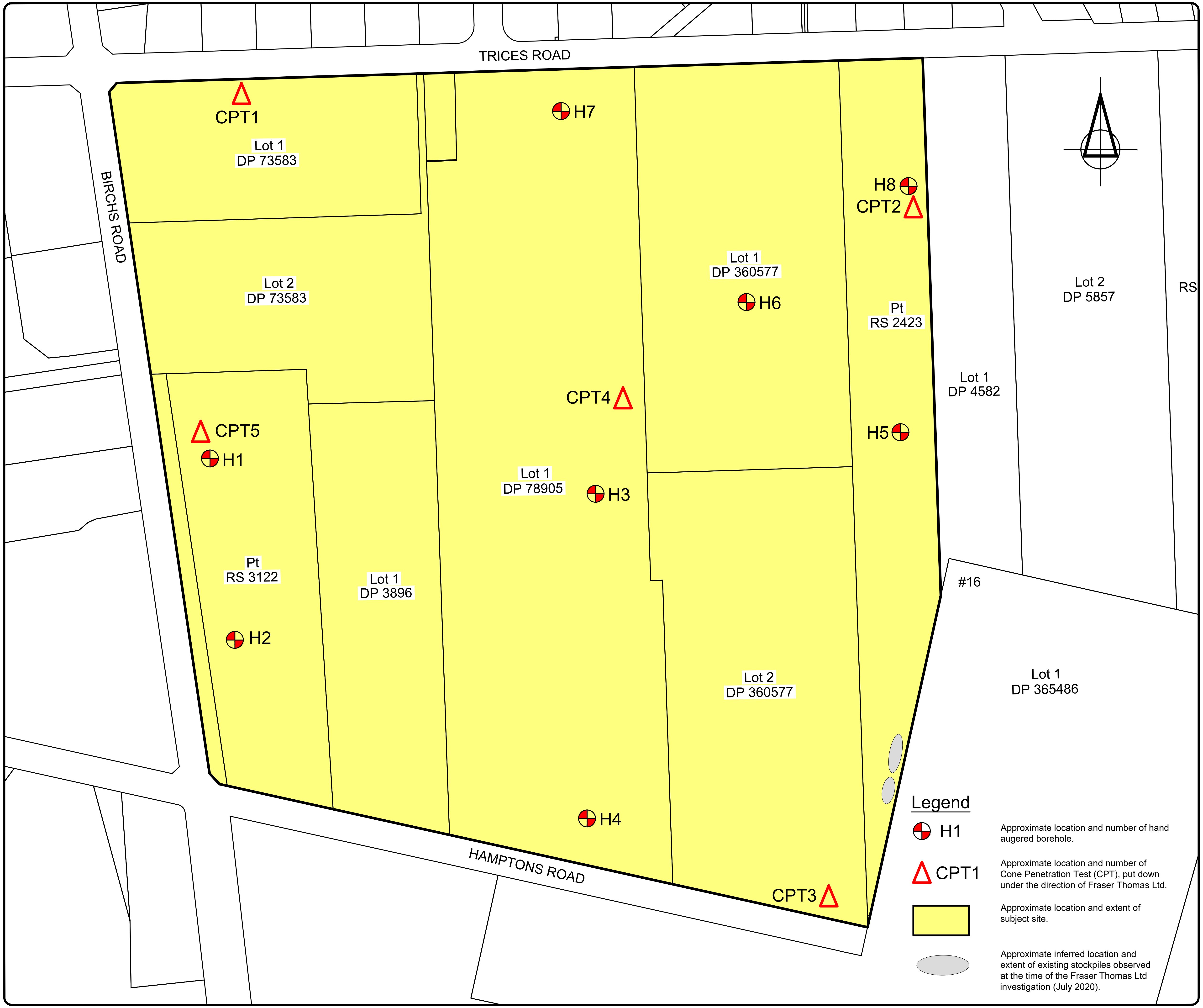

Legend

⊕ H1	Approximate location and number of hand augered borehole.
△ CPT1	Approximate location and number of Cone Penetration Test (CPT), put down under the direction of Fraser Thomas Ltd.
🟨	Approximate location and extent of subject site.
⬭	Approximate inferred location and extent of existing stockpiles observed at the time of the Fraser Thomas Ltd investigation (July 2020).

SURVEYED		
DESIGNED		
DRAWN	JTG	13/07/2020
CAD	GS	14/07/2020
CHECKED		

APPROVED DATE

MVR

REVISION	CHANGES	CHECKED	DATE
A	Issued for information	MVR	22/07/20

NOTES

1. This plan has been adopted from Quick map. The location and extent of the site boundaries and site features are therefore considered to be approximate only.

CLIENT

TRICES ROAD REZONING GROUP

PROJECT

TRICES ROAD AREA
PREBBLETON

TITLE

SITE PLAN

Fraser Thomas

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SCALE 1:2500 (A3)

DRAWING No G00417-01

REVISION A

SHEET 1 of 1