Rosemerryn Subdivision, Lincoln

Stage 25 Geotechnical Investigation Report

Fulton Hogan Land Development Limited

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Executive Summary

Introduction

Fulton Hogan Land Development Limited (FHLD) is proposing to subdivide approximately 2.1ha of rural land in Lincoln, for Stage 25 of the Rosemerryn residential subdivision. The site is located on the eastern edge of the wider Rosemerryn Subdivision that is currently being developed and will be an extension of the subdivision to Ellesmere Road.

FHLD has engaged Aurecon New Zealand Ltd (Aurecon) to undertake a geotechnical investigation and assessment for Stage 25 of the Rosemerryn Subdivision, which is continuation of our work on the wider site since 2005. The purpose of the investigation is to assess the suitability of the land for residential development, and to characterise the risk of liquefaction and lateral spreading to the development.

Geotechnical Investigations

The geotechnical investigations for Stage 25 comprised four Cone Penetration Tests (CPTs). The investigations also drew on the results of the extensive testing that was carried out for the adjacent Stages 19 to 24.

Based on the results of our geotechnical investigations, the ground conditions across the site can be separated into two different ground profiles based on the depth to the underlying gravel. To the north, gravel is at relatively shallow depths of 1m or less, with the depth to gravel deepening towards the south and at the southern corner of the site the gravels are approximately 4m below ground level. The gravel is overlain by interbedded loose to medium dense sands and silty sands, and firm to stiff sandy silts and silts.

Groundwater is at approximately 1.5m depth. It is noted that groundwater levels will vary seasonally or following prolonged rainfall.

Liquefaction Assessment

A liquefaction assessment has been carried out at the site and the results indicate the following:

- The site experienced significant ground shaking during the 4 September 2010 Darfield Earthquake which resulted in no observed or recorded ground damage.
- Due to the level of shaking during the Deerfield Earthquake the site has been assessed a being 'sufficiently tested' (MBIE Guidelines, 2012) to well in excess of the Serviceability Limit State earthquake event without any observed ground damage.
- The site has been assessed as having a low to moderate liquefaction hazard, with a greater risk towards the southern end where the upper gravelly soils are located at a greater depth.

Technical Category Classification

Based on our liquefaction assessment we consider that the northern part of Stage 25 is consistent with the classifications of **Technical Category 1 (TC1)** and the remainder of the site is consistent with the classification of **Technical Category 2 (TC2)**. Across Stage 25 future land damage from liquefaction is unlikely in the Technical Category 1 area, and possible in the Technical Category 2 area in future large earthquakes. The locations of the Technical Category zones are shown on see Figure 2 in Appendix A.

RMA Section 106 Assessment

A risk assessment approach has been undertaken on the significant geotechnical hazards that may affect the site (see Appendix I). Based on this assessment we consider that there are no significant geotechnical hazards at the site other than the potential for earthquake induced soil liquefaction. Provided that the geotechnical recommendations provided within this report are followed, and the appropriate engineering measures are implemented, then we consider that the development is unlikely to be affected by significant geotechnical hazards nor will the development worsen, accelerate or result in material damage. **Therefore, from a geotechnical perspective we consider that the residential subdivision development will comply with the requirements of RMA Clause 106.**

The geotechnical investigations were aimed at assessing the site for geotechnical suitability for subdivision into residential lots with associated access roads and rights-of-way. Detailed design of house foundations is not part of this report and will need to be undertaken by the individual lot owner. This report shall be read as a whole, and our Explanatory Statement is provided in Section 1 below.

1 Introduction

1.1 General

Fulton Hogan Land Development Limited is proposing to subdivide approximately 2.1ha of rural land in Lincoln, for Stage 25 of the Rosemerryn residential subdivision. The site is located on the eastern edge of the wider Rosemerryn Subdivision that is currently being developed and will be an extension of the subdivision towards the east up to Ellesmere Road.

Fulton Hogan Land Development Limited (FHLD) has engaged Aurecon New Zealand Ltd (Aurecon) to undertake a geotechnical investigation and assessment for Stage 25 of the Rosemerryn Subdivision, which is continuation of our work on the wider site since 2005. The purpose of the investigation is to assess the suitability of the land for residential development, and to characterise the risk of liquefaction and lateral spreading to the development along with any other applicable geotechnical hazards. The scope of the works undertaken was as follows:

- A detailed desk study of readily available geological and geotechnical information available for this site.
- A site walkover by a Geotechnical Engineer.
- Review the existing geotechnical work carried out in the area by Aurecon.
- Undertake further geotechnical investigations comprising four cone penetrometer tests.
- A liquefaction analysis using latest MBIE and NZGS (2021) Guidelines to identify the liquefaction potential of the underlying natural soils and to confirm the technical categories across the site based on the liquefaction assessment.
- Provide recommendations on potential liquefaction remediation options for the site.
- Provide recommendations for further testing (if required).
- Assess the site against Section 106 of the Resource Management Act (RMA).
- Prepare a geotechnical investigation report for Rosemerryn Subdivision Stage 25.

This geotechnical report presents the results of our geotechnical investigations and assessment, confirms the suitability of the land for residential development, as well providing recommendations for site development.

Our work has been carried out under the existing ACENZ/IPENZ Short Form Agreement between FHLD and Aurecon, as per Aurecon's fee proposals dated 31 January 2022.

This report shall be read as a whole.

1.2 Explanatory Statement

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

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

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

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

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2 Site Conditions

2.1 Site Description

The site is located in Lincoln, southwest of Christchurch, on the eastern side of the wider Rosemerryn subdivision. Site is bounded by Ellesmere Road to the east and previous Rosemerryn subdivision stages to the west, which is presented in Figure 2 of Appendix A. The main features are:

- The site has an approximate area of 2.1ha and has a triangular shape
- The site topography is relatively flat with less than 1.5m height change across the area.
- The site is bounded to the north by rural land, to the west by previous stages of the Rosemerryn Subdivision and to the east and south by Ellesmere Road.
- There is a small stream which runs through the Rosemerryn subdivision at the south end of Stage 25. The stream is approximately 0.5m deep and 2m to 3m wide with gently sloping sides.
- The site is currently being used for pastoral and crop farming and is covered in grass with an existing dwelling on site.
- Current drainage is inferred to be via direct soakage to the ground or via runoff to the small stream.

2.2 Regional Geology

The geology of the site is shown on the Geological and Nuclear Sciences Map 16, Geology of Christchurch area, scale 1:25,000 (compiled by Forsyth, Barrell and Jongens, 2008). The map indicates that the site is underlain by *grey river alluvium beneath plains of low-level terraces (Q1a).*

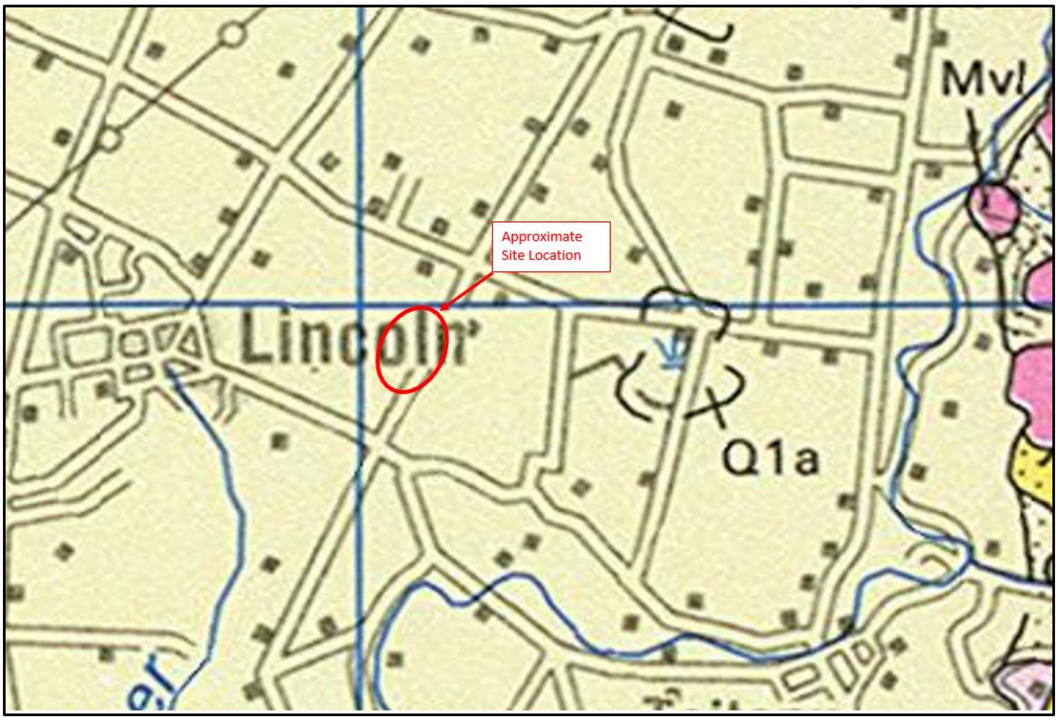

Figure 1: Geological map of site

2.3 Seismicity

The GNS Science Active Fault System database (GNS, 2012a and 2012b) indicates that the site is within an area of recent seismic activity known as the Canterbury Earthquake Sequence (CES) and is approximately:

- 12km south-east of the eastern extension of the Greendale Fault, which was responsible for the Magnitude M_w7.1 Darfield (Canterbury) Earthquake on 4 September 2010.
- 16km south-west of the epicentre of the Magnitude M_w6.2 Christchurch Earthquake on 22 February 2011 (GNS, 2011b); and
- 21km south-west of the epicentre of the Magnitude M_w6.0 major aftershock on 13 June 2011 (GNS, 2011b); and
- 23km south-west of the epicentre of the Magnitude M_w5.9 major aftershock on 23 December 2011 (GNS, 2011b).

Based on Bradley (2012), Lincoln School, which is approximately 2km west of site, experienced a 0.44g PGA in the September 2010 earthquake.

2.4 Recorded Earthquake Damage

Based on the GNS report "*Review of liquefaction hazard information in eastern Canterbury, including Christchurch City and parts of Selwyn, Waimakariri and Hurunui*" (GNS, 2012), there was no observed liquefaction induced ground damage after the 4 September 2010 or 22 February 2011 earthquakes. Minor surface expression of liquefaction was observed in areas 500m southeast of the site.

Based on reviews of aerial photography, discussions with Fulton Hogan staff who are familiar with the site, and Aurecon site walk overs in 2011, 2012, 2013, 2015 and 2018, no surface expression of liquefaction or land cracking occurred within the proposed subdivision. The lack of observed liquefaction induced ground damage is consistent with the GNS report.

2.5 MBIE Land Classification

The current land classification for the site, according to the Ministry of Business Innovation and Employment (MBIE) Technical Categories map, is "*N/A – Rural & Unmapped*". To the east of the site on the eastern side of Ellesmere Road it is classified as "*Technical Category 2*" and to the west of the site it is classified as "*Technical Category 1*".

"*N/A – Rural & Unmapped*" means that normal consenting procedures apply in these areas. "*Technical Category 1*" means that future land damage from liquefaction is unlikely, and ground settlements are expected to be within normally accepted tolerances. Standard foundations (NZS 3604) are acceptable in TC 1 areas subject to shallow geotechnical investigation. "*Technical Category 2*" means that minor to moderate land damage from liquefaction is possible in future large earthquakes. Standard foundations (NZS 3604) cannot be used. Lightweight construction or enhanced foundations are likely to be required such as enhanced concrete raft foundations (i.e. stiffer floor slabs that tie the structure together).

3 Geotechnical Investigations

3.1 General

The objective of the geotechnical review and site investigation was to determine the ground and groundwater conditions across the site in order to assess the suitability of the site for subdividing into residential sections.

Geotechnical investigations have been carried out across the site at various stages since August 2011 with more recent investigations in Stages 19 to 24 carried out in May 2018. As part of our assessment for the site we have reviewed previous investigations on and around Stages 19 to 24 (adjacent to Stage 25), as well as the results from the recent investigations.

The geotechnical review and investigation included the following information:

- Readily available Environment Canterbury well logs from Canterbury Maps.
- Previous geotechnical investigations, which comprised geotechnical boreholes, test pits, cone penetration tests (CPT) and Multi-channel Analysis of Surface Waves (MASW).
- Additional investigations which comprised four CPTs to target depths of 10m or refusal.

Details of the geotechnical investigations are presented in the following sections.

3.2 Environment Canterbury Well Logs

A review of the Canterbury Maps and Environment Canterbury GIS Database (ECan, 2015) indicates three Environmental boreholes with logs on the site. The borehole logs, locations, and depths are summarised in Table 1 below.

Table 1: Summary of ECan borehole logs

Borehole	Location	Depth	Groundwater Depth	Summary of Stratigraphy
M36/8675	Southern end of site	5.8m	1.5m	0-0.2m - Topsoil 0.2-3.6m - Silty Clay 3.6-5.8m - Silty Sandy gravel
M36/7299	Northern end of site	18m	2.2m	0-0.2m - Topsoil 0.2-4.5m - Clay 4.5-11m – Sandy gravels 11-16m – Claybound gravels 16-18m – Sandy gravels
M36/3324	Western side of site	42m	Unknown	0-0.5m Topsoil 0.5-9m Gravel and pug 9-14m Gravel 14-20m pug and wood 20-42m Gravel interbedded with clay

The locations of the ECan borehole locations are presented in Figure 5 in Appendix A and the borehole logs presented in Appendix B.

3.3 Previous Geotechnical Investigations

Aurecon has completed a series of staged ground investigations as part of the development for the wider Rosemerryn subdivision to the west of the site. These investigations are detailed in full in the subdivision consent report for Stages 19 to 24, 224464-0004-REP-GG-0001, Rev0, dated 22 June 2018.

Previous investigations carried out on and around Stages 19 to 24 have comprised of geotechnical boreholes, test pits, cone penetration tests (CPT) and Multi-channel Analysis of Surface Waves (MASW). A summary of the previous investigations is presented in Table 2.

Table 2: Summary of relevant previous investigations

Year	Testing type	Relevant Test
2011	Boreholes	BH3 and BH4
2011	CPTs	CPT18 to CPT27
2011	Test Pits	TP33 to TP47
2012	CPTs	CPT1, CPT2, CPT4 and CPT27
2012	Test Pits	TP1
2013	CPTs	CPT19, CPT21, and CPT22
2015	Boreholes	BH102 and BH103
2015	MASW	3.1km of MASW line carried out of which approximately 1.1km is in Stage 19 to 24.
2018	CPTs	CPT201, CPT202, CPT203, CPT204, CPT205, CPT206, CPT207, CPT208, CPT209, CPT210, CPT211, CPT212, CPT213, CPT214 and CPT215

The locations of these investigations are presented in Figures 1 in Appendix A.

We have considered these investigations, alongside our understanding of the wider site geological environment, to help constrain the subsoil profile in Stage 25.

3.4 Recent Aurecon (2022) Investigations

3.4.1 Cone Penetration Testing

Four Cone Penetration Tests (CPT) were undertaken as part of Stage 25 on 11 February 2022. The CPT's were undertaken by McMillian Drilling using a track mounted CPT rig and the tests were undertaken to effective refusal (tip pressure reaching 40MPa) of the rig at 2m to 7m depth. The CPT locations are shown in Figure 1 in Appendix A and the logs are present in Appendix C.

4 Engineering Considerations

4.1 General

Fulton Hogan Land Development Limited is proposing to subdivide 2.1ha of rural land in Lincoln into Rosemerryn Stage 25. The Ministry of Business, Innovation and Employment (MBIE, 2012) guidelines on residential development, requires that ground conditions and geotechnical hazards, including liquefaction, are assessed and based on the result of this assessment, mitigation measures (if required) can be developed.

This section of the report presents the:

- Geotechnical ground model for the site.
- Potential for seismically induced liquefaction.
- Implications for building foundations.
- Assessment against the Resource Management Act (RMA) Section 106.

Considerations for this section have been made with the previous knowledge of extensive ground investigations completed in Stages 19 to 24 of the western edge of Stage 25. A full analysis for these investigations is presented in the subdivision consent report for Stages 19 to 24, 224464-0004-REP-GG-0001, Rev0, dated 22 June 2018.

4.2 Geotechnical Ground Model

4.2.1 Ground Conditions

Based on the results of our geotechnical site investigation results, which ranged from ECan borelogs on site, previous borehole investigations (BH201, BH202, BH203), and current CPT investigations, the ground profile can be summarised as two separate models, Profile One being north ed of the site and profile Two being the south end of the site. Both profiles comprise similar materials, the main difference being the depth to the top of the shallow gravel layer being shallower at the north end of the site. These profiles are summarised in the Tables 3 and 4 below.

Table 3: Inferred ground profile 1 (Northern section of site)

Unit	Depth to Start of Layer	Depth to End of Layer	Material
N1	Surface	0.2 to 0.3m	Topsoil
N2	0.2 to 0.3m	0.9 to 1m	Loose to medium dense Sand and silty sands interbedded with gravelly sand, firm to dense silty sands and Sand
N3	0.9 to 1m	10m onwards	Gravel and sandy gravel with occasional sand lenses

Table 4: Inferred ground profile 2 (Southern section of site)

Unit	Depth to Start of Layer	Depth to End of Layer	Material
S1	Surface	0.2 to 0.3m	Topsoil
S2	0.2 to 0.3m	4 to 4.2m	Loose to medium dense Sand and silty sands interbedded with clayey silts, firm to dense silty sands and Sand and Gravel
S3	4 to 4.2m	10m onwards	Gravel and sandy gravels

Figure 2 presented in Appendix A shows the demarcation line between these two soil profiles.

The key difference between the soil profiles is the depth to the gravel layer. The gravel is at relatively shallow depths in the north part of the site and deepens to the south. Aspects of note are as follows:

- Sand lenses are present within the gravel in the northern section of the site (Ground Profile 1), as noted in Borehole BH102 at 4.56m depth, MASW Line 4 Chainage 20m, and MASW Line 10 Chainage 217m in the previous investigations. The sand lenses appear to be limited in extent, with one lens logged as approximately 1.5m thick.
- In the upper soil profile in the southern section of the site there are soft silt layers interbedded with firm and stiff silt layers. Generally, these soft layers are limited in thickness ranging from 0.2m to 0.5m thick and are typically below 2.5m depth.

The ground conditions encountered in Stage 25 are consistent with those inferred from the previous subdivision stages immediately to the west.

4.2.2 Groundwater

The depth to groundwater is considered critical in determining the likely site performance and therefore our assessment of the groundwater level has been carried out based on the ECan groundwater model, piezometer readings and groundwater levels encountered during the investigations.

From recent CPTs, ground water was encountered at depths of 1.5m to 1.8m below ground level but these levels are potentially inaccurate due to the short time the holes were open not allowing groundwater levels to equalise.

From investigations in previous stages, shallow piezometer readings indicate groundwater levels in the order of 1.4m to 1.5m depth, with the exception of BH203 adjacent to the stream, which indicates groundwater at 1m depth.

For design purposes, and accounting for the expected seasonal variation in groundwater level we have adopted a design groundwater level of 1.5m below ground level.

4.3 Site Flexibility

We have assessed the site flexibility based on the following:

- Site stratigraphy comprises approximately sands and silts underlain by gravels to at least 15m depth (maximum depth investigated at the site).
- Clause 3.1.3 and Table 3.2 of NZS 1170.5:2004.

We consider that the site subsoil category in terms of NZS 1170.5:2004 Clause 3.1.3 is Class D (Deep soil site).

4.4 Liquefaction Assessment

4.4.1 General

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

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

- How has the site performed during the major seismic events of the Canterbury earthquake sequence?
- Which layers have liquefied?
- What is the likelihood of further liquefaction in the future?
- How the potential liquefaction affects the development?

Each of these is considered below.

Observations after Previous Major Earthquake Events

As outlined in Section 2.4 there is no evidence of surface expression of liquefaction observed at the site after the 4 September 2010 Darfield Earthquake or any subsequent earthquakes during the Canterbury Earthquake Sequence. This lack of expression suggests limited potential for soil liquefaction at the site for shaking levels close to a ULS design event.

Potential for Liquefaction

Three primary factors contribute to liquefaction potential:

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

Each of these is discussed below.

Soil Grading and Density

The CPT logs show layers of loose to medium dense sands, silty sands and sandy silts. These layers are considered to be potentially susceptible to liquefaction from a soil grading and density perspective.

Groundwater

We have adopted a groundwater level of 1.5m below ground level based on piezometer readings from Stages 19 to 24 and the information from Stage 25 investigations. It should be noted that groundwater levels are subject to seasonal changes.

Earthquake Intensity and Level of Shaking

The level of ground shaking is one of the key factors in determining whether liquefaction will or will not occur. For this analysis, we have assessed three design levels of shaking. The residential structures to be constructed on site will likely be classified as Importance Level 2 (IL2) structures in accordance with Table 3.2 of the New Zealand structural loadings standard (NZS 1170.0.2004) and the building will have a nominal 50 year design life. To determine the design level for earthquake shaking we have adopted the MBIE/NZGS (2021) recommendations, which correspond to design level earthquake events as follows:

- ULS shaking a M_w7.5 earthquake with 0.35g peak ground acceleration (PGA)
- SLS-a shaking a M_w7.5 earthquake with 0.13g PGA
- SLS-b shaking a M_w6.0 earthquake with 0.19g PGA

For an Ultimate Limit State (ULS) earthquake, buildings are expected to retain their structural integrity and form and not endanger life. Some plastic deformation of structural elements within the structure is expected to occur but ideally the damage can be repaired and the structure can be returned to service after the event, although repair may be uneconomical.

For a Serviceability Limit State (SLS) earthquake, buildings are expected to perform well for the SLS event and be returned to service after limited repair.

Based on the PGA model from BA Bradley (2012) and MBIE Guidelines (2012) the site has been 'sufficiently tested' as the PGA for the 4 September 2010 event exceeded 170% of the SLS PGA (i.e 1.7 x 0.13g = 0.22g). The levels of shaking used for our analysis are presented in Table 5.

Table 5: Design earthquake parameters

Earthquake Event	Magnitude	Peak Ground Acceleration
ULS	M_w7.5	0.35g
SLS-a	M_w7.5	0.13g
SLS-b	M_w6.0	0.19g

4.4.2 Liquefaction Potential Assessment

Liquefaction in the Deeper Soil Layers

Sand lenses within the underlying gravels were encountered in Borehole BH102 (2015) and are inferred to be present based on the current CPT traces. MASW soundings from previous stage testing also indicates sand lenses, where shear wave velocities are between 180m/s and 220m/s. The sand lenses appear to be localised in the northern part of the site. A full discussion is presented in the subdivision consent report for Stages 19 to 24, 224464-0004-REP-GG-0001, Rev0, dated 22 June 2018.

To assess the liquefaction potential of these lenses, we have used the investigation findings from the previous stages, which indicate or possibly imply but not infer a continuation of the sandier lenses. We consider that liquefaction in these deeper lenses does not present a significant geotechnical risk to the proposed shallow founded structures, based on the following:

- When loose non-cohesive soils are saturated and rapid loading occurs under undrained conditions, the soils densification causes pore water pressure to increase. The increase in pore water pressure results in a loss of soil strength due to a decrease in effective stress and eventually liquefaction occurs when the effective stress drops to zero. However, as these sand lenses as surrounded by gravel, drainage is likely to occur, limiting and reducing the build-up of excess pore water pressure, and thus reducing the liquefaction potential of these sand lenses.

- The log of Borehole BH102 indicates 4.5m of medium to very dense gravels overlying the potentially liquefiable sand lenses, while the MASW profiles indicate 6.5m to 7m of medium to very dense gravels overlying the potentially liquefiable sand lenses. This depth of gravel will form a thick non-liquefiable crust, which, based on observations in Christchurch during the CES, is likely to supress liquefaction induced ground damage on shallow founded structures, even if these sand layers were to liquefy.
- No ground damage, including settlement or land cracking, was observed across areas with and without sand lenses, which suggests that either theses layers did not liquefy, or the upper gravel layer has supressed the surface expression of liquefaction in these areas. Noting that the site has been shaken to a significant level well in excess of SLS levels and nearing ULS levels with no observed ground damage.

Based on this assessment we consider that liquefaction effects occurring in these deeper localised sand lenses will have minimal, if any, effect on shallow founded domestic structures and therefore we have not considered it further in our assessment. Instead, we have focussed on liquefaction in the upper soils as the main mechanism that could drive land damage in Stage 25.

Liquefaction in the Upper Soil Layers

Methodology

The ability for the subsoils to resist the effect of ground shaking associated with the design level events has been assessed from the upper subsoil information obtained from the CPTs. The liquefaction assessment was carried out using the methods outlined in MBIE Guidelines (2018) and the results are summarised in Table 6.

Table 6: Liquefaction assessment methodology summary

Test	Liquefaction Assessment(1)	Fines Content	Liquefaction Cut Off	Liquefaction Settlement Method(2)
CPT	Boulanger and Idriss (2014)	Based on a soil Character Index (I_c) with a Co-efficient for Fines Content (C_{fc}) =0	Based on a 2.6 I_c cut off	Zhang et al (2002)

(1) A 15% probability of liquefaction (PL) has been considered with all methods.
(2) We note that there is an inherent uncertainty when identifying liquefiable layers in CPT analysis, due to this inherent uncertainty, calculated settlements will likely differ from actual settlements experienced on site.

The fines content fitting parameter has been set as 0 as no laboratory testing has been undertaken on the soils at the site. Layers within the upper soils were inferred to be clayey silts to organic silts (I_c greater than 2.6). As limited laboratory testing has been carried out to aid in determining a liquefaction cut off on the soils underlying the site, soils have been assumed to be non-liquefiable where the CPT Soil Character Index, I_c, is greater than 2.6.

Liquefaction Effects

Liquefaction can have a number of effects on buildings and land. In this assessment we have considered the following effects:

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

These are discussed in the following sections.

Liquefiable Layers

The layers which may liquefy in a design level event are critical in regard to the foundation performance. The Boulanger and Idriss (2014) method has been used in this assessment and it has been assumed that soils are liquefiable when the factor of safety is below one.

Liquefaction Induced Settlement

The method of Zhang et. al. (2004) was used for calculating the potential liquefaction induced reconsolidation settlements in the CPT analysis. Due to the presence of dense gravel from the CPT refusal depth to at least 10m below ground level, index settlements in the upper 10m of the soil profile have been calculated from the CPT data.

Liquefaction Induced Ground Damage

We have used two methods to assess the potential for liquefaction induced ground damage as presented below:

a) Published information (after Ishihara, 1985) can be used to assess the potential for surface expression of liquefaction and hence the likelihood of inducing damage. Ishihara's method is for a single non-liquefiable layer overlying a single liquefiable layer only. The liquefaction analysis indicates multiple liquefiable layers within the CPT profiles and to account for this we have taken the thickness of the non-liquefied crust as the thickness from the ground surface to the top of the uppermost critical liquefiable layer, and the thickness of the critical liquefied layer as the sum of the thicknesses of all critical liquefiable layers.

   Ishihara's plots do not explicitly indicate ground damage curves for specific PGAs such as 0.13g which is the SLS level PGA. To simplify the analysis, we have used following curves to assess the ground damage:

   - The 0.20g curve when assessing damage under SLS design levels of ground shaking and the lower bound 4 September 2010 Darfield Earthquake.
   - The 0.40g curve when assessing damage under ULS design level of ground shaking and the 4 September 2010 Darfield Earthquake.

b) Tonkin & Taylor (T&T) developed the Liquefaction Severity Number (LSN) (Tonkin & Taylor 2013) based on investigation data and observations made following major earthquake events in Christchurch. The LSN uses the settlements calculated from the Idriss and Boulanger (2008) method with the Robertson and Wride (1998) fines content method and the Zhang et. al. (2004) settlement method to assess the expected ground damage that could be caused by liquefaction in future earthquakes. The corresponding level of ground damage associated with a given LSN number range is summarised in Table 7.

Table 7: LSN Descriptions

LSN Range	Predominate Performance
0-10	Little to no expression of liquefaction, minor effects
10-20	Minor expression of liquefaction, some sand boils
20-30	Moderate expression of liquefaction, with sand boils and some structural damage
30-40	Moderate to severe expression of liquefaction, settlement can cause structural damage
40-50	Major expression of liquefaction, undulations and damage to ground surface, severe total and differential settlement of structures
>50	Severe damage, extensive evidence of liquefaction at surface, severe total and differential settlement affecting structures, damage to services

Upper Liquefaction Results

The results of the liquefaction assessment is presented in Table 8 below

Table 8: Liquefaction Assessment Summary

Earthquake Event	Earthquake Effects	Results
SLS-a (M_w7.5, 0.13g)	Potentially Liquefiable Layers(1)	None anticipated
	Indexed Settlement(2)	<5mm
	Expected Ground Damage	Minor to moderate surface expression of liquefaction
SLS-b (M_w6.0, 0.19g)	Potentially Liquefiable Layers(1)	Minor liquefaction in southern end of the site. None in the northern end.
	Indexed Settlement(2)	0 – 30mm
	Expected Ground Damage	Minor to moderate surface expression of liquefaction
ULS (M_w7.5, 0.35g)	Potentially Liquefiable Layers(1)	Minor liquefaction in southern end of the site. None in the northern end.
	Indexed Settlement(2)	0 – 55mm
	Expected Ground Damage	Minor expression of liquefaction, some sand boils possible at the southern end. None to minor ground damage expected at the northern end of the site.
Notes: 1. Settlements rounded to the nearest 5mm 2. Potential ground damage estimated from LSN, based on Tonkin and Taylor (2013)		

Lateral Spreading

Lateral spreading is a co-seismic effect where surface soils move on a layer, or layers, of liquefied soil downslope or towards a free edge, such as a river or basin. Lateral spreading can occur during an earthquake under seismic loading and following the earthquake until the excess pore water pressure caused by ground shaking dissipate and the soil regains strength.

When assessing the liquefaction induced lateral spreading potential we considered the following:

- There is a small stream which runs south of the site which is approximately 0.5m deep and 2m to 3m wide with gently sloping banks.??
- In the south east corner of the site is a stormwater basin that was installed as part of the overall Rosemerryn Subdivision development, which is in the order of 0.5m deep.
- No other significant rivers or significant changes in height are in close proximity to the site.
- The site is relatively level and we understand that there will be no significant change in the site levels once the development is undertaken.
- We understand that no additional stormwater basins or open channels will be built as part of this development.

Based on the site topography, the depth of the stream and stormwater basin, and the depth to groundwater across the site we consider that the global lateral movement and lateral stretch potentials across the site are minor or less and will not affect the assessment of a MBIE Technical Category Classification

Technical Classification

We have assessed the risk of future liquefaction in terms of the technical category classification system as per the MBIE Guidelines (2018). This classification system is divided into three technical categories that reflect both the liquefaction experience to date and future performance expectations. The categories and corresponding criteria are summarised as follows:

- **Technical Category 1 (TC1)** – Future land damage from liquefaction is unlikely, and ground settlements are expected to be within normally accepted tolerances.
- **Technical Category 2 (TC2)** – Minor to moderate land damage form liquefaction is possible in future large earthquakes.
- **Technical Category 3 (TC3)** – Moderate to significant land damage from liquefaction is possible in future large earthquakes.

MBIE has indicated the following liquefaction and lateral spreading deformation limits for house foundations as summarised in Table 9.

Table 9: Liquefaction deformation limits and house foundation implications

Technical Category	Index Liquefaction Deformation Limits				Likely Implication for House Foundations (subject to individual assessment)
	Vertical		Lateral Spread		
	SLS	ULS	SLS	ULS	
TC1	15mm	25mm	Nil	Nil	Standard NZS3604 type foundations with tied slabs
TC2	50mm	100mm	50mm	100mm	MBIE enhanced foundation solutions
TC3	>50mm	>100mm	>50mm	>100mm	Site specific foundation solution

Discussion

As indicated by Bradley (2012), the site experienced a PGA of 0.44g during the 4 September 2010 Darfield Earthquake event. Based on the MBIE Guidelines (2012) the site has been ‘sufficiently tested’ as the median value for the PGA for the 4 September 2010 earthquake event exceeded 170% of the SLS PGA (I.e. 1.7 x 0.13g = 0.22g).

During the 4 September earthquake event there was no damage on site due to liquefaction. Based on this actual response, we infer that the liquefaction assessment method overestimates likely settlement and damage under future large earthquakes.

Under SLS conditions, the maximum settlement expected is to not exceed 30mm and under ULS conditions not exceed 50mm. Under SLS conditions, there is expected to be none to minor ground damage across both ground models, while under ULS conditions, minor liquefaction could occur in Profile One in the south end of site.

Based on these settlements, the northern (Profile One) end of the site is consistent with MBIE TC1 classification and the southern (Profile Two) end is consistent with TC2.

In summary, based off our liquefaction assessment, and observed ground damage we infer that minor to moderate land damage is possible in future large earthquakes. Areas of TC1 and TC2 classified land are shown in Figure 2 in Appendix A.

4.4.3 Summary of MBIE Technical Category Liquefaction Assessment

The liquefaction analysis indicates the following:

- Based on Bradley (2012) PGA model the site has been “sufficiently tested” (MBIE Guidelines (2012)) as the median value for the PGA for the 4 September 2010 event exceeded 170% of the SLS PGA (i.e. 1.7 x 0.13g = 0.22g). Therefore, we have used the lack of ground damage observed at the site after the 4 September 2010 earthquake event to calibrate our liquefaction assessment.
- The GNS report on liquefaction (GNS, 2012), a review of aerial photography, and site observations made by Aurecon and Fulton Hogan staff confirms there was no evidence of liquefaction observed at the site

after the 4 September 2010 Darfield earthquake, or any subsequent earthquakes in the Canterbury Earthquake Sequence.

- In the northern part of the site liquefaction induced settlements and damage are likely to be minimal and are consistent with a TC1 classification while elsewhere the calculated liquefaction induced settlements and assessed ground damage are consistent with a TC2 classification. However, when compared to actual site performance, the level of calculated damage is overstated, as the back analysis indicates that moderate to major ground damage should have occurred, when only limited to minor damage was observed at and around the site.
- The liquefaction induced lateral spreading potential is considered to be minor.
- Based on our liquefaction assessment and observed ground damage we infer that minor to moderate land damage from liquefaction is possible in future large earthquakes at parts of the site.

Therefore, based on our liquefaction assessment, we consider that the northern part of Stage 25 is consistent with a **Technical Category 1 (TC1)** classification and the remainder of the site is consistent a **Technical Category 2 (TC2)** classification, see Figure 2 in Appendix A for further details.

4.5 Liquefaction Mitigation

4.5.1 General

We consider that parts of the site in its current assessed state are susceptible to varying degrees of seismically induced liquefaction in a future major seismic event. In terms of liquefaction hazard mitigation at this site, and considering the proposed site layout and development, there are two basic approaches available as follows:

Building Strengthening

Structurally design the building to accommodate the effects of liquefaction. Examples of this include using raft or piled foundations. These methods do not remove the liquefaction hazard but reinforce the structure in such a way that it maintains stability during a liquefaction event. This approach is recommended in the TC2 equivalent area.

Ground Improvement

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

1. Densify the soil so that soil grain skeleton will not collapse under earthquake loading. Examples of this include compaction and replacement (refilling with material which will not liquefy).
2. Soil reinforcement. Examples include stone columns, driven piles to densify and stiffen the soil, deep soil mixing, soil cement columns etc.
3. Allow dissipation of excess pore water pressure so that liquefaction is reduced. Examples of this include installation of drains, drainage blankets, and or stone columns.

The recommended approach for liquefaction mitigation in each Technical Category classification zone is discussed below.

4.5.2 Technical Category 1

As per the MBIE (2012) Guidelines with TC1 sites *"Future land damage from liquefaction is unlikely, and ground settlements from liquefaction effects are expected to be within normal accepted tolerances"*. For Technical Category 1 areas the MBIE Guidelines recommend Standard NZS3604:2011 type foundations with tied slabs provided there is suitable bearing.

MBIE Guidelines recommend that a site specific geotechnical assessment be carried out by suitability qualified chartered engineer with experience in residential house development at the detailed house design stage.

4.5.3 Technical Category 2

This section provides generic foundation advice for the wider subdivision development. It ***does not*** constitute a detailed design of house foundations. Additional investigations will be required at the building consent stage for each house to determine the appropriate foundations and to support a building consent application.

It is considered that parts of the site in its current assessed state is consistent with a MBIE TC2 classification. Land with the deformation characteristics of TC2 does not meet the definition of "good ground" as per the New Zealand Standards (NZS3604 *'Timber Framed Buildings'* and NZS4229 '*Concrete Masonry Buildings not requiring Specific Engineering Design'*) without modification to the standard foundation system as described below. The generic foundation types in these standards are not appropriate due to their potential for damage in liquefaction events.

The risk of building damage due to liquefaction in TC2 land can be mitigated by providing strengthened foundations, which reduce the differential settlement of the building and are designed to be readily re-levellable following a major earthquake. There are a range of standard foundation types available for TC2 land which are presented in the MBIE Guidelines and include enhanced raft or rib raft foundations.

Although it is not an explicit consent requirement, we recommend that lightweight cladding and roofing materials are used on all dwellings in TC2 areas, as reducing the dwelling mass will lead to reduced foundation movements and less building damage in future large earthquakes.

As part of the detailed foundation design, particular attention should be paid to detailing the connection joints of buried services (water and sewer pipes, power conduits, etc.) between the house foundation and the in-situ ground. The design should allow sufficient movement and ductility to account for seismic shaking and liquefaction induced movement, and to allow for easy reinstatement if they were to be damaged during a future seismic event.

Other foundation solutions are available (i.e. ground improvement to achieve TC1 site characteristics etc.). However, these options are unlikely to be economic viable to the options below.

It should be noted that this report provides guidance only on residential foundation design and should not be taken as detailed design. MBIE Guidelines require that for detailed house design, a site-specific geotechnical assessment shall be carried out by suitability qualified chartered engineer with experience in residential house development.

5 Assessment Against the RMA

Section 106 of the Resource Management Act (RMA) (2017) states *inter alia*

Consent authority may refuse subdivision consent in certain circumstances

1) A consent authority may refuse to grant a subdivision consent, or may grant a subdivision consent subject to conditions, if it considers that—

- *a) there is a significant risk from natural hazards; or*
- *b) Repealed*
- *c) sufficient provision has not been made for legal and physical access to each allotment to be created by the subdivision.*

1A) For the purpose of subsection (1) (a), an assessment of the risk from natural hazards requires a combined assessment of—

- *a) the likelihood of natural hazards occurring (whether individually or in combination); and*
- *b) the material damage to land in respect of which the consent is sought, other land, or structures that would result from natural hazards; and*
- *c) any likely subsequent use of the land in respect of which the consent is sought that would accelerate, worsen, or result in material damage of the kind referred to in paragraph (b).*

2) Conditions under subsection (1) must be—

- *a) for the purposes of avoiding, remedying, or mitigating the effects referred to in subsection (1); and*
- b) *of a type that could be imposed under section 108.*

A risk assessment approach has been undertaken on the significant geotechnical hazards that may affect the site, which is presented in Appendix E.

Based on this assessment we consider that at the site there are no significant geotechnical hazards other than the potential for earthquake induced soil liquefaction of varying degrees. However, provided that the geotechnical recommendations provided within this report are followed, and the appropriate engineering measures are implemented, then we consider that the development is unlikely to be significant affected by geotechnical hazards nor will the development worsen, accelerate or result in material damage. Therefore, from a geotechnical perspective we consider that Stage 25 of the Rosemerryn residential subdivision development can proceed.

6 References

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A

Figures

Coordinate System: New Zealand Transverse Mercator 2000 Date: 14/03/2022

Path: C:\Users\Marcus.Dooney\Documents\ArcGIS\Projects\Rosemerryn Subdivision\Rosemerryn Subdivision.aprx

aurecon
www.aurecongroup.com

CLIENT: Fulton Hogan LAND DEVELOPMENT LTD.

REV	DATE	REVISION DETAILS	APPROVED
A	08/03/22	PRELIMINARY	JK

SCALE	SIZE
1:4,000	A3

DRAWN: M DOONEY
REVIEWED: A HILLS
VERIFIED: D MAHONEY

PRELIMINARY
NOT FOR CONSTRUCTION

APPROVED: J KUPEC
DATE 14.03.22

PROJECT: ROSEMERRYN SUBDIVISION EXTENSION

TITLE: FIGURE 1 - GEOTECHNICAL INVESIGATION PLAN

DOCUMENT	PROJECT	WBS	TYPE	DISC	NUMBER	SHEET	REVISION
	520194	0000	DRG	GG	0001	01	A

Date: 14/03/2022
Coordinate System: New Zealand Transverse Mercator 2000
Path: C:\Users\Marcus.Dooney\Documents\ArcGIS\Projects\Rosemerryn Subdivision\Rosemerryn Subdivision.aprx

aurecon
www.aurecongroup.com

CLIENT: Fulton Hogan LAND DEVELOPMENT LTD.

REV	DATE	REVISION DETAILS	APPROVED
A	09/03/22	PRELIMINARY	JK

SCALE: 1:1,750
SIZE: A3
DRAWN: M DOONEY
REVIEWED: A HILLS
VERIFIED: D MAHONEY

PRELIMINARY
NOT FOR CONSTRUCTION

APPROVED: J KUPEC
DATE: 14.03.22

PROJECT: ROSEMERRYN SUBDIVISION EXTENSION

TITLE: FIGURE 2 - GROUND PROFILE SECTION AND TECHNICAL CATEGORY CLASSIFICATION

DOCUMENT	PROJECT	WBS	TYPE	DISC	NUMBER	SHEET	REVISION
	520194	0000	DRG	GG	0001	02	A

B

ECan Logs

Borelog for well M36/3324

Environment Canterbury Regional Council
Kaunihera Taiao ki Waitaha

Grid Reference (NZTM): 1560404 mE, 5167892 mN
Location Accuracy: 50 - 300m
Ground Level Altitude: 8.7 m +MSD Accuracy: < 2.5 m
Driller: McMillan Drilling Ltd
Drill Method: Unknown
Borelog Depth: 42.0 m Drill Date: 17-Dec-1985

Depth(m)	Full Drillers Description	Formation Code
0.50m	Earth	SP
5.00m	Blue gravels	SP
9.00m	Blue pug	SP
14.00m	Free gravels	RI
20.00m	Pug and wood	RI
25.50m	Free Brown stained gravels	RI
30.00m	Claybound gravel	BR
36.00m	Very hard claybound gravel	BR
42.00m	Free Brown stained gravel	LI-1

Scale(m): 5, 10, 15, 20, 25, 30, 35, 40

Borelog for well M36/7299

Environment Canterbury Regional Council
Kaunihera Taiao ki Waitaha

Grid Reference (NZTM): 1560504 mE, 5168062 mN
Location Accuracy: 50 - 300m
Ground Level Altitude: 8.7 m +MSD Accuracy: < 2.5 m
Driller: Smiths Welldrilling
Drill Method: Rotary Rig
Borelog Depth: 18.0 m Drill Date: 20-Nov-2002

Depth(m)	Full Drillers Description	Formation Code
0.20m	Soil	SP
4.50m	Clay	SP
5.00m	Sand	SP
6.00m	Blue sandy gravels	SP
7.00m	Sandy gravels	SP
8.00m	Orange pug	SP
11.00m	Free sandy gravels	RI
16.00m	Claybound gravels	RI
18.00m	Free sandy gravels	RI

Scale(m): 5, 10, 15

Borelog for well M36/8675

Grid Reference (NZTM): 1560265 mE, 5167627 mN
Location Accuracy: 2 - 15m
Ground Level Altitude: 8.0 m +MSD Accuracy: < 2.5 m
Driller: Not Known
Drill Method: Rotary/Percussion
Borelog Depth: 5.8 m Drill Date: 09-Oct-2008

Environment Canterbury Regional Council
Kaunihera Taiao ki Waitaha

Depth(m)	Full Drillers Description	Formation Code
0.20m	Dark grey loamy topsoil	
1.20m	Soft grey intermixed with brown silty clay. Speckled with patches of orange and some timber	
3.20m	Soft blue grey silty clay with pieces of timber	
3.60m	Grey brown sand mixed with soft grey brown silty clay	
5.80m	Grey silty sandy gravel	

C

Recent Investigations

CONE PENETRATION TEST (CPT) REPORT

McMILLAN Drilling

Client: Aurecon NZ Ltd

Location: Rosemerryn Subdivision
642 Ellesmere Road, Lincoln

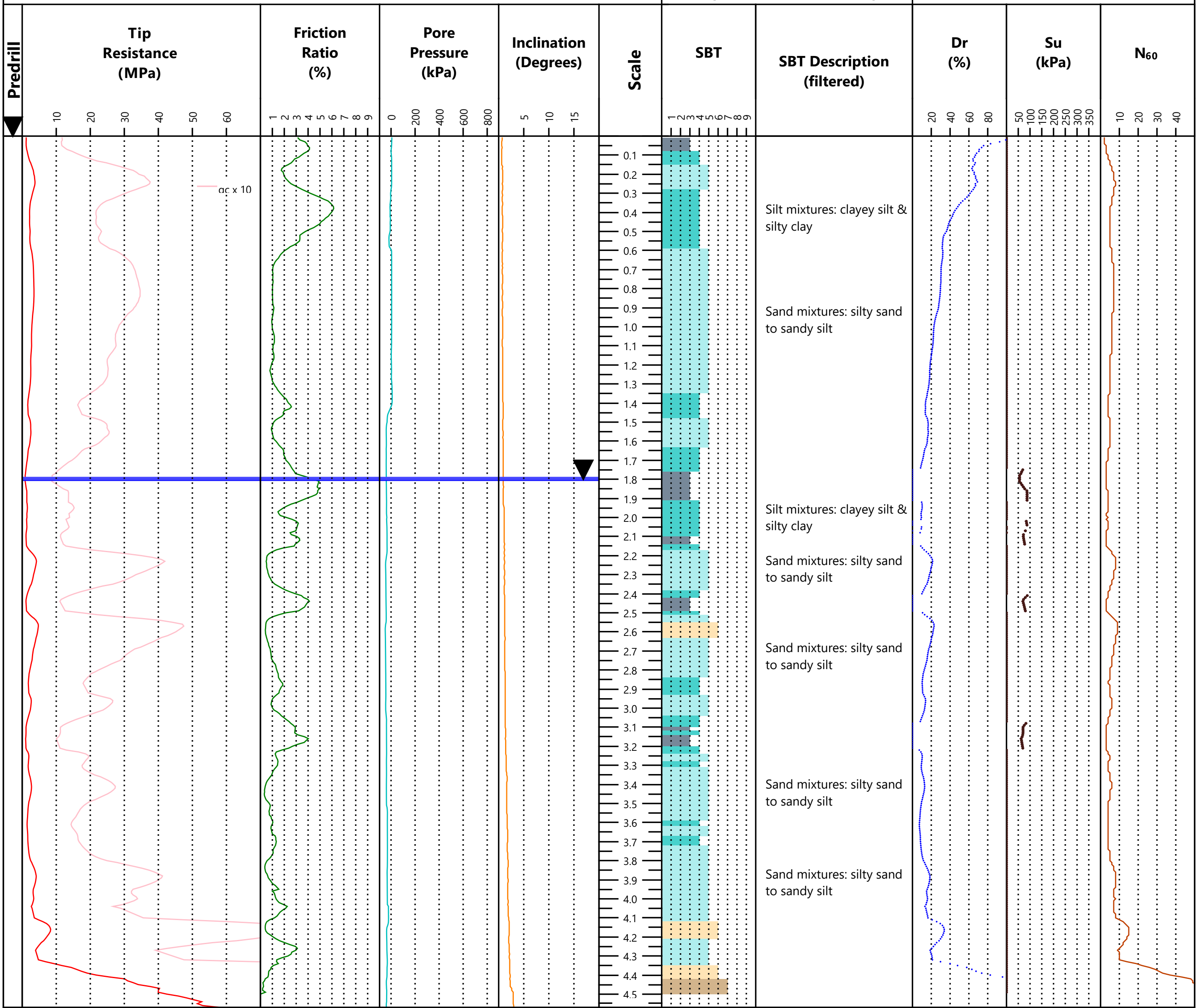
Printed: 15/02/2022

McMILLAN Drilling	**Client:** Aurecon NZ Ltd	**Bore No.:** CPTu001
	Project: Rosemerryn Subdivision	**Job No.:** 20686

Site Location: 642 Ellesmere Road, Lincoln **Date:** 11/2/2022

Grid Reference: 1560389.3m E, 5167821.88m N (NZTM) - Map or aerial photograph **Rig Operator:** B. Wilson

Elevation: 0.00m **Datum:** Ground **Equipment:** Geomil Panther 100



EOH: 4.57m

Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 100992
Cone Area Ratio: 0.75
Standards: ISO 22476-1:2012

Predrill: -
Water Level: 1.80m
Collapse: 3.6m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.5479	0.6689
Local Friction	0.0154	0.0088
Pore Pressure	-0.0019	0.0006

Termination

Target Depth	☐
Effective Refusal	
Tip	☑
Gauge	☐
Inclinometer	☐
Other	☐

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

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

Notes & Limitations
Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P. K. Robertson and K.L. Cabal (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 4th Edition. The interpretations are presented only as a guide for geotechnical use, and should be carefully reviewed by the user. No warranty is provided as to the correctness or the applicability of any of the geotechnical soil and design parameters shown and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used to derive data shown in this report.

Remarks

McMILLAN Drilling

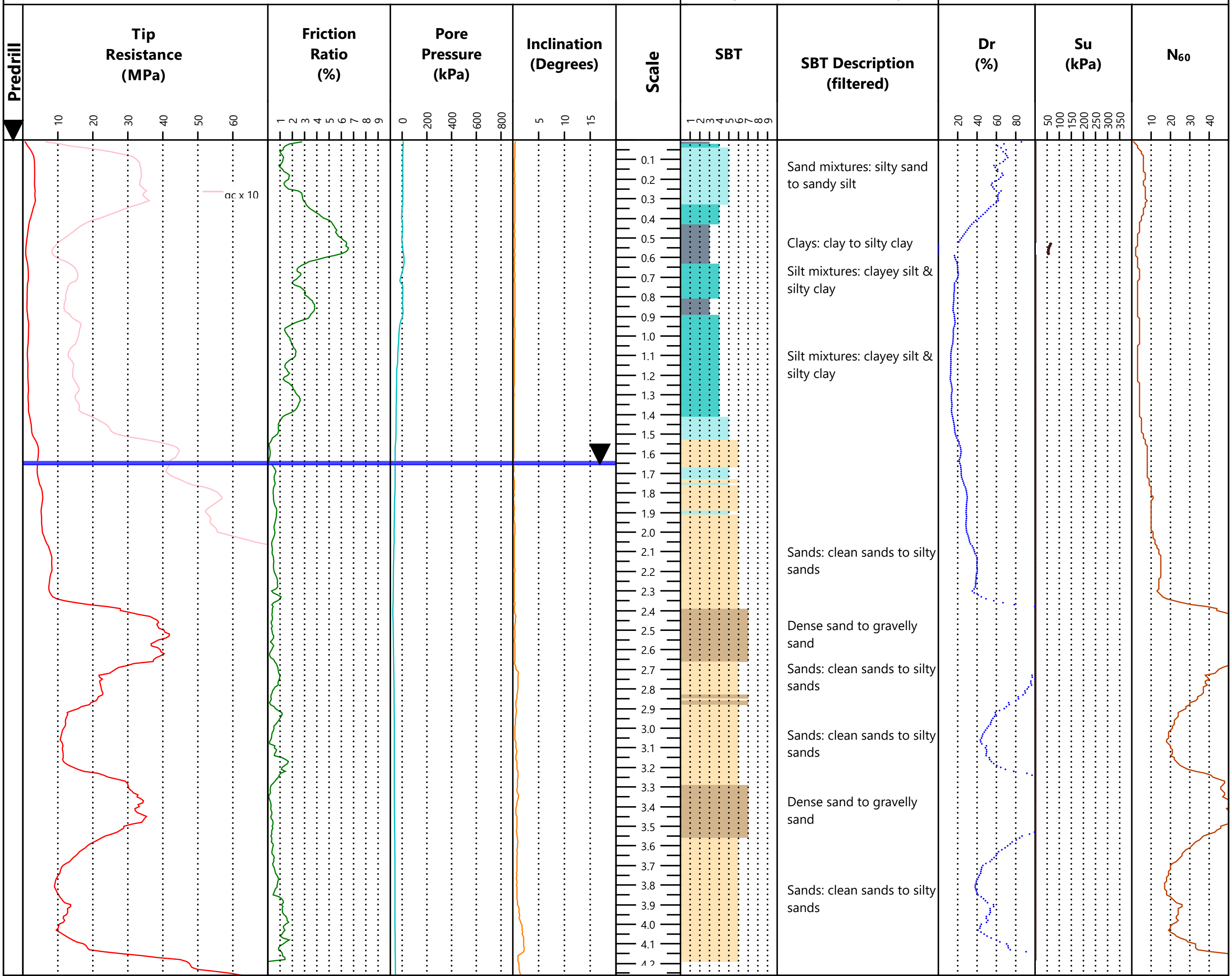
Client: Aurecon NZ Ltd	**Bore No.:** CPTu002
Project: Rosemerryn Subdivision	**Job No.:** 20686

Site Location: 642 Ellesmere Road, Lincoln **Date:** 11/2/2022

Grid Reference: 1560447.09m E, 5167886.33m N (NZTM) - Map or aerial photograph **Rig Operator:** B. Wilson

Elevation: 0.00m **Datum:** Ground **Equipment:** Geomil Panther 100

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



EOH: 4.26m

Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 151125
Cone Area Ratio: 0.75
Standards: ISO 22476-1:2012

Predrill: -
Water Level: 1.65m
Collapse: 3.0m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	-0.1163	-0.0209
Local Friction	0.0038	0.0036
Pore Pressure	-0.0011	-0.0002

Termination

Target Depth ☐

Effective Refusal

- Tip ☑
- Gauge ☐
- Inclinometer ☐
- Other ☐

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

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

Notes & Limitations

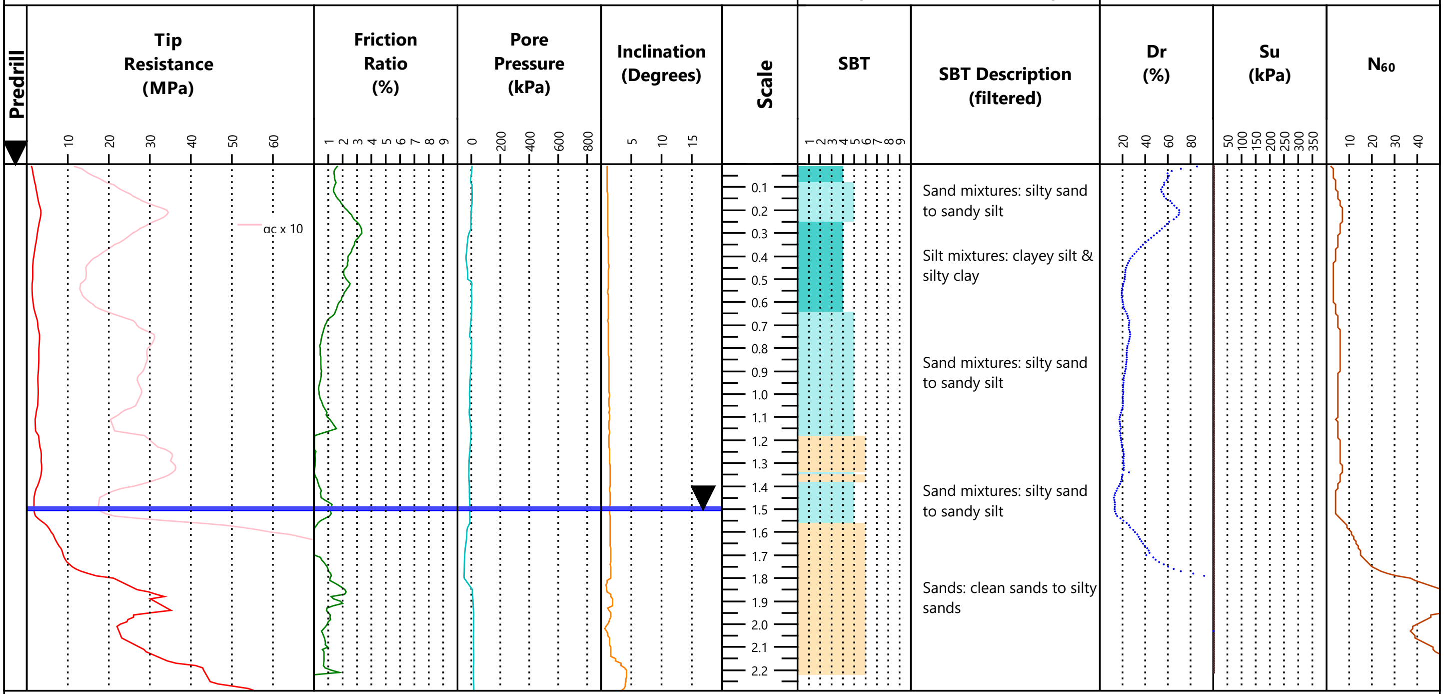
Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P. K. Robertson and K.L. Cabal (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 4th Edition. The interpretations are presented only as a guide for geotechnical use, and should be carefully reviewed by the user. No warranty is provided as to the correctness or the applicability of any of the geotechnical soil and design parameters shown and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used to derive data shown in this report.

Remarks

McMILLAN Drilling	Client: Aurecon NZ Ltd	Bore No.: CPTu003
	Project: Rosemerryn Subdivision	Job No.: 20686

Site Location: 642 Ellesmere Road, Lincoln
Date: 11/2/2022
Grid Reference: 1560445.54m E, 5167975.32m N (NZTM) - Map or aerial photograph
Rig Operator: B. Wilson
Elevation: 0.00m **Datum:** Ground
Equipment: Geomil Panther 100

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



EOH: 2.29m

Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 100992
Cone Area Ratio: 0.75
Standards: ISO 22476-1:2012

Predrill: -
Water Level: 1.50m
Collapse: 1.95m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.5135	0.5879
Local Friction	0.0087	0.0084
Pore Pressure	0.0121	0.0125

Termination

Target Depth	☐
Effective Refusal	
Tip	☑
Gauge	☐
Inclinometer	☐
Other	☐

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

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

Notes & Limitations
Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P. K. Robertson and K.L. Cabal (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 4th Edition. The interpretations are presented only as a guide for geotechnical use, and should be carefully reviewed by the user. No warranty is provided as to the correctness or the applicability of any of the geotechnical soil and design parameters shown and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used to derive data shown in this report.

Remarks

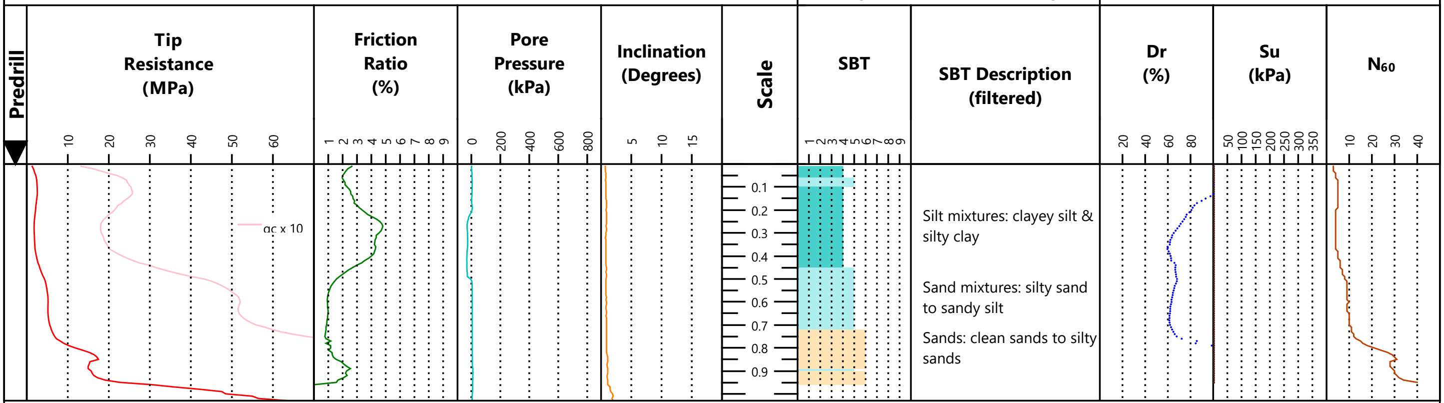
McMILLAN Drilling	**Client:** Aurecon NZ Ltd	**Bore No.:** CPTu004
	Project: Rosemerryn Subdivision	**Job No.:** 20686

Site Location: 642 Ellesmere Road, Lincoln **Date:** 11/2/2022

Grid Reference: 1560529.15m E, 5168014.35m N (NZTM) - Map or aerial photograph **Rig Operator:** B. Wilson

Elevation: 0.00m **Datum:** Ground **Equipment:** Geomil Panther 100

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



EOH: 1.03m

Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 151125
Cone Area Ratio: 0.75
Standards: ISO 22476-1:2012

Predrill: -
Water Level: -
Collapse: 1.0m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	-0.1001	-0.1242
Local Friction	0.0055	0.0032
Pore Pressure	-0.0014	-0.0009

Termination

Target Depth	☐
Effective Refusal	
Tip	☑
Gauge	☐
Inclinometer	☐
Other	☐

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

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

Notes & Limitations

Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P. K. Robertson and K.L. Cabal (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 4th Edition. The interpretations are presented only as a guide for geotechnical use, and should be carefully reviewed by the user. No warranty is provided as to the correctness or the applicability of any of the geotechnical soil and design parameters shown and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used to derive data shown in this report.

Remarks

TEST DETAIL

PointID: CPTu001
Sounding: 1

Operator: B. Wilson
Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 100992
Cone Area Ratio: 0.75

Date: 11/2/2022
Predrill: 0.00m
Water Level: 1.80m
Collapse: 3.6m

Termination

Target Depth ☐

Effective Refusal
Tip ☑
Gauge ☐
Inclinometer ☐
Other ☐

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.5479	0.6689
Local Friction	0.0154	0.0088
Pore Pressure	-0.0019	0.0006

PointID: CPTu002
Sounding: 2

Operator: B. Wilson
Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 151125
Cone Area Ratio: 0.75

Date: 11/2/2022
Predrill: 0.00m
Water Level: 1.65m
Collapse: 3.0m

Termination

Target Depth ☐

Effective Refusal
Tip ☑
Gauge ☐
Inclinometer ☐
Other ☐

Zero load outputs (MPa)	Before test	After test
Tip Resistance	-0.1163	-0.0209
Local Friction	0.0038	0.0036
Pore Pressure	-0.0011	-0.0002

PointID: CPTu003
Sounding: 3

Operator: B. Wilson
Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 100992
Cone Area Ratio: 0.75

Date: 11/2/2022
Predrill: 0.00m
Water Level: 1.50m
Collapse: 1.95m

Termination

Target Depth ☐

Effective Refusal
Tip ☑
Gauge ☐
Inclinometer ☐
Other ☐

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.5135	0.5879
Local Friction	0.0087	0.0084
Pore Pressure	0.0121	0.0125

PointID: CPTu004
Sounding: 4

Operator: B. Wilson
Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 151125
Cone Area Ratio: 0.75

Date: 11/2/2022
Predrill: 0.00m
Water Level: -
Collapse: 1.0m

Termination

Target Depth ☐

Effective Refusal
Tip ☑
Gauge ☐
Inclinometer ☐
Other ☐

Zero load outputs (MPa)	Before test	After test
Tip Resistance	-0.1001	-0.1242
Local Friction	0.0055	0.0032
Pore Pressure	-0.0014	-0.0009

CPT CALIBRATION AND TECHNICAL NOTES

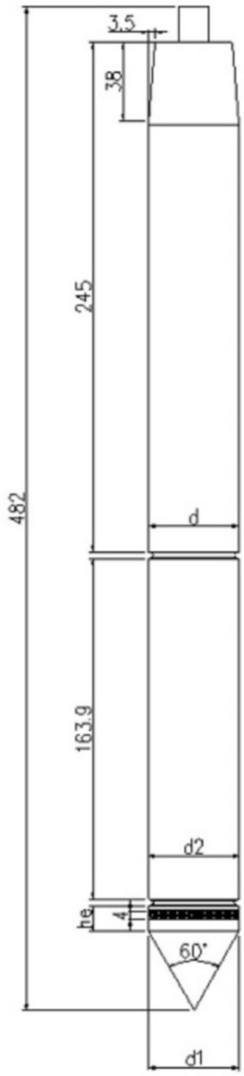
These notes describe the technical specifications and associated calibration references pertaining to the following cone types:

- I-CFXY-10 measuring cone resistance, sleeve friction and inclination (standard cone, 10cm²);
- I-CFXY-15 measuring cone resistance, sleeve friction and inclination (standard cone, 15cm²);
- I-CFXYP20-10 measuring cone resistance, sleeve friction, inclination and pore pressure (piezocone, 10cm²);
- I-CFXYP100-10 measuring cone resistance, sleeve friction, inclination and high range pore pressure (piezocone, 10cm²);
- I-C2xFXYP100-10 measuring cone resistance, high range sleeve friction, inclination and high range pore pressure (piezocone, 10cm²);
- I-C5F0p15XYP20-10 measuring sensitive cone resistance, sleeve friction, inclination and pore pressure (piezocone, 10cm²).
- I-CFXYP20-15 measuring cone resistance, sleeve friction, inclination and pore pressure (piezocone, 15cm²);

Dimensions

Dimensional specifications for all cone types are detailed below. All tolerances are routinely checked prior to testing and measurements taken are electronically recorded. All records are kept on file and available on request.

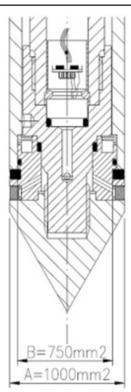
A.P. van den Berg Machinefabriek tel.: +31 (0)513-631355 info@apvandenberg.com	**DEVIATION of Straightness + MINIMUM Dimensions tip, friction jacket, cone adapter**	Standards: EN ISO 22476-1 APB-standard
Type of cone:	Icone 10 cm²	Icone 15 cm²
ALLOWABLE SIZE VARIATION		
Diameter of tip:	$35{,}3 \le d1 \le 36{,}0$	$43{,}2 \le d_1 \le 44{,}1$
Diameter of centering ring CFP	$35{,}3 \le d1 \le 36{,}0$	$43{,}2 \le d_1 \le 44{,}1$
Diameter of friction jacket:	$d_1 \le d_2 < d_1 + 0{,}35$	$d_1 \le d_2 < d_1 + 0{,}43$
Height dimension of tip edge:	$7 \le h_e \le 10$	$9 \le h_e \le 12$
PRODUCTION DIMENSIONS		
Tip:	$d_1 = 35{,}7^{+0,2}_{0}$	$d_1 = 43{,}8^{+0,2}_{0}$
Jacket (C-cone):	$d_2 = 35{,}7^{+0,2}_{0}$	$d_2 = 43{,}7^{+0,2}_{0}$
Friction jacket (CF-cone):	$d_2 = 35{,}9^{+0,1}_{0}$	$d_2 = 44{,}0^{+0,1}_{0}$
Tip for used cone:	$d_1 = 35{,}5^{+0,1}_{0}$	$d_1 = 43{,}5^{+0,1}_{0}$
MINIMUM DIMENSIONS		
Minimum diameter jacket (C-cone):	$d_2 = 35{,}2$ (APB standard)	$d_2 = 43{,}0$ (APB standard)
Minimum diameter friction jacket (CF-cone):	$d_2 = 35{,}3$	$d_2 = 43{,}2$
Use "used cone"-tip when friction jacket diameter:	$d_2 \le 35{,}65$	$d_2 \le 43{,}7$
Minimum diameter of cone adaptor:	$d = 35{,}3$	$d = 43{,}8$
Maximum deviation of straightness:	1 mm on a length of 1000 mm (max. oscillation 1,0 mm.)	1 mm on a length of 1000 mm (max. oscillation: 2.0 mm)



Tip and Local Friction sensor displacement

The different distances of the sensors are compensated depending on the cone types:

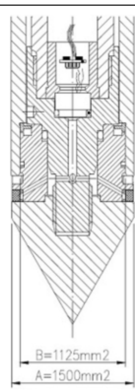
- 10cm² cones: 80mm
- 15cm² cones: 100mm



Cone area ratio

α = B / A = 0.75

β = 1 - B / A = 0.25



McMILLAN Drilling

CPT CALIBRATION AND TECHNICAL NOTES

Calibration

Each cone has a unique identification number that is electronically recorded and reported for each CPT test. The identification number enables the operator to compare ‘zero-load offsets’ to manufacturer calibrated zero-load offsets.

The recommended maximum zero-load offset for each sensor is determined as ± 5% of the nominal measuring range.

In addition to maximum zero-load offsets, the difference in zero load offset before and after the test is limited as ± 2% of the maximum measuring range. See table below:

	Tip (MPa)		**Friction (MPa)**			**Pore Pressure (MPa)**	
Maximum Measuring Range:	150	15 *	1.50	0.3 *	3 **	3	15 ***
Nominal Measuring Range:	75	7.5 *	1.00	0.15 *	1 **	2	10 ***
Max. ‘zero-load offset’:	7.5	0.75 *	0.10	0.015 *	0.1 **	0.2	1 ***
Max ‘before and after test’:	3	0.3 *	0.03	0.006 *	0.06 **	0.06	0.3 ***

***** I-C5F0p15XYP20-10 ("sensitive")
** I-C2xFXYP100-10 (high range friction and pore water pressure sensors)
******* I-CFXYP100-10 (high range pore water pressure sensor)

Note: The zero offsets are electronically recorded and reported for each test in the same units as that of each sensor.

Calibration Certificate

CALIBRATION RvA K 178

a.p. van den berg

1.1 General

Probe number:	100992
Probe type:	I-CFXYP20-10
Description:	Tip 75 MPa Sleeve 1.00 MPa Inclinometer 20° Pore 2MPa
Part number:	0100277B
Certificate number:	100992-2
Manufacturer:	A.P. van den Berg, Heerenveen (NL)
Calibration lab.:	A.P. van den Berg Ingenieursburo, IJzerweg 4, 8445 PK, Heerenveen (NL) RvA accredited laboratory according to ISO/IEC 17025:2017
Location of calibration:	Heerenveen (NL)
Client:	McMillan Drilling Ltd 120 High Street SOUTHBRIDGE, CANTERBURY New Zealand

1.2 Calibration equipment

Reference measuring equipment:

DAQ MX238B 0177FD	March 2021 (HBM: 92591)
DAQ MX440B 0182F3	March 2021 (HBM: 92778)
Loadcell 100kN H54435	August 2020 (HBM: 86959 2020-07)
Loadcell 20kN D16200	July 2020 (HBM: 86871 2020-07)
Sensor 20 Bar 240310140	Sept 2020 (ZMK: 02-1194 2020-09)
ACS-080-SC00-HE2-PM 12/17 2321909	April 2021 (Trescal: 2103-24007)
Temperature logger: 620-2326 SN:170800101	March 2021 (AVANTOR 219001540)

1.3 Laboratory conditions:

Ambient temperature: 23.8 ± 2 °C

1.4 Measurement uncertainty

The expanded combined uncertainty (k=2) of the sensor at laboratory conditions was analysed according to ISO/IEC Guide 98-3:2008 and is based on the standard uncertainty of the measurement multiplied by a coverage factor k, such that the coverage probability corresponds to approximately 95%. The results of the measurement uncertainty analysis of the different parameters are as listed below:

Cone resistance	5,6 + 0,165%	(kPa)
Sleeve friction	0,17 + 0,105%	(kPa)
Pore Pressure 2 MPa sensor	4,16 + 0,037%	(kPa)
Inclination	0,42	(degrees)

1.5 Standard and method of calibration

EN ISO 22476-1 2012 Class 2

1.6 Results

The probe complies with the requirements of the above-mentioned standard and indicated calibration class. The calibrated sensors comply if the measured deviations over the nominal measuring range are within the accuracy limits of the standard (decision rule). The deviations and standard limits are shown in graphs in the Calibration Report.

Calibrated by:	D.Bisschops
Calibration Date:	23 November 2021
Signature:	
QA Manager:	N.R.E. de Jong
Date:	23 November 2021
Signature:	
Expiration date according to EN ISO 22476-1:	24 May 2022

1.7 Remarks

The calibration results only relate to the probe identified in this certificate. This new calibration certificate replaces all previously issued certificates for this probe. The calibration certificate documents the traceability to national and international standards, which realize the units of measurement according to the International System of Units (SI). This calibration certificate may not be reproduced other than in full and except with permission of the issuing laboratory. Calibration certificates without signature are not valid.

Calibration Certificate CALIBRATION RvA K 176 a.p. van den berg

1.1 General

Probe number:	151125
Probe type:	I-CFXYP20-10
Description:	Tip 75 MPa Sleeve 1.00 MPa Inclinometer 20° Pore 2MPa
Part number:	0100277B
Certificate number:	151125-3
Manufacturer:	A.P. van den Berg, Heerenveen (NL)
Calibration lab.:	A.P. van den Berg Ingenieursburo, IJzerweg 4, 8445 PK, Heerenveen (NL) RvA accredited laboratory according to ISO/IEC 17025:2017
Location of calibration:	Heerenveen (NL)
Client:	McMillan Drilling Ltd 120 High Street SOUTHBRIDGE, CANTERBURY New Zealand

1.2 Calibration equipment

Reference measuring equipment:

DAQ MX238B 0177FD	March 2021 (HBM: 92591)
DAQ MX440B 0182F3	March 2021 (HBM: 92778)
Loadcell 100kN H54435	August 2020 (HBM: 86959 2020-07)
Loadcell 20kN D16200	July 2020 (HBM: 86871 2020-07)
Sensor 20 Bar 240310140	Sept 2020 (ZMK: 02-1194 2020-09)
ACS-080-SC00-HE2-PM 12/17 2321909	April 2021 (Trescal: 2103-24007)
Temperature logger: 620-2326 SN:170800101	March 2021 (AVANTOR 219001540)

1.3 Laboratory conditions:

Ambient temperature: 23.0 ± 2 °C

1.4 Measurement uncertainty

The expanded combined uncertainty (k=2) of the sensor at laboratory conditions was analysed according to ISO/IEC Guide 98-3:2008 and is based on the standard uncertainty of the measurement multiplied by a coverage factor k, such that the coverage probability corresponds to approximately 95%. The results of the measurement uncertainty analysis of the different parameters are as listed below:

Cone resistance	5,6 + 0,165%	(kPa)
Sleeve friction	0,17 + 0,105%	(kPa)
Pore Pressure 2 MPa sensor	4,16 + 0,037%	(kPa)
Inclination	0,42	(degrees)

1.5 Standard and method of calibration

EN ISO 22476-1 2012 Class 2

1.6 Results

The probe complies with the requirements of the above-mentioned standard and indicated calibration class. The calibrated sensors comply if the measured deviations over the nominal measuring range are within the accuracy limits of the standard (decision rule). The deviations and standard limits are shown in graphs in the Calibration Report.

Calibrated by:	D.Bisschops
Calibration Date:	24 November 2021
Signature:	
QA Manager:	N.R.E. de Jong
Date:	24 November 2021
Signature:	
Expiration date according to EN ISO 22476-1:	25 May 2022

1.7 Remarks

The calibration results only relate to the probe identified in this certificate. This new calibration certificate replaces all previously issued certificates for this probe. The calibration certificate documents the traceability to national and international standards, which realize the units of measurement according to the International System of Units (SI). This calibration certificate may not be reproduced other than in full and except with permission of the issuing laboratory. Calibration certificates without signature are not valid.

Certificate version 1.20 Certificate number: 151125-3 Page 1/6

D

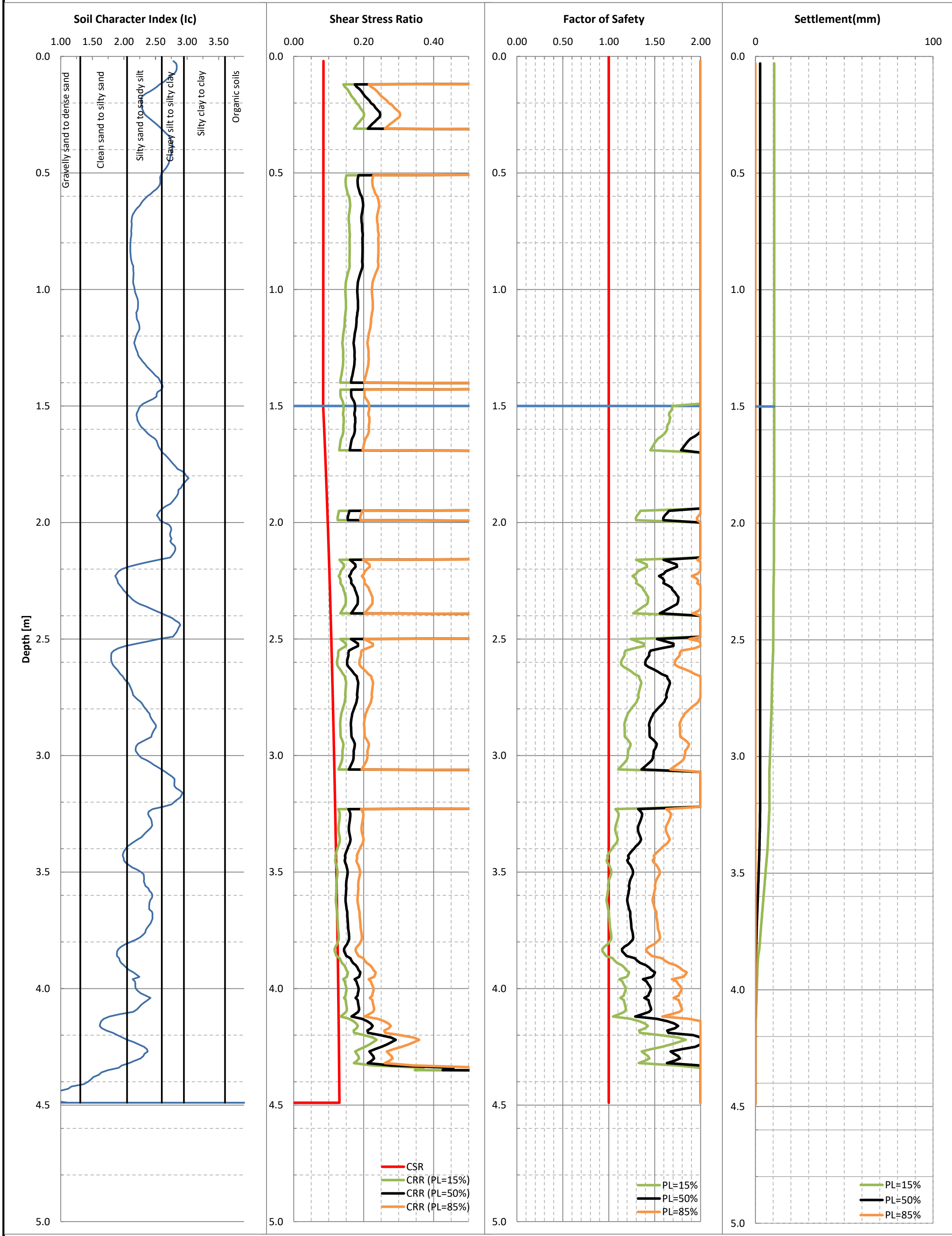
Liquefaction Assessment

LIQUEFACTION ANALYSIS

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

I_c cut off: 2.60
C_{FC}: 0.00

Water Table [m]	**1.50**
Magnitude	**7.50**
Acceleration [g]	**0.13**



Indexed' Settlement (PL=15%) [mm]: N/A
Indexed' Settlement (PL=50%) [mm]: N/A
Indexed' Settlement (PL=85%) [mm]: N/A
Indexed' LSN: N/A

Total Settlement (PL=15%) [mm]: 11
Total Settlement (PL=50%)[mm]: 3
Total Settlement (PL=85%)[mm]: 0
LSN: 3

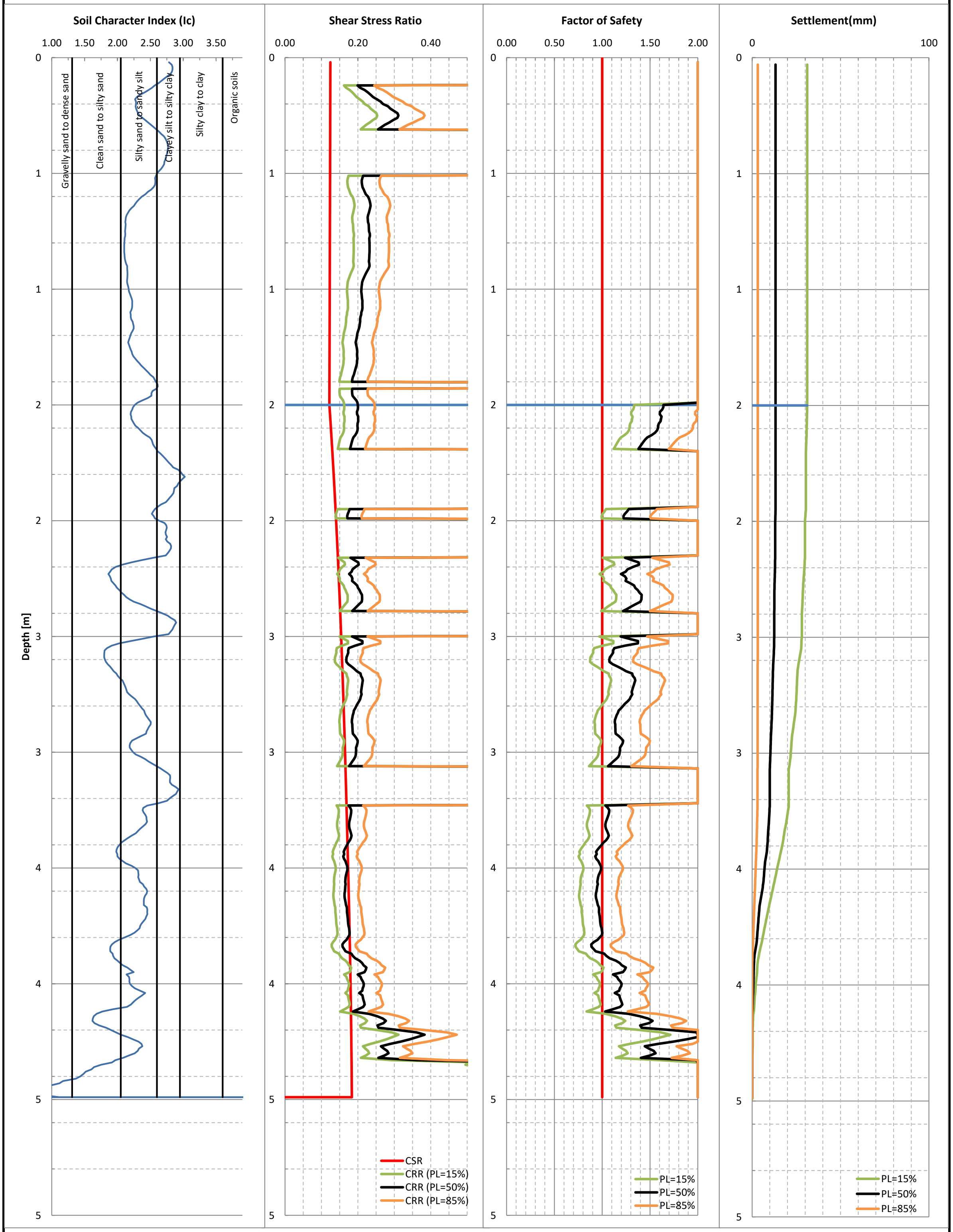
aurecon

Client	Location
Fulton Hogan Land Developments	**Rosemerryn Subdivision**
Project No. **520194**	Test No. **CPT 1**
Design Event **SLSa**	Date **11 February 2022**

LIQUEFACTION ANALYSIS

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

I_c cut off:	2.60	Water Table [m]	1.50
C_{FC}:	0.00	Magnitude	6.00
		Acceleration [g]	0.19



Indexed' Settlement (PL=15%) [mm]:	N/A	Total Settlement (PL=15%) [mm]:	31
Indexed' Settlement (PL=50%) [mm]:	N/A	Total Settlement (PL=50%)[mm]:	13
Indexed' Settlement (PL=85%) [mm]:	N/A	Total Settlement (PL=85%)[mm]:	3
Indexed' LSN:	N/A	LSN:	10

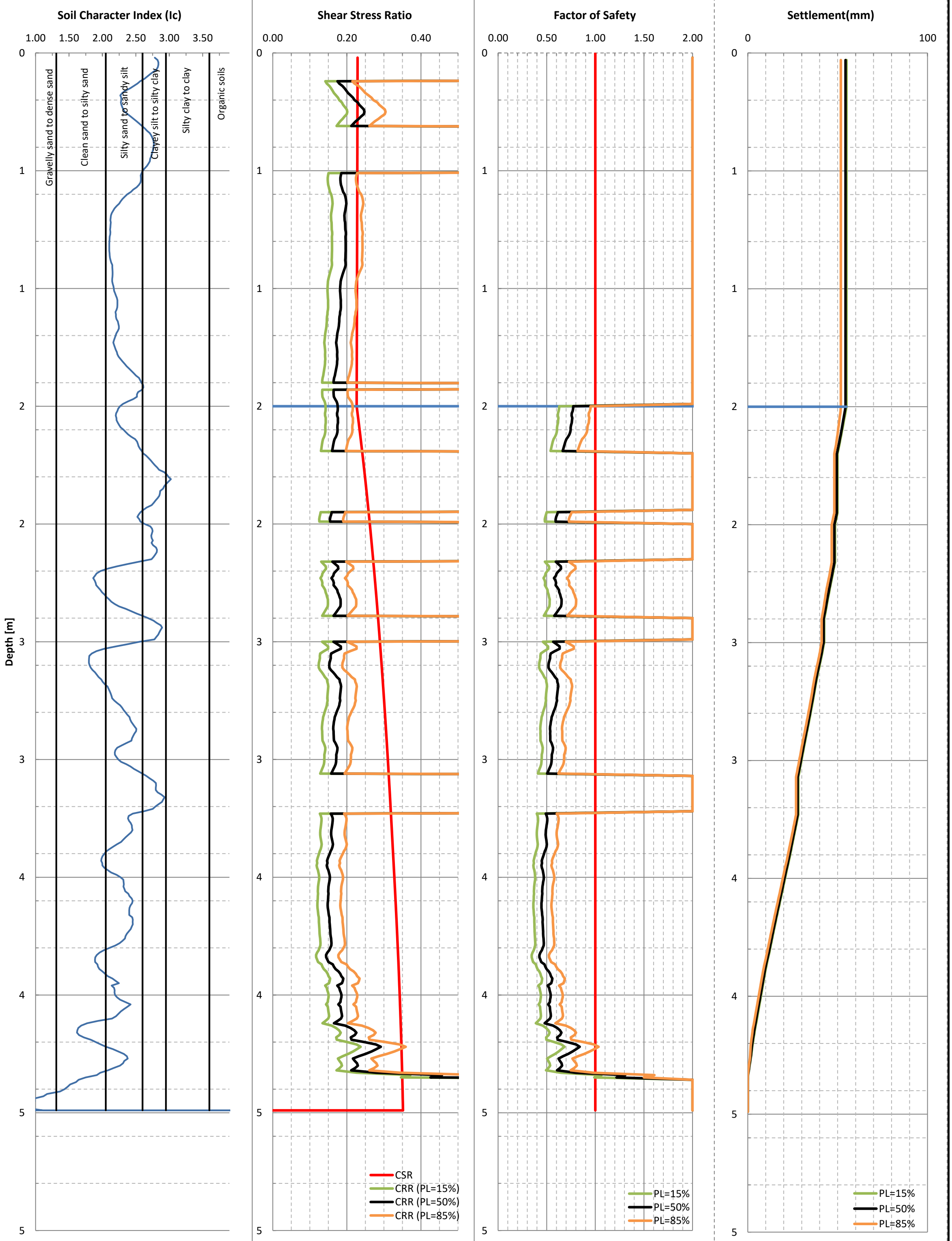
aurecon

Client: **Fulton Hogan Land Developments**	Location: **Rosemerryn Subdivision**
Project No.: **520194**	Test No.: **CPT 1**
Design Event: **SLSb**	Date: **11 February 2022**

LIQUEFACTION ANALYSIS

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

I_c cut off:	**2.60**
C_{FC}:	**0.00**
Water Table [m]	**1.50**
Magnitude	**7.50**
Acceleration [g]	**0.35**



Indexed' Settlement (PL=15%) [mm]:	N/A
Indexed' Settlement (PL=50%) [mm]:	N/A
Indexed' Settlement (PL=85%) [mm]:	N/A
Indexed' LSN:	N/A
Total Settlement (PL=15%) [mm]:	**55**
Total Settlement (PL=50%)[mm]:	**54**
Total Settlement (PL=85%)[mm]:	**52**
LSN:	**19**

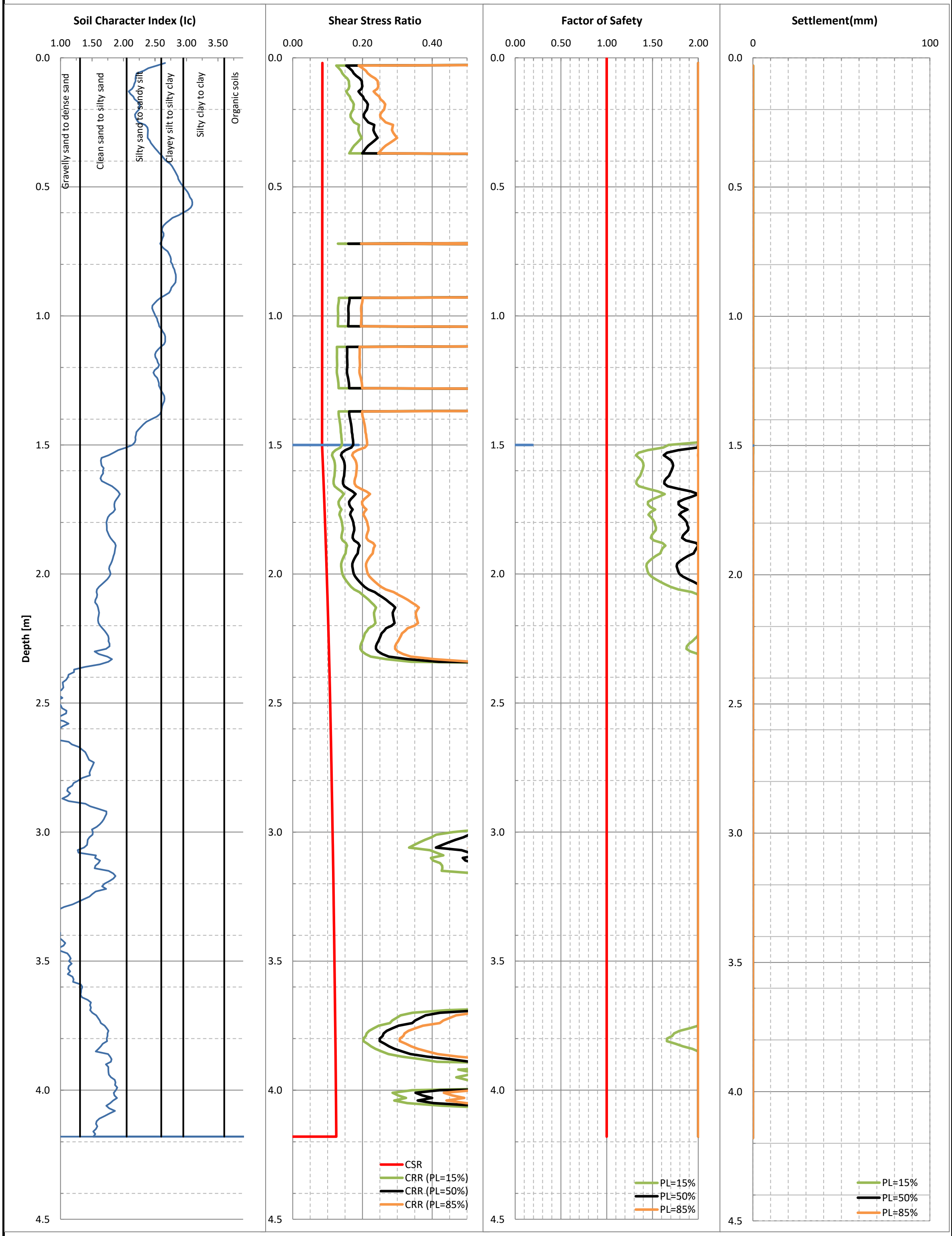
aurecon

Client **Fulton Hogan Land Developments**	Location **Rosemerryn Subdivision**
Project No. **520194**	Test No. **CPT 1**
Design Event **ULS**	Date **11 February 2022**

LIQUEFACTION ANALYSIS

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

I_c cut off:	2.60
C_{FC}:	0.00
Water Table [m]	1.50
Magnitude	7.50
Acceleration [g]	0.13



Indexed' Settlement (PL=15%) [mm]:	N/A	Total Settlement (PL=15%) [mm]:	0
Indexed' Settlement (PL=50%) [mm]:	N/A	Total Settlement (PL=50%)[mm]:	0
Indexed' Settlement (PL=85%) [mm]:	N/A	Total Settlement (PL=85%)[mm]:	0
Indexed' LSN:	N/A	LSN:	0

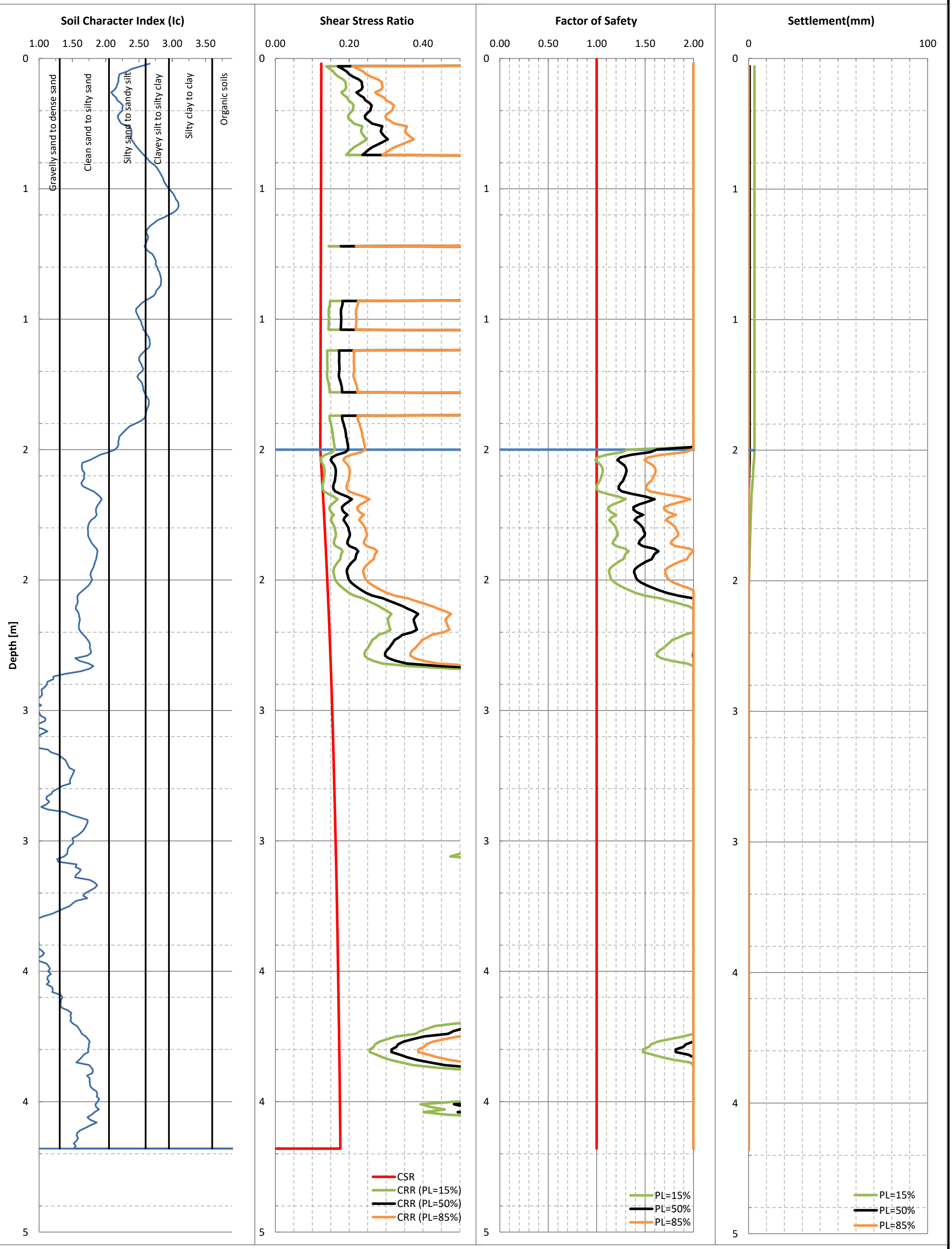
aurecon	Client: Fulti Hogan Land Developments	Location: Rosemerryn Subdivision
	Project No.: 520194	Test No.: CPT 2
	Design Event: SLSa	Date: 11 February 2022

LIQUEFACTION ANALYSIS

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

I_c cut off: 2.60
C_{FC}: 0.00

Water Table [m] 1.50
Magnitude 6.00
Acceleration [g] 0.19



Indexed' Settlement (PL=15%) [mm]: N/A
Indexed' Settlement (PL=50%) [mm]: N/A
Indexed' Settlement (PL=85%) [mm]: N/A
Indexed' LSN: N/A

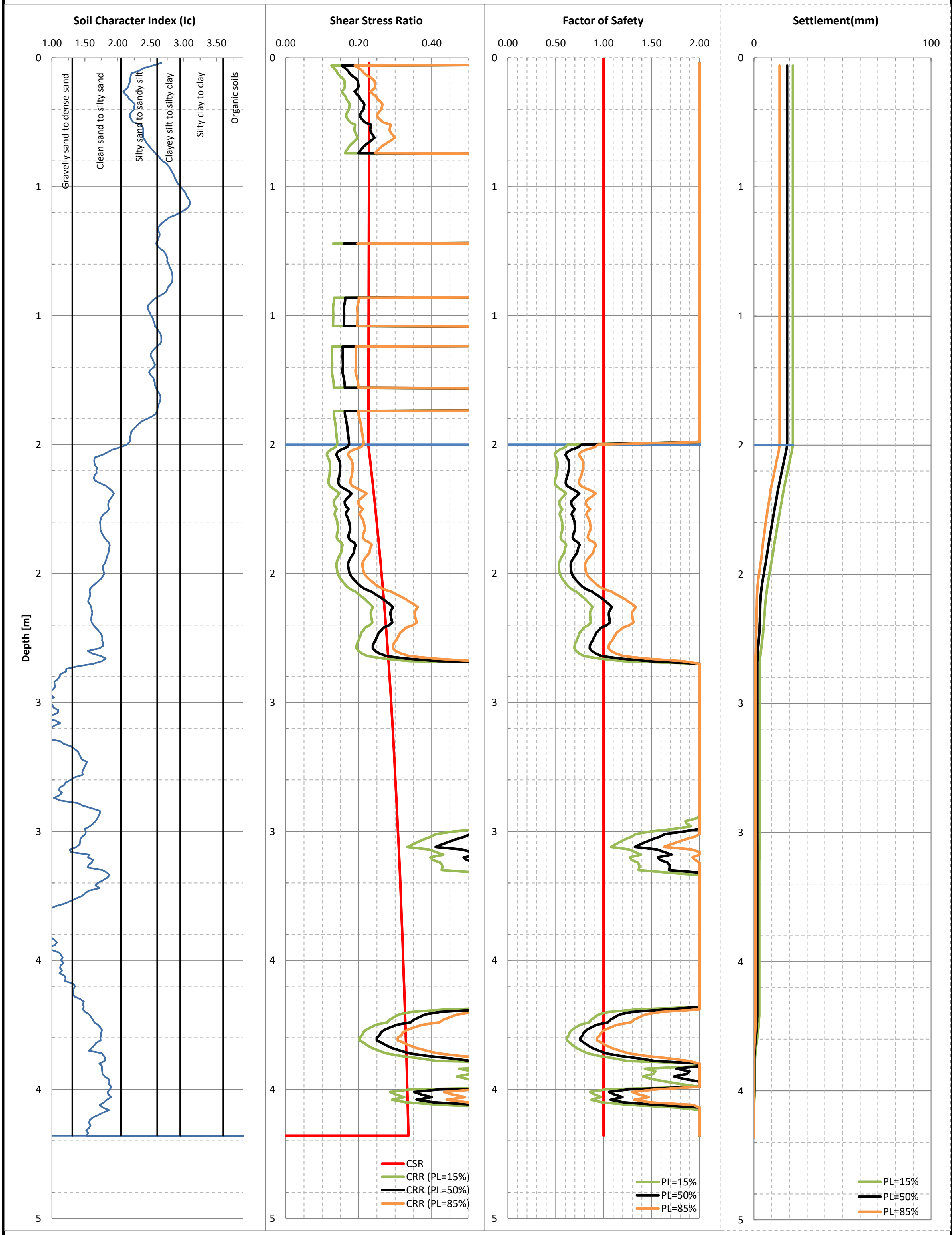
Total Settlement (PL=15%) [mm]: 3
Total Settlement (PL=50%)[mm]: 1
Total Settlement (PL=85%)[mm]: 0
LSN: 2

aurecon	Client: Fulti Hogan Land Developments	Location: Rosemerryn Subdivision
	Project No.: 520194	Test No.: CPT 2
	Design Event: SLSb	Date: 11 February 2022

LIQUEFACTION ANALYSIS

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

I_c cut off:	2.60	Water Table [m]	1.50
C_{FC}:	0.00	Magnitude	7.50
		Acceleration [g]	0.35



Indexed' Settlement (PL=15%) [mm]:	N/A	**Total Settlement (PL=15%) [mm]:**	**22**
Indexed' Settlement (PL=50%) [mm]:	N/A	**Total Settlement (PL=50%)[mm]:**	**19**
Indexed' Settlement (PL=85%) [mm]:	N/A	**Total Settlement (PL=85%)[mm]:**	**14**
Indexed' LSN:	N/A	**LSN:**	**11**

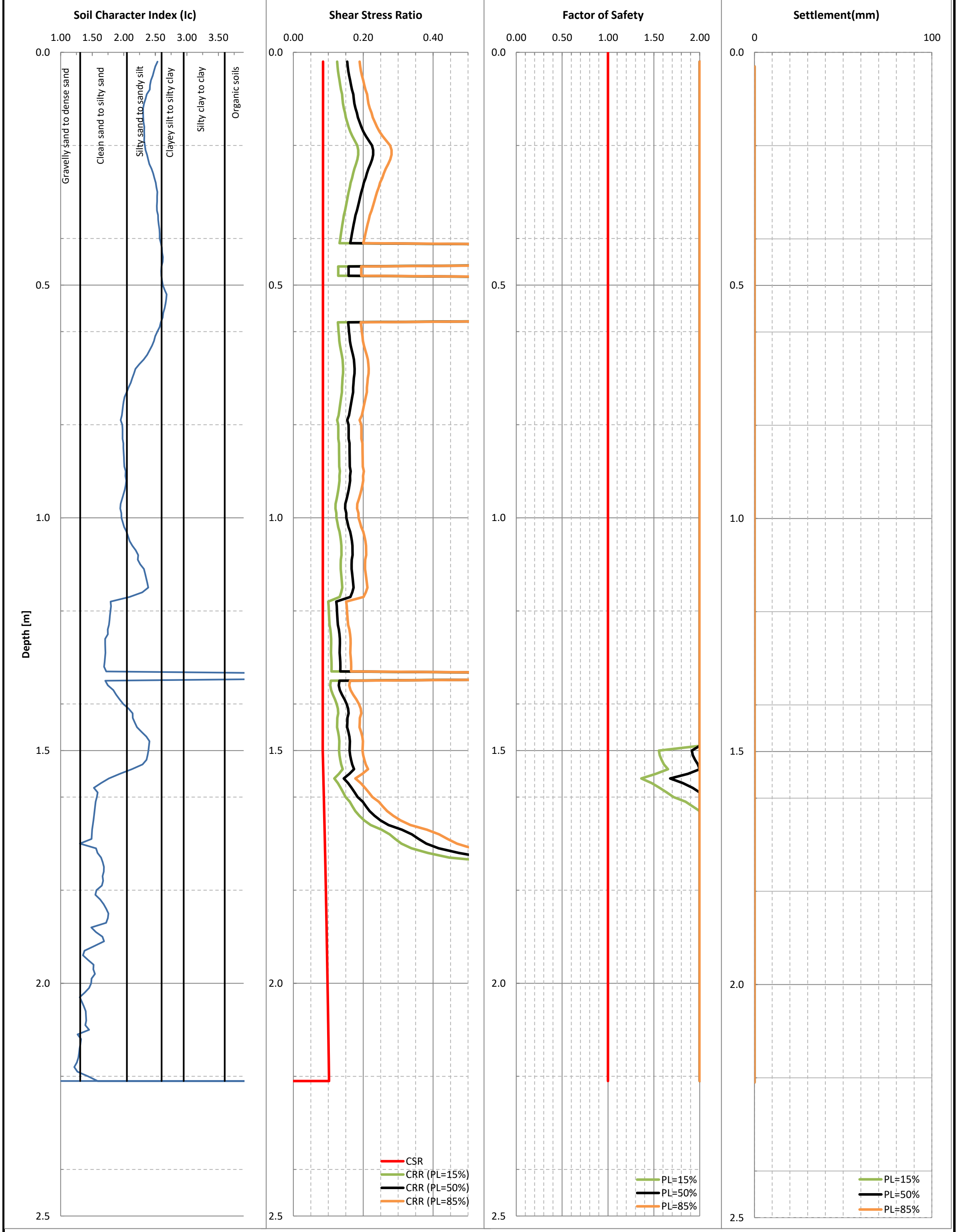
aurecon		
	Client **Fulti Hogan Land Developments**	Location **Rosemerryn Subdivision**
	Project No. **520194**	Test No. **CPT 2**
	Design Event **ULS**	Date **11 February 2022**

LIQUEFACTION ANALYSIS

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

I_c cut off: 2.60
C_{FC}: 0.00

Water Table [m] 1.50
Magnitude 7.50
Acceleration [g] 0.13



Indexed' Settlement (PL=15%) [mm]: N/A
Indexed' Settlement (PL=50%) [mm]: N/A
Indexed' Settlement (PL=85%) [mm]: N/A
Indexed' LSN: N/A

Total Settlement (PL=15%) [mm]: 0
Total Settlement (PL=50%)[mm]: 0
Total Settlement (PL=85%)[mm]: 0
LSN: 0

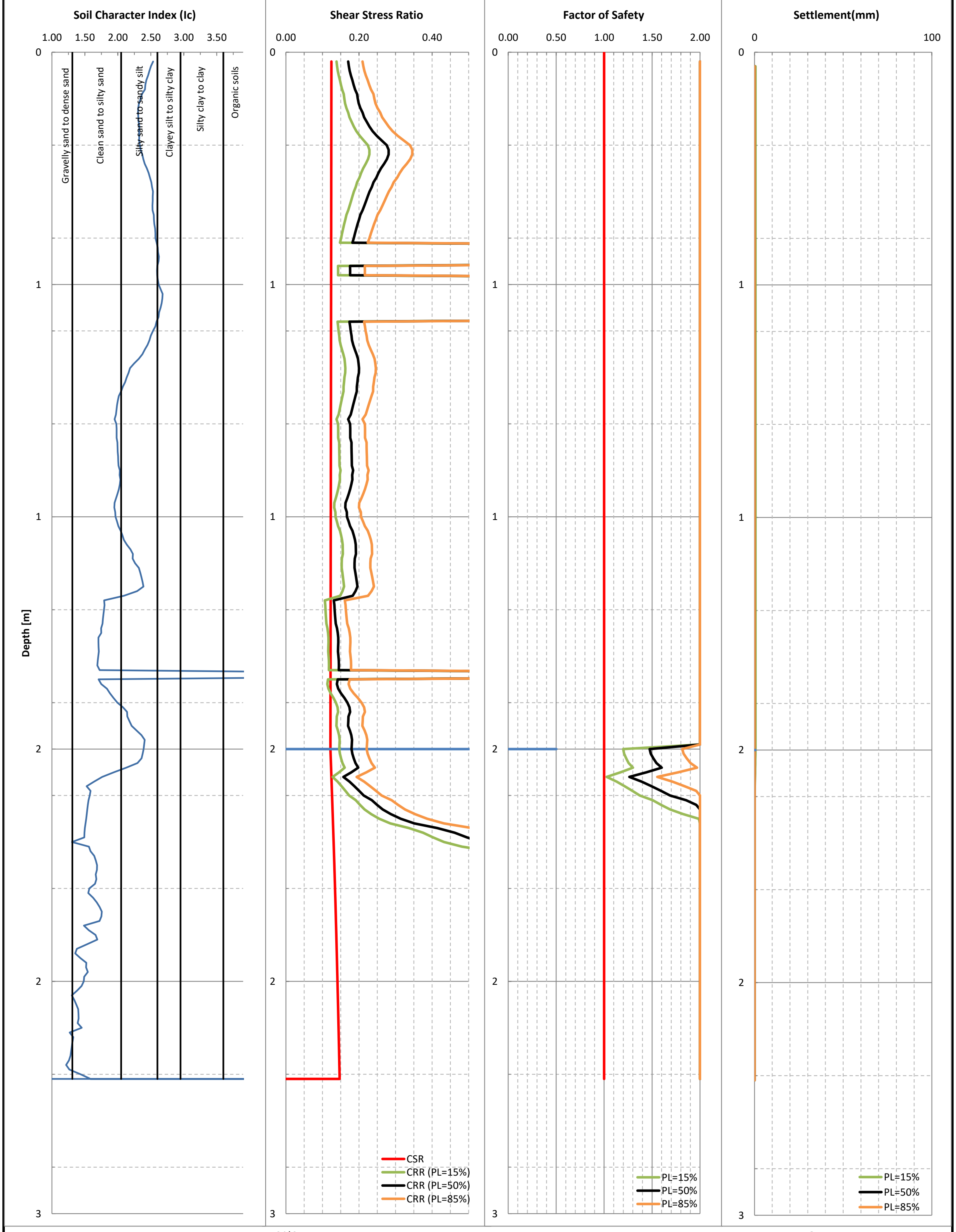
aurecon

Client	Location
Fulton Hogan Land Developments	**Rosemerryn Subdivision**
Project No. **520194**	Test No. **CPT 3**
Design Event **SLSa**	Date **11 February 2022**

LIQUEFACTION ANALYSIS

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

I_c cut off:	2.60	Water Table [m]	1.50
C_{FC}:	0.00	Magnitude	6.00
		Acceleration [g]	0.19



Indexed' Settlement (PL=15%) [mm]:	N/A	**Total Settlement (PL=15%) [mm]:**	1
Indexed' Settlement (PL=50%) [mm]:	N/A	**Total Settlement (PL=50%)[mm]:**	0
Indexed' Settlement (PL=85%) [mm]:	N/A	**Total Settlement (PL=85%)[mm]:**	0
Indexed' LSN:	N/A	**LSN:**	0

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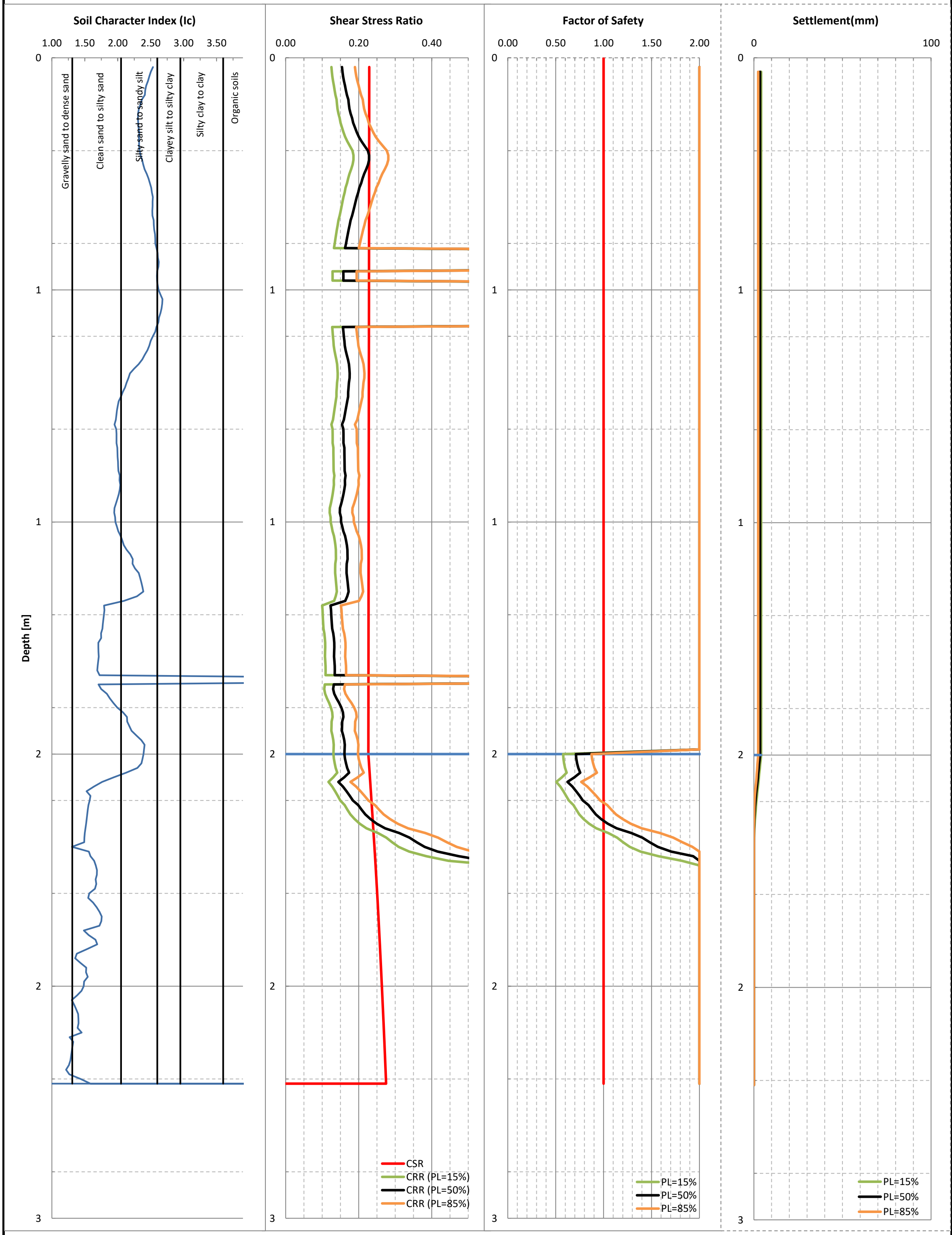
Client **Fulton Hogan Land Developments**	Location **Rosemerryn Subdivision**
Project No. **520194**	Test No. **CPT 3**
Design Event **SLSb**	Date **11 February 2022**

LIQUEFACTION ANALYSIS

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

I_c cut off:	2.60	Water Table [m]	1.50
C_{FC}:	0.00	Magnitude	7.50
		Acceleration [g]	0.35



Indexed' Settlement (PL=15%) [mm]:	N/A	**Total Settlement (PL=15%) [mm]:**	4
Indexed' Settlement (PL=50%) [mm]:	N/A	**Total Settlement (PL=50%)[mm]:**	3
Indexed' Settlement (PL=85%) [mm]:	N/A	**Total Settlement (PL=85%)[mm]:**	2
Indexed' LSN:	N/A	**LSN:**	3

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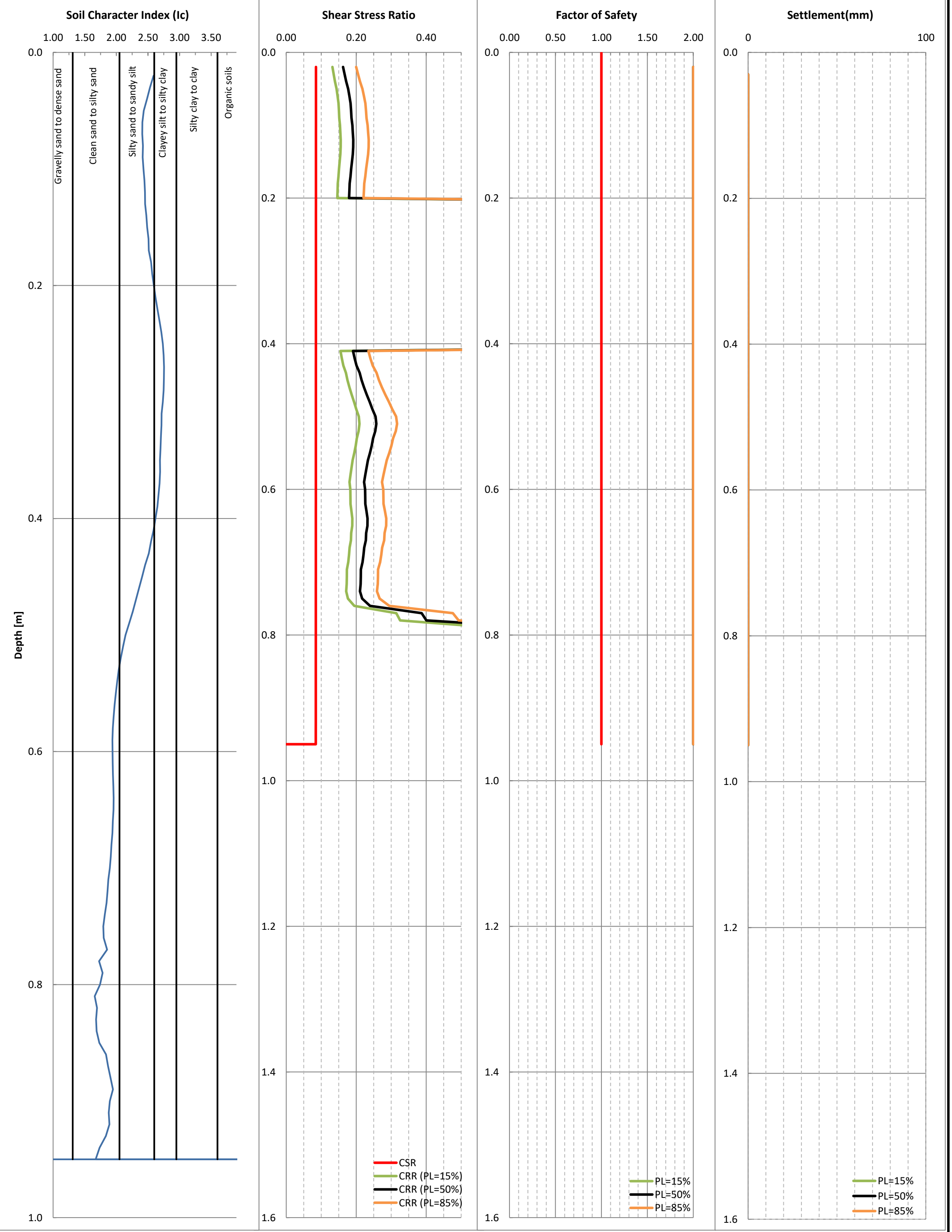
Client **Fulton Hogan Land Developments**	Location **Rosemerryn Subdivision**
Project No. **520194**	Test No. **CPT 3**
Design Event **ULS**	Date **11 February 2022**

LIQUEFACTION ANALYSIS

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

Water Table [m]	1.50
Magnitude	7.50
Acceleration [g]	0.13

I_c cut off: 2.60
C_{FC}: 0.00



Indexed' Settlement (PL=15%) [mm]: N/A
Indexed' Settlement (PL=50%) [mm]: N/A
Indexed' Settlement (PL=85%) [mm]: N/A
Indexed' LSN: N/A

Total Settlement (PL=15%) [mm]: 0
Total Settlement (PL=50%)[mm]: 0
Total Settlement (PL=85%)[mm]: 0
LSN: 0

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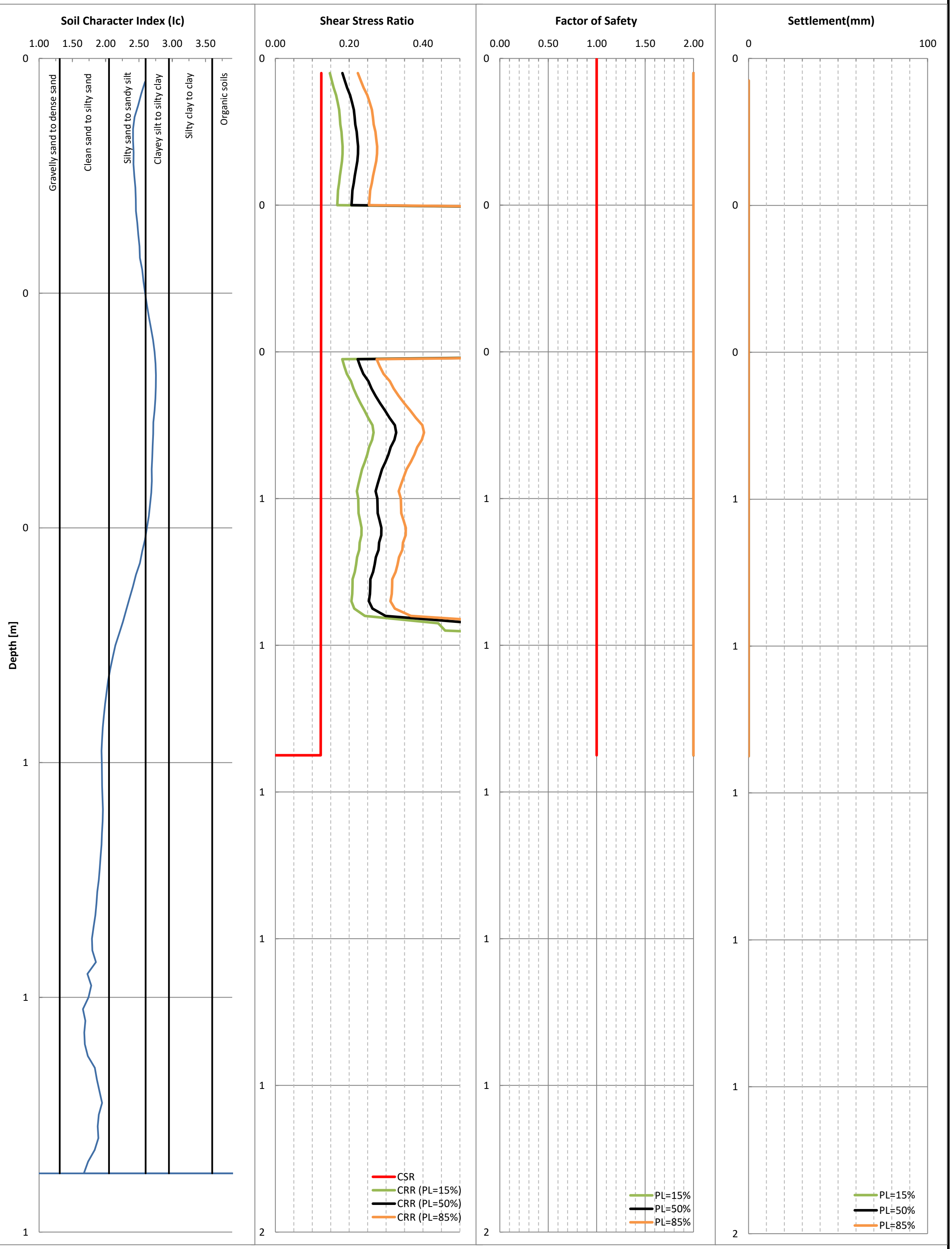
Client **Fulton Hogan Land Developments**	Location **Rosemerryn Subdivision**
Project No. **520194**	Test No. **CPT 4**
Design Event **SLSa**	Date **11 February 2022**

LIQUEFACTION ANALYSIS

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

Water Table [m]	**1.50**
Magnitude	**6.00**
Acceleration [g]	**0.19**

I_c cut off: 2.60
C_{FC}: 0.00



Indexed' Settlement (PL=15%) [mm]: N/A
Indexed' Settlement (PL=50%) [mm]: N/A
Indexed' Settlement (PL=85%) [mm]: N/A
Indexed' LSN: N/A

Total Settlement (PL=15%) [mm]: 0
Total Settlement (PL=50%)[mm]: 0
Total Settlement (PL=85%)[mm]: 0
LSN: 0

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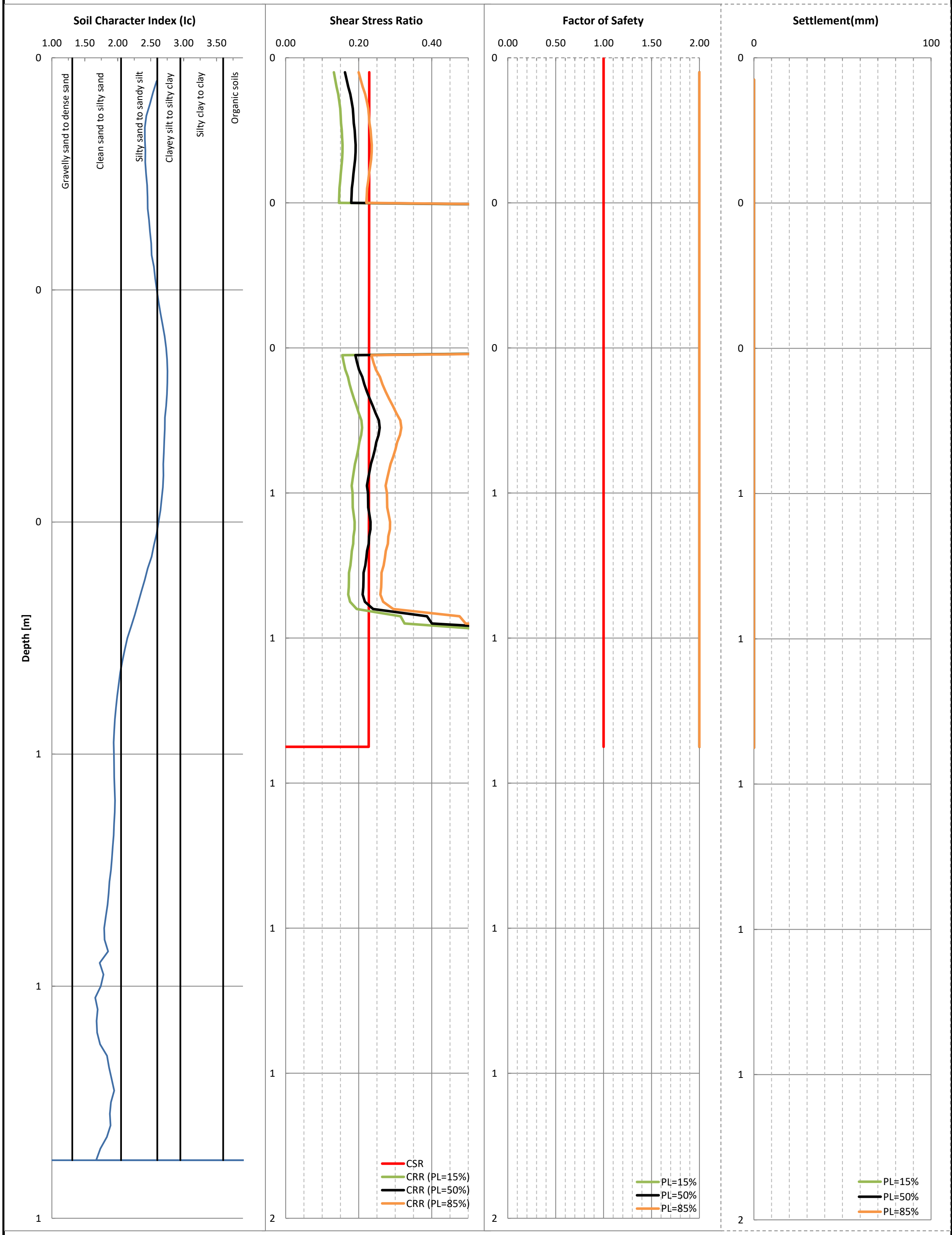
Client	Location
Fulton Hogan Land Developments	**Rosemerryn Subdivision**
Project No. **520194**	Test No. **CPT 4**
Design Event **SLSb**	Date **11 February 2022**

LIQUEFACTION ANALYSIS

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

Water Table [m]	**1.50**
Magnitude	**7.50**
Acceleration [g]	**0.35**

I_c cut off: 2.60
C_{FC}: 0.00



Indexed' Settlement (PL=15%) [mm]:	N/A
Indexed' Settlement (PL=50%) [mm]:	N/A
Indexed' Settlement (PL=85%) [mm]:	N/A
Indexed' LSN:	N/A

Total Settlement (PL=15%) [mm]:	**0**
Total Settlement (PL=50%)[mm]:	**0**
Total Settlement (PL=85%)[mm]:	**0**
LSN:	**0**

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Client **Fulton Hogan Land Developments**	Location **Rosemerryn Subdivision**
Project No. **520194**	Test No. **CPT 4**
Design Event **ULS**	Date **11 February 2022**

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RMA Assessment

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RMA Section 106 (1 & 1A) Assessment – Rosemerryn Subdivision - 520194

Client	Fulton Hogan Land Development	**Project No.**	520194
Prepared by	Tom Tremain	**Reviewed by**	I.McPherson

Risk Rating Matrix

	Likelihood of occurrence				
Most Likely Consequence	5 - Very likely	4 - Good chance	3 - Likely	2 - Unlikely	1 - Very unlikely
A - Disastrous	Extreme	Extreme	Extreme	Extreme	High
B - Critical	Extreme	Extreme	Extreme	High	High
C - Serious	Extreme	High	High	Moderate	Moderate
D - Significant	High	High	Moderate	Low	Low
E - Minor	Moderate	Moderate	Low	Low	Low

IDENTIFY NATURAL HAZARD		ASSESS RISK Section 1A (a) & (b)				RESIDUAL RISK ASSESSMENT Section 1A (a) & (b)				
Risk Source (Hazard)	**Damage**	**Likelihood**	**Consequence**	**Risk Rating**	**Control Measure (Risk Treatment)**	**Likelihood**	**Consequence**	**Risk Rating**	**Subsequent use of the land accelerate, worsen, or result in material damage resulting from hazard Section 1A (c)**	**Comments or Recommendations**
Earthquake/Seismic										
Liquefaction induced ground damage (settlement, sand boils, cracking)	Liquefaction in major seismic events is likely but is likely to be TC2 equivalent	3 - Likely	D - Significant	Moderate	Mitigation strategies in the form of strengthened structural foundations or ground improvement have been provided.	1 - Very unlikely	E - Minor	Low	No	Development can proceed provided recommendations in this report are followed and appropriate engineering measures implemented.
Liquefaction induced lateral spreading	Liquefaction induced lateral spreading is unlikely due to the lack of free, sloping faces.	1 - Very unlikely	E - Minor	Low	No specific mitigation measure proposed at this stage	1 - Very unlikely	E - Minor	Low	No	
Seismic Induced Slope Instability (incl Mass Movement)	The site is relatively flat and as such is not likely to be at risk from seismically induced mass movement.	1 - Very unlikely	E - Minor	Low	No specific mitigation measure proposed at this stage	1 - Very unlikely	E - Minor	Low	No	
Seismic Induced Rockfall	No rockfall sources above site.	1 - Very unlikely	E - Minor	Low	No specific mitigation measure proposed at this stage	1 - Very unlikely	E - Minor	Low	No	
Seismic Induced Cliff Collapse	No cliff above site.	1 - Very unlikely	E - Minor	Low	No specific mitigation measure proposed at this stage	1 - Very unlikely	E - Minor	Low	No	
Fault Rupture	No known active faults near the site.	1 - Very unlikely	E - Minor	Low	No specific mitigation measure proposed at this stage	1 - Very unlikely	E - Minor	Low	No	
Landslip/Landslide/Land Instability/Subsidence										
Landslide/Landslip	No evidence of slips around the development sites and due to lack of slopes, slips are unlikely.	1 - Very unlikely	E - Minor	Low	N/A	1 - Very unlikely	E - Minor	Low	No	Development can proceed provided recommendations in this report are followed and appropriate engineering measures implemented.
Deep Seated Landslide	No evidence of deep seated instability	1 - Very unlikely	E - Minor	Low	N/A	1 - Very unlikely	E - Minor	Low	No	
Earth/Debris flows	No earthflow sources above site nor any evidence of previous earthflows affecting site	1 - Very unlikely	E - Minor	Low	N/A	1 - Very unlikely	E - Minor	Low	No	
Rockfall or Topple	No rockfall sources above site	1 - Very unlikely	E - Minor	Low	N/A	1 - Very unlikely	E - Minor	Low	No	
Other										
Soft Ground Settlement	Potential for settlement of building foundations and other infrastructure due to the presence of soft silts, at depths of 2m to 3m.	2 - Unlikely	D - Significant	Low	Soft softs are reasonable depth so unlikely to cause settlement provided appropriate foundation design is undertaken and includes the use of enhanced slabs.	2 - Unlikely	E - Minor	Low	No	Development can proceed provided recommendations in this report are followed and appropriate engineering measures implemented.

RMA Section 106 (1 & 1A) Assessment – Rosemerryn Subdivision - 520194

Client	Fulton Hogan Land Development	**Project No.**	520194
Prepared by	Tom Tremain	**Reviewed by**	I.McPherson

Risk Rating Matrix

	Likelihood of occurrence				
Most Likely Consequence	5 - Very likely	4 - Good chance	3 - Likely	2 - Unlikely	1 - Very unlikely
A - Disastrous	Extreme	Extreme	Extreme	Extreme	High
B - Critical	Extreme	Extreme	Extreme	High	High
C - Serious	Extreme	High	High	Moderate	Moderate
D - Significant	High	High	Moderate	Low	Low
E - Minor	Moderate	Moderate	Low	Low	Low

IDENTIFY NATURAL HAZARD		ASSESS RISK Section 1A (a) & (b)			Control Measure (Risk Treatment)	RESIDUAL RISK ASSESSMENT Section 1A (a) & (b)			Subsequent use of the land accelerate, worsen, or result in material damage resulting from hazard Section 1A (c)	Comments or Recommendations
Risk Source (Hazard)	**Damage**	**Likelihood**	**Consequence**	**Risk Rating**		**Likelihood**	**Consequence**	**Risk Rating**		
Erosion	Due to finer nature of soil, erosion is possible either by concentrated stormwater runoff or subsurface seepages.	3 - Likely	E - Minor	Low	Adequate site stormwater control to be incorporated with site development and exposed soil covered with topsoil/vegetation.	2 - Unlikely	E - Minor	Low	No	As part of the civil design of the subdivision adequate stormwater and erosion control will be required. If subsoil seeps are encountered during site development then these will need to be assessed

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