GEOLOGY
GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES

22 April 2024

Cabra Developments Limited
Unit 9B, 30 Foundry Road
Silverdale
Auckland 0932

Attn: Duncan Unsworth

Dear Duncan

Private Plan Change Geotechnical Assessment - Cabra Sites, Hobsonville, Auckland

(Our Reference: 23849.000.006_04)

1 Introduction

ENGEO Limited was requested by Cabra Developments Limited to undertake a geotechnical assessment to support a private plan change (PPC) application to Auckland Council for rezoning of the properties located at 10, 12, 14 and 16 Sinton Road and 15, 17 and 17a Clarks Lane, Hobsonville, Auckland (herein referred to as “the site”, as shown on Figure 1). The proposed plan change would rezone the land from ‘Future Urban Zone’ to a residential zone (a combination of Single House, Mixed Housing Suburban and / or Mixed Housing Urban) under the Auckland Unitary Plan. This work has been carried out in accordance with our signed agreement dated 12 April 2024.

Figure 1: Site Area

Left: Red dash indicates wider peninsula area

Right: Blue perimeter indicates private plan change area

2 Scope of the Assessment

ENGEO has previously undertaken geotechnical and environmental assessments of 10, 14 and 16 Sinton Road and 15 Clarks Lane, Hobsonville, however we understand that the PPC will also incorporate 12 Sinton Road, and 17 and 17A Clarks Lane (pink areas shown on Figure 1) into a single application.

This assessment has been completed with the intention of identifying key geotechnical constraints or data gaps that may preclude a future conversion from 'Future Urban Zone' to a residential zone.

Geotechnical reports previously prepared by ENGEO for these sites are as follows:

- Geotechnical Investigation – 10 Sinton Road, Hobsonville, ref. 23849.000.002_03, 10 November 2023.
- Geotechnical Investigation – 16 Sinton Road, Hobsonville, ref. 23849.000.003_03, 10 November 2023.
- Geotechnical Investigation – 15 Clarks Lane, Hobsonville, ref. 23849.000.004_02, 10 November 2023.
- Geotechnical Investigation – 14 Sinton Road, Hobsonville, ref. 23849.000.005_01, 22 February 2024.

We consider that the previous site investigations undertaken by ENGEO for four of the properties within the PPC area are suitable to support this plan change application, and no further site investigation works are proposed on the remaining land parcels. Therefore, this assessment has comprised a desk-based assessment of the three additional lots not previously investigated by ENGEO, in addition to the wider peninsula (pink shaded areas and red outlined area shown on Figure 1, respectively).

3 Site Description

The site is situated in a rural residential area and is bound by Sinton Road or Clarks Lane to the southeast, lifestyle blocks to the southwest and northeast, and by the Waiarohia Inlet to the northwest. The properties within the PPC area are developed as rural-residential properties with primarily undeveloped land supporting horticulture and grazing.

The landform slopes broadly towards the north, with the coastal margin defined by 5 to 10 m high slopes ranging between 10 to 50 degrees, with low height (1 to 2 m typically) soft cliffs comprising soil or very weak rock in the tidal zone over much of the coastal margin. The Waiarohia Inlet comprises mud flats and mangroves, and where slope instability or gully development has occurred on the coastal margin, those slopes are typically less than 10 degrees.

Beyond the PPC area over the wider peninsula, the landform and land use are broadly comparable. A residential subdivision has recently been completed on the southern side of Ockleston Landing, beyond the eastern end of Clarks Lane. Similar subdivisions are also underway, or have been completed, on the southern side of the Upper Harbour Motorway.

4 Desktop Study

The following sections present a summary of the desktop study undertaken to support this assessment, and the key findings from a geotechnical perspective.

4.1 Aerial Photographs

Aerial photographs dating from 1940 to 2023 were reviewed for relevant visible features over the PPC properties and the surrounding area. The aerial photos were sourced from Auckland Council GeoMaps, Retrolens and Nearmaps.

From 1940 to the later 1980s the site and wider peninsula (red area shown on Figure 1) primarily comprises agricultural land with associated residential buildings. The Upper Harbour Motorway was constructed to the south of the site circa 2010. Residential (high-density) and commercial redevelopment of land to the south of the motorway, and southeast of the site commences after this time.

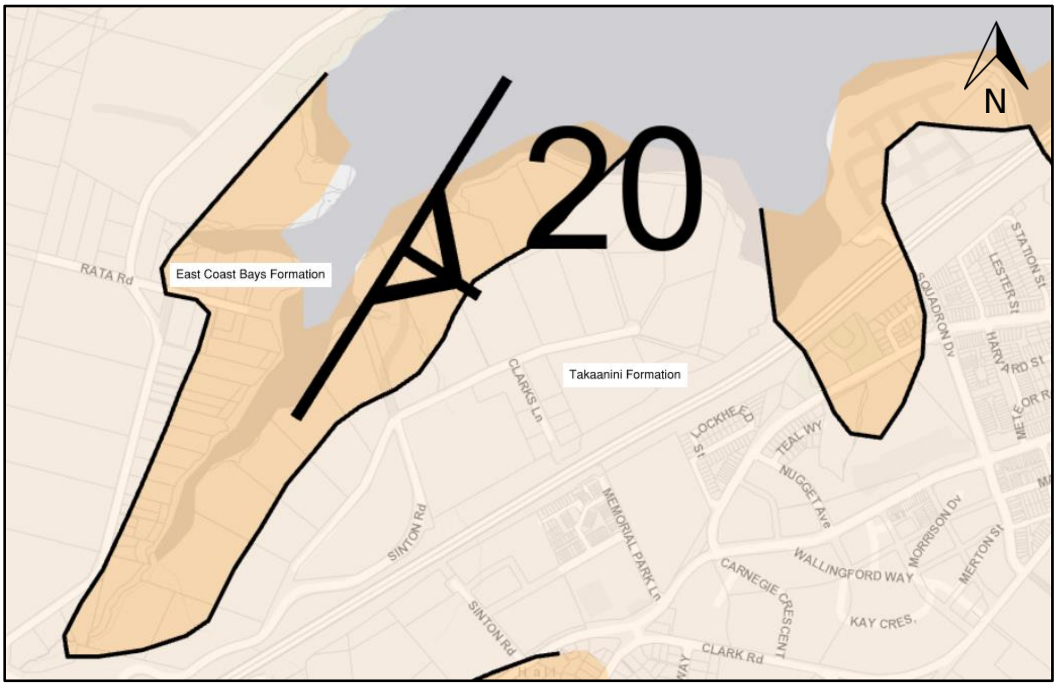
Evidence for local land development is visible within individual properties at varying times over the last 80 years. This is observed as formation of new tracks and driveways, new sheds and residential buildings, and in the case of 14 Sinton Road, some earthworks on the north-western site boundary adjacent to the Waiarohia Inlet. Evidence for earthworks over an area of approximately 4,000 m^2 between 2006 and 2015 are also visible at 17 Sinton Road on the southern side of the property. The land disturbance appears to have been intermittent, with the area revegetated in grass after 2015.

Some geomorphic evidence for historical slope instability is visible in the aerial photographs. These are typically constrained to the slopes adjacent to the Waiarohia Inlet and associated tributary gullies and are discussed in further detail in the following sections.

4.2 Published Geology

The PPC area is mapped by GNS Science as being located across a geological boundary between East Coast Bays Formation on the northern side of the properties, and undifferentiated Takaanini Formation on the southern side of the properties (Figure 2).

Figure 2: Geological Map



GNS Science Geology Webmap, sourced April 2024.

East Coast Bays Formation rock comprises alternating sandstone and mudstone with variable volcanic content and interbedded volcaniclastic grits. East Coast Bays Formation residual soils (weathered from the underlying rock) are generally described as orange and grey silts and clays with varying sand content. The geological map in Figure 2 shows a locally mapped bedding dip of 20 degrees towards the southeast, which is why East Coast Bays Formation soils and rock form the low sea cliffs across the northern side of the peninsula but are overlain by Takaanini Formation alluvium to the south.

The Takaanini Formation (formerly named Puketoka Formation) comprises relatively young and weak sedimentary strata encountered across much of the Auckland area. Based on investigation data over the PPC area and the wider peninsula, the local members are likely to comprise the Runciman Member (carbonaceous sand or mud, organic material prominent), Ardmore Member (peat / lignite, organic material dominant by volume), and the Pahurehure Member (predominantly fluvial sediments comprising gravel, sand, or silt / clay soils).

4.3 Geomorphological Appraisal

Site walkovers of the 10, 14 and 16 Sinton Road and 15 Clarks Lane properties were undertaken by ENGEO in late 2023 to observe the landform across those sites. These site walkovers were supplemented by a desktop study and site-specific investigations for each property. Specific details for each site are presented in the corresponding Geotechnical Investigation Reports issued by ENGEO and are summarised in the following sections.

4.3.1 Topography and Drainage

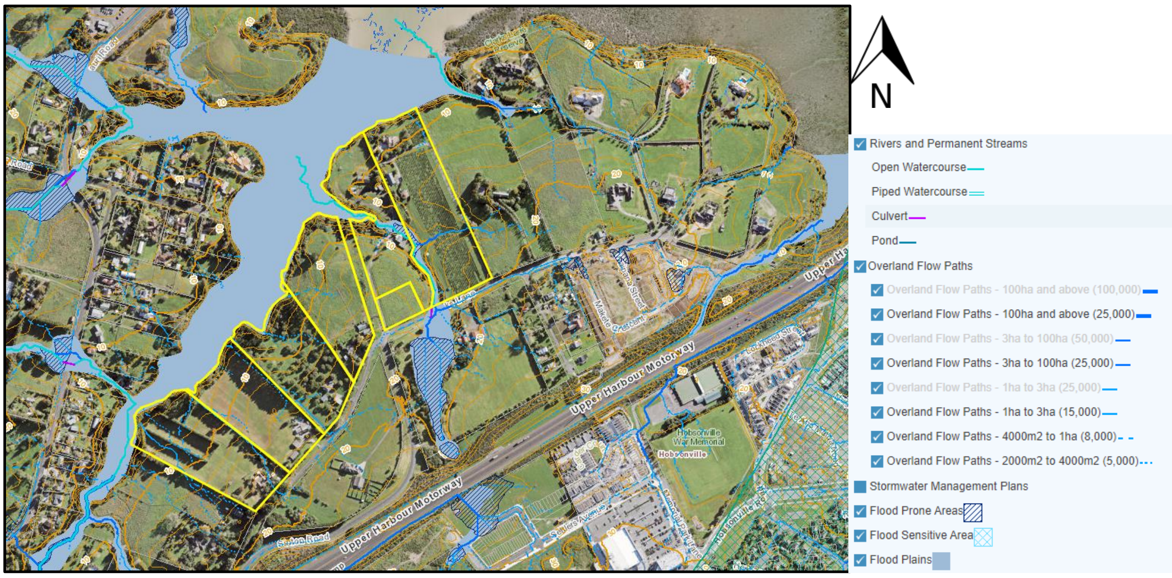
The landform is characterised by broadly undulating topography descending from the southern side of Clarks Lane and Sinton Road towards the Waiarohia and Wallace Inlets to the north and northeast. The coastal margin is characterised by low height (typically 5 to 10 m) slopes between 30 and 50 degrees with a sub-vertical soil or very weak rock cliff exposed in the tidal zone.

A network of overland flow paths and permanent streams dissect the landscape and drain towards the Waiarohia Inlet to the northwest. A man-made pond has been formed on the southern side of Clarks Lane and is connected to the gully through 15 and 17 Clarks Lane via a culvert beneath the road.

The Auckland Council GeoMaps Catchments and Hydrology layer (Figure 3) identifies areas of land that could be affected by flooding during and / or following periods of heavy rain. Portions of the site labelled as flood prone, or flood plains are limited to areas immediately adjacent to Waiarohia Inlet and its associated tributaries, including the pond.

Evidence for localised swampy ground, surface ponding and associated softened ground was observed in the vicinity of overland flow paths on the sites assessed by ENGEO.

Figure 3: Topography and Drainage



Catchments and Hydrology layer sourced from Auckland Council GeoMaps, April 2024.

4.3.2 Slope Instability

There is evidence for historical slope instability on the coastal margin, most notably at 10 Sinton Road which has been affected by two landslides (Figure 4, Image 1). The landslides occurred prior to 1940 (the earliest available aerial photographs), and do not appear to have been reactivated in recent years based on our desktop and site observations. Those landslides have been assessed in the ENGEO geotechnical report for that site (referenced in Section 2).

The property at 17 Sinton Road has not been investigated as part of this assessment, however aerial photographs indicate possible small-scale slope instability in the vicinity of the gully flanks in the form of bare earth scars adjacent to the steep slopes (Figure 4, Images 2 and 3).

Elsewhere, evidence for active soil creep was observed on some of the densely vegetated northern slopes where ENGEO has completed a site walkover. For sites not assessed by ENGEO, evidence of instability may be obscured by dense vegetation cover.

Figure 4: Observed Slope Instability

Image 1: 10 Sinton Road, two head scarps as shown

Image 2: 17 Sinton Road, 2000 aerial photograph

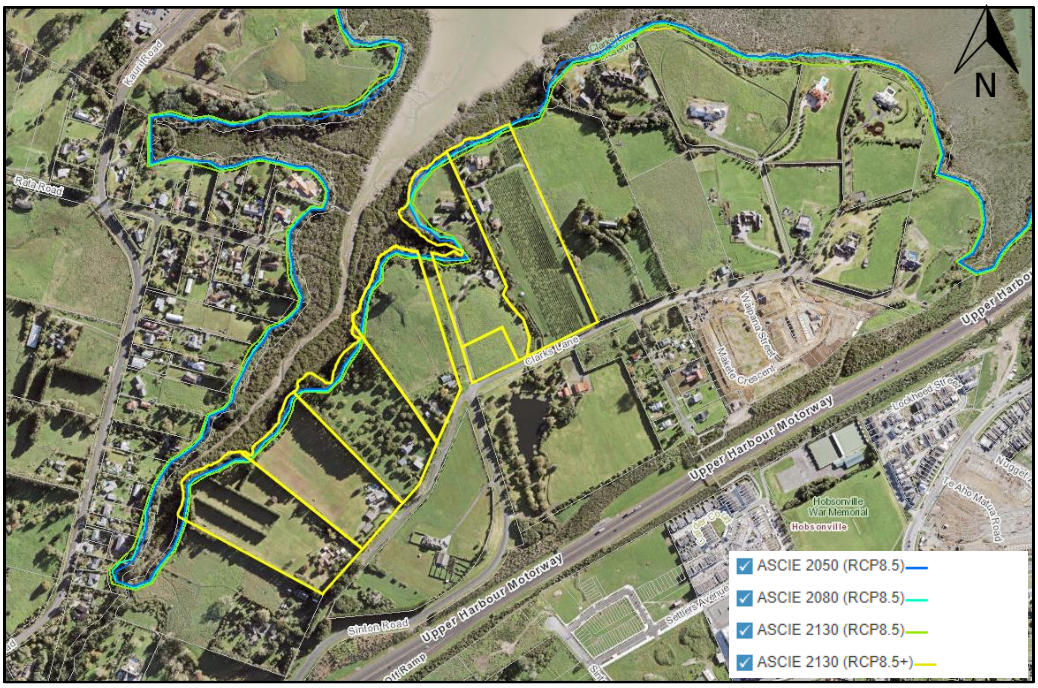
Image 3: 17 Sinton Road, 2001 aerial photograph

4.3.3 Coastal Hazards

The Auckland Council GeoMaps Areas Susceptible to Coastal Instability and Erosion (ASCIE) layer identifies a zone up to approximately 25 m wide that extends along the northern boundary of the PPC area. Land within that zone is assessed as likely to be susceptible to inundation, erosion, and land instability as a result of sea level rise and dynamic coastal processes over the next 100 years.

The work undertaken to develop the ASCIE layer was completed at a regional level and is intended to be used as a planning tool to indicate where a site-specific assessment will be necessary to support a change in future land use. A site-specific assessment has been completed by SLR Consulting New Zealand (ref. 850.016583.00001, dated April 2024) for the PPC area to support the plan change application.

Figure 5: Areas Susceptible to Coastal Instability and Erosion



Areas Susceptible to Coastal Instability and Erosion layer sourced from Auckland Council GeoMaps, April 2024.

4.3.4 Land Modification and Existing Fill

Evidence for land modification associated with horticultural land use and residential development was observed across limited portions of the PPC area in the historical aerial photograph review and on-site. Evidence of domestic refuse tipping was also observed at some sites during the geotechnical investigations by ENGEO (e.g., northern boundary of 10 Sinton Road).

Where earthworks have been completed within the wider peninsula area for residential development or on a large scale, it is reasonable to expect that these would have been undertaken under the guidance of a Geotechnical Engineer and records should be available with respect to fill quality and placement. However, away from those areas it is reasonable to expect that fill placed to form dams, farm tracks and accessways, or to support other horticultural land use operations is unlikely to have been placed to an engineered standard. To that end, future land development within the PPC area will need to consider the potential for existing fill and address its suitability to remain in place.

4.4 Investigation Data

ENGEO has previously undertaken geotechnical investigations of the properties at 10, 14 and 16 Sinton Road and 15 Clarks Lane (refer to Section 2). The investigations comprised a combination of site walkovers, hand auger boreholes and cone penetrometer tests (CPTs). Full details of the investigation findings are included in the relevant reports.

The New Zealand Geotechnical Database has also been consulted for data over the wider peninsula area. There is a substantial data set available for the area as shown in Figure 6.

Figure 6: Investigation Data

ENGEO investigation data compiled from ENGEO database, NZGD data imported in April 2024.

4.4.1 ENGEO Investigation Findings

The ENGEO investigation findings generally concur with the published geological mapping. East Coast Bays Formation residual soils were typically encountered over the northern portions of the sites investigated, with Takaanini Formation (formerly named Puketoka Formation) alluvial soils encountered through the central and southern portions of the sites. The residual soils typically comprised stiff to hard silt and clay soils that generally increase in strength with depth, while the alluvial soils comprised interfingered layers of sands, silts and clays with occasional organic soil and peat lenses. Soil strengths within the alluvium were variable with depth but generally ranged from stiff to hard.

The investigations also encountered undocumented fill around existing buildings and in farm operations areas, as well as colluvium on the northern end of 10 Sinton Road in association with the landslides identified there.

4.4.2 Groundwater

Groundwater was measured across the sites at hand auger borehole locations on the day of drilling, and during the CPT progression. The data indicates that groundwater levels vary across the area and are influenced by soil strata, proximity to overland flow paths or water courses, and proximity to the coastline. Groundwater may be expected within 5 m of the ground surface across much of the site area.

5 Geohazards

The following geohazards have been assessed as they have the potential to affect the safe and economic development of the PPC area.

5.1 Seismic Hazards

The site is located approximately 38 km from the nearest active fault (the Waikopua Fault) and is therefore unlikely to be affected by surface rupture.

Due to the presence of saturated sands and unrestrained surfaces (free faces) there is a potential for lateral spreading or localised movement under seismic accelerations. This is discussed further in Section 5.1.1.

The risk of seismically induced tsunami inundation is low due to the sheltered location of the PPC area within the upper Waitemata Harbour, and the elevation of the developable land (5 to 10 m above sea level). Land susceptible to tsunami inundation is expected to be limited to the low-lying intertidal land.

5.1.1 Liquefaction and Lateral Spread

Site-specific liquefaction assessments were undertaken for the properties at 10, 14 and 16 Sinton Road and 15 Clarks Lane supported by CPT testing for each. The assessments confirmed that the soils identified at those sites are not susceptible to liquefaction under SLS accelerations, and settlements under ULS accelerations are expected to fall within the “insignificant” to “mild” categories (L_0/L_1) in accordance with Table 5.1 of MBIE / NZGS Module 3.

Based on these findings, it is reasonable to expect that future land development will be possible using conventional earthworks and / or building foundation design to mitigate the calculated liquefaction-induced settlements.

5.1.2 Seismic Site Soil Class

Based on the investigation data available for the PPC area, a seismic site classification of “Class C – Shallow Soil Sites” in line with NZS 1170.5:2004 is considered appropriate for seismic design.

5.2 Expansive Soils

Laboratory testing undertaken for the properties at 10, 14 and 16 Sinton Road and 15 Clarks Lane indicates that the expansive soil site class for the soils over the PPC area range from non-expansive through to M (moderately) expansive with respect to NZS 3604 (from Section 3.2 of B1/AS1 November 2019 Amendment).

Based on our experience in this part of Auckland with these particular soils, we recommend a preliminary conservative expansive soil site class of M (moderately) expansive is assumed for preliminary design purposes, with site-specific testing at the detailed design and construction verification stages to confirm the ground conditions at subgrade levels for future building developments.

5.3 Compressible Soils

Young alluvium of the Takaanini Formation can be susceptible to consolidation settlements as they are typically normally consolidated, have higher groundwater levels than their residual soil counterparts, and may contain organic soil and peat layers of variable strength.

Organic soils and peat layers were encountered in boreholes drilled at each end of the PPC area (15 Clarks Lane and 16 Sinton Road) and are documented in other boreholes in the area (sourced from the NZGD).

The organic soils encountered within the PPC area are typically laterally discontinuous and less than 1.0 m thick, so are not expected to present a significant risk to residential development. The risk of settlements induced by fill and building loads can be managed using conventional earthworks approaches (e.g., minimising cut and fill earthworks in areas where compressible soils are present near the surface) and through specific foundation design.

5.4 Slope Instability

Evidence for historical slope instability was observed at 10 Sinton Road (two landslides) and has been observed elsewhere on the coastal margin through aerial photograph review. There is also evidence for active shallow-seated instability in the form of soil creep on the steep coastal and gully-adjacent slopes. These processes are common across the Auckland region and present a manageable risk for which conventional mitigation measures are readily available.

The landslide features that extend into, or are located immediately adjacent to, future development areas will require specific investigation at the development design stage to inform design of appropriate mitigation measures. These may include a combination of specifically designed geotechnical drainage, retaining wall(s), and / or bulk earthworks solutions to support stable building platforms and associated infrastructure. Alternatively, these features may be avoided through implementation of building restriction lines to impose appropriate setbacks.

5.5 Coastal Instability and Erosion

SLR Consulting New Zealand Limited have prepared a Coastal Hazard Assessment report (ref. 850.016583.00001, dated April 2024) for the PPC area. The report concludes that the overall risk to the subject site from coastal hazards is considered low and will be included with the PPC application for reference.

5.6 Uncontrolled Fill

Uncontrolled fill associated with historical building development and horticultural / agricultural land use can be expected across portions of the sites within the PPC area. The desktop study has identified larger areas of earthworks (e.g., 17 Sinton Road) for which no details have yet been reviewed. However, in general it can be reasonably expected that localised earthworks are likely to have involved site-won native soils and may be suitable to remain in place or to be included in future earthworks, subject to assessment by a Geotechnical Engineer. Where organic, domestic refuse or otherwise unsuitable inclusions are identified the fill would be unsuitable for use and would need to be cut to waste.

6 Conclusions

Based on the findings of the existing ENGEO geotechnical reports for properties within the PPC area, as well as this desktop study of the wider peninsula area for context, we have not identified geohazards which would be likely to preclude future conversion of this area to residential land use provided that the normal geotechnical investigation, analysis and design process is followed. The geohazards identified in this assessment are typical of land development in the Auckland region and are able to be addressed through conventional engineering design approaches.

This report is not intended to replace the need for a site-specific geotechnical investigation for properties not already assessed as part of a future redevelopment. In some cases, supplementary geotechnical investigation and analysis has been recommended for properties already assessed to inform design of mitigation measures to address land instability, liquefaction and consolidation settlement geohazards.

7 Limitations

i. We have prepared this report in accordance with the brief as provided. This report has been prepared for the use of our client, Cabra Developments Limited, their professional advisers and the relevant Territorial Authorities in relation to the specified project brief described in this report. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity.

ii. The recommendations in this report are based on the ground conditions indicated from published sources, site assessments and subsurface investigations described in this report based on accepted normal methods of site investigations. Only a limited 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 judgement and it should be appreciated that actual conditions could vary from the assumed model.

iii. 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.

iv. This Limitation should be read in conjunction with the Engineering NZ / ACENZ Standard Terms of Engagement.

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

We trust that this information meets your current requirements. Please do not hesitate to contact the undersigned on (09) 972 2205 if you require any further information.

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Project Number 23849.000.004

Geotechnical Investigation

15 Clarks Lane, Hobsonville, Auckland

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ENGEO Document Control:

Report Title	Geotechnical Investigation - 15 Clarks Lane, Hobsonville		
Project No.	23849.000.004	**Doc ID**	04
Client	Cabra Developments Limited	**Client Contact**	Duncan Unsworth
Distribution (PDF)	Duncan Unsworth (Cabra), Tom Lemon (Capture Land Development Consultants)		

Date	**Revision**	**Description**	**Author**	**Reviewer**	**WP**
10/11/2023	0	Client Issue	RD	PF	DF
06/05/2024	1	Issued to Client	JT	HL	JT

1 Introduction

ENGEO Limited was requested by Cabra Developments Limited to undertake a geotechnical investigation of the property at 15 Clarks Lane, Hobsonville, Auckland (herein referred to as ‘the site’; shown in Appendix 1). The purpose of this assessment was to support a Resource Consent application for the proposed redevelopment of the site. This work has been carried out in accordance with our signed agreement dated 31 July 2023.

Our scope of works for this report included:

- Desktop review of existing geotechnical reports and drawings for the site, a review of publicly available geological and geotechnical data and aerial photographs.
- Undertake a site walkover to assess current site conditions and observe for geomorphological evidence of land disturbance, active and historical slope instability, and in the case for this particular property being adjacent to the coastline, soil and rock outcrops and groundwater seepages that may be observed in the sea cliff.
- Drill up to eight hand auger boreholes to a target depth of 5.0 m below ground level (bgl) with associated strength tests across the site to provide geotechnical data on the shallow soil profile.
- Undertake two Cone Penetration Tests (CPTs) to target depths of 15.0 m bgl to support a liquefaction assessment for the alluvial soils.
- Recover a representative soil sample from near surface soils for laboratory expansive soils classification testing.
- Undertake a liquefaction and settlement assessment using primary CPT data from our intrusive investigations.
- Preparation of this Geotechnical Investigation Report presenting the findings of our investigation and geohazard assessments to support the Resource Consent application.

2 Site Description

The site comprises 3.3955 ha section of joint residential and pastoral zoned land legally described as LOT 2 DP 92753. The site is accessed via a private driveway directly off Clarks Lane in the south-eastern corner of the site. Clarks Lane forms the southern site boundary. The site is currently comprised of orchards and grass fields with three residential dwellings as shown in Appendix 1.

The residential dwellings are located within the north-western corner and near the central western boundary of the site, along with minor vegetation. The central and north-eastern portion of the site is comprised of orchards. The site is bound by Clarks Lane to the south, lifestyle blocks and residential dwellings to the east and west, and by the Waiarohia Inlet to the north. There is dense vegetation in the north and western extents of the site along the boundaries.

There are three overland flow paths that run through the site. A flow path runs from east to west, cutting across the centre of the site. The other two flow paths originate within the northeast and northwest corners of the site and drain northward to the Waiarohia Inlet. These flow paths divert surface water run-off from the upslope areas to the Waiarohia Inlet.

There are no existing public services that run through the site. There is a private Hi-Tech septic tank located in the northwest corner by one of the residential properties and two standard septic tanks located in the central eastern portion near the other two residential properties.

The site falls gently from 18.5 m RL in the south-eastern corner down to 7.0 m RL in the north-western corner at approximately 3 degrees. From here the site slopes moderately steeply into the Waiarohia Inlet from 7.0 to 1.5 m RL at approximately 32.3 degrees. Minor changes in elevation can be noted along the alignment of the overland flow paths throughout the site.

3 Proposed Development

ENGEO has been provided with the Capture proposed development plans (Appendix 2, ref CLEN-1100, undated). These plans depict the proposed development of 83 residential lots with three accessways, one of which will be a main road with a north to south alignment and two COALs. These plans do not depict service lines nor retaining wall structures.

The provided cut fill plan depicts minor cuts and fills, with up to 2.5 m of fill to be placed across the central overland flow path, and cuts to central parts of the site and northern end of the site of up to 4.5 m in depth.

4 Desktop Study

4.1 Published Geology

Published geological maps (GNS Science) indicate that the site contains a geological boundary within the southern third of the site. East Coast Bays Formation is mapped to the north of this geological boundary and Puketoka Formation to the south.

Puketoka Formation soils of the Tauranga Group comprise pumiceous mud, sand and gravel with muddy peat and lignite: rhyolite pumice, including non-welded ignimbrite, tephra and alluvial pumice deposits; massive micaceous sand.

East Coast Bays Formation rock comprises alternating sandstone and mudstone with variable volcanic content and interbedded volcaniclastic grits. East Coast Bays Formation residual soils (weathered from the underlying rock), are generally described as orange and grey silts and clays with varying sand contents.

4.2 Auckland Council GeoMaps

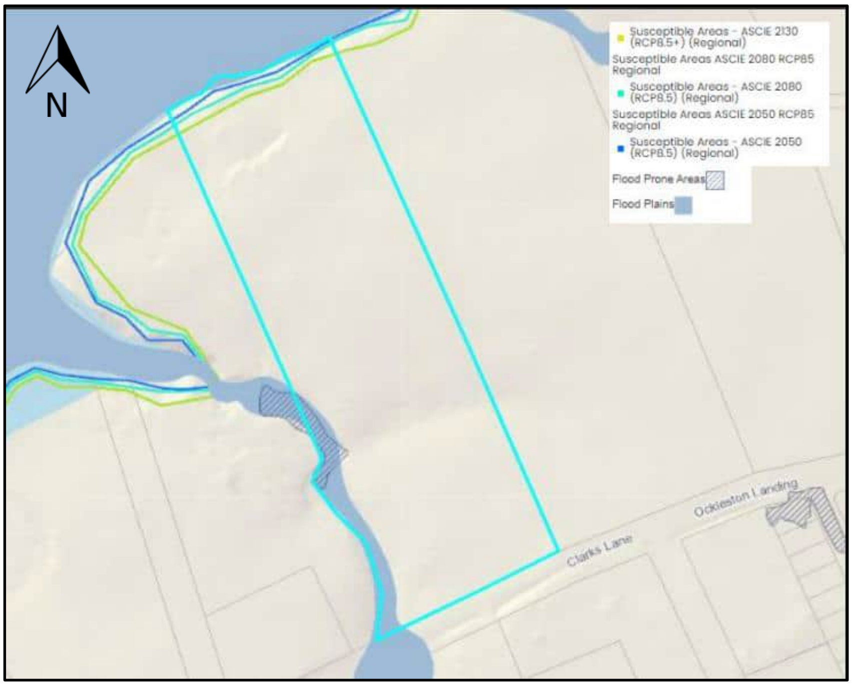
4.2.1 Coastal Instability and Erosion

The Auckland Council GeoMaps layer ‘Areas Susceptible to Coastal Instability and Erosion’ identifies areas of coastline in Auckland that could be affected by coastal erosion and instability under a range of climate change scenarios and timeframes. The potential regression lines for 2050, 2080 and 2130 for this site are shown in Figure 1. These areas are limited to the northern slopes, along the Waiarohia Inlet.

4.2.2 Flood Plains & Prone Areas

The Auckland Council GeoMaps layer 'Flood Plains & Flood Prone Areas' identifies areas of land in Auckland that could be affected by flooding during and / or following periods of heavy rain. Portions of the site labelled as flood prone or flood plains are shown in Figure 1 and are limited to areas adjacent to the western boundary, in the southern portion of the site.

Figure 1: Auckland Council Hazard Map



4.3 Historical Aerial Photography Review

Aerial photographs of the site dating from 1940 to 2023 have been accessed from Auckland Council GeoMaps, Retrolens, Nearmaps and Google Earth Pro and reviewed under the context of understanding past site use and to identify evidence for historical landform modifications. Table 1 provides a summary of our review findings. Reviewed aerial photographs are attached in Appendix 3.

Table 1: Summary of Historical Aerial Photographs

Date	Description
1940	The site and surrounding area comprise agricultural land. The site itself appears to be used for grazing. A small structure is present in the northwest portion of the site. A residential development is present to the southeast of the site.
1950	The small structure in the north of the site appears to have been demolished; a lighter area at this location may represent demolition debris. No other significant changes to the site are noted.
1963	A shelterbelt or fence divides the northern and southern portions of the site; stock are grazing in the north. Earthworks are observed on land directly east of the site.

Date	Description
1980	A minor area of bare ground is observed on the southwest boundary. Two linear features are observed on the neighbouring site to the east, where earthworks were previously observed.
1988	Shelterbelts have been planted throughout the site, separating grazing areas. The southern portion of the site appears to be subject to horticultural activity. The octagonal shaped dwelling referenced in property file documents is observed in the northwest corner of site. A tributary of the Waitemata Harbour intersects the site at the centre of the western boundary and runs along the western boundary. This area is demarcated by shelterbelts and darker vegetation. Directly to the west of the site, a residential building and ancillary buildings have been constructed, which are still present today.
1996	The shelterbelts throughout the site have been cleared, and the land appears to be pasture. A large shed had been constructed to the south of the dwelling, along the western boundary. A small building is present to the east of the dwelling, the location of a pump shed identified in property file documentation. Additional structures have been constructed on properties to the east, south and west of the site.
2000	The majority of the site had been converted into vineyard. The small building in the northern portion of the site has been relocated or demolished. Minor earthworks appear to have occurred to the east of the dwelling. No significant changes to the surrounding land.
2004	The octagonal shaped dwelling in the north has been relocated approximately 80 meters south, with a new structure taking its place. No significant changes to the surrounding area are observed.
2017	A circular feature is observed near the western boundary, in the southern portion of the site. No significant changes to the surrounding area are observed.
2019	A small mound is observed at the centre of the western portion of the site. No significant changes to the surrounding area are observed.
2020	The mound has been removed. Not other significant changes are observed.
2021	The vineyards have been cleared and the mound at the northern end appears to comprise of vegetation. No significant changes to the surrounding area are observed.
2023	No significant changes to the site or surrounding area.

No significant earthworks, large-scale movement and / or geomorphological changes to the landform were observed during our review of historical aerials.

4.4 New Zealand Geotechnical Database

We have reviewed the New Zealand Geotechnical Database (NZGD) as part of our desktop study. A third-party geotechnical investigation was carried out approximately 250 m east of the site as part of the Ockleston Landing Development project by Geotek Services Limited during 2014 and comprised eighteen hand auger boreholes.

Hand augers were drilled to depths between 3.0 and 5.0 m bgl on a site that resembled a similar geomorphological landform at similar elevations to 15 Clarks Lane. Hand auger logs generally interpreted topsoil underlain by 0.0 to 0.6 m of fill and Puketoka Formation alluvial clays and silts with varying sand contents between 0.3 and 5.0 m bgl, with measured shear strengths between 55 and 138+ kPa.

5 Site Investigation

5.1 Site Walkover

ENGEO visited site on 17 August 2023 to complete a site walkover, assess current site conditions and identify evidence of potential geohazards. During this site walkover, we made the following observations:

- The site sloped generally into the central overland flow path and to the northwest, into the Waiarohia inlet. The south-western corner of the site slopes from northeast to the southwest and inlets to a shallow gully system.
- The overland flow path in the south-eastern portion of the site flows from northeast to southwest. The ground in the vicinity of the flow path was observed to be soft and had minor ponding.
- The northern most dwelling has been constructed on the downslope side of the northwest aspect slope. The change in elevation in this area is characterised by a retaining wall, a driveway and gently sloping landscaped area.
- The majority of site is grassed horticultural land with shelter belts along the southwest, northeast, and southeast site boundaries.
- The coastal margin is connected to site via a ~2.0 m wide boat ramp to the northwest of the site and the rest is through terraced landscaped areas.

Figure 2: Site Photos

Photo 1: Looking to the northeast up the overland flow path in the south-eastern portion of the site.

Photo 2: Looking to the southwest down the overland flow path in the south-eastern portion of the site.

Photo 3: Image of the steepened slope in the northern portion of the site.

Photo 4: Image looking north across the site.

5.2 Subsurface Investigations

ENGEO attended site on 17 August 2023 to complete a subsurface investigation. This investigation comprised eight hand auger boreholes, HA01 through HA08, drilled to depths of between 3.9 and 5.0 m bgl across the site. Two CPTs, CPT01 and CPT02 were carried out until practical refusal on hard ground at depths of 15.54 m and 13.96 m, respectively. Test locations are shown on the Investigation Location Plan in Appendix 1.

All hand auger boreholes were drilled to target depths of 5.0 m bgl except HA05 and HA06. HA05 was terminated at 3.9 m bgl due to practical refusal through hole collapse. HA06 reached practical refusal on hard material at 3.9 m bgl. Full hand auger borehole logs are presented in Appendix 4. Logs have been prepared in general accordance with the New Zealand Geotechnical Society Guideline for the field classification and description of soil and rock for engineering purposes (NZGS, 2005).

5.3 Investigation Findings

Ground conditions encountered across the site are summarised as follows:

- Surficial topsoil was encountered to depths of up to 0.3 m bgl across the site within all hand auger borehole locations except HA02. Buried topsoil was encountered in HA02 (adjacent to the existing dwelling), from 0.15 to 0.9 m bgl underlying 0.15 m of fill material. Buried topsoil was also encountered within HA06 (drilled in the central overland flow path) from 1.1 to 2.0 m bgl underlying 1.0 m of fill and 0.1 m of surficial topsoil.

- Native Puketoka Formation soils were encountered below the topsoil, at all borehole locations. These soils were observed to comprise clays and silts and amorphous, plastic peat with variable sand and organic content. These stiff to hard soils returned shear strengths between 52 and 220+ kPa and presented variations in plasticity.

- Native East Coast Bays Formation soils were encountered below Puketoka Formation soils within boreholes HA01 to HA04 and HA06. These soils were observed to comprise clays, silts and sands. These stiff to hard soils returned shear strengths between 72 and 220+ kPa and presented variations in plasticity.

A geological cross section has been drawn along the line of Section AA’ as shown on the Site Plan in Appendix 1. The geological section is presented in Appendix 7.

5.3.1 Groundwater

Groundwater was measured at various levels between 1.8 and 4.8 m bgl when the boreholes were dipped at the conclusion of the drilling. Groundwater was not encountered within borehole HA08 though soils were logged as wet form 4.0 m depth. These levels should be considered indicative only as they were recorded on the day of drilling and may not represent longer term levels.

5.4 Laboratory Testing

A soil sample was collected from borehole HA02 for Atterberg Limits and Linear Shrinkage testing. This testing was undertaken in accordance with NZS4402:1986. Full results can be found in Appendix 5 and are summarised in Table 2.

Table 2: Laboratory Test Results Summary

Sample ID	Sample Depth (m)	Water Content	Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage (%)
HA02	0.50 – 1.00	23.6	42	22	20	12

Expansive soils are classified in NZS 3604 as soils with a liquid limit of greater than 50% and a linear shrinkage greater than 15%.

6 Geohazard and Geotechnical Assessment

6.1 Soil Classification

Based on the findings of our desktop and subsurface investigation, as well as our experience of regional ground conditions, we consider the preliminary seismic site classification to be ‘Class C – Shallow Soil Sites’ in line with NZS 1170.5:2004 for the purpose of seismic design.

6.2 Seismic Hazards

Potential seismic hazards resulting from nearby moderate to major earthquakes can be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, ground lurching, regional subsidence or uplift, soil liquefaction, lateral spreading, landslides, tsunamis, flooding, or seiches. Based on topographic and lithologic data, risk from earthquake-induced regional subsidence / uplift, ground lurching, and seiches are considered negligible at the site. The following sections present a discussion of ground rupture, liquefaction risk, and other geohazards as they apply to the site.

6.2.1 Ground Rupture

There are no known active faults located within the site. Based on regional mapping, and the results of our field observations, it is our opinion that fault-related ground rupture is unlikely at the subject property.

6.2.2 Liquefaction Analysis

Liquefaction analysis was undertaken utilising on-site CPT investigations to assess if the ground conditions on-site are susceptible to liquefaction induced settlement.

Soil liquefaction and lateral spread result from loss of strength during cyclic loading, such as those imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded sands. Empirical evidence indicates that loose to medium dense gravels, silty sands, low-plasticity silts, and some low-plasticity clays are also potentially liquefiable, with the more clayey soils more likely to experience softening rather than liquefaction.

Liquefaction Methodology

Peak horizontal ground accelerations (amax) in accordance with New Zealand Geotechnical Society (NZGS) Earthquake Geotechnical Engineering Practice Module 1, Appendix A1 (2021) are 0.19 g (ULS) and 0.05 g (SLS) for a 6.5 magnitude earthquake.

We have carried out liquefaction analysis using the data from the on-site CPTs (Appendix 4) in accordance with NZGS Module 1, Appendix A. We assessed for both ULS and SLS using peak horizontal ground accelerations and groundwater levels of between 1.7 m and 2.0 m depth. For this assessment, we assessed existing ground levels and have not accounted for future earthworks.

The liquefaction potential assessments have been carried out with computer software (GeoLogismiki, CLiq v.2.3.1.15) using Boulanger & Idriss (2014) methodology for liquefaction triggering, and detailed results are included in Appendix 4.

Liquefaction Discussion

The liquefaction assessment results indicate that the site soils are not susceptible to liquefaction under SLS accelerations. Calculated seismically induced settlements and lateral spreading were below 10 mm under ULS seismic conditions.

The MBIE/ NZGS module for identification, assessment and mitigation of liquefaction hazards indicates the ULS event liquefaction induced settlements on this site are expected to fall within the insignificant category (L0). The consequences are described as “no significant excess pore water pressures (no liquefaction)”.

6.3 Settlement

Deposits of the Puketoka Formation were found to locally contain thin horizons of peat and organic clay up to 0.7 m thick. The Puketoka Formation comprises alluvial sediments; in alluvial environments, peat forms in areas with low sediment input, typically on the margins on small, slow flowing channels. These become buried beneath sediment as the channel migrates subsequently forming a peat containing paleo-channel. Based on the spacing of our investigations the presence of one or more paleo-channels on-site cannot be ruled out and the maximum potential depth of peat on-site may not have been encountered in our boreholes.

Peat is considered an unacceptable bearing stratum for foundations as it is highly susceptible to consolidation due to its high water content (peat may contain ten times its own weight in water). Under the load of fill and building foundations, peat can reduce its volume by up to 75% resulting in significant vertical settlement. Peat is also vulnerable to wasting where it is found above the groundwater table as oxidation of the biomass results in the peat decaying / decomposing. Primary settlement of peat may take days whereas secondary creep consolidation settlement behaviour due to the decay of organic material may continue over 50+ years.

Additional investigations should be undertaken prior to Building Consent in order to better characterise the extent of the peat on-site and, where peat is located below proposed structures, carry out detailed settlement analysis. Where settlements caused by peat are found to be beyond building code tolerances; suitable solutions may include undercutting and replacing the peat with engineered fill or piled foundations extending below the peat.

6.4 Expansive Soils

Expansive soils shrink and swell as a result of seasonal fluctuation in moisture content. This can cause heaving and cracking of on-grade slabs, pavements, and structures founded on shallow foundations.

Building damage due to volume changes associated with expansive soils can be reduced through proper foundation design. Successful performance of structures on expansive soils requires special attention during design and construction. It is imperative that exposed soils be kept moist prior to placement of concrete for foundation construction. It is extremely difficult to re-moisturise clayey soils without excavation, moisture conditioning, and re-compaction.

Although our laboratory test results indicate non expansive soils, based on our experience with similar soils within the Hobsonville area we consider a preliminary soil classification of M (moderately) expansive with respect to NZS 3604 (from Section 3.2 of B1/AS1 November 2019 Amendment) is suitable for this site.

It is considered that this preliminary recommendation may be refined with further site-specific testing at the Geotechnical Completion Report stage, following earthworks.

6.5 Coastal Regression Hazard

The northern boundary of the site has been identified by Auckland Council as being potentially susceptible to coastal instability and erosion. The potential regression lines for 2050, 2080 and 2130 are mapped within the proposed council esplanade area and are shown in Figure 1. As such, a site-specific coastal hazard assessment undertaken by a Coastal Engineer will be required to support a Resource Consent application.

6.6 Flooding Hazard

As mentioned in Section 4.2.2, a Flood Plain and Flood Prone Area has been identified along the western boundary in the southern portion of the site. This should be considered by the Civil Engineer during the design phase to mitigate this hazard.

6.7 Slope Stability

Stability of the proposed site topography has been assessed as per the following sections.

6.7.1 Soil Parameters

The soil parameters given in Table 3 have been assigned to the geological units identified earlier in this report and used in slope stability analysis. These parameters are derived from the *in situ* soil strength testing, published effective stress parameters used for similar soils at other sites and our experience with these soil units.

Table 3: Slope Stability Parameters

Geological Unit	Unit Weight (kN/m^3)	Effective Stress Parameters		Total Stress Parameters
		Ø' (°)	c (kPa)	Su (kPa)
Engineered Fill (Cohesive)	18	32	5	100
Non-Engineered Fill (Cohesive)	18	28	2	50
Puketoka Formation	18	28	3	80
East Coast Bays Formation residual soils	18	32	5	n/a
East Coast Bays Formation Rock	20	40	10	n/a

6.7.2 Analysis Methodology

Numerical slope stability analyses were conducted using the software package SLIDE2, produced by Rocscience Limited.

We considered the existing unsupported slope geometry and ground conditions identified on-site using the GLE / Morgenstern Price method.

Based on our observation of local slope failures, circular analysis was undertaken under proposed development conditions along two critical cross sections (B-B' & C-C'). Future development loads of 20 kPa and 12 kPa have been applied across the proposed building platforms and road reserves, respectively.

Three conditions were considered to assess the final stability of the slope:

- Normal condition - measured groundwater levels.
- Transient condition – with elevated ‘worst credible’ groundwater profile.
  - A conservative groundwater water elevation approximately 1.0 m higher than measured (noting this followed a wet period) has been assumed for the transient slope stability modelling case.
- Seismic condition ULS – a seismic coefficient of 0.19 was used to model the behaviour of the slope during a 1 in 500-year seismic event. This seismic coefficient has been derived from NZS1170 and MBIE/NZGS Module 1 (2021).

The Factor of Safety (FoS) is a ratio of the forces resisting failure to the forces driving the slope toward failure.

Factor of Safety = Resisting Forces / Driving Forces

A FoS in excess of 1.0 is considered to be stable, while a FoS of less than one is considered unstable. Factor of safety criteria have been adopted from Auckland Code of Practice for Land Development and Subdivision document, dated July 2022 (Version 2.0). Table 4 includes the FoS required for residential development in Auckland.

Table 4: Required Factors of Safety

Residential Subdivision / Development	
Building Platform Area	
Conditions	**Factor of Safety Required**
Long Term groundwater condition	1.5
Transient groundwater condition	1.3
Seismic condition - in 500-year return period event	1.0

6.7.3 Assessment of Proposed Slope Stability

In order to assess the stability of the proposed landforms, two cross section profiles were generated through the critical slopes across the site (B-B’ & C-C’). These sections were chosen as they include previously identified prevalent geomorphological features, extend down the primary slopes throughout the site and are representative of the worst-case proposed post development contours i.e., these sections extend through locations where additional fill is proposed to create desired finished levels. The locations of the cross sections are shown on the Investigation Location Plan in Appendix 1.

Results of analysis of the proposed slopes are summarised in Table 5. Analysis output sheets for all scenarios are presented in Appendix 6.

Table 5: Summary of Stability Analyses for Existing Slopes

Section	Scenario	Calculated Minimum FOS within Lot Boundaries
Section BB’	Static	2.02
	Transient	1.49
	Seismic	1.49
Section CC’	Static	2.66
	Transient	2.21
	Seismic	1.77

The analysis detailed in Appendix 6 and summarised in Table 5 indicates that under post-development conditions, minimum slope stability factors of safety (FoS) within the lot boundaries are expected to achieve the FoS required by Auckland Council (Table 4). However, we note that this analysis should be revisited once more detailed earthworks proposals have been developed for the site.

Further, this analysis demonstrates that the proposed development does not negatively impact the existing slope stability along the critical cross section alignments.

The slope stability analyses are limited by the assumptions used in developing the ground model. This includes the conservative soil parameters due to the absence of site-specific laboratory testing and assumed worst-credible groundwater conditions.

6.8 RMA Section 106 Assessment and Development Suitability

Section 106 of the Resource Management Act (RMA) states that a consent authority may refuse to grant a Subdivision Consent, or may grant a consent subject to specific consent conditions if it considers that:

- There is significant risk from natural hazards; or
- Sufficient provision has not been made for legal or physical access to each allotment to be created by the subdivision.

An assessment of the risk from natural hazards as required by the RMA includes the following:

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

We have assessed the risk of natural hazards at the site in accordance with Section 106 of the Resource Management Act (RMA) and considered the risk to the site from erosion, rockfall, inundation (debris), slope stability, subsidence, flooding and tsunami.

Based on our investigation, assessment and site observations, we consider it is unlikely for the site to be subject to the aforementioned natural hazards providing suitable engineering measures are included in the site development (as discussed in Section 7). As such, the site is considered to be conditionally suitable for the proposed residential development from a geotechnical perspective.

7 Geotechnical Recommendations

Based on the results of our geotechnical investigation and subsequent assessment, we consider the site to be generally suitable for the proposed development subject to our geotechnical recommendations being followed.

However, as mentioned in Section 6 the site is at risk from a number of identified geohazards including the following:

- Instability of the over steepened north-western slope bordering Waiarohia Inlet.
- Portions of the site may be vulnerable to settlement due to the potential presence of compressible alluvial soils.
- Shallow site soils are moderately expansive and may be susceptible to shrinkage and heave.

7.1 Foundations

Based on the draft masterplan provided it is likely that building foundations are likely to bear within stiff to hard silts and clays of the Puketoka Formation or East Coast Bays Formation. We consider these deposits to be generally suitable as a foundation subgrade.

Notwithstanding the above, where the Puketoka Formation was found to contain layers of peat, shallow foundations may be vulnerable to intolerable differential settlement as a result of long term consolidation and wasting of the peat. Where peat soils are identified within finished lots these will require undercutting and replacement with engineering fill or alternatively the use of piled foundations.

It is our preliminary recommendation that the site soils following earthworks will likely be suitable for a geotechnical ultimate bearing capacity of 300 kPa for shallow foundations constructed on identified competent natural ground beneath any topsoil and existing non-engineered fill or on engineer certified fill.

This preliminary recommendation will be revisited in the geotechnical completion report to be issued for the site following the satisfactory completion of the proposed earthworks.

It is considered likely that the soils on-site may be M (moderately) expansive with respect to NZS 3604 (from Section 3.2 of B1/AS1 November 2019 Amendment). This will be reassessed as part of the completion reporting for this site.

7.2 Earthworks

7.2.1 General

- All topsoil and pre-existing fill shall be removed from any building platforms or areas to receive fill.

- Earthworks will be carried out in Puketoka Formation alluvial soils. There is potential to encounter flowable sands, organic soils / peat while carrying out cuts for building platforms, road cuttings, service lines and general earthworks. Depending on the thickness of these soils, shallow undercuts will be required to remove the flowable sands / organic material in full, or partial undercuts and replacement with suitably compacted fills up to finish level will be required.

- Excavations and temporary cuts should not exceed a batter angle of 1V:2H up to 2 m in height and should not be left unsupported for longer than two weeks. Cuts beyond this height should be referred to the Geotechnical Engineer for stability assessment.

- Where vertical and subvertical faces higher than 1.0 m are required, we recommend that this is done in shortened sections (< 5 m) and the faces are left unsupported for a minimal time period (i.e., one week) or temporarily shored.

- All temporary cuts and batters proximal to boundaries should take into account the potential surcharge and risk of undermining neighbouring property.

- Suitable drainage channels must be put in place to divert surface water from unsupported cut faces. Subsurface drains should also be considered for the toe of the long-term slopes.

- If any permanent cuts have a batter steeper than 1V:4H and are to be higher than 1.5 m, they should be supported with a specifically designed retaining wall (approved by a chartered Geotechnical Engineer) or be referred back to the Geotechnical Engineer for stability assessment and specific batter design.

- All cuts and batters should be in line with the WorkSafe Good Practice Guidelines for Excavation Safety (July 2016). Permanent fill batters should not exceed 1V:3H and should be reviewed by the Geotechnical Engineer as part of the site development and earthworks proposal review. Fill batters exceeding 1V:3H will require specific geotechnical assessment.

- All excavations should be inspected by ENGEO (or a suitably qualified Geotechnical professional), prior to constructing foundation elements to verify founding conditions are as anticipated.

- Suitable underfill drainage should be considered for any filling on slopes, within stream gully features and wherever seepage is observed within the stripped surface.

- All engineered or structural fill should be placed in ≤ 200 mm compacted lifts and be compacted to a minimum of 95% of maximum dry density, at no less than optimum moisture content. Maximum dry density for granular fill materials may be obtained from the source quarry, a geotechnical laboratory or from plateau testing undertaken on-site. Compaction should be achieved using standard plant and methodology suitable for the imported material. A water source should be maintained on-site for moisture control.

- All excavated soil should be removed from site or placed in an engineer approved stockpile to avoid unfavorable loading on construction or preconstruction slope batters.

7.2.2 Material Suitability

With the exception of topsoil, peat and organic clay, we consider the site won native soils to be suitable for reuse as compacted engineered fill provided that appropriate moisture content be maintained.

Moisture contents will increase with depth in the cut areas and are likely to be higher in lower lying areas. Material conditioning and compaction can likely be achieved with standard earthworks machinery.

Our experience with the types of native soils present on this site indicates that when they are exposed to the weather their strengths may be significantly reduced. We therefore recommend that trafficked areas and building platforms are only trimmed to final levels immediately prior to placing hardfill / topsoil and that at all times the site is shaped to avoid water ponding during rain, thereby limiting the need for additional undercuts. On no account should areas of trimmed subgrade be left exposed to allow the ingress of water, nor should subgrade areas be trafficked prior to drying out after rain.

7.2.3 Unsuitable Materials

Topsoil and organic soils are not suitable for bearing foundations or for reworking and re-use as engineered fill and should be undercut and stockpiled away from the earthworks area.

Buried topsoil and pre-existing fill were identified around the existing house in the northeast corner of the site and within the southern overland flow path. Given the rural setting of the site and the historic / current land use it is possible that further areas of pre-existing fills, burn pits and buried topsoil are present which may not have been identified as part of our investigation. Where encountered during subdivision development works, any pre-existing fill will need to be inspected by ENGEO to assess its suitability to remain in place, or to be used on-site in structural fills.

It is unlikely that any existing fill at the site will have been placed to an Engineer Certified standard, (given the historical land use as pasture) and accordingly the requirement to undercut any fill to expose the underlying natural ground should be allowed for in the development scope.

Provided that the identified fill is inorganic and free of any deleterious inclusions it may be suitable, with conditioning, for placement as structural fills as part of the development works. However, if the material is considered to be unsuitable for use it will need to be cut to waste.

7.3 Service Lines

The construction and installation of new services lines within alluvial material may intercept flowable sands and organic / peat layers. Particular attention should be paid to drainage and stability of trench walls under such circumstances.

Where the base of service line trenches encounters weak, flowable sands and / or organic soils, increased bedding depths of up to 70% and undercuts of approximately 300 mm plus geotextile wrapping of the bedding may be required to provide adequate support to the services and limit the chance of differential settlement along low gradient service alignment. Specific bedding modifications are best prescribed when the trenches are excavated and the material at invert level are examined in detail by a geotechnical professional.

Construction of services during the winter months may pose a risk of trench wall collapse within soft alluvial soils partly due to raised groundwater, leading to the need for additional support, alternative construction methodology and / or dewatering. This should be allowed for on-site by the contractors. Methods to deal with this could be, but not limited to, trench shields to support service trench walls, benching or excavations to a safe temporary works angle (e.g., 1):H): 1(V)).

Should flowable sands and / or organic soil layers be encountered during service line trenching, the contractor shall contact ENGEO.

7.4 Soakage

Based on the presence of near surface alluvial silt and clay material (Section 5.3), we consider that soil infiltration rates at the site will be poor (i.e., less than 2 mm per hour). This should be verified by site-specific soakage testing at the detailed design stage.

7.5 Retaining Walls

Currently, there are no retaining structures explicitly shown on the development plans. Any future retaining should be designed to accommodate for the soils encountered on-site. Based on our subsurface investigations, we expect the proposed retaining structures will generally support native Puketoka Formation or East Coast Bays Formation.

7.5.1 Preliminary Retaining Wall Parameters

Based on the results of our investigation and ground conditions at the site, future retaining walls should be designed using the following geotechnical parameters:

Table 6: Soil Parameters for Retaining Wall Design

Material Type	Unit Weight	Friction Angle (°)	Effective Cohesionc' (kPa)	Undrained Shear Strength Su (kPa)
Puketoka Formation (Stiff to very stiff)	18	28	3	80
East Coast Bays Formation (residual soil)	18	32	5	100
Cohesive Engineered Fill	18	32	5	100
Granular Engineered Fill	20	38	0	-

Retaining wall design should include appropriate drainage which must outlet into an approved stormwater disposal system.

We recommend that design of retaining walls should be carried out in line with Module 6 of the Ministry of Business, Innovation and Employment Guidance.

7.6 Surface Water Management

During construction, appropriate measures shall be undertaken to control and treat stormwater runoff, with silt and erosion controls complying with Auckland Council Guidance for Erosion & Sediment Control (GD05).

This is particularly relevant for the site due to the proximity to a stormwater receptor, being the inlet to the north. Surface cut-off drains or appropriate stormwater flow paths should be maintained outside of the proposed development area, both during and following construction. These drains and impervious surfaces will divert water away from any buildings and minimise possible movement in sensitive soils during and post construction.

Stormwater from paved areas shall be taken in a piped system and disposed of into an approved stormwater system. Uncontrolled discharge onto land or uncontrolled disposal via in-ground systems must be avoided.

All service trenches should be capped with low permeability materials, so that excavations do not become points of entry for surface run-off.

7.7 Pavement Subgrade CBR

Inferred CBRs of approximately 3% may be adopted for native soils and 6% for cohesive engineered fill areas are considered to be suitable for preliminary design purposes. These values are derived from the soils encountered in our hand auger boreholes and our knowledge of the soil type on-site.

It should be noted that actual CBR values can be highly affected by moisture content (i.e., exposure to the elements) and trafficking.

A programme of CBR testing should be carried out on the stripped subgrade level within roading corridors to confirm actual values.

8 Future Work

ENGEO should be given the opportunity to review detailed earthworks and development design drawings (walls, structures, etc.) for the development to confirm that the recommendations in this report have been interpreted as intended. If changes are made to the plans cited within this report, we reserve the right to revisit and modify our recommendations when updated plans are made available.

ENGEO should be engaged to undertake the following future works required for this site:

- Detailed review of the final earthworks design if revised from that assessed herein, and if necessary, undertake supplementary investigation to verify ground conditions in the vicinity of proposed deep cuts or significant fills in the context of slope instability and potentially compressible soils, and undertake revised slope stability analysis if required.

- Preparation of a Geotechnical earthworks specification.

- Observation and certification of earthworks and retaining walls including all stripping and undercuts and engineered fill in accordance with the earthworks and retaining wall specifications.

- Geotechnical Completion Reporting / Producer Statements.

9 Limitations

i. We have prepared this report in accordance with the brief as provided. This report has been prepared for the use of our client, Cabra Developments Limited, their professional advisers and the relevant Territorial Authorities in relation to the specified project brief described in this report. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity.

ii. The recommendations in this report are based on the ground conditions indicated from published sources, site assessments and subsurface investigations described in this report based on accepted normal methods of site investigations. Only a limited amount of information has been collected to meet the specific 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 judgement and it should be appreciated that actual conditions could vary from the assumed model.

iii. 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.

iv. This Limitation should be read in conjunction with the Engineering NZ/ACENZ Standard Terms of Engagement.

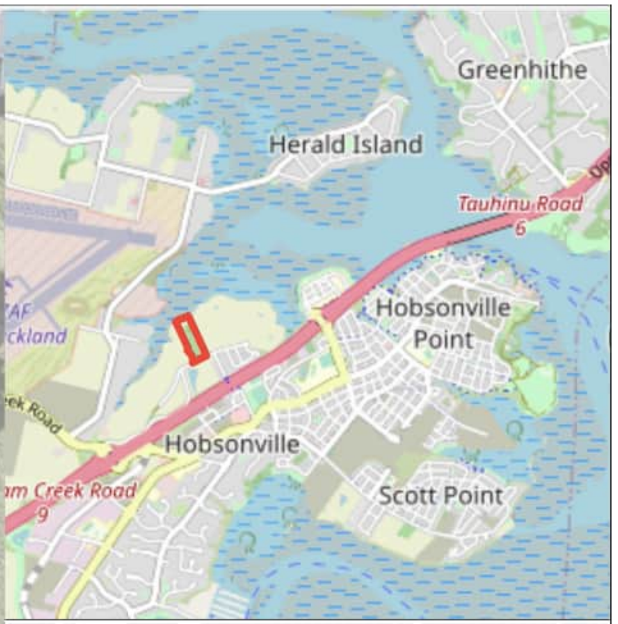
v. This report is not to be reproduced either wholly or in part without our prior written permission.

We trust that this information meets your current requirements. Please do not hesitate to contact the undersigned on (09) 972 2205 if you require any further information.

Report prepared by	Report reviewed by
Jamie Lott Engineering Geologist	**Paul Fletcher, CMEngNZ (CPEng)** Principal Geotechnical Engineer
Alex Keijzer Geotechnical Engineer	**Heather Lyons, CMEngNZ (PEngGeol)** Associate Engineering Geologist
Jamie Thomas Geotechnical Engineer	

APPENDIX 1:

Investigation Location Plan



Legend

- ⊕ ENGEO Hand Auger Boreholes 2023
- ▲ ENGEO CPT 2023
- — Cross Section Alignment
- — Overland Flow Path
- 🟥 Site Boundary
- — Contours

0 25 m 50 m

LINZ CC BY 4.0 © Imagery Basemap contributors, , public

ENGEO

Produced by **Datanest.earth**

Title: Investigation Location Plan

Client:
Cabra Developments Limited

Project:
15 Clarks Lane,
Whenuapai

Drawn: RD

Appendix: 1a
Size: A3

Date: 26-10-2023

Checked: JC

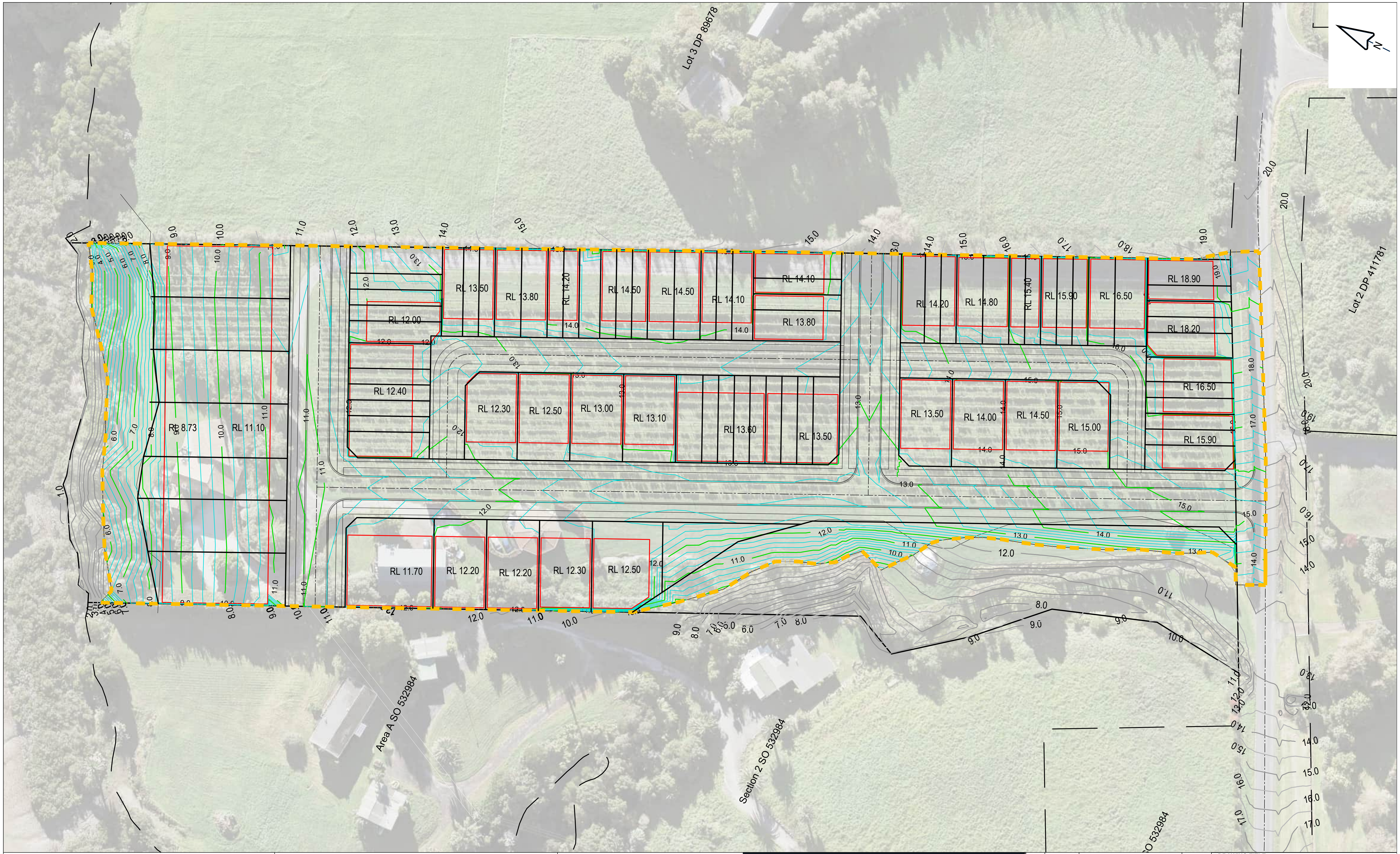
Proj No.:
23849.000.001_02

Scale: 1:1500

Version: 01

APPENDIX 2:

Earthworks and Development Plans



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CLIENT

CABRA
LAND & PROPERTY DEVELOPMENT

PROJECT

SITE A
15 CLARKS LANE
WHENUAPAI

CAPTURE
Land Development Consultants

REV	DATE	REVISION DETAILS	ISSUED
A	DATE	DRAFT	DL

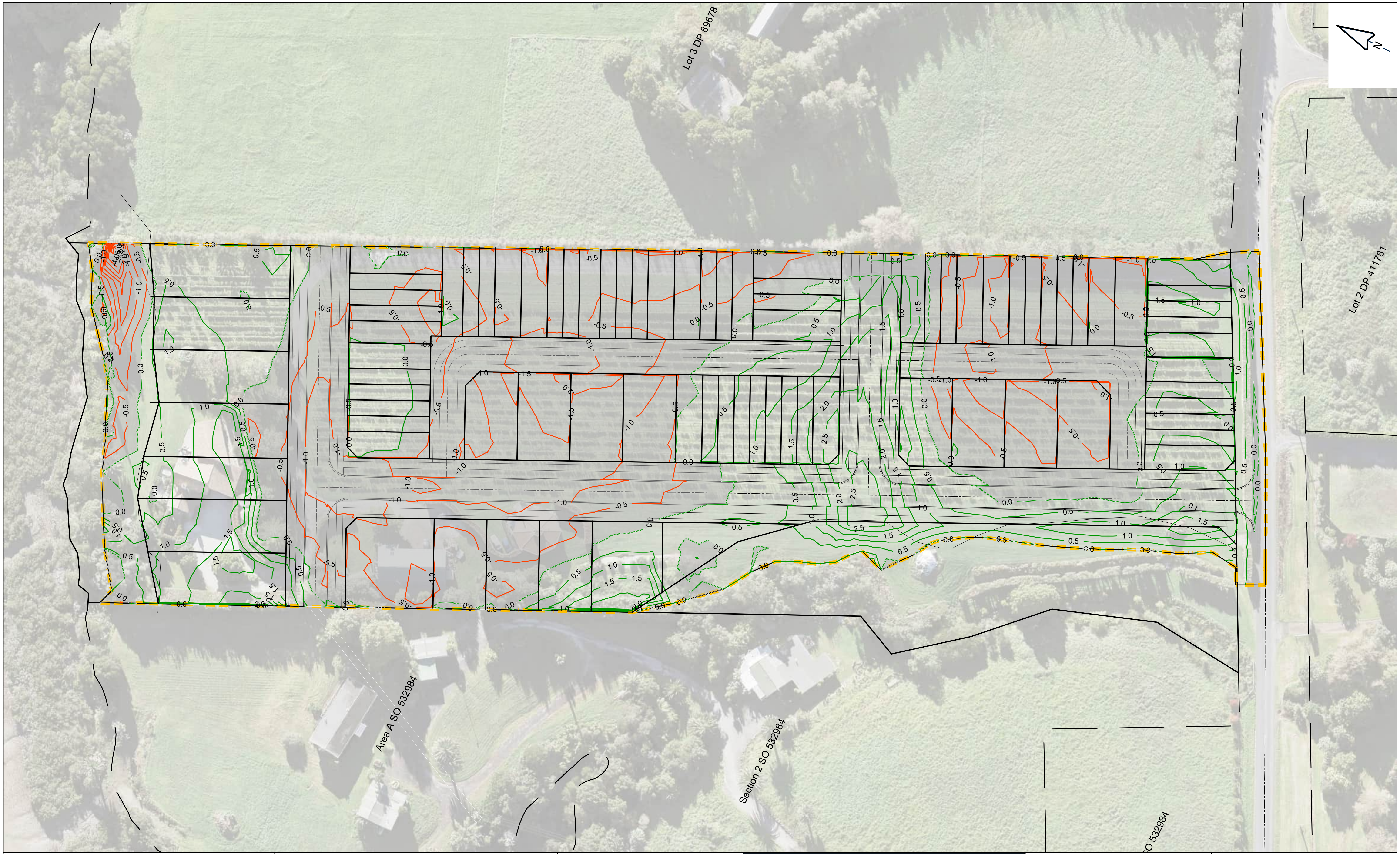
DRAWING TITLE

SITE A
PROPOSED CONTOURS

STATUS	*SCALE*	*SIZE*
DRAFT	1:1000	A3

PROJECT NO	*DRAWING NO*	*REVISION*
1100	CL EN-101	A

DRAFT



CLIENT

CABRA
LAND & PROPERTY DEVELOPMENT

PROJECT

SITE A
15 CLARKS LANE
WHENUAPAI

CAPTURE
Land Development Consultants

REV	DATE	REVISION DETAILS	ISSUED
A	DATE	DRAFT	DL

DRAWING TITLE

SITE A
CUT & FILL CONTOURS

STATUS	***SCALE***	***SIZE***
DRAFT	1:1000	A3

PROJECT NO	***DRAWING NO***	***REVISION***
1100	CL EN 110	A

DRAFT

APPENDIX 3:

Historical Aerial Photographs

1940 (Retrolens NZ)

1950 (Retrolens NZ)

1963 (Retrolens NZ)

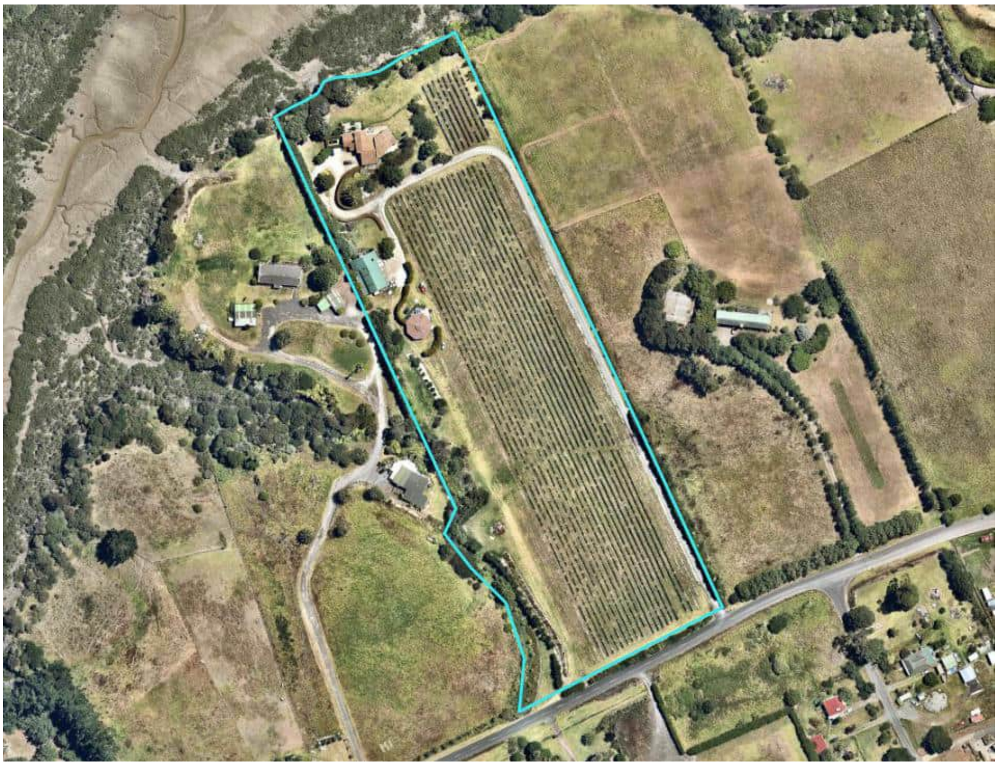
1980 (Retrolens NZ)

1988 (Retrolens NZ)

1996 (Auckland Council GeoMaps)

2000 (Auckland Council GeoMaps)

2004 (Auckland Council GeoMaps)



2017 (Nearmap)

2019 (Nearmap)

2020 (Nearmap)

2021 (Nearmap)

2023 (Nearmap)

APPENDIX 4:

Borehole logs

ENGEO

LOG OF AUGER HA01

Geotechnical Investigation
15 Clarks Lane
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_02
Date : 17-08-2023
Hole Depth : 5 m
Hole Diameter : 50 mm
Shear Vane No : 2853
Logged By : LM
Reviewed By : RD
Latitude : -36.7919438
Longitude : 174.6436263

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm 2 4 6 8 10 12
	TS	OL	TOPSOIL.					N/A		
	PUKETOKA FORMATION	ML	SILT with some fine sand; yellowish brown and white intermixed. Low plasticity.		13			VSt	111/26	
0.5 1.0 1.5 2.0 2.5		CH	Silty CLAY with trace fine sand; yellow and light grey intermixed. High plasticity. 0.8 m: Becomes light grey with orange and yellow mottles. 1.8 m: Encountered orange red mottles. 2.4 m: Becomes yellow, grey and black intermixed.		12 11		M	VSt	157/72 187/80 174/59 167/90 177/139 125/83 144/86 162/75	
3.0		OL	Organic clayey SILT with minor coarse sand and minor rootlet and decomposing bark inclusions; blackish purple with grey and white mottles. Low plasticity.		10			St - VSt	102/53 65/37	
3.5		CH	Silty CLAY with minor fine sand; yellow and light grey intermixed. High plasticity. 3.6 m: Encountered standing groundwater.			▼		St	72/32	
4.0 4.5 5.0		ML	Clayey SILT with some fine sand; grey with yellow mottles. Low plasticity. Poor recovery.		9		W	VSt	98/53 149/70 149/79 148/82	

End of Hole Depth: 5 m
Termination Condition: Target depth

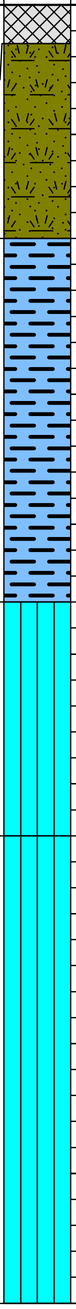
Hand Auger met target depth at 5 m.
Dip test showed standing water at 3.6 m depth.
Elevation and coordinates estimated from Auckland Council GeoMaps.
N/A = Not Applicable; TS = Topsoil

ENGEO

LOG OF AUGER HA02

Geotechnical Investigation
15 Clarks Lane
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_02
Date : 17-08-2023
Hole Depth : 5 m
Hole Diameter : 50 mm
Shear Vane No : 2853
Logged By : LM
Reviewed By : RD
Latitude : -36.7912316
Longitude : 174.6428976



Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm 2 4 6 8 10 12
	F	ML	[FILL] Clayey SILT with minor fine sand and topsoil; yellow with brown and white mottles. Low plasticity.			W	N/A		
0.5	BURIED TOPSOIL	OL	BURIED TOPSOIL. 0.4 to 0.8 m: No Recovery.	7			N/A		
1.0 1.5	PUKETOKA FORMATION	CH	Silty CLAY with some fine sand; light grey. High plasticity. 1.8 m: Encountered standing groundwater. Becomes saturated.	6	▼ (1.8)	W S	VSt	136/55 144/80 159/93 135/67	
2.0 2.5 3.0		ML	Clayey SILT with some fine sand; light grey. Low plasticity.	5		S	St - VSt	98/47 102/46 89/32	
3.5 4.0 4.5 5.0		ML	Fine sandy SILT with some clay; yellow and light grey intermixed. Low plasticity. 3.45 m: Becomes very stiff. 3.5 m: Becomes light grey with yellow mottles. 4.3 m: Becomes grey.	4 3		S	St VSt	92/39 129/47 145/55 113/39 132/42 164/57	

End of Hole Depth: 5 m
Termination Condition: Target depth

Hand Auger met target depth at 5 m.
Dip test showed standing water at 1.8 m depth.
Elevation and coordinates estimated from Auckland Council GeoMaps.
N/A = Not Applicable; F = Fill

LOG OF AUGER HA03

Geotechnical Investigation
15 Clarks Lane
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_02
Date : 17-08-2023
Hole Depth : 5 m
Hole Diameter : 50 mm
Shear Vane No : 3840
Logged By : KE
Reviewed By : RD
Latitude : -36.7910546
Longitude : 174.6435834

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm 2 4 6 8 10 12
	TS	OL	TOPSOIL.			M	N/A		
	PUKETOKA FORMATION	ML	Clayey SILT; light brown. Low plasticity.			M	VSt	129/47	
0.5 1.0 1.5 2.0 2.5		CH	Silty CLAY with minor fine to medium sand; light grey with orange brown streaks. High plasticity. 2.2 m: Encountered standing groundwater. Becomes saturated. 2.7 m: Becomes orange brown with light grey mottles.	8 7 6	▼	M W	VSt	172/96 196/122 143/91 157/60 132/60 140/58 129/53 157/47	
3.0		ML	SILT with some fine to medium sand; light brown with orange brown streaks. Low plasticity.			W	VSt	135/56	
3.5 4.0 4.5 5.0		ML	Fine to medium sandy SILT; light grey to grey. Low plasticity. 4.05 m: Becomes hard. 4.5 m: Poor recovery.	5 4		W	VSt H	149/60 166/60 172/56 200+ 200+	
			End of Hole Depth: 5 m Termination Condition: Target depth						

Hand Auger met target depth at 5 m.
Dip test showed standing water at 2.2 m depth.
Elevation and coordinates estimated from Auckland Council GeoMaps.
N/A = Not Applicable; TS = Topsoil

ENGEO

LOG OF AUGER HA04

Geotechnical Investigation
15 Clarks Lane
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_02
Date : 17-08-2023
Hole Depth : 5 m
Hole Diameter : 50 mm
Shear Vane No : 3840
Logged By : KE
Reviewed By : RD
Latitude : -36.7915381
Longitude : 174.6438657

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm 2 4 6 8 10 12
	TS	OL	TOPSOIL.				W	N/A	53/25	
	PUKETOKA FORMATION	ML	Clayey SILT; light brown. High plasticity.				M	St		
0.5 1.0 1.5 2.0 2.5 3.0		CH	Silty CLAY; light grey with occasional orange brown streaks. Low plasticity. 2.0 m: Becomes wet. 2.2 m - Encountered 100 mm band of fine sandy SILT. 2.5 m: Encountered standing groundwater.		12 11 10	▼	W	VSt - H	141/64 187/111 185/116 180/111 200+ 172/102 113/78 107/50 118/42	
3.5		CH	Silty CLAY with some fine to medium sand; light brown with orange brown streaks. High plasticity.		9			St - VSt	94/45	
4.0 4.5 5.0	EAST COAST BAYS FORMATION	ML	SILT with some fine to medium sand; grey. Low plasticity.		8			VSt - H	151/45 160/64 200+ 188/50 200+	
			End of Hole Depth: 5 m Termination Condition: Target depth							

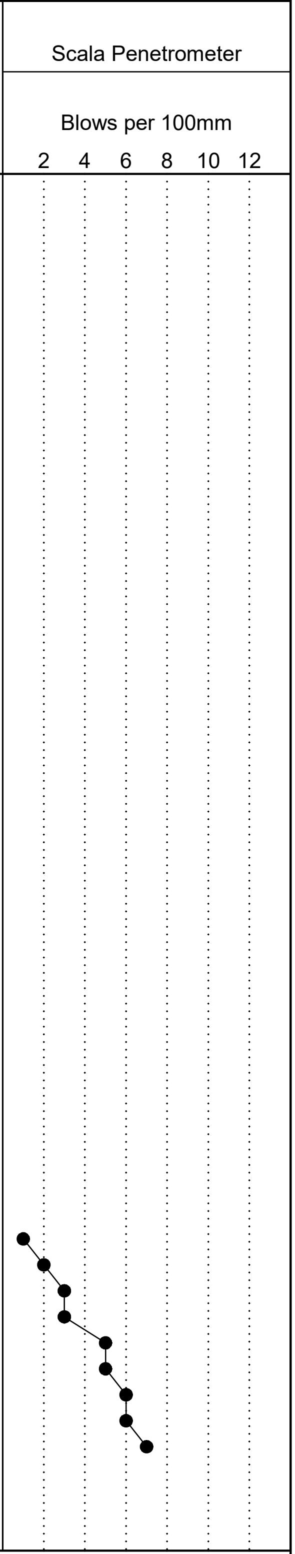
Hand Auger met target depth at 5 m.
Dip test showed standing water at 2.5 m depth.
Elevation and coordinates estimated from Auckland Council GeoMaps.
N/A = Not Applicable; TS = Topsoil

LOG OF AUGER HA05

Geotechnical Investigation
15 Clarks Lane
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_02
Date : 17-08-2023
Hole Depth : 3.9 m
Hole Diameter : 50 mm
Shear Vane No : 3840
Logged By : KE
Reviewed By : RD
Latitude : -36.7920068
Longitude : 174.6442598

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
	TS	OL	TOPSOIL.			W	N/A	
	PUKETOKA FORMATION	CH	Silty CLAY; brown. High plasticity.				VSt	185/55
0.5			0.5 m: Becomes moist. 0.55 m: Becomes orange brown.			M		176/97
								188/110
1.0				14				185/105
1.5			1.3 m: Becomes light grey with orange brown streaks.					176/102
								144/64
2.0		ML	Clayey SILT; light grey with orange brown streaks. Low plasticity.	13		W	VSt	154/63
2.5								157/60
								157/83
3.0				12				144/56
			3.15 m: Becomes stiff.				St	
			3.3 m: Encountered standing groundwater. Becomes saturated.		▼	S		67/36
3.5								53/19
4.0			End of Hole Depth: 3.9 m Termination Condition: Practical refusal					78/24
4.5								
5.0								



Hand Auger met practical refusal at 3.9 m depth due to hole collapse.
Scala Penetrometer met target depth at 4.9 m.
Dip test showed standing water at 3.3 m depth.
Elevation and coordinates estimated from Auckland Council GeoMaps.

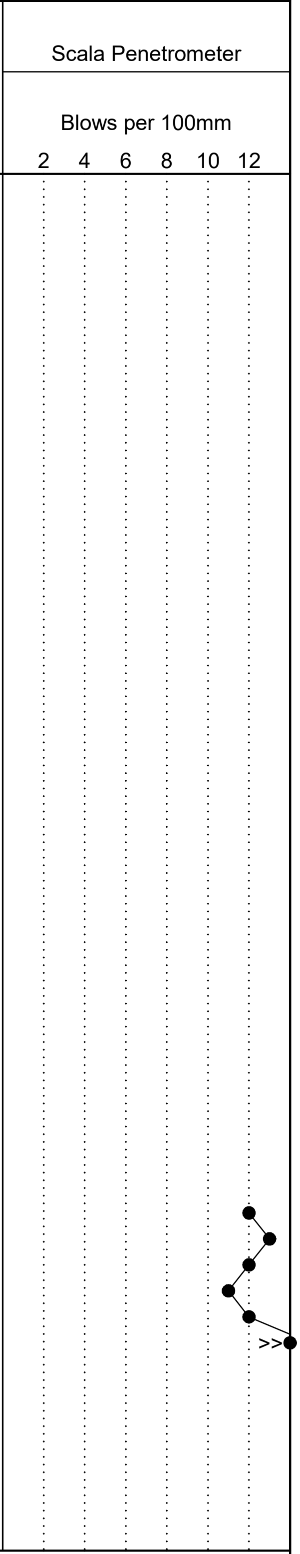
N/A = Not Applicable; TS = Topsoil

LOG OF AUGER HA06

Geotechnical Investigation
15 Clarks Lane
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_02
Date : 17-08-2023
Hole Depth : 4 m
Hole Diameter : 50 mm
Shear Vane No : 2853
Logged By : LM
Reviewed By : RD
Latitude : -36.7926807
Longitude : 174.6442945

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
	T	OL	TOPSOIL.			W	N/A	
0.5 1.0	FILL	ML	[FILL] Clayey SILT with minor fine sand; light grey, white and yellow intermixed. Low plasticity.	11			St - VSt	106/33 73/20 177/55
1.5	BURIED TOPSOIL	OL	BURIED TOPSOIL.	10	▼		N/A	
2.0 2.5	PUKETOKA FORMATION	ML	Clayey SILT with some fine sand; grey with orange yellow streaks. Low plasticity. 2.4 m: Becomes grey with orange mottles.	9		S	St - VSt	76/17 93/29 136/49
3.0 3.5		CH	Silty CLAY interbedded with Clayey SILT with some fine sand; grey with occasional black streaks. High plasticity. Beds are approximately 100mm thick and sub horizontal. 3.15 m: Becomes hard.	8			VSt H	136/47 200+ 200+ 200+
4.0 4.5 5.0			End of Hole Depth: 4 m Termination Condition: Practical refusal					



Hand Auger met practical refusal at 4 m depth on hard material.
Scala Penetrometer met practical refusal at 4.5 m depth.
Dip test showed standing water at 2.0 m depth.
Elevation and coordinates estimated from Auckland Council GeoMaps.

N/A = Not Applicable; T = Topsoil

LOG OF AUGER HA07

Geotechnical Investigation
15 Clarks Lane
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_02
Date : 17-08-2023
Hole Depth : 5 m
Hole Diameter : 50 mm

Shear Vane No : 3840
Logged By : KE
Reviewed By : RD
Latitude : -36.7931942
Longitude : 174.64492

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm 2 4 6 8 10 12
	TS	OL	TOPSOIL.			W	N/A	200+	
0.5	PUKETOKA FORMATION	ML	Fine to coarse sandy SILT; light brown. Low plasticity.				VSt - H	125/39	
1.0 1.5		CH	Silty CLAY; light grey. High plasticity.	16		M	VSt	157/77 169/107 113/63 132/69	
2.0 2.5		PT	Plastic, Amorphous PEAT; black. 2.1 m: Encountered standing groundwater. Becomes saturated.	15	▼	W S	N/A	47/14 69/16	
3.0 3.5 4.0 4.5		CH	CLAY; grey. High plasticity. 2.85 m: Becomes stiff. 2.9 m: Becomes light grey.	14 13			H St	200+ 91/39 97/50 63/41 63/38 53/3 52/31 67/38	
5.0			End of Hole Depth: 5 m Termination Condition: Target depth	12					

Hand Auger met target depth at 5 m.
Dip test showed standing water at 2.1 m depth.
Elevation and coordinates estimated from Auckland Council GeoMaps.
N/A = Not Applicable; TS = Topsoil

LOG OF AUGER HA08

Geotechnical Investigation
15 Clarks Lane
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_02
Date : 17-08-2023
Hole Depth : 5 m
Hole Diameter : 50 mm
Shear Vane No : 2853
Logged By : LM
Reviewed By : RD
Latitude : -36.7932589
Longitude : 174.644209

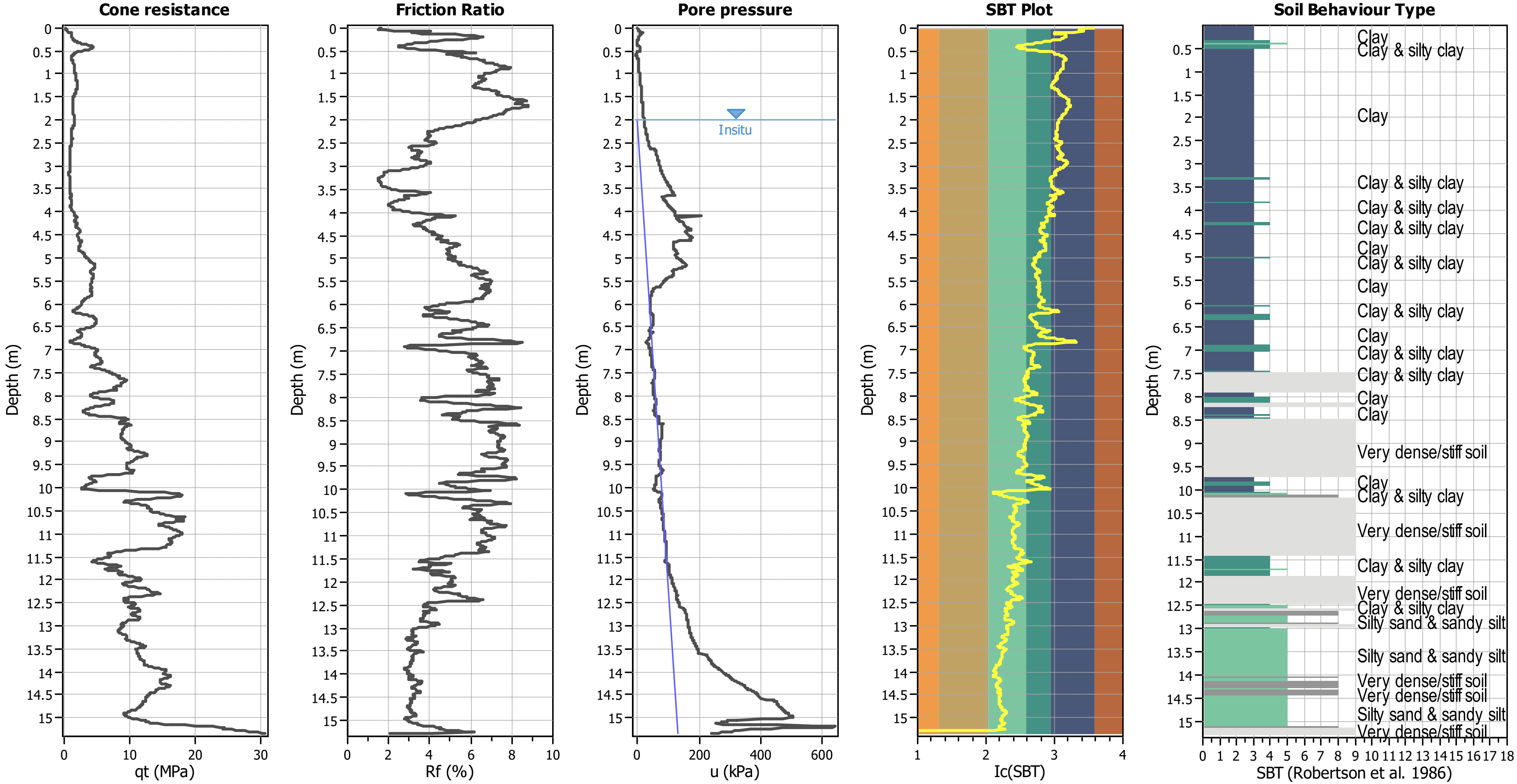
Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index
0.0	TS	OL	TOPSOIL.	13		M	N/A
0.5	PUKETOKA FORMATION	ML	Clayey SILT with trace fine sand; light grey with yellow mottles and black veins. Low plasticity.				VSt
1.0 1.5		CH	Silty CLAY with trace fine sand; light grey with orange yellow mottles. High plasticity.	12			VSt
2.0		ML	Clayey SILT with minor fine to coarse pumiceous sand; light grey with yellow mottles. Low plasticity.	11			VSt
2.5 3.0 3.5 4.0 4.5		ML	Fine sandy SILT with minor clay; light grey. Low plasticity. 3.3 m: Becomes orange yellow with light grey mottles. 3.7 m: Becomes light yellowish brown with light grey and orange yellow mottles. 4.0 m: Becomes wet.	10 9		M W	St - VSt
5.0			End of Hole Depth: 5 m Termination Condition: Target depth				

Shear Vane Undrained Shear Strength (kPa) Peak/Remolded:
161/88
184/56
172/82
118/78
167/60
101/49
115/37
164/47
171/47
139/42
95/43
145/45
171/69
164/63
162/66
152/56

Scala Penetrometer
Blows per 100mm
2 4 6 8 10 12

Hand Auger met target depth at 5 m.
Standing groundwater was not encountered
Elevation and coordinates estimated from Auckland Council GeoMaps.
N/A = Not Applicable; TS = Topsoil

CPT basic interpretation plots



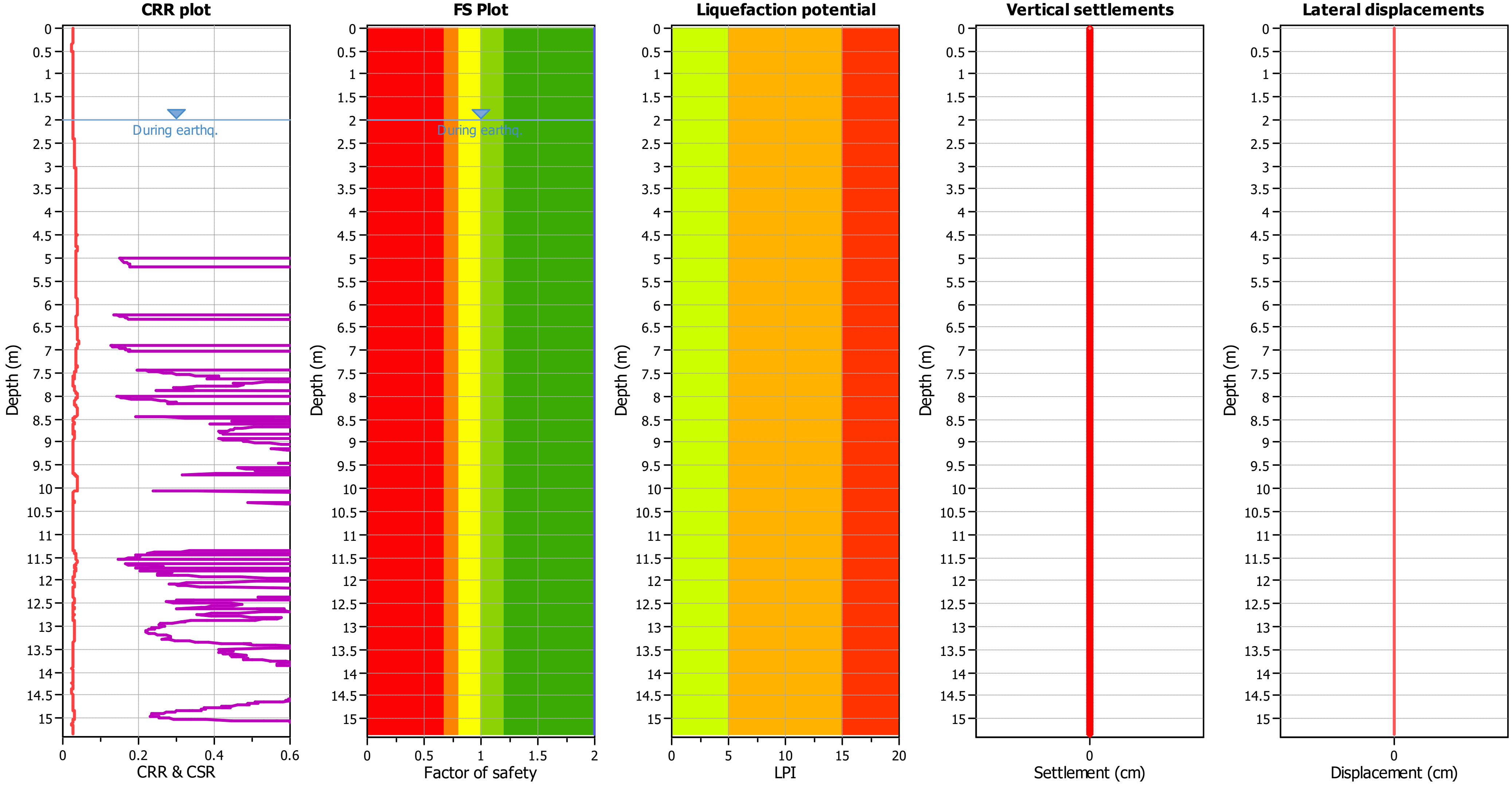
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.00 m	Fill height:	N/A	Limit depth:	N/A

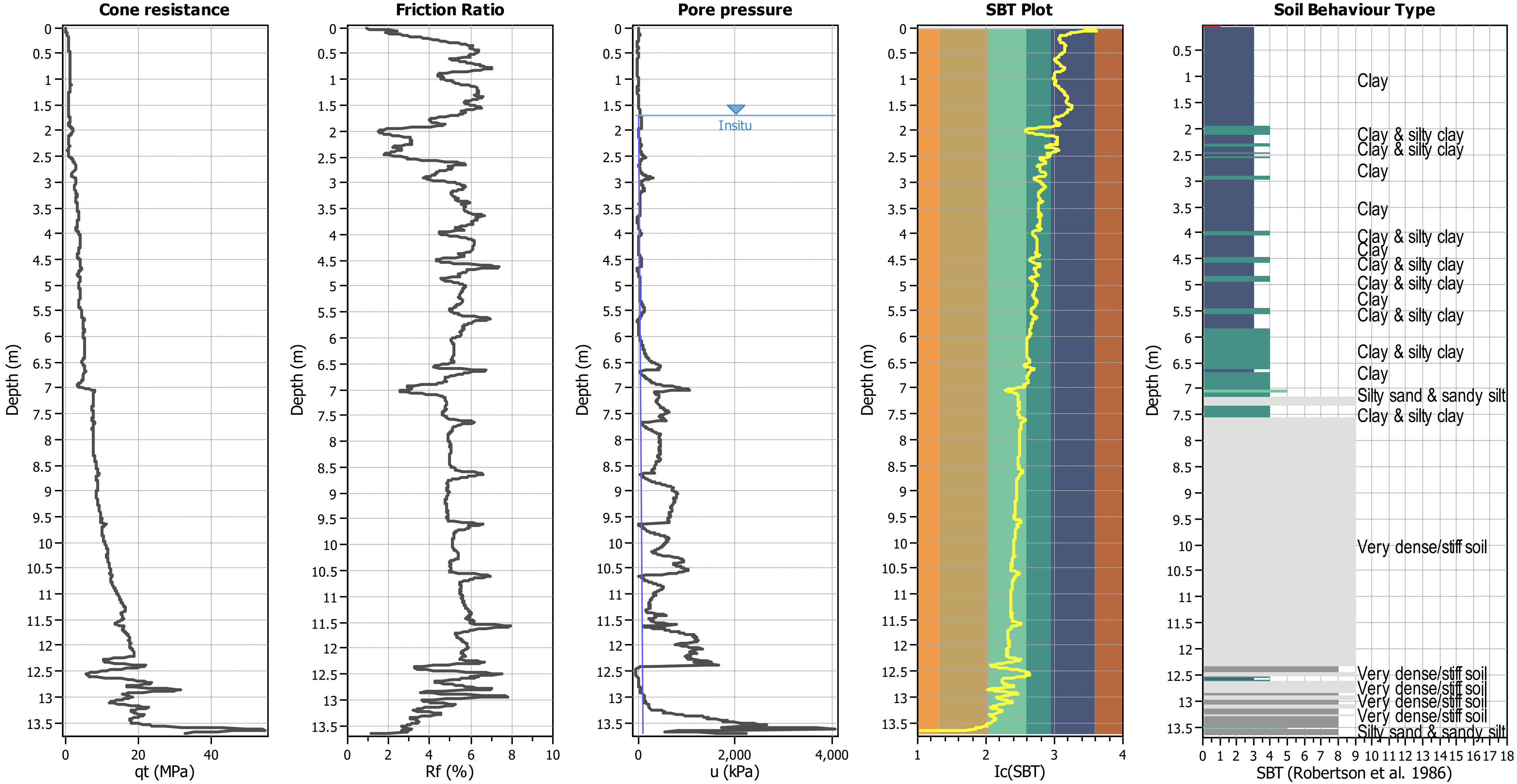
F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

CPT basic interpretation plots



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: 5.90
Peak ground acceleration: 0.05
Depth to water table (insitu): 1.70 m

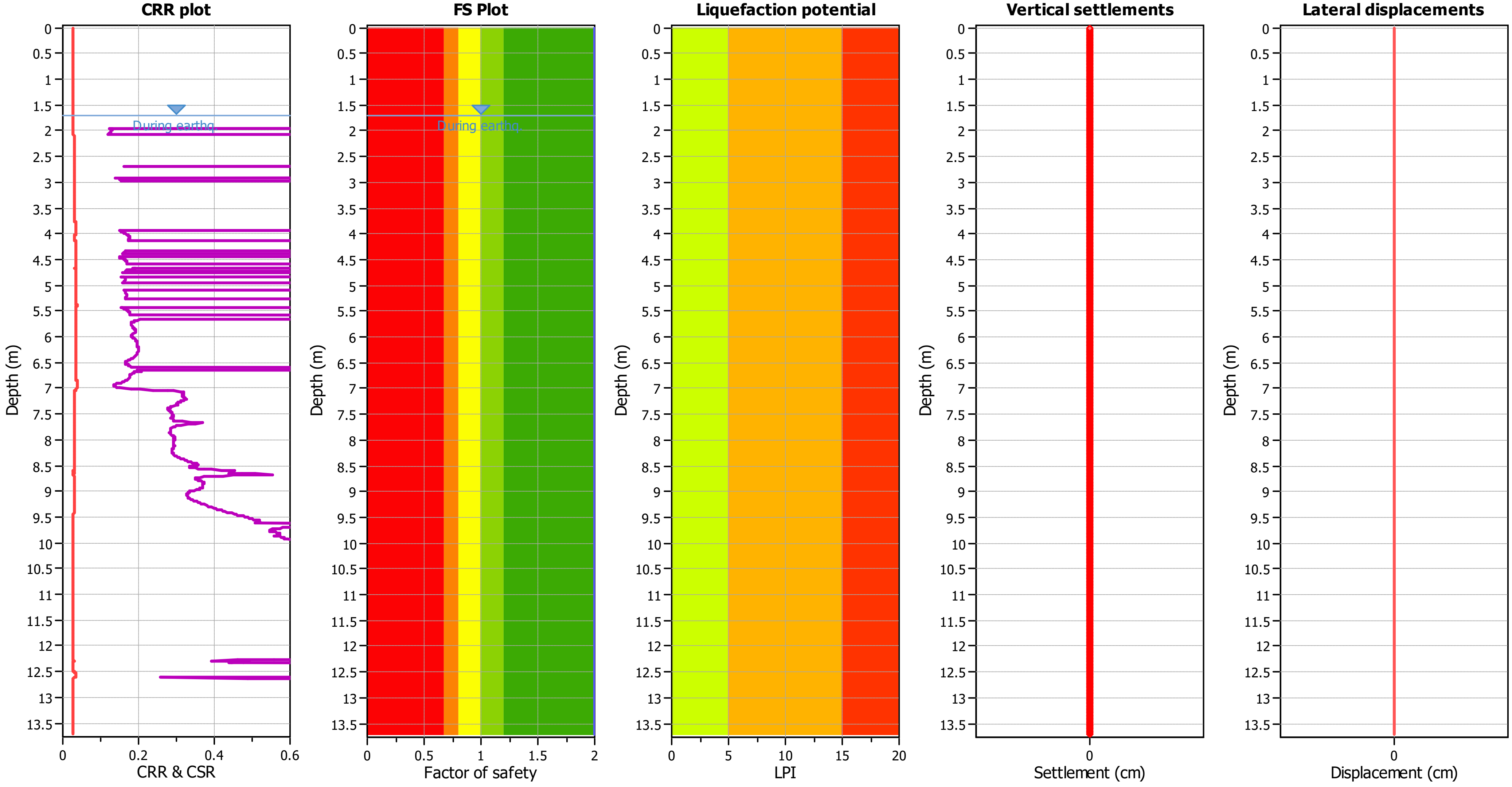
Depth to GWT (erthq.): 1.70 m
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: No
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.70 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.70 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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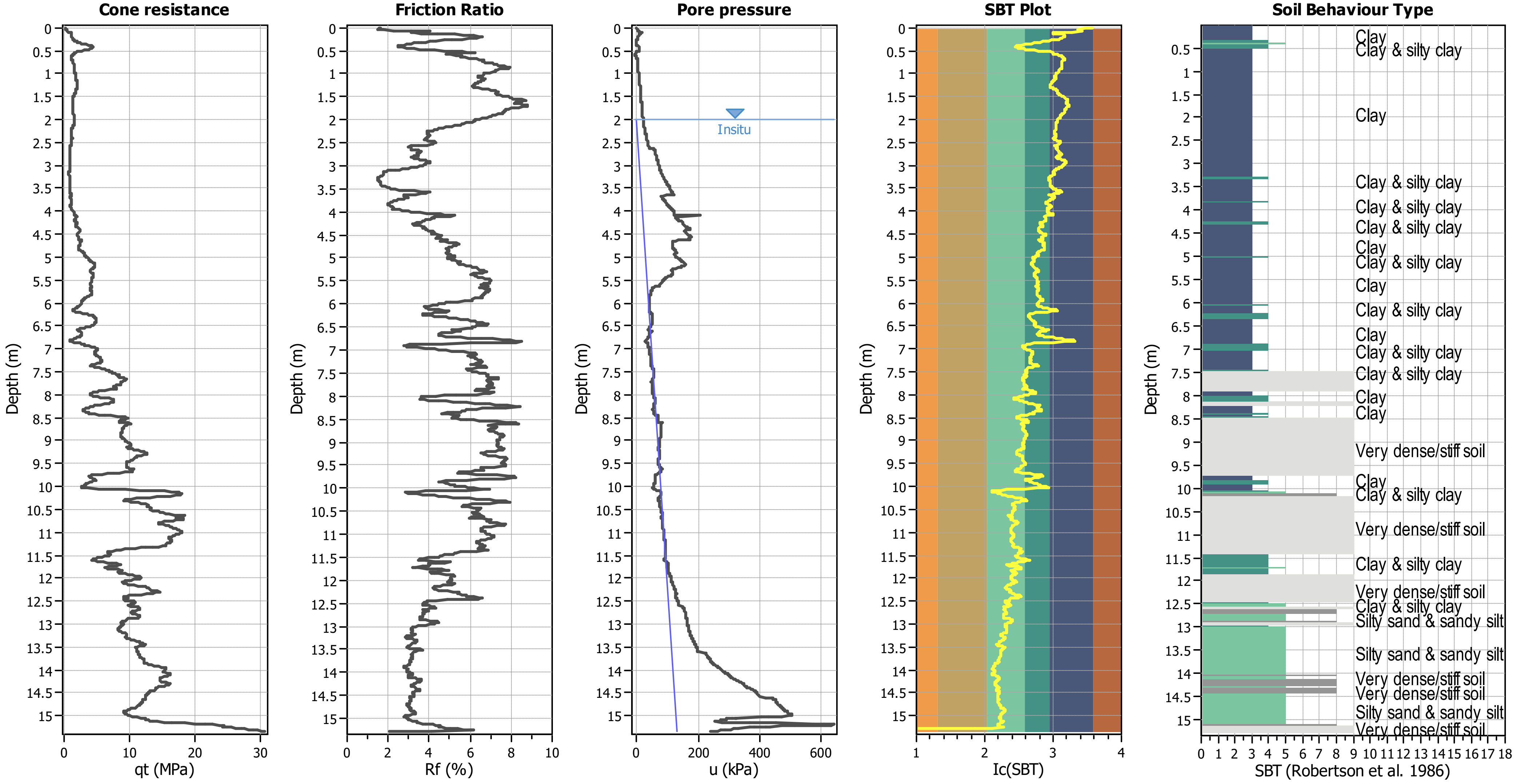
CPT basic interpretation plots



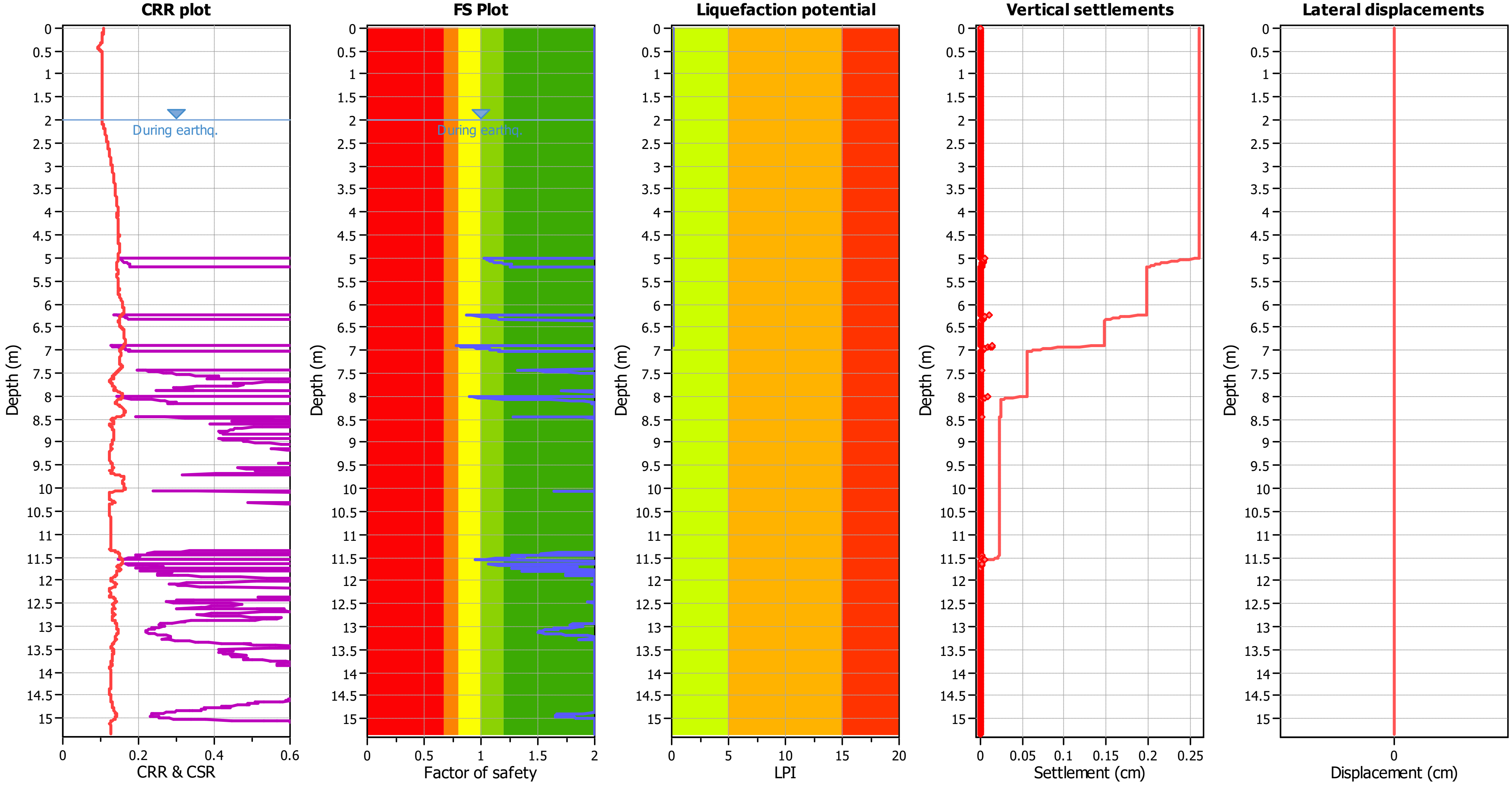
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.00 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
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LPI color scheme

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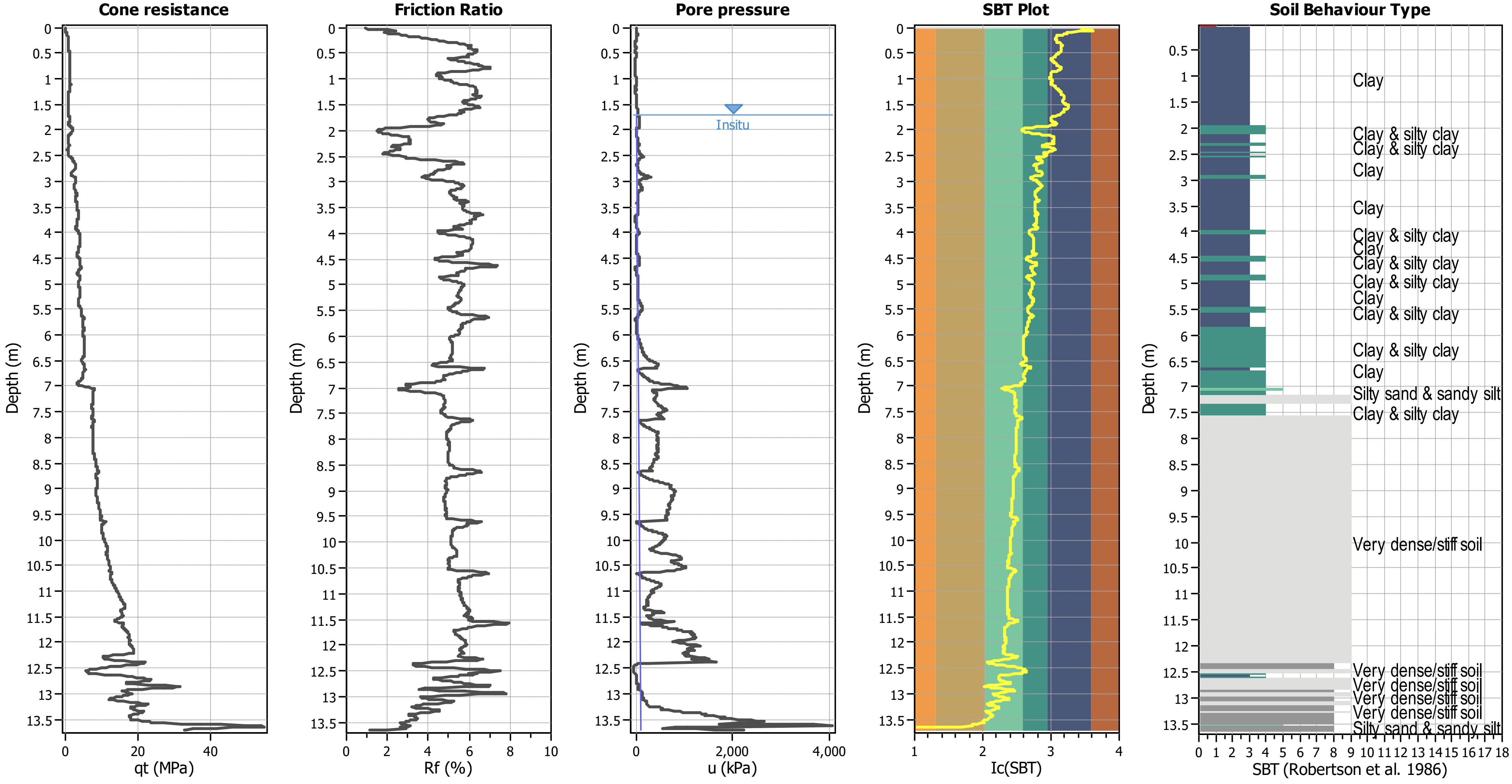
CPT basic interpretation plots



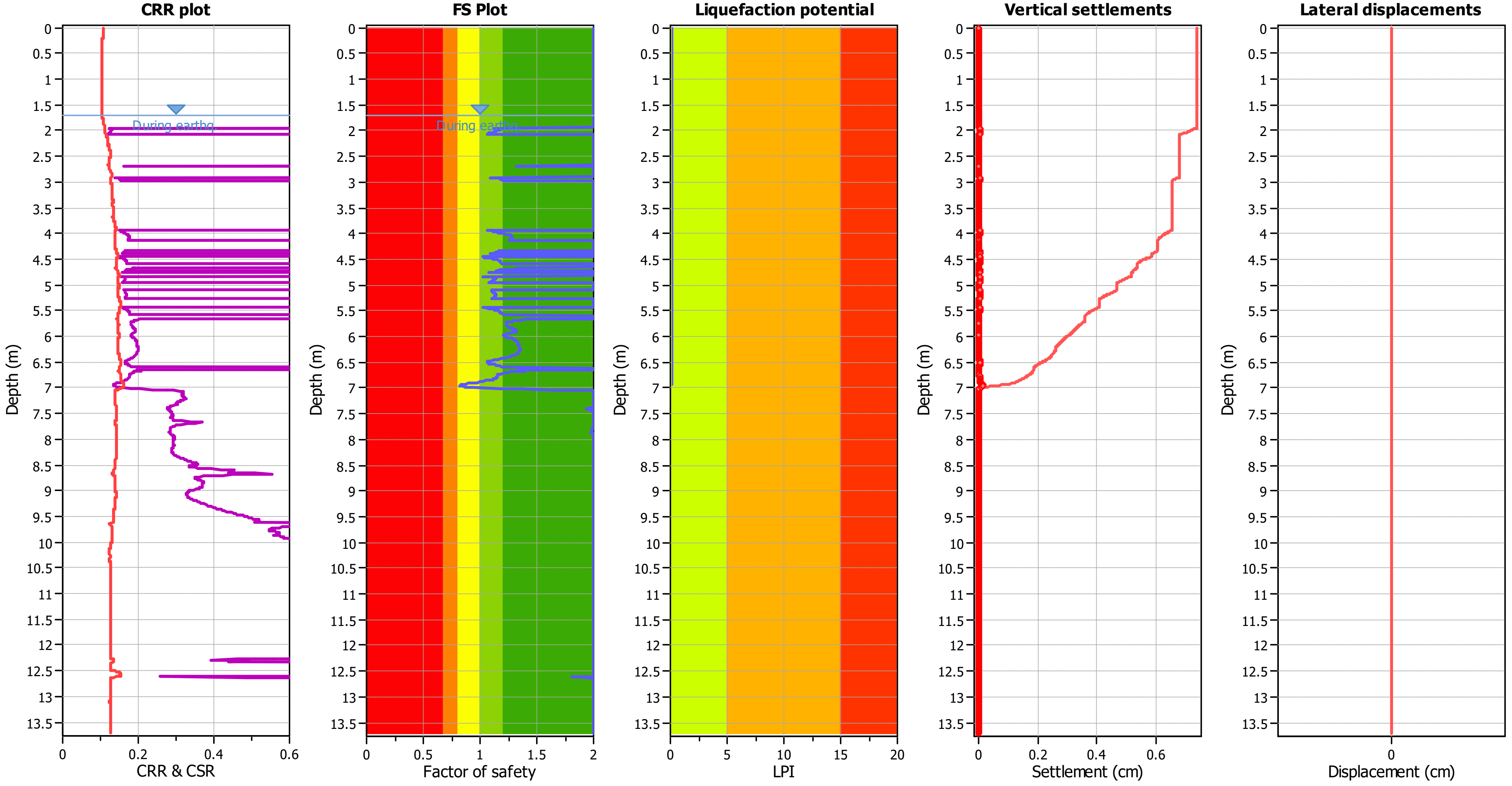
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.70 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.70 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.70 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.70 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

APPENDIX 5:

Laboratory Test Results

Babbage Geotechnical Laboratory

Babbage Geotechnical Laboratory
Level 4
68 Beach Road
Auckland 1010
Telephone
E-mail

P O Box 2027
New Zealand
64-9-367 4954
wec@babbage.co.nz

Please reply to: W.E. Campton

ENGEO LTD.
PO Box 33-1527
Takapuna
Auckland 0740

Attention: **HEATHER LYONS**

Job Number: 66273#L
BGL Registration Number: 3064
Checked by: WEC

31st August 2023

ATTERBERG LIMITS & LINEAR SHRINKAGE TESTING

Dear Heather,

Re: 15 CLARKS LANE, HOBSONVILLE

Your Reference: 23849.000.004
Report Number: 66273#L/AL 15 Clarks Lane

The following report presents the results of Atterberg Limits & Linear Shrinkage testing at BGL of a soil sample delivered to this laboratory on the 21st of August 2023. Test results are summarised below, with page 3 showing where the sample plots on the Unified Soil Classification System (Casagrande) Chart. Test standards used were:

Water Content:	NZS4402:1986:Test 2.1
Liquid Limit:	NZS4402:1986:Test 2.2
Plastic Limit:	NZS4402:1986:Test 2.3
Plasticity Index:	NZS4402:1986:Test 2.4
Linear Shrinkage:	NZS4402:1986:Test 2.6

Borehole Number	Sample Number	Depth (m)	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage (%)*
HA02	**Sample 1**	**0.50 – 1.00**	**23.6**	**42**	**22**	**20**	**12**

The amount of shrinkage of the sample as a percentage of the original sample length.

The whole soil was used for the water content test (the soil was in a natural state), and for the liquid limit, plastic limit and linear shrinkage tests. The soil was wet up and dried where required for the liquid limit, plastic limit and linear shrinkage tests.

[redacted]

As per the reporting requirements of NZS4402: 1986: Test 2.1: water content is reported to two significant figures for values below 10%, and to three significant figures for values of 10% or greater. Test 2.2: liquid limit, test 2.3: plastic limit, and test 2.6: linear shrinkage are reported to the nearest whole number.

Please note that the test results relate only to the sample as-received, and relate only to the sample under test.

Thank you for the opportunity to carry out this testing. If you have any queries regarding the content of this report please contact the person authorising this report below at your convenience.

Yours faithfully,

Justin Franklin
Key Technical Person
Assistant Laboratory Manager
Babbage Geotechnical Laboratory

ACCREDITED
IANZ
TESTING LABORATORY
№ 126

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation. This report may not be reproduced except in full & with written approval from BGL.

[redacted]

BGL Babbage Geotechnical Laboratory	**Job Number:**	**66273#L**	**Sheet 1 of 1**	**Page 3 of 3**
	Reg. Number:	**3064**	**Version No:**	**7**
	Report No:	**66273#L/AL 15 Clarks Lane**	**Version Date:**	**July 2022**
	Project:	**15 CLARKS LANE, HOBSONVILLE**		

DETERMINATION OF THE LIQUID LIMIT, PLASTIC LIMIT & THE PLASTICITY INDEX

Test Methods: NZS4402: 1986: Test 2.2, Test 2.3 and Test 2.4

Tested By:	JL	August 2023
Compiled By:	JF	31/08/2023
Checked By:	JF	31/08/2023

SUMMARY OF TESTING

Borehole Number	Sample Number	Depth (m)	Liquid Limit	Plastic Limit	Plasticity Index	Soil Classification Based on USCS Chart Below
HA02	**Sample 1**	**0.50 - 1.00**	42	22	**20**	CL

The chart below & soil classification terminology is taken from ASTM D2487-17^{e1} "Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)", April 2020, & is based on the classification scheme developed by A. Casagrande in the 1940's (Casagrande, A., 1948: Classification and identification of soil. Transactions of the American Society of Civil Engineers, v. 113, p. 901-930). The chart below & the soil classification given in the table above are included for your information only, and are not included in the IANZ endorsement for this report.

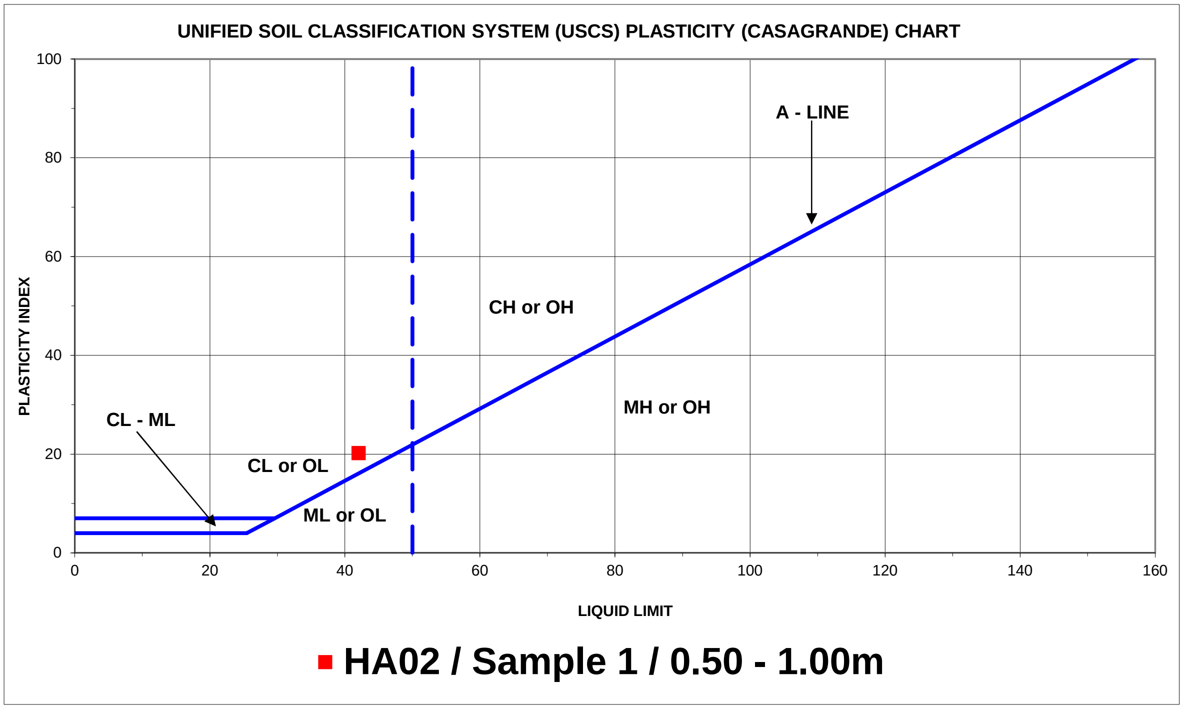


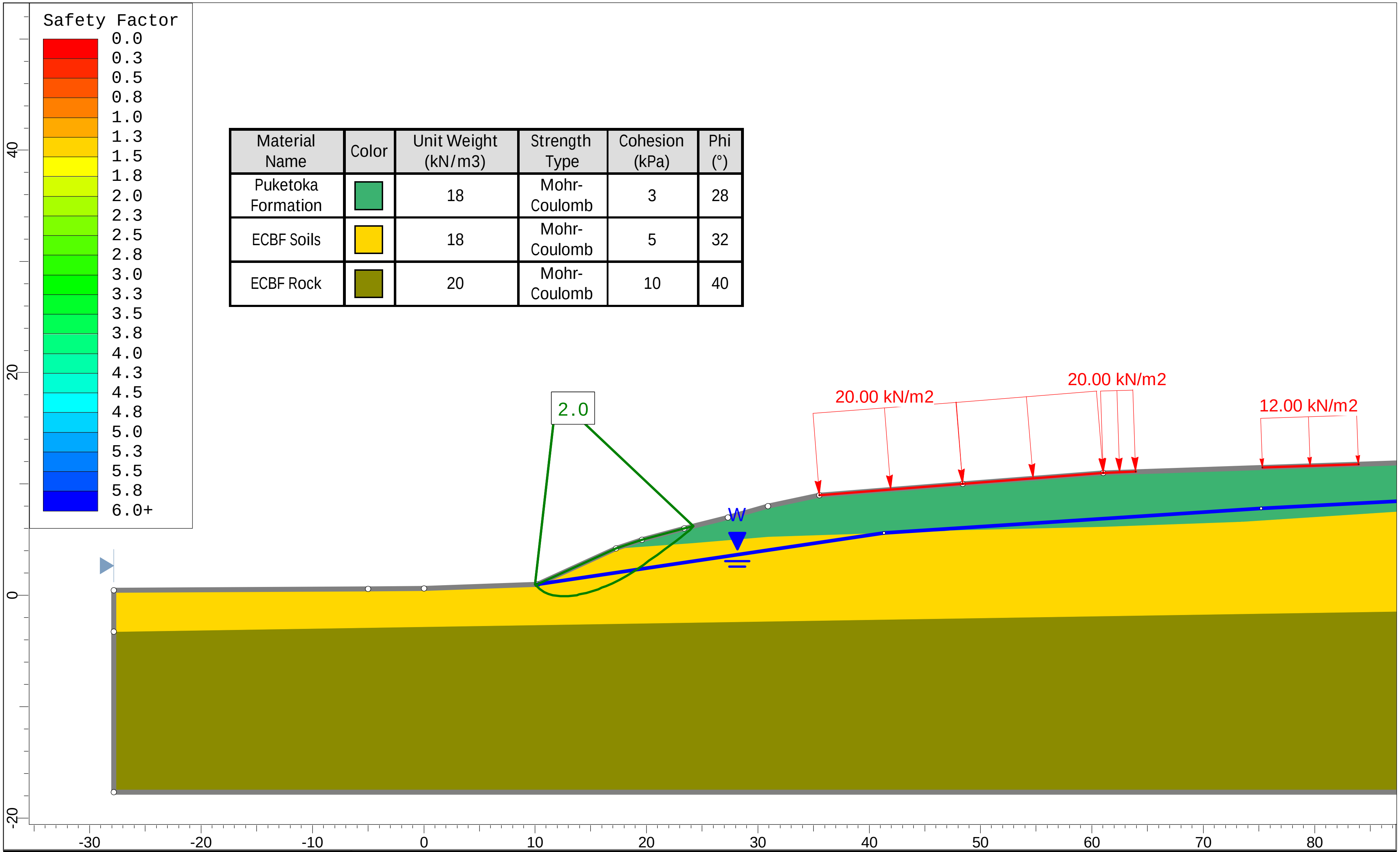
CHART LEGEND

CL = CLAY, low plasticity ('lean' clay)
OL = ORGANIC CLAY or ORGANIC SILT, low liquid limit
ML = SILT, low liquid limit
CL - ML = SILTY CLAY

CH = CLAY, high plasticity ('fat' clay)
OH = ORGANIC CLAY or ORGANIC SILT, high liquid limit
MH = SILT, high liquid limit ('elastic silt')

APPENDIX 6:

Slide Outputs

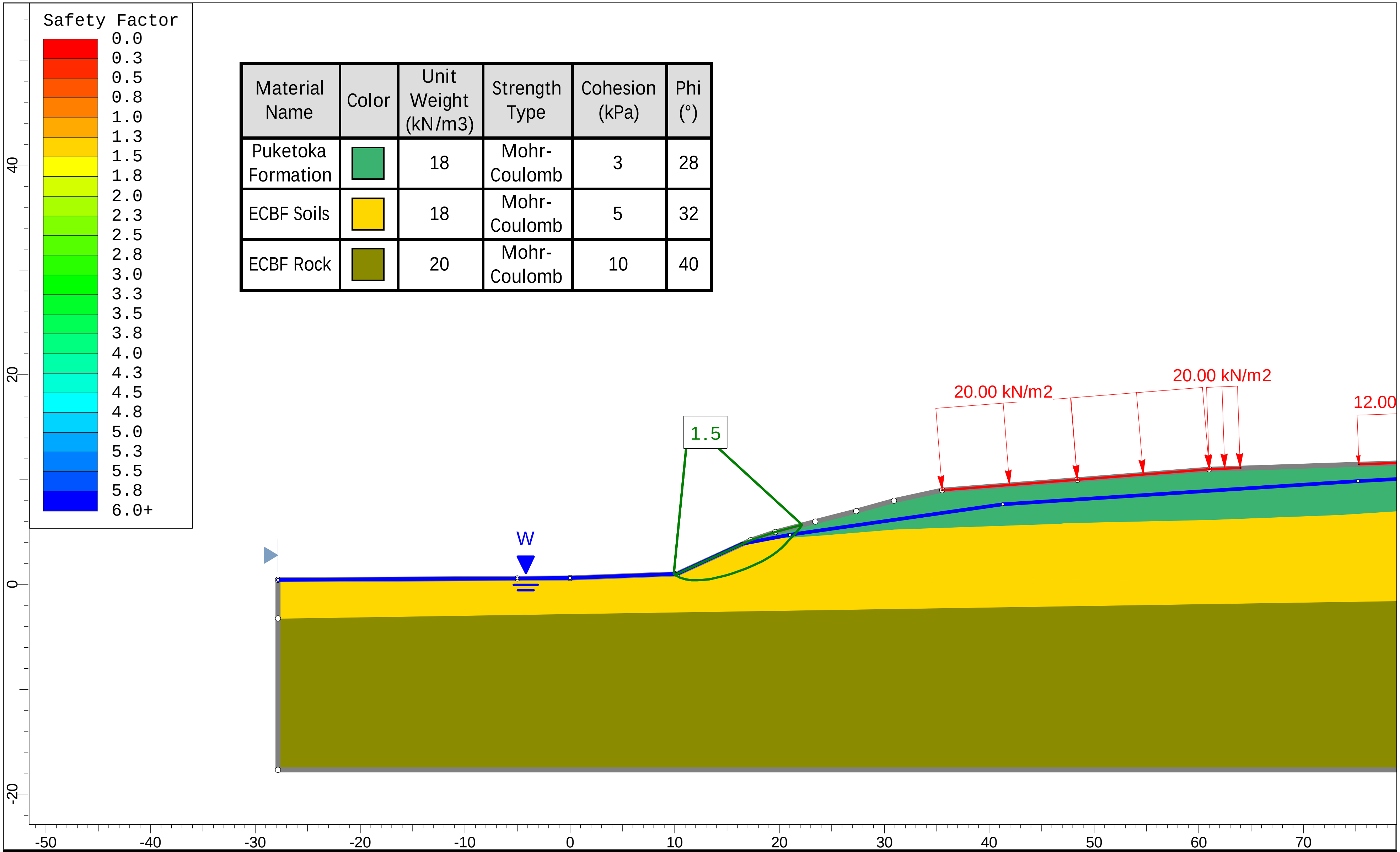


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Puketoka Formation	🟩	18	Mohr-Coulomb	3	28
ECBF Soils	🟨	18	Mohr-Coulomb	5	32
ECBF Rock	🟫	20	Mohr-Coulomb	10	40

rocscience

SLIDEINTERPRET 9.030

Project	15 Clarks Lane, Hobsonville
Group	Post Development - Section B
Scenario	Normal
Drawn By	JT
Company	ENGEO Ltd.
Date	30/10/2023, 11:23:51 am
File Name	X-Section 3_JT.slmd

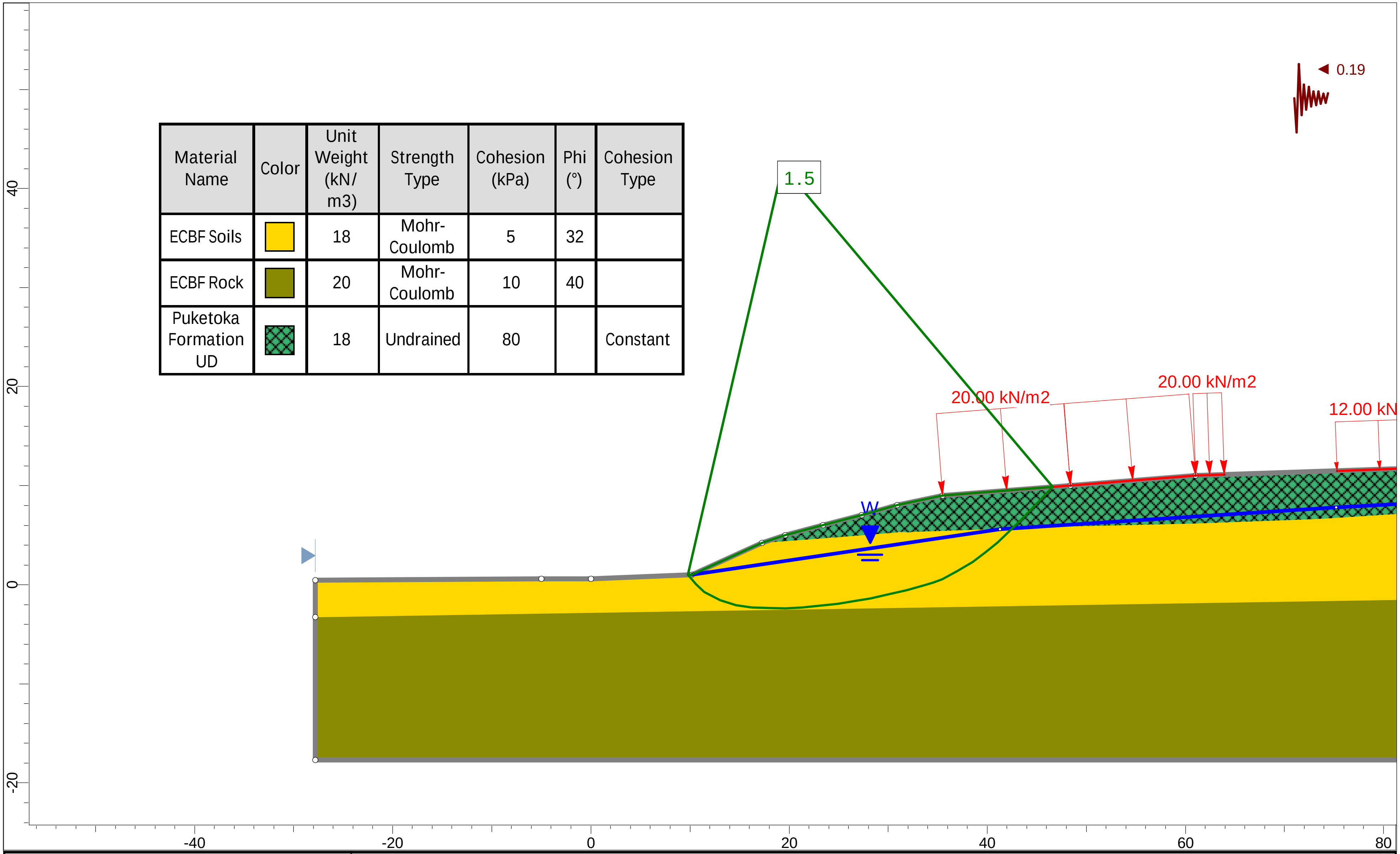


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Puketoka Formation	🟩	18	Mohr-Coulomb	3	28
ECBF Soils	🟨	18	Mohr-Coulomb	5	32
ECBF Rock	🟫	20	Mohr-Coulomb	10	40

rocscience

SLIDEINTERPRET 9.030

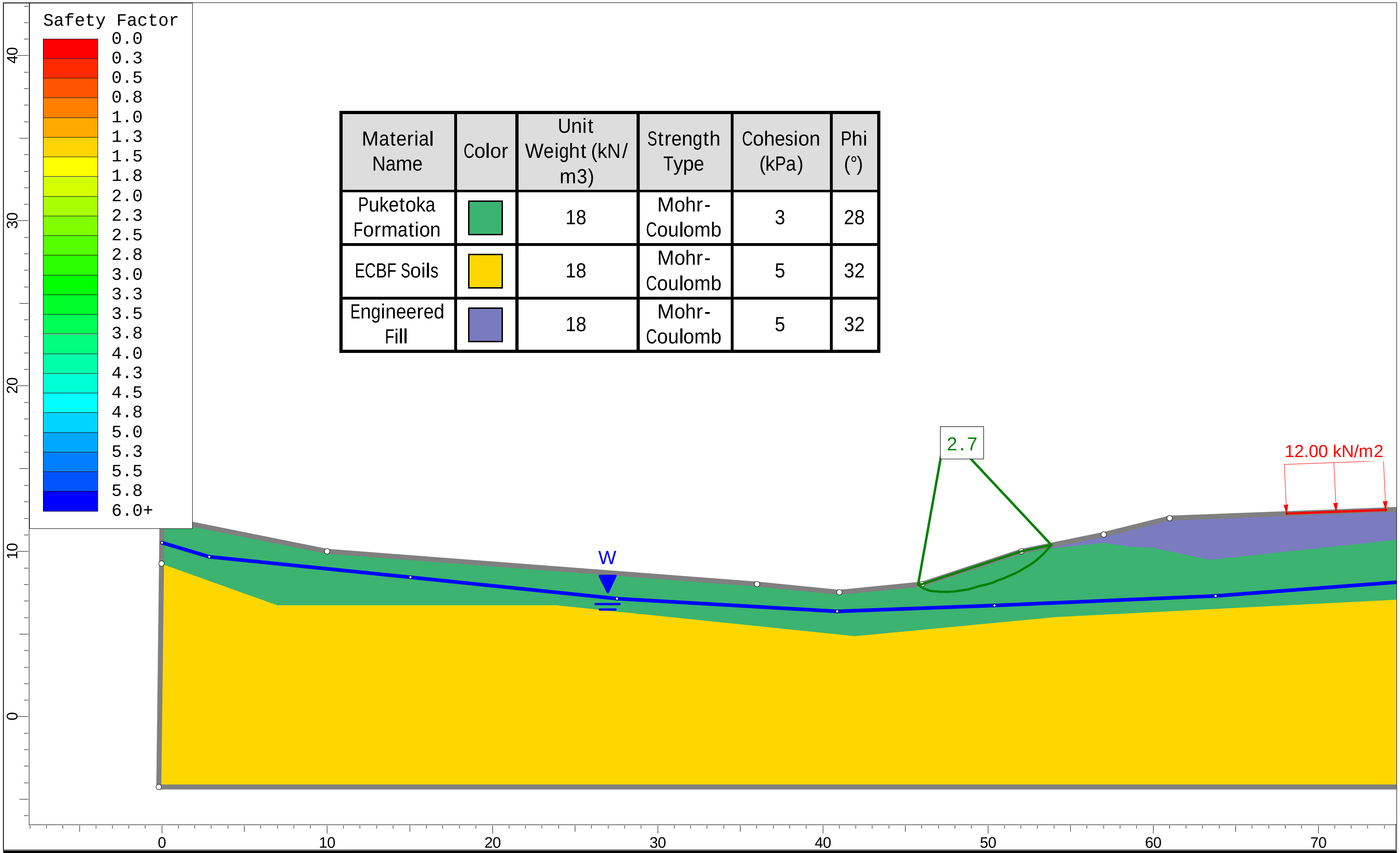
Project	15 Clarks Lane, Hobsonville		
Group	Post Development - Section B	*Scenario*	Transient
Drawn By	JT	*Company*	ENGEO Ltd.
Date	30/10/2023, 11:23:51 am	*File Name*	X-Section 3_JT.slmd



Material Name	Color	Unit Weight (kN/ m3)	Strength Type	Cohesion (kPa)	Phi (°)	Cohesion Type
ECBF Soils	🟨	18	Mohr-Coulomb	5	32	
ECBF Rock	🟫	20	Mohr-Coulomb	10	40	
Puketoka Formation UD	▨	18	Undrained	80		Constant

Project	15 Clarks Lane, Hobsonville		
Group	Post Development - Section B	*Scenario*	Seismic
Drawn By	JT	*Company*	ENGEO Ltd.
Date	30/10/2023, 11:23:51 am	*File Name*	X-Section 3_JT.slmd

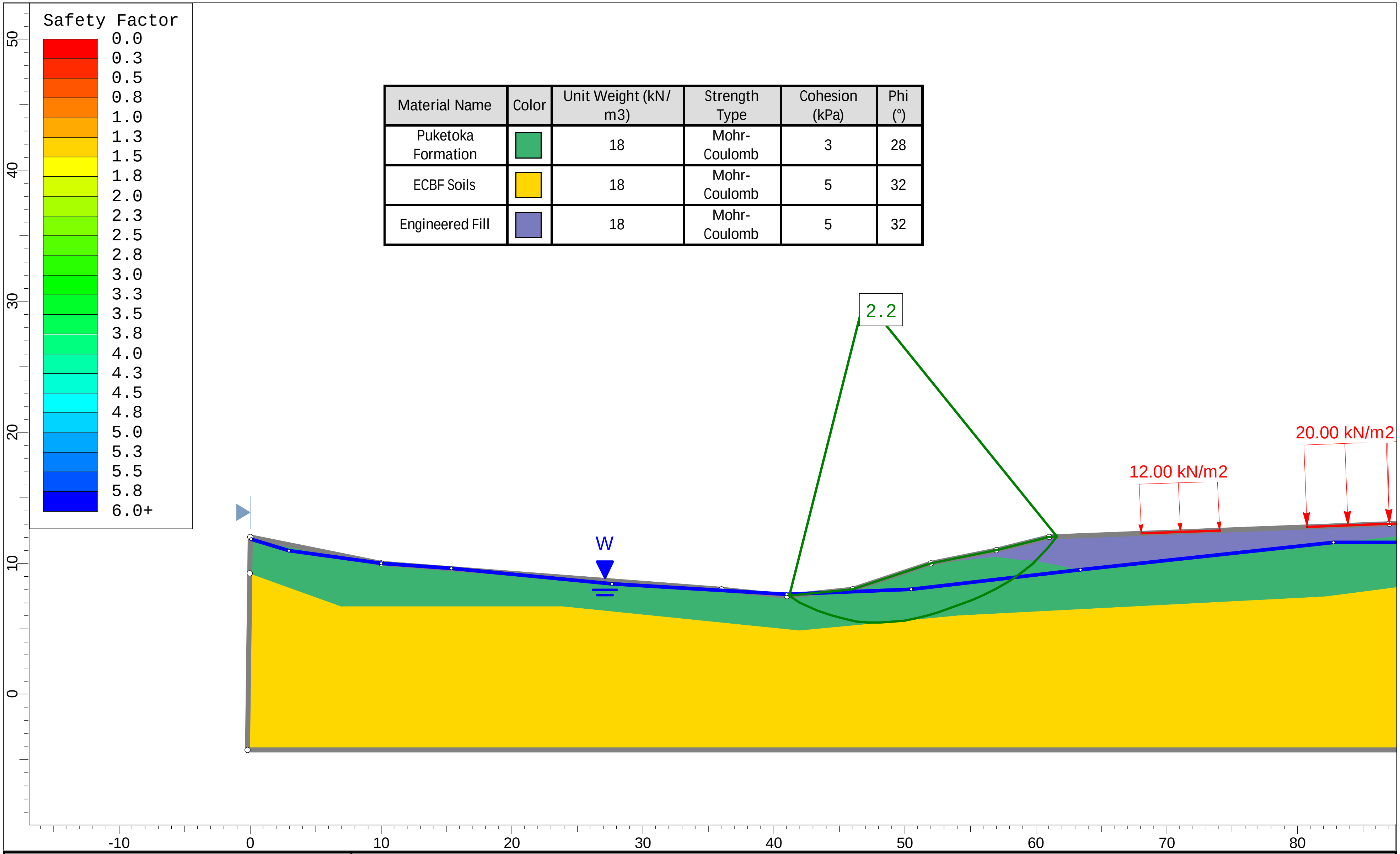
SLIDEINTERPRET 9.030



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Puketoka Formation	🟩	18	Mohr-Coulomb	3	28
ECBF Soils	🟨	18	Mohr-Coulomb	5	32
Engineered Fill	🟪	18	Mohr-Coulomb	5	32

Project	15 Clarks Lane, Hobsonville		
Group	Post Development - Section C	*Scenario*	Normal
Drawn By	JT	*Company*	ENGEO Ltd.
Date	30/10/2023, 11:23:51 am	*File Name*	X-Section 3_JT.slmd

SLIDEINTERPRET 9.030

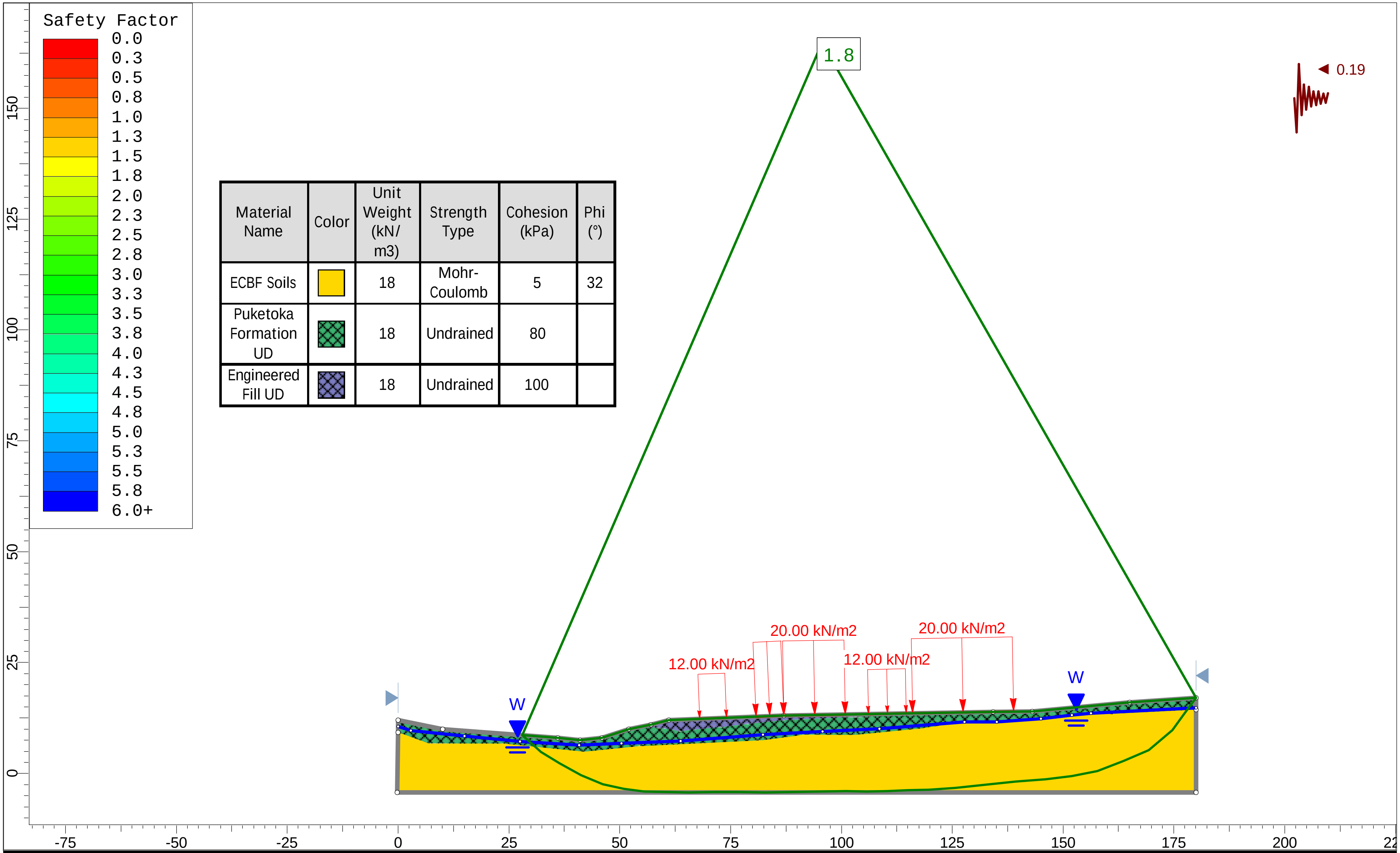


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Puketoka Formation	🟩	18	Mohr-Coulomb	3	28
ECBF Soils	🟨	18	Mohr-Coulomb	5	32
Engineered Fill	🟪	18	Mohr-Coulomb	5	32

rocscience

SLIDEINTERPRET 9.030

Project	15 Clarks Lane, Hobsonville		
Group	Post Development - Section C	*Scenario*	Transient
Drawn By	JT	*Company*	ENGEO Ltd.
Date	30/10/2023, 11:23:51 am	*File Name*	X-Section 3_JT.slmd



Material Name	Color	Unit Weight (kN/ m3)	Strength Type	Cohesion (kPa)	Phi (°)
ECBF Soils	🟨	18	Mohr-Coulomb	5	32
Puketoka Formation UD	🟩	18	Undrained	80	
Engineered Fill UD	🟪	18	Undrained	100	

rocscience

SLIDEINTERPRET 9.030

Project	15 Clarks Lane, Hobsonville		
Group	Post Development - Section C	*Scenario*	Siesmic
Drawn By	JT	*Company*	ENGEO Ltd.
Date	30/10/2023, 11:23:51 am	*File Name*	X-Section 3_JT.slmd

APPENDIX 7:

Geological Cross Section

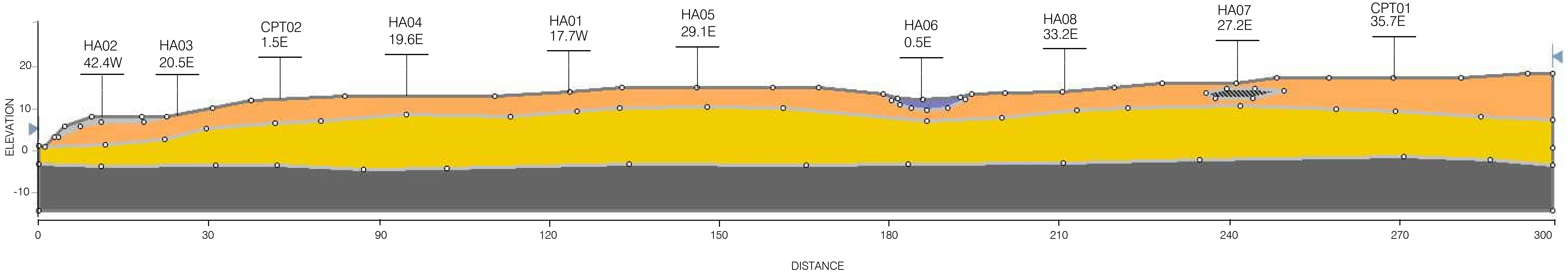
DATE PLOTTED: 1 June 2023 12:06 pm BY : CAROLINE DJERDJERIAN

Legend

- ⬜ Fill
- 🟪 Gully Formation
- 🟧 Puketoka Formation
- 🟨 East Coast Bays Formation
- ⬛ Inferred competent East Coast Bays Formation Rock
- ⬛ Peat

CROSS SECTION A-A'

SCALE 1:750



		DRAFT		
A	6.23	Draft for review	CD	-
Rev	Date	Description	Drwn	Chkd

ENGEO

Auckland Office
8 Greydene Place
Takapuna 0622
Auckland
Tel: 09 972 2205
www.engeo.co.nz

Title:

GEOLOGICAL CROSS SECTION

Client: Cabra Developments Limited

Project:
15 Clarks Lane,
Hobsonville, Auckland

Designed: RL
Drawn: RL
Checked:
Date: 6.23
Scale: 1:750

Proj No: 23849.000.001 02

Figure No:
1

Size: A3
Rev: A

CAD File: Z:\Drafting\21001 to 22000\21133 - Fernleigh Business Park\21133 -GeoXS-FernleighBusinessPark-523.dwg
XREF's:

ORIGINAL FIGURE PRINTED IN COLOUR

ENGEO Limited
8 Greydene Place, Takapuna, Auckland 0622
PO Box 33-1527, Takapuna, Auckland 0740
T: +64 9 972 2205
www.engeo.co.nz

Project Number 23849.000.002

Geotechnical Investigation

10 Sinton Road, Hobsonville, Auckland

Submitted to:
Cabra Developments Limited
Unit 9B
30 Foundry Road
Silverdale Auckland

Contents

Tables

Figures

Appendices

ENGEO Document Control:

Report Title	Geotechnical Investigation - 10 Sinton Road, Hobsonville				
Project No.	23849.000.002	**Doc ID**	04		
Client	Cabra Developments Limited	**Client Contact**	Duncan Unsworth		
Distribution (PDF)	Duncan Unsworth (Cabra)				
Date	**Revision**	**Description**	**Author**	**Reviewer**	**WP**
10/11/2023	0	Issued to client	JL	PF	DF
06/05/2024	1	Issued to client	JL	HL	DF

1 Introduction

ENGEO Ltd was requested by Cabra Developments Limited to undertake a geotechnical investigation of the property at 10 Sinton Road, Hobsonville, Auckland (herein referred to as ‘the site’; shown in Figure 1). The purpose of this assessment was to support a Resource Consent application for the proposed redevelopment of the site. This work has been carried out in accordance with our signed agreement dated 31 July 2023.

We have been provided with an unnumbered draft masterplan of the site by Forme Planning Limited.

Our scope of works included:

- Desktop review of existing geotechnical reports and drawings for the site and a review of publicly available geological and geotechnical data, and aerial photographs.
- Undertake a site walkover to assess current site conditions and observe geomorphological evidence of land disturbance, active and historical slope instability, and in the case for this particular property being adjacent to the coastline, soil and rock outcrops and groundwater seepages that may be observed in the cliff.
- Drill up to two hand auger boreholes to a target depth of 5.0 m below ground level (bgl) with associated strength tests across the site to provide geotechnical data on the shallow soil profile.
- Undertake two Cone Penetration Tests (CPTs) to target depths of 15.0 m bgl to support a liquefaction assessment for the alluvial soils.
- Recover a representative soil sample from near surface soils for laboratory expansive soils classification testing.
- Undertake a liquefaction assessment using primary CPT data from our intrusive investigations.
- Undertake slope stability analysis for the two landslides on-site.
- Preparation of this Geotechnical Investigation Report presenting the findings of our investigation and geohazard assessments to support the Resource Consent application.

To support a Resource Consent application this report is required to reflect the earthworks proposals, particularly with respect to the slope stability assessment, which have not yet been developed. A supplementary assessment will be required to address the development proposals when available.

2 Site Description

The site comprises 2.7291 ha parcel of joint residential and pastoral land legally described as Lot 23 ALLOT 2 SO 958, located on an elevated coastal terrace bordered to the northwest by a tidal creek; the Waiarohia Inlet.

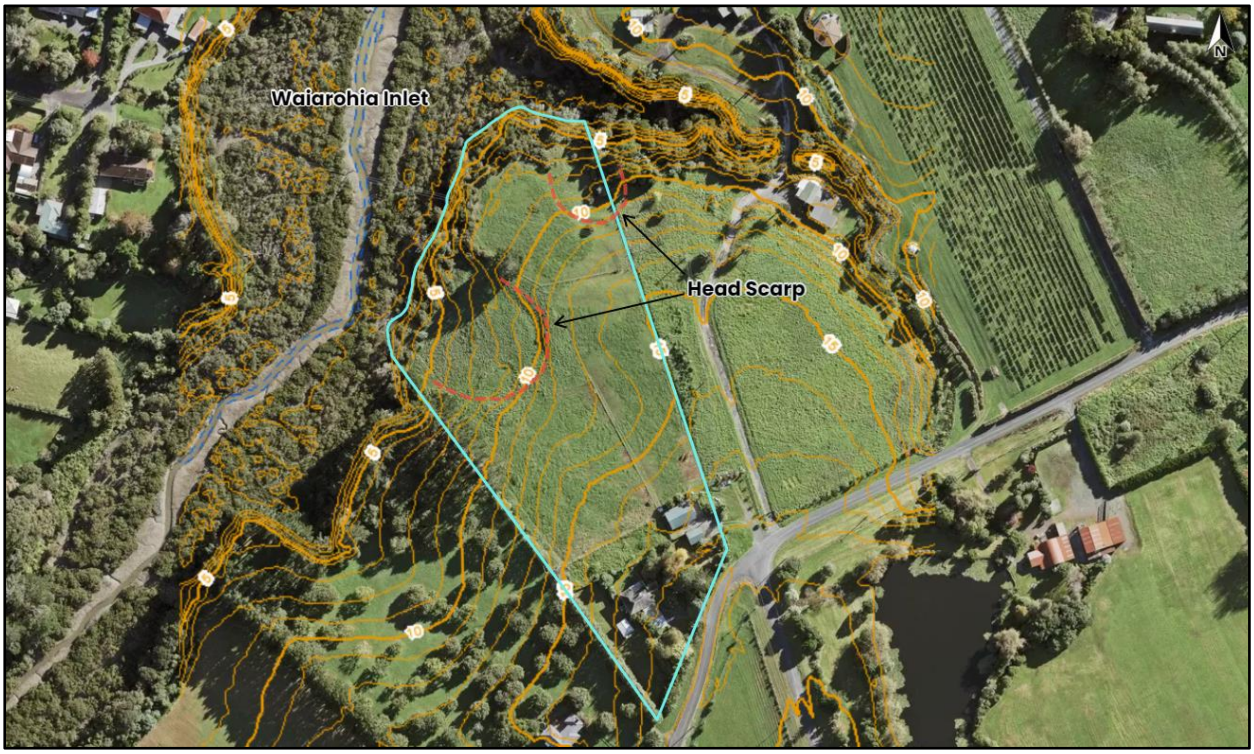
The site is accessed via two private driveways, one directly off the intersection between Sinton Road and Clarks Lane to the southeast and one directly off Sinton Road to the south. Site features are shown on the plan in Figure 1.

Two dwellings are located within the southern portion of the site, along with small sheds or garages and landscaped areas. The northern portion of the site is comprised of former grazing paddocks that have been left fallow and are not currently in use. The site is bound by Sinton Road to the south, lifestyle blocks and residential dwellings to the east and west, and by the Waiarohia Inlet to the north. The northern and southern site boundaries are covered in dense vegetation.

An overland flow path runs from the centre of the site toward the north-western portion of the site. This flow path diverts surface water run-off from the upslope areas to the Waiarohia Inlet. There are no existing public services that run through the site.

The topography on-site displays two distinctive concave landslide head scarps and associated debris lobes located towards the northern end of the site failing towards to Waiarohia Inlet. The western landslide has failed in the north-western direction, whilst the eastern landslide has failed in the northern direction. There is a total cross fall of approximately 16 metres across site in the south-eastern to north-western direction. The land south of the landslides and coastal margins has an approximate slope angle of 2º. The head scarp of the western landslide has an approximate slope angle of 19º, whilst the head scarp of the eastern landslide has an approximate slope angle of 8º. The steepest slope of the coastal margin sits at 51º, whilst the shallowest sits at 11º.

Figure 1: Site Features Plan



NTS. Aerial imagery from LINZ. Site boundary shown in blue. Contours shown in orange.

3 Proposed Development

We have been provided with an excerpt of a draft masterplan of the proposed development by Forme Planning Limited which shows that the development will comprise a 62 lot residential subdivision. The proposed lots are to be of a range of typologies. Lots along the eastern site boundary are to contain terraced town houses while the lots along the western boundary are to contain single detached dwellings. Larger lots are proposed towards the north of the site bordering an esplanade which follows the northern site boundary.

The excerpt is of too low resolution to reproduce within this report. No earthworks plans or proposed contours have been provided to ENGEO at the time of writing.

4 Desktop Study

4.1 Geology and Geomorphology

The site is regionally mapped (1:250,000) by GNS Science[1] as spanning the Geological boundary between interbedded sandstones and mudstones of the East Coast Bays Formation to the north and Late Pliocene to Middle Pleistocene pumiceous river deposits of the Tauranga Group (Puketoka Formation) comprising silts clays and sands to the south.

Structural data presented in the 1:250,000 GNS map indicate that these strata dip shallowly at 20° to the southeast.

The geological boundary is mapped as occurring immediately to the north of the existing dwelling on-site, roughly following the boundary between residential and pastoral land. The boundary is shown as inferred and it should also be considered that GNS maps are regional in scale and therefore the mapped boundary may not reflect its true location.

4.2 Previous Study

We have been provided with an historical geotechnical investigation letter prepared by CMW Geosciences in November 2016[2]. The investigation described by that letter comprised six hand auger boreholes drilled to a maximum depth of 5.0 m below ground level.

Six of the boreholes were drilled on the gently sloping portion of the site, one through the western landslide head scarp, and one in the north-western corner of the site through the landslide debris lobe.

Beneath a surface layer of topsoil the augers on the gently sloping landform and head scarp crest encountered soils described as Tauranga Group alluvium comprising stiff to hard, grey and brown clay. In borehole HA05, topsoil inclusions were identified in the soil profile to 1.2 m depth; although not identified within the report, this may be indicative of local, shallow fill soils.

Groundwater was encountered in these boreholes between 2.2 m and 3.5 m below ground level.

---

[1] https://data.gns.cri.nz/geology/

[2] Geotechnical Investigation Appraisal and Site Walkover for 10 Sinton Road, Hobsonville. CMW, AKL2016_0605aa Rev A (2016).

Borehole HA08 drilled within the north-western landslide debris lobe encountered soils identified as colluvium, comprising mottled firm to very stiff clays, overlying Tauranga alluvium to a depth of 2.2 m although it is possible that colluvium extends to a depth of 2.9 m based on a transition between saturated and moist soils and a noted increase in shear strength at this depth.

Deposits of the East Coast Bays Formation soil were not identified in the boreholes, however boreholes HA07-16 located in the north of the site, met practical refusal at 3.0 m depth and HA08 met practical refusal at 4.0 m. This may indicate the presence of shallow rock.

4.3 New Zealand Geotechnical Database

ENGEO reviewed the data held on the New Zealand Geotechnical Database (NZGD) October 2023. No relevant deep investigation records were available.

4.4 Auckland Council GeoMaps

4.4.1 Coastal Instability and Erosion

The Auckland Council GeoMaps layer ‘Areas Susceptible to Coastal Instability and Erosion’ identifies areas of coastline in Auckland that could be affected by coastal erosion and instability under a range of climate change scenarios and timeframes. The potential regression lines for 2050, 2080 and 2130 for this site are shown in Figure 2. These areas are limited to the northern slopes, along the Waiarohia Inlet.

4.4.2 Flood Plains & Prone Areas

The Auckland Council GeoMaps layer ‘Flood Plains & Flood Prone Areas’ identifies areas of land in Auckland that could be affected by flooding during and / or following periods of heavy rain. Portions of the site labelled as flood prone or flood plains are shown in Figure 2 and are limited to areas adjacent to Waiarohia Inlet.

Figure 2: Auckland Council Hazard Map

4.5 Seismicity

The GNS New Zealand Active Fault Database[3] indicates that the nearest mapped active fault is the Waikopua Fault (ref# 7540) located approximately 38.5 km to the southeast of the site. The Waikopua Fault is a normal fault with recurrence rate and slip rate unknown to GNS or ENGEO.

4.6 Historical Aerial Photography Review

Aerial photographs of the site dating from 1940 to 2023 have been accessed from Auckland Council Geomaps[4], Retrolens[5], Nearmaps and Google Earth Pro and reviewed in order to identify evidence for historical changes to the site of geotechnical significance. Table 1, below, provides a summary of our findings.

---

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

[4] https://www.aucklandcouncil.govt.nz/geospatial/geomaps/Pages/default.aspx

[5] https://retrolens.co.nz

Table 1: Summary of Aerial Photographs

Year	Description
1940	The site is covered by pasture. The existing western dwelling is present on the southern site boundary, however the existing ancillary buildings are absent. A small building is present in a paddock adjacent to the eastern boundary. Both head scarps appear visible at the northern end of the site. Sinton Road is absent and the property is accessed by a small track.
1950	Several small ancillary structures have been constructed around the dwelling.
1959	A barn or large shed has been constructed in the middle of the paddock towards the southern end of the site.
1963	The building on the eastern site boundary has been removed. Three additional farm buildings have been constructed around the shed/barn.
1980	Sinton Road has been constructed. The farm buildings have been removed and the site is now mostly covered by a circular track. The second existing building have been constructed.
2000	The site is in its present arrangement with all existing dwellings and ancillary structures complete.
2008	No significant changes.
2010	No significant changes.
2023	No significant changes.

Based on our findings no significant landscape modification has occurred which may influence future development. However, it should be borne in mind that local undocumented fills associated with historical building foundations and farming activities may be encountered on-site.

Based on the low resolution and long time-gap of the aerial photograph records it is inconclusive whether there has been continuous movement of the landslides at site between 1940 and the present day.

5 Site Investigation

5.1 Site Observations

ENGEO visited site on 15 August 2024 to complete a site walkover, assess current site conditions and identify evidence for potential geohazards that may affect a future land use change at this site. During our site walkover we made the following observations:

- The majority of the site comprises gently undulating grassed paddocks falling to the north-northeast (Photo 1).

- Drainage of the paddocks is generally poor with sporadic patches of saturated ground.
- The western landslide head scarp forms a distinct break in slope. No further signs of instability were observed behind the crest of the scarp (Photo 2).
- An overland flow path drains down the headscarp and flows northwest to an incised gully at the edge of the debris lobe. This has been used for fly tipping (Photo 3).
- The toe of the debris lobe shows evidence of local instability including overturning trees (Photo 4).
- The eastern landslide head scarp is partially obscured by vegetation (Photo 5).
- The debris lobe below the scarp showed evidence of ongoing soil movement; many of the trees were overturned and several fallen trees were observed at the toe of the slope (Photo 6).

Figure 3: Site Photographs

Photo 1: Site facing northwest.

Photo 2: Western head scarp facing southeast.

Photo 3: Incised gully facing west.

Photo 4: Overturned trees at base of western debris lobe.

Photo 5: Eastern head scarp facing east.

Photo 5: Overturned trees at base of eastern debris lobe.

5.2 Investigations Completed

ENGEO attended site on 15 August to complete a subsurface investigation to supplement the investigation previously completed by CMW. The investigation comprised two hand auger boreholes, HA01 and HA02 completed through the eastern debris lobe, alongside two CPT tests, CPT01 and CPT02 completed through the western debris lobe and gently sloping landform. Investigation locations are shown on the geotechnical investigation plan in Appendix 1.

Hand auger boreholes HA01 and HA02 were drilled to depths of 4.9 and 4.5 m respectively, where they met practical refusal. and were logged in general accordance with the New Zealand Geotechnical Society guidelines[6] by an ENGEO engineering geologist. CPT01 and CPT02 were completed to depths of 8.78 m and 15. 5 m by Ground Investigation Ltd.

Full geotechnical hand auger logs are presented in Appendix 2 and CPT results are presented in Appendix 3. A summary of the findings of our investigation are presented in Section 5.3.

5.3 Investigation Findings

Ground conditions encountered across the site are summarised as follows:

- Topsoil was encountered to depths of 0.2 m in borehole HA01 and 0.3 m in HA02.
- Colluvium was encountered underlying topsoil in the eastern debris lobe. In borehole HA01 this extended to 3.5 m depth, in HA02 colluvium was encountered to 3.2 m. These soils typically comprised stiff to very stiff mottled orange and grey silts and clays.
- Beneath the colluvium, deposits identified as East Coast Bays Formation were encountered to the base of both boreholes. In HA01 these soils comprised hard grey silt and in HA02 medium dense to dense grey sand. Both boreholes met practical refusal.
- Based on a low cone resistance and marked reduction in friction ratio, the results of CPT01 indicate that colluvium was encountered to a depth of 3.0 m in the western debris lobe.

---

[6] Field description of soil and rock, guideline for the field classification and description of soil and rock for engineering purposes, NZGS (2005)

- Underneath the colluvium CPT01 penetrated soils consistent with Puketoka Formation to a depth of 6 m overlying soils of the East Coast Bays formation to 8.78 m where the test met practical refusal.
- CPT02 encountered Puketoka Formation soils to 7.5 m depth, overlying what are inferred to be soils of the East Coast Bays Formation to a depth of 15.75 m where the test met practical refusal.

5.3.1 Groundwater

Groundwater was measured in borehole HA01 at 4.9 m. Groundwater was not measured in HA02 or either of the CPTs. Table 2 presents a summary of groundwater observations at the site, including results from the previous CMW investigation. It should be noted that groundwater levels may fluctuate both seasonally and in the long term.

Table 2: Groundwater Observation Summary

Investigation Locations	Depth to groundwater (m)	Date
HA01	4.9	15/08/2023
HA02	Not encountered	15/08/2023
CPT01	Not measured	16/08/2023
CPT02	Not measured	16/08/2023
HA01-16	2.8	08/11/2016
HA02-16	2.5	08/11/2016
HA03-16	3.5	08/11/2016
HA04-16	2.2	09/11/2016
HA05-16	3.8 (encountered) 3.1 (following borehole completion)	08/11/2016
HA06-16	3.2	10/11/2016
HA07-16	Not encountered	08/11/2023
HA08-16	1.6	08/11/2023

5.4 Laboratory Testing

A soil sample was collected from the Puketoka Formation for Atterberg Limits and Linear Shrinkage testing. This testing was undertaken in accordance with NZS4402:1986. Full results can be found in Appendix 4 and are summarised in Table 3.

Table 3: Laboratory Testing Summary

Sample ID	Sample Depth (m)	Water Content	Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage
S1	0.25 – 0.75	23.1	45	20	25	13

Expansive soils are classified in NZS 3604 as soils with a liquid limit of greater than 50% and a linear shrinkage greater than 15%.

6 Geohazard and Geotechnical Assessment

6.1 Soil Classification

Based on the findings of our desktop and subsurface investigation, as well as our experience of regional ground conditions we consider the preliminary seismic site classification to be 'Class C – Shallow Soil Sites' in line with NZS 1170.5:2004[7] for the purpose of seismic design.

6.2 Seismic Hazards

Potential seismic hazards resulting from nearby moderate to major earthquakes can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, ground lurching, regional subsidence or uplift, soil liquefaction, lateral spreading, and landslides, tsunamis and seiches. Based on topographic and lithological

The following sections present a discussion of seismic hazards as they apply to the site.

6.2.1 Ground Rupture

There are no known active faults located within the site. Based on regional mapping, and the results of our field observations, it is our opinion that fault-related ground rupture is unlikely at the subject property.

6.2.2 Landslides

Landslides, while primarily found to occur during or following high rainfall events, can be triggered by earthquakes. Ground accelerations produced by earthquakes can significantly reduce the stability of inclined masses of soil, particularly where the soil is vulnerable to strain softening.

As the proposed building locations are within the vicinity of sloping ground, consideration must be given to the effects of earthquake loading on the stability of the slope. We have considered these factors in our slope stability analyses, see Section 6.5.

6.2.3 Ground Shaking

Ground shaking and subsequent effects on structures, infrastructure and engineering systems can be extensive. The intensity, frequency and duration of ground shaking drives the effect of earthquake loading on structures, while the severity of ground shaking drives the level of ground deformation.

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[7] Standards New Zealand. (2004). Structural design actions – Part 5: Earthquake actions – New Zealand. Published 21/12/04.

The level of ground shaking to which a building must be designed to withstand is dependent on the building's Importance Level as described in clause A3 of the Building Code. As the planned development is residential, we have assumed all buildings will be Importance Level 2 or lower. According to NZS 1170.5:2004, Importance Level 2 buildings are required to retain their structural integrity and not collapse or endanger life during an earthquake with a 500 year return period; the Ultimate Limit State (ULS) design seismic loading. They are further required to sustain little or no structural damage during an earthquake with a 25 year return period; the Serviceability Limit State (SLS) design seismic loading.

Peak horizontal ground accelerations (a_{max}) in accordance with NZGS Earthquake Geotechnical Engineering Practice Module 1, Appendix A1[8] are 0.19 g (ULS) and 0.05 g (SLS).

6.2.4 Liquefaction Analysis

We have assessed the potential of liquefaction triggering and liquefaction induced settlement occurring at the site by performing liquefaction analyses on the CPT data.

Soil liquefaction and lateral spreading results from the loss of strength during cyclic loading, such as that imposed by earthquakes. Soils most susceptible to liquefaction are typically identified as clean, loose, saturated, cohesionless materials. Empirical evidence indicates that some silty sands, low plasticity silts and low plasticity clays are also potentially liquefiable or may be subject to strain softening. Lateral spreading occurs as a result of liquefied material moving toward a sloping area or free face. This is most common in sloping ground, backfills behind retaining walls, open stormwater channels and water frontage areas. Thin layers, particularly those that are not laterally extensive, are unlikely to liquefy if they are surrounded by non-liquefiable soils.

Liquefaction Methodology

We have assessed the potential of liquefaction triggering and liquefaction induced settlement occurring at the site by performing liquefaction analyses on the CPT data based on the liquefaction triggering methodologies presented by Boulanger and Idriss[9] and using the proprietary software CLiq v.2.3.1.15.

Our analysis included the following assumptions and inputs:

- Ground motion parameters as outlined in Section 6.2.3.
- A maximum earthquake magnitude groundwater level of 1.6 m to reflect the shallowest groundwater level observed within the hand auger boreholes.
- The Zhang and Brachman[10] (2002) procedure for estimating volumetric strain and vertical settlement for the CPT settlement.

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[8] New Zealand Geotechnical Society (NZGS) and Ministry of Business, Innovation and Employment (MBIE) (2021). Earthquake geotechnical engineering practice Module 1: Overview of the guidelines, Version 1, November 2021.

[9] Boulanger, R.W. and Idriss, I.M. (2014). CPT and SPT based liquefaction triggering procedures. Centre for Geotechnical Modeling. Department of Civil & Environmental Engineering, University of California. Report No. UCD/CGM-14/01. April 2014.

[10] Zhang, G.; Robertson, P.K.; and Brachman, R.W.I. (2002). Estimating liquefaction-induced ground settlements from CPT for level ground. Canadian Geotechnical Journal 39: 1168–1180. DOI: 10.1139/T02-047

- The Boulanger and Idriss relationship between fines content and Soil Behaviour Type (Ic) with a fitting parameter (CFC) of 0.0 for the CPT analysis (no soil laboratory testing available for calibration of the parameter.

Liquefaction Discussion

Full results of our analyses are presented in Appendix 5, a summary only is presented in Table 4 below:

Table 4: Liquefaction Analysis Summary

CPT	LPI	LSN	Calculated vertical Settlement (SLS)	Calculated Vertical Index Settlement (ULS)	MBIE Module 3 Performance Level
CPT01	Negligible	<1	Negligible	< 2mm	L_0
CPT02	1	1.5	Negligible	10 mm	L_1

LPI and LSN presented are for ULS case.

Our analysis indicates that under SLS conditions the site soils are not vulnerable to liquefaction. Under ULS conditions limited liquefaction is predicted in sporadic sandy layers within the Puketoka Formation.

In CPT01, minor liquefaction is predicted in very thin (< 0.2 m), isolated strata between 4.0 m and 6.5 m depth. These strata are sufficiently thin that the likelihood of these layers liquefying is considered negligible. Should liquefaction trigger in these layers < 2 mm of settlement is predicted with little to no surface expression.

In CPT02, liquefaction is predicted to occur in several < 0.5 m thick layers between 6.5 and 9.5 m depth. A Liquefaction Potential Index (LPI) of < 2 indicates a low risk of liquefaction triggering. Should liquefaction trigger, 10 mm of global settlement is predicted with approximately half of this to be expressed as differential settlement at the surface. The low Liquefaction Severity Number (LSN) predicts that there will be little to no surface expression to liquefaction.

Table 5.1 of MBIE / NZGS Module 3[11] indicates that the ULS liquefaction induced settlements on this site are within the insignificant to mild categories (L_0 and L_1). The consequences are described as ‘No significant excess pore water pressures (no liquefaction’ and ‘Limited excess pore water pressures; negligible deformation of the of the ground and small settlements’).

6.3 Expansive Soils

Expansive soils shrink and swell as a result of seasonal fluctuation in moisture content. This can cause heaving and cracking of on-grade slabs, pavements and structures founded on shallow foundations.

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[11] New Zealand Geotechnical Society (NZGS) and Ministry of Business, Innovation and Employment (MBIE) (2021). Earthquake geotechnical engineering practice Module 3: Identification, assessment and mitigation of liquefaction hazards, November 2021.

Building damage due to volume changes associated with expansive soils can be reduced through proper foundation design and construction. It is imperative that exposed soils be kept moist prior to placement of concrete for foundation construction. It is extremely difficult to re-moisturise clayey soils without excavation, moisture conditioning and re-compaction.

The results of our laboratory testing indicates that with reference to NZS 3604.11[12] that the near surface soils at site fall within the bounds of 'good ground' concerning expansive soils. Although our laboratory test results indicate non expansive soils, based on our experience with similar soils within the Hobsonville area we consider a preliminary soil classification of M (moderately) expansive with respect to NZS 3604 (from Section 3.2 of B1/AS1 November 2019 Amendment) is suitable for this site.

It is considered that this preliminary recommendation may be refined with further site-specific testing at the Geotechnical Completion Report stage, following earthworks.

6.4 Coastal Regression Hazard

The northern boundary of the site has been identified by Auckland Council as being potentially susceptible to coastal instability and erosion. The potential regression lines for 2050, 2080 and 2130 are mapped within the proposed council esplanade area and are shown in Figure 2. As such, a site-specific coastal hazard assessment undertaken by a Coastal Engineer will be required to support a Resource Consent application.

6.5 Slope Stability

ENGEO has completed slope stability analyses for the two landslide features on-site in order to determine the nature of the existing failures and to assess the feasibility of potential mitigation methods to reduce the risk of future movement of these features from influencing the development. Our assessment methodology was as follows:

- Determination of a geological cross section through both features using the results of our investigation and understanding of the local geology. A-A' crosses the western feature while B-B' passes through the eastern feature.
- Back analyses of the existing slope using the proprietary software SLIDE2 to determine the soil parameters and likely failure surfaces of existing features under three conditions.
    - Long term static conditions – observed groundwater levels.
    - Short term transient conditions – elevated groundwater levels.
    - Seismic conditions – 500 year return period event.
- Slope stability analysis using SLIDE2 to determine a suitable mitigation method that satisfies the design criteria for residential development in Auckland in the above three conditions.

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[12] Standards New Zealand. Timber-framed buildings – New Zealand. (2011).

6.5.1 Design Criteria

The requisite factors of safety (FoS) for residential development in Auckland are outlined in Table 5.

Table 5: Slope Stability Factor of Safety Requirements

Scenario	Requisite Factor of Safety
Long term static conditions	1.5
Short term transient conditions	1.3
Seismic Conditions	1.0

These FoS have been assessed using Spencer and GLE-Morgenstern Price methods for non-circular failure. Based on the presence of saturated, marshy soils on the elevated portions of the site, our analyses of the transient groundwater condition has considered soils to be fully saturated. For the seismic scenario ULS peak ground acceleration as determined in Section 6.2.3 has been adopted (0.19 g).

Our analysis has not considered the surcharge of any building loads or loading from placed fill. These will need to be considered as part of detailed design.

6.5.2 Material Parameters

Material parameters were adopted for our slope stability and remediation analyses based on *in situ* testing within our hand augers, Su and CPT correlations, local experience and back analysis of colluvial soils.

The failure surface between the colluvium and underlying soils was modelled as a thin layer of low strength material and a preferential failure surface plotted within it.

A summary of these derived parameters is presented in Table 6.

Table 6: Geotechnical Parameters

Geological Unit	Unit Weight (kN/m^3)	Effective Stress Parameters		
		Ø' (°)	c' (kPa)	Undrained Shear Strength (kPa)
Tauranga Group Alluvium	18	28	3	80
Colluvium	16	23	2	50
East Coast Bays Formation Soils	18	32	5	n/a
Shear Zone	15	14	0	-
East Coast Bays Formation Rock	20	40	10	-

6.5.3 Slope Stability Results

Outputs of our slope stability analyses are presented in Appendix 6.

The results of our analysis indicate that under static conditions, both landslides are comparatively stable, with minor failures predicted internally within the colluvium, but no remobilisation of the failed soils indicated. This is consistent with our site observations of creep features and shallow failures towards the toe of the slope (Photo 4).

Under the transient condition, FoS are close to 1.0 with failures predicted along the entire base of the colluvium which suggests that these conditions are likely to have resulted in the initial slope failure. Under seismic conditions A-A’ is unstable with internal failures predicted within the colluvium and B-B’ is stable. Failures in A-A’ exit through the narrowest part of the colluvium which indicates that failures of this type are dependent on the current landform and are less likely to have resulted in the initial failure.

A summary of our analyses results are presented below:

Table 7: Summary of Slope Stability Analyses

Section	Condition	Factor of Safety
A-A’	Static	1.95
	Transient	1.03
	Seismic	0.78
B-B’	Static	1.47
	Transient	0.62
	Seismic	1.20
Note: red = FoS below requirements, green = satisfies requirements		

Discussion

It should be noted that there is a degree of uncertainty in the results of our analysis. As no machine borehole testing was carried out, and hand auger drilling results in significant soil disturbance, it is possible that the failure plane is in a different position to that plotted within our models.

Furthermore, the presence of two failures at site indicates that there is a risk of further failures along the border of the inlet. Structural data provided by the GNS indicate that the soils at site dip at 20° to the southeast, directly away from the inlet in the location of the intact slope. As the direction of failure of the existing landslides is oblique to the dip direction it may be the case that dip provides a structural control on slope failures in this location, this cannot be confirmed without further deep investigation and analysis.

6.5.4 Remediation

In order to address the risk of slope failure influencing the development and maintain the requisite factors of safety, in ground palisade walls located along the boundary of the residential lots may be a suitable solution (although this does not improve the FoS of the esplanade reserve area). A 10 m pile was modelled for section A-A’ and a 9 m pile was modelled for section B-B’. The results of our analysis are presented in Appendix 7.

For section A-A’ a minimum 300 kN pile shear strength at 1.0 m centres is required. This would necessitate the use of large diameter reinforced concrete piles.

Section B-B’ requires a minimum pile shear strength of 100 kN at 1.0 m centres which may be achieved using timber piles.

These remediation measures are conceptual only as earthworks plans have not yet been developed for the sites, and changes in existing levels will have a corresponding effect on the slope stability which will need to be taken into account at the detailed design stages.

A summary of our analyses is presented in Table 8.

Table 8: Palisade Wall Stability Analysis

Section	Condition	Factor of Safety
A-A’	Static	2.15
	Transient	1.98
	Seismic	1.86
B-B’	Static	1.36*
	Transient	1.36
	Seismic	1.37
Note: green = achieves requisite FoS *Failure surfaces with FoS <1.5 are limited to the outer extent of the site and do not significantly affect the developable area		

6.6 Settlement

The Puketoka Formation comprises alluvial sediments. In alluvial environments, peat forms in areas with low sediment input, typically on the margins on small, slow flowing channels. These become buried beneath sediment as the channel migrates subsequently forming a peat containing paleo-channel. Although not encountered at this site, peat and organic soils have been encountered at other sites in the area and extensive organic deposits are known to be present south of the site in the vicinity of the Upper Harbour Motorway.

Peat is considered an unacceptable bearing stratum for foundations as it is highly susceptible to consolidation due to its high-water content (peat may contain ten times its own weight in water). Under the load of fill and building foundations, peat can reduce its volume by up to 75% resulting in significant vertical settlement. Peat is also vulnerable to wasting where it is found above the groundwater table as oxidation of the biomass results in the peat decaying / decomposing. Primary settlement of peat may take days whereas secondary creep consolidation settlement behaviour due to the decay of organic material may continue over 50+ years.

We should be given the opportunity to review the earthworks proposals for the site when they are developed, prior to building consent, to assess whether the magnitude of cut or fill earthworks may present a settlement risk to the development. Additional investigations may be recommended to confirm the presence or absence of organic or otherwise weak / compressible soils in the vicinity of deep excavations or large fills to appropriately characterise the settlement risk. Where potential consolidation settlements are found to be beyond building code tolerances, suitable solutions may include undercutting and replacing the peat with engineered fill or piled foundations extending below the peat.

6.7 RMA Section 106 Assessment and Development Suitability

Section 106 of the Resource Management Act (RMA) states that a consent authority may refuse to grant a Subdivision Consent, or may grant a consent subject to specific consent conditions if it considers that:

- There is significant risk from natural hazards; or
- Sufficient provision has not been made for legal or physical access to each allotment to be created by the subdivision.

An assessment of the risk from natural hazards as required by the RMA includes the following:

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

We have assessed the risk of natural hazards at the site in accordance with Section 106 of the Resource Management Act (RMA) and considered the risk to the site from erosion, rockfall, inundation (debris), slope stability, subsidence, flooding and tsunami.

Based on our investigation, assessment and site observations, we consider it is unlikely for the site to be subject to the aforementioned natural hazards providing suitable engineering measures are included in the site development (as discussed in Section 7). As such, the site is considered to be conditionally suitable for the proposed residential development from a geotechnical perspective.

7 Geotechnical Recommendations

Based on the results of our geotechnical investigation and subsequent assessment, we consider the site to be generally suitable for the proposed development subject to our geotechnical recommendations being followed.

However, as mentioned in Section 6 the site is at risk from a number of identified geohazards including the following:

- Instability of the over steepened north-western slope bordering Waiarohia Inlet.
- Portions of the site may be vulnerable to settlement due to the potential presence of compressible alluvial soils.
- Shallow site soils may be susceptible to shrinkage and heave.

7.1 Foundations

Shallow soils at the site typically comprised very stiff to hard clays and silts of the Puketoka Formation. It is our preliminary recommendation that site soils will likely be suitable for a geotechnical ultimate bearing capacity for shallow foundations constructed on competent natural ground beneath any topsoil and existing undocumented fill or on engineer certified fill.

This preliminary recommendation will be revisited once an earthwork plan has been provided as significant cuts may expose weaker soil horizons with a reduced bearing capacity. Any bearing capacities provided during the design phase are subject to change and revision in the geotechnical completion report to be issued for the site following the satisfactory completion of earthworks.

It is considered likely that the soils on-site may be M (moderately) expansive with respect to NZS 3604 (from Section 3.2 of B1/AS1 November 2019 Amendment). This will be reassessed as part of the completion reporting for this site.

7.2 Earthworks

- As noted in Section 5, possible undocumented fill is present on-site. Any undocumented fill soils should be undercut to the depth that native soils are exposed.
- Excavations and temporary cuts should not exceed a batter angle of 1V:2H up to 2 m in height and should not be left unsupported for longer than two weeks. Cuts beyond this height should be referred to the Geotechnical Engineer for stability assessment.
- Where vertical and subvertical faces higher than 1.0 m are required, we recommend that this is done in shortened sections (< 5 m) and the faces are left unsupported for a minimal time period (i.e., one week) or temporarily shored.
- All temporary cuts and batters proximal to boundaries should take into account the potential surcharge and risk of undermining neighbouring property.
- Suitable drainage channels must be put in place to divert surface water from unsupported cut faces. Subsurface drains should also be considered for the toe of the long-term slopes.
- If any permanent cuts have a batter steeper than 1V:4H and are to be higher than 1.5 m, they should be supported with a specifically designed retaining wall (approved by a chartered Geotechnical Engineer) or be referred back to the Geotechnical Engineer for stability assessment and specific batter design.

- All cuts and batters should be undertaken in line with the WorkSafe Good Practice Guidelines for Excavation Safety (July 2016). Permanent fill batters should not exceed 1V:3H and should be reviewed by the Geotechnical Engineer as part of the site development and earthworks proposal review. Fill batters exceeding 1V:3H will require specific geotechnical assessment.

- All excavations should be inspected by ENGEO (or a suitably qualified Geotechnical professional), prior to constructing foundation elements to verify founding conditions are as anticipated.

- Suitable underfill drainage should be considered for any filling on slopes, within stream gully features and wherever seepage is observed within the stripped surface.

- All engineered or structural fill should be placed in ≤ 200 mm compacted lifts and be compacted to a minimum of 95% of maximum dry density, at no less than optimum moisture content. Maximum dry density for granular fill materials may be obtained from the source quarry, a geotechnical laboratory or from plateau testing undertaken on-site. Compaction should be achieved using standard plant and methodology suitable for the imported material. A water source should be maintained on-site for moisture control.

- All excavated soil should be removed from site or placed in an engineer approved stockpile to avoid unfavorable loading on construction or preconstruction slope batters.

Material Suitability

Earthworks' operations involving borrow materials, usually from the elevated portions of the site, should be relatively straightforward. Generally, both the cuts and fills will involve inorganic, alluvial clayey silts and silty clays that should be suitable, with conditioning for handling and compaction by conventional earthmoving plant. It should be noted though that moisture contents will increase with depth in the cut areas and also in the lower lying areas.

Our experience with the types of native soils present on this site indicates that when they are exposed to the weather their strengths may be significantly reduced. We therefore recommend that trafficked areas and building platforms are only trimmed to final levels immediately prior to placing hardfill / topsoil and that at all times the site is shaped to avoid water ponding during rain, thereby limiting the need for additional undercuts. On no account should areas of trimmed subgrade be left exposed to allow the ingress of water, nor should subgrade areas be trafficked prior to drying out after rain.

Unsuitables

Topsoil and organic soils are not suitable for bearing foundations or for reworking and re-use as engineered fill and should be undercut and stockpiled away from the earthworks area. Undocumented fills encountered on-site may be suitable for re-use as engineered fill following approval of the Geotechnical Engineer.

7.3 Service Lines

The construction and installation of new services lines within alluvial material may intercept flowable sands and organic / peat layers. Particular attention should be paid to drainage and stability of trench walls under such circumstances.

Where the base of service line trenches encounters weak, flowable sands and / or organic soils, increased bedding depths of up to 70% and undercuts of approximately 300 mm plus geotextile wrapping of the bedding may be required to provide adequate support to the services and limit the chance of differential settlement along low gradient service alignment. Specific bedding modifications are best prescribed when the trenches are excavated and the material at invert level are examined in detail by a geotechnical professional.

Construction of services during the winter months may pose a risk of trench wall collapse within soft alluvial soils partly due to raised groundwater, leading to the need for additional support, alternative construction methodology and / or dewatering. This should be allowed for on-site by the contractors. Methods to deal with this could be, but not limited to, trench shields to support service trench walls, benching or excavations to a safe temporary works angle (e.g., 1):H): 1(V)).

Should flowable sands and / or organic soil layers be encountered during service line trenching, the contractor shall contact ENGEO.

7.4 Retaining Walls

7.4.1 Internal Retaining Walls

Currently there are no internal retaining structures shown on the development plans. Any future retaining should be designed to accommodate for the soils encountered on-site. Based on our subsurface investigations, we expect internal retaining structures to support native Puketoka Formation.

Preliminary Retaining Wall Parameters

Based on the results of our investigation and the ground conditions at site, future retaining walls should be designed using the following geotechnical parameters:

Table 9: Retaining Wall Parameters

Material Type	Unit Weight	Friction Angle (°)	Effective Cohesion c’ (kPa)	Undrained shear Strength Su (kPa)
Puketoka Formation	18	28	3	80
Cohesive Engineered Fill	18	32	5	100
Granular Engineered Fill	20	38	0	-

7.4.2 Boundary Palisade Walls

Palisade walls constructed to stabilise the landslides on-site will require specific geotechnical investigation and design. Deep boreholes through the debris lobe will be required to confirm the location of the existing failure plane.

Additionally, we recommend further intrusive geotechnical investigation to assess the slope crest that has not failed and also note that it may be prudent to extend a future palisade wall along the entire north-western boundary of the residential lots.

These walls should be designed to support any future building loads or loading resulting from earthworks.

7.5 Stormwater and Effluent Disposal

ENGEO have not been provided with plans showing the preferred methods of stormwater and wastewater disposal.

Based on the preliminary plans that have been provided we anticipate that wastewater will be disposed of via reticulated Council services.

Due to the proximity of the steep and unstable slopes to the proposed development, we do not recommend in-ground soakage systems are adopted for the site. All stormwater collected from hard standing areas and roofing should be collected and reticulated to Council services.

Overland flows should be directed away from existing slopes to reduce the risk of ponding and erosion exacerbating slope instability concerns.

7.6 Pavement Subgrade CBR

Inferred CBRs of approximately 3% may be adopted for native soils and 6% for cohesive engineered fill areas are considered to be suitable for preliminary design purposes. These values are derived from the soils encountered in our hand auger boreholes and our knowledge of the soil type on-site.

The above CBR values are preliminary only. Specific *in situ* and laboratory testing of the exposed subgrade is recommended following earthworks and prior to finalising pavement designs, including the use of *in situ* and soaked CBR testing and falling weight deflectometer. Where localised uncontrolled fill is encountered, it will be necessary to remove this fill and replace it with engineered fill. Additional subgrade improvement requirements may be necessary to achieve council requirements. This may include undercut and replacements, and / or the use of triaxial geogrid.

8 Future Work

We recommend ENGEO's involvement in the following future activities:

- Deep machine borehole investigations to fully characterise the landslide features on-site prior to design of mitigation methods.
- Detailed review of landform / earthworks design and revised slope stability analysis to reflect design ground profiles in the context of slope instability and potentially compressible soils.
- Preparation of a Geotechnical earthworks specification.
- Observation and certification of earthworks and retaining walls including all stripping and undercuts and engineered fill in accordance with the earthworks and retaining wall specifications.
- Geotechnical Completion Reporting / Producer Statements.

9 Limitations

i. We have prepared this report in accordance with the brief as provided. This report has been prepared for the use of our client, Cabra Developments Limited, their professional advisers and the relevant Territorial Authorities in relation to the specified project brief described in this report. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity.

ii. The recommendations in this report are based on the ground conditions indicated from published sources, site assessments and subsurface investigations described in this report based on accepted normal methods of site investigations. Only a limited amount of information has been collected to meet the specific 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 judgement and it should be appreciated that actual conditions could vary from the assumed model.

iii. 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.

iv. This Limitation should be read in conjunction with the Engineering NZ/ACENZ Standard Terms of Engagement.

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

We trust that this information meets your current requirements. Please do not hesitate to contact the undersigned on (09) 972 2205 if you require any further information.

Report prepared by

Jamie Lott

Geotechnical Engineer

Report reviewed by

Paul Fletcher, CMEngNZ (CPEng)

Principal Geotechnical Engineer

Heather Lyons, CMEngNZ (PEngGeol)

Associate Engineering Geologist

APPENDIX 1:

Geotechnical Investigation Plan

Legend

- ⊕ ENGEO Hand Augers
- ▲ ENGEO CPT (August 2023)
- ⊕ CMW Hand Auger Boreholes (November 2016)
- - - - Head Scarp
- — Cross Section B-B'
- — Cross Section A-A'
- ☐ Site Boundary

0 25 m 50 m

© Nearmap, , public,

ENGEO

Produced by **Datanest.earth**

Title: Investigation location Plan

Client:
Cabra Developments Limited

Project:
10 Sinton Road…

Drawn: JL

Figure No.:
2

Size:
A4

Date: 10-11-2023

Checked: P..

Proj No.:
23849.000.001…

Scale:
1:1500

Version:
1

APPENDIX 2A:

ENGEO Hand Auger Logs

ENGEO

LOG OF AUGER HA01

Geotechnical Investigation
10 Sinton Road
Whenuapai, Auckland
23849.000.001_03

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_03
Date : 15-08-2023
Hole Depth : 4.9 m
Hole Diameter : 50 mm

Shear Vane No : 3840
Logged By : KE
Reviewed By : JL
Latitude : -36.7928307
Longitude : 174.6421347

Material	USCS Symbol	DESCRIPTION	Moisture Cond.	Consistency/ Density Index
TS	OL	TOPSOIL	W	NA
COLLUVIUM	ML	SILT; light brown. Low plasticity.	M	St
	ML	Fine sandy SILT; light grey. Low plasticity.		St
	CH	Silty CLAY with trace fine sand; light grey with orange brown mottling. High plasticity.		St
	ML	Fine sandy SILT; light grey with orange brown streaks. Low plasticity.		St
	CH	Silty CLAY; orange brown with light grey streaks. High plasticity.		VSt
	CH	CLAY with some fine sand; light grey with orange brown streaks. High plasticity.		St
	CH	CLAY; orange brown with light grey streaks. High plasticity.		St - VSt
	CH	Silty CLAY; light grey with orange brown streaks. High plasticity.	W	VSt
	ML	Fine sandy SILT with some clay; grey. Low plasticity.		VSt
EAST COAST BAYS FORMATION	ML	SILT; dark grey. Low plasticity.		H

End of Hole Depth: 4.9 m
Termination Condition: Practical refusal

Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
96/16
80/33
83/41
74/28
125/36
86/28
91/28
97/33
183/34
157/44
152/47
220+
220+
220+
220+
220+

Scala Penetrometer
Blows per 100mm
2 4 6 8 10 12

Hand Auger met practical refusal at 4.9 m depth. due to hole collapse
Dip test showed standing water at 4.9 m depth.
Coordinates obtained via handheld GPS.
Elevation obtained via Auckland Council GIS.

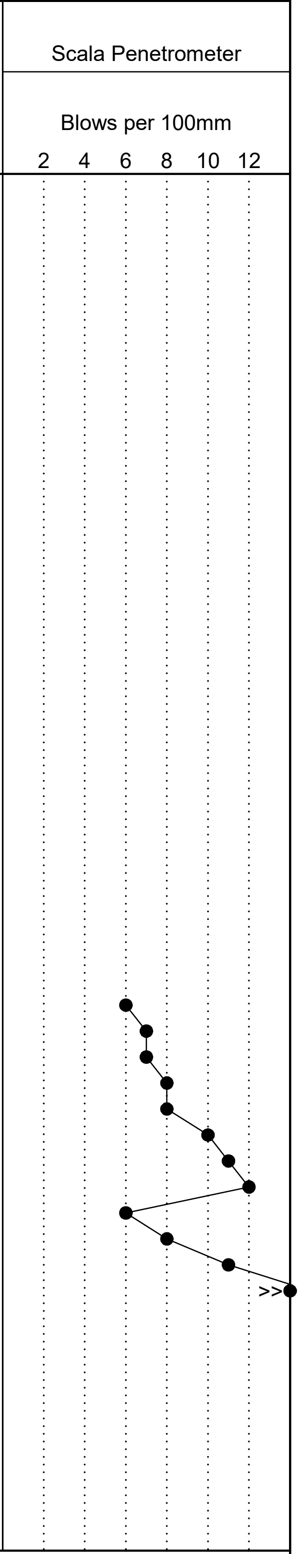
TS = Topsoil; NA = Not Assessed.

LOG OF AUGER HA02

Geotechnical Investigation
10 Sinton Road
Whenuapai, Auckland
23849.000.001_03

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_03
Date : 15-08-2023
Hole Depth : 4.5 m
Hole Diameter : 50 mm
Shear Vane No : 2853
Logged By : LM
Reviewed By : JL
Latitude : -36.7926733
Longitude : 174.6420805

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
	TS	OL	TOPSOIL				W	NA	
0.5	COLLUVIUM	CH	Silty CLAY; light brown with orange mottling. High plasticity.		7		M	VSt	125/73 119/63
1.0		ML	Clayey SILT with trace fine sand; light grey with orange streaks. Low plasticity.					St - VSt	96/59 121/33
1.5 2.0 2.5		ML	Fine sandy SILT with trace clay; light grey with orange mottling. Low plasticity. 1.7 m: Becomes orange with light grey mottling. 1.9 to 2.1 m: Encountered limonite granules. 2.8 m: Encountered limonite granules.		6 5			St - VSt	102/27 85/32 79/29 111/32 102/42
3.0		CH	Silty CLAY with some fine sand; orange brown with light grey streaks. High plasticity.					VSt	157/50
3.5 4.0 4.5	EAST COAST BAYS FORMATION	SM	Silty fine to medium SAND interbedded with sandy SILT with minor clay; grey. Beds are 100-200 mm thick and sub horizontal. 4.0 m: Becomes wet.		4		W	MD - D	
5.0			End of Hole Depth: 4.5 m Termination Condition: Practical refusal						



Hand auger met practical refusal at 4.5 m depth. on hard material
Scala Penetrometer met practical refusal at 4.3 m depth.
Standing groundwater was not encountered
Coordinates obtained via handheld GPS.

Elevation obtained via Auckland Council GIS.
TS = Topsoil; NA = Not Assessed.

APPENDIX 2B:

Previous Hand Auger Logs

HAND AUGER BOREHOLE - HA01-16

Client: Cabra Developments Limited
Project: 10 Sinton Road
Site Address: Hobsonville
Project No: AKL2016_0605
Date: 08/11/2016
Borehole Location: Refer to site plan.

CMW Geosciences

1:25 Sheet 1 of 1

Logged by: KP
Checked by: JF

Position: E.1746546.0m N.5926615.0m (NZTM)
Survey Source: Hand Held GPS

Elevation:
Datum:

Hole Diameter: 50mm
Angle from horizontal: 90°

Unit	Groundwater	RL (m)	Depth (m)	Graphic Log	Material Description Soil: USC; Soil type; colour; structure; strength; moisture; bedding; plasticity; sensitivity; additional comments Rock: Weathering; colour; fabric; rock name; strength; additional comments	Moisture Condition	Sensitivity	Shear Strengths (kPa) Peak (Residual)	Dynamic Cone Penetrometer (Blow/100 mm) 5 10 15 20	Comments
Tauranga Group					OL: TOPSOIL					
					CH: Silty CLAY: orange with limonite staining. Very stiff, moist, high plasticity.					
					...becoming light grey		MS	V-139(61)		
							MS	V-142(70)		
			1							
							IS	V-122(65)		
						M				
							MS	V-163(75)		
			2		CH: Silty CLAY: grey with orange streaks. Very Stiff, moist, high plasticity.		MS	V-148(70)		
							MS	V-119(49)		
	▼				CH: Sandy CLAY with minor silt: grey, streaked orange. Stiff, wet, high plasticity. Sand is fine grained.		IS	V-87(49)		
			3			W				
					CH: Sandy CLAY with minor silt: orange, streaked grey. Stiff, moist, high plasticity. Sand is fine grained.		IS	V-105(58)		
					CH: Silty CLAY: orange, streaked grey. Very stiff, moist, high plasticity.		MS	V-113(55)		
					CH: Sandy CLAY: grey. Hard, moist, high plasticity. Sand is fine grained.					
			4					V-UTP		
						M				
								V-UTP		
								V-UTP		
			5		Borehole terminated at 5.0m					

Termination reason: Target Depth Reached

Remarks: Standing groundwater was encountered at a depth of 2.8m

This report is based on the attached field description for soil and rock, New Zealand, Geotechnical Society Inc 2005.

HAND AUGER BOREHOLE - HA02-16

Client: Cabra Developments Limited
Project: 10 Sinton Road
Site Address: Hobsonville
Project No: AKL2016_0605
Date: 08/11/2016
Borehole Location: Refer to site plan.

1:25 Sheet 1 of 1

Logged by: RHD
Checked by: JF

Position: E.1746547.0m N.5926727.0m (NZTM)
Survey Source: Hand Held GPS

Elevation:
Datum:

Hole Diameter: 50mm

Unit	Groundwater	RL (m)	Depth (m)	Graphic Log	Material Description Soil: USC; Soil type; colour; structure; strength; moisture; bedding; plasticity; sensitivity; additional comments Rock: Weathering; colour; fabric; rock name; strength; additional comments	Moisture Condition	Sensitivity	Shear Strengths (kPa) Peak (Residual)	Dynamic Cone Penetrometer (Blow/100 mm) 5 10 15 20	Comments
Tauranga Group					OL: TOPSOIL					
					CH: CLAY with minor silt: light brown, streaked orange. Very stiff, moist, high plasticity.		MS	V-125(48)		
					...grading; light grey, streaked orange		MS	V-154(69)		
			1			M	MS	V-134(66)		
							IS	V-147(75)		
			2				IS	V-111(61)		
	▼				CH: CLAY: light grey, streaked orange. Stiff, moist to wet, high plasticity.	M to W	IS	V-83(46)		
					...becoming wet	W	IS	V-72(39)		
			3				IS	V-62(35)		
					CH: CLAY with minor silt and minor fine sand: light grey/orange. Very stiff, wet to saturated, high plasticity.	W to S	MS	V-103(42)		
			4		CH: CLAY with minor silt: grey. stiff, moist to wet, high plasticity.	M to W	MS	V-93(46)		
					CH: CLAY with some fine sand: grey. Stiff, saturated, high plasticity.	S	MS	V-91(44)		
					SC: Sandy CLAY: grey. Stiff, saturated, low plasticity. Sand is fine to medium grained.		MS	V-97(40)		
			5		Borehole terminated at 5.0m					

Termination reason: Target Depth Reached

Remarks: Standing groundwater was encountered at a depth of 2.5m. Groundwater was dipped at a depth of 1.5m following borehole completion.

This report is based on the attached field description for soil and rock, New Zealand, Geotechnical Society Inc 2005.

HAND AUGER BOREHOLE - HA03-16

Client: Cabra Developments Limited
Project: 10 Sinton Road
Site Address: Hobsonville
Project No: AKL2016_0605
Date: 08/11/2016
Borehole Location: Refer to site plan.

CMW Geosciences

1:25 Sheet 1 of 1

Logged by: RHD
Checked by: JF

Position: E.1746517.0m N.5926712.0m (NZTM)
Survey Source: Hand Held GPS

Elevation:
Datum:

Hole Diameter: 50mm

Unit	Groundwater	RL (m)	Depth (m)	Graphic Log	Material Description Soil: USC; Soil type; colour; structure; strength; moisture; bedding; plasticity; sensitivity; additional comments Rock: Weathering; colour; fabric; rock name; strength; additional comments	Moisture Condition	Sensitivity	Shear Strengths (kPa) Peak (Residual)	Dynamic Cone Penetrometer (Blow/100 mm) 5 10 15 20	Comments
Tauranga Group					OL: TOPSOIL	M				
					CH: CLAY with some silt: brown, mottled orange/light grey. Very stiff, moist, high plasticity.		MS	V-125(42)		
					...grading; light grey, streaked light brown					
								V-194+		
					...becoming hard					
					CH: CLAY: light grey, streaked orange. Very stiff to hard, moist, high plasticity.					
			1							
								V-194+		
							IS	V-166(104)		
			2				IS	V-140(86)		
					CL: CLAY with some silt: light grey, streaked orange. Stiff, moist, low plasticity.					
					...grading; high plasticity		IS	V-83(44)		
					...becoming moist to wet	M to W				
							MS	V-82(39)		
			3		CL: CLAY with some silt and trace fine sand: white/light grey, mottled orange. Stiff, wet, low plasticity.	W				
							MS	V-69(32)		
	▼									
					CL: Silty CLAY: light grey. Stiff, saturated, low plasticity.	S	MS	V-66(32)		
			4				IS	V-72(47)		
							IS	V-78(50)		
							IS	V-78(44)		
			5		Borehole terminated at 5.0m					

Termination reason: Target Depth Reached

Remarks: Standing groundwater was encountered at a depth of 3.5m.

This report is based on the attached field description for soil and rock, New Zealand, Geotechnical Society Inc 2005.

HAND AUGER BOREHOLE - HA04-16

Client: Cabra Developments Limited
Project: 10 Sinton Road
Site Address: Hobsonville
Project No: AKL2016_0605
Date: 09/11/2016
Borehole Location: Refer to site plan.

1:25 Sheet 1 of 1

Logged by: KP
Checked by: JF

Position: E.1746523.0m N.5926747.0m (NZTM)
Survey Source: Hand Held GPS

Elevation:
Datum:

Hole Diameter: 50mm
Angle from horizontal: 90°

Unit	Groundwater	RL (m)	Depth (m)	Graphic Log	Material Description Soil: USC; Soil type; colour; structure; strength; moisture; bedding; plasticity; sensitivity; additional comments Rock: Weathering; colour; fabric; rock name; strength; additional comments	Moisture Condition	Sensitivity	Shear Strengths (kPa) Peak (Residual)	Dynamic Cone Penetrometer (Blow/100 mm) 5 10 15 20	Comments
Tauranga Group					OL: TOPSOIL	M				
					CL: Silty CLAY: orange/brown. Very Stiff, moist, low plasticity. Friable.					
					CL: Silty Clay: grey. Very Stiff, moist, low plasticity,			V-UTP		
					CH: CLAY: grey. Very Stiff, moist, high plasticity.		MS	V-134(58)		
			1				MS	V-160(75)		
							MS	V-157(64)		
			2				IS	V-116(58)		
	▼				CL: CLAY: grey. Stiff, saturated, low plasticity.	S				
					...becoming wet.	W	IS	V-84(44)		
					CH: Sandy CLAY: orange, streaked brown. Very stiff, moist, high plasticity.	M	IS	V-116(55)		
			3				IS	V-102(51)		
					CH: Sandy CLAY: grey. Very stiff, moist, low plasticity.		MS	V-109(49)		
			4				MS	V-131(55)		
							MS	V-138(61)		
					CH: CLAY: grey. Very stiff, moist, high plasticity.		MS	V-152(67)		
			5		Borehole terminated at 5.0m					

Termination reason: Target Depth Reached

Remarks: Standing groundwater was encountered at a depth of 2.2m

This report is based on the attached field description for soil and rock, New Zealand, Geotechnical Society Inc 2005.

HAND AUGER BOREHOLE - HA05-16

Client: Cabra Developments Limited
Project: 10 Sinton Road
Site Address: Hobsonville
Project No: AKL2016_0605
Date: 08/11/2016
Borehole Location: Refer to site plan.

1:25 Sheet 1 of 1

Logged by: RHD
Checked by: JF

Position: E.1746501.0m N.5926787.0m (NZTM)
Survey Source: Hand Held GPS

Elevation:
Datum:

Hole Diameter: 50mm

Unit	Groundwater	RL (m)	Depth (m)	Graphic Log	Material Description Soil: USC; Soil type; colour; structure; strength; moisture; bedding; plasticity; sensitivity; additional comments Rock: Weathering; colour; fabric; rock name; strength; additional comments	Moisture Condition	Sensitivity	Shear Strengths (kPa) Peak (Residual)	Dynamic Cone Penetrometer (Blow/100 mm) 5 10 15 20	Comments
Tauranga Group					OL: TOPSOIL					
					CL: Silty CLAY: light brown, mottled orange. Very stiff, moist, low plasticity.		MS	V-166(62)		
					CH: CLAY with some silt: light brown, streaked grey. Very stiff, moist, high plasticity. Occasional topsoil inclusion.		MS	V-159(62)		
			1							
					CH: Sandy CLAY: light grey, streaked orange. Very stiff, moist, low plasticity. Sand is fine to medium grained.	M		V-194+		
					...becoming low plasticity					
							MS	V-183(80)		
					...with minor fine sand					
			2		CH: CLAY: light grey, streaked light brown. Very stiff, moist, high plasticity.			V-194+		
					CL: CLAY with some silt, minor fine sand : cream, streaked orange. Very stiff, moist, low plasticity.		MS	V-173(83)		
					CH: CLAY with some fine sand: light grey/orange. Very stiff, moist to wet, high plasticity. Minor fine to medium sized limonite clasts.	M to W				
					CH: CLAY with minor fine sand: cream, streaked orange. Very stiff, moist, high plasticity.		S	V-152(35)		
			3		*...becoming orange with minor fine to coarse gravel limonite clasts*					
	▼					M	IS	V-83(50)		
							IS	V-71(39)		
	▽				CH: CLAY: light grey/orange. Stiff, wet to saturated, high plasticity.					
			4			W to S	IS	V-68(37)		
							IS	V-78(39)		
					CH: CLAY: dark grey. Very stiff, moist, high plasticity.	M	MS	V-115(48)		
			5		Borehole terminated at 5.0m					

Termination reason: Target Depth Reached

Remarks: Standing groundwater was encountered at a depth of 3.8m. Groundwater was dipped at a depth of 3.1m following borehole completion.

This report is based on the attached field description for soil and rock, New Zealand, Geotechnical Society Inc 2005.

HAND AUGER BOREHOLE - HA06-16

Client: Cabra Developments Limited
Project: 10 Sinton Road
Site Address: Hobsonville
Project No: AKL2016_0605
Date: 10/11/2016
Borehole Location: Refer to site plan.

1:25

Sheet 1 of 1

Logged by: KP
Checked by: JF

Position: E.1746485.0m N.5926784.0m (NZTM)
Survey Source: Hand Held GPS

Elevation:
Datum:

Hole Diameter: 50mm
Angle from horizontal: 90°

Unit	Groundwater	RL (m)	Depth (m)	Graphic Log	Material Description Soil: USC; Soil type; colour; structure; strength; moisture; bedding; plasticity; sensitivity; additional comments Rock: Weathering; colour; fabric; rock name; strength; additional comments	Moisture Condition	Sensitivity	Shear Strengths (kPa) Peak (Residual)	Dynamic Cone Penetrometer (Blow/100 mm) 5 10 15 20	Comments
Tauranga Group					OL: TOPSOIL	M				
					CL: Silty CLAY: creamy grey. Very stiff, moist, low plasticity.		MS	V-174(44)		
					CH: Silty CLAY: orange, streaked brown. Very stiff, moist, high plasticity.		MS	V-122(55)		
			1							
					becoming orange, streaked grey.		MS	V-134(64)		
					becoming grey, streaked orange.		IS	V-122(61)		
			2				IS	V-116(64)		
							IS	V-119(61)		
					CH: Sandy CLAY: orange with grey streaks. Very stiff, moist, high plasticity.		IS	V-116(58)		
			3		CH: Sandy CLAY: grey medium to fine. Very stiff, wet, low plasticity. Sand is fine grained.	W				
	▼						IS	V-84(55)		
							IS	V-78(46)		
			4		CH: Sandy CLAY: grey medium to fine. Stiff, moist, low plasticity. Sand is fine grained.	M	IS	V-131(73)		
							MS	V-145(64)		
								V-UTP		
			5		Borehole terminated at 5.0m					

Termination reason: Target Depth Reached

Remarks: Standing groundwater was encountered at a depth of 3.2m

This report is based on the attached field description for soil and rock, New Zealand, Geotechnical Society Inc 2005.

HAND AUGER BOREHOLE - HA07-16

Client: Cabra Developments Limited
Project: 10 Sinton Road
Site Address: Hobsonville
Project No: AKL2016_0605
Date: 08/11/2016
Borehole Location: Refer to site plan.

CMW Geosciences

1:25

Sheet 1 of 1

Logged by: KP	Position: E.1746494.0m N.5926833.0m (NZTM)	Elevation:	Hole Diameter: 50mm
Checked by: JF	Survey Source: Hand Held GPS	Datum:	Angle from horizontal: 90°

Unit	Groundwater	RL (m)	Depth (m)	Graphic Log	Material Description Soil: USC; Soil type; colour; structure; strength; moisture; bedding; plasticity; sensitivity; additional comments Rock: Weathering; colour; fabric; rock name; strength; additional comments	Moisture Condition	Sensitivity	Shear Strengths (kPa) Peak (Residual)	Dynamic Cone Penetrometer (Blow/100 mm) 5 10 15 20	Comments
Tauranga Group					OL: TOPSOIL	M				
					CL: Silty CLAY: creamy grey. Very Stiff, moist, low plasticity.		MS	V-136(67)		
					CH: Silty CLAY: orange with brown streaks. Very stiff, moist, high plasticity.		IS	V-128(64)		
			1		SM: Silty SAND: light grey. Very stiff, moist high plasticity.			V-UTP		
					CL: Sandy CLAY with fine gravels: orange. Very stiff, moist, low plasticity. Sand is fine to course grained.			V-UTP V-UTP V-UTP		
			2							
			3		Borehole terminated at 3.0m					
			4							
			5							

Termination reason: Refusal. Unable to penetrate further.

Remarks: Groundwater was not encountered.

This report is based on the attached field description for soil and rock, New Zealand, Geotechnical Society Inc 2005.

HAND AUGER BOREHOLE - HA08-16

Client: Cabra Developments Limited
Project: 10 Sinton Road
Site Address: Hobsonville
Project No: AKL2016_0605
Date: 08/11/2016
Borehole Location: Refer to site plan.

CMW Geosciences

1:25 Sheet 1 of 1

Logged by: RHD
Checked by: JF

Position: E.1746428.0m N.5926776.0m (NZTM)
Survey Source: Hand Held GPS

Elevation:
Datum:

Hole Diameter: 50mm

Unit	Material Description Soil: USC; Soil type; colour; structure; strength; moisture; bedding; plasticity; sensitivity; additional comments Rock: Weathering; colour; fabric; rock name; strength; additional comments
Colluvium	OL: TOPSOIL
	CH: CLAY: light grey. Very stiff, moist, high plasticity. Trace rootlets. *...becoming moist to wet. Stiff*
	CL: Silty CLAY, with trace fine sand: grey. Stiff, moist to wet, low plasticity. *...becoming firm*
	CH: Sandy CLAY: grey. Firm, wet to saturated, low plasticity.
Tauranga Group	CH: CLAY with trace fine sand: light grey, streaked orange. Stiff, saturated, high plasticity. *...becoming stiff*
	CL: CLAY with some sand: light grey, mottled light brown. Stiff, saturated, low plasticity. Sand is fine to medium grained.
	CH: CLAY: dark grey. Stiff, moist to wet, high plasticity.
	CL: Silty CLAY: dark grey. Hard, moist to wet, low plasticity.
	Borehole terminated at 4.0m

Moisture Condition: M; M to W; W to S; S; M to W

Sensitivity	Shear Strengths (kPa) Peak (Residual)
MS	V-150(48)
IS	V-86(44)
MS	V-72(26)
IS	V-44(28)
IS	V-44(22)
MS	V-55(24)
MS	V-94(35)
	V-194+
	V-UTP
	V-UTP

Dynamic Cone Penetrometer (Blow/100 mm): 5 10 15 20

Comments

Depth (m): 1 2 3 4 5

Termination reason: Refusal. Unable to penetrate further.

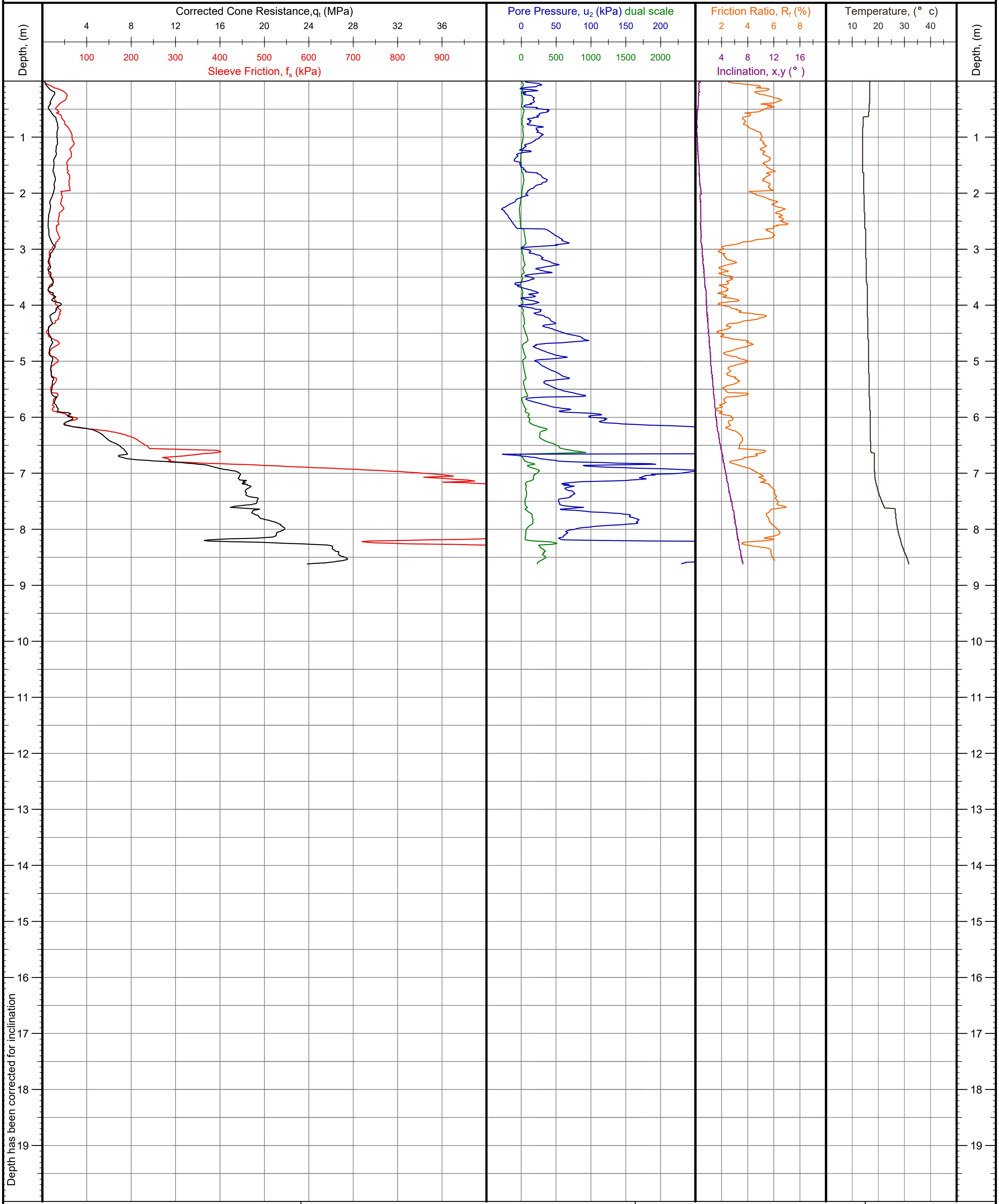
Remarks: Standing groundwater encountered at 1.6m.

This report is based on the attached field description for soil and rock, New Zealand, Geotechnical Society Inc 2005.

APPENDIX 3:

CPT Results

CONE PENETRATION TEST (CPT) LOG



Project: 10 & 16 Sinton Rd & 15 Clarks Ln

Location: Hobsonville, Auckland

NZTM 2000 N, E (m): 5926783.36, 1746465.64
WGS84 (deg): -36.793452,174.641630
Location Method: Handheld GPS
Elevation (m): Unknown

Comments: Tested at 10 Sinton Road.

Operator: Cesar Etchevarne
Cone Ref: MKJ311
Cone Type: $10cm^2$ Compression
Date of Test: 16/08/2023
Depth (m): 8.78

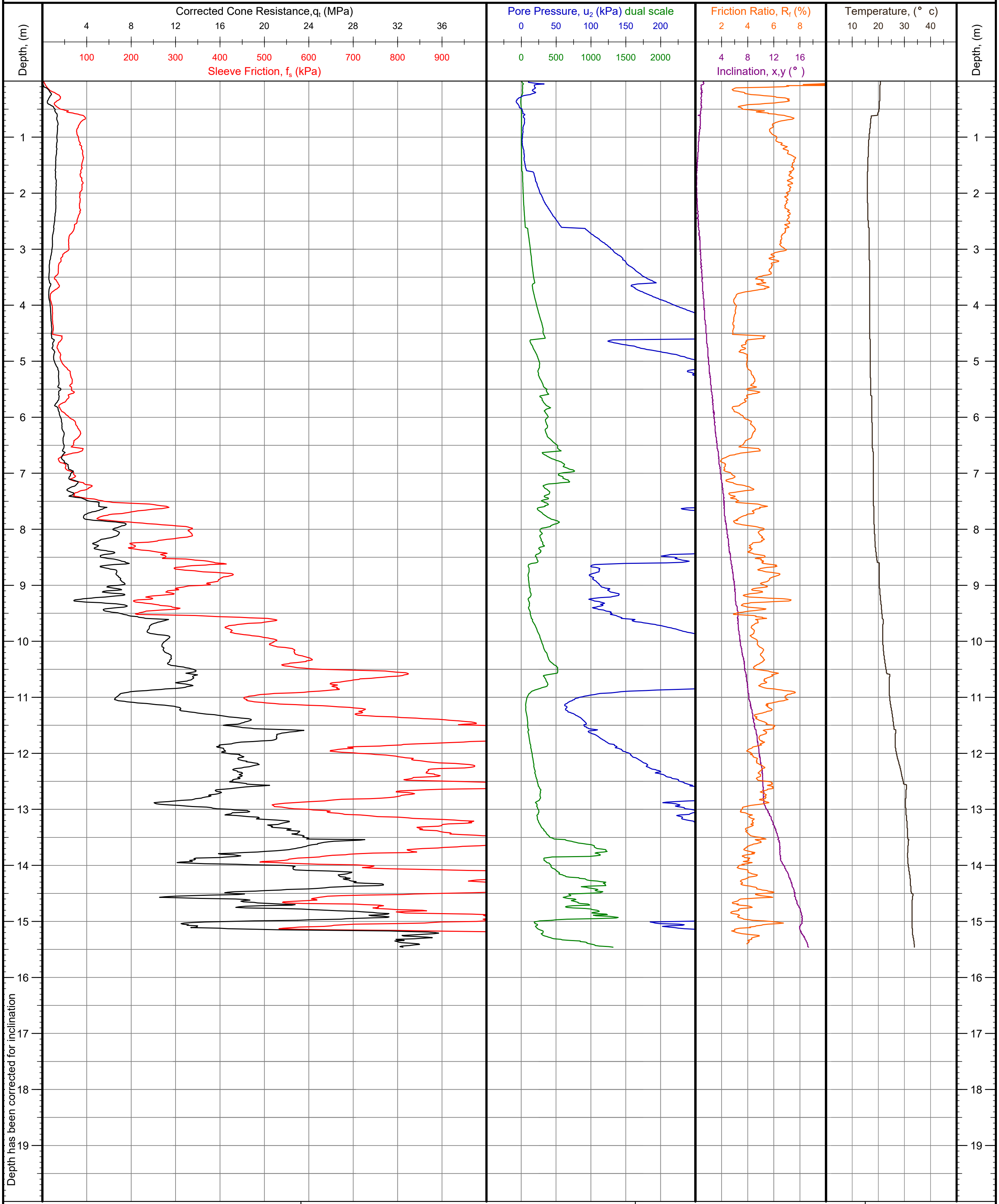
Test Number:
CPT01-23849.000.002

G.I. Job Ref:
230738

Termination Reason: High friction resistance

Where possible GWL is measured after testing, or estimated in the office. This may not represent the true GWL

CONE PENETRATION TEST (CPT) LOG



Project: 10 & 16 Sinton Rd & 15 Clarks Ln

Location: Hobsonville, Auckland

NZTM 2000 N, E (m): 5926730.18, 1746557.37

WGS84 (deg): -36.793917,174.642668

Location Method: Handheld GPS

Elevation (m): Unknown

Operator: Cesar Etchevarne

Cone Ref: MKJ311

Cone Type: 10cm^2 Compression

Date of Test: 16/08/2023

Depth (m): 15.75

Test Number: CPT02-23849.000.002

G.I. Job Ref: 230738

Comments: Tested at 10 Sinton Road.

Termination Reason: High friction resistance

Where possible GWL is measured after testing, or estimated in the office. This may not represent the true GWL

APPENDIX 4:

Lab Test Results

BGL
Babbage Geotechnical Laboratory

Babbage Geotechnical Laboratory
Level 4
68 Beach Road
Auckland 1010
Telephone
E-mail

P O Box 2027
New Zealand
64-9-367 4954
wec@babbage.co.nz

Please reply to: W.E. Campton

Page 1 of 3

ENGEO LTD.
PO Box 33-1527
Takapuna
Auckland 0740

Attention: **HEATHER LYONS**

Job Number: 66273#L
BGL Registration Number: 3064
Checked by: WEC

31st August 2023

ATTERBERG LIMITS & LINEAR SHRINKAGE TESTING

Dear Heather,

Re: 10 SINTON ROAD, HOBSONVILLE

Your Reference: 23849.000.002
Report Number: 66273#L/AL 10 Sinton Rd

The following report presents the results of Atterberg Limits & Linear Shrinkage testing at BGL of a soil sample delivered to this laboratory on the 21st of August 2023. Test results are summarised below, with page 3 showing where the sample plots on the Unified Soil Classification System (Casagrande) Chart. Test standards used were:

Water Content:	NZS4402:1986:Test 2.1
Liquid Limit:	NZS4402:1986:Test 2.2
Plastic Limit:	NZS4402:1986:Test 2.3
Plasticity Index:	NZS4402:1986:Test 2.4
Linear Shrinkage:	NZS4402:1986:Test 2.6

Borehole Number	Sample Number	Depth (m)	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage (%)*
SS	**Sample 1**	**0.25 – 0.75**	**23.1**	**45**	**20**	**25**	**13**

**The amount of shrinkage of the sample as a percentage of the original sample length.*

The whole soil was used for the water content test (the soil was in a natural state), and for the liquid limit, plastic limit and linear shrinkage tests. The soil was wet up and dried where required for the liquid limit, plastic limit and linear shrinkage tests.

As per the reporting requirements of NZS4402: 1986: Test 2.1: water content is reported to two significant figures for values below 10%, and to three significant figures for values of 10% or greater. Test 2.2: liquid limit, test 2.3: plastic limit, and test 2.6: linear shrinkage are reported to the nearest whole number.

Please note that the test results relate only to the sample as-received, and relate only to the sample under test.

Thank you for the opportunity to carry out this testing. If you have any queries regarding the content of this report please contact the person authorising this report below at your convenience.

Yours faithfully,

Justin Franklin
Key Technical Person
Assistant Laboratory Manager
Babbage Geotechnical Laboratory

ACCREDITED
IANZ
TESTING LABORATORY
№ 126

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation. This report may not be reproduced except in full & with written approval from BGL.

[redacted]

BGL Babbage Geotechnical Laboratory	Job Number:	66273#L	Sheet 1 of 1	Page 3 of 3
	Reg. Number:	3064	Version No:	7
	Report No:	66273#L/AL 10 Sinton Rd	Version Date:	July 2022
	Project:	**10 SINTON ROAD, HOBSONVILLE**		

DETERMINATION OF THE LIQUID LIMIT, PLASTIC LIMIT & THE PLASTICITY INDEX

Test Methods: NZS4402: 1986: Test 2.2, Test 2.3 and Test 2.4

Tested By:	JL	August 2023
Compiled By:	JF	31/08/2023
Checked By:	JF	31/08/2023

SUMMARY OF TESTING

Borehole Number	Sample Number	Depth (m)	Liquid Limit	Plastic Limit	Plasticity Index	Soil Classification Based on USCS Chart Below
SS	**Sample 1**	**0.25 - 0.75**	45	20	**25**	CL

The chart below & soil classification terminology is taken from ASTM D2487-17^{e1} "Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)", April 2020, & is based on the classification scheme developed by A. Casagrande in the 1940's (Casagrande, A., 1948: Classification and identification of soil. Transactions of the American Society of Civil Engineers, v. 113, p. 901-930). The chart below & the soil classification given in the table above are included for your information only, and are not included in the IANZ endorsement for this report.

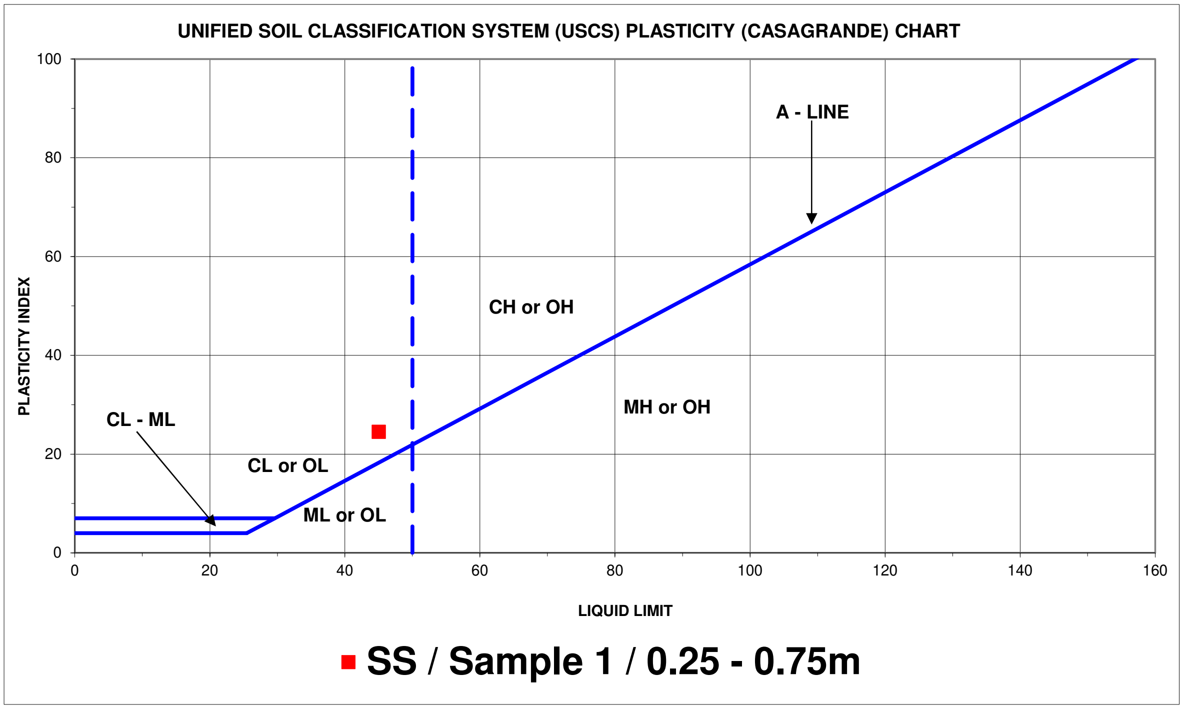


CHART LEGEND

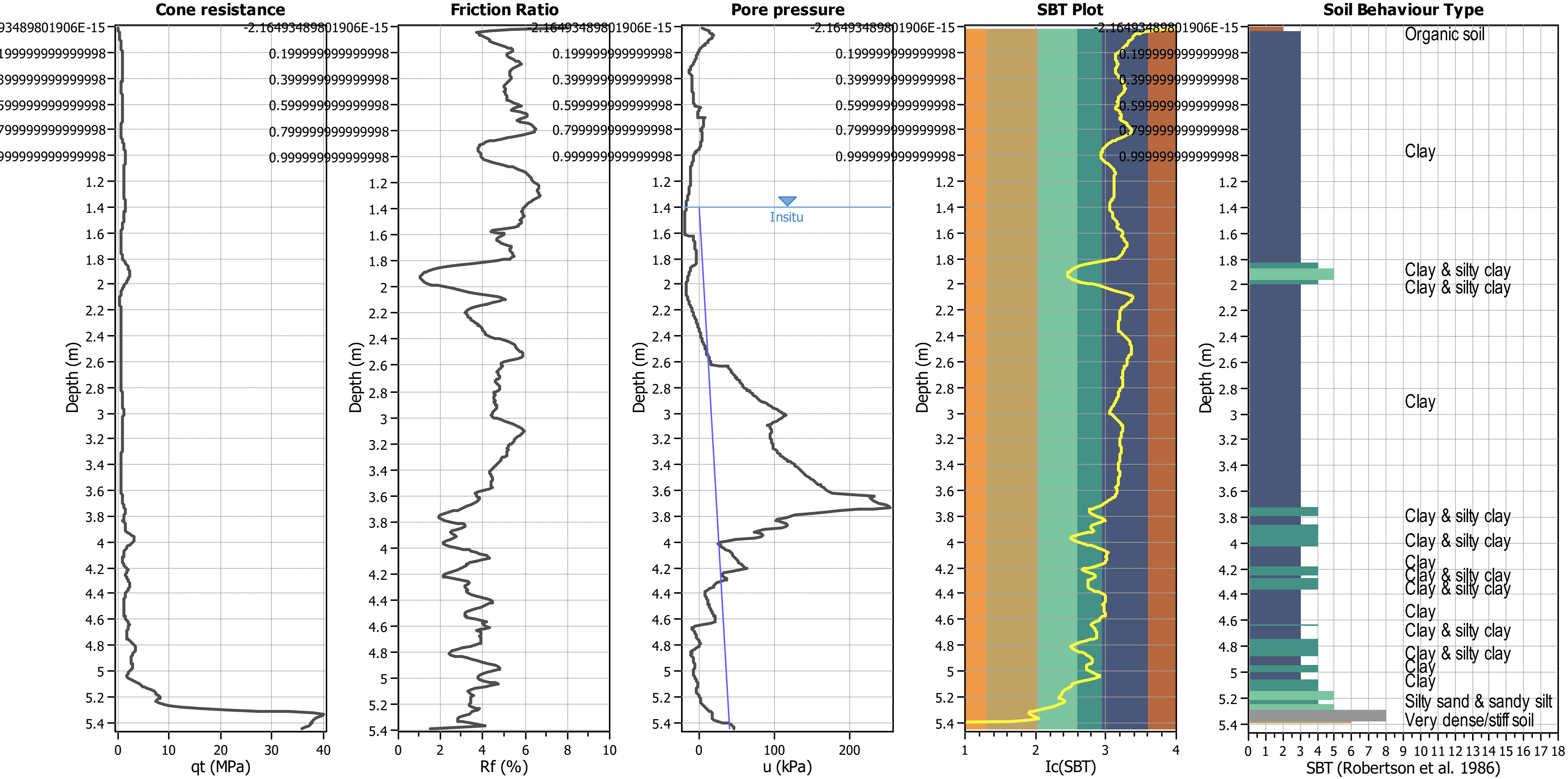
CL = CLAY, low plasticity ('lean' clay)
OL = ORGANIC CLAY or ORGANIC SILT, low liquid limit
ML = SILT, low liquid limit
CL - ML = SILTY CLAY

CH = CLAY, high plasticity ('fat' clay)
OH = ORGANIC CLAY or ORGANIC SILT, high liquid limit
MH = SILT, high liquid limit ('elastic silt')

APPENDIX 5:

Liquefaction Analysis Results

CPT basic interpretation plots



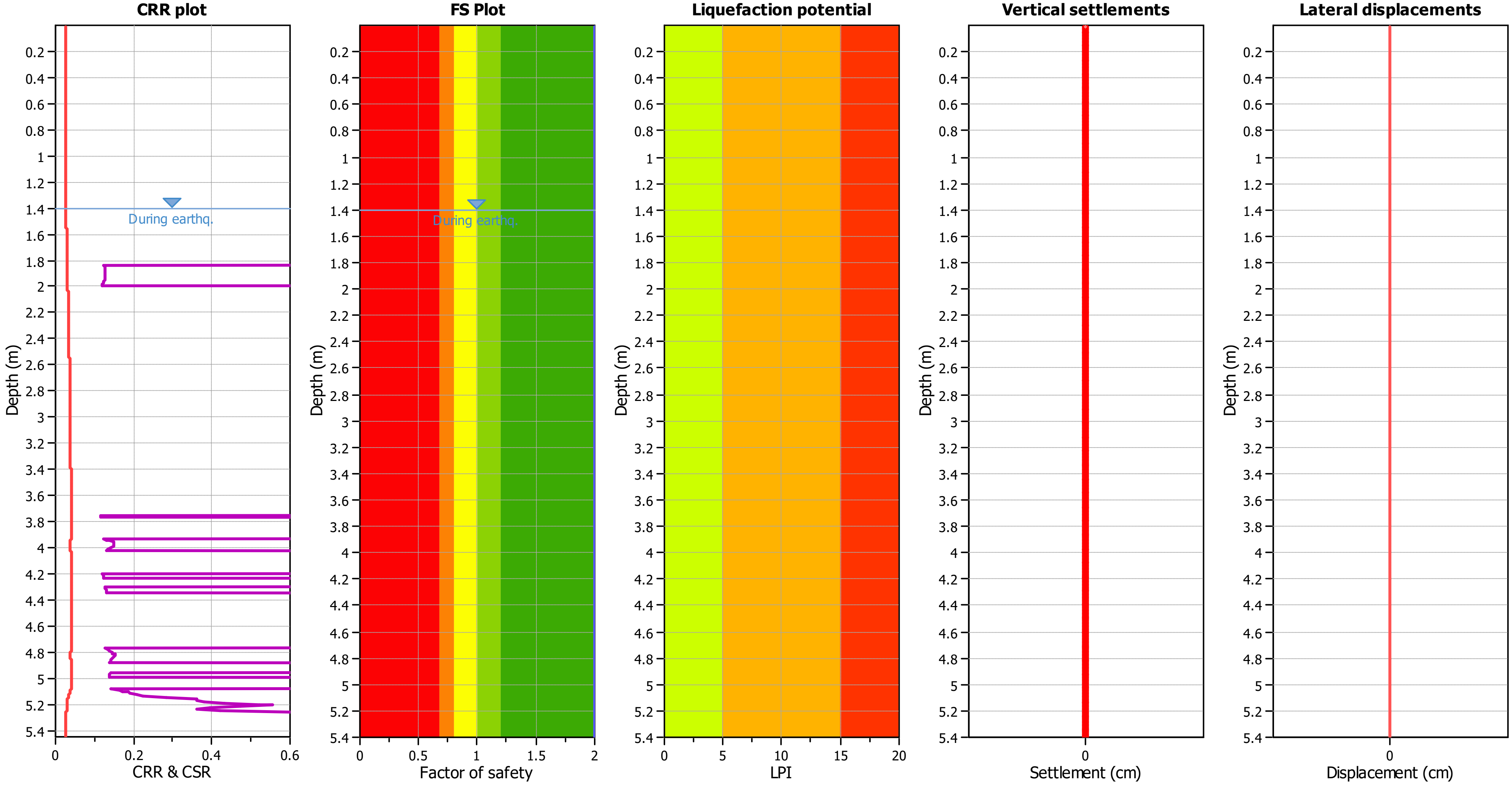
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.40 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.40 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.40 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.40 m	Fill height:	N/A	Limit depth:	N/A

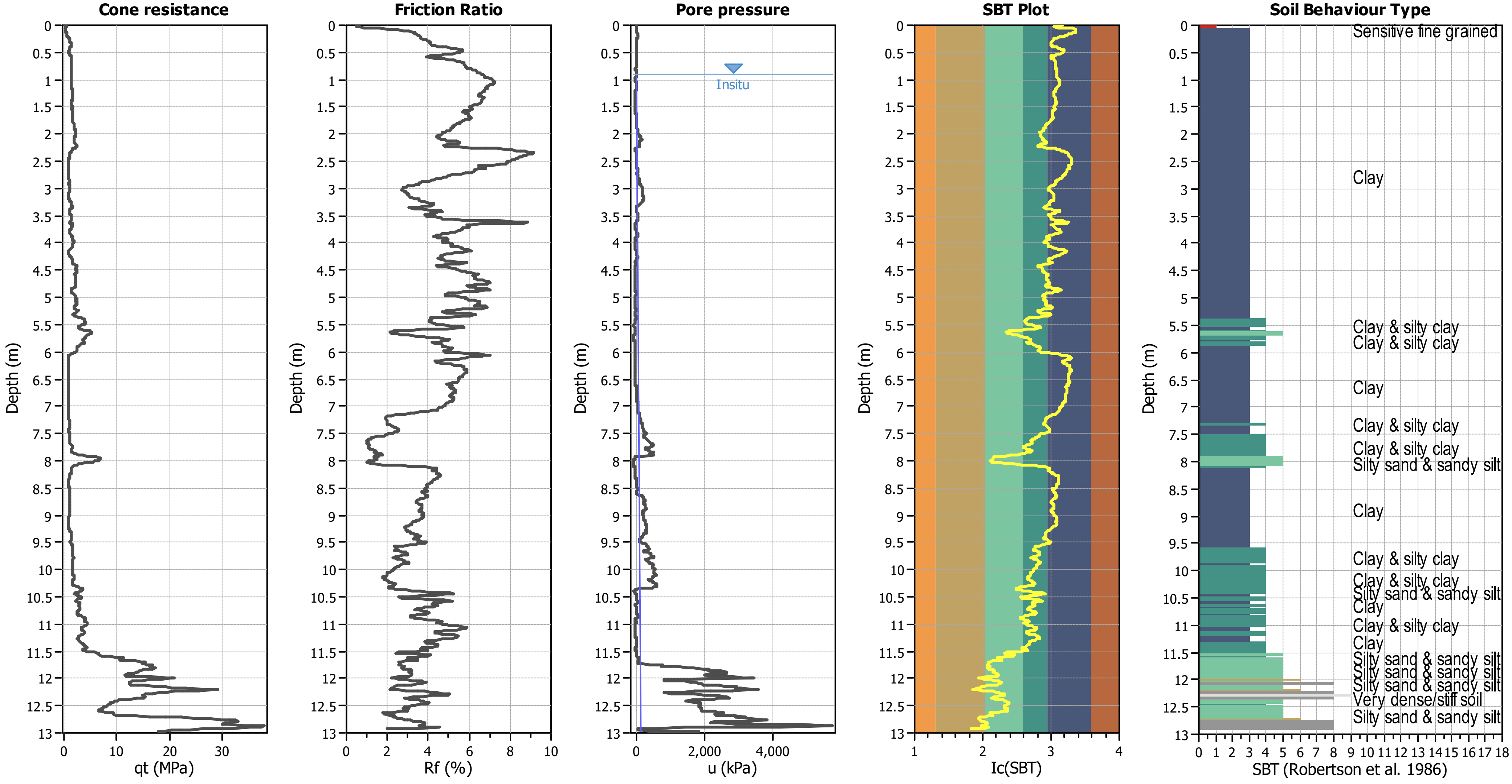
F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

CPT basic interpretation plots



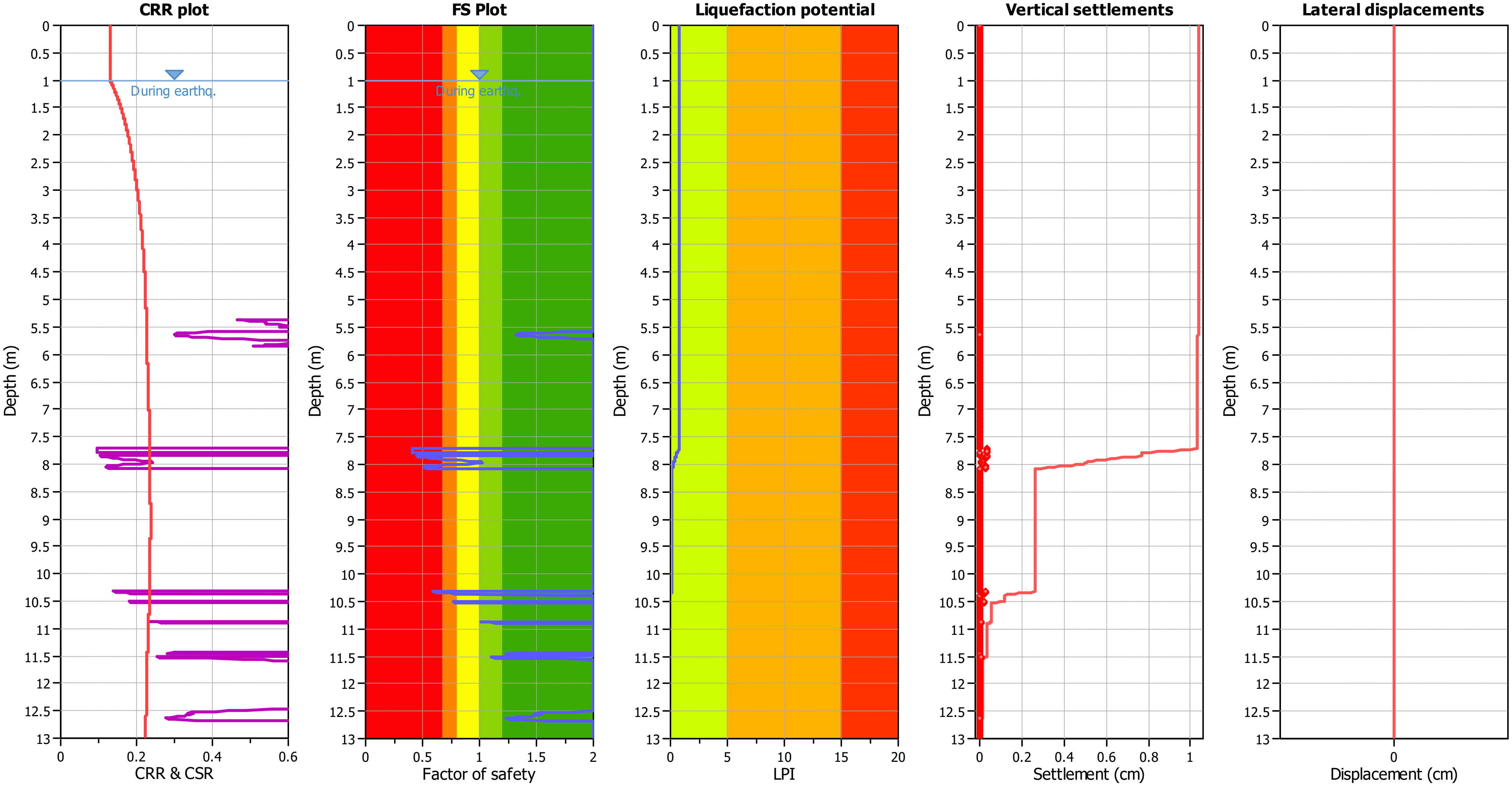
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.24	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	0.90 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.24	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	0.90 m	Fill height:	N/A	Limit depth:	N/A

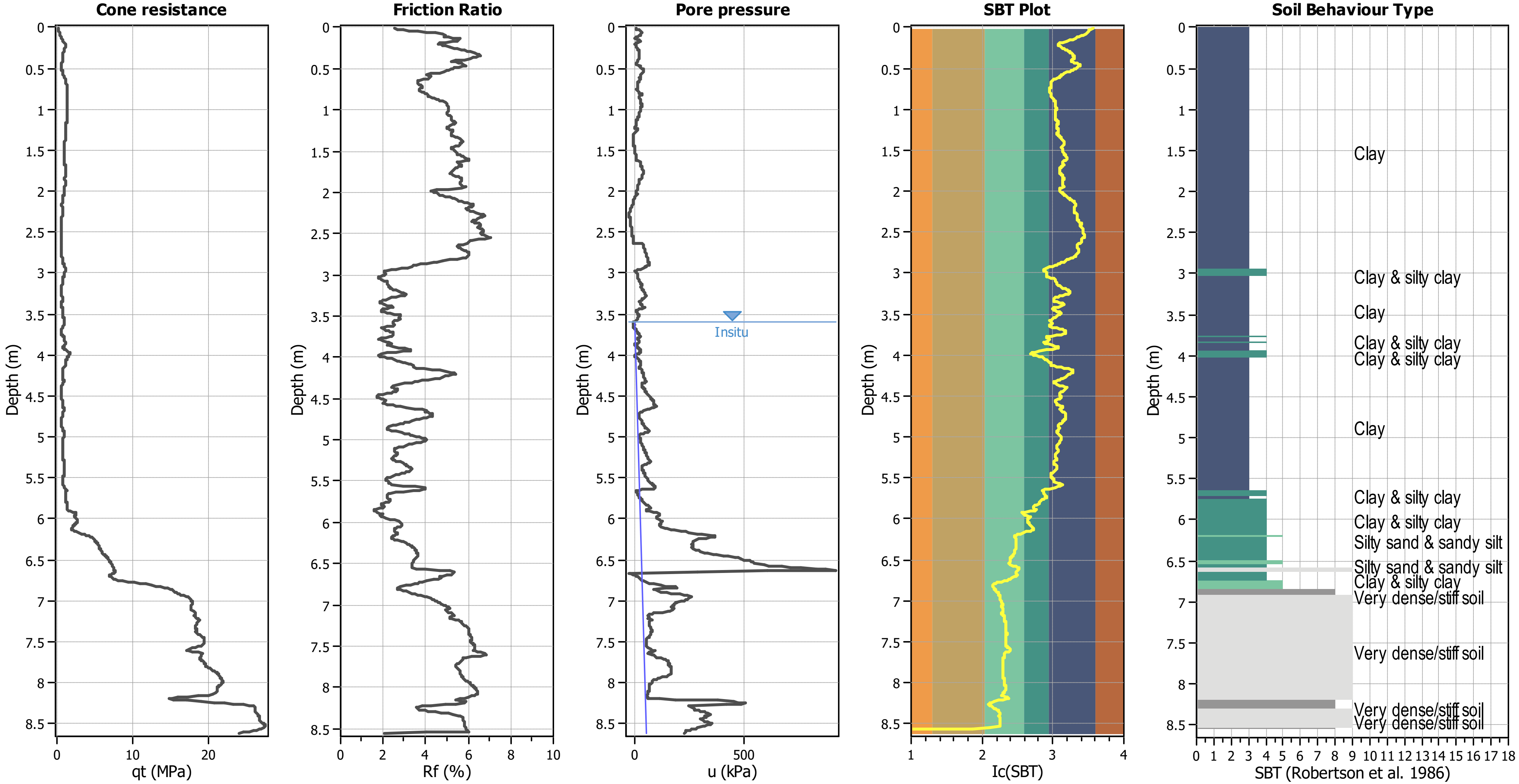
F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

CPT basic interpretation plots



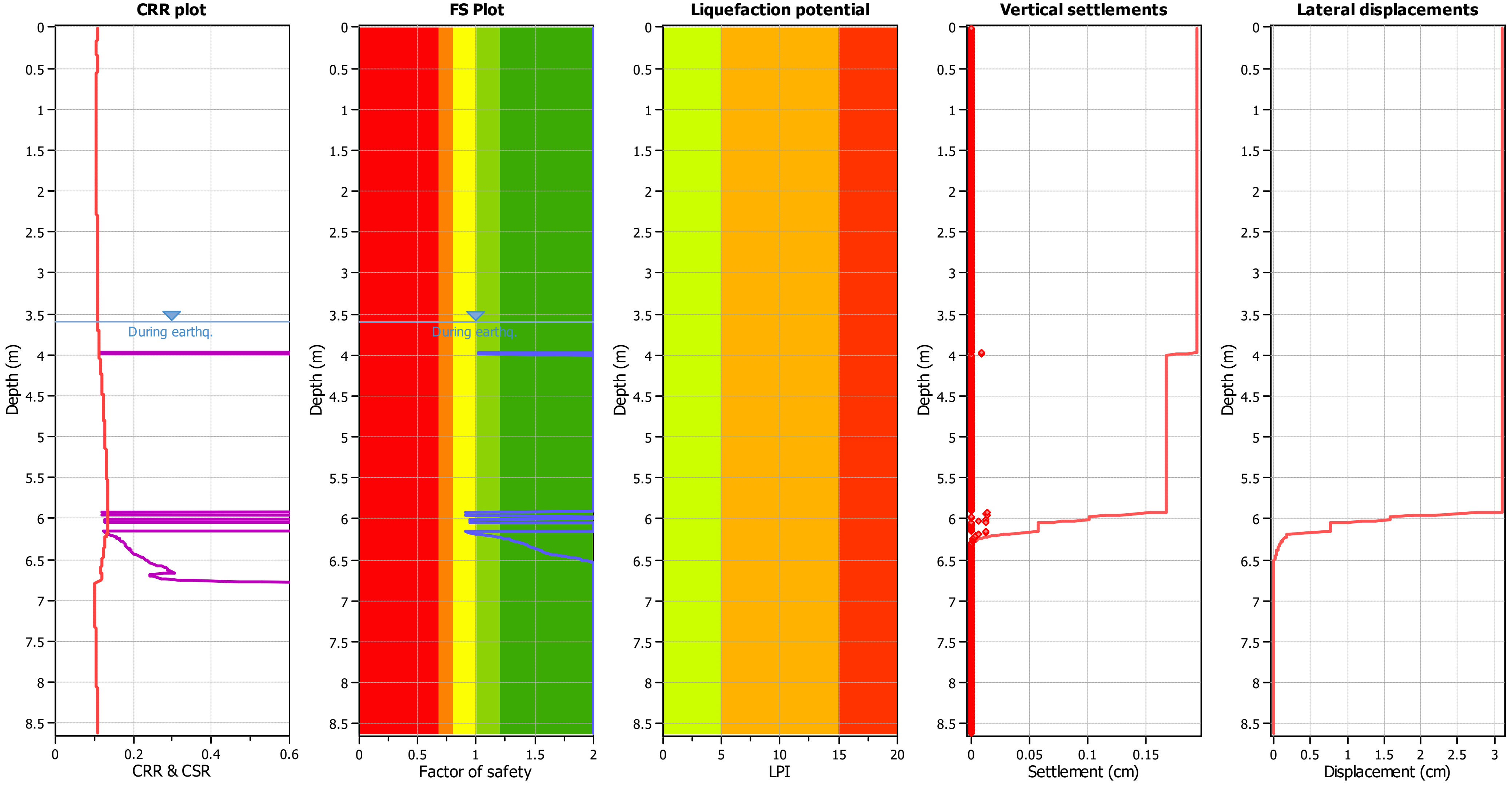
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.60 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.60 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	3.60 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	3.60 m	Fill height:	N/A	Limit depth:	N/A

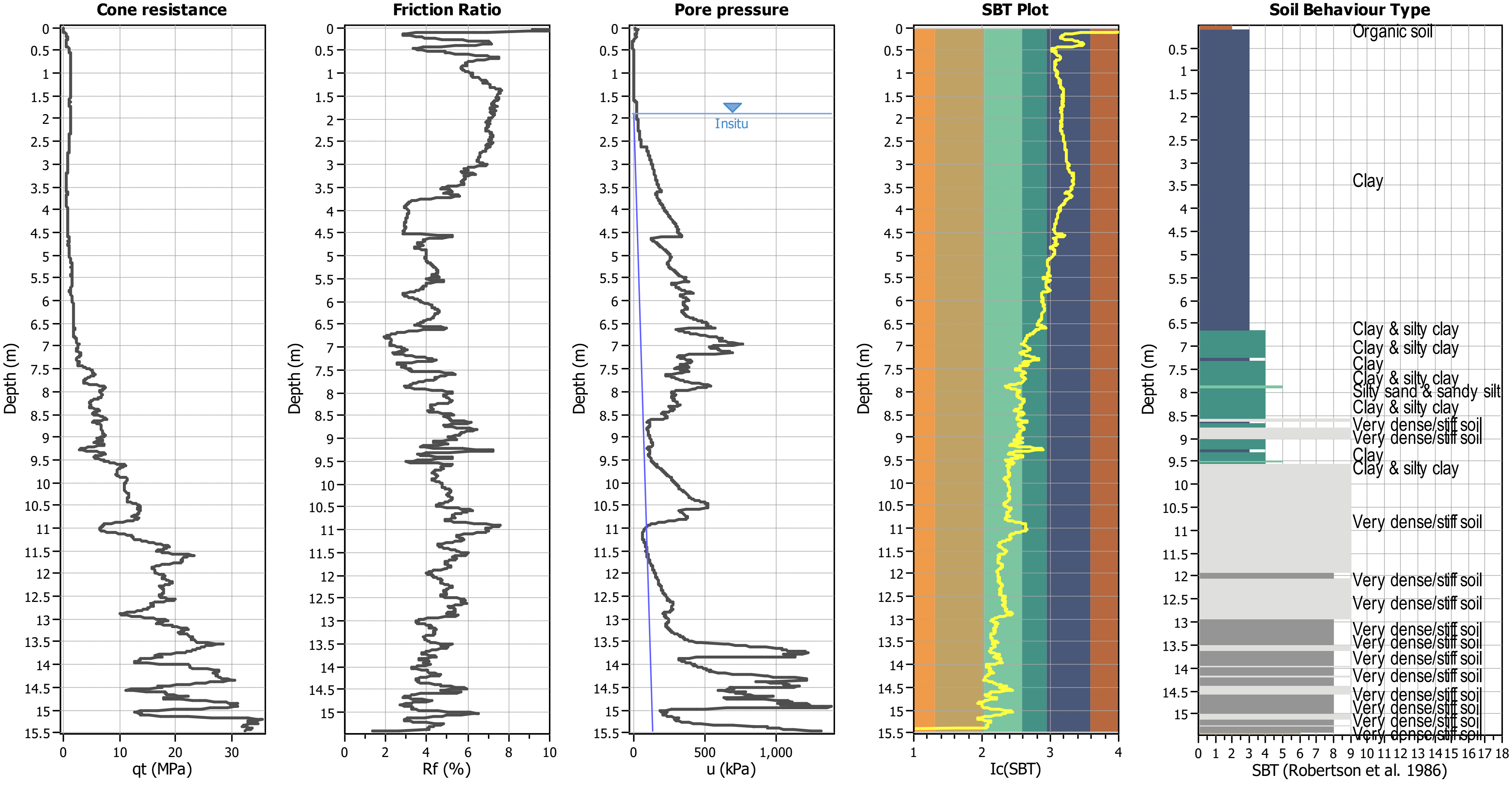
F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟩 Low risk

CPT basic interpretation plots



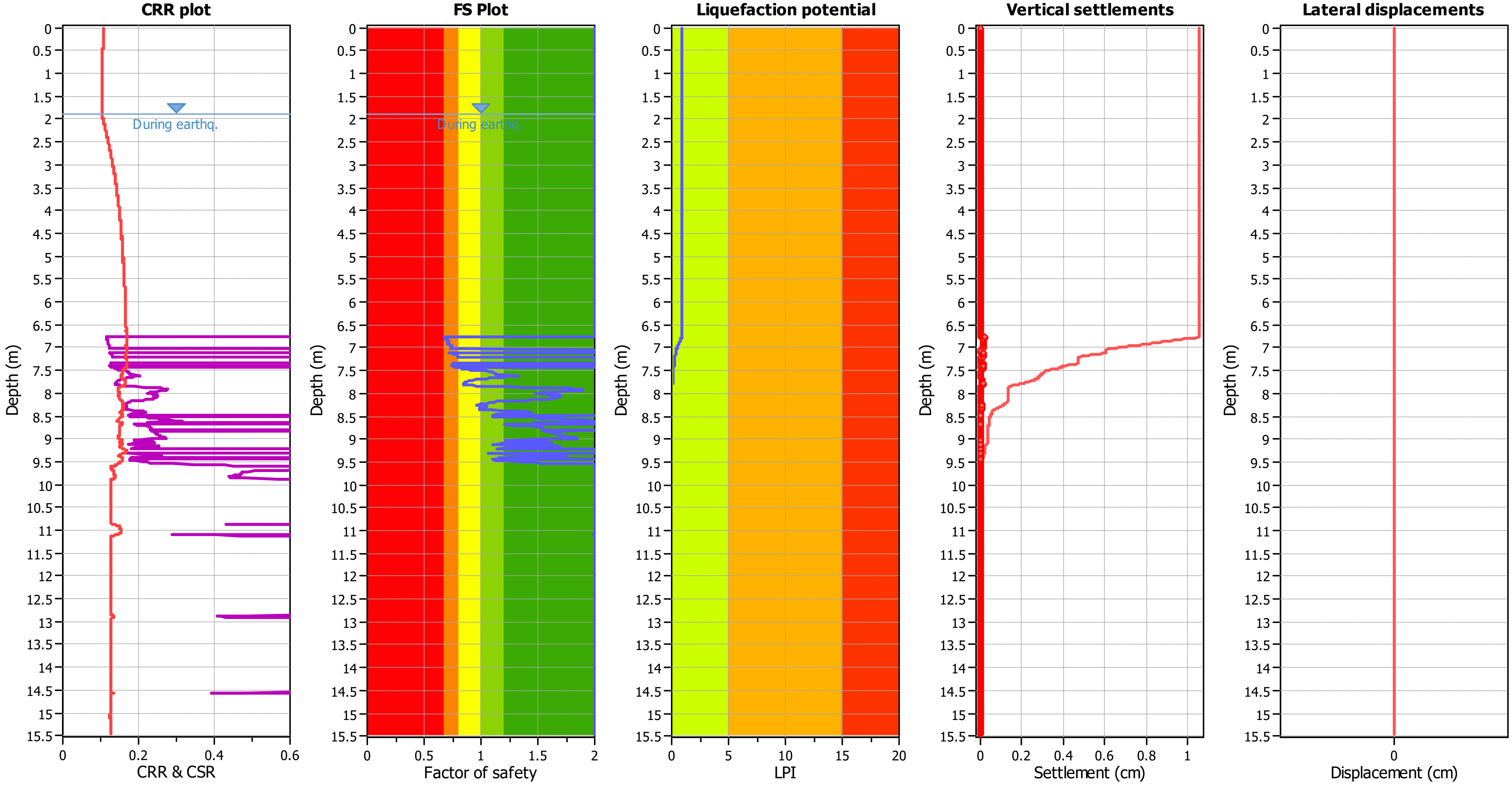
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.90 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.90 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.90 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.90 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

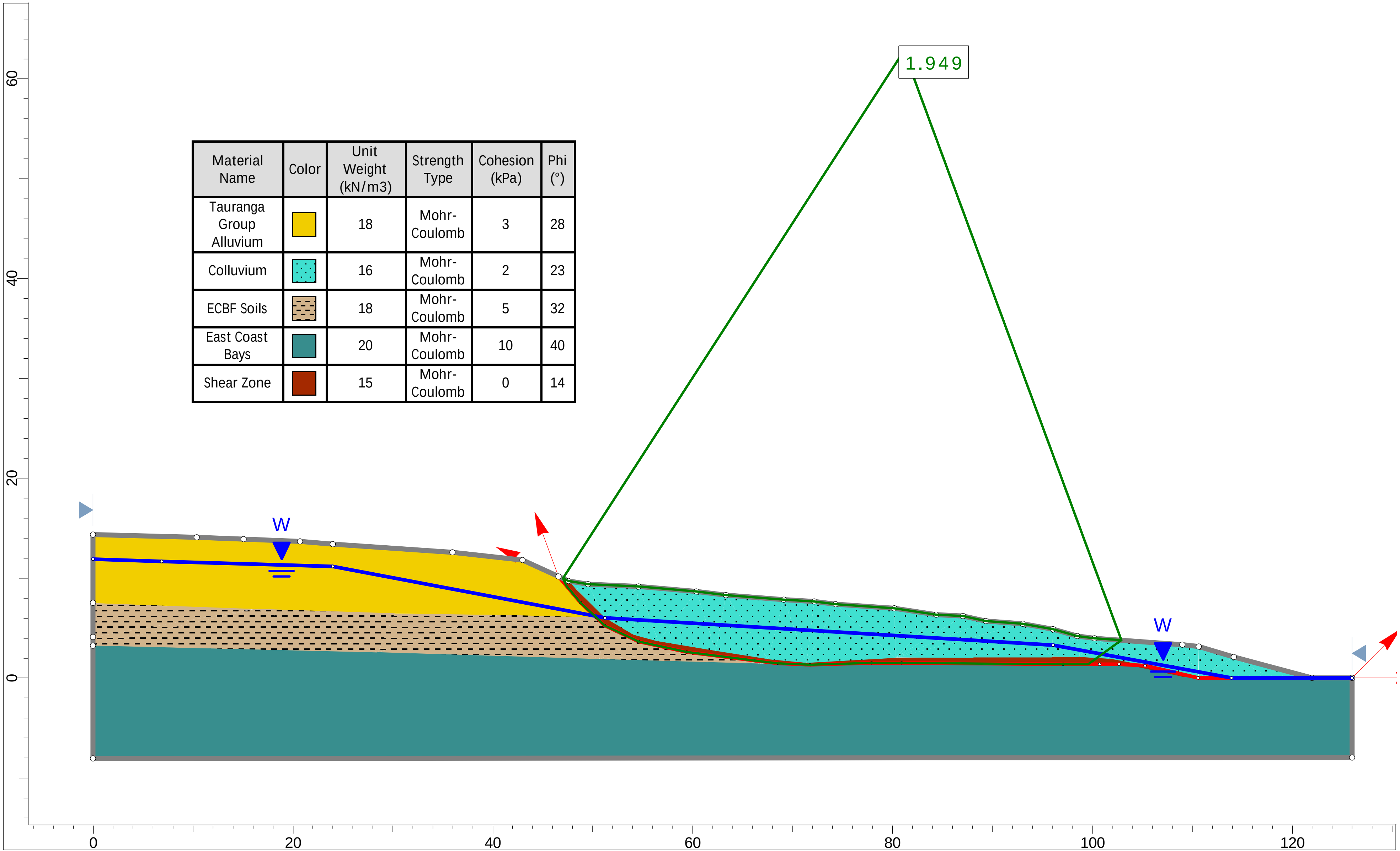
- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟩 Low risk

APPENDIX 6A:

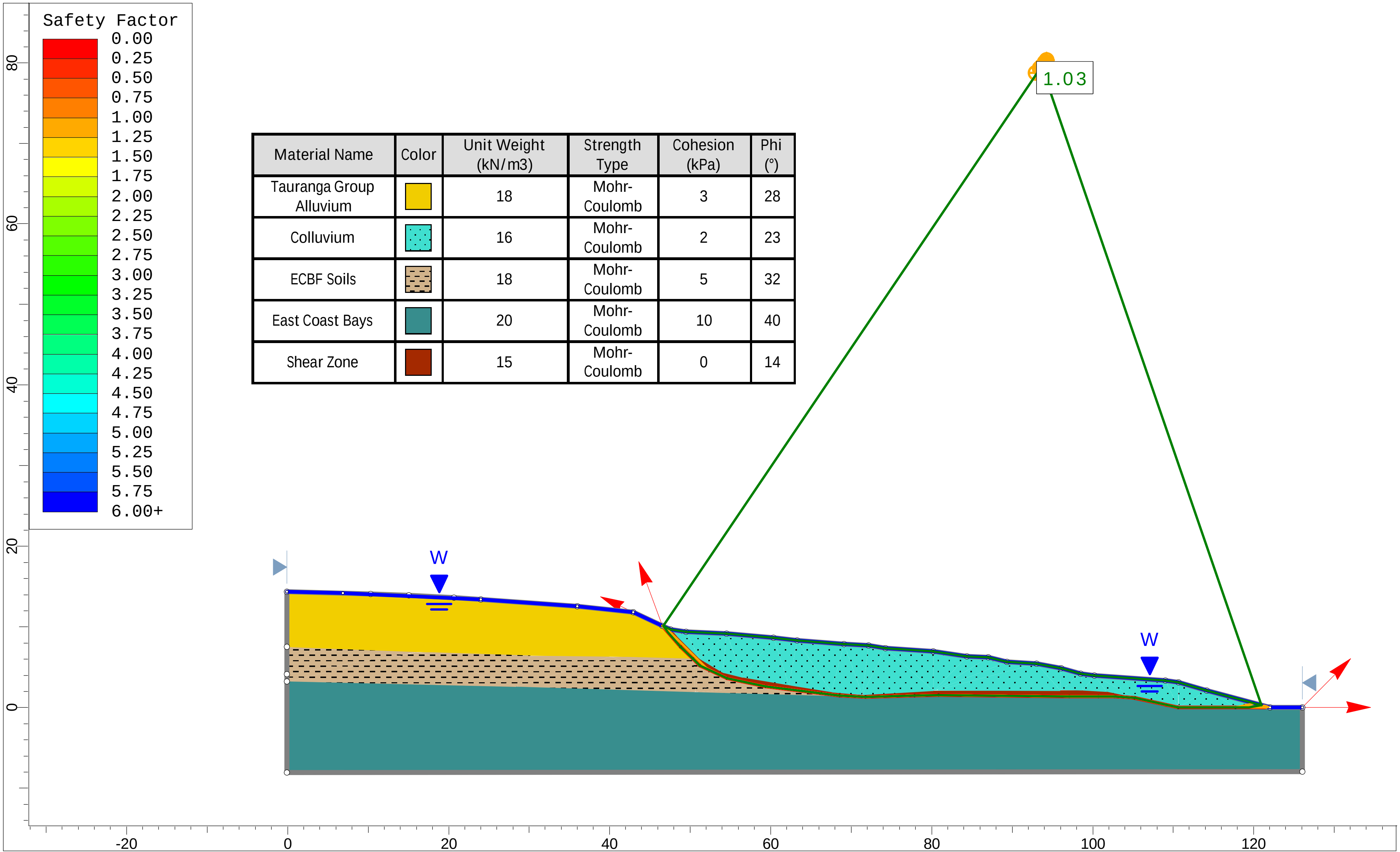
Section A-A’ Slope Stability Analysis



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Tauranga Group Alluvium	🟨	18	Mohr-Coulomb	3	28
Colluvium	░	16	Mohr-Coulomb	2	23
ECBF Soils	▤	18	Mohr-Coulomb	5	32
East Coast Bays	🟦	20	Mohr-Coulomb	10	40
Shear Zone	🟥	15	Mohr-Coulomb	0	14

Project	10 Sinton Road, Hobsonville		
Group	A-A'	*Scenario*	Static
Drawn By	JT	*Company*	ENGEO Ltd.
Date	18/10/2023, 9:27:53 am	*File Name*	10 Sinton Rd JT.slmd

SLIDEINTERPRET 9.030



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Tauranga Group Alluvium	🟨	18	Mohr-Coulomb	3	28
Colluvium	🟦	16	Mohr-Coulomb	2	23
ECBF Soils	🟫	18	Mohr-Coulomb	5	32
East Coast Bays	🟦	20	Mohr-Coulomb	10	40
Shear Zone	🟥	15	Mohr-Coulomb	0	14

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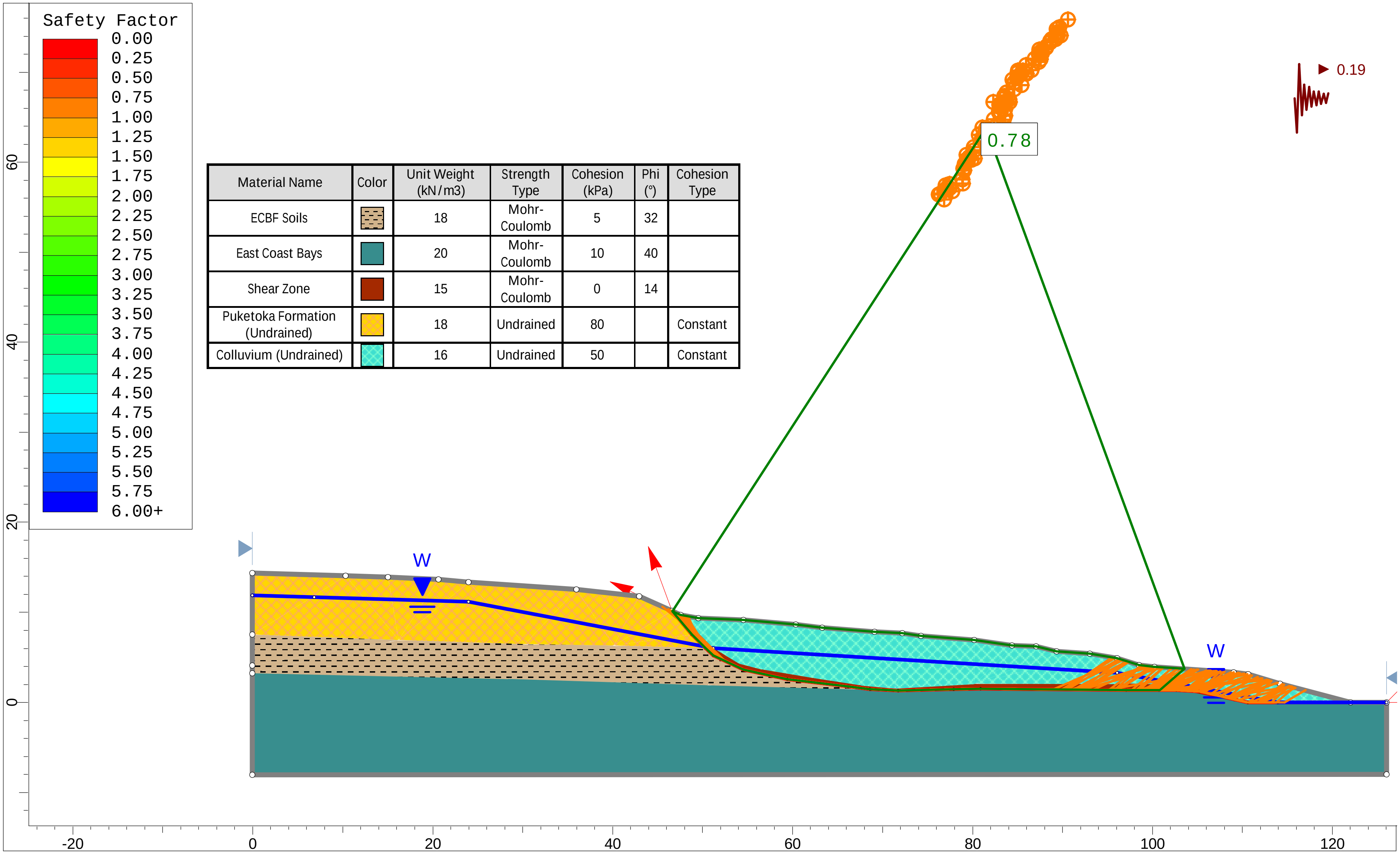
SLIDEINTERPRET 9.030

Project	10 Sinton Road, Hobsonville		
Group	A-A'	*Scenario*	Saturated slope
Drawn By	JT	*Company*	ENGEO Ltd.
Date	18/10/2023, 9:27:53 am	*File Name*	10 Sinton Rd JT.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Cohesion Type
ECBF Soils	▨	18	Mohr-Coulomb	5	32	
East Coast Bays	■	20	Mohr-Coulomb	10	40	
Shear Zone	■	15	Mohr-Coulomb	0	14	
Puketoka Formation (Undrained)	▨	18	Undrained	80		Constant
Colluvium (Undrained)	▨	16	Undrained	50		Constant

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SLIDEINTERPRET 9.030

Project
10 Sinton Road, Hobsonville

Group
A-A'

Scenario
Seismic

Drawn By
JT

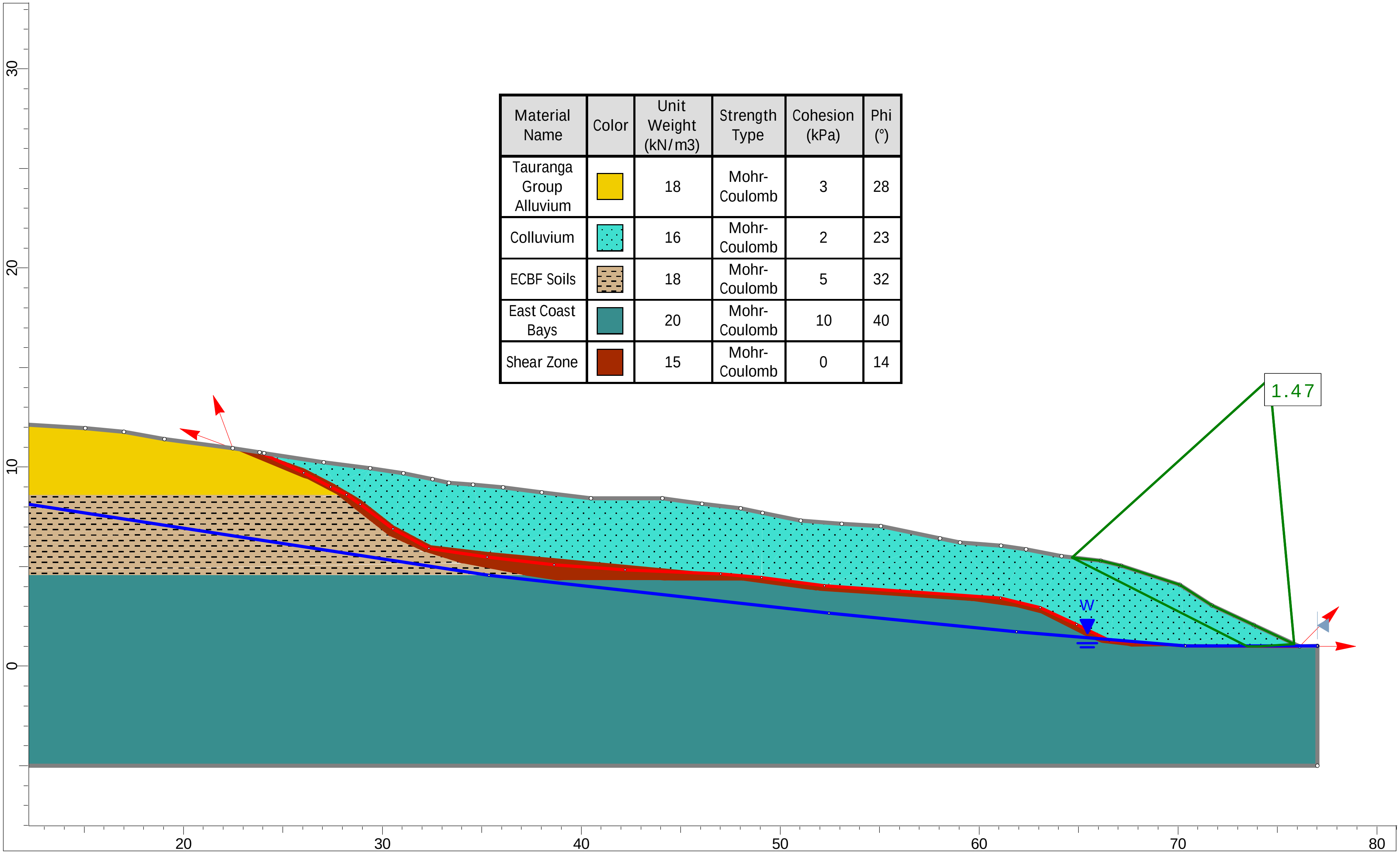
Company
ENGEO Ltd.

Date
18/10/2023, 9:27:53 am

File Name
10 Sinton Rd JT.slmd

APPENDIX 6B:

Section B-B’ Slope Stability Analysis



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Tauranga Group Alluvium	🟨	18	Mohr-Coulomb	3	28
Colluvium	▒	16	Mohr-Coulomb	2	23
ECBF Soils	▨	18	Mohr-Coulomb	5	32
East Coast Bays	🟦	20	Mohr-Coulomb	10	40
Shear Zone	🟥	15	Mohr-Coulomb	0	14

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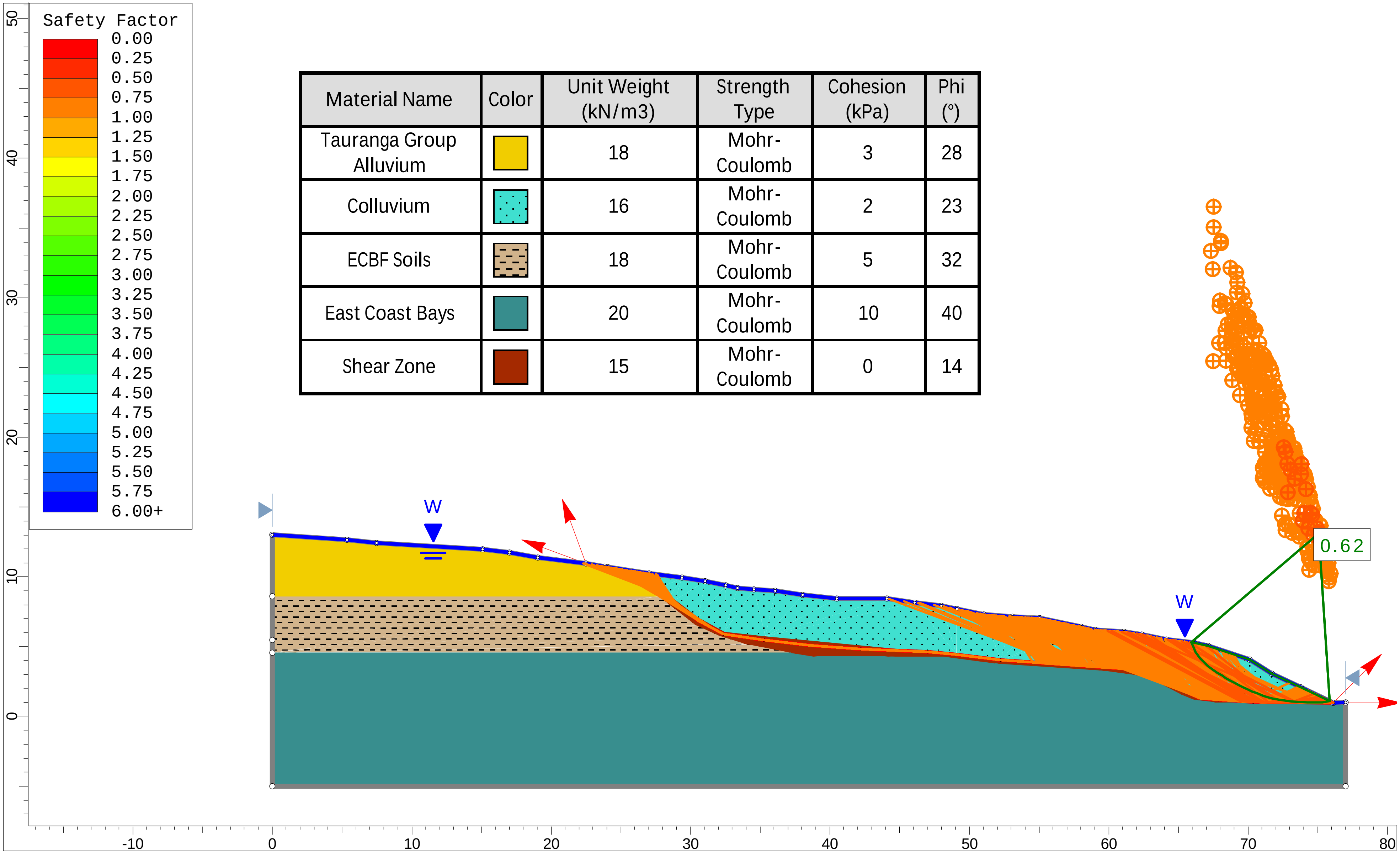
SLIDEINTERPRET 9.030

Project	10 Sinton Road, Hobsonville
Group	B-B'
Scenario	static
Drawn By	JT
Company	ENGEO Ltd.
Date	18/10/2023, 9:27:53 am
File Name	10 Sinton Rd JT.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Tauranga Group Alluvium	🟨	18	Mohr-Coulomb	3	28
Colluvium	▨	16	Mohr-Coulomb	2	23
ECBF Soils	▤	18	Mohr-Coulomb	5	32
East Coast Bays	🟦	20	Mohr-Coulomb	10	40
Shear Zone	🟫	15	Mohr-Coulomb	0	14

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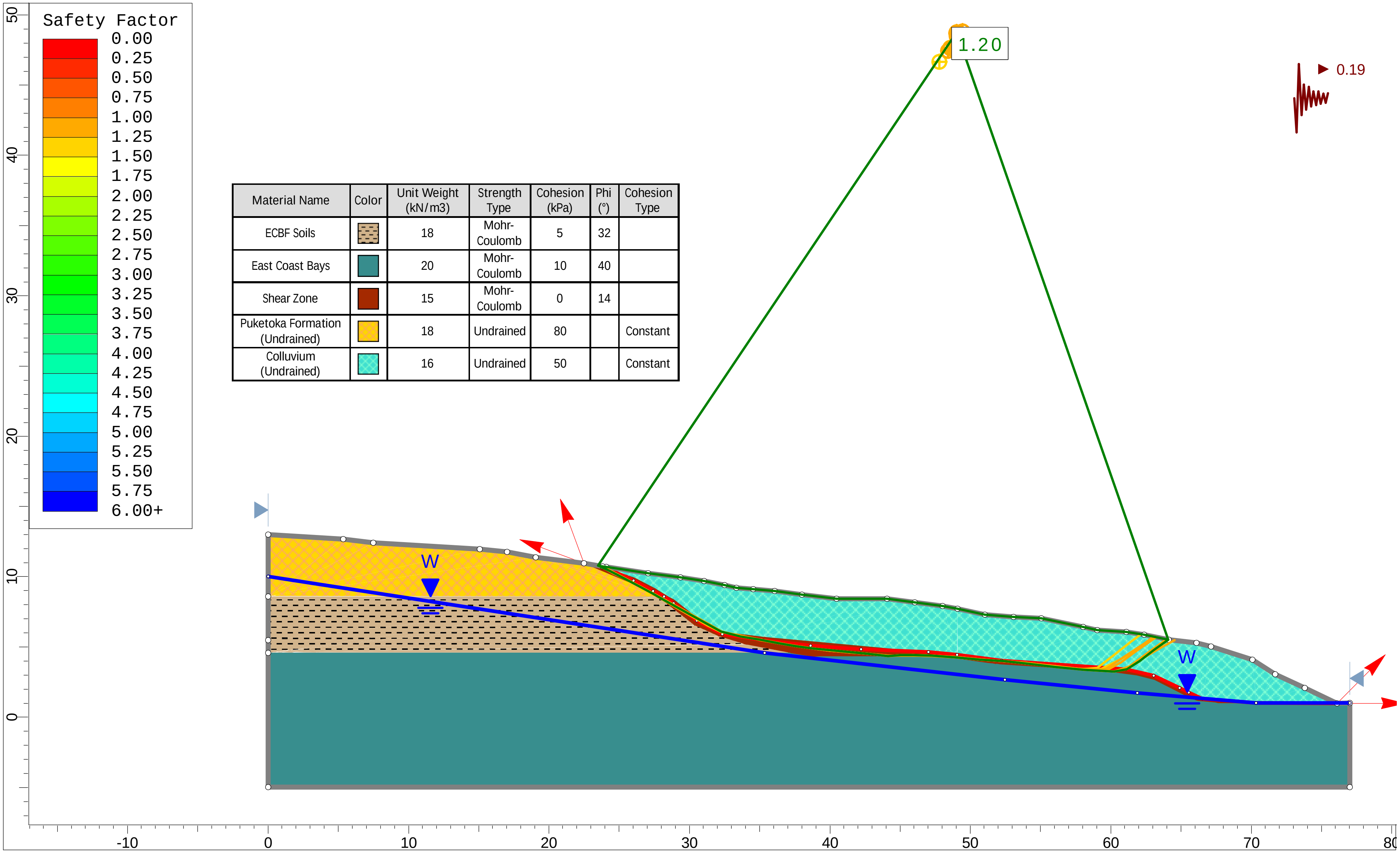
SLIDEINTERPRET 9.030

Project	10 Sinton Road, Hobsonville
Group	B-B'
Scenario	saturated slope
Drawn By	JT
Company	ENGEO Ltd.
Date	18/10/2023, 9:27:53 am
File Name	10 Sinton Rd JT.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Cohesion Type
ECBF Soils	▨	18	Mohr-Coulomb	5	32	
East Coast Bays	🟦	20	Mohr-Coulomb	10	40	
Shear Zone	🟥	15	Mohr-Coulomb	0	14	
Puketoka Formation (Undrained)	🟨	18	Undrained	80		Constant
Colluvium (Undrained)	▨	16	Undrained	50		Constant

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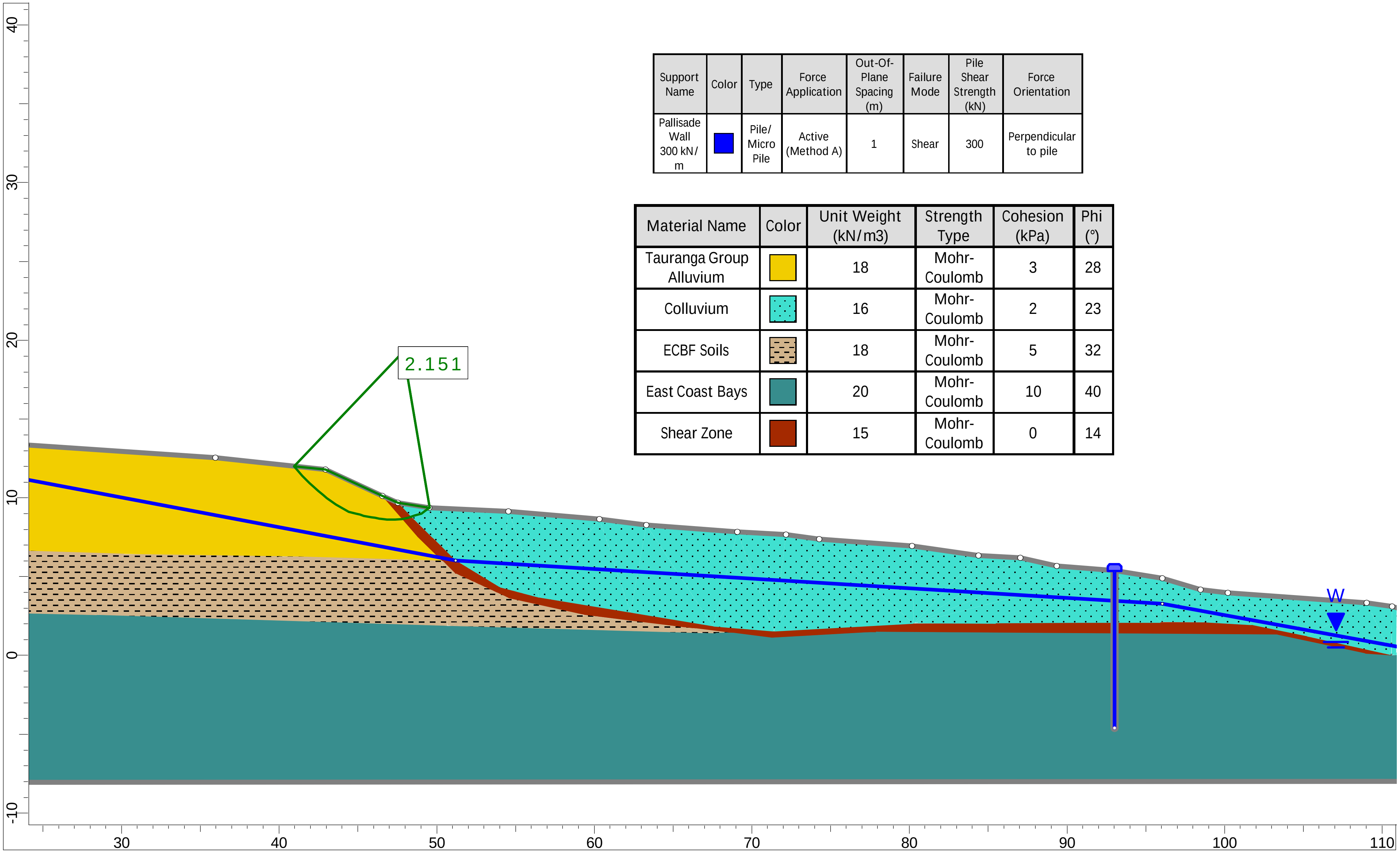
SLIDEINTERPRET 9.030

Project	10 Sinton Road, Hobsonville		
Group	B-B'	*Scenario*	Seismic
Drawn By	JT	*Company*	ENGEO Ltd.
Date	18/10/2023, 9:27:53 am	*File Name*	10 Sinton Rd JT.slmd

APPENDIX 7A:

Section A-A’ Stability Remediation



Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Failure Mode	Pile Shear Strength (kN)	Force Orientation
Pallisade Wall 300 kN/m	🟦	Pile/ Micro Pile	Active (Method A)	1	Shear	300	Perpendicular to pile

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Tauranga Group Alluvium	🟨	18	Mohr-Coulomb	3	28
Colluvium	🟦	16	Mohr-Coulomb	2	23
ECBF Soils	🟫	18	Mohr-Coulomb	5	32
East Coast Bays	🟩	20	Mohr-Coulomb	10	40
Shear Zone	🟥	15	Mohr-Coulomb	0	14

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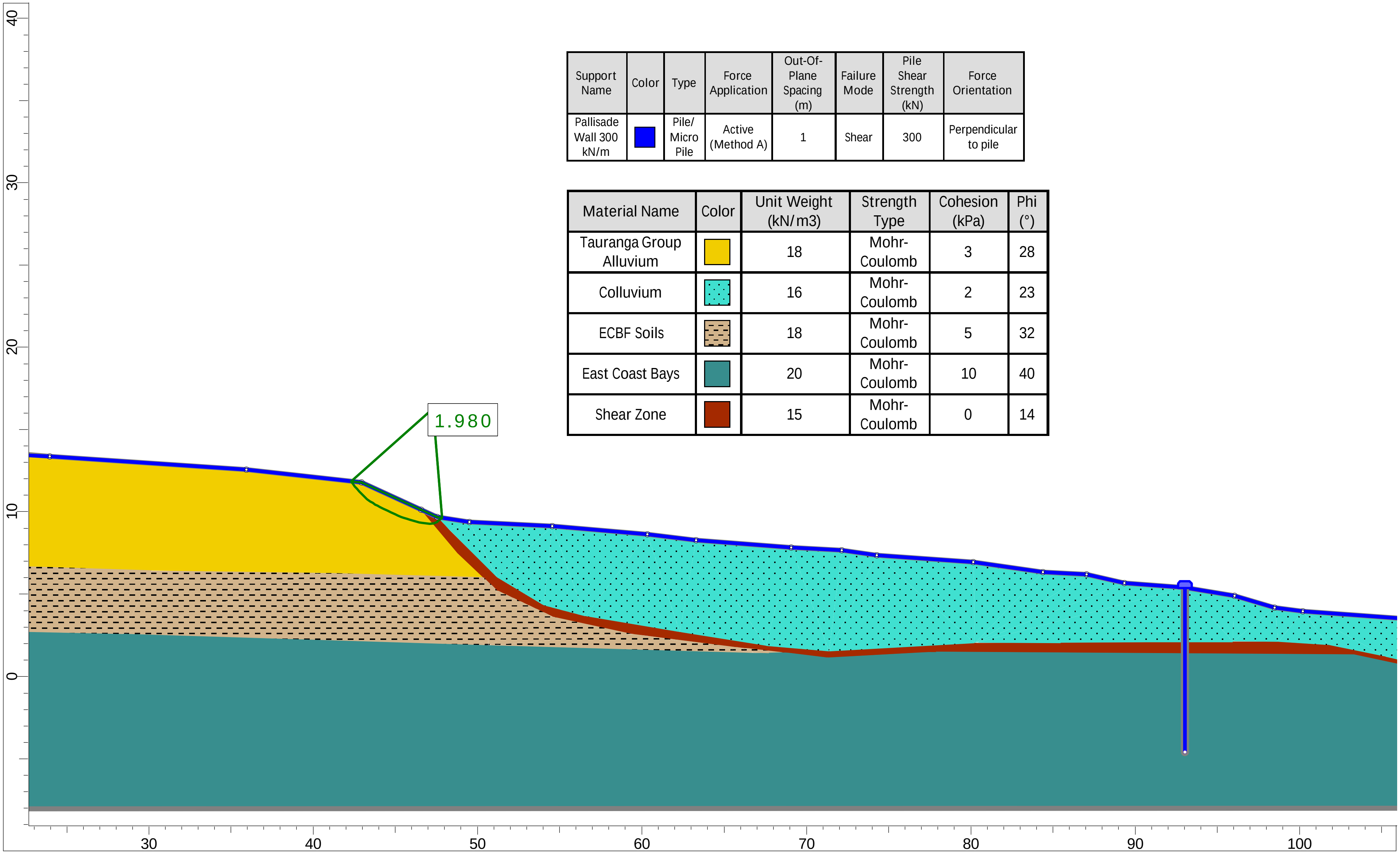
SLIDEINTERPRET 9.030

Project	10 Sinton Road, Hobsonville
Group	Section A-A' Palisade Wall
Scenario	Static
Drawn By	JT
Company	ENGEO Ltd.
Date	18/10/2023, 9:27:53 am
File Name	10 Sinton Rd JT.slmd

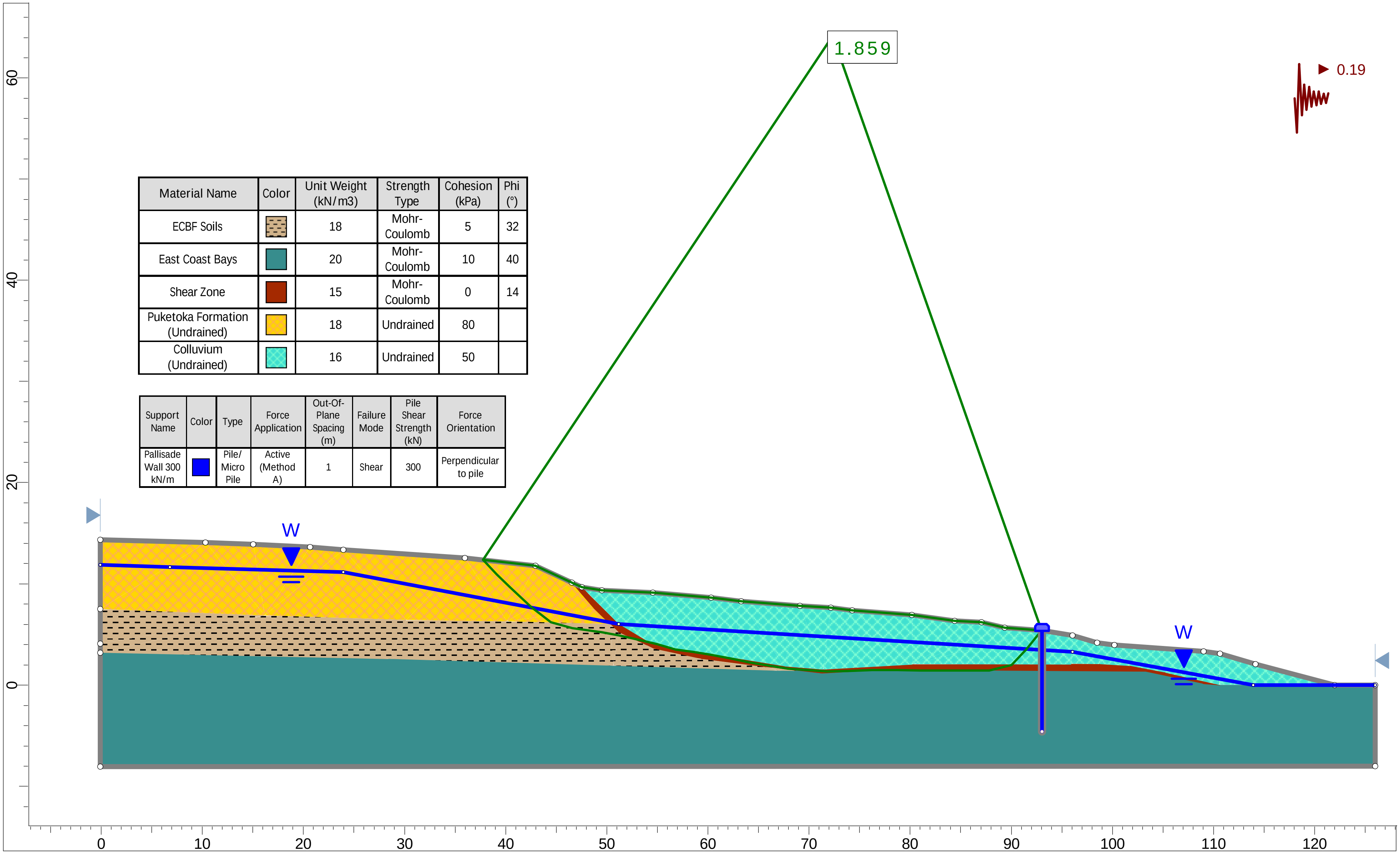


Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Failure Mode	Pile Shear Strength (kN)	Force Orientation
Pallisade Wall 300 kN/m	🟦	Pile/ Micro Pile	Active (Method A)	1	Shear	300	Perpendicular to pile

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Tauranga Group Alluvium	🟨	18	Mohr-Coulomb	3	28
Colluvium	▒	16	Mohr-Coulomb	2	23
ECBF Soils	▤	18	Mohr-Coulomb	5	32
East Coast Bays	🟩	20	Mohr-Coulomb	10	40
Shear Zone	🟥	15	Mohr-Coulomb	0	14

Project	10 Sinton Road, Hobsonville		
Group	Section A-A' Palisade Wall	*Scenario*	Saturated slope
Drawn By	JT	*Company*	ENGEO Ltd.
Date	18/10/2023, 9:27:53 am	*File Name*	10 Sinton Rd JT.slmd

SLIDEINTERPRET 9.030



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
ECBF Soils	▨	18	Mohr-Coulomb	5	32
East Coast Bays	■	20	Mohr-Coulomb	10	40
Shear Zone	🟥	15	Mohr-Coulomb	0	14
Puketoka Formation (Undrained)	🟨	18	Undrained	80	
Colluvium (Undrained)	▨	16	Undrained	50	

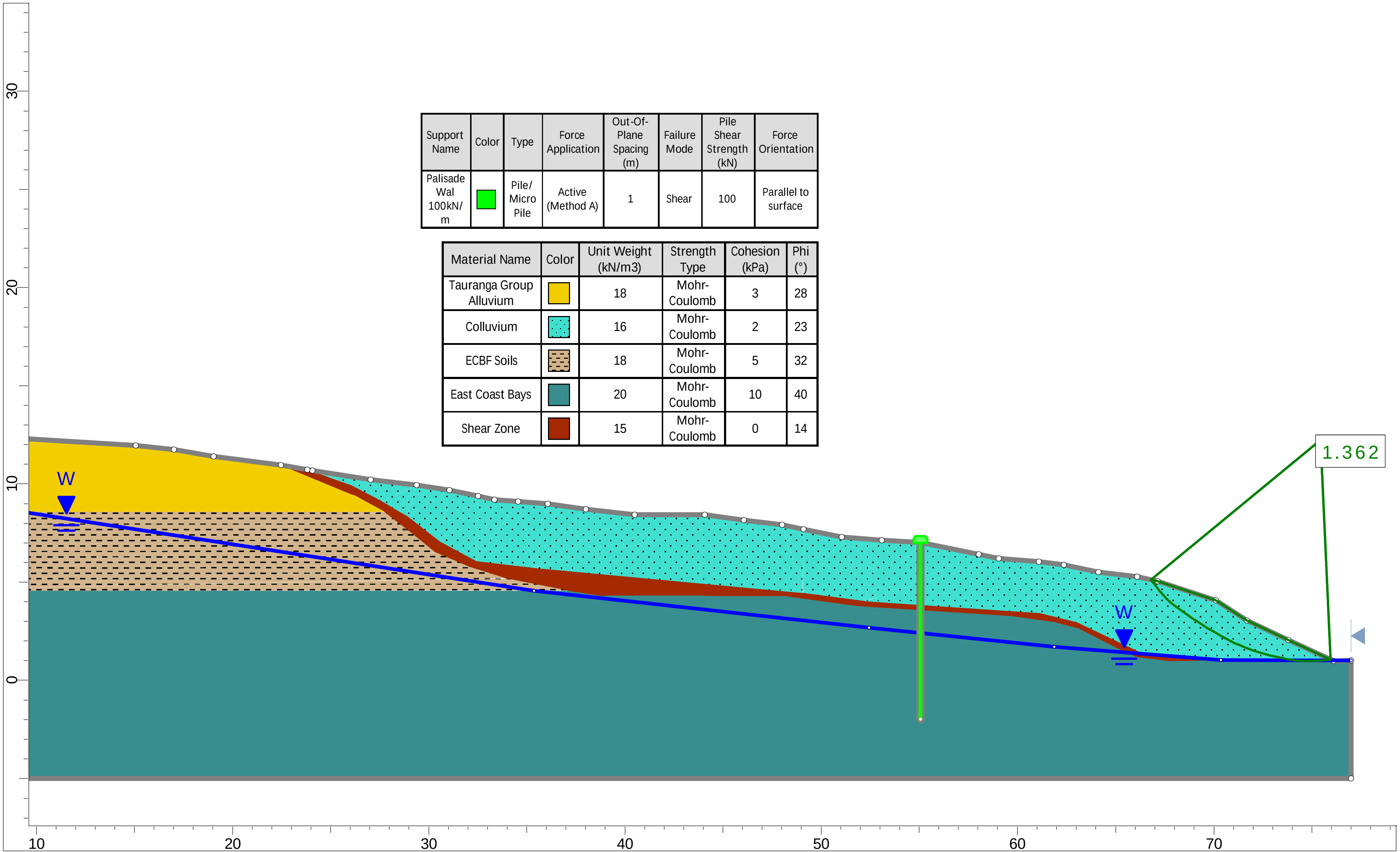
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Failure Mode	Pile Shear Strength (kN)	Force Orientation
Pallisade Wall 300 kN/m	🟦	Pile/ Micro Pile	Active (Method A)	1	Shear	300	Perpendicular to pile

Project	10 Sinton Road, Hobsonville		
Group	Section A-A' Palisade Wall	*Scenario*	Seismic
Drawn By	JT	*Company*	ENGEO Ltd.
Date	18/10/2023, 9:27:53 am	*File Name*	10 Sinton Rd JT.slmd

SLIDEINTERPRET 9.030

APPENDIX 7B:

Section B-B’ Stability Remediation



Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Failure Mode	Pile Shear Strength (kN)	Force Orientation
Palisade Wal 100kN/m	🟩	Pile/ Micro Pile	Active (Method A)	1	Shear	100	Parallel to surface

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Tauranga Group Alluvium	🟨	18	Mohr-Coulomb	3	28
Colluvium	▒	16	Mohr-Coulomb	2	23
ECBF Soils	▨	18	Mohr-Coulomb	5	32
East Coast Bays	🟦	20	Mohr-Coulomb	10	40
Shear Zone	🟥	15	Mohr-Coulomb	0	14

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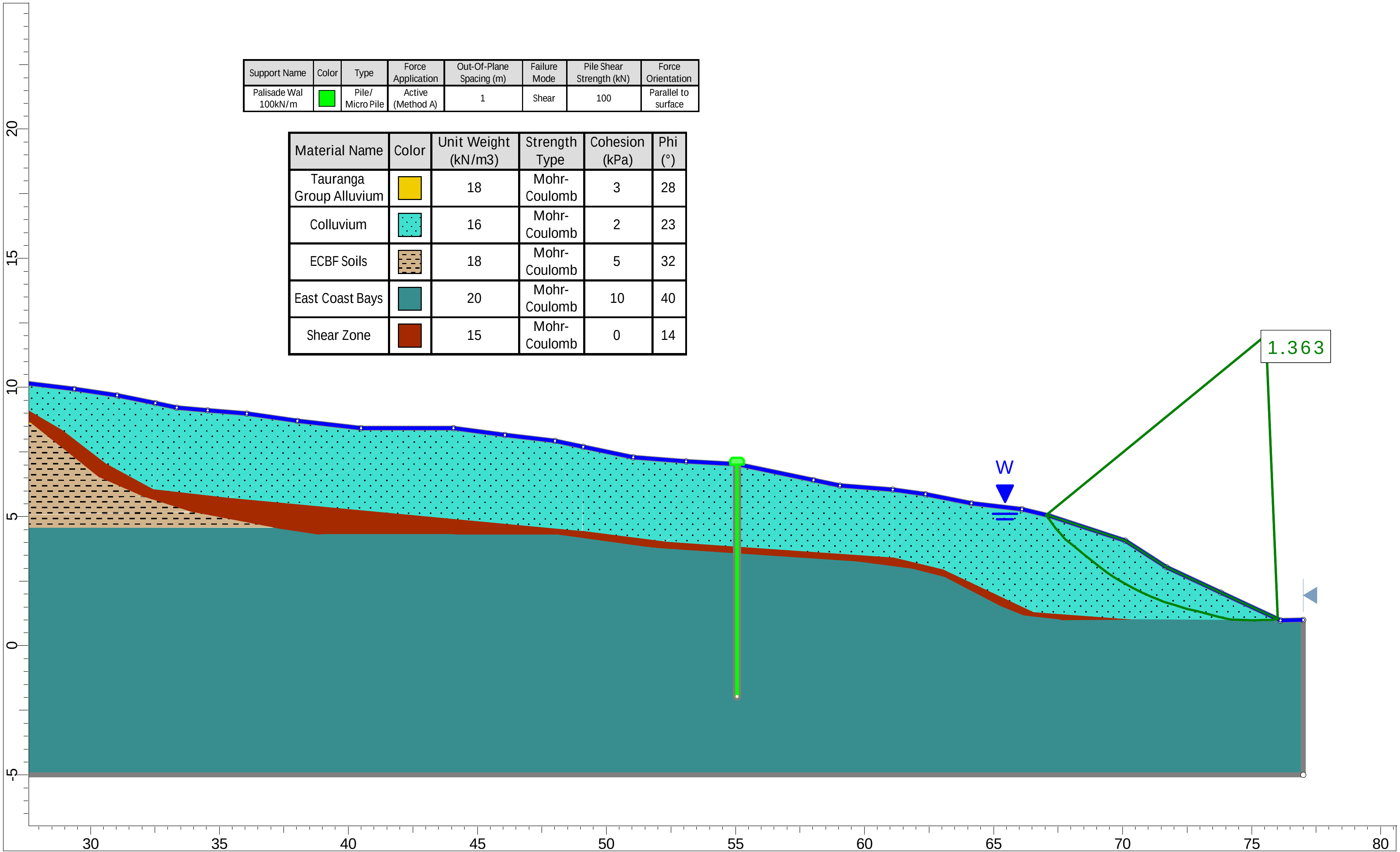
SLIDEINTERPRET 9.030

Project	10 Sinton Road, Hobsonville
Group	Section B-B'' Palisade Wall
Scenario	static
Drawn By	JT
Company	ENGEO Ltd.
Date	18/10/2023, 9:27:53 am
File Name	10 Sinton Rd JT.slmd

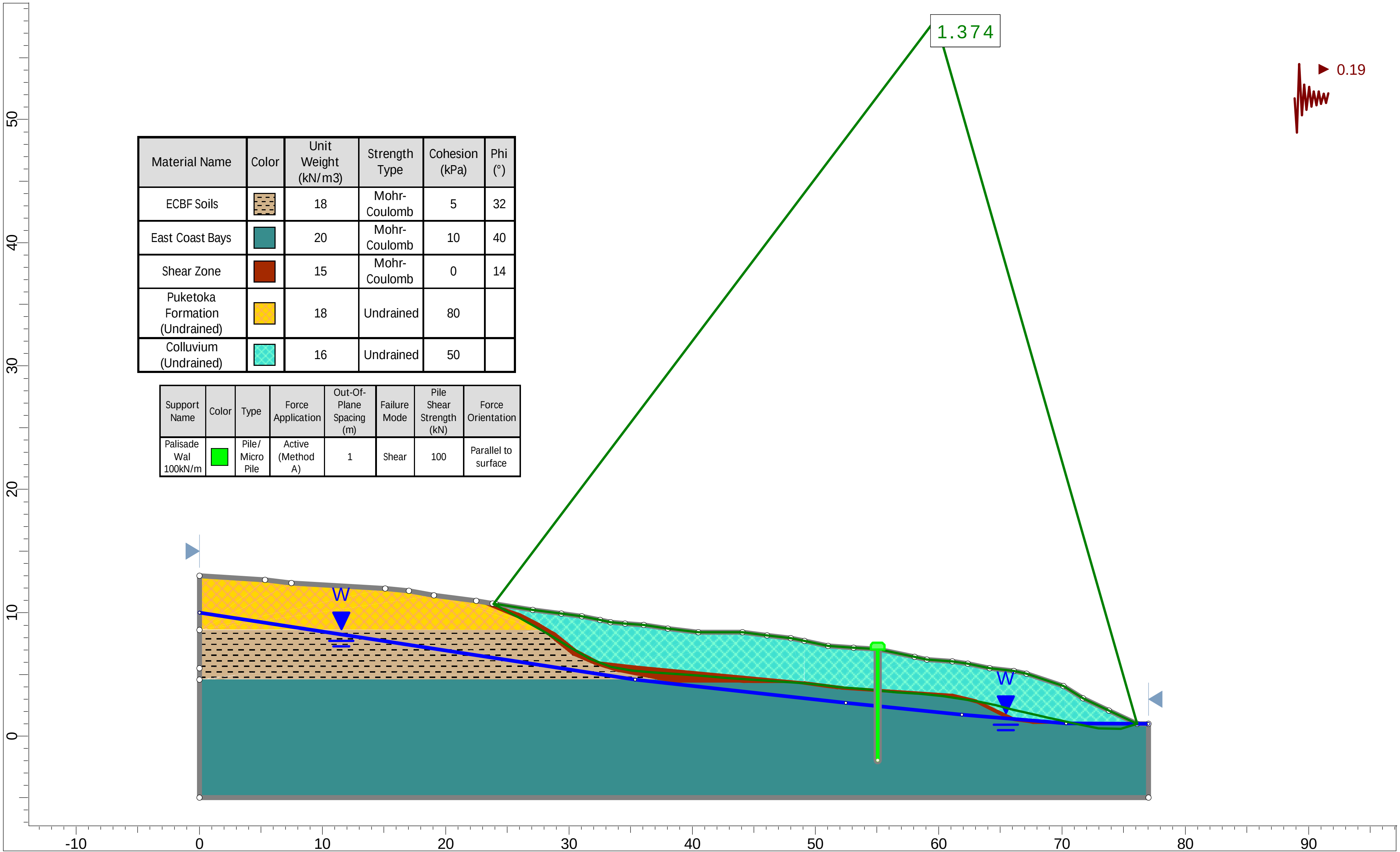


Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Failure Mode	Pile Shear Strength (kN)	Force Orientation
Palisade Wal 100kN/m	🟩	Pile/ Micro Pile	Active (Method A)	1	Shear	100	Parallel to surface

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Tauranga Group Alluvium	🟨	18	Mohr-Coulomb	3	28
Colluvium	🟦	16	Mohr-Coulomb	2	23
ECBF Soils	▨	18	Mohr-Coulomb	5	32
East Coast Bays	🟦	20	Mohr-Coulomb	10	40
Shear Zone	🟥	15	Mohr-Coulomb	0	14

SLIDEINTERPRET 9.030

Project	10 Sinton Road, Hobsonville		
Group	Section B-B'' Palisade Wall	*Scenario*	saturated slope
Drawn By	JT	*Company*	ENGEO Ltd.
Date	18/10/2023, 9:27:53 am	*File Name*	10 Sinton Rd JT.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
ECBF Soils	▨	18	Mohr-Coulomb	5	32
East Coast Bays	🟦	20	Mohr-Coulomb	10	40
Shear Zone	🟥	15	Mohr-Coulomb	0	14
Puketoka Formation (Undrained)	🟨	18	Undrained	80	
Colluvium (Undrained)	▨	16	Undrained	50	

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Failure Mode	Pile Shear Strength (kN)	Force Orientation
Palisade Wal 100kN/m	🟩	Pile/ Micro Pile	Active (Method A)	1	Shear	100	Parallel to surface

rocscience

SLIDEINTERPRET 9.030

Project	10 Sinton Road, Hobsonville		
Group	Section B-B'' Palisade Wall	*Scenario*	Seismic
Drawn By	JT	*Company*	ENGEO Ltd.
Date	18/10/2023, 9:27:53 am	*File Name*	10 Sinton Rd JT.slmd

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Project Number 23849.000.005

Geotechnical Investigation

14 Sinton Road, Hobsonville , Auckland

Submitted to:
Cabra Developments Limited
Unit 9b, 30 Foundry Road
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Auckland 0932

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Client		Cabra Developments Limited	**Client Contact**	Duncan Unsworth	
Distribution (PDF)		Duncan Unsworth (Cabra)			
Date	**Revision**	**Description**	**Author**	**Reviewer**	**WP**
22/02/2024	0	Issued to Client	JC	PF	DF
06/05/2024	1	Issued to Client	JC	HL	DF

1 Introduction

ENGEO Limited was requested by Cabra Developments Limited to undertake a geotechnical investigation of the property at 14 Sinton Road, Hobsonville , Auckland (herein referred to as 'the site'; shown in Figure 1). The purpose of this assessment is to support a Resource Consent application for the proposed redevelopment of the site. This work has been carried out in accordance with our signed agreement dated 11 November 2023.

We have been provided with an unnumbered draft masterplan of the site by DKO Architecture, dated 13 November 2023. This is attached in Appendix 1.

Our scope of works included:

- Desktop review of existing drawings for the site, a review of publicly available geological and geotechnical data and aerial photographs.
- Undertaking a site walkover to assess current site conditions and observe for geomorphological evidence of land disturbance, active and historical slope instability.
- Drilling of nine hand auger boreholes to a target depth of 5.0 m below ground level (bgl) with associated strength tests across the site to provide geotechnical data on the shallow soil profile.
- Undertaking six Cone Penetration Tests (CPTs) to a target depth of 15.0 m bgl to support a liquefaction assessment for the alluvial soils.
- Recovering two representative soil samples from near surface soils for laboratory expansive soils classification testing.
- Undertaking a liquefaction assessment using CPT data.
- Assessment of slope stability based on the geologic model developed from the site investigation and walkover data.
- Preparation of this Geotechnical Investigation Report presenting the findings of our investigation and geohazard assessments to support the Resource Consent application.

To support a Resource Consent application this report is required to reflect the earthworks proposals, particularly with respect to the slope stability assessment, which have not yet been developed. A supplementary assessment will be required to address the development proposals when available.

2 Site Description

The site comprises 2.3674 ha of joint residential and pastoral zoned land legally described as LOT 8 DP 57408. The site is located on an elevated coastal terrace bordered to the northwest by the Waiarohia Inlet, which is a tidal creek.

The site is accessed via two private driveways directly off 14 Sinton Street on the south-eastern side of the site. There is one dwelling with three smaller sheds or garages located in the south-eastern portion of the site. The northwestern side of the site contains grazing paddocks. To the east and west, the site is bounded by lifestyle blocks and residential blocks, to the south by Sinton Rd and to the north by the Waiarohia Inlet, the northern end of the site is densely vegetated.

An overland flow path runs southeast to northwest along the northern boundary and another smaller overland flow path feeds from the site into the neighbouring property, these flow paths divert surface water from the upslope portion of the site into the Waiarohia Inlet to the northwest.

3 Proposed Development

We have been provided with an unnumbered draft masterplan of the site by DKO Architecture, dated 13 November 2023. The masterplan shows two separate options; option 1, which indicates a total of 65 planned units; and option 2, which combines the sites at 14 and 16 Sinton Road for a total of 141 planned units. The plans also show proposed access roads. While the type of housing is not specified, we assume these will consist of one- to two-story residential structures.

No earthworks plans or proposed contours have been provided to ENGEO at the time of writing.

4 Desktop Study

4.1 Published Geology

The site is regionally mapped (1:250,000) by GNS Science[1] as spanning the geological boundary between the East Coast Bays Formation to the north and the Puketoka Formation of the Tauranga Group.

Puketoka Formation soils typically comprise pumiceous mud, sand and gravel with muddy peat and lignite. The alluvial nature of the soils means that it may commonly include sediment from a range of eroded sources and reworked material from underlying stratigraphic units including the East Coast Bays Formation.

East Coast Bays Formation comprises of alternating sandstone and mudstone with variable volcanic content and interbedded volcaniclastic grits. East Coast Bays Formation residual soils are generally described as orange and grey silts and clays with varying sand contents.

The boundary between these two units is mapped as inferred and runs across southwest-to-northeast across the centre of the site. Based on the scale of the regional mapping and the inferred nature of the contact it should be considered that the mapped boundary may not reflect the exact location of the geological contact within the site.

4.2 Auckland Council GeoMaps

4.2.1 Coastal Instability and Erosion

The Auckland Council GeoMaps layer ‘Areas Susceptible to Coastal Instability and Erosion’ identifies areas of coastline in Auckland that could be affected by coastal erosion and instability under a range of climate change scenarios and timeframes. The potential regression lines for 2050, 2080 and 2130 for this site are shown in Figure 1. These areas are limited to the northern slopes, along the Waiarohia Inlet.

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[1] https://data.gns.cri.nz/geology/

4.2.2 Flood Plains & Prone Areas

The Auckland Council GeoMaps layer ‘Flood Plains & Flood Prone Areas’ identifies areas of land in Auckland that could be affected by flooding during and / or following periods of heavy rain. Portions of the site labelled as flood prone or flood plains are shown in Figure 1 and are limited to areas adjacent to Waiarohia Inlet.

Figure 1: Auckland Council Hazard Map

4.3 Historical Aerial Photography Review

Aerial photographs of the site dating from 1940 to 2023 have been accessed from Auckland Council GeoMaps, Retrolens, Nearmaps and Google Earth Pro and these photos have been reviewed under the context of understanding past site use and to identify evidence of historical landform modifications. Table 1 provides a summary of our review findings. Aerial images are presented in Appendix 2.

Table 1: Summary of Historical Aerial Photographs

Date	Description
1940	The site and surrounding area comprise agricultural land; the site itself appears to be used for grazing. The northwest end of the site is vegetated and forms the edge of the Waiarohia Inlet.
1950	No significant changes to the site are noted.
1959	Some of the vegetation at the northwest end of the site has been cleared.
1963	No significant changes to the site are observed.
1968	Image quality is too poor to assess finer details; however, the west side of the site appears to have been divided into crop areas. In addition, two buildings have been constructed in the eastern portion of the site.

Date	Description
1972	Two small sheds appear to have been constructed on-site; one in the northwest section of the site close to the inlet, and one along the northeast boundary of the site.
1975	A rectangular shaped building has been constructed in the southern portion of the site. Two additional sheds have also been constructed towards the center of the site.
1978	A smaller building has been constructed to the south of the two original buildings (first observed in the 1968 aerial photograph). Fences appear to have been erected in the eastern quarter of site, and a small area to the northeast of the original buildings may be being utilised as an orchard.
1980	No significant changes to the site observed.
1988	The site has been divided into four sections. The northern, southern and western sections appear to be used for horticultural activity.
1996	Image quality is too poor to assess finer details; however, a swimming pool appears to have been constructed north of the original two buildings. In addition, three brown circular features can be seen in the northern section of the site that may represent stockpiled material.
2000	The circular features are no longer observed. The three sections previously used for horticultural uses appear to have been cleared.
2006	No significant change to the site observed.
2008	The shelterbelts dividing the site into sections appear to have been removed. Another light circular feature that seems to represent a stockpile has appeared on the north side of the site. An additional structure has been constructed to the north of the southern rectangular building. The structure at the center of the site appears to have been removed.
2010 / 2011	The previously observed stockpile appears to have been removed.
2012	No significant changes to the site are observed.
2017	No significant changes to the site are observed.
2018	No significant changes to the site are observed.
2019	A small building appears to have been constructed in the south side of the site.
2020	The previously mentioned building appears to have been removed.
2023	No significant changes to the site or surrounding area are observed.

No significant earthworks or landscape modification and no major slope instability is evident from these photos.

4.4 New Zealand Geotechnical Database

The New Zealand Geotechnical Database (NZGD) reveals that there have been a handful of past intrusive investigations in the vicinity of this site. These investigations comprise:

- Four hand auger boreholes along Sinton Road with the nearest two adjacent to the southern and eastern corner of the site extending approximately 200 m up the road (NZGD ID: HA_96977, HA_96979, HA_96981, HA_96984):
    - These boreholes were drilled by Maunsell Ltd in November 2005 to depths of 5.0 m bgl.
    - Materials encountered:
        - Topsoil between 0.0 and 0.3 m bgl.
        - Puketoka Formation clays and silts with varying sand contents between 0.0 and 5.0 m bgl, with measured shear strengths between 53 and 185+ kPa.
        - East Coast Bays Formation silts and sands with varying clay contents between 3.4 and 5.0 m bgl, with measured shear strengths between 99 and 185+ kPa.
- Four machine boreholes along the Upper Harbour Motorway (SH18), located approximately 200 m southeast of the site (NZGD ID: BH_205768, BH_205769, BH_205770, BH_205772).
    - These boreholes were drilled by Tonkin & Taylor between April and June 2021 to depths between 30.0 and 30.1 m bgl.
    - Materials encountered:
        - Topsoil between 0.0 and 0.9 m bgl.
        - Silt, clay and gravel fill between 0.0 and 3.0 m bgl.
        - Puketoka Formation clays and silts with varying sand contents between 0.9 and 10.08 m bgl, with measured shear strengths between 40 and 107 kPa.
        - East Coast Bays Formation silts and sands with varying clay contents between 4.45 and 13.13 m bgl, with measured shear strengths between 33 and 149 kPa.
        - East Coast Bays Formation sandstone and siltstone between 8.25 and 30.01 m bgl, with measured N values of 17 and 50+.
- One machine borehole in between Sinton Road and the Upper Harbor Motorway (SH18), located approximately 200 m southeast of the site (NZGD ID: BH96865)
    - This borehole was drilled by Meritec 2 May 2001 to a depth of 17.0 m bgl.
    - Materials encountered:
        - Topsoil between 0.0 and 0.2 m bgl.

- Completely weathered, very weak sandstone of the Waitemata Group comprising clayey silts, silty clays, and silty sands between 0.2 m and 3.5 m bgl, with measured N values of 3 and 4.
- Highly weathered, very weak sandstone [sic] of the Waitemata Group between 3.5 and 7.0 m bgl, with two measured N values of 14 and 14.
- Moderately weathered, very weak sandstone of the Waitemata Group from 7.0 m bgl to bottom of borehole at 17.0 m bgl, with measured N value of 25, 50, 44, 50+, 50+ and 50+.

5 Site Investigation

ENGEO visited site on 11 December 2023 to complete the following intrusive investigations:

- Six hand auger boreholes to a maximum depth of 5.0 m bgl with *in situ* strength testing (shear vane / Scala penetrometer testing).
- Three hand auger boreholes to a maximum depth of 3.0 m bgl with *in situ* strength testing (shear vane / Scala penetrometer testing).
- Two Scala penetrometer tests to 1.0 m bgl to attain CBR data.
- Collection of two samples from hand auger boreholes eight and nine at depths of 0.25 m to 0.8 m bgl and 0.8 m to 1.7 m bgl respectively.
- Six Cone Penetration Tests (CPTs) to a maximum depth of 15.0 m bgl.

While at the site we completed site observations and noted observed geomorphic features of concern or interest. The south-eastern portion of the site accommodated multiple dwellings, two driveways and also a pool. The investigation took place within the paddocks northwest of the dwellings. These paddocks were occupied by two horses and a chicken coop, with temporary and permanent fencing dividing the paddocks up.

The site gently sloped from south-east to northwest and at the north-western boundary it dropped at a moderate incline down into the Waiarohia Inlet. No tension cracking or ground rupture was observed within the heavily vegetated area in the north-western section of site. There were no evidence of severe instability within the site, although subtle signs may have been obscured by vegetation and long grass.

5.1 Subsurface Conditions

Topsoil

Our investigation generally identified a layer of organic topsoil between 0.1 m and 0.3 m thick covering the site.

Undocumented Fill

Hand auger borehole HA09 drilled on the eastern boundary of the site encountered undocumented fill at 0.25 m bgl. This undocumented fill was logged as very stiff clayey silt.

Puketoka Formation Alluvium

Investigation locations HA04, HA05, HA06 and HA08 encountered Puketoka Formation alluvial soils underlying topsoil. These soils consisted of stiff to very stiff cohesive silty clays and clayey silts.

Borehole HA05 encountered some minor amorphous organics within silty clay at a depth of 2.7 m bgl. Underlying this silty clay layer a sandy clay layer was encountered with an *in situ* strength profile ranging from very stiff at 4.2 m bgl dropping to soft at 4.8 m bgl.

East Coast Bays Formation

Hand auger boreholes HA01, HA02, HA03 and HA07 all encountered completely weathered East Coast Bays Formation soils underlying topsoil. These completely weathered soils were also encountered in HA06 underlying alluvial soils and in HA09 underlying undocumented fill.

These completely weathered soils were typically logged as clays, clayey silts and at greater depths sandy silts and silty sands which were dark grey, very stiff, or medium dense to dense.

5.1.1 Groundwater

Groundwater was measured at various levels when the boreholes were dipped at the conclusion of the drilling.

Table 2 presents a summary of groundwater observations at the site, including results from the previous CMW investigation. It should be noted that groundwater levels may fluctuate both seasonally and in the long term.

Table 2: Groundwater Observation Summary

Investigation Locations	Depth to groundwater (m)	Date
CPT01	8.5	11/12/2023
CPT02	Not measured	11/12/2023
CPT03	Not measured	11/12/2023
CPT04	1.0	11/12/2023
CPT05	0.9	11/12/2023
CPT06	Not measured	11/12/2023
HA01	2.2	11/12/2023
HA02	1.4	11/12/2023
HA03	1.1	11/12/2023
HA04	1.2	11/12/2023

Investigation Locations	Depth to groundwater (m)	Date
HA05	2.2	11/12/2023
HA06	1.3	11/12/2023
HA07	2.6	11/12/2023
HA08	2.4	11/12/2023
HA09	1.1	11/12/2023

These levels should be considered indicative only as they were recorded on the day of drilling and may not represent longer term levels.

5.2 Laboratory Testing

A soil sample was collected from boreholes HA08 and HA09 (logs in Appendix 3) for Atterberg Limits and Linear Shrinkage testing. This testing was undertaken in accordance with NZS4402:1986. Full results can be found in Appendix 4 and are summarised in Table 3.

Table 3: Atterberg Limits Testing

Sample ID	Sample Depth (m)	Water Content	Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage (%)
HA08	0.25 – 0.80	27.8	53	23	30	14
HA09	0.80 – 1.70	31.4	70	23	47	18

Expansive soils are classified in NZS 3604 as soils with a liquid limit of greater than 50% and a linear shrinkage greater than 15%.

6 Geohazard and Geotechnical Assessment

6.1 Soil Classification

Based on the findings of our desktop and subsurface investigation, as well as our experience of regional ground conditions, we consider the preliminary seismic site classification to be ‘Class C – Shallow Soil Sites’ in line with NZS 1170.5:2004[2] for the purpose of seismic design.

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[2] Standards New Zealand. (2004). Structural design actions – Part 5: Earthquake actions – New Zealand. Published 21/12/04.

6.2 Seismic Hazards

Potential seismic hazards resulting from nearby moderate to major earthquakes can be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, ground lurching, regional subsidence or uplift, soil liquefaction, lateral spreading, landslides, tsunamis, flooding, or seiches. Based on topographic and lithologic data, risk from earthquake-induced regional subsidence / uplift, ground lurching, and seiches are considered negligible at the site.

The following sections present a discussion of ground rupture, liquefaction risk, and other geohazards as they apply to the site.

6.2.1 Ground Rupture

There are no known active faults located within the site. Based on regional mapping, and the results of our field observations, it is our opinion that fault-related ground rupture is unlikely at the subject property.

6.2.2 Landslides

Landslides, while primarily found to occur during or following high rainfall events, can be triggered by earthquakes. Ground accelerations produced by earthquakes can significantly reduce the stability of inclined masses of soil, particularly where the soil is vulnerable to strain softening.

As the proposed lots are within the vicinity of sloping ground and historical landslides (at 10 Sinton Road), consideration must be given to the effects of earthquake loading on the stability of these features. These factors are considered in the slope stability analyses in Section 7.5.

6.2.3 Ground Shaking

Ground shaking and subsequent effects on structures, infrastructure and engineering systems can be extensive. The intensity, frequency and duration of ground shaking drives the effect of earthquake loading on structures, while the severity of ground shaking drives the level of ground deformation.

The level of ground shaking to which a building must be designed to withstand is dependent on the building's Importance Level as described in clause A3 of the Building Code. As the planned development is residential, we have assumed all buildings will be Importance Level 2 or lower. According to NZS 1170.5:2004, Importance Level 2 buildings are required to retain their structural integrity and not collapse or endanger life during an earthquake with a 500 year return period; the Ultimate Limit State (ULS) design seismic loading. They are further required to sustain little or no structural damage during an earthquake with a 25 year return period; the Serviceability Limit State (SLS) design seismic loading.

Peak horizontal ground accelerations (a_{max}) in accordance with NZGS Earthquake Geotechnical Engineering Practice Module 1, Appendix A1[3] are 0.19 g (ULS) and 0.05 g (SLS).

6.2.4 Liquefaction Analysis

We have assessed the potential of liquefaction triggering and liquefaction induced settlement occurring at the site by performing liquefaction analyses on the CPT data.

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[3] New Zealand Geotechnical Society (NZGS) and Ministry of Business, Innovation and Employment (MBIE) (2021). Earthquake geotechnical engineering practice Module 1: Overview of the guidelines, Version 1, November 2021.

Soil liquefaction and lateral spreading results from the loss of strength during cyclic loading, such as that imposed by earthquakes. Soils most susceptible to liquefaction are typically identified as clean, loose, saturated, cohesionless materials. Empirical evidence indicates that some silty sands, low plasticity silts and low plasticity clays are also potentially liquefiable or may be subject to strain softening. Lateral spreading occurs as a result of liquefied material moving toward a sloping area or free face. This is most common in sloping ground, backfills behind retaining walls, open stormwater channels and water frontage areas. Thin layers, particularly those that are not laterally extensive, are unlikely to liquefy if they are surrounded by non-liquefiable soils.

Liquefaction Methodology

We have assessed the potential of liquefaction triggering and liquefaction induced settlement occurring at the site by performing liquefaction analyses on the CPT data based on the liquefaction triggering methodologies presented by Boulanger and Idriss[4] and using the proprietary software CLiq v.2.3.1.15.

Our analysis included the following assumptions and inputs:

- Ground motion parameters as outlined in Section 6.2.3.
- A maximum earthquake magnitude groundwater level of 1.0 m to reflect the shallowest groundwater level observed within the hand auger boreholes.
- The Zhang and Brachman[5] (2002) procedure for estimating volumetric strain and vertical settlement for the CPT settlement.
- The Boulanger and Idriss relationship between fines content and Soil Behaviour Type (lc) with a fitting parameter (CFC) of 0.0 for the CPT analysis (no soil laboratory testing available for calibration of the parameter.

Liquefaction Discussion

Full results of our analyses are presented in Appendix 5, a summary is presented in Table 4 below:

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[4] Boulanger, R.W. and Idriss, I.M. (2014). CPT and SPT based liquefaction triggering procedures. Centre for Geotechnical Modeling. Department of Civil & Environmental Engineering, University of California. Report No. UCD/CGM-14/01. April 2014.

[5] Zhang, G.; Robertson, P.K.; and Brachman, R.W.I. (2002). Estimating liquefaction-induced ground settlements from CPT for level ground. Canadian Geotechnical Journal 39: 1168–1180. DOI: 10.1139/T02-047

Table 4: Ultimate Limit State LSN, LPI and Calculated Vertical Settlement

CPT	LPI	LSN	Calculated Vertical Index Settlement (SLS)	Calculated Vertical Index Settlement (ULS)	MBIE Module 3 Performance Level
CPT01	Negligible	0.3	Negligible	2 mm	L_0
CPT02	Negligible	Negligible	Negligible	Negligible	L_0
CPT03	Negligible	0.2	Negligible	1 mm	L_0
CPT04	Negligible	Negligible	Negligible	Negligible	L_0
CPT05	Negligible	0.2	Negligible	< 1 mm	L_0
CPT06	Negligible	Negligible	Negligible	Negligible	L_0

This analysis indicates that under SLS conditions, liquefaction is not predicted to occur at site. Under ULS conditions, liquefaction is predicted to occur in several silty sand and sandy silt horizons that are typically less than 0.1 to 0.2 m thick.

Based on the distribution and size of the liquefiable layers, and the low Liquefaction Potential Index (LPI) and Liquefaction Severity Number (LSN), we anticipate the surface effects of ULS liquefaction to be minor with settlements within building code tolerances.

Table 5.1 of MBIE / NZGS Module 3[6] indicates that the ULS liquefaction induced settlements on this site are within the insignificant category (L_0). The consequences are described as 'No significant excess pore water pressures (no liquefaction)'.

6.3 Expansive Soils

Expansive soils shrink and swell as a result of seasonal fluctuation in moisture content. This can cause heaving and cracking of on-grade slabs, pavements, and structures founded on shallow foundations.

Building damage due to volume changes associated with expansive soils can be reduced through proper foundation design. Successful performance of structures on expansive soils requires special attention during design and construction. It is imperative that exposed soils be kept moist prior to placement of concrete for foundation construction. It is extremely difficult to re-moisturise clayey soils without excavation, moisture conditioning, and re-compaction.

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[6] New Zealand Geotechnical Society (NZGS) and Ministry of Business, Innovation and Employment (MBIE) (2021). Earthquake geotechnical engineering practice Module 3: Identification, assessment and mitigation of liquefaction hazards, November 2021.

Based on our laboratory assessment of the near surface soils and our experience with similar soils within the region, we consider a preliminary soil classification of M (moderately) expansive with respect to NZS 3604 (from Section 3.2 of B1/AS1 November 2019 Amendment) is suitable for this site.

It is considered that this preliminary recommendation may be refined with further site-specific testing at the Geotechnical Completion Report stage, following earthworks.

6.4 Coastal Regression Hazard

The northern boundary of the site has been identified by Auckland Council as being potentially susceptible to coastal instability and erosion. The potential regression lines for 2050, 2080 and 2130 are mapped within the proposed council esplanade area and are shown Figure 1. As such, a site-specific coastal hazard assessment undertaken by a Coastal Engineer will be required to support a Resource Consent application.

6.5 Slope Stability

ENGEO has completed a slope stability analyses along line of section AA' shown on the investigation plan, capturing the steepest part of the creek bank slope. Numerical slope stability analyses were conducted using the software package SLIDE2, produced by Rocscience Limited. Analyses were completed using the GLE / Morgenstern-Price method to identify areas of possible circular and non-circular slope instability using Cuckoo search method with optimisation of failure surfaces being enabled via the Surface Altering option. We considered three scenarios in the stability analysis: Static (Long Term) Groundwater (using the measured groundwater levels), Transient (Elevated) Groundwater (considering the worst credible groundwater level) and Seismic (considering the seismic loadings).

6.5.1 Design Criteria

The requisite factors of safety (FoS) for residential development in Auckland are outlined in Table 5.

Table 5: Slope Stability Factor of Safety Requirements

Scenario	Requisite Factor of Safety
Long term static conditions	1.5
Short term transient conditions	1.3
Seismic Conditions	1.0

These FoS have been assessed using the GLE-Morgenstern Price method for non-circular failure. For the seismic scenario ULS peak ground acceleration as determined in Section 6.2.3 has been adopted (0.19 g).

For conservatism, we have assumed a constant 20 kPa surcharge to account for potential building, traffic and other live loads. These can be refined once more detailed plans become available.

6.5.2 Material Parameters

Material parameters were adopted for our slope stability and remediation analyses based on *in situ* testing within our hand augers, Su and CPT correlations, and local experience.

A summary of these derived parameters is presented in Table 6.

Table 6: Geotechnical Parameters

Geological Unit	Unit Weight (kN/m³)	Effective Stress Parameters		
		Ø' (°)	c' (kPa)	Undrained Shear Strength (kPa)
East Coast Bays Residual Soil (Clay)	18	32	5	100
East Coast Bays Residual Soil (Sandy/Silty)	18	34	2	n/a
East Coast Bays Formation Transition Zone*	18	36	10	n/a
East Coast Bays Formation Rock	20	40	10	n/a

*Inferred from CPT tip resistance and soil behaviour type

6.5.3 Slope Stability Results

Outputs of our slope stability analyses are presented in Appendix 6.

The results of our analysis indicate that under static, transient and seismic conditions, the slope section meets the target Factors of Safety under existing site conditions.

A summary of our analyses results are presented below:

Table 7: Summary of Slope Stability Analyses

Condition	Factor of Safety
Static	1.63
Transient	1.30
Seismic	1.23
Note: red = FoS below requirements, green = satisfies requirements	

6.6 Settlement

The Puketoka Formation comprises alluvial sediments. In alluvial environments, peat forms in areas with low sediment input, typically on the margins on small, slow flowing channels. These become buried beneath sediment as the channel migrates subsequently forming a peat containing paleo-channel. Although not encountered at this site, peat and organic soils have been encountered at other sites in the area and extensive organic deposits are known to be present south of the site in the vicinity of the Upper Harbour Motorway.

Peat is considered an unacceptable bearing stratum for foundations as it is highly susceptible to consolidation due to its high-water content (peat may contain ten times its own weight in water). Under the load of fill and building foundations, peat can reduce its volume by up to 75% resulting in significant vertical settlement. Peat is also vulnerable to wasting where it is found above the groundwater table as oxidation of the biomass results in the peat decaying / decomposing. Primary settlement of peat may take days whereas secondary creep consolidation settlement behaviour due to the decay of organic material may continue over 50+ years.

We should be given the opportunity to review the earthworks proposals for the site when they are developed, prior to building consent, to assess whether the magnitude of cut or fill earthworks may present a settlement risk to the development. Additional investigations may be recommended to confirm the presence or absence of organic or otherwise weak / compressible soils in the vicinity of deep excavations or large fills to appropriately characterise the settlement risk. Where potential consolidation settlements are found to be beyond building code tolerances, suitable solutions may include undercutting and replacing the peat with engineered fill or piled foundations extending below the peat.

6.7 RMA Section 106 Assessment and Development Suitability

Section 106 of the Resource Management Act (RMA) states that a consent authority may refuse to grant a Subdivision Consent, or may grant a consent subject to specific consent conditions if it considers that:

- There is significant risk from natural hazards; or
- Sufficient provision has not been made for legal or physical access to each allotment to be created by the subdivision.

An assessment of the risk from natural hazards as required by the RMA includes the following:

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

We have assessed the risk of natural hazards at the site in accordance with Section 106 of the Resource Management Act (RMA) and considered the risk to the site from erosion, rockfall, inundation (debris), slope stability, subsidence, flooding and tsunami.

Based on our investigation, assessment and site observations, we consider it is unlikely for the site to be subject to the aforementioned natural hazards providing suitable engineering measures are included in the site development (as discussed in Section 7). As such, the site is considered to be conditionally suitable for the proposed residential development from a geotechnical perspective.

7 Geotechnical Recommendations

Based on the results of our geotechnical investigation and subsequent assessment, we consider the site to be generally suitable for the proposed development subject to our geotechnical recommendations being followed.

However, as mentioned in Section 6 the site is at risk from a number of identified geohazards including the following:

- Instability of the over steepened north-western slope bordering Waiarohia Inlet.
- Portions of the site may be vulnerable to settlement due to the potential presence of compressible alluvial soils.
- Shallow site soils are moderately expansive and may be susceptible to shrinkage and heave.

7.1 Foundations

Shallow soils at the site typically comprised stiff to hard clays and silts of the Puketoka and East Coast Bays Formations. It is our preliminary recommendation that site soils will likely be suitable for a geotechnical ultimate bearing capacity of 300 kPa for shallow foundations constructed on competent natural ground beneath any topsoil and existing undocumented fill or on engineer certified fill.

This preliminary recommendation will be revisited once an earthwork plan has been provided as significant cuts may expose weaker soil horizons with a reduced bearing capacity. Any bearing capacities provided during the design phase are subject to change and revision in the geotechnical completion report to be issued for the site following the satisfactory completion of earthworks.

It is considered likely that the soils on-site will fall within site class M (moderately expansive), with respect to NZS 3604 (from Section 3.2 of B1/AS1 November 2019 Amendment). This will be reassessed as part of the completion reporting for this site.

7.2 Earthworks

- As noted in Section 5, possible undocumented fill is present on-site. Any undocumented fill soils should be undercut to the depth that native soils are exposed.
- Excavations and temporary cuts should not exceed a batter angle of 1V:2H up to 2 m in height and should not be left unsupported for longer than two weeks. Cuts beyond this height should be referred to the Geotechnical Engineer for stability assessment.
- Where vertical and subvertical faces higher than 1.0 m are required, we recommend that this is done in shortened sections (< 5 m) and the faces are left unsupported for a minimal time period (i.e. one week) or temporarily shored.
- All temporary cuts and batters proximal to boundaries should take into account the potential surcharge and risk of undermining neighbouring property.
- Suitable drainage channels must be put in place to divert surface water from unsupported cut faces. Subsurface drains should also be considered for the toe of the long-term slopes.
- If any permanent cuts have a batter steeper than 1V:4H and are to be higher than 1.5 m, they should be supported with a specifically designed retaining wall (approved by a chartered Geotechnical Engineer) or be referred back to the Geotechnical Engineer for stability assessment and specific batter design.

- All cuts and batters should be in line with the WorkSafe Good Practice Guidelines for Excavation Safety (July 2016). Permanent fill batters should not exceed 1V:3H and should be reviewed by the Geotechnical Engineer as part of the site development and earthworks proposal review. Fill batters exceeding 1V:3H will require specific geotechnical assessment.

- All excavations should be inspected by ENGEO (or a suitably qualified Geotechnical professional), prior to constructing foundation elements to verify founding conditions are as anticipated.

- Suitable underfill drainage should be considered for any filling on slopes, within stream gully features and wherever seepage is observed within the stripped surface.

- All engineered or structural fill should be placed in ≤ 200 mm compacted lifts and be compacted to a minimum of 95% of maximum dry density, at no less than optimum moisture content. Maximum dry density for granular fill materials may be obtained from the source quarry, a geotechnical laboratory or from plateau testing undertaken on-site. Compaction should be achieved using standard plant and methodology suitable for the imported material. A water source should be maintained on-site for moisture control.

- All excavated soil should be removed from site or placed in an engineer approved stockpile to avoid unfavorable loading on construction or preconstruction slope batters.

Material Suitability

Earthworks' operations involving borrow materials, usually from the elevated portions of the site, should be relatively straightforward. Generally, both the cuts and fills will involve inorganic, alluvial clayey silts and silty clays that should be suitable, with conditioning for handling and compaction by conventional earthmoving plant. It should be noted though that moisture contents will increase with depth in the cut areas and also in the lower lying areas.

Our experience with the types of native soils present on this site indicates that when they are exposed to the weather their strengths may be significantly reduced. We therefore recommend that trafficked areas and building platforms are only trimmed to final levels immediately prior to placing hardfill / topsoil and that at all times the site is shaped to avoid water ponding during rain, thereby limiting the need for additional undercuts. On no account should areas of trimmed subgrade be left exposed to allow the ingress of water, nor should subgrade areas be trafficked prior to drying out after rain.

Unsuitables

Topsoil and organic soils are not suitable for bearing foundations or for reworking and re-use as engineered fill and should be undercut and stockpiled away from the earthworks area. Undocumented fills encountered on-site may be suitable for re-use as engineered fill following approval of the Geotechnical Engineer.

7.3 Service Lines

The construction and installation of new services lines within alluvial material may intercept flowable sands and organic / peat layers. Particular attention should be paid to drainage and stability of trench walls under such circumstances.

Where the base of service line trenches encounters weak, flowable sands and / or organic soils, increased bedding depths of up to 70% and undercuts of approximately 300 mm plus geotextile wrapping of the bedding may be required to provide adequate support to the services and limit the chance of differential settlement along low gradient service alignment. Specific bedding modifications are best prescribed when the trenches are excavated and the material at invert level are examined in detail by a geotechnical professional.

Construction of services during the winter months may pose a risk of trench wall collapse within soft alluvial soils partly due to raised groundwater, leading to the need for additional support, alternative construction methodology and / or dewatering. This should be allowed for on-site by the contractors. Methods to deal with this could be, but not limited to, trench shields to support service trench walls, benching or excavations to a safe temporary works angle (e.g., 1(H):1(V)).

Should flowable sands and / or organic soil layers be encountered during service line trenching, the contractor shall contact ENGEO.

7.4 Retaining Walls

7.4.1 Internal retaining walls

Currently there are no internal retaining structures shown on the development plans. Any future retaining should be designed to accommodate for the soils encountered on-site. Based on our subsurface investigations, we expect internal retaining structures to support native Puketoka Formation and East Coast Bays Formation soils.

Preliminary Retaining Wall Parameters

Based on the results of our investigation and the ground conditions at site, future retaining walls should be designed using the following geotechnical parameters:

Table 8: Retaining Wall Parameters

Material Type	Unit Weight	Friction Angle (°)	Effective Cohesion c' (kPa)	Undrained shear Strength Su (kPa)
Native (Clay)	18	32	5	80
Native (Silts/Sands)	18	34	2	n/a
Cohesive Engineered Fill	18	32	5	100
Granular Engineered Fill	20	38	0	-

7.5 Stormwater and Effluent Disposal

ENGEO have not been provided with plans showing the preferred methods of stormwater and wastewater disposal.

Based on the preliminary plans that have been provided we anticipate that wastewater will be disposed of via reticulated council services.

Overland flows should be directed away from existing slopes to reduce the risk of ponding and erosion exacerbating slope instability concerns.

7.6 Pavement Subgrade CBR

Based on the Scala penetrometer tests performed on-site, we recommend that a CBR of approximately 4% be adopted for native soils and 6% for cohesive engineered fill areas are considered to be suitable for preliminary design purposes. These values are derived from the soils encountered in our hand auger boreholes and our knowledge of the soil type on-site.

The above CBR values are preliminary only. Specific *in situ* and laboratory testing of the exposed subgrade is recommended following earthworks and prior to finalising pavement designs, including the use of *in situ* and soaked CBR testing and falling weight deflectometer. Where localised uncontrolled fill is encountered, it will be necessary to remove this fill and replace it with engineered fill. Additional subgrade improvement requirements may be necessary to achieve council requirements. This may include undercut and replacements, and / or the use of triaxial geogrid.

8 Future Work

We recommend ENGEO’s involvement in the following future activities:

- Detailed review of landform / earthworks design and revised slope stability analysis to reflect design ground profiles in the context of slope instability and potentially compressible soils.
- Preparation of a Geotechnical earthworks specification.
- Observation and certification of earthworks and retaining walls including all stripping and undercuts and engineered fill in accordance with the earthworks and retaining wall specifications.
- Geotechnical Completion Reporting / Producer Statements.

9 Limitations

i. We have prepared this report in accordance with the brief as provided. This report has been prepared for the use of our client, Cabra Developments Limited, their professional advisers and the relevant Territorial Authorities in relation to the specified project brief described in this report. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity.

ii. The recommendations in this report are based on the ground conditions indicated from published sources, site assessments and subsurface investigations described in this report based on accepted normal methods of site investigations. Only a limited amount of information has been collected to meet the specific 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 judgement and it should be appreciated that actual conditions could vary from the assumed model.

iii. 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.

iv. This Limitation should be read in conjunction with the Engineering NZ/ACENZ Standard Terms of Engagement.

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

We trust that this information meets your current requirements. Please do not hesitate to contact the undersigned on (09) 972 2205 if you require any further information.

Report prepared by

Jerry Chen
Geotechnical Engineer

Report reviewed by

Paul Fletcher, CMEngNZ (CPEng)
Principal Geotechnical Engineer

Heather Lyons, CMEngNZ (PEngGeol)
Associate Engineering Geologist

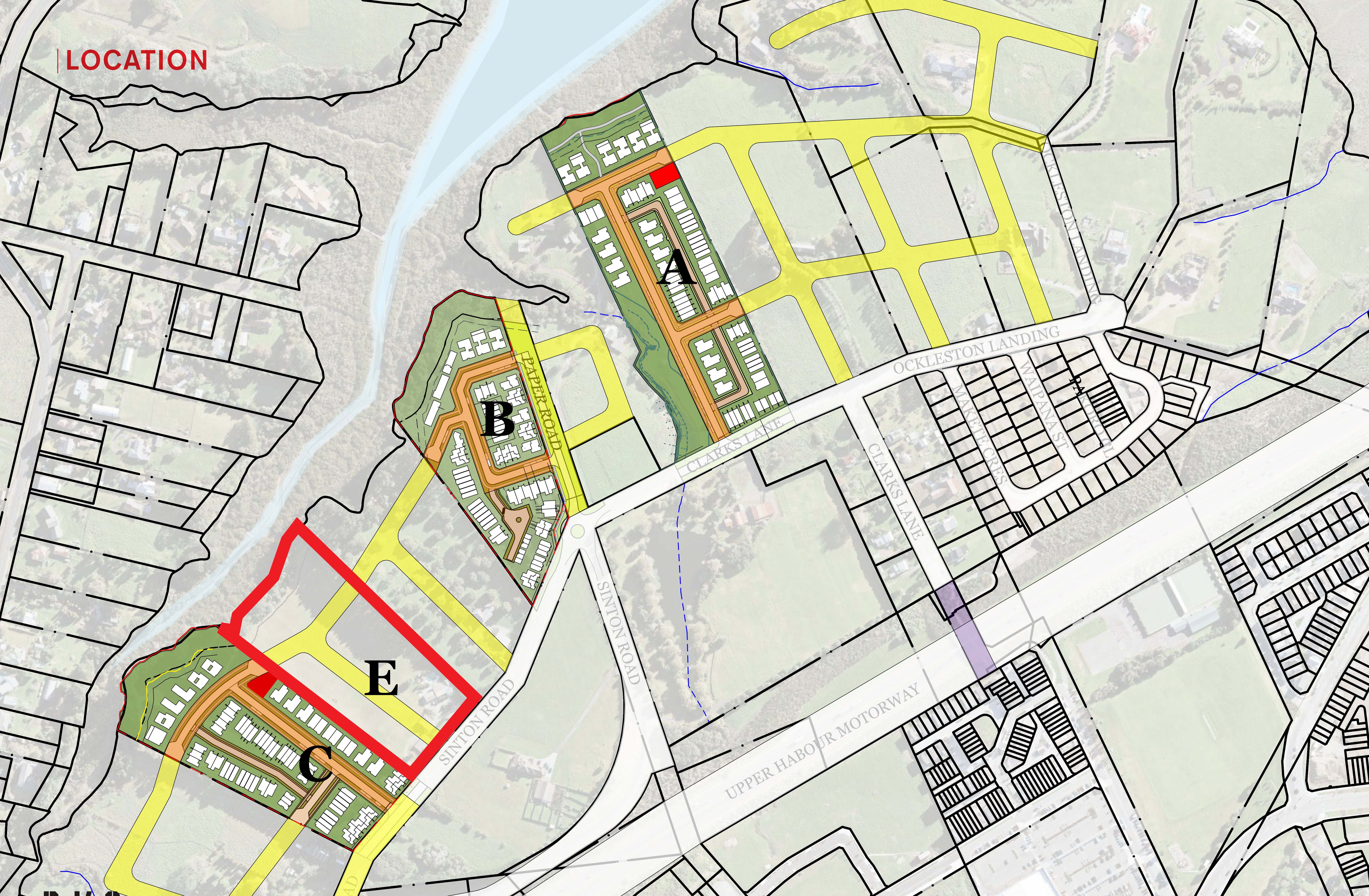
APPENDIX 1:

DKO Plans

14 Sinton Road - Site E

CONCEPT MASTERPLAN

LOCATION



CLIENT NAME
CABRA
DEVELOPMENTS

PROJECT #00013151
14 SINTON ROAD, WHENUAPAI

OVERALL MASTERPLAN - OPTION 1

DKO

CLIENT NAME

CABRA
DEVELOPMENTS

PROJECT #00013151
14 SINTON ROAD, WHENUAPAI

OPTION 1

SITE E
65 UNITS

SITE C
73 UNITS

6 ADDITIONAL UNITS

144 TOTAL UNITS

DKO

CLIENT NAME
CABRA DEVELOPMENTS

PROJECT #00013151
14 SINTON ROAD, WHENUAPAI

Merging Site C and E

OVERALL MASTERPLAN - OPTION 2

OPTION 2

141 TOTAL UNITS

- Increased number of units.
- Improved lot-to-building ratio for greater efficiency.
- Achieve a even distribution of density across the two sites by reducing the 4.75m width typologies.
- Enhanced steetscape frontage for a better overall appearance.
- Less COAL's.
- Reducing the intersections along Sinton Road to alleviate congestion by establishing single point of entry for vehicles.
- Diverse typologies variations.

DKO

CLIENT NAME
CABRA DEVELOPMENTS

PROJECT #00013151
14 SINTON ROAD, WHENUAPAI

Thank you.

APPENDIX 2:

Historical Aerials

1940 (Retrolens NZ)

1950 (Retrolens NZ)

1963 (Retrolens NZ)

1968 (Retrolens NZ)

1972 (Retrolens NZ)

1975 (Retrolens NZ)

1978 (Retrolens NZ)

1980 (Retrolens NZ)

1988 (Retrolens NZ)

1996 (Auckland Council GeoMaps)

2000 (Auckland Council GeoMaps)

2006 (Auckland Council GeoMaps)

2008 (Auckland Council GeoMaps)

2010/2011 (Auckland Council GeoMaps)

2012 (Auckland Council GeoMaps)

2017 (Auckland Council Geomaps)

2018 (Nearmaps)

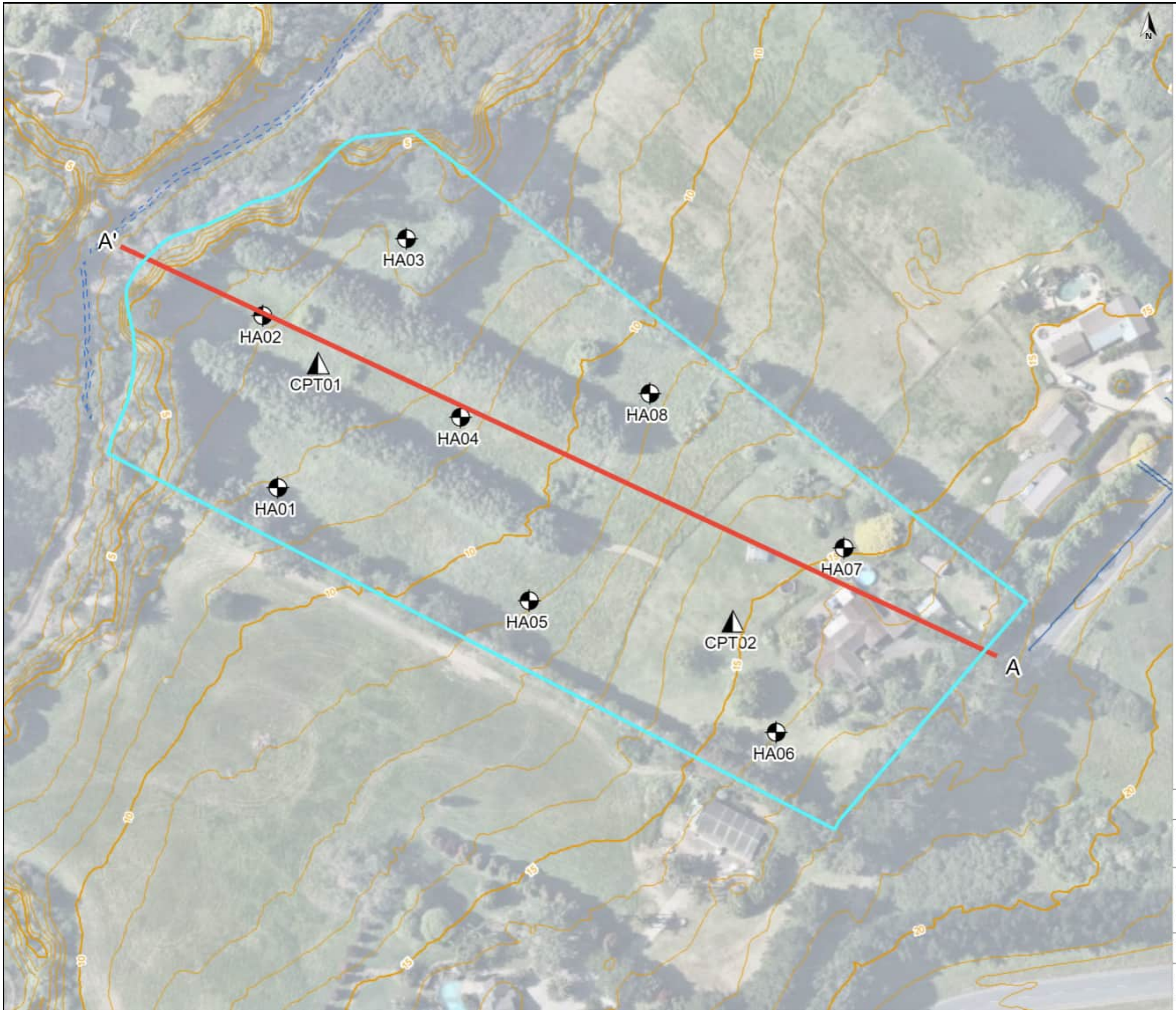
2019 (Nearmaps)

2020 (Nearmaps)

2023 (Nearmaps)

APPENDIX 3:

Site Investigations



Legend

□ Site Boundary

— Interpretive Geological Cross -Section A-A'

⊕ ENGEO Hand Auger Boreholes (August 2023)

▲ ENGEO CPT (August 2023)

0 25 m 50 m

© Nearmap, , public

ENGEO

Produced by **Datanest.earth**

Title: Investigation Location Plan

Client: Cabra Developments Limited

Size: A4

Project: 16 Sinton Road, Whenupai

Drawn: JL

Appendix No.: 1

Date: 03-05-2024

Checked: HL

Proj No: 23849.000.001_04

Scale: 1:1570

Version: 1

ENGEO

LOG OF AUGER HA01

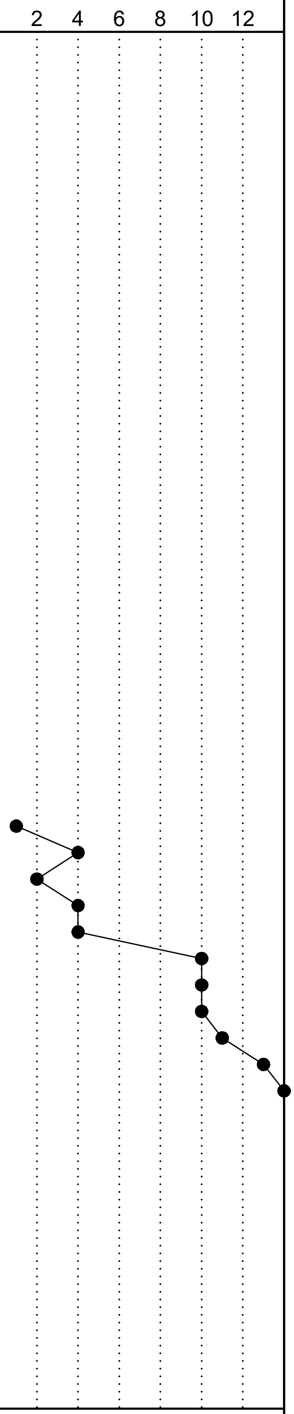
Geotechnical Investigation
14 Sinton Road
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.005
Date : 11-12-2023
Hole Depth : 4 m
Hole Diameter : 50 mm
Shear Vane No : 3843
Logged By : RL
Reviewed By : NM
Latitude : -36.7949748
Longitude : 174.6399653

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
	TS	OL	[TOPSOIL]				M	N/A	169/51
0.5	EAST COAST BAYS FORMATION	ML	Clayey SILT with minor sand; orange with brown and light grey streaks. Low plasticity; sand is fine. 0.8 m - Becomes light grey with occasional orange mottles.					VSt	157/29 159/34
1.0 1.5 2.0		CH	CLAY with some silt and trace sand; light grey with orange mottles. High plasticity; sand is fine.		7 6	▽		VSt	152/44 160/74 169/67 148/59
2.5		ML	Sandy SILT with minor clay; light grey with orange streaks. Low plasticity; sand is fine. 2.6 - 2.95 m - Poor recovery.				W	VSt	148/37 165/40
3.0 3.5		SM	Silty SAND with minor silt and trace clay; dark grey. Poorly graded; sand is fine to medium. 3.5 m - Becomes dense.		5		S	L-MD D	
4.0 4.5 5.0			End of Hole Depth: 4 m Termination Condition: met practical refusal		4				

Scala Penetrometer

Blows per 100mm



Hand auger met practical refusal at 4 m depth on hard material.
Scala Penetrometer met practical refusal at 4 m depth.
Dip test showed standing water at 2.2 m depth during drilling.
Elevations and coordinates estimated from Auckland Council GeoMaps.

N/A = Not Applicable; TS = Topsoil

ENGEO

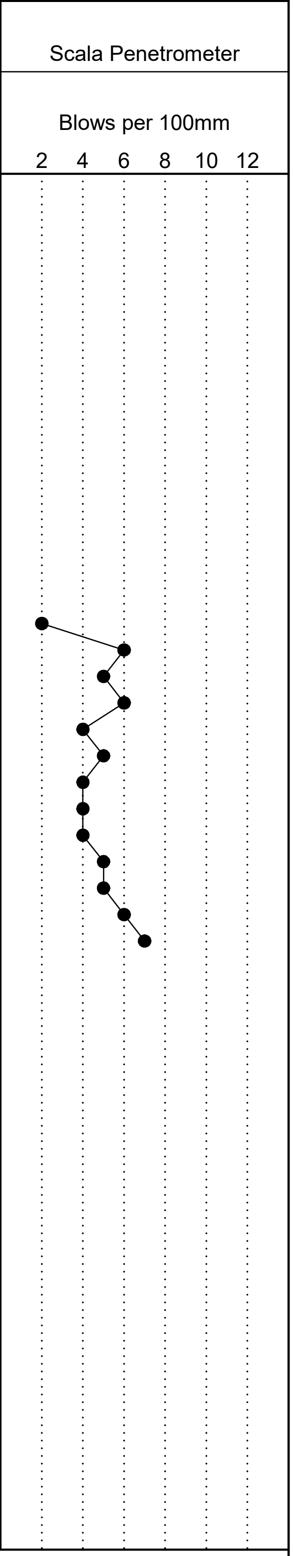
LOG OF AUGER HA02

Geotechnical Investigation
14 Sinton Road
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.005
Date : 11-12-2023
Hole Depth : 3 m
Hole Diameter : 50 mm
Shear Vane No : 3843
Logged By : RL
Reviewed By : NM
Latitude : -36.7951129
Longitude : 174.6401559

Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
TS	OL	[TOPSOIL]			M	N/A	
EAST COAST BAYS FORMATION	CH	CLAY with minor silt and trace sand; light grey with orange mottles. High plasticity; sand is fine. 0.4 m - Becomes very stiff.	8	▽		St VSt	84/20 121/34 182/98 138/47
	ML	Clayey SILT with some sand; light grey. Low plasticity; sand is fine to medium.			W	VSt	118/39
	SM	Silty SAND; light grey with occasional orange streaks. Poorly graded; sand is fine to medium. 2.1 m - Becomes saturated.	7		W S	L - MD	
	ML	Sandy SILT with minor silt and trace clay; dark grey with orange streaks. Well graded; sand is fine to coarse. 2.7 m - 3.0 m - Poor recovery.	6		S	St - VSt*	

End of Hole Depth: 3 m
Termination Condition: met target depth



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

Hand Auger met target depth at 3 m depth.
Scala Penetrometer met target depth.
Dip test showed standing water at 1.4 m depth during drilling.
Elevations and coordinates estimated from Auckland Council GeoMaps.

N/A = Not Applicable; TS = Topsoil
*Consistency determined from tactile assessment (NZGS, 2005).

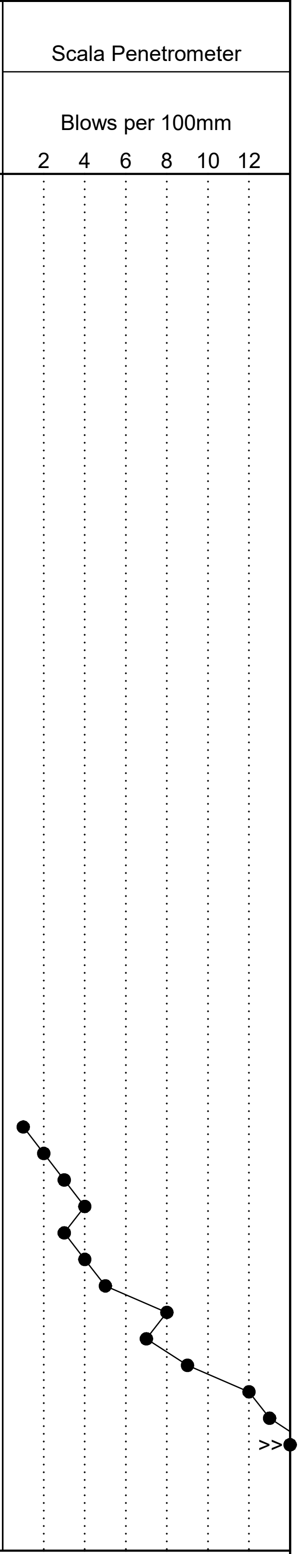
ENGEO

LOG OF AUGER HA03

Geotechnical Investigation
14 Sinton Road
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.005
Date : 11-12-2023
Hole Depth : 5 m
Hole Diameter : 50 mm
Shear Vane No : 3843
Logged By : RL
Reviewed By : NM
Latitude : -36.7952443
Longitude : 174.6403705

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
	TS	OL	[TOPSOIL]			M	N/A	
0.5	EAST COAST BAYS FORMATION	CH	Silty CLAY with trace sand; brownish orange with occassional light grey streaks. High plasticity; sand is fine.	10			VSt	152/57 162/84
1.0 1.5 2.0		CH	CLAY with some silt and trace sand; light grey with orange mottles. High plasticity; sand is fine.	9	∇		VSt - H	200+ 194/101 179/84 148/84 150/83
		CH	Silty CLAY with some sand; dark brownish grey. High plasticity; sand is fine to coarse.				VSt	115/51
2.5 3.0		OH	Sandy SILT with minor clay; light grey with thin orange streaks. Low plasticity; sand is fine to medium.	8		W	St - VSt	98/34 115/46
			3.1 m - Becomes saturated.			S		118/51
3.5		ML	Sandy SILT with trace clay; dark grey with occasional black carbonaceous inclusions. Poorly graded; sand is fine to medium.	7			F*	
4.0			3.8 m - Becomes medium dense.				St*	
4.5			4.3 m - Becomes dense.	6			VSt - H*	
5.0			End of Hole Depth: 5 m Termination Condition: met target depth					



Hand Auger met target depth at 5 m depth.
Scala Penetrometer met practical refusal at 4.8 m depth.
Dip test showed standing water at 1.1 m depth during drilling.
Elevations and coordinates estimated from Auckland Council GeoMaps.

N/A = Not Applicable; TS = Topsoil
*Consistency determined from tactile assessment (NZGS, 2005).

LOG OF AUGER HA04

Geotechnical Investigation
14 Sinton Road
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.005
Date : 12-12-2023
Hole Depth : 5 m
Hole Diameter : 50 mm
Shear Vane No : 2524
Logged By : JM
Reviewed By : NM
Latitude : -36.7958642
Longitude : 174.6405977

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm 2 4 6 8 10 12
	TS	OL	[TOPSOIL]			M	N/A		
0.5 1.0	PUKETOKA FORMATION	ML	SILT with some clay and trace fine sand; orange brown with occasional black and orange mottles. Low plasticity.	12			St - VSt	98/35 105/39 123/45	
1.5 2.0 2.5 3.0 3.5		CH	Silty CLAY ; light grey with orange streaks. High plasticity. 1.2 m - Becomes wet. 1.4 m - Becomes saturated. 1.8 m - Encountered trace fine sand. 2.9 m - Sand becomes fine to coarse.	11 10	∇	W S	VSt	188/89 181/112 168/91 173/101 181/112 156/101 123/56 165/114 159/95	
4.0 4.5		ML	Sandy SILT with some clay; dark blackish grey with greyish brown streaks. Low plasticity; sand is fine to medium.	9			VSt	154/91 151/75 126/77	
5.0		CH	Silty CLAY with minor fine to medium sand; dark grey. Low plasticity.	8			VSt	156/109	
			End of Hole Depth: 5 m Termination Condition: met target depth						

Hand Auger met target depth at 5 m depth.
Elevations and coordinates estimated from Auckland Council GeoMaps.
Dip test showed standing water at 1.2 m depth during drilling.
N/A = Not Applicable; TS = Topsoil

ENGEO

LOG OF AUGER HA05

Geotechnical Investigation
14 Sinton Road
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.005
Date : 12-12-2023
Hole Depth : 5 m
Hole Diameter : 50 mm
Shear Vane No : 2524
Logged By : JM
Reviewed By : NM
Latitude : -36.7958474
Longitude : 174.6411531

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm 2 4 6 8 10 12
	TS	OL	[TOPSOIL]	▒			M	N/A		
	FILL	ML	[FILL] SILT with some clay and trace fine sand; orange brown with occasional black mottles. Low plasticity.	▩			M	VSt	179/34	
0.5	PUKETOKA FORMATION	CH	Silty CLAY with trace fine sand; dark grey with occasional orange streaks and black mottles. High plasticity.	▤	14		M	St	67/21	
1.0 1.5 2.0	PUKETOKA FORMATION	CH	Silty CLAY; light grey with orange streaks. High plasticity. 1.9 m - Encountered trace fine sand.	▤	13		M	VSt	119/63 133/77 117/70 126/70	
2.5	PUKETOKA FORMATION	ML	Sandy SILT; light grey with orange streaks. Low plasticity; sand is fine to coarse. 2.2 m - Becomes wet.	▥	12	▽	M W	St - VSt	193/87 154/61	
3.0 3.5	PUKETOKA FORMATION	CH	Silty CLAY with minor amorphous organics; dark blackish grey with occasional light brown streaks. High plasticity. 3.4 m - Becomes saturated. 3.8 m - Encountered trace fine sand.	▤	11		W S	St - VSt	95/50 77/47 126/91 154/77 147/98	
4.0 4.5 5.0	PUKETOKA FORMATION	CH	Sandy CLAY with some silt; greenish grey. High plasticity; sand is fine to medium.	▤	10		S	S - VSt	126/91 92/39 20/6	
			End of Hole Depth: 5 m Termination Condition: met target depth							

Hand auger met target depth at 5 m depth.
Elevations and coordinates estimated from Auckland Council GeoMaps.
Dip test showed standing water at 2.2 m depth during drilling.
N/A = Not Applicable; TS = Topsoil

ENGEO

LOG OF AUGER HA06

Geotechnical Investigation
14 Sinton Road
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.005
Date : 11-12-2023
Hole Depth : 5 m
Hole Diameter : 50 mm

Shear Vane No : 3843
Logged By : RL
Reviewed By : NM
Latitude : -36.7953335
Longitude : 174.6409108

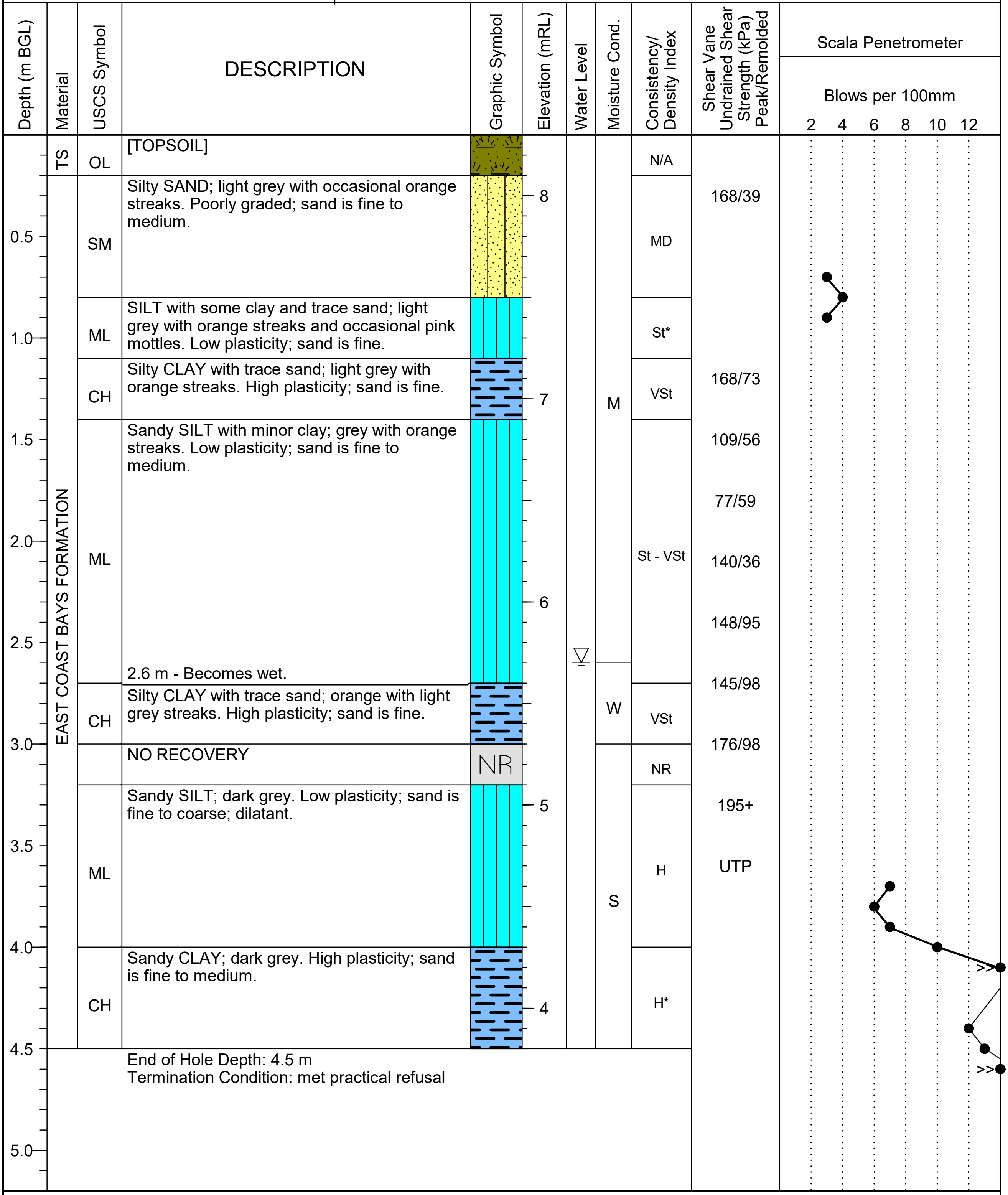
Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm 2 4 6 8 10 12
	T	OL	[TOPSOIL]					N/A		
0.5	PUKETOKA FORMATION	CH	Silty CLAY with minor sand; brownish orange with occasional light grey streaks. High plasticity; sand is fine.		13		M	VSt	142/64 138/73 135/67	
1.0 1.5		CH	CLAY with minor silt and trace sand; light grey with orange streaks. High plasticity; sand is fine. 1.2 m - Becomes dark brownish grey.		12	▽		VSt VSt - H	125/67 200+ 150/76	
2.0		CH	Silty CLAY with trace sand; light grey with orange streaks. High plasticity; sand is fine. 2.2 m - Encountered minor fine sand.					VSt	121/51 121/57	
2.5 3.0		ML	Sandy SILT with minor clay; light grey with faint orange streaks. Low plasticity; sand is fine. 2.7 m - Becomes saturated.		11		W S	St - VSt	88/37 111/49	
3.5		MH	Clayey SILT with some sand; light grey with orange streaks. Low plasticity; sand is fine.		10		W	VSt	142/83 150/51	
4.0		CH	CLAY with some silt and minor sand; dark brownish orange with occasional dark grey streaks. High plasticity; sand is fine.				M	H	200+	
4.5	ECBF	ML	Sandy SILT with trace clay; dark grey with black carbonaceoous streaks. Poorly graded; sand is fine to medium.		9		W	VSt - H	135/64 200+ 200+	
5.0			End of Hole Depth: 5 m Termination Condition: met target depth							

Hand Auger met target depth at 5 m depth.
Elevations and coordinates estimated from Auckland Council GeoMaps.
Dip test showed standing water at 1.3 m depth during drilling.
N/A = Not Applicable; T = Topsoil; ECBF = East Coast Bays Formation

LOG OF AUGER HA07

Geotechnical Investigation
14 Sinton Road
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.005
Date : 12-12-2023
Hole Depth : 4.5 m
Hole Diameter : 50 mm
Shear Vane No : 2524
Logged By : JM
Reviewed By : NM
Latitude : -36.7953488
Longitude : 174.6398361



Material	USCS Symbol	DESCRIPTION	Moisture Cond.	Consistency/ Density Index
TS	OL	[TOPSOIL]	M	N/A
EAST COAST BAYS FORMATION	SM	Silty SAND; light grey with occasional orange streaks. Poorly graded; sand is fine to medium.		MD
	ML	SILT with some clay and trace sand; light grey with orange streaks and occasional pink mottles. Low plasticity; sand is fine.		St*
	CH	Silty CLAY with trace sand; light grey with orange streaks. High plasticity; sand is fine.		VSt
	ML	Sandy SILT with minor clay; grey with orange streaks. Low plasticity; sand is fine to medium. 2.6 m - Becomes wet.		St - VSt
	CH	Silty CLAY with trace sand; orange with light grey streaks. High plasticity; sand is fine.	W	VSt
		NO RECOVERY	S	NR
	ML	Sandy SILT; dark grey. Low plasticity; sand is fine to coarse; dilatant.		H
	CH	Sandy CLAY; dark grey. High plasticity; sand is fine to medium.		H*

End of Hole Depth: 4.5 m
Termination Condition: met practical refusal

Shear Vane Undrained Shear Strength (kPa) Peak/Remoded

168/39
168/73
109/56
77/59
140/36
148/95
145/98
176/98
195+
UTP

Hand Auger met practical refusal at 4.5 m depth on hard material.
Scala Penetrometer met practical refusal at 4.6 m.
Dip test showed standing water at 2.6 m depth during drilling.
Elevations and coordinates estimated from Auckland Council GeoMaps.

N/A = Not Applicable; TS = Topsoil; NR = Not Recorded
* Inferred consistency from tactile assessment (NZGS, 2005).

ENGEO

LOG OF AUGER HA08

Geotechnical Investigation
14 Sinton Road
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.005
Date : 12-12-2023
Hole Depth : 3 m
Hole Diameter : 50 mm
Shear Vane No : 2524
Logged By : JM
Reviewed By : NM
Latitude : -36.7955543
Longitude : 174.6403999

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm 2 4 6 8 10 12
	TS	OL	[TOPSOIL]				M	N/A	140/67	
0.5 1.0 1.5	PUKETOKA FORMATION	CH	Silty CLAY; light grey with orange streaks. High plasticity.		11 10			VSt	184/109 184/114 168/101 126/77 147/70	
2.0		CH	Silty CLAY with minor sand; dark grey with occasional orange and black streaks. High plasticity; sand is fine. 2.2 m - Encountered some fine to coarse sand.		9	▽		VSt - H	195+ 142/42	
2.5 3.0		CH	Sandy CLAY; grey with orange and black streaks. High plasticity; sand is fine to medium. 2.9 m - Becomes orange with grey streaks.				W	VSt	154/84 145/81	

End of Hole Depth: 3 m
Termination Condition: met target depth

3.5
4.0
4.5
5.0

Hand Auger met target depth at 3 m depth.
Elevations and coordinates estimated from Auckland Council GeoMaps.
Dip test showed standing water at 2.4 m depth during drilling.
N/A = Not Applicable; TS = Topsoil

LOG OF AUGER HA09

Geotechnical Investigation
14 Sinton Road
Hobsonville, Auckland

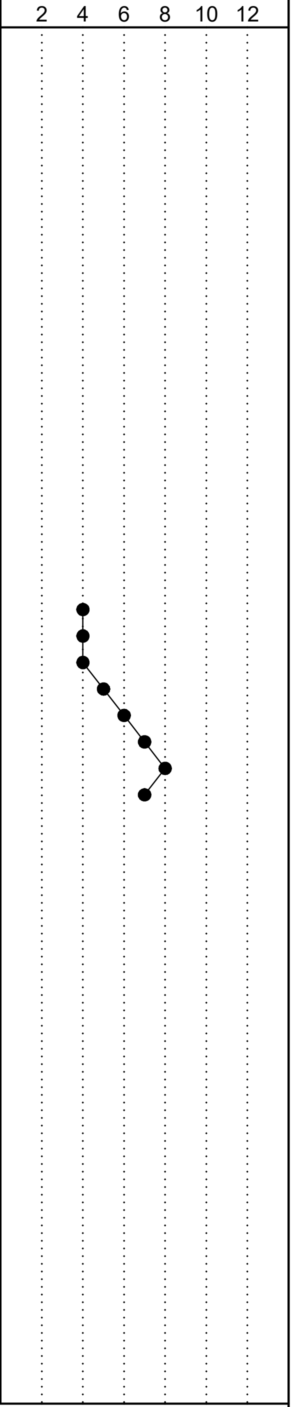
Client	: Cabra Developments Ltd	**Shear Vane No**	: 3843
Client Ref.	: 23849.000.005	**Logged By**	: RL
Date	: 11-12-2023	**Reviewed By**	: NM
Hole Depth	: 3 m	**Latitude**	: -36.7950725
Hole Diameter	: 50 mm	**Longitude**	: 174.6406398

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
	TS	OL	[TOPSOIL]				M	N/A	
0.5	FILL	ML	[FILL] Clayey SILT with minor sand; intermixed light brown, orange and light grey. Low plasticity; sand is fine.	▨				VSt	140/32
1.0 1.5	EAST COAST BAYS FORMATION	CH	Silty CLAY with trace sand; brownish grey with orange streaks. High plasticity; sand is fine. 1.0 m - Becomes light grey with occasional orange mottles.		11 10	∇		VSt	148/57 169/81 152/73 186/71
2.0		ML	Clayey SILT with some sand; light grey with orange streaks. Low plasticity; sand is fine.					H	189/67 200+
2.5 3.0		SW	Silty SAND with trace clay; light grey with occasional orange streaks. Well graded; sand is fine to coarse. 2.7 m - Becomes dark grey with black carbonaceous inclusions.		9		W	MD	

End of Hole Depth: 3 m
Termination Condition: met target depth

Scala Penetrometer

Blows per 100mm



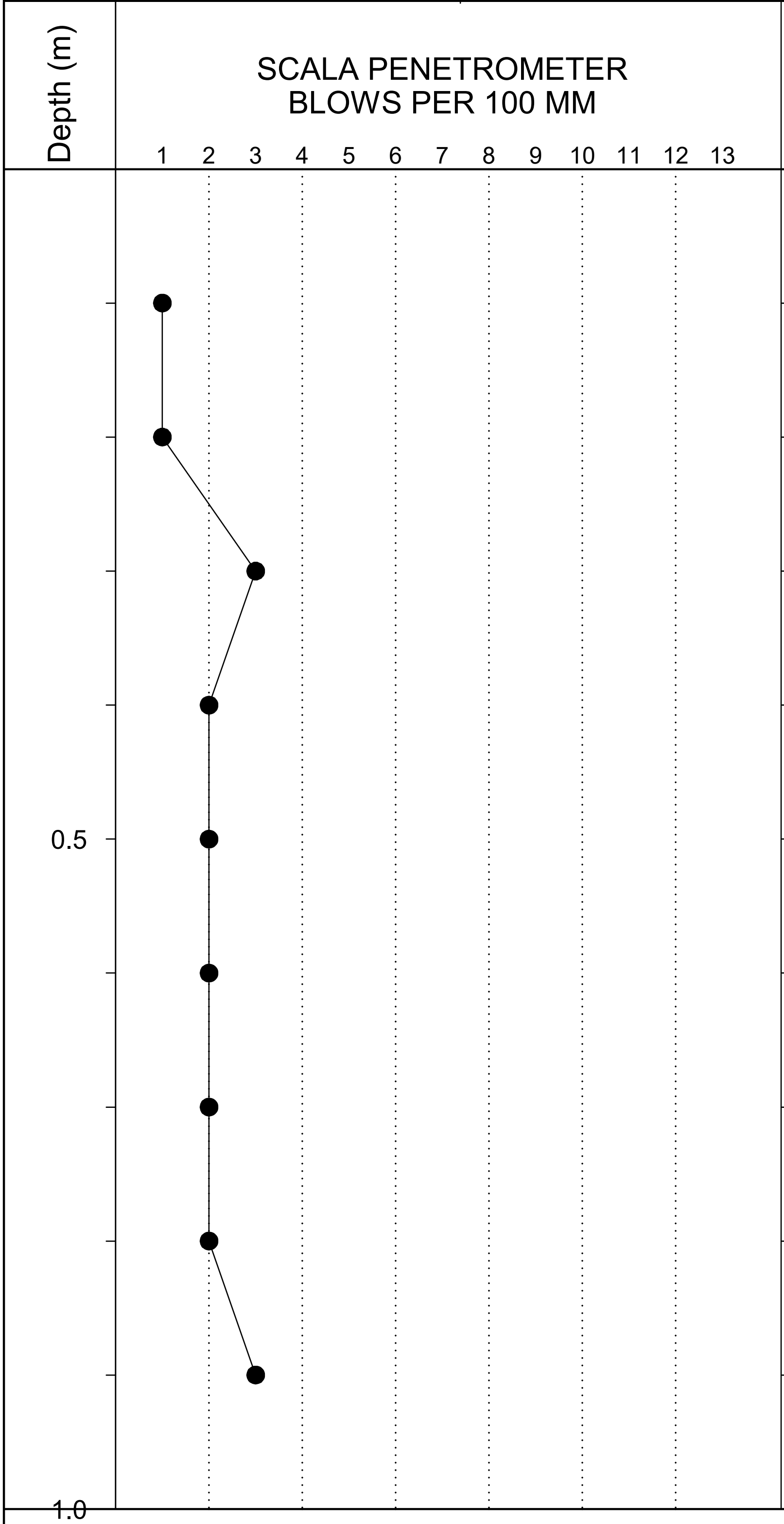
Hand Auger met target depth at 3 m depth.
Scala Penetrometer met target depth.
Dip test showed standing water at 1.1 m depth during drilling.
Elevations and coordinates estimated from Auckland Council GeoMaps.

N/A = Not Applicable; TS = Topsoil

LOG OF SCALA Scala 1

Geotechnical Investigation
14 Sinton Road
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.005
Date : 11-12-2023
Scala Depth : 1 m
Logged By : JM
Reviewed By : NM
Latitude : -36.7958746
Longitude : 174.6409308



BLOWS	NOTES/ REMARKS
1	
1	
3	
2	
2	
2	
2	
2	
3	

Scala Penetrometer met target depth.
Elevations and coordinates estimated from Auckland Council GeoMaps.
Standing groundwater was not encountered.

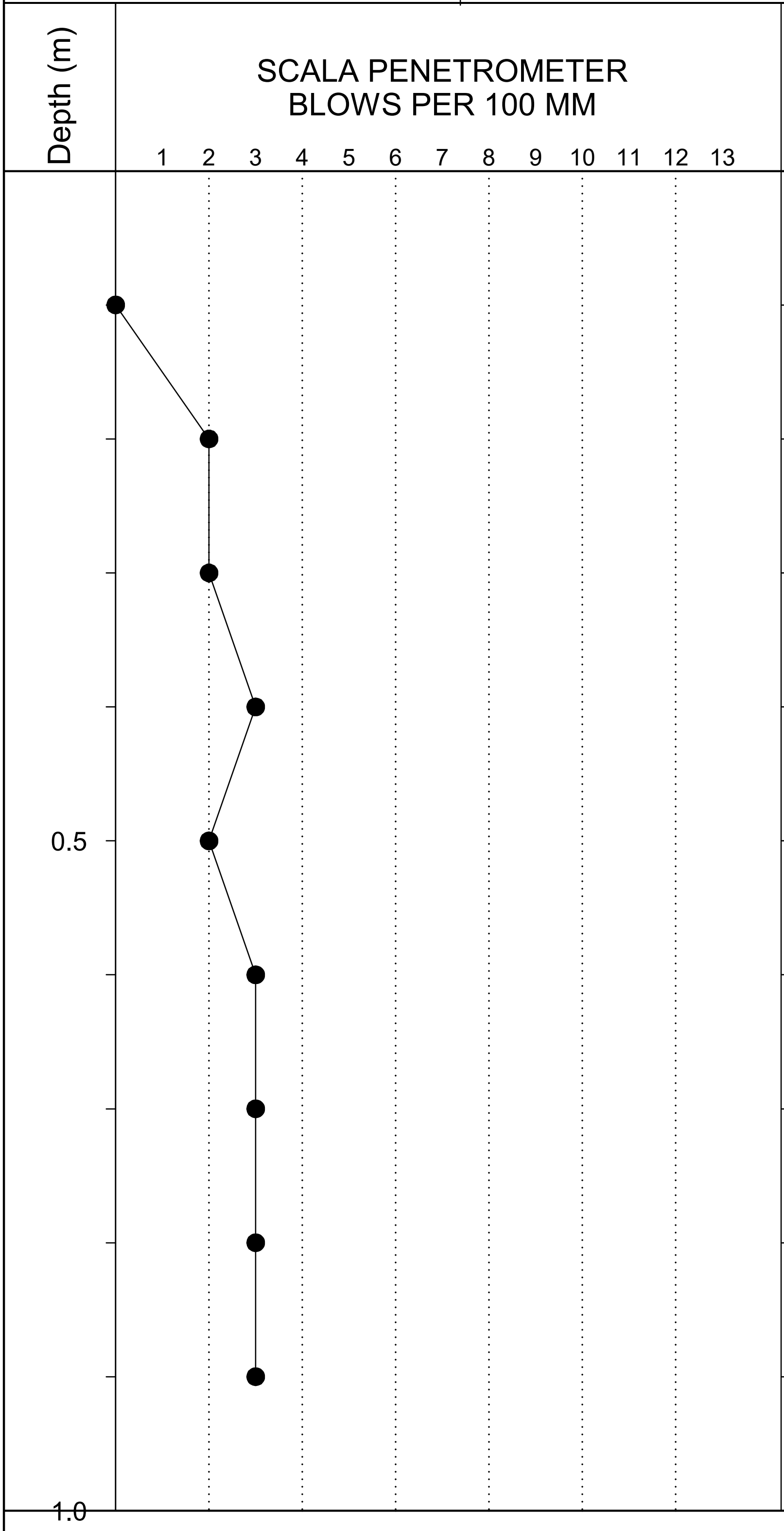
ENGEO

LOG OF SCALA Scala 2

Geotechnical Investigation
14 Sinton Road
Hobsonville, Auckland

Client : Cabra Developments Ltd
Client Ref. : 23849.000.005
Date : 11-12-2023
Scala Depth : 1 m

Logged By : JM
Reviewed By : NM
Latitude : -36.7950384
Longitude : 174.6403477



BLOWS	NOTES/ REMARKS
0	
2	
2	
3	
2	
3	
3	
3	
3	

Scala Penetrometer met target depth.
Elevations and coordinates estimated from Auckland Council GeoMaps.
Standing groundwater was not encountered.

LandTech CONSULTING

CONE PENETRATION TEST (CPT) LOG

HOLE NO.: CPT01

CLIENT: ENGEO Limited

PROJECT: CPT Testing

JOB NO.: LTA23348

SITE LOCATION: 14 Sinton Road, Hobsonville, Auckland

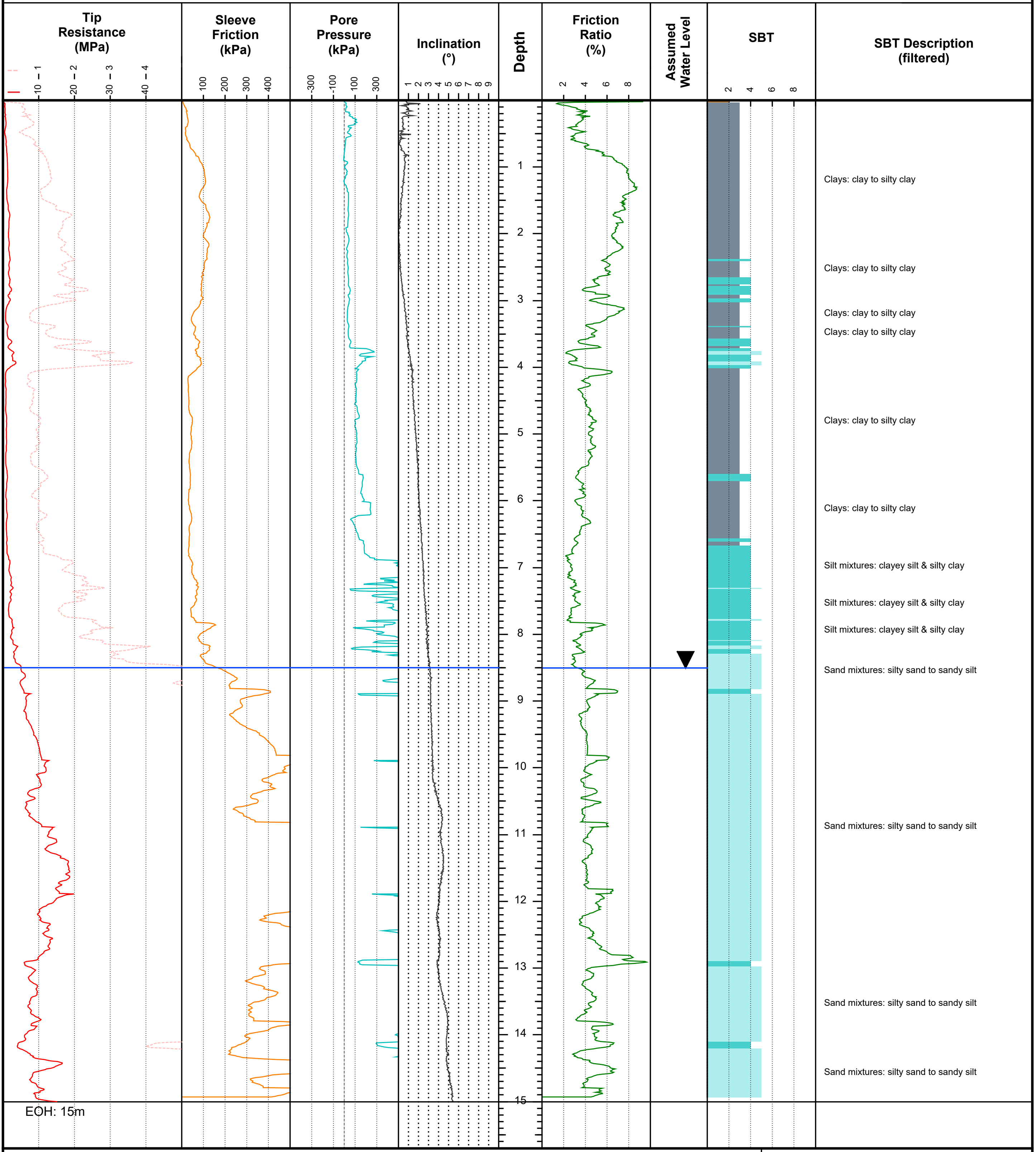
OPERATOR: CW

START DATE: 11/12/2023

CO-ORDINATES: 1746415.00mE, 5926483.00mN (NZTM2000)

ELEVATION: 15m (NZVD2016)

END DATE: 11/12/2023



REMARKS:

Groundwater measured at 8.5m (Dipmeter)
Coordinates from handheld GPS accurate to +/-4m
Pagani TG63-150 Rig, 10 cm² piezocone

NOTES:

TEST DETAILS:

Cone Number	Mks953
Cone Type	PC
Area Ratio	0.80
Filter Location	u2
Termination Reason	Target depth

CONE PENETRATION TEST (CPT) LOG

HOLE NO.: CPT02

CLIENT: ENGEO Limited

PROJECT: CPT Testing

JOB NO.: LTA23348

SITE LOCATION: 14 Sinton Road, Hobsonville, Auckland

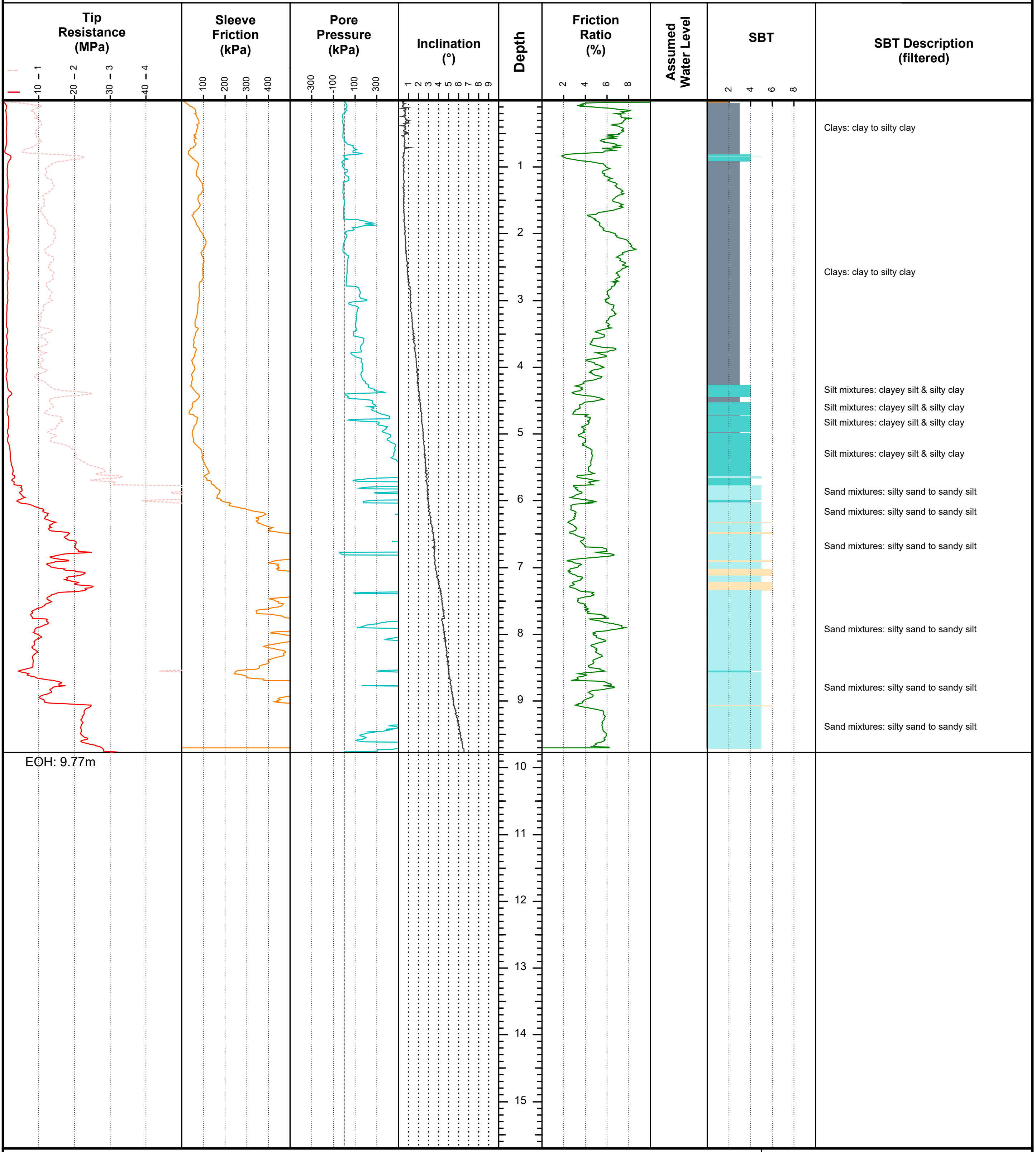
OPERATOR: CW

START DATE: 11/12/2023

CO-ORDINATES: 1746323.00mE, 5926564.00mN (NZTM2000)

ELEVATION: 9m (NZVD2016)

END DATE: 11/12/2023



REMARKS:

Groundwater not measured due to hole collapse at 0.7m
Coordinates from handheld GPS accurate to +/-4m
Pagani TG63-150 Rig, 10 cm² piezocone

NOTES:

TEST DETAILS:

Cone Number	Mks953
Cone Type	PC
Area Ratio	0.80
Filter Location	u2
Termination Reason	Fs refusal

Generated with CORE-GS by Geroc - LandTech CPT - V1.03.01 - 11/12/2023 4:19:39 pm

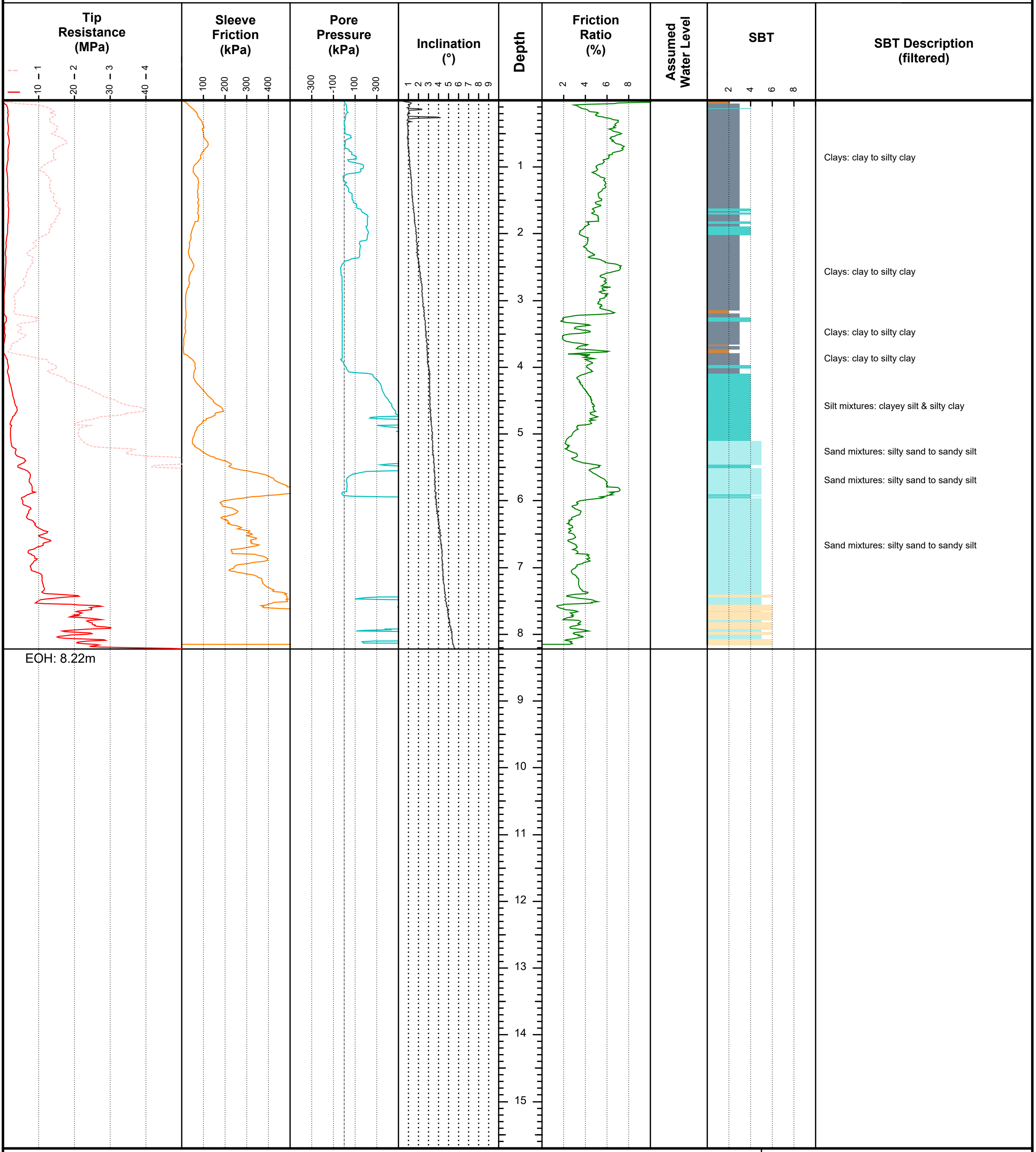
LandTech CONSULTING

CONE PENETRATION TEST (CPT) LOG

HOLE NO.: CPT03

CLIENT: ENGEO Limited
PROJECT: CPT Testing
JOB NO.: LTA23348

SITE LOCATION: 14 Sinton Road, Hobsonville, Auckland
OPERATOR: CW
START DATE: 11/12/2023
CO-ORDINATES: 1746357.00mE, 5926623.00mN (NZTM2000)
ELEVATION: 9.5m (NZVD2016)
END DATE: 11/12/2023



REMARKS:

Groundwater not measured due to hole collapse at 1.7m
Coordinates from handheld GPS accurate to +/-4m
Pagani TG63-150 Rig, 10 cm² piezocone

NOTES:

TEST DETAILS:

Cone Number	Mks953
Cone Type	PC
Area Ratio	0.80
Filter Location	u2
Termination Reason	Qc refusal

CONE PENETRATION TEST (CPT) LOG

HOLE NO.: CPT04

CLIENT: ENGEO Limited

PROJECT: CPT Testing

JOB NO.: LTA23348

SITE LOCATION: 14 Sinton Road, Hobsonville, Auckland

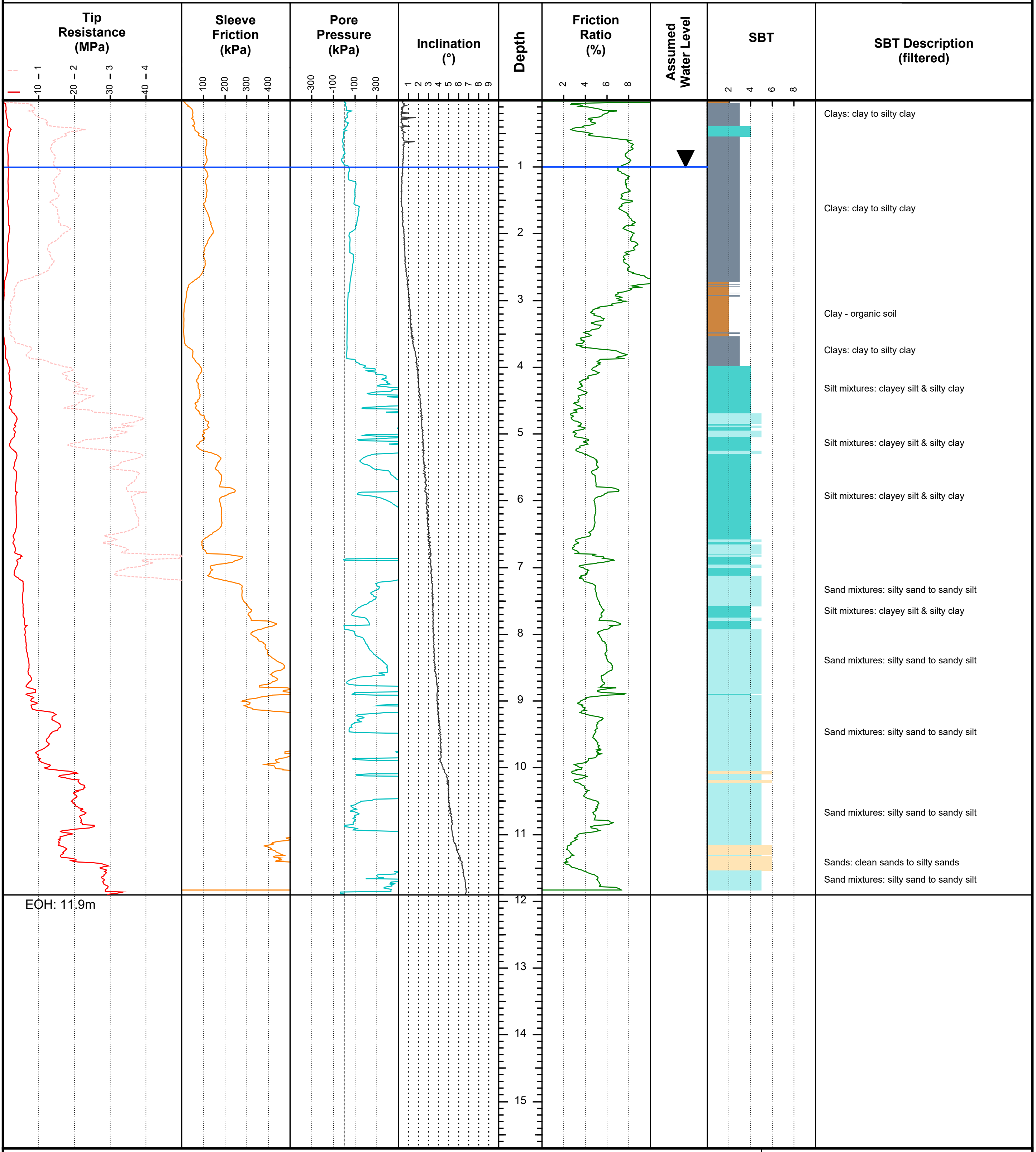
OPERATOR: CW

START DATE: 11/12/2023

CO-ORDINATES: 1746377.00mE, 5926554.00mN (NZTM2000)

ELEVATION: 13m (NZVD2016)

END DATE: 11/12/2023



REMARKS:

Groundwater measured at 1.0m (Dipmeter)
Coordinates from handheld GPS accurate to +/-4m
Pagani TG63-150 Rig, 10 cm² piezocone

NOTES:

TEST DETAILS:

Cone Number	Mks953
Cone Type	PC
Area Ratio	0.80
Filter Location	u2
Termination Reason	Fs refusal

Generated with CORE-GS by Geroc - LandTech CPT - V1.03.01 - 11/12/2023 4:19:39 pm

CONE PENETRATION TEST (CPT) LOG

HOLE NO.: CPT05

CLIENT: ENGEO Limited

PROJECT: CPT Testing

JOB NO.: LTA23348

SITE LOCATION: 14 Sinton Road, Hobsonville, Auckland

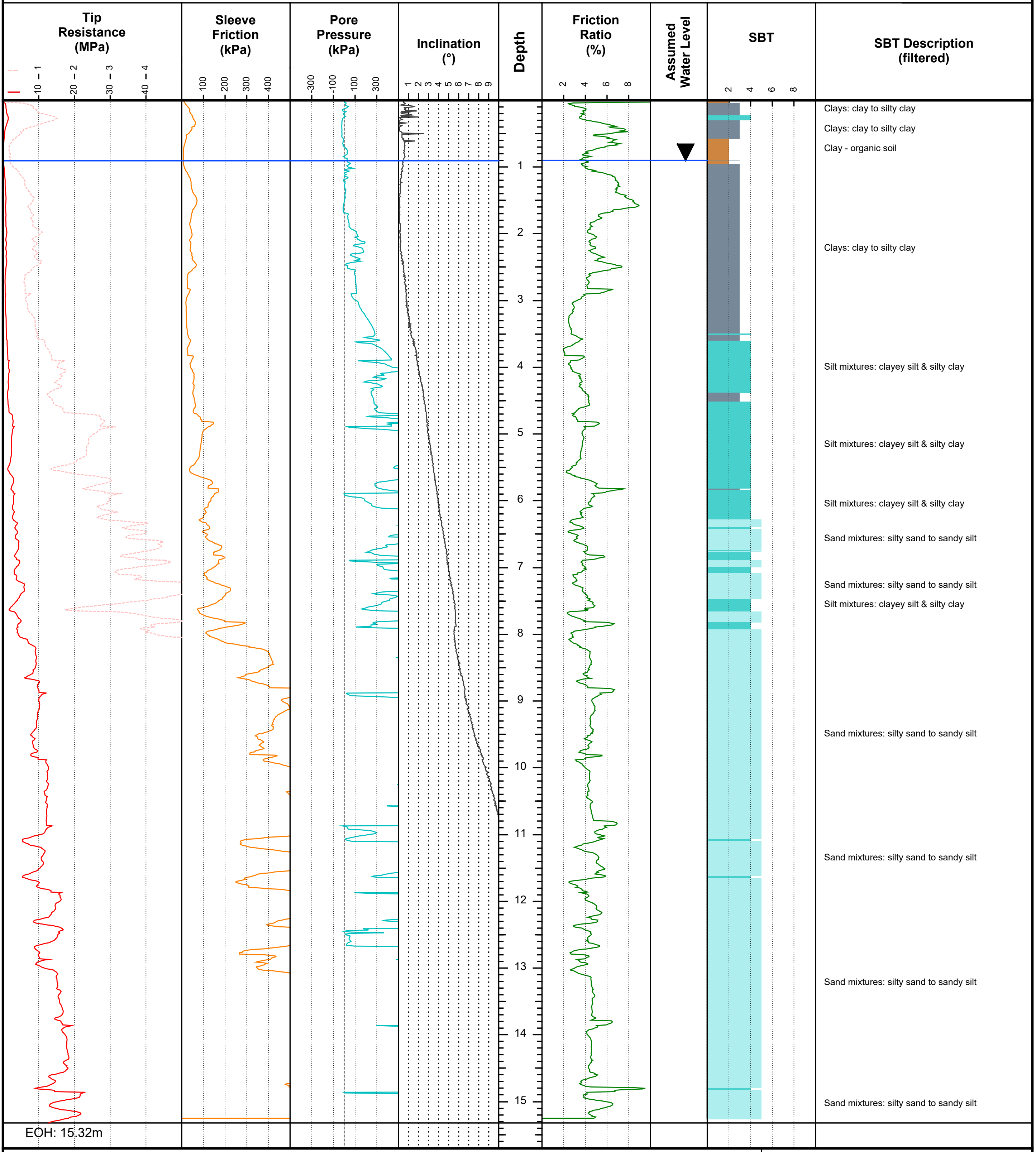
OPERATOR: CW

START DATE: 11/12/2023

CO-ORDINATES: 1746436.00mE, 5926553.00mN (NZTM2000)

ELEVATION: 14m (NZVD2016)

END DATE: 11/12/2023



REMARKS:

Groundwater measured at 0.9m (Dipmeter)
Coordinates from handheld GPS accurate to +/-4m
Pagani TG63-150 Rig, 10 cm² piezocone

NOTES:

TEST DETAILS:

Cone Number	Mks953
Cone Type	PC
Area Ratio	0.80
Filter Location	u2
Termination Reason	Target depth

CONE PENETRATION TEST (CPT) LOG

HOLE NO.: CPT06

CLIENT: ENGEO Limited

PROJECT: CPT Testing

JOB NO.: LTA23348

SITE LOCATION: 14 Sinton Road, Hobsonville, Auckland

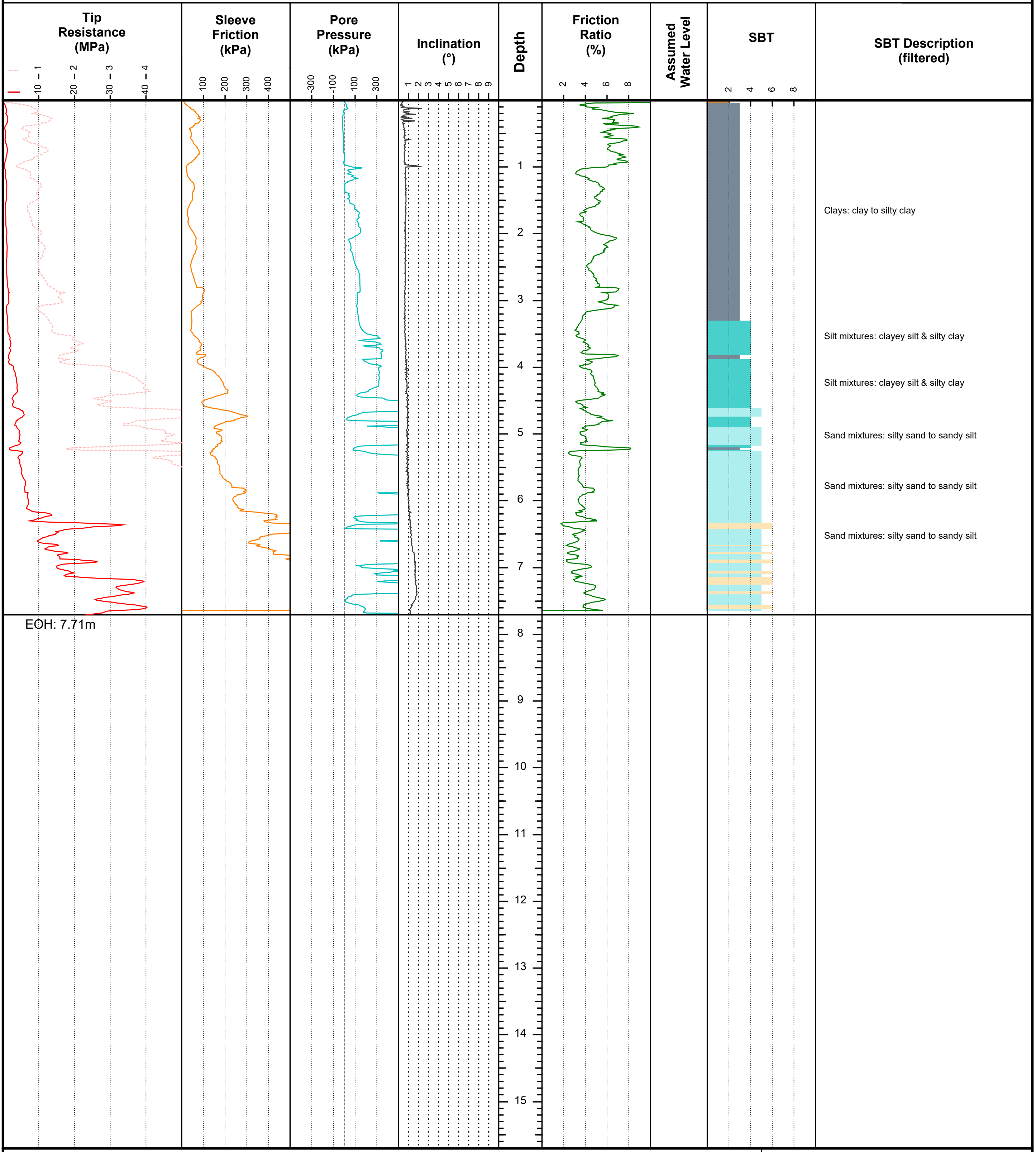
OPERATOR: CW

START DATE: 11/12/2023

CO-ORDINATES: 1746323.00mE, 5926615.00mN (NZTM2000)

ELEVATION: 8m (NZVD2016)

END DATE: 11/12/2023



REMARKS:

Groundwater not measured due to hole collapse at 0.9m
Coordinates from handheld GPS accurate to +/-4m
Pagani TG63-150 Rig, 10 cm² piezocone

NOTES:

TEST DETAILS:

Cone Number	Mks953
Cone Type	PC
Area Ratio	0.80
Filter Location	u2
Termination Reason	Fs refusal

APPENDIX 4:

Babbage Laboratory Results

Babbage Geotechnical Laboratory

Babbage Geotechnical Laboratory
Level 4
68 Beach Road
Auckland 1010
Telephone
E-mail

P O Box 2027
New Zealand
64-9-367 4954
wec@babbage.co.nz

Please reply to: W.E. Campton

ENGEO LTD.
PO Box 33-1527
Takapuna
Auckland 0740

Attention: **JERRY CHEN**

Job Number: 66273#L
BGL Registration Number: 3064
Checked by: WEC

19th December 2023

ATTERBERG LIMITS & LINEAR SHRINKAGE TESTING

Dear Jerry,

Re: 14 SINTON ROAD, HOBSONVILLE

Your Reference: 23849
Report Number: 66273#L/AL 14 Sinton Rd

The following report presents the results of Atterberg Limits & Linear Shrinkage testing at BGL of soil samples delivered to this laboratory on the 12th of December 2023. Test results are summarised below, with page 3 showing where the samples plot on the Unified Soil Classification System (Casagrande) Chart.

Test standards used were:

Water Content:	NZS4402:1986:Test 2.1
Liquid Limit:	NZS4402:1986:Test 2.2
Plastic Limit:	NZS4402:1986:Test 2.3
Plasticity Index:	NZS4402:1986:Test 2.4
Linear Shrinkage:	NZS4402:1986:Test 2.6

Borehole Number	Sample Number	Depth (m)	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage (%)*
HA09	**Sample 1**	**0.25 – 0.80**	**27.8**	**53**	**23**	**30**	**14**
HA08	**Sample 2**	**0.80 – 1.70**	**31.4**	**70**	**23**	**47**	**18**

The amount of shrinkage of the sample as a percentage of the original sample length.

The whole soils were used for the water content tests (the soils were in a natural state), and for the liquid limit, plastic limit & linear shrinkage tests. The soils were wet up and dried where required for the liquid limit, plastic limit & linear shrinkage tests.

As per the reporting requirements of NZS4402: 1986: Test 2.1: water content is reported to two significant figures for values below 10%, and to three significant figures for values of 10% or greater. Test 2.2: liquid limit, test 2.3: plastic limit, and test 2.6: linear shrinkage are reported to the nearest whole number.

Please note that the test results relate only to the samples as-received, and relate only to the samples under test.

Thank you for the opportunity to carry out this testing. If you have any queries regarding the content of this report please contact the person authorising this report below at your convenience.

Yours faithfully,

Justin Franklin
Key Technical Person
Assistant Laboratory Manager
Babbage Geotechnical Laboratory

№ 126

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation. This report may not be reproduced except in full & with written approval from BGL.

BGL Babbage Geotechnical Laboratory	Job Number:	66273#L	Sheet 1 of 1	Page 3 of 3
	Reg. Number:	3064	Version No:	7
	Report No:	66273#L/AL 14 Sinton Rd	Version Date:	July 2022
	Project:	**14 SINTON ROAD, HOBSONVILLE**		

DETERMINATION OF THE LIQUID LIMIT, PLASTIC LIMIT & THE PLASTICITY INDEX

Test Methods: NZS4402: 1986: Test 2.2, Test 2.3 and Test 2.4

Tested By:	WEC	December 2023
Compiled By:	JF	19/12/2023
Checked By:	JF	19/12/2023

SUMMARY OF TESTING

Borehole Number	Sample Number	Depth (m)	Liquid Limit	Plastic Limit	Plasticity Index	Soil Classification Based on USCS Chart Below
HA09	**Sample 1**	**0.25 - 0.80**	53	23	**30**	CH
HA08	**Sample 2**	**0.80 - 1.70**	70	23	**47**	CH

The chart below & soil classification terminology is taken from ASTM D2487-17^{e1} "Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)", April 2020, & is based on the classification scheme developed by A. Casagrande in the 1940's (Casagrande, A., 1948: Classification and identification of soil. Transactions of the American Society of Civil Engineers, v. 113, p. 901-930). The chart below & the soil classification given in the table above are included for your information only, and are not included in the IANZ endorsement for this report.

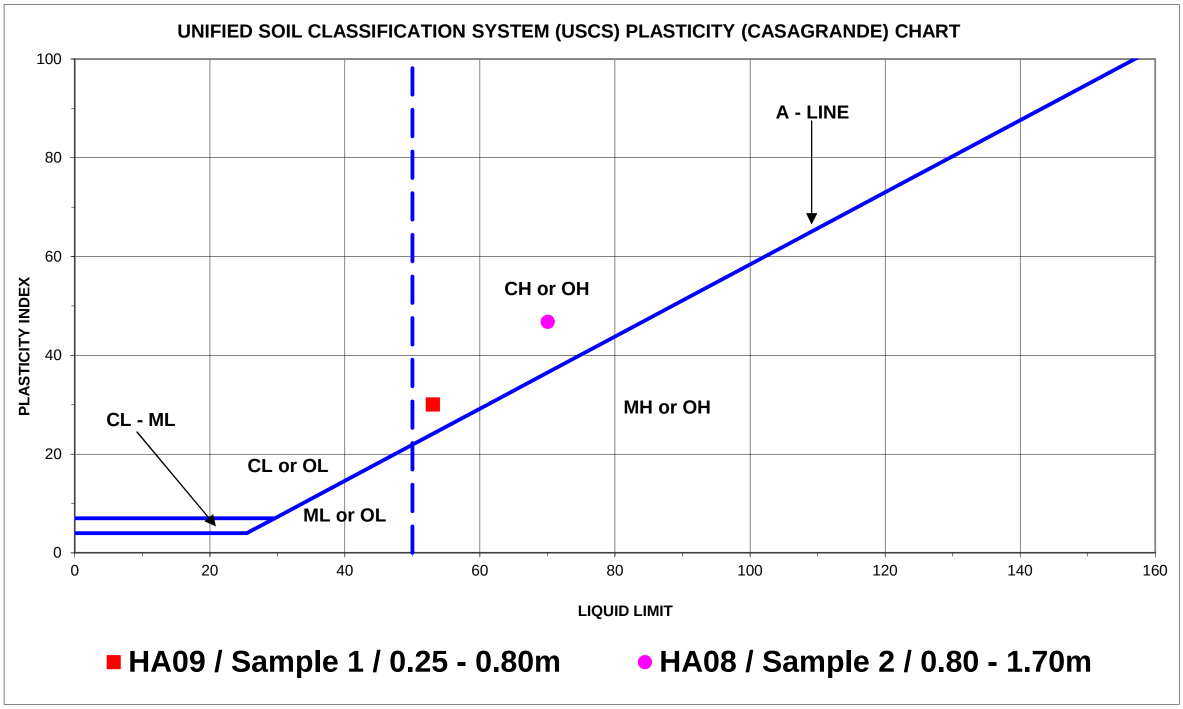


CHART LEGEND

CL = CLAY, low plasticity ('lean' clay)
OL = ORGANIC CLAY or ORGANIC SILT, low liquid limit
ML = SILT, low liquid limit
CL - ML = SILTY CLAY
CH = CLAY, high plasticity ('fat' clay)
OH = ORGANIC CLAY or ORGANIC SILT, high liquid limit
MH = SILT, high liquid limit ('elastic silt')

APPENDIX 5:

Liquefaction Analyses

GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

LIQUEFACTION ANALYSIS REPORT

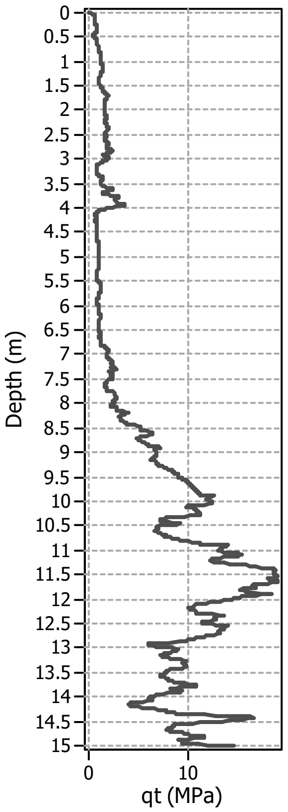
Project title : **Location :**

CPT file : CPT-01

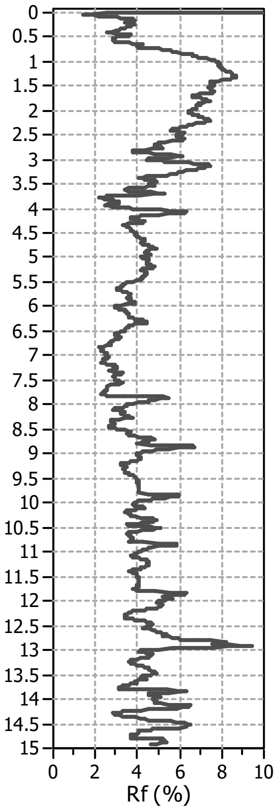
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.05	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

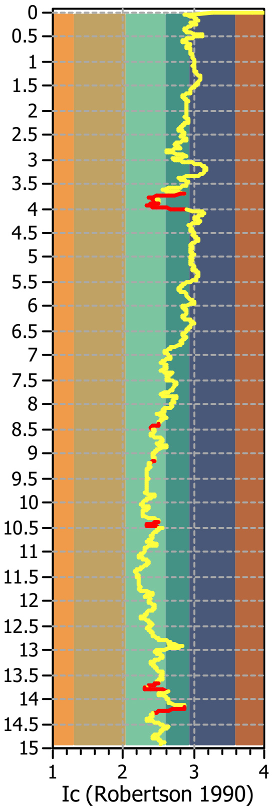
Cone resistance



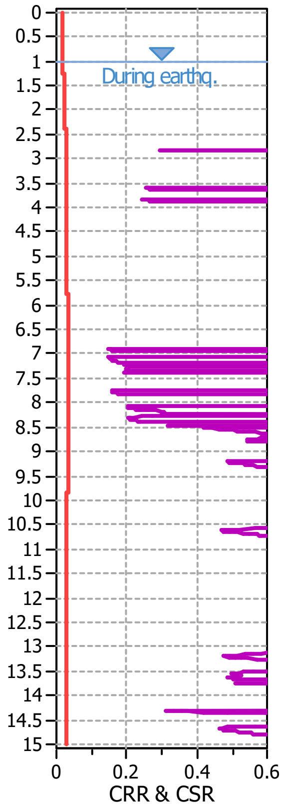
Friction Ratio



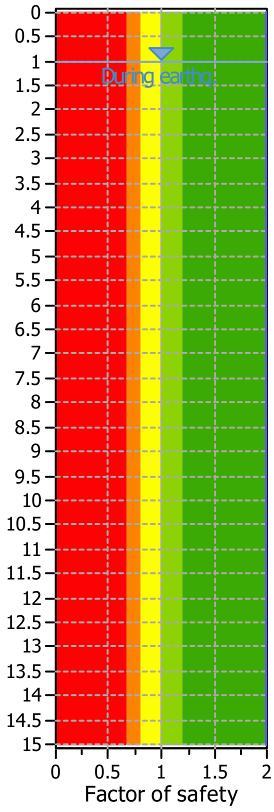
SBTn Plot



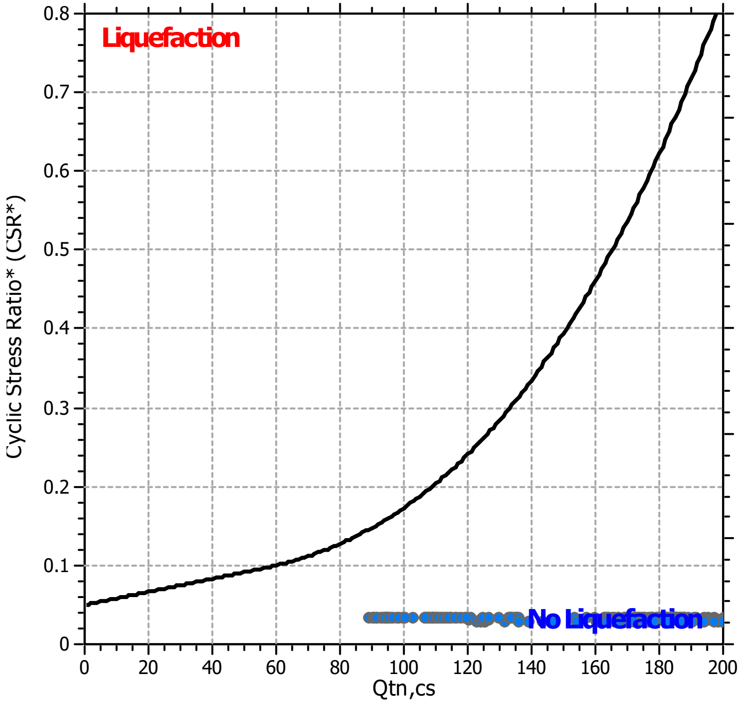
CRR plot



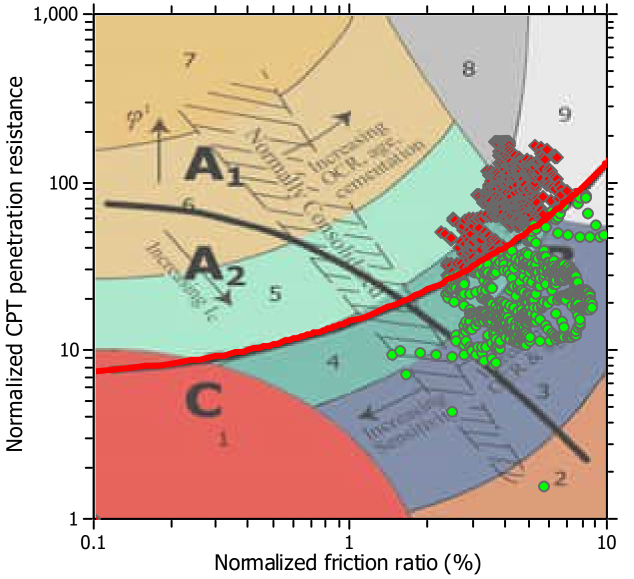
FS Plot



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

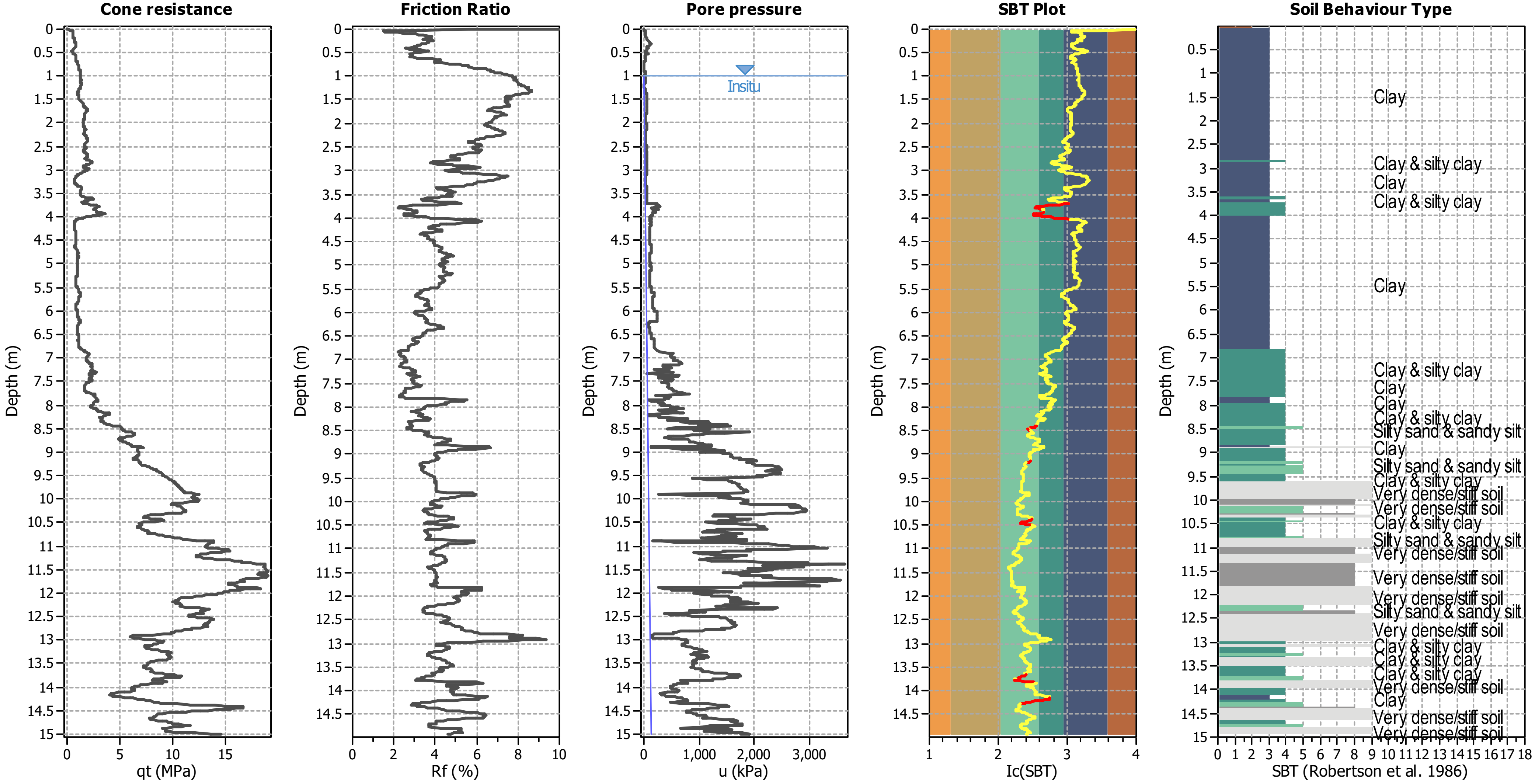


Summary of liquefaction potential



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

CPT basic interpretation plots



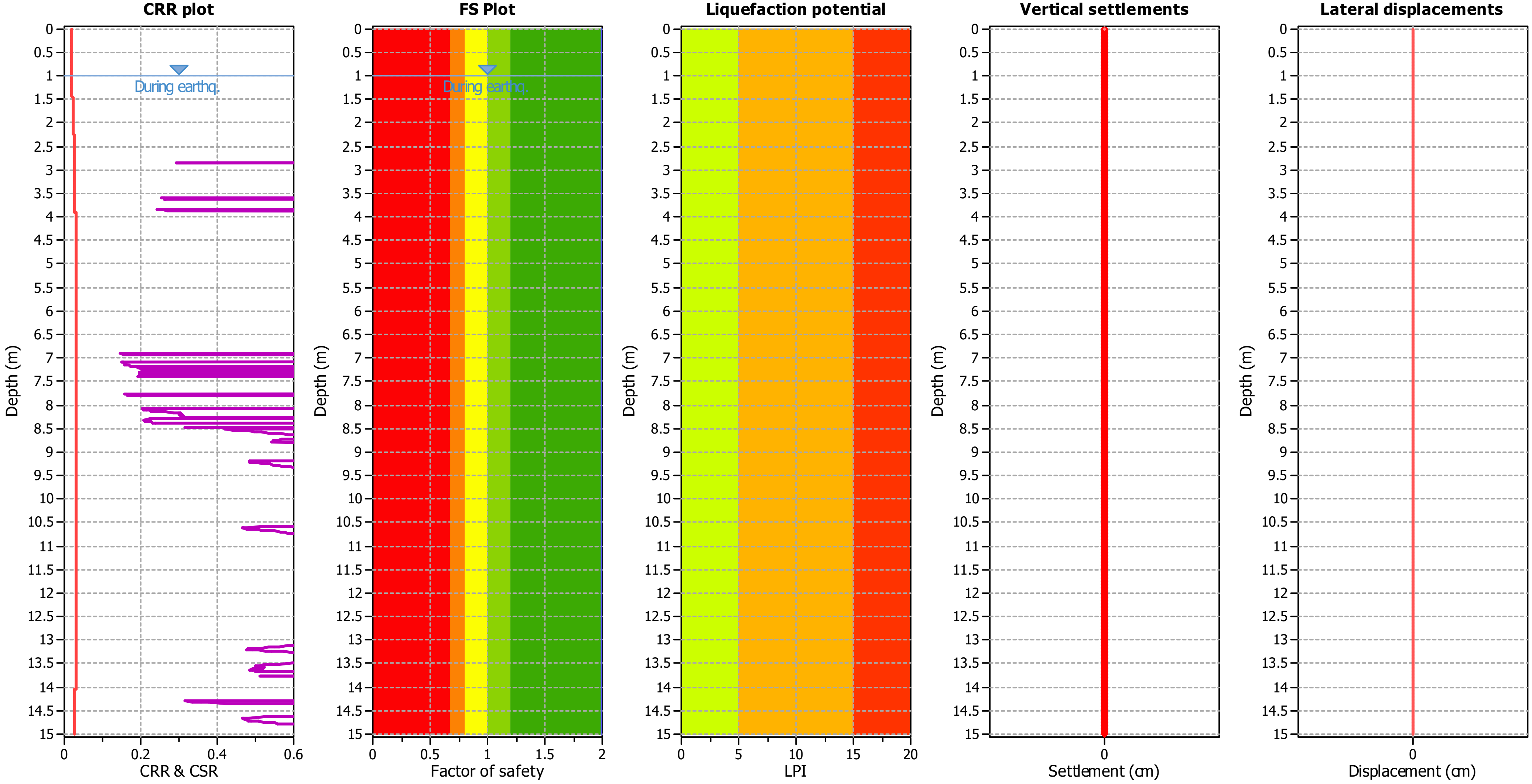
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

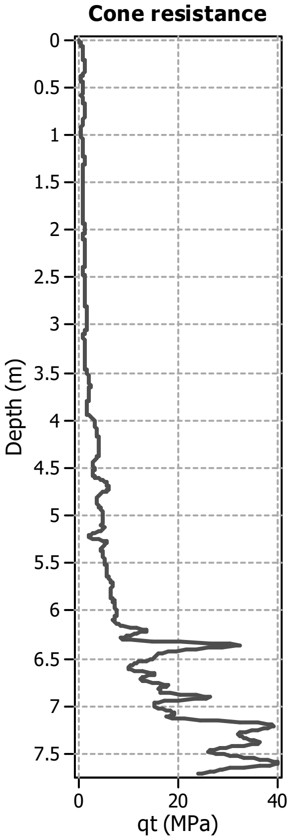
LIQUEFACTION ANALYSIS REPORT

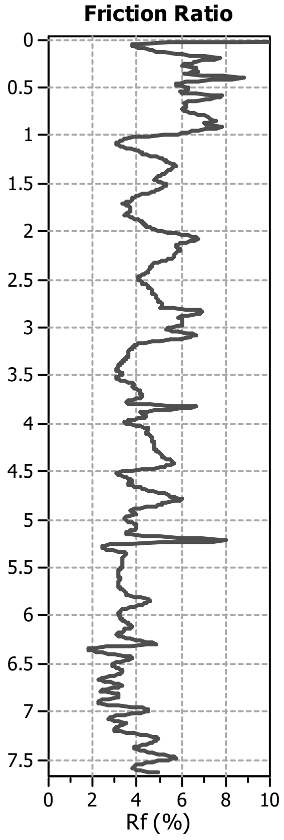
Project title : **Location :**

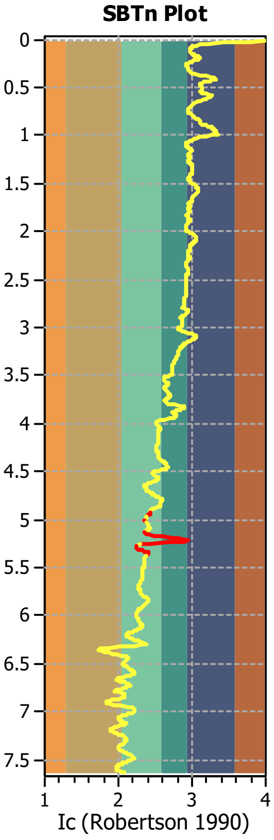
CPT file : CPT-06

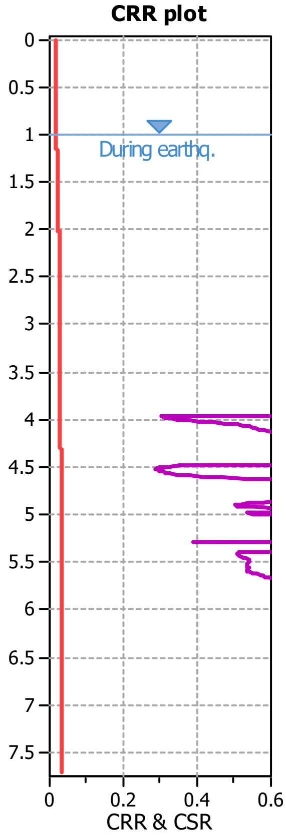
Input parameters and analysis data

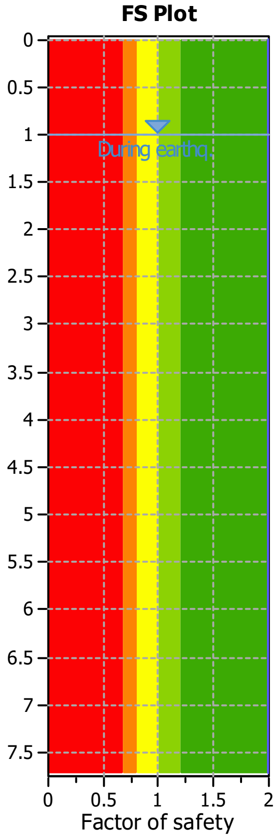
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.05	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

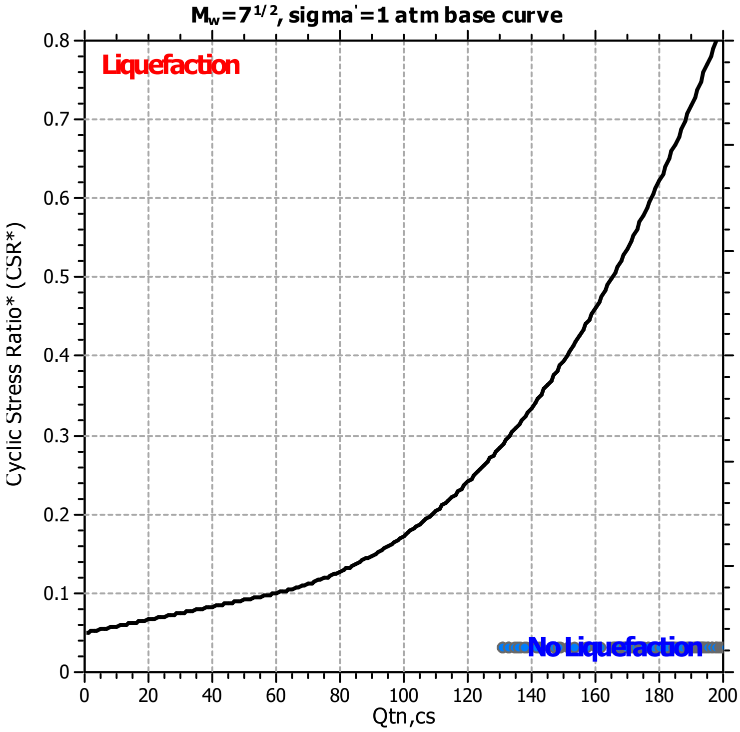

Cone resistance

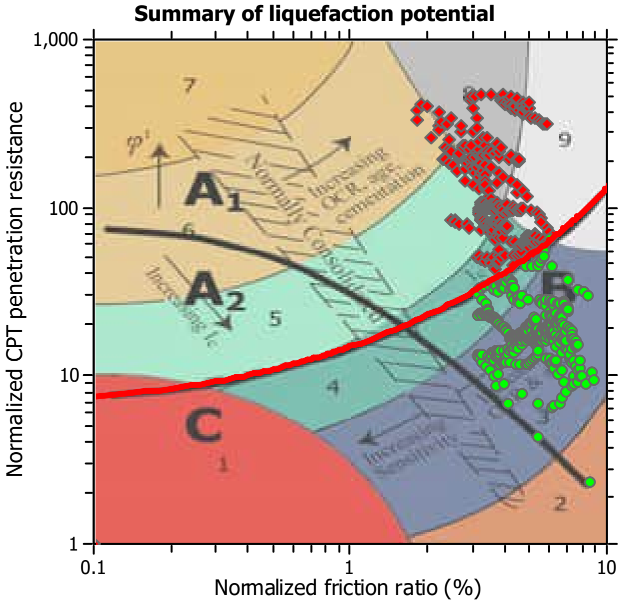

Friction Ratio


SBTn Plot


CRR plot

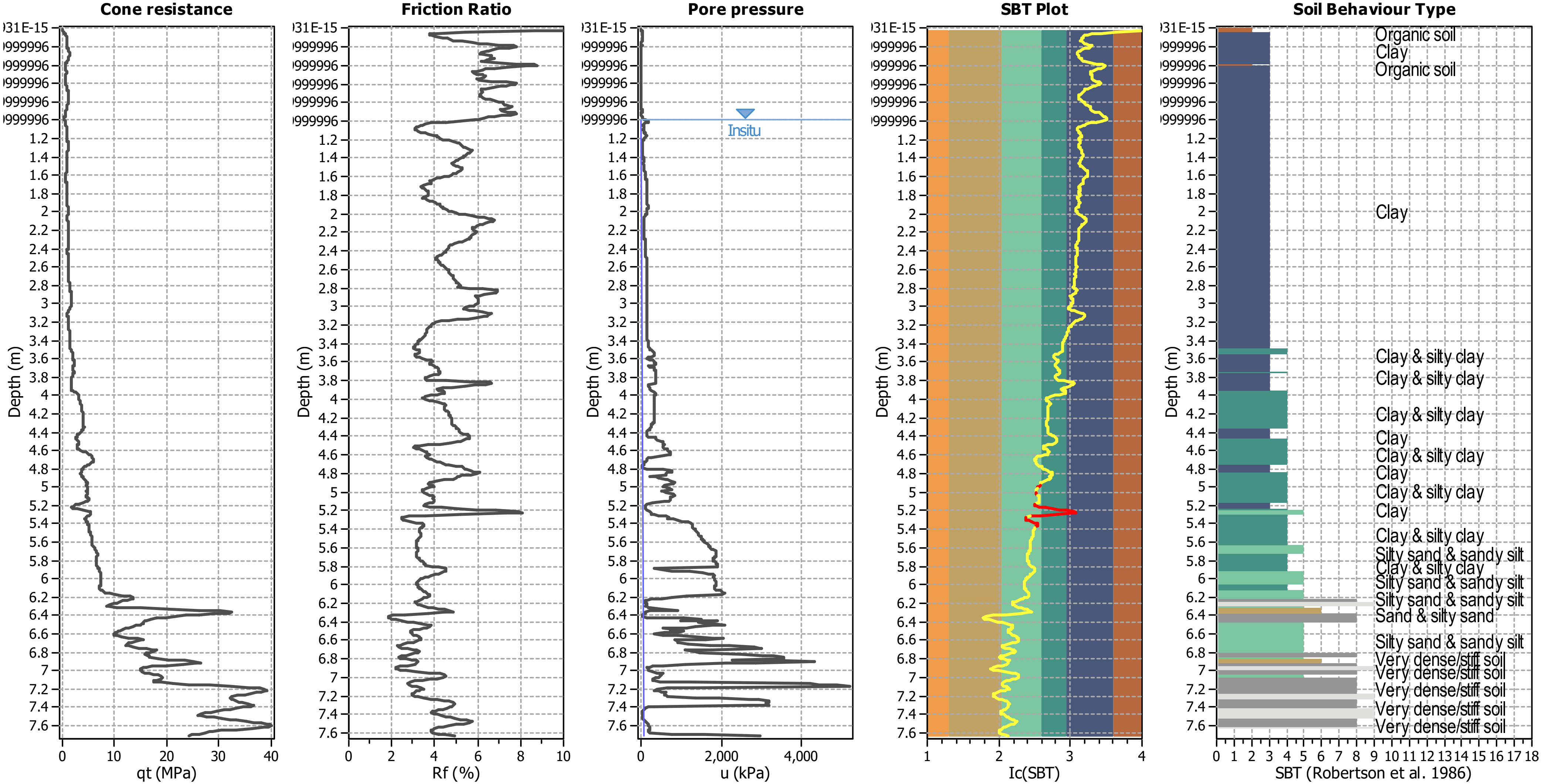

FS Plot


$M_w = 7^{1/2}$, 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

CPT basic interpretation plots



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 5.90
Peak ground acceleration: 0.05
Depth to water table (insitu): 1.00 m

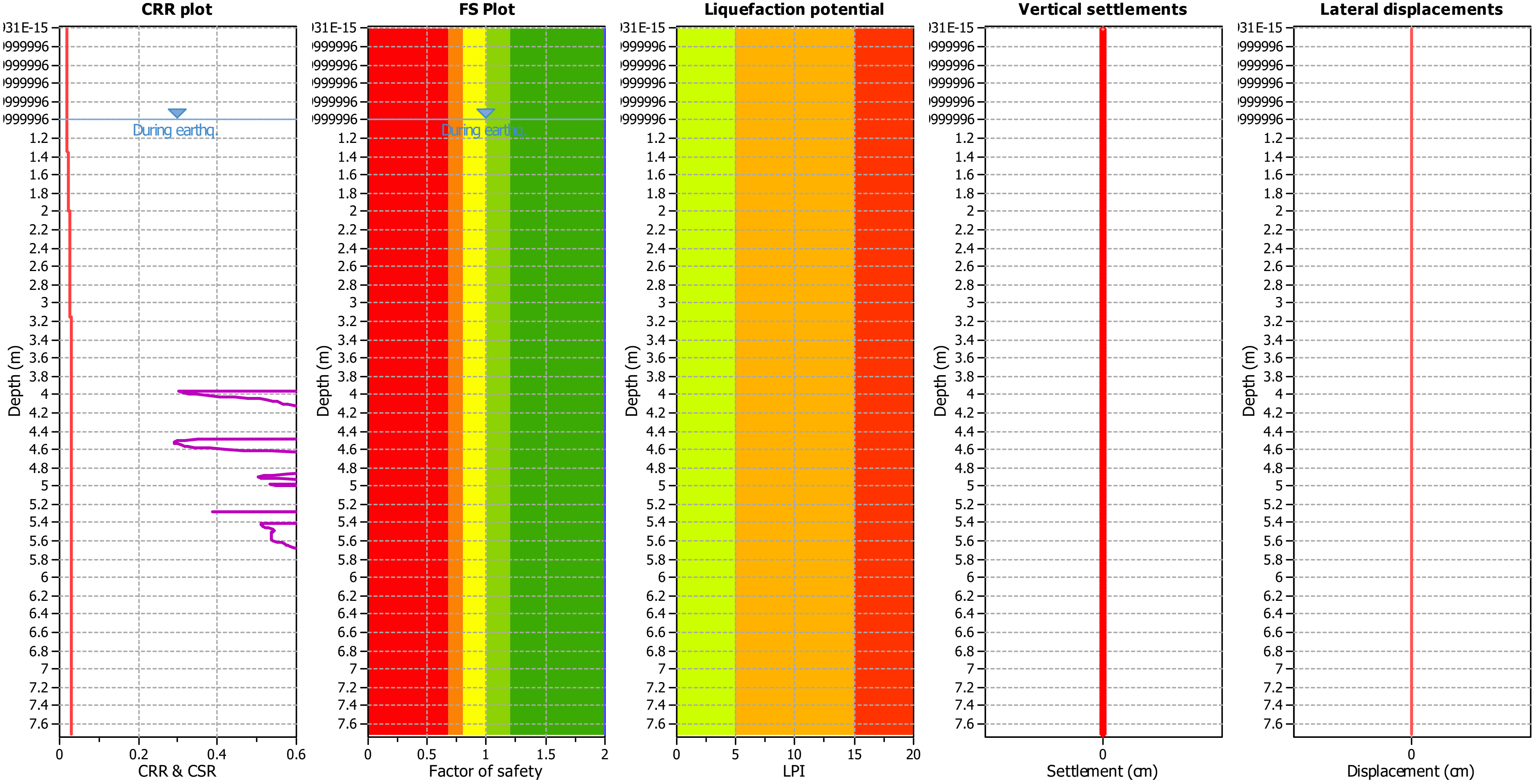
Depth to water table (erthq.): 1.00 m
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: Yes
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟩 Low risk

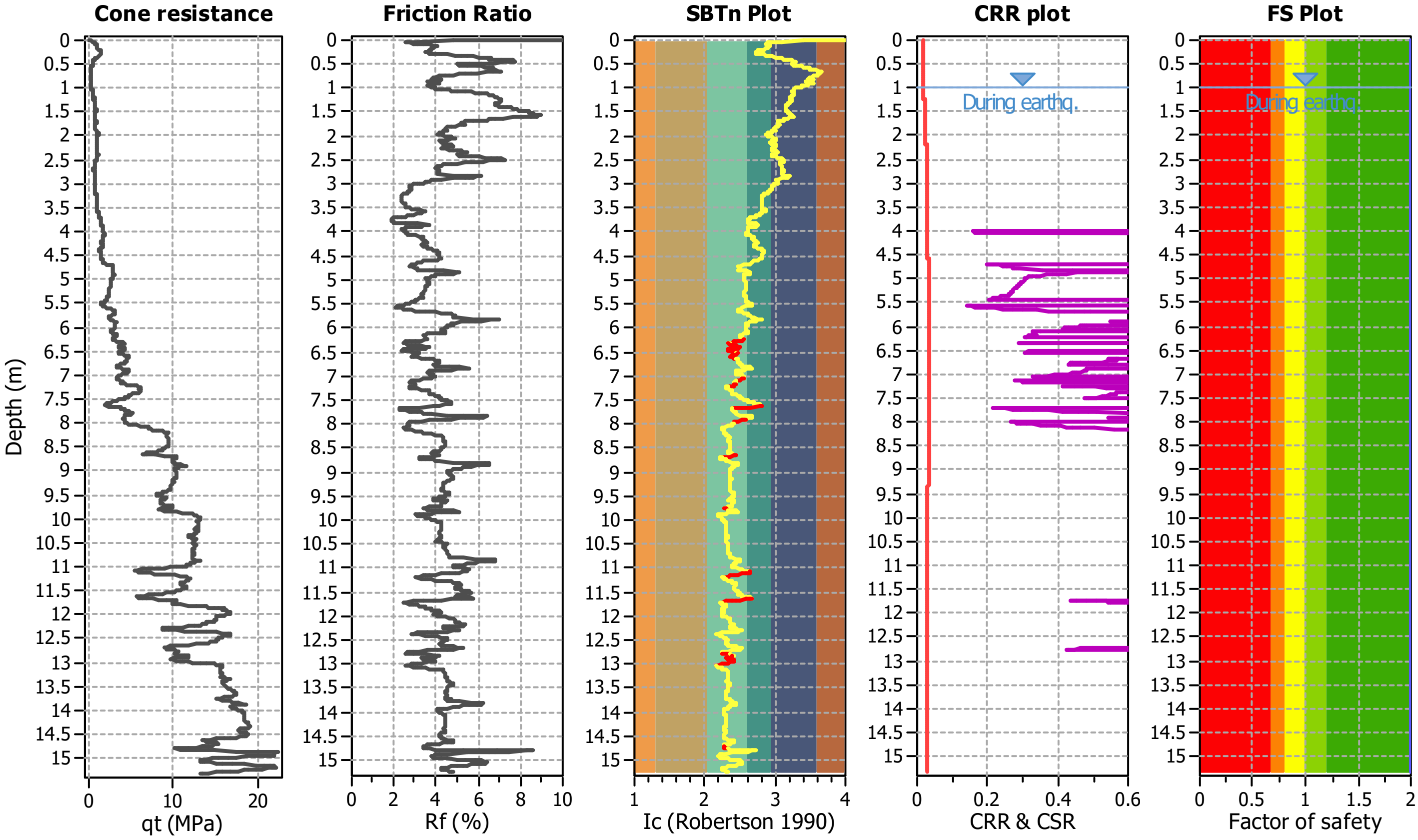
LIQUEFACTION ANALYSIS REPORT

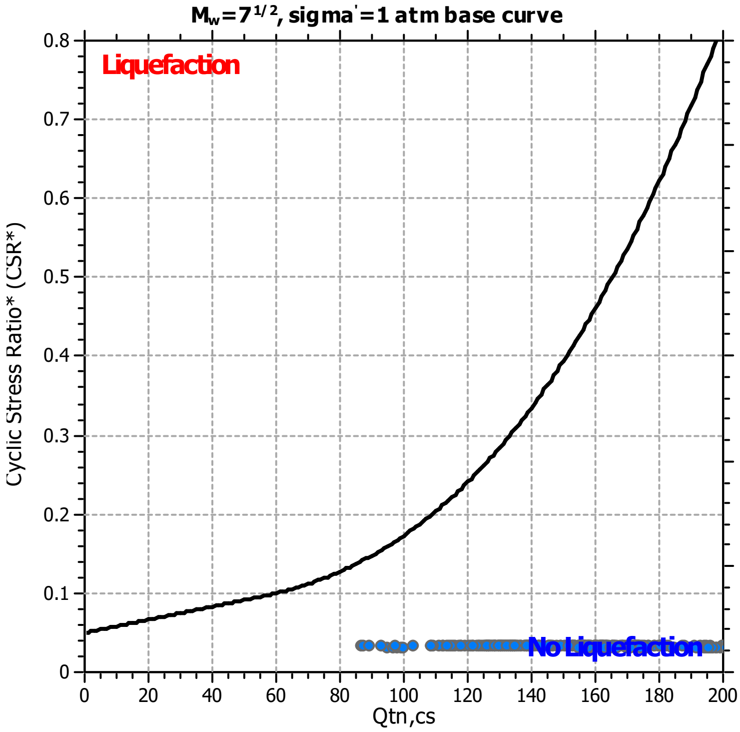
Project title : **Location :**

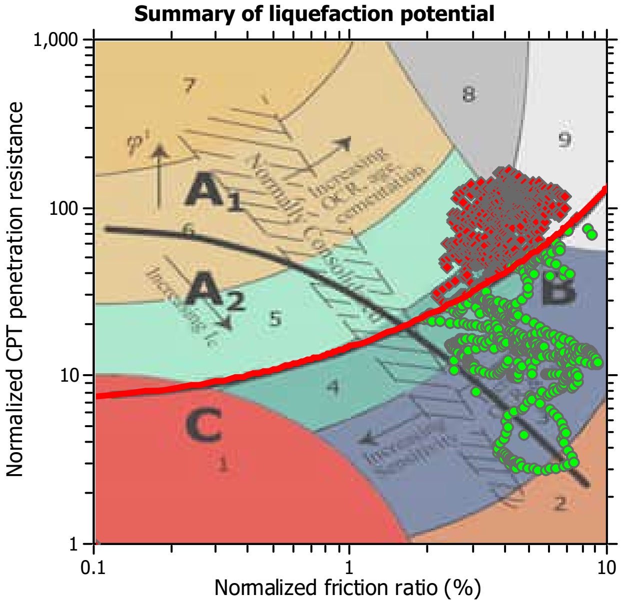
CPT file : CPT-05

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.05	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

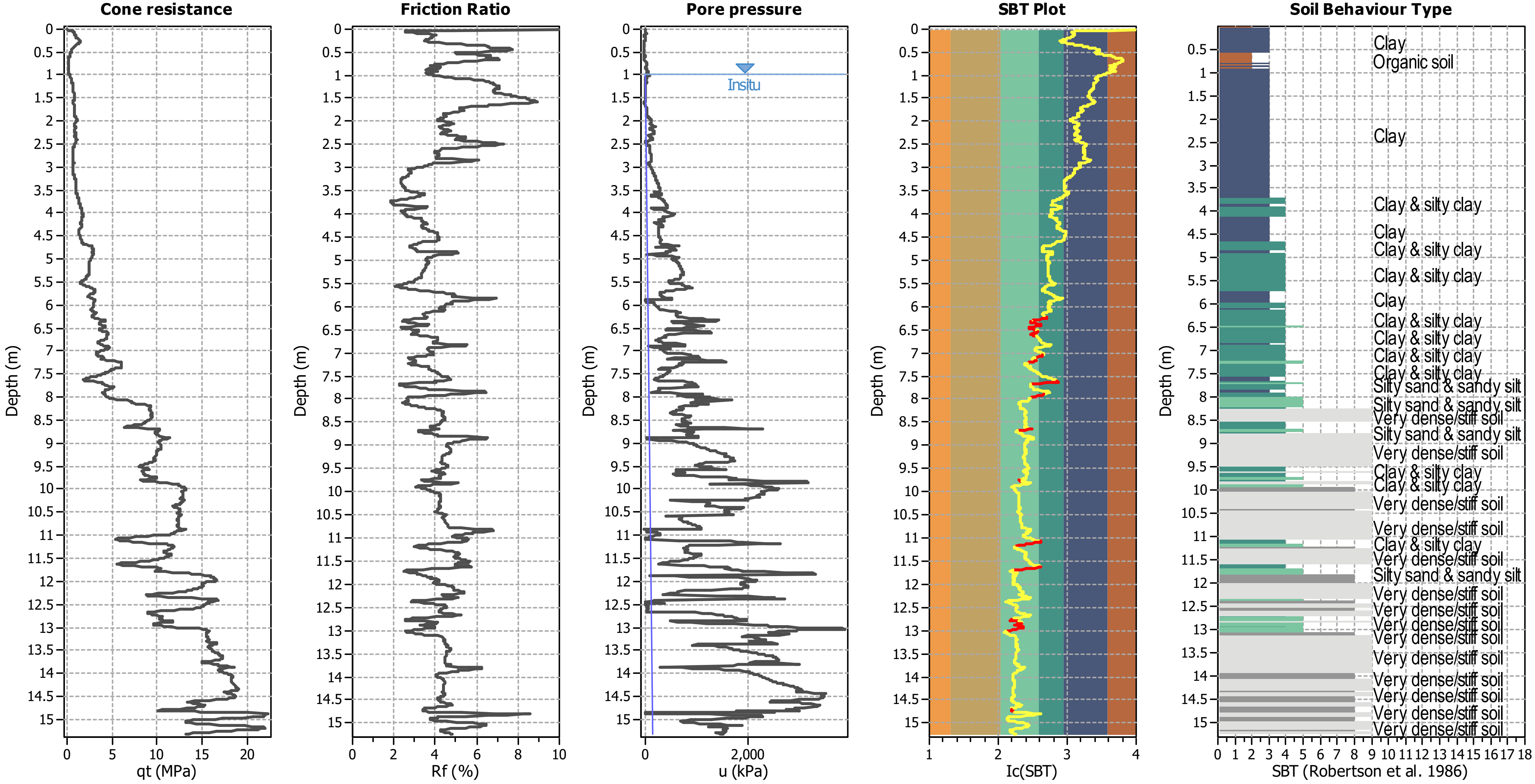






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



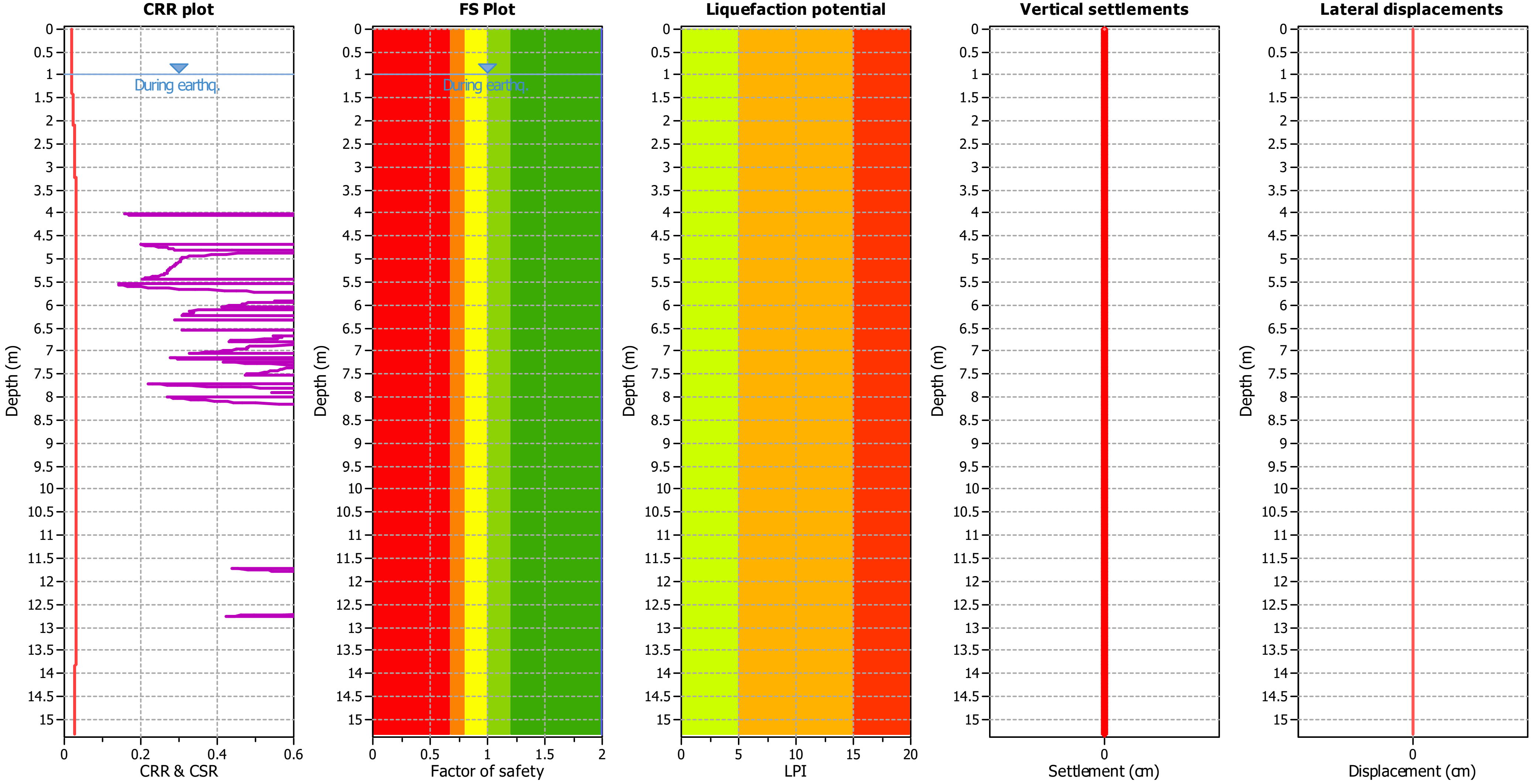
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 5.90
Peak ground acceleration: 0.05
Depth to water table (insitu): 1.00 m

Depth to water table (erthq.): 1.00 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: Sands only
Limit depth applied: No
Limit depth: N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

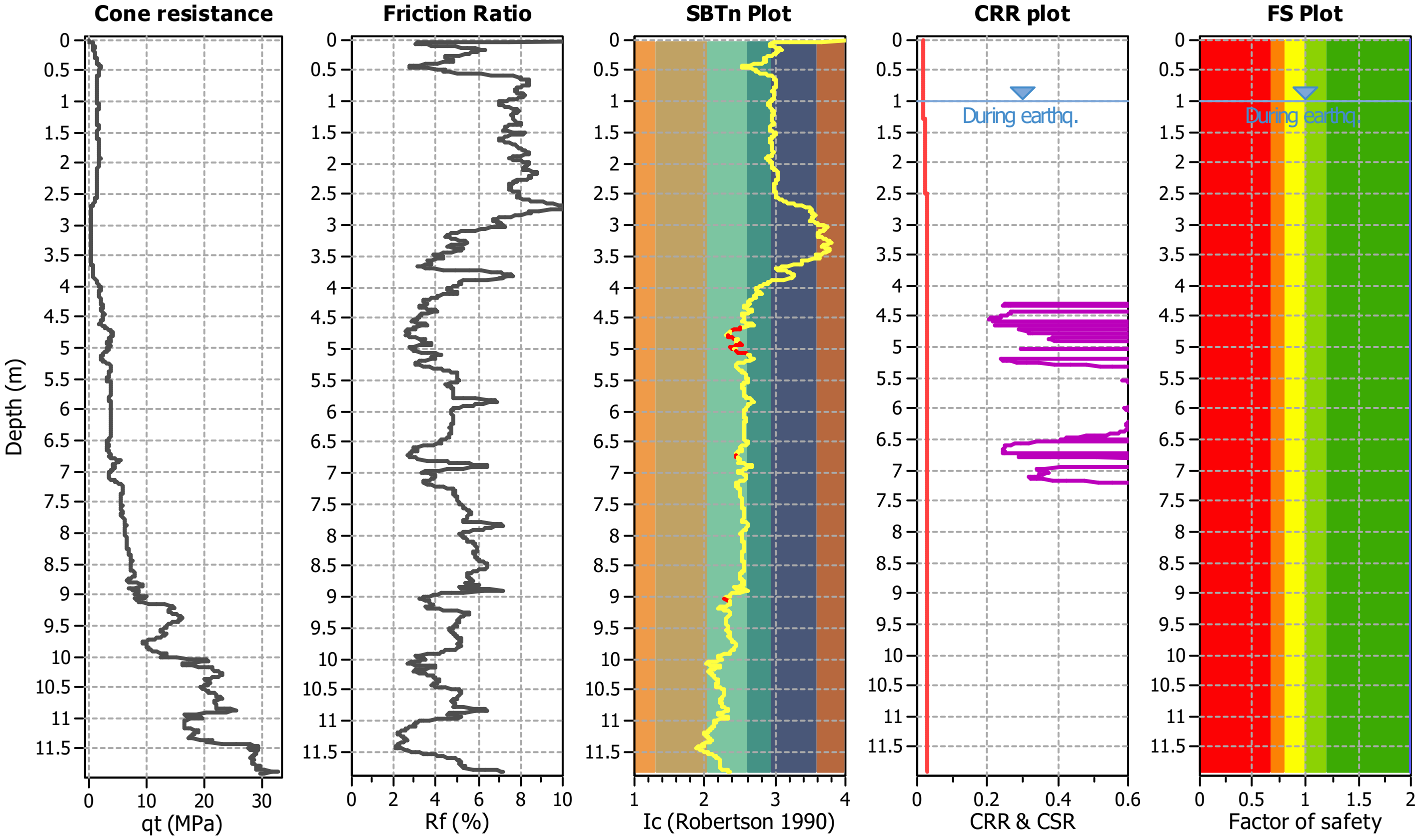
LIQUEFACTION ANALYSIS REPORT

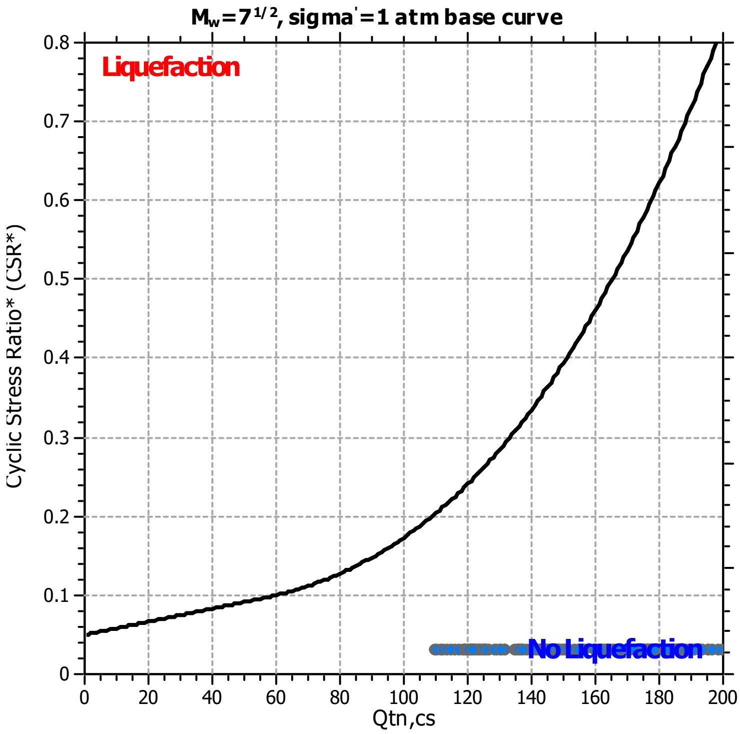
Project title : **Location :**

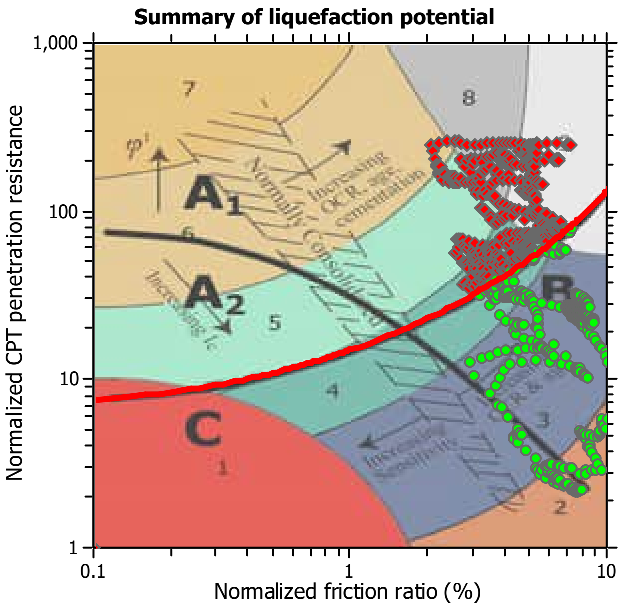
CPT file : CPT-04

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.05	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

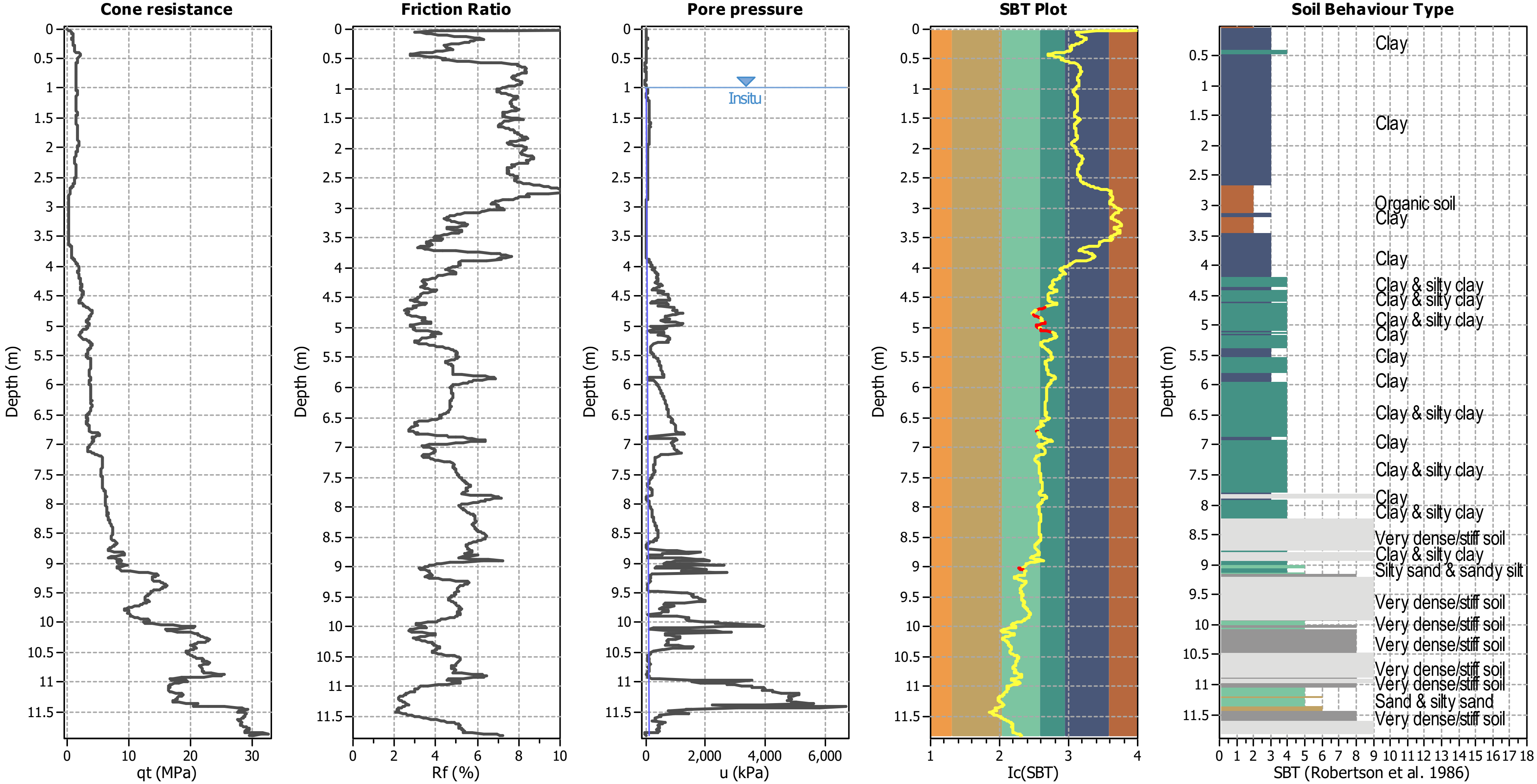






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



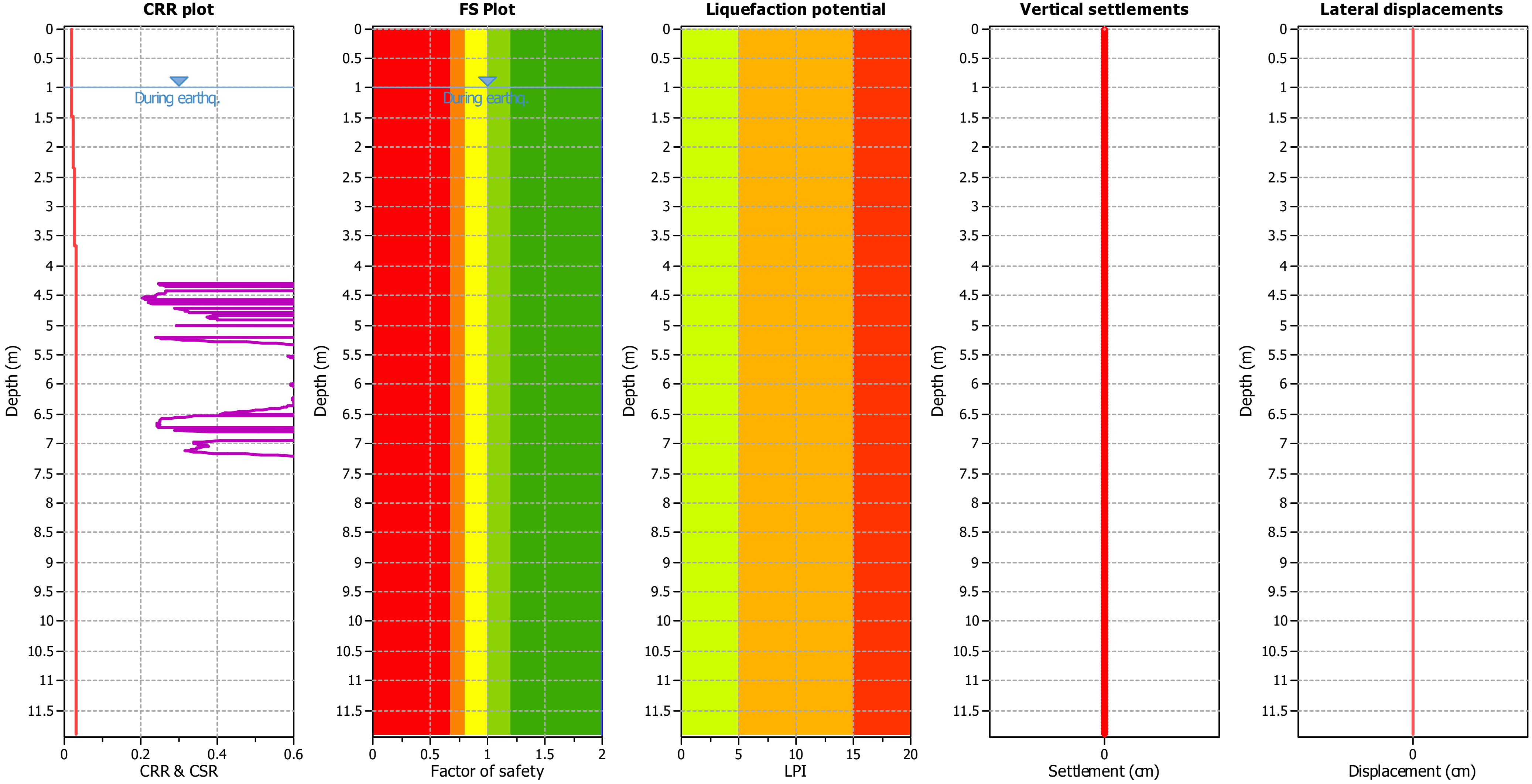
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

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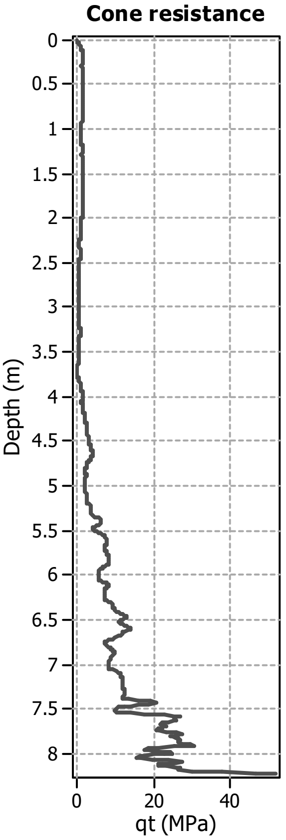
LIQUEFACTION ANALYSIS REPORT

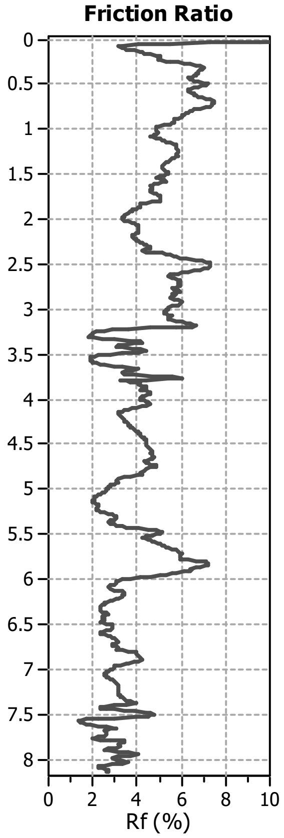
Project title : **Location :**

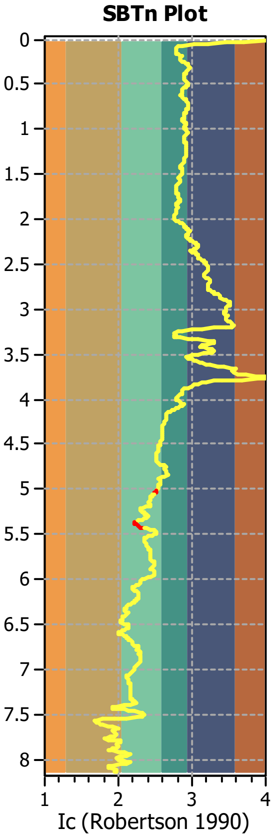
CPT file : CPT-03

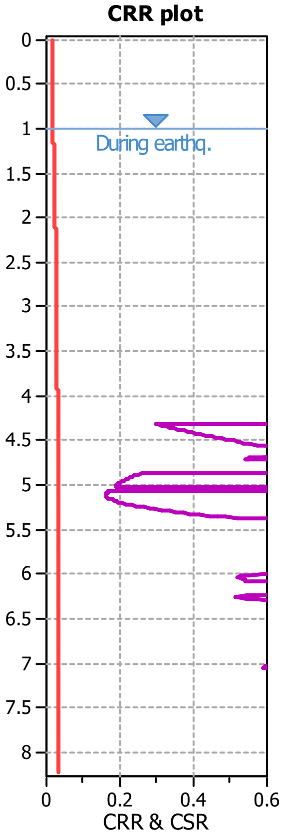
Input parameters and analysis data

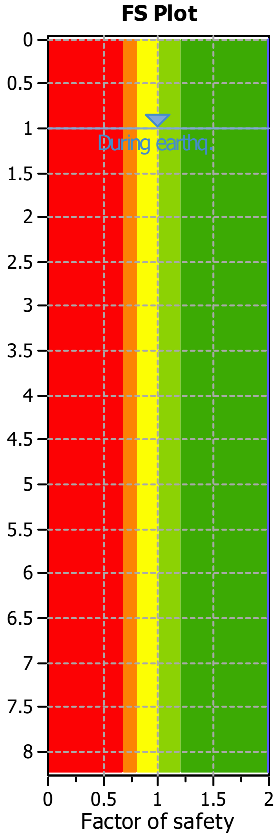
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.05	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

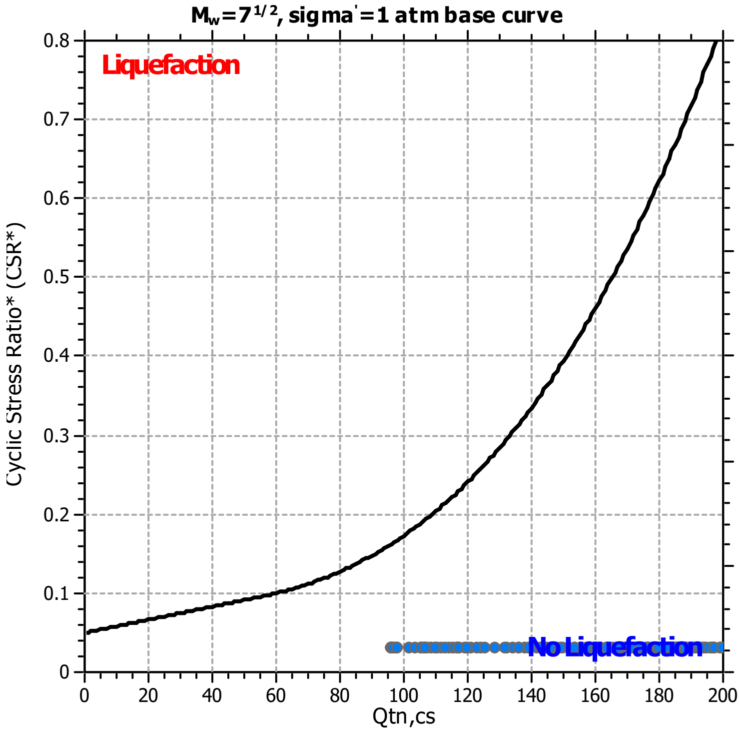


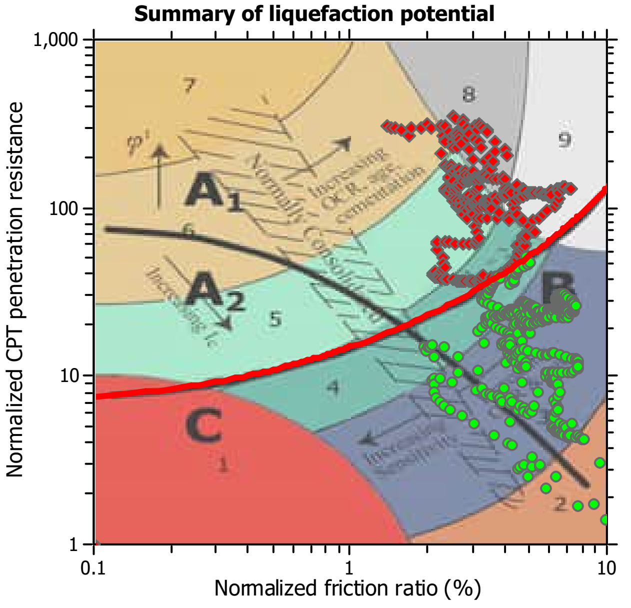






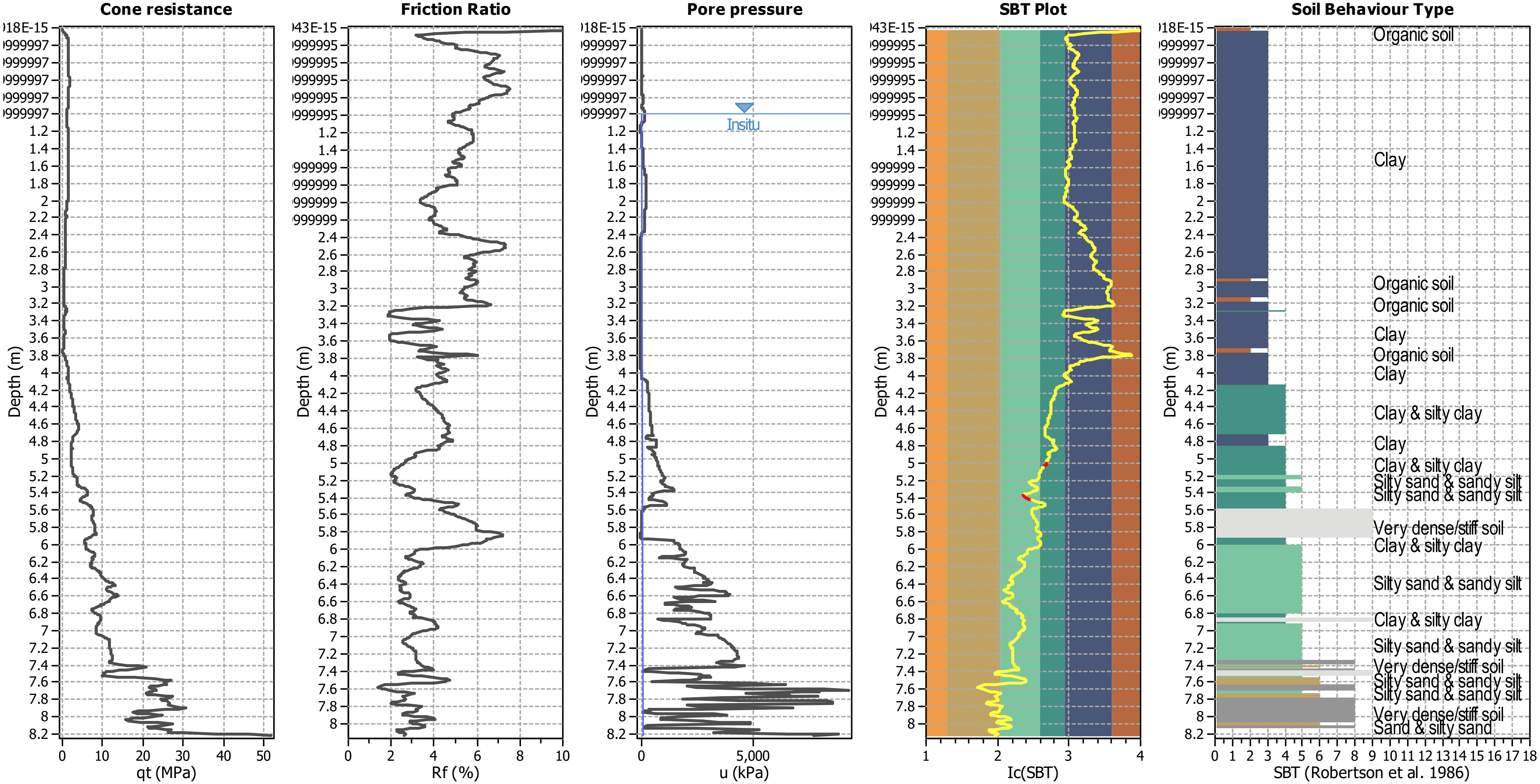






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



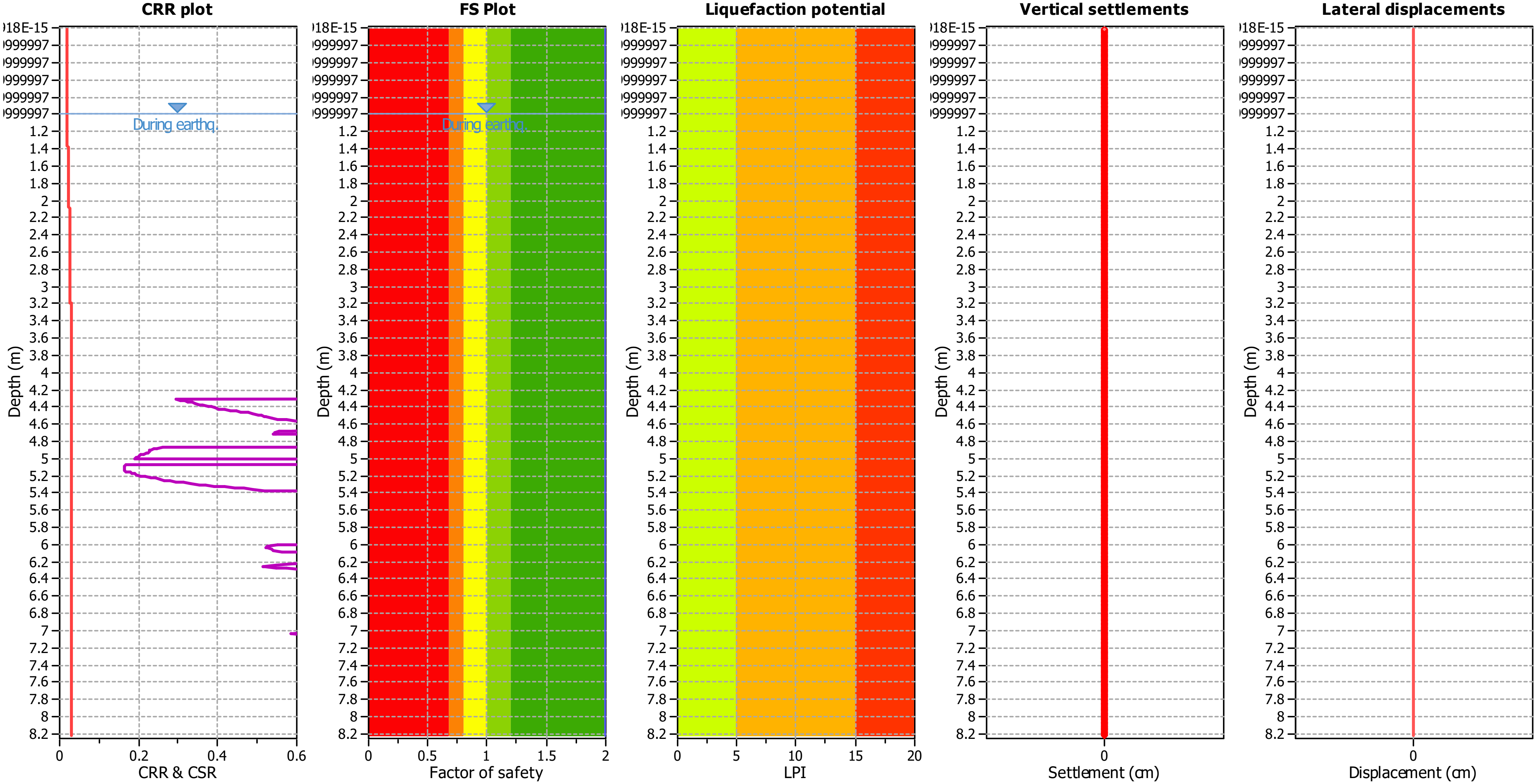
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

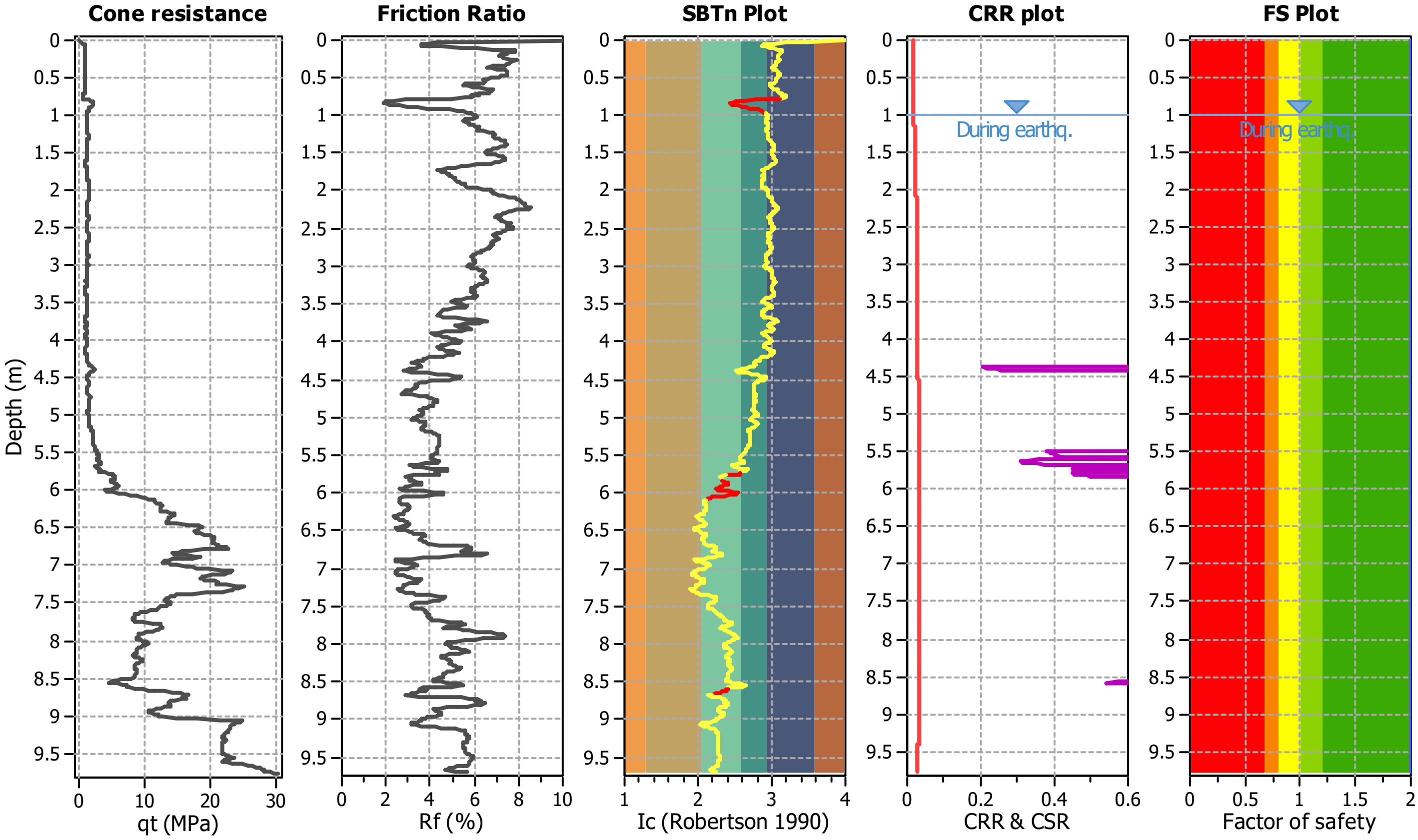
LIQUEFACTION ANALYSIS REPORT

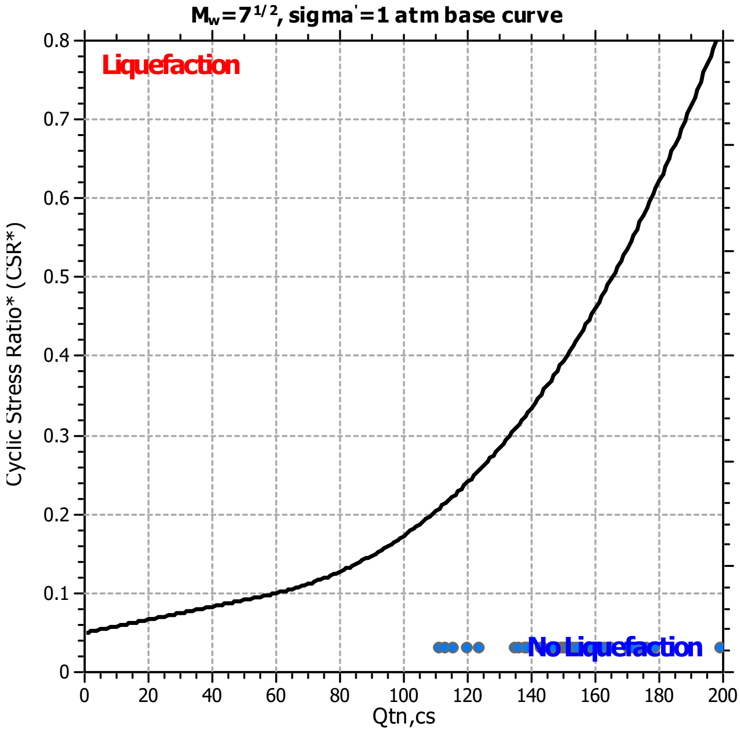
Project title : **Location :**

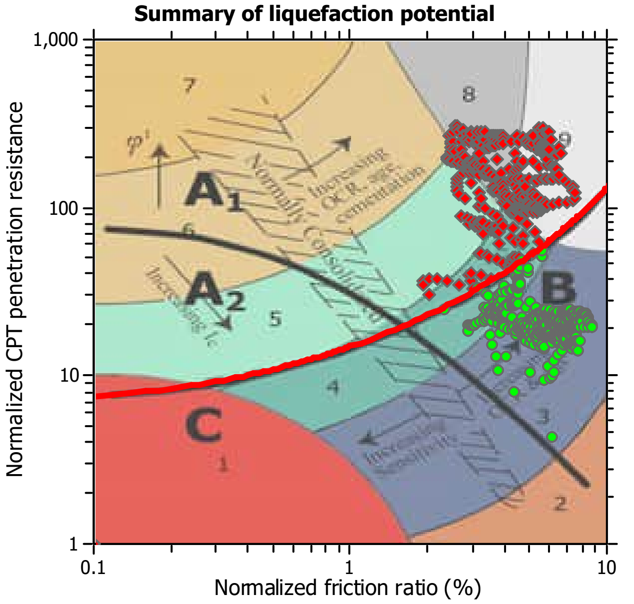
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.05	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

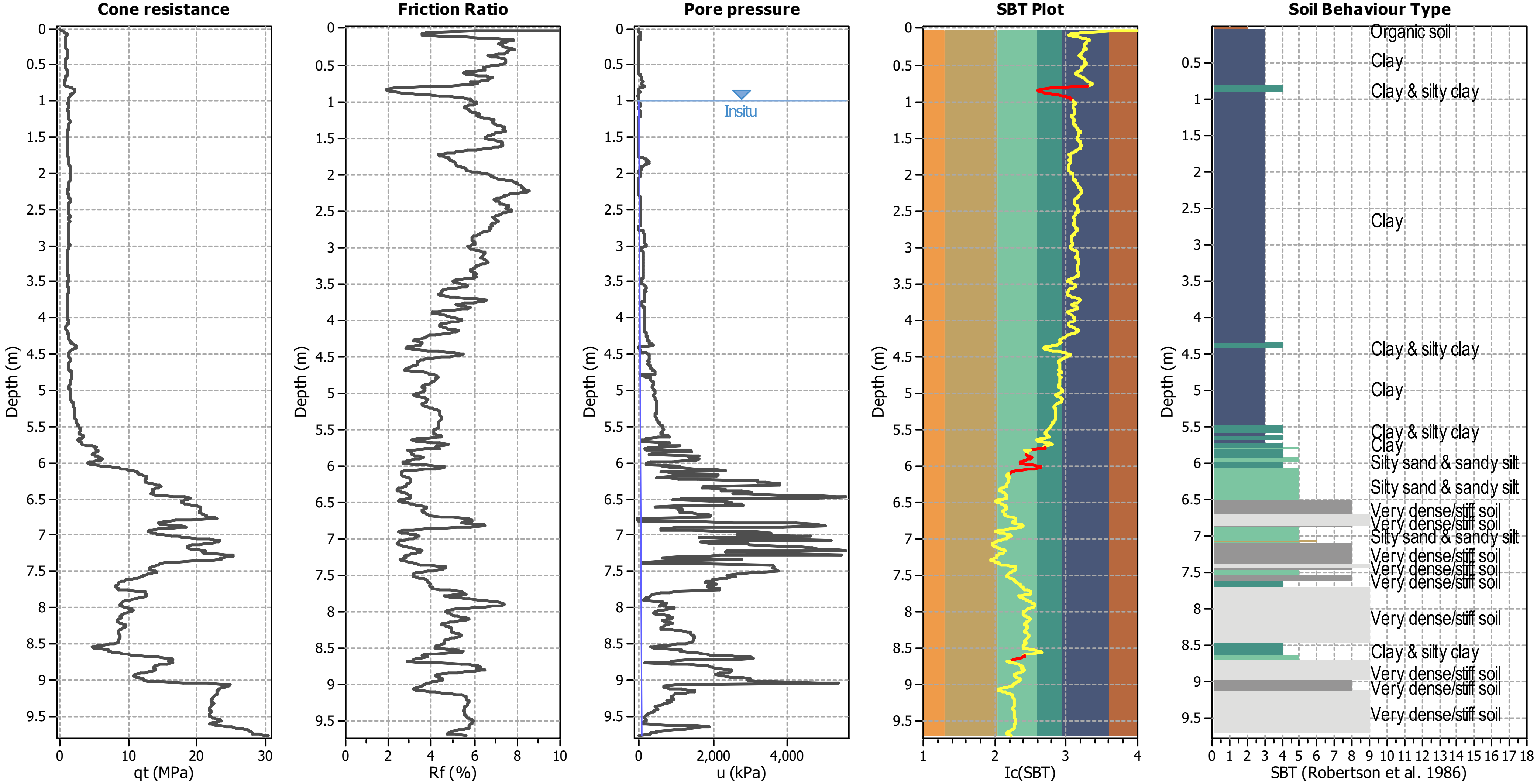






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



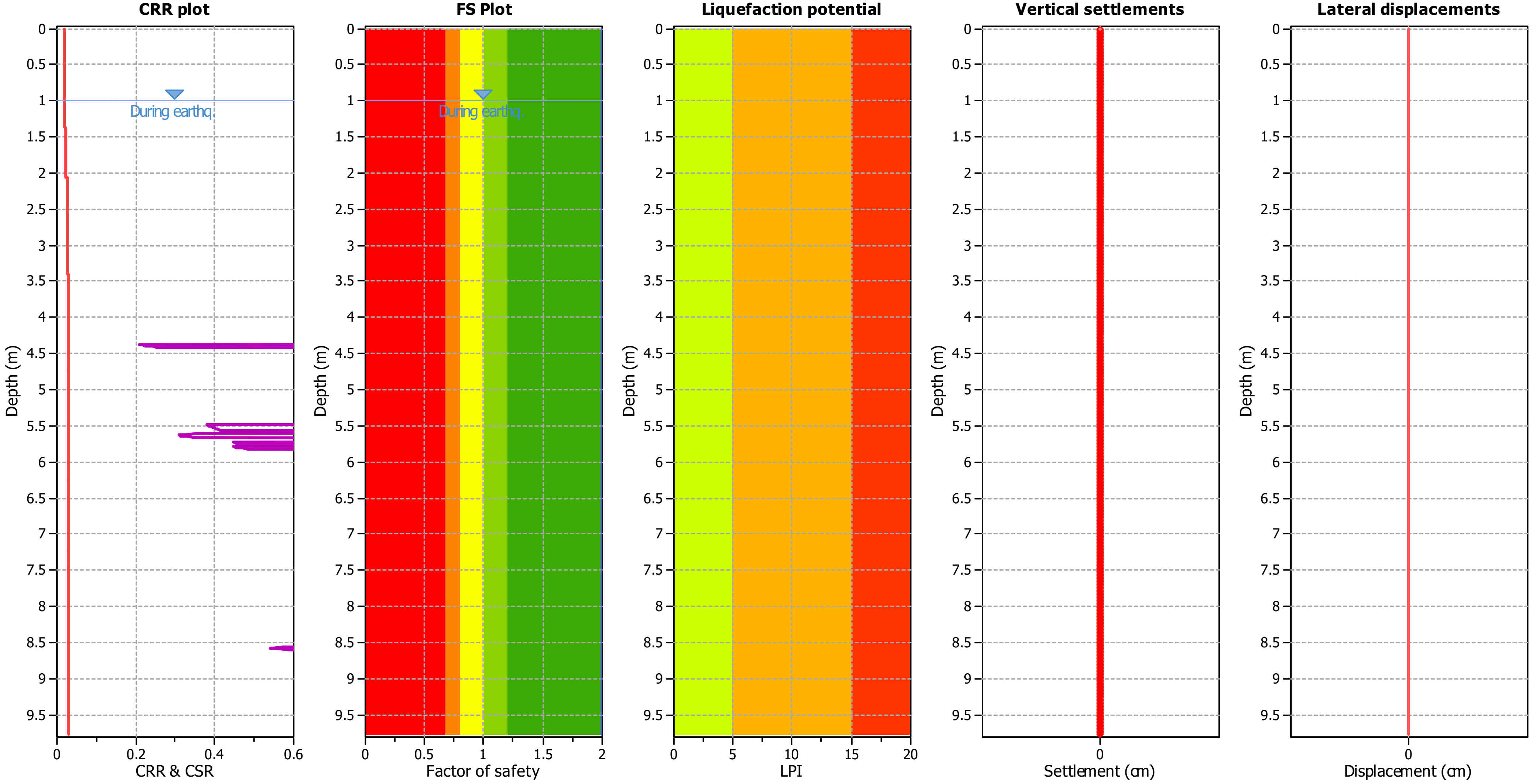
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 5.90
Peak ground acceleration: 0.05
Depth to water table (insitu): 1.00 m
Depth to water table (erthq.): 1.00 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: Sands only
Limit depth applied: No
Limit depth: N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

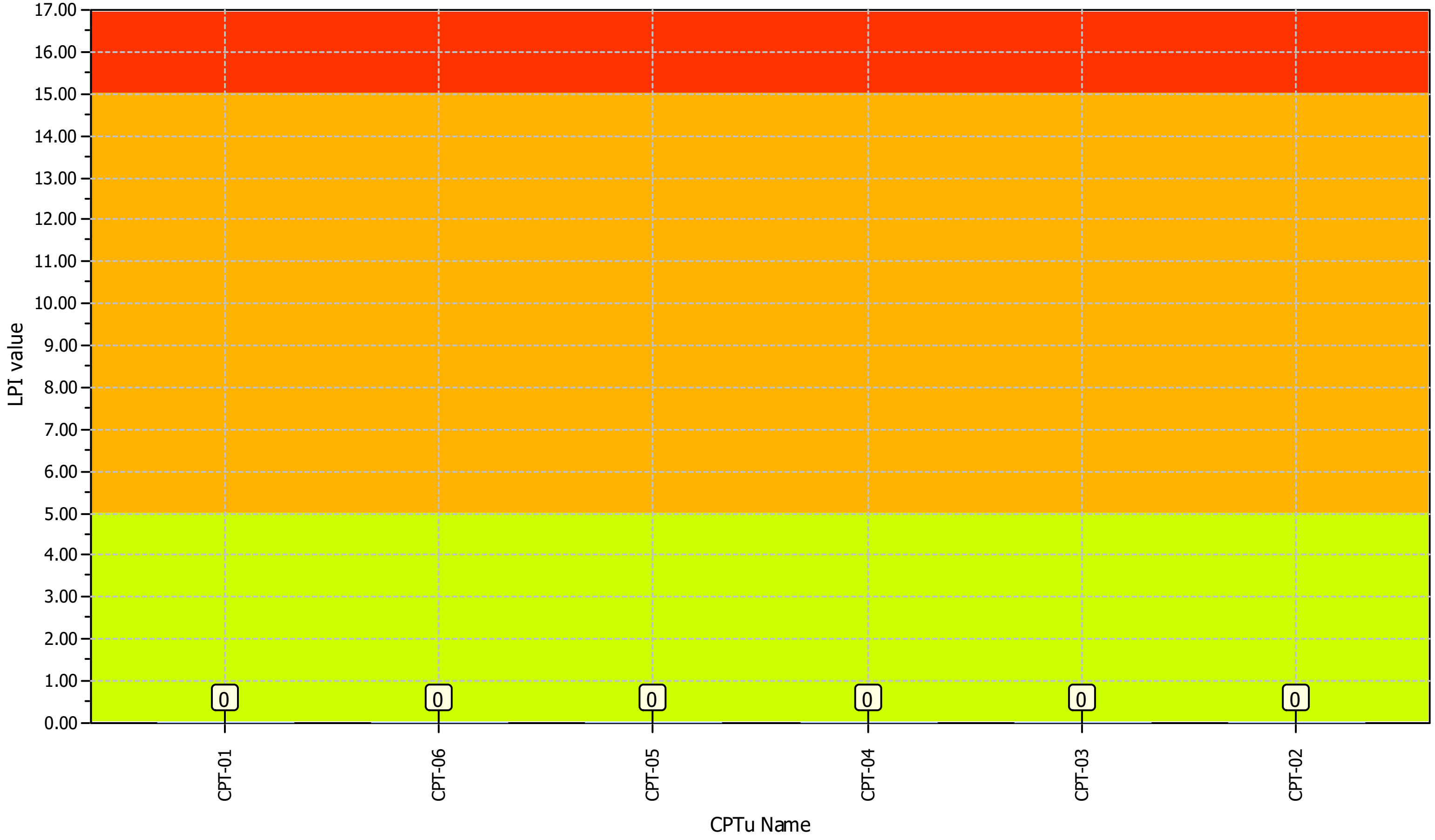
LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

Project title :

Location :

Overall Liquefaction Potential Index report



LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟩 Low risk

Basic statistics

Total CPT number: 6

100% low risk

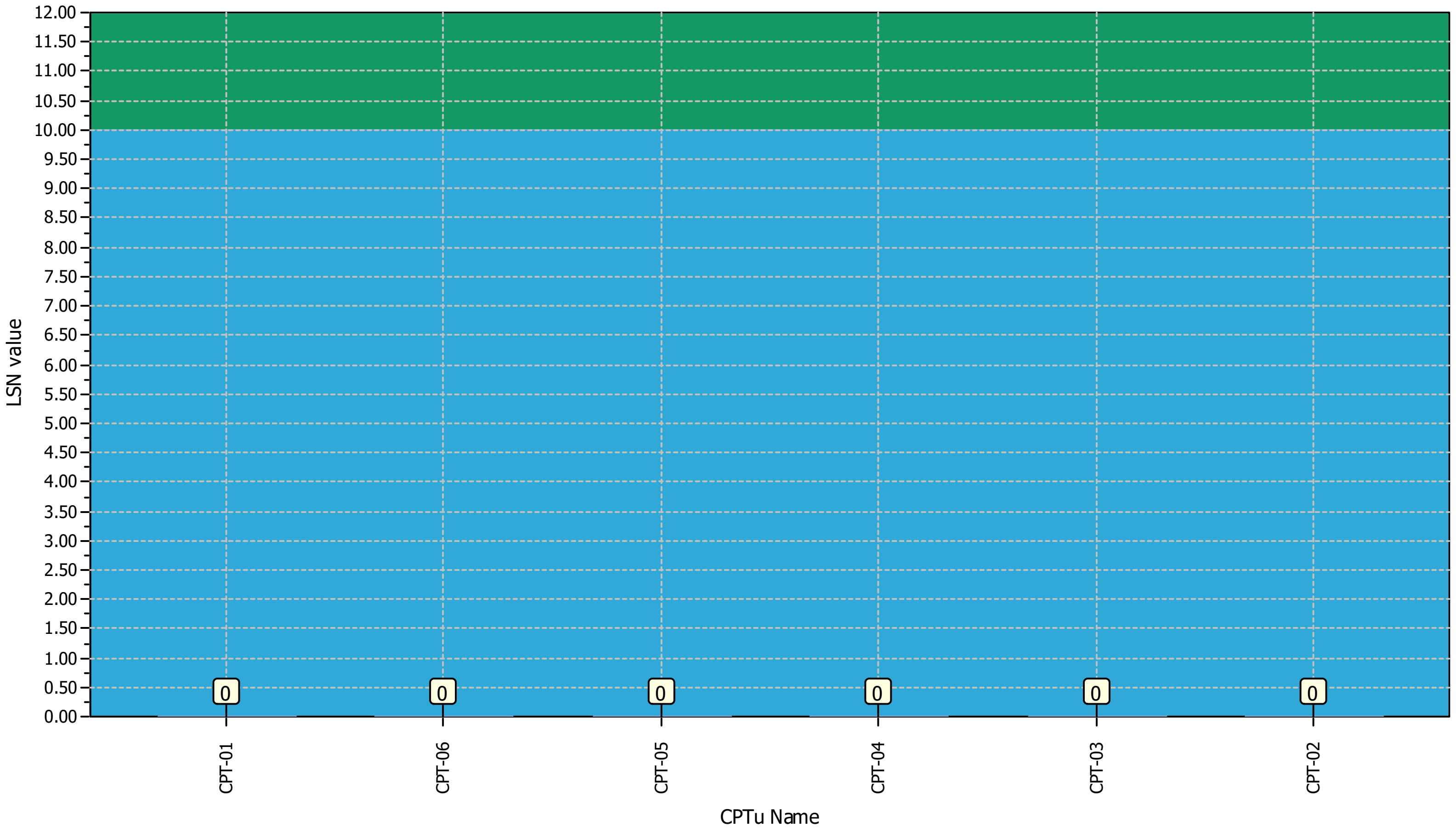
0% high risk

0% very high risk

Project title :

Location :

Overall Liquefaction Severity Number report



LSN color scheme

🟥 Severe damage
🟥 Major expression of liquefaction
🟧 Moderate to severe exp. of liquefaction
🟨 Moderate expression of liquefaction
🟩 Minor expression of liquefaction
🟦 Little to no expression of liquefaction

Basic statistics

Total CPT number: 6

100% little liquefaction

0% minor liquefaction

0% moderate liquefaction

0% moderate to major liquefaction

0% major liquefaction

0% severe liquefaction

GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

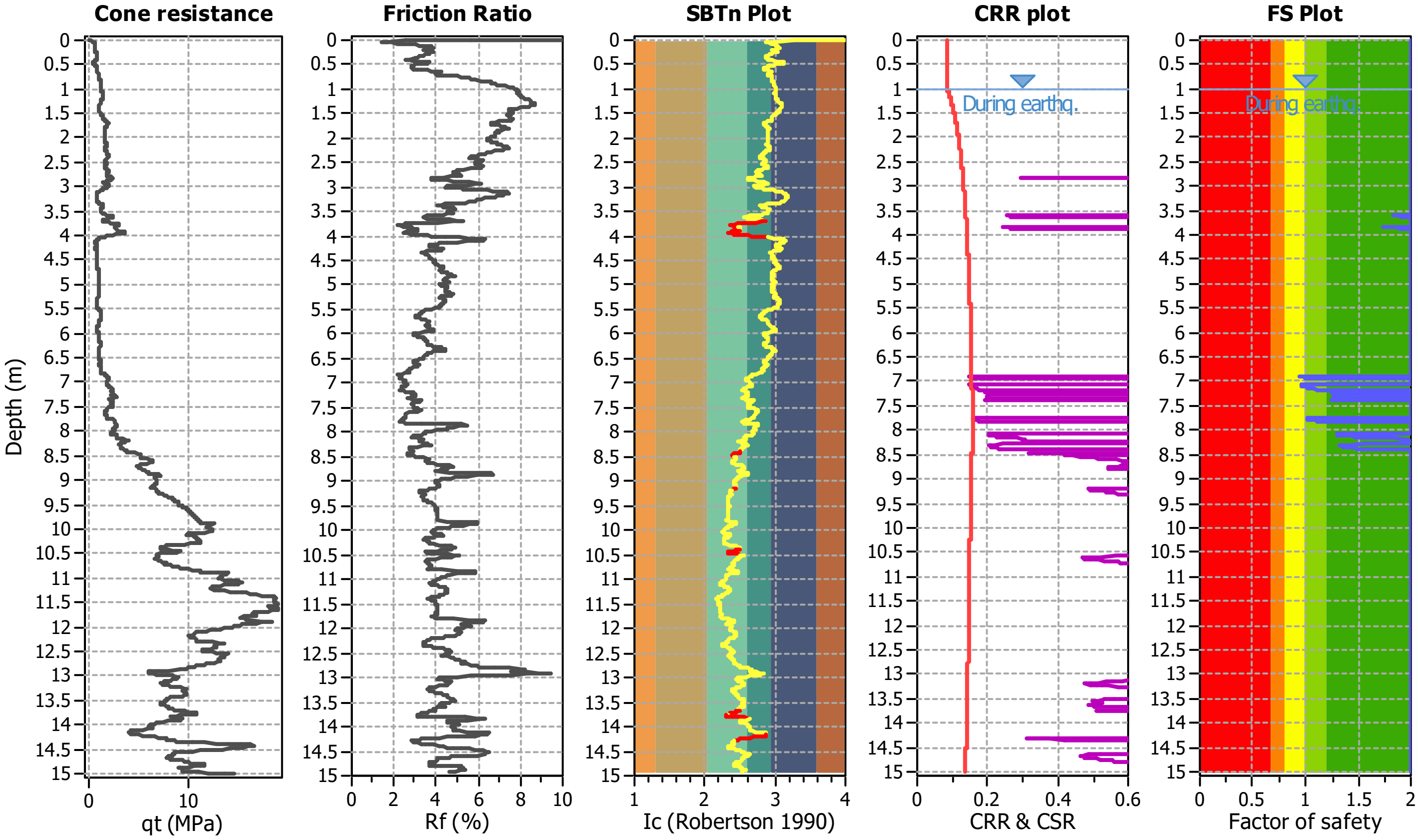
LIQUEFACTION ANALYSIS REPORT

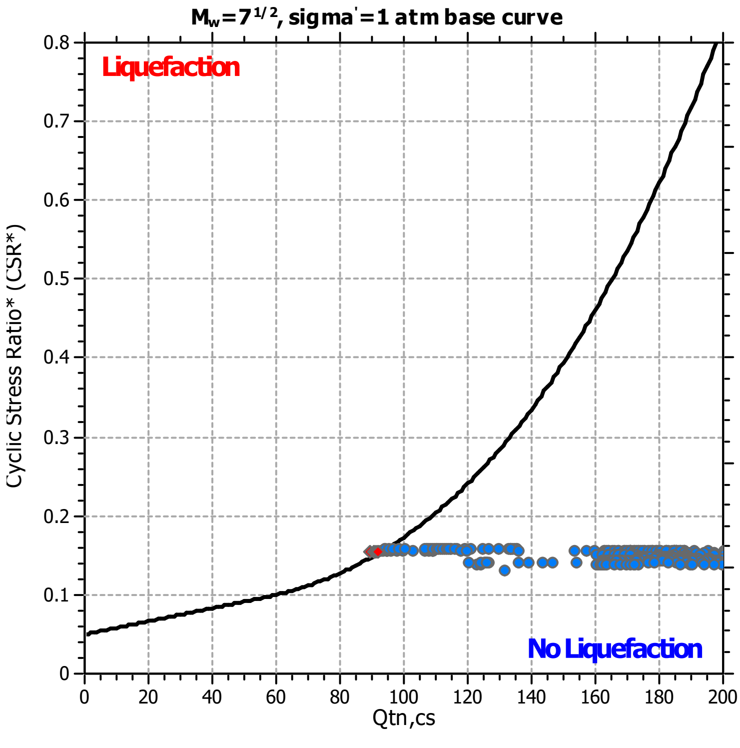
Project title : **Location :**

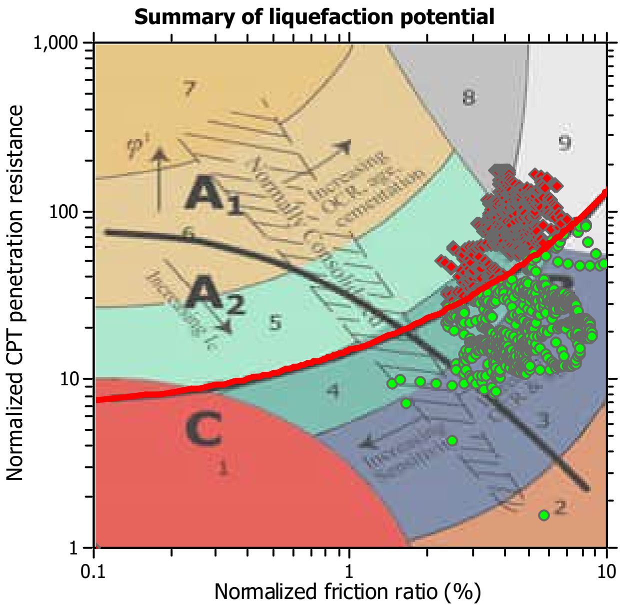
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	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		

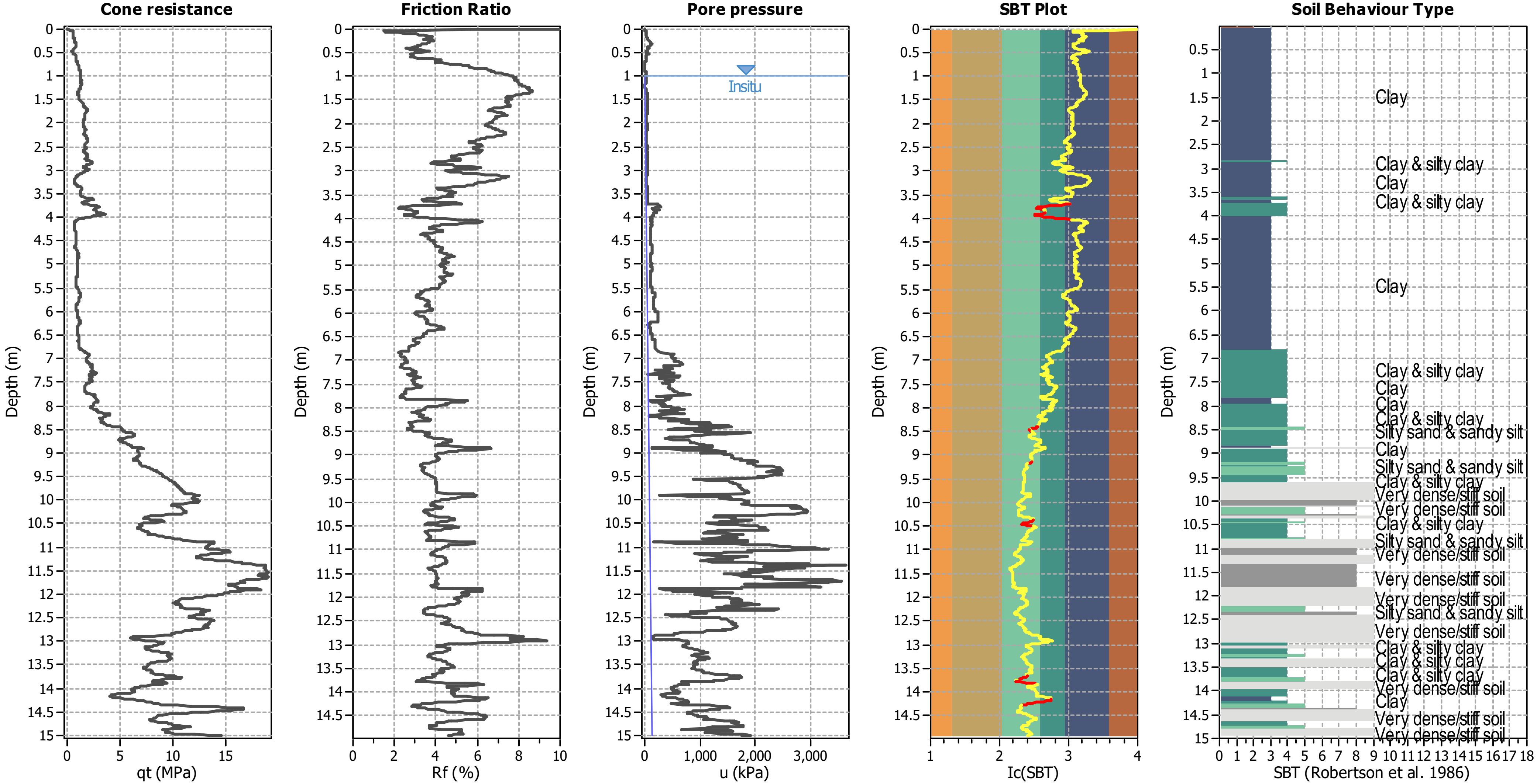






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



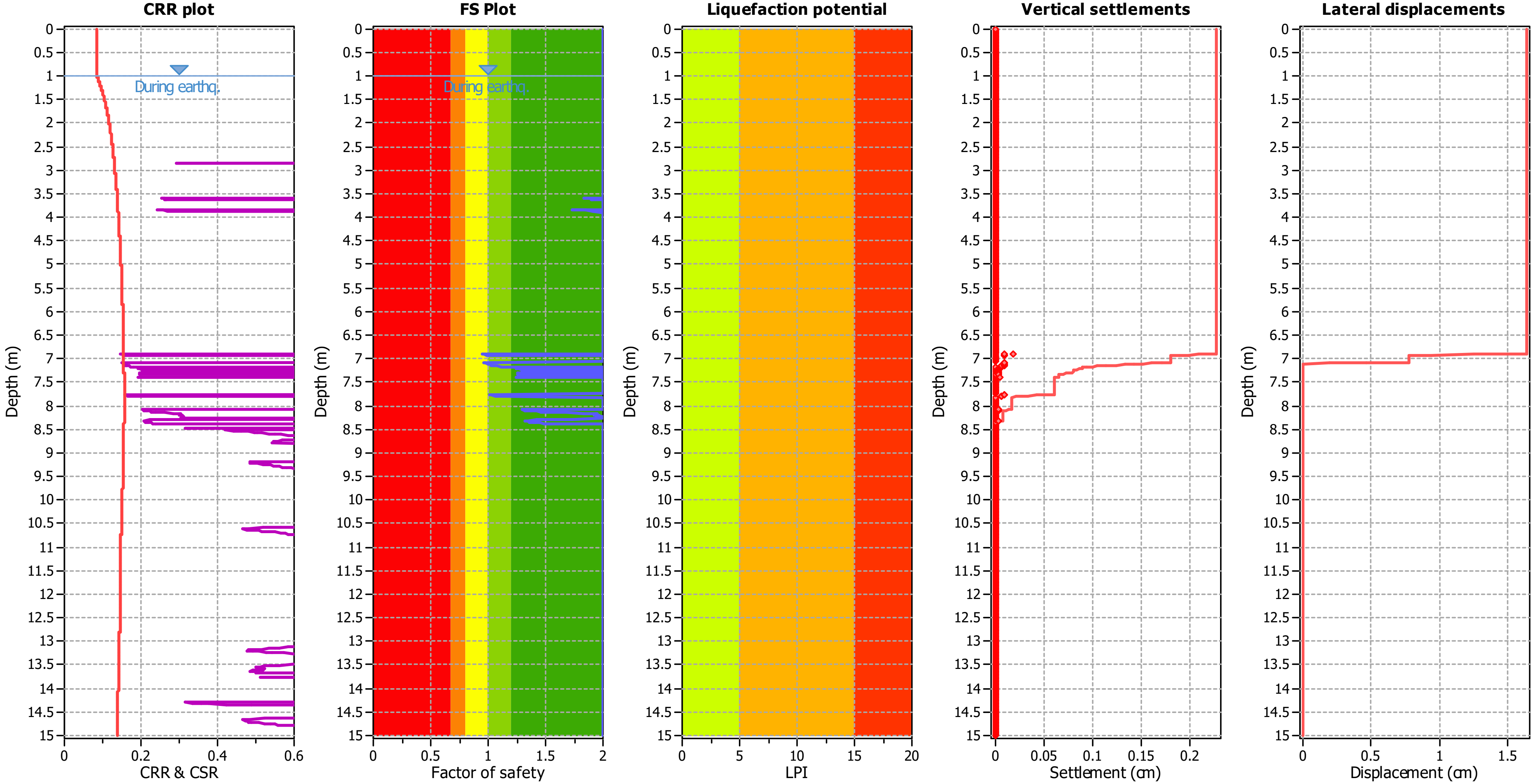
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.50
Peak ground acceleration: 0.19
Depth to water table (insitu): 1.00 m

Depth to water table (erthq.): 1.00 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: Sands only
Limit depth applied: No
Limit depth: N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

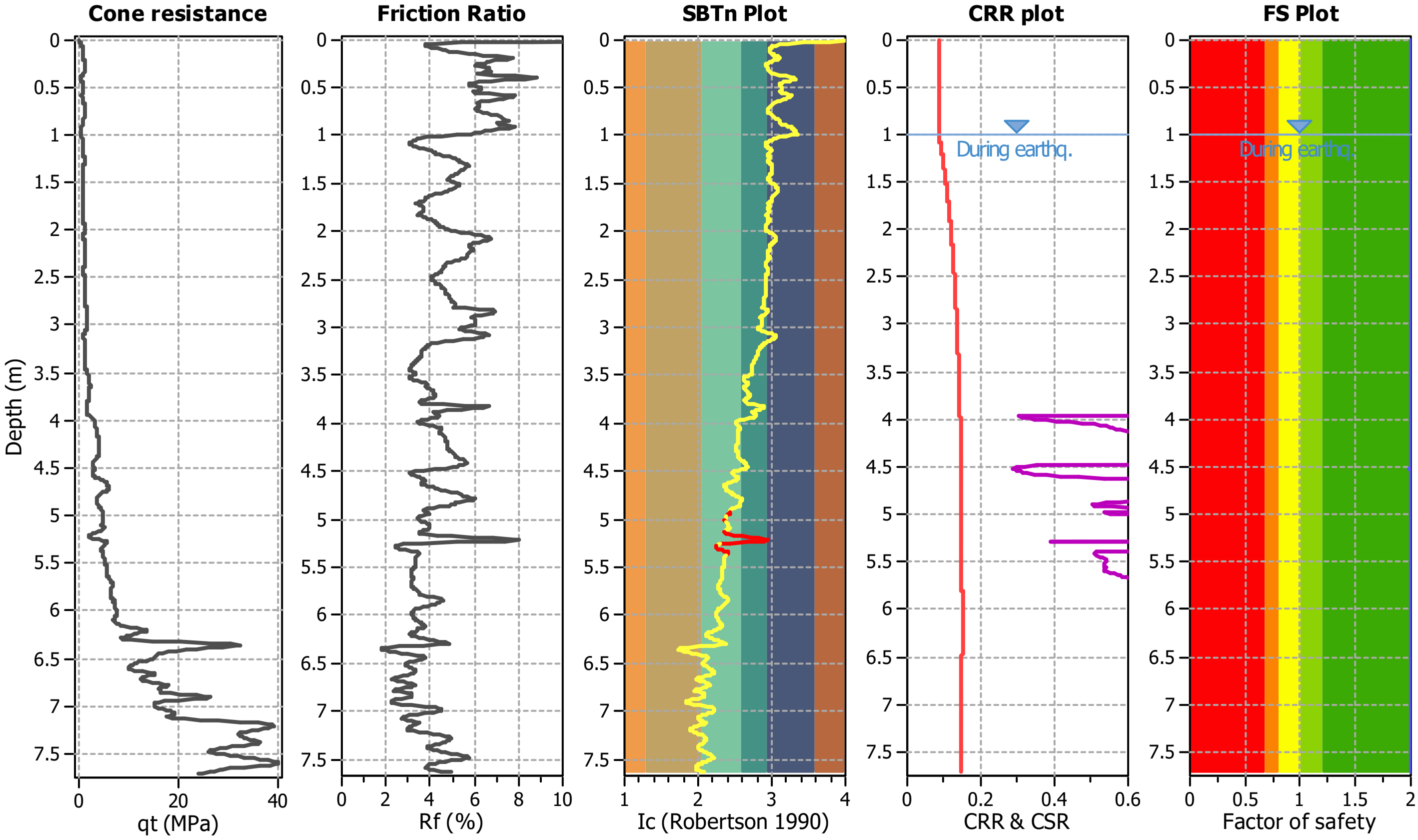
LIQUEFACTION ANALYSIS REPORT

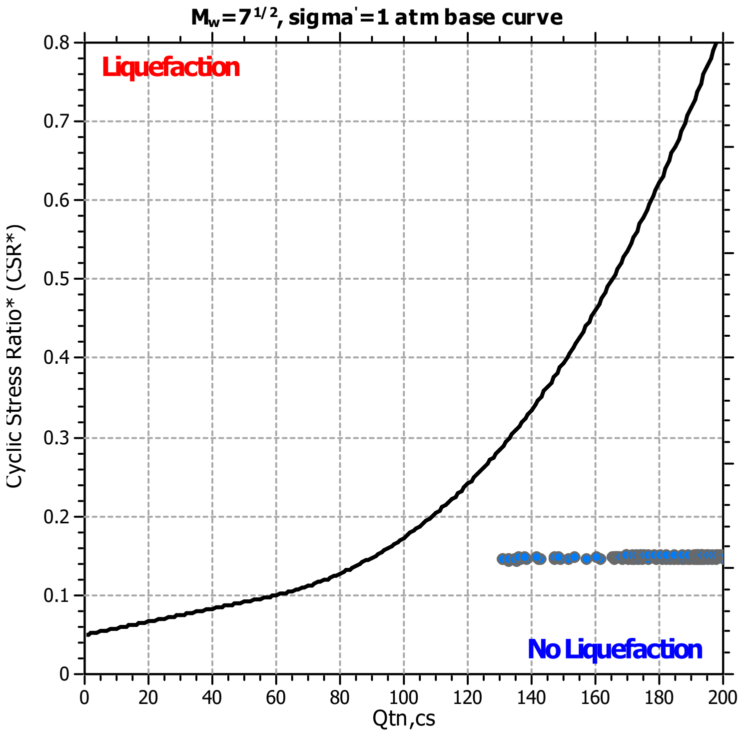
Project title : **Location :**

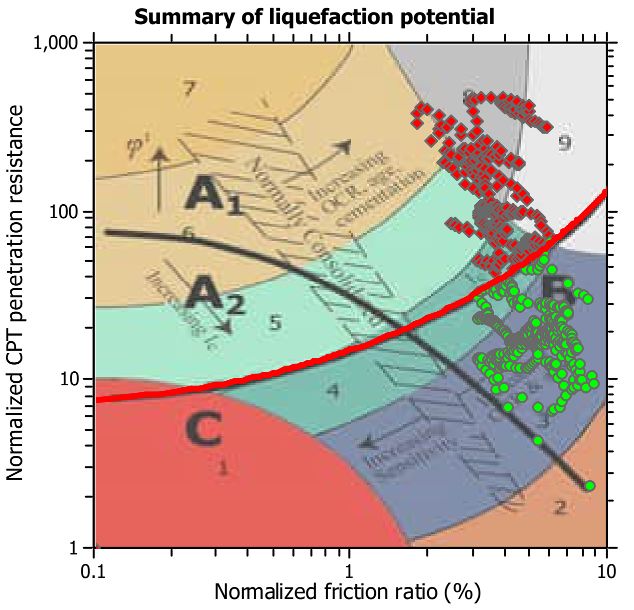
CPT file : CPT-06

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	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		

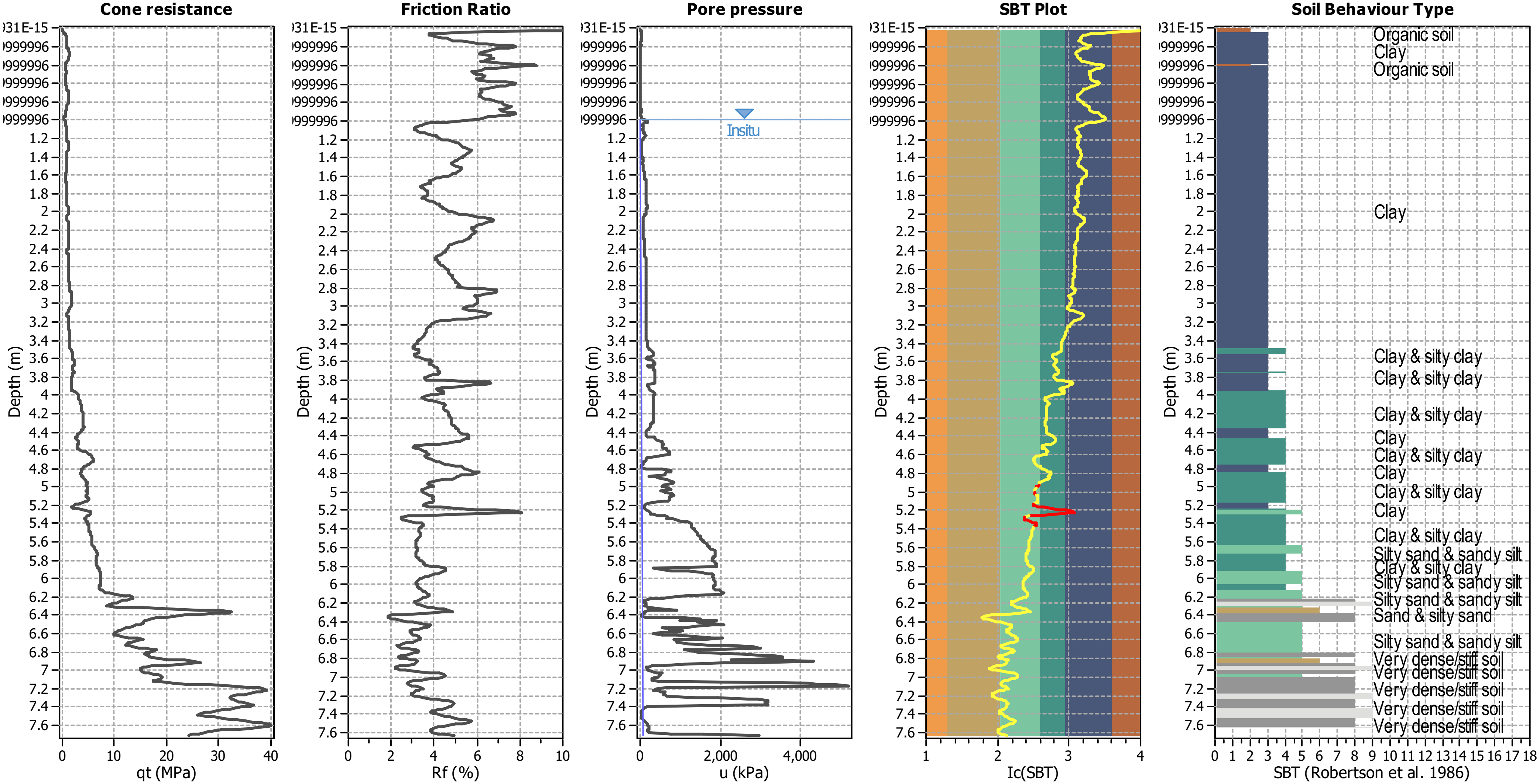






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



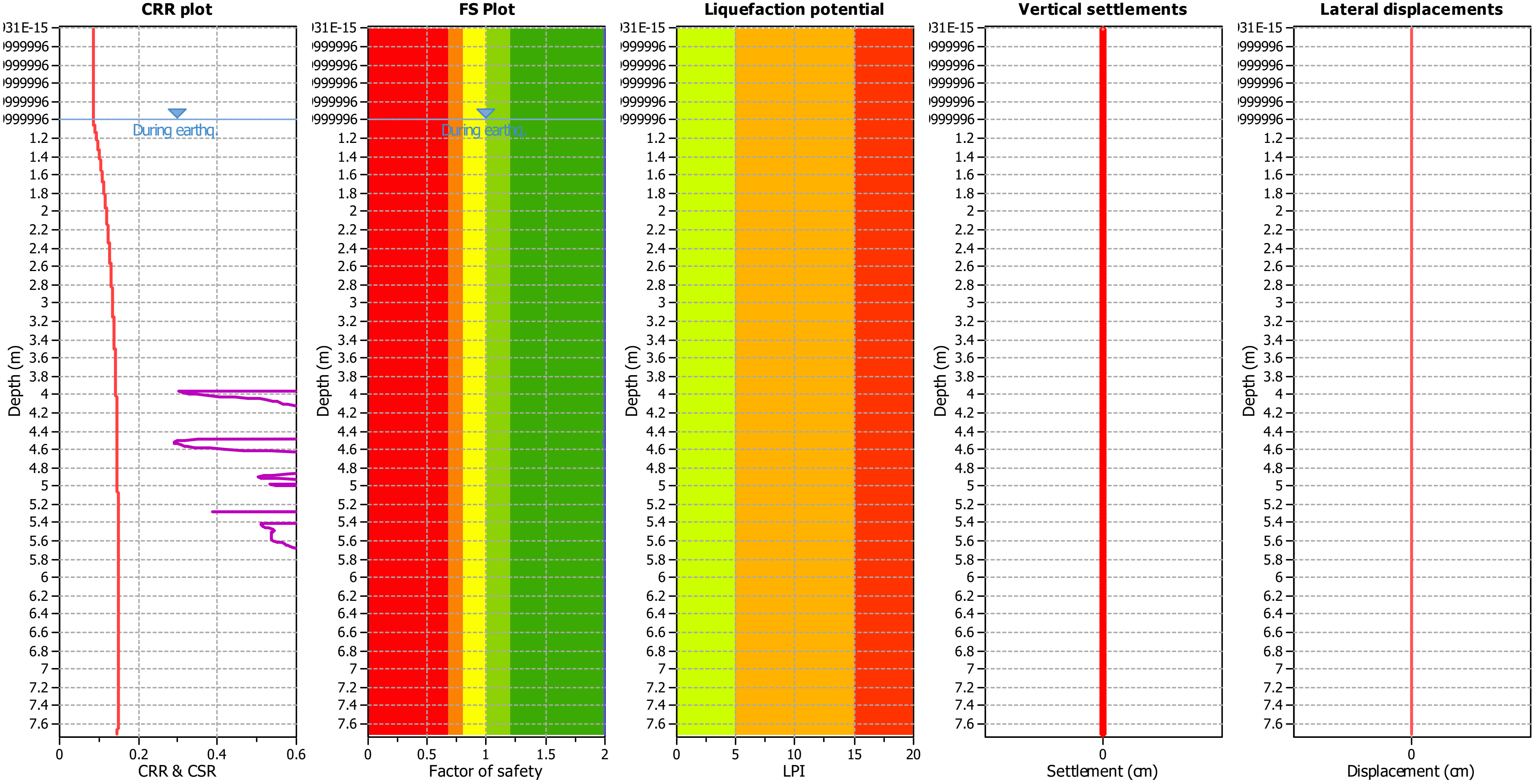
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

LIQUEFACTION ANALYSIS REPORT

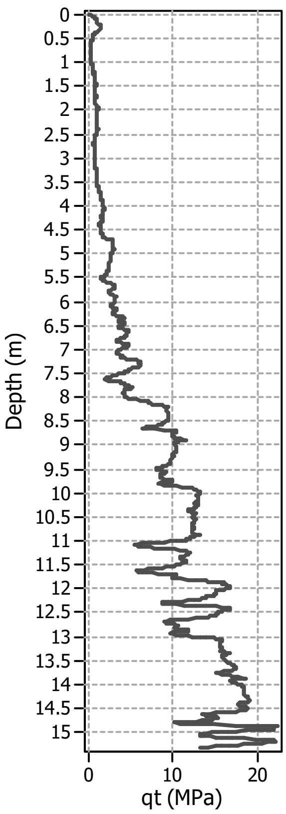
Project title : **Location :**

CPT file : CPT-05

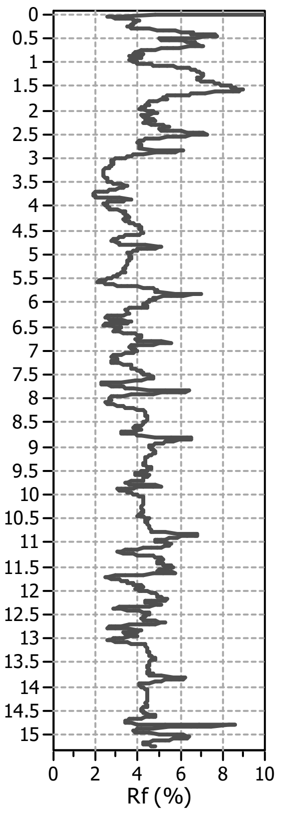
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	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		

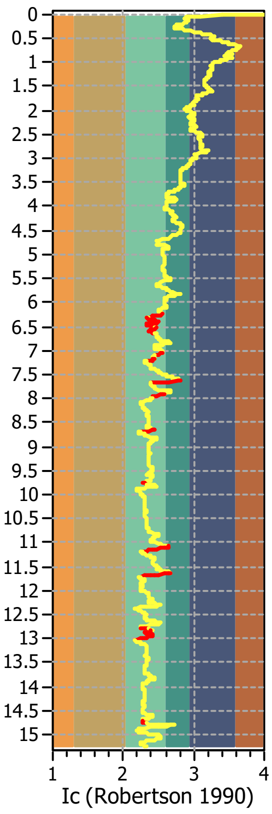
Cone resistance



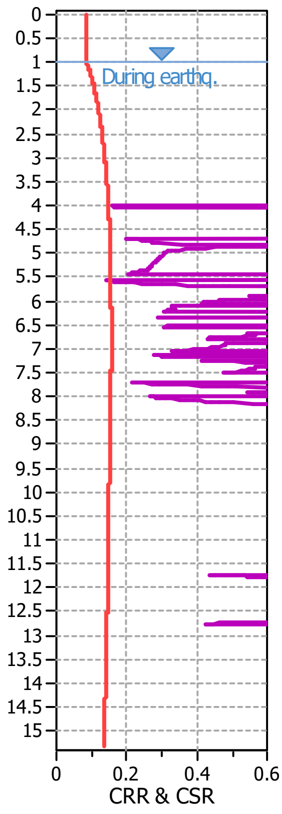
Friction Ratio



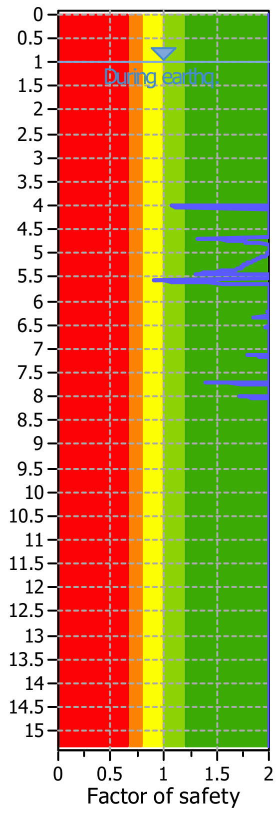
SBTn Plot



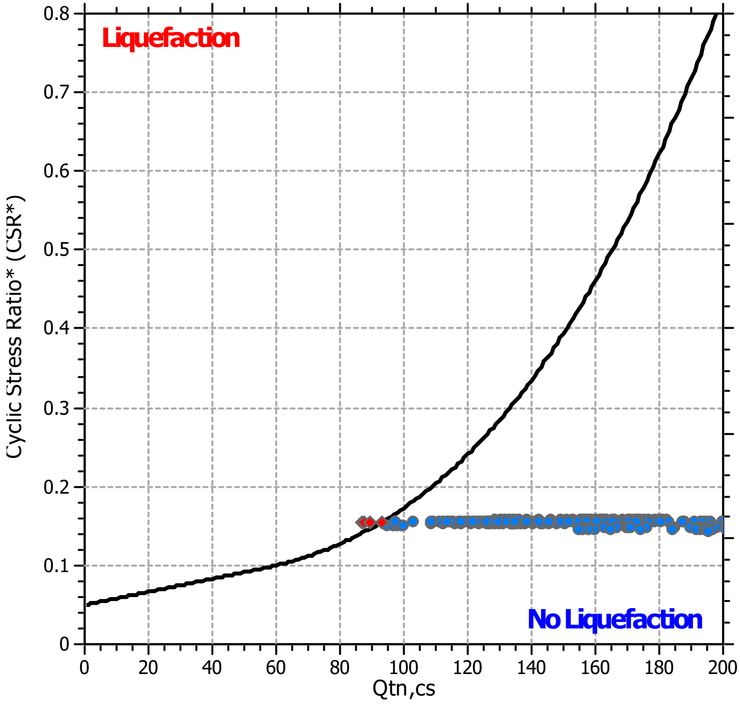
CRR plot



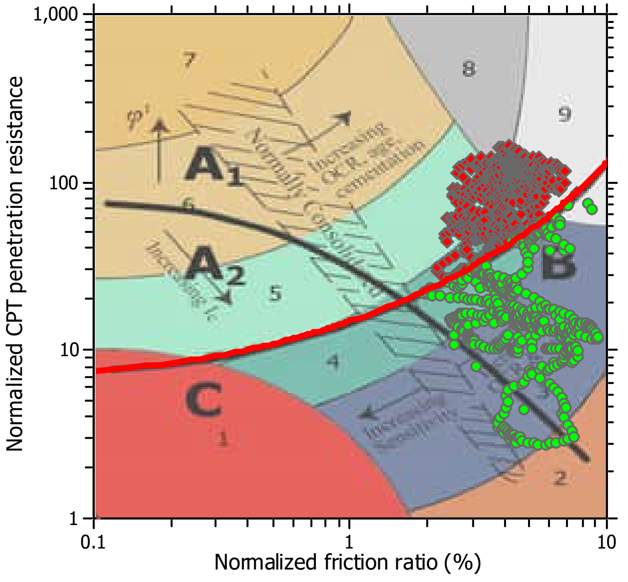
FS Plot



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

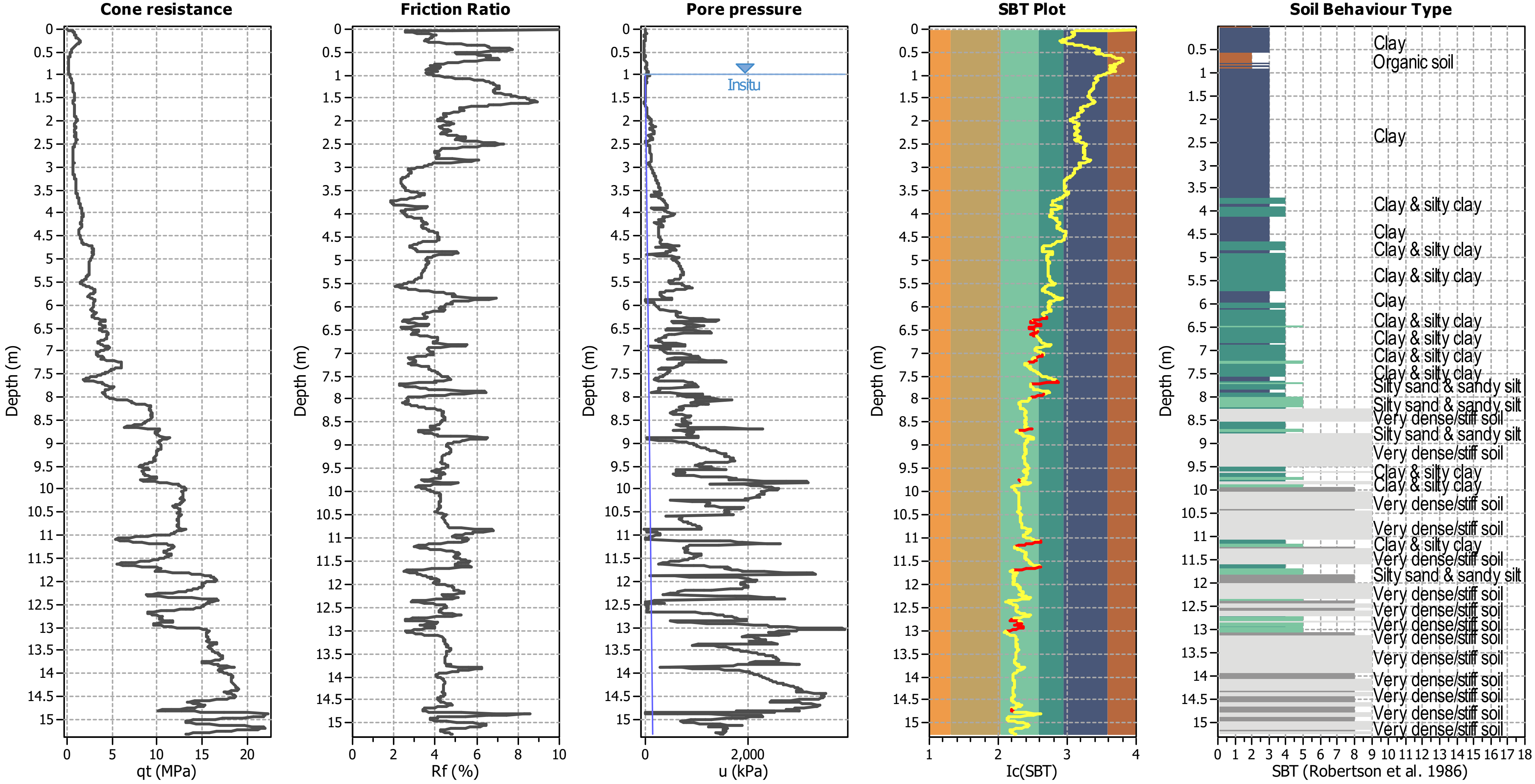


Summary of liquefaction potential



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

CPT basic interpretation plots



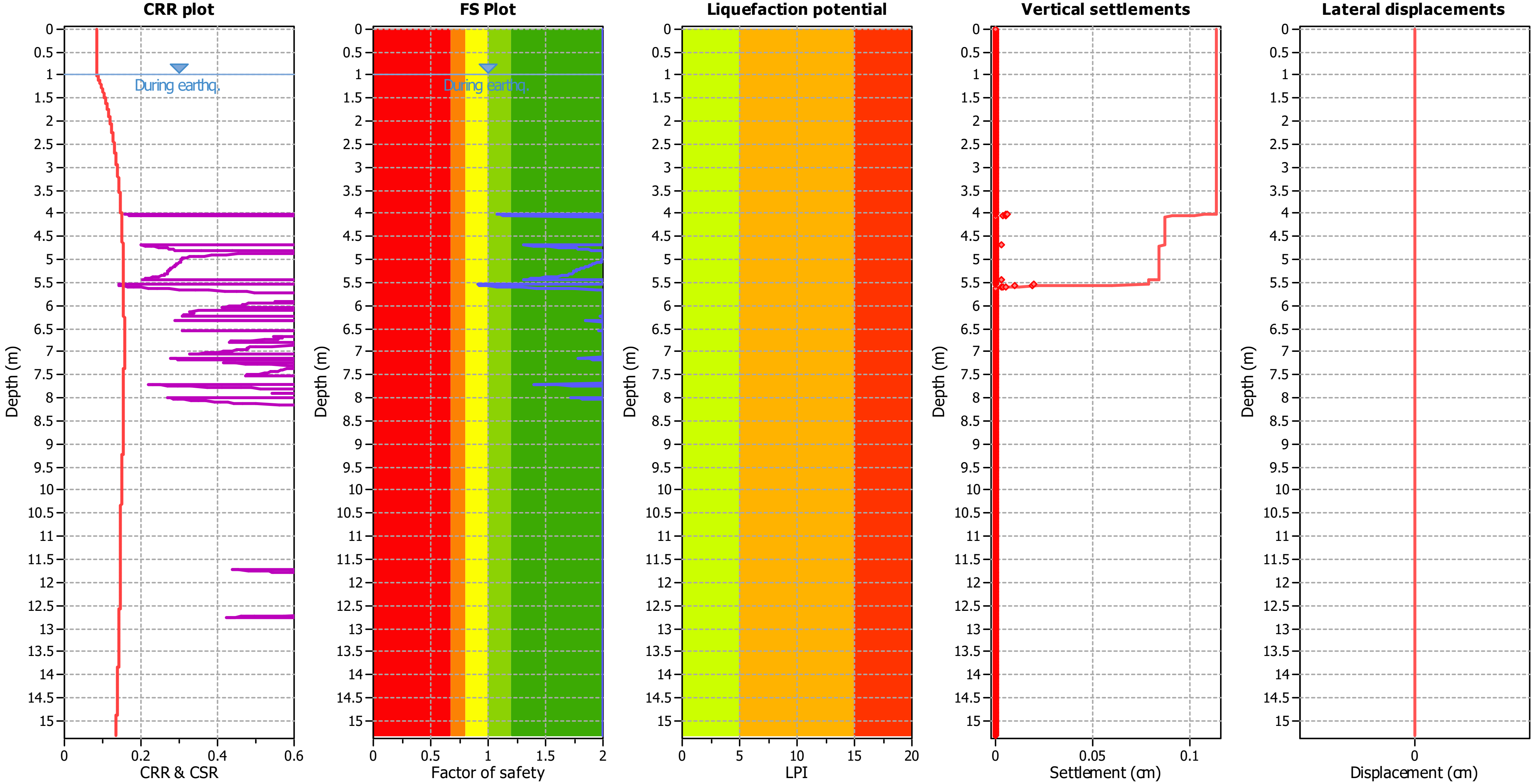
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.50
Peak ground acceleration: 0.19
Depth to water table (insitu): 1.00 m

Depth to water table (erthq.): 1.00 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: Sands only
Limit depth applied: No
Limit depth: N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

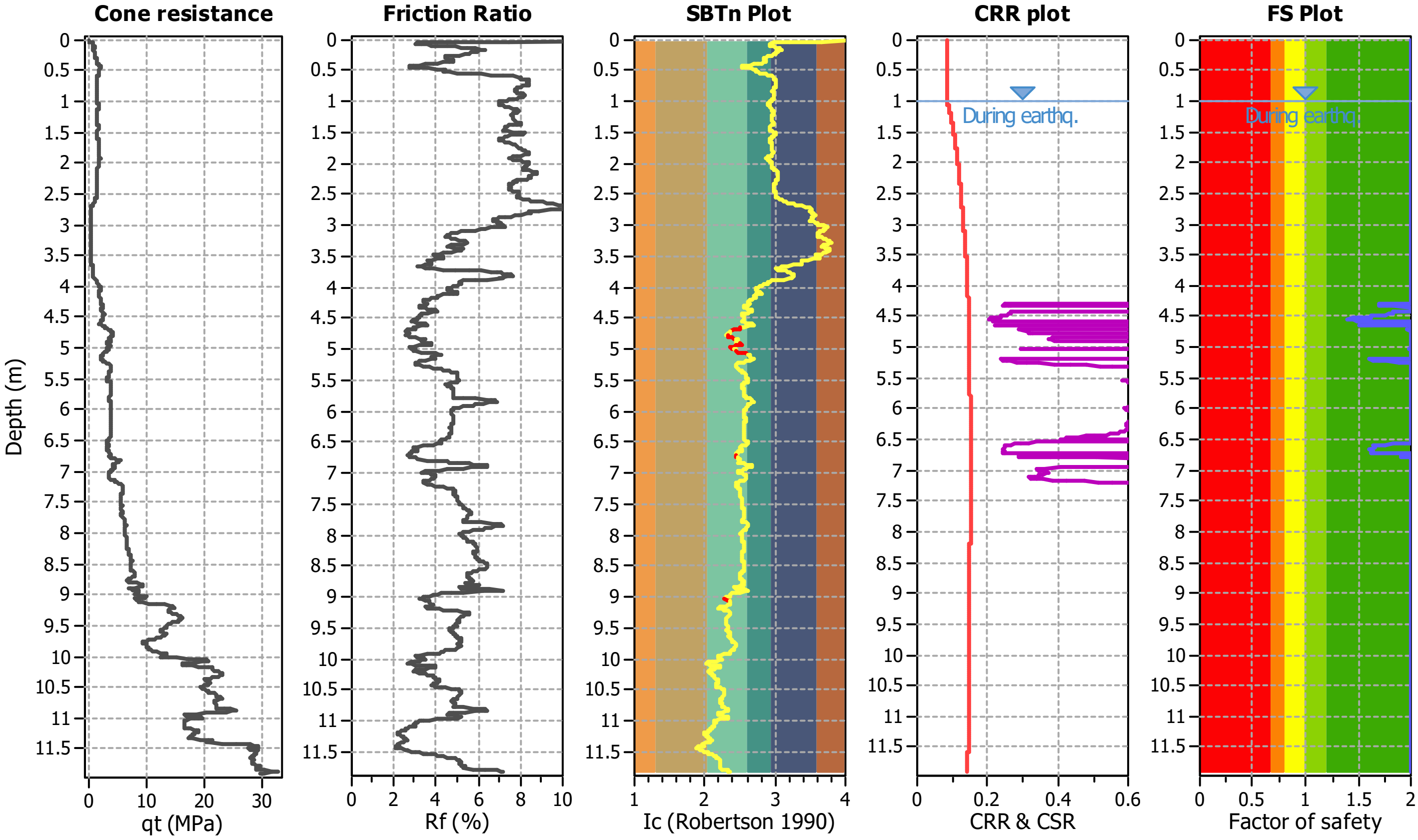
LIQUEFACTION ANALYSIS REPORT

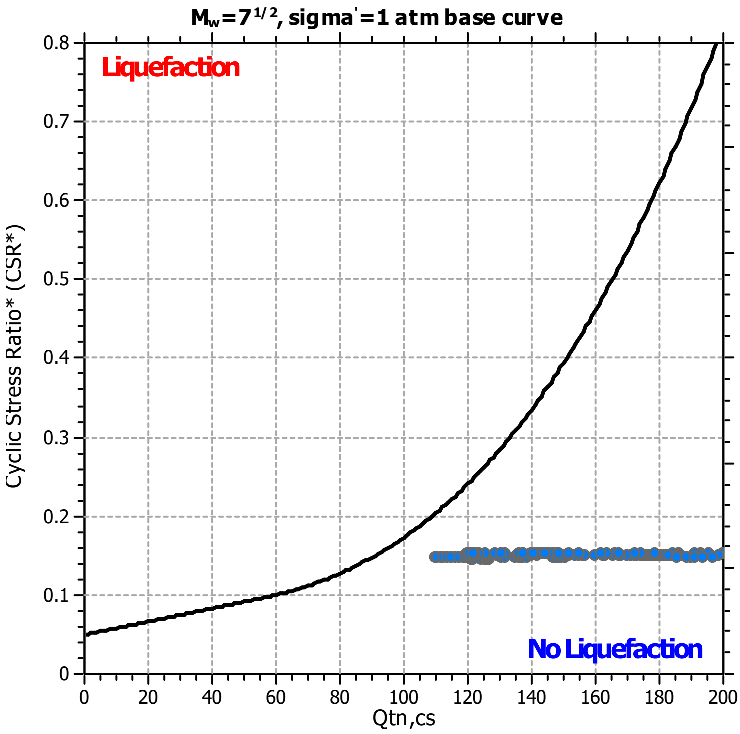
Project title : **Location :**

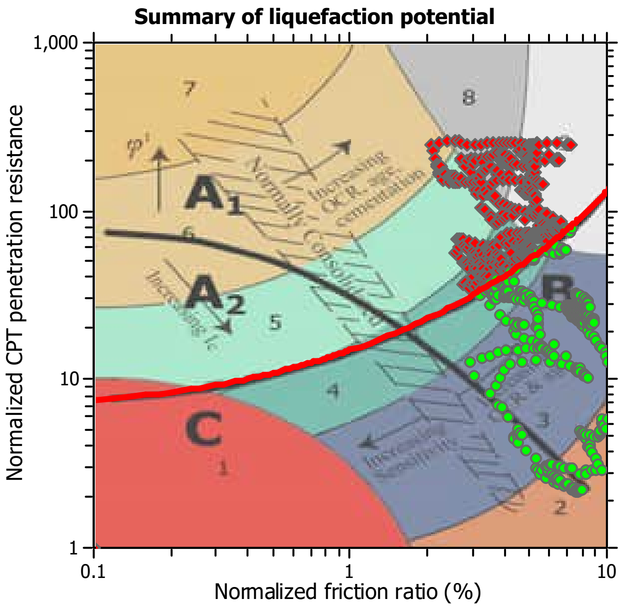
CPT file : CPT-04

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	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		

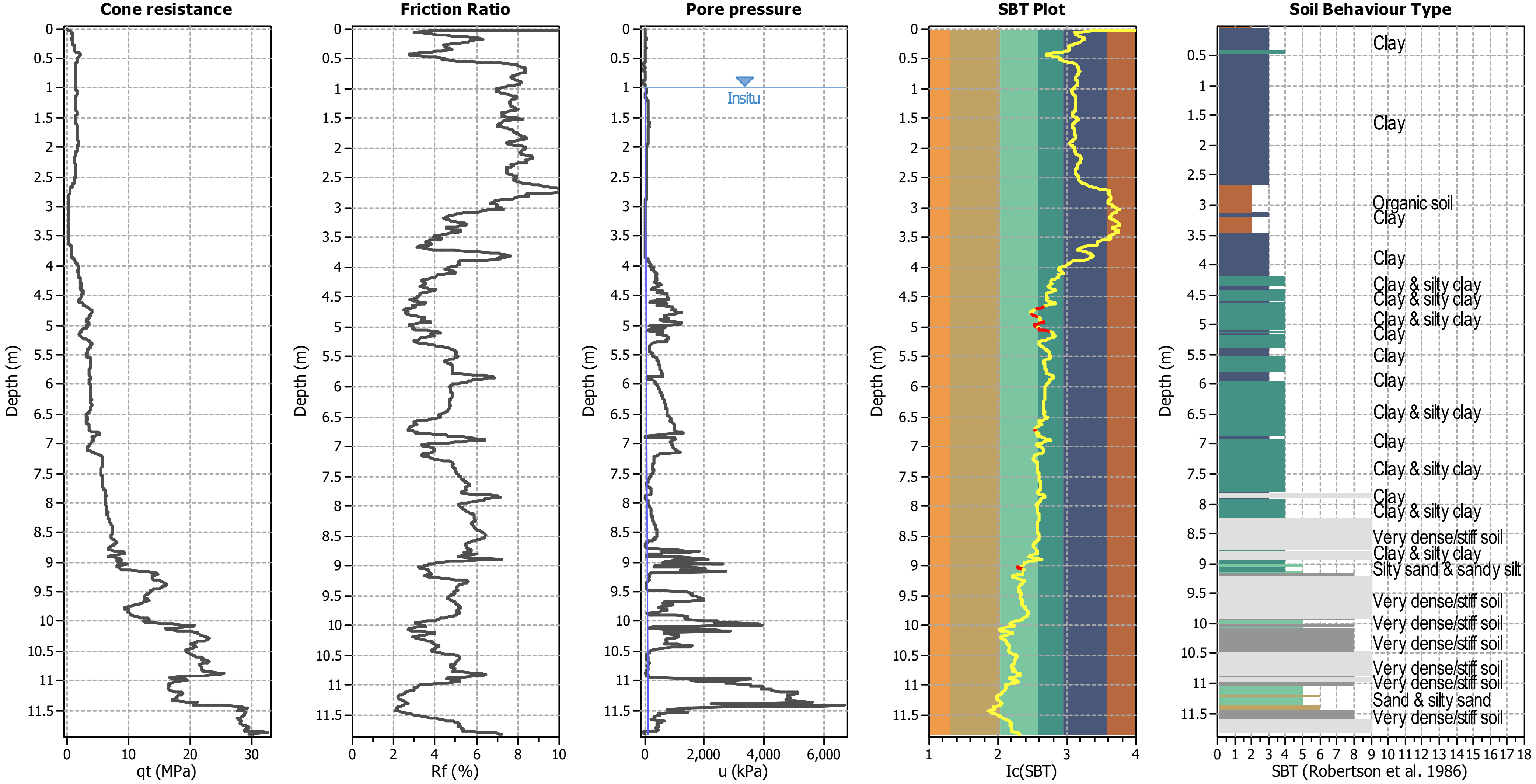

Cone resistance
Friction Ratio
SBTn Plot
CRR plot
FS Plot


$M_w=7^{1/2}$, 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

CPT basic interpretation plots



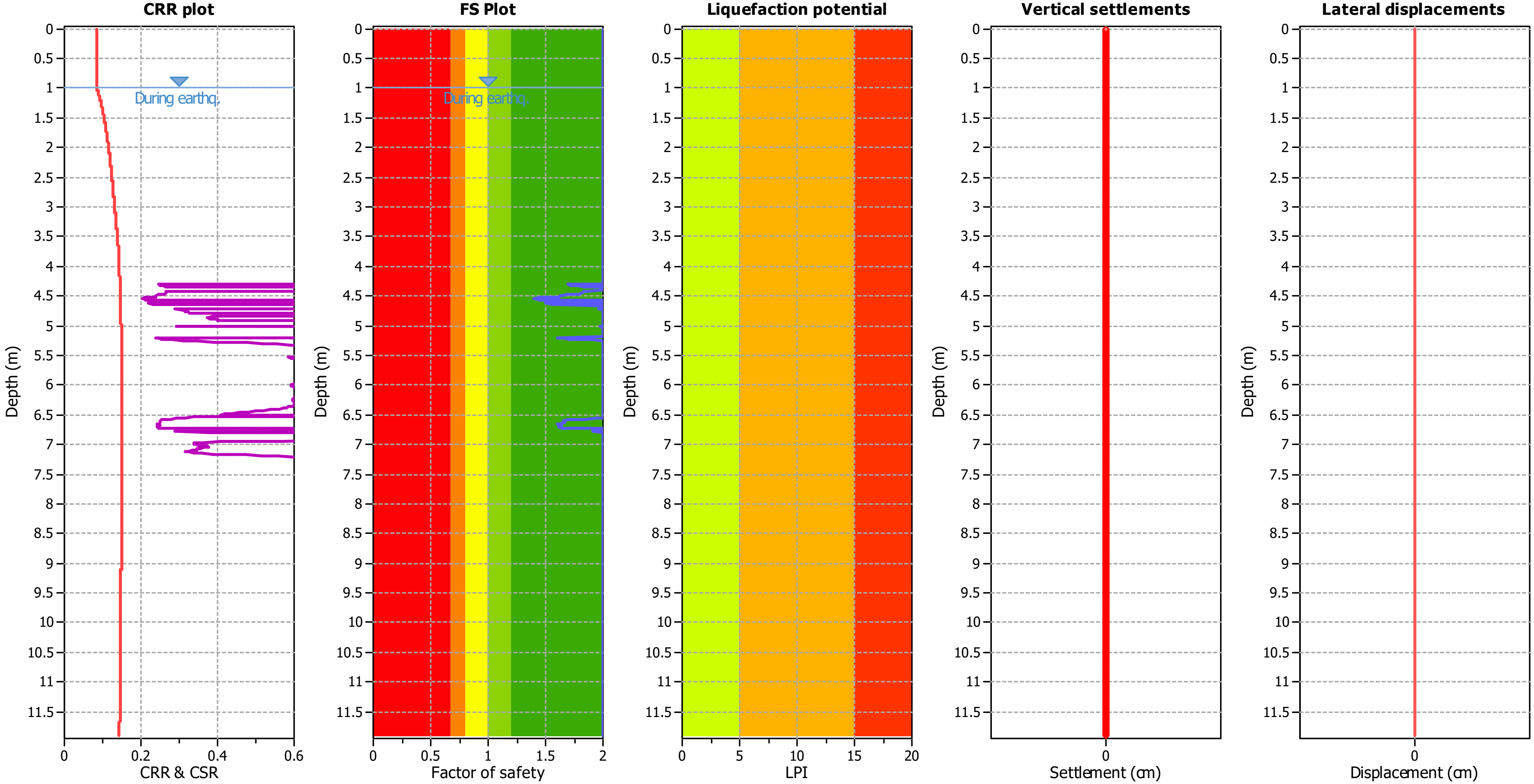
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

LIQUEFACTION ANALYSIS REPORT

Project title : **Location :**

CPT file : CPT-03

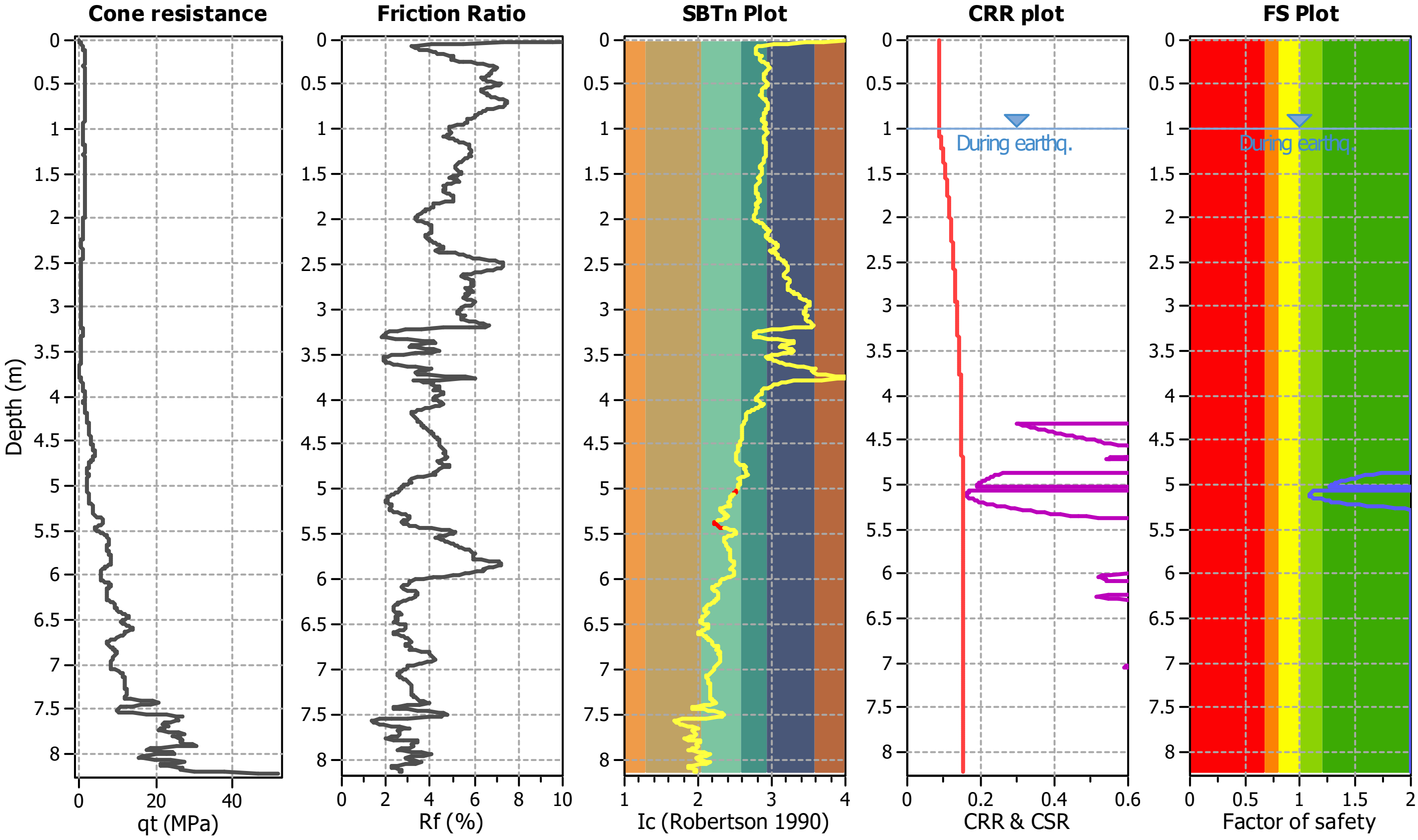
Input parameters and analysis data

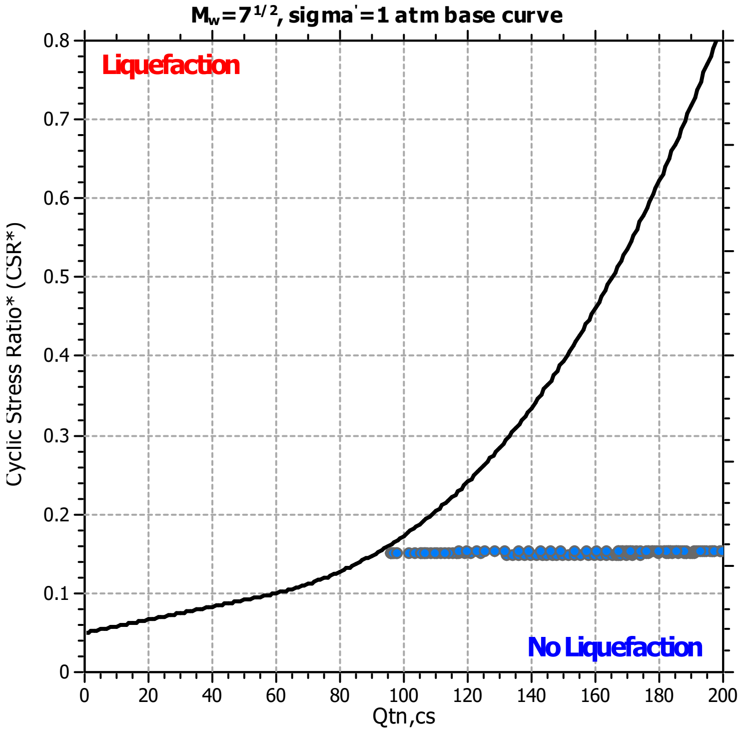
Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.50
Peak ground acceleration: 0.19

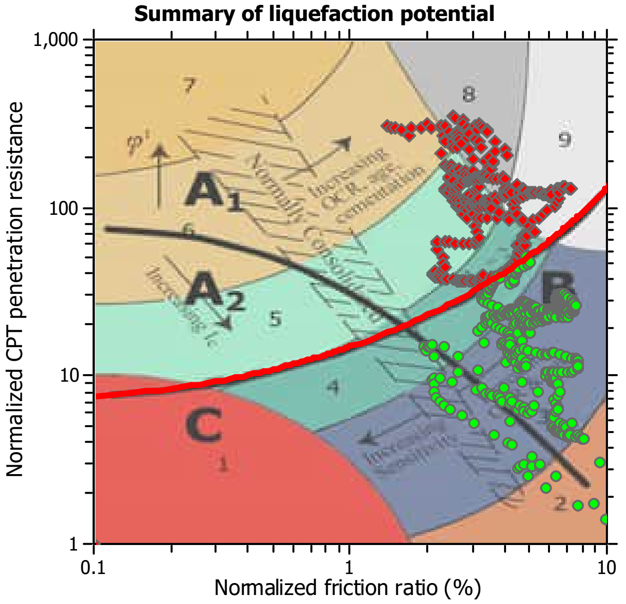
G.W.T. (in-situ): 1.00 m
G.W.T. (earthq.): 1.00 m
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT

Use fill: No
Fill height: N/A
Fill weight: N/A
Trans. detect. applied: Yes
K_σ applied: Yes

Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A
MSF method: Method based

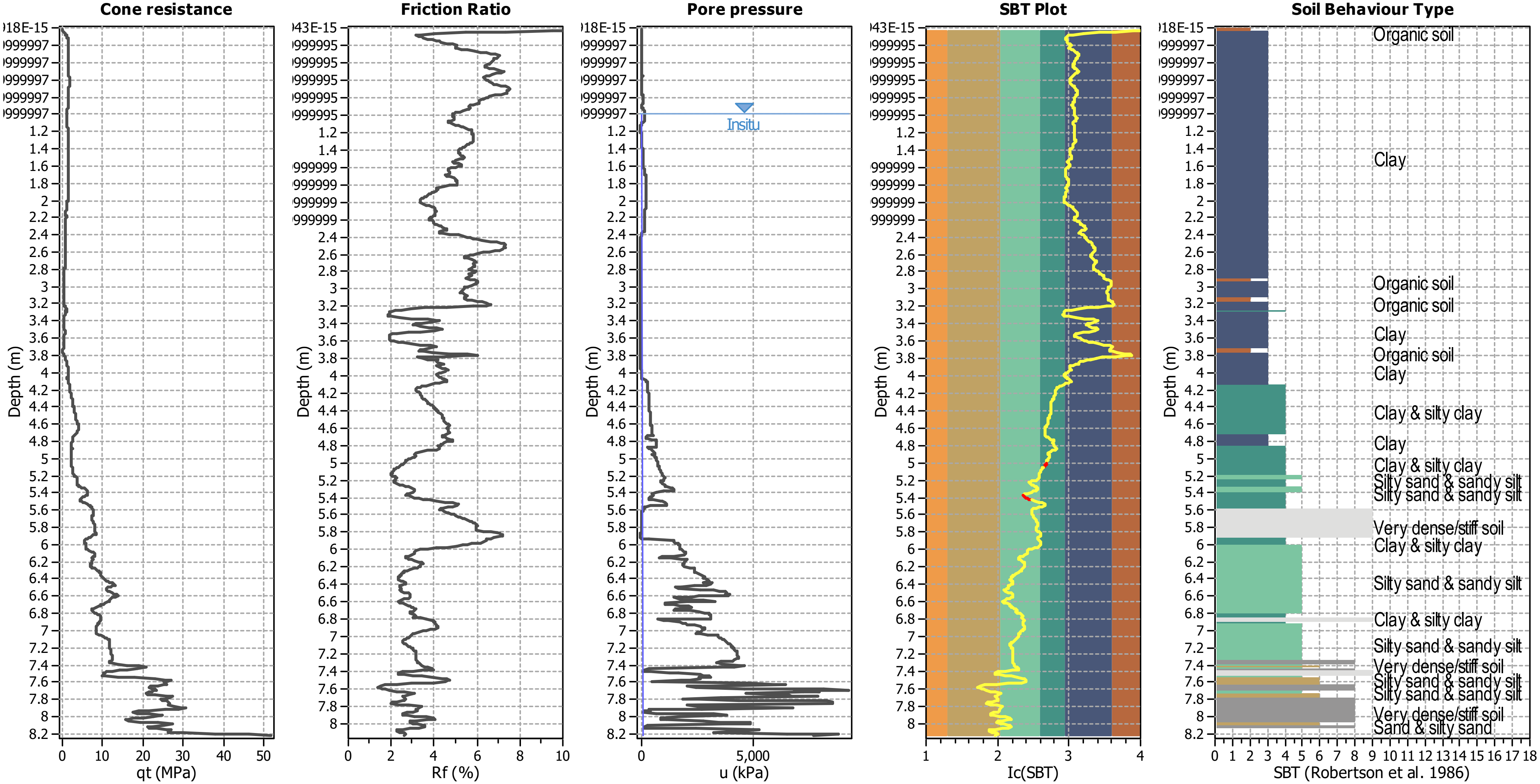






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



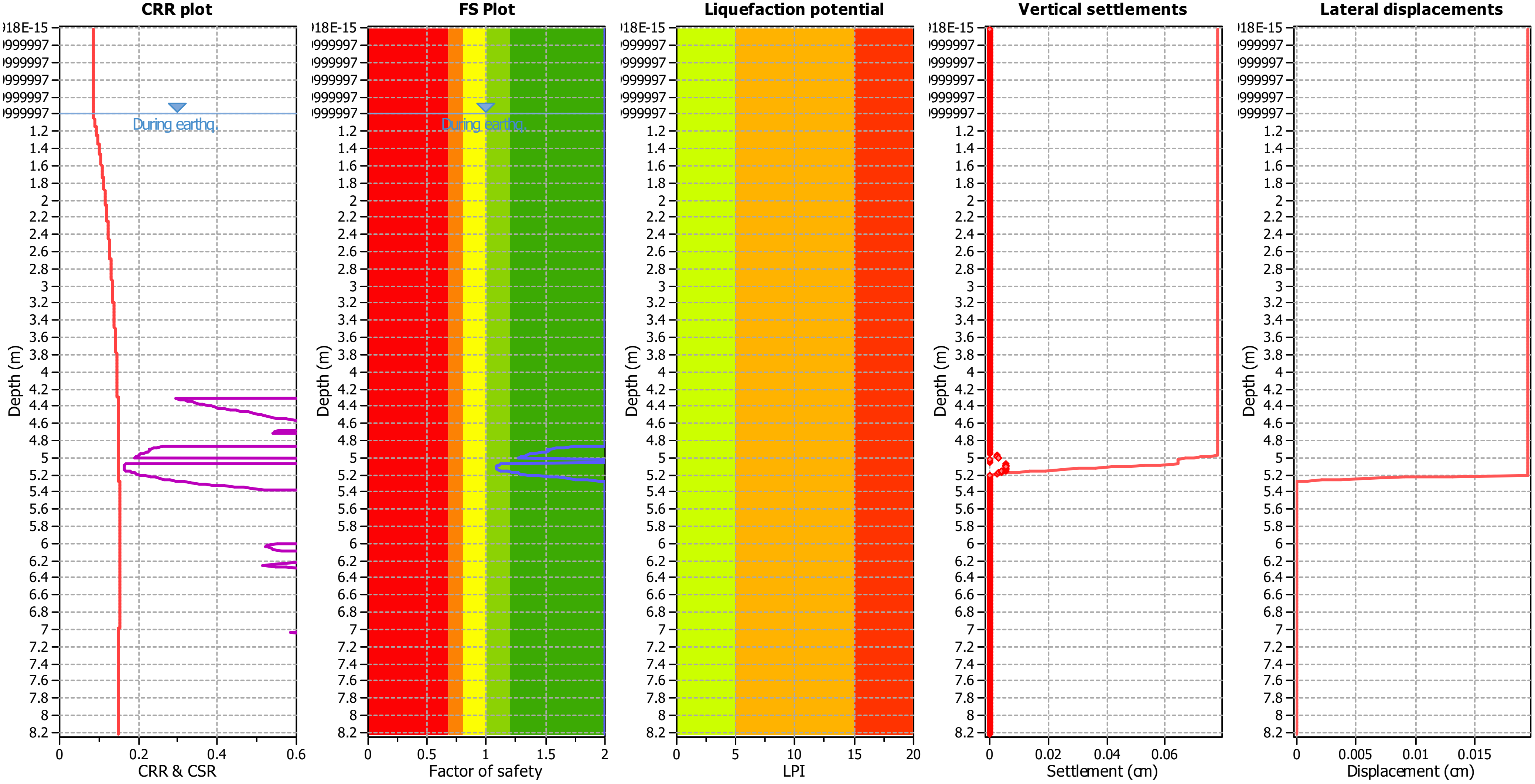
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	NCEER (1998)	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:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

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Fines correction method:	NCEER (1998)	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:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

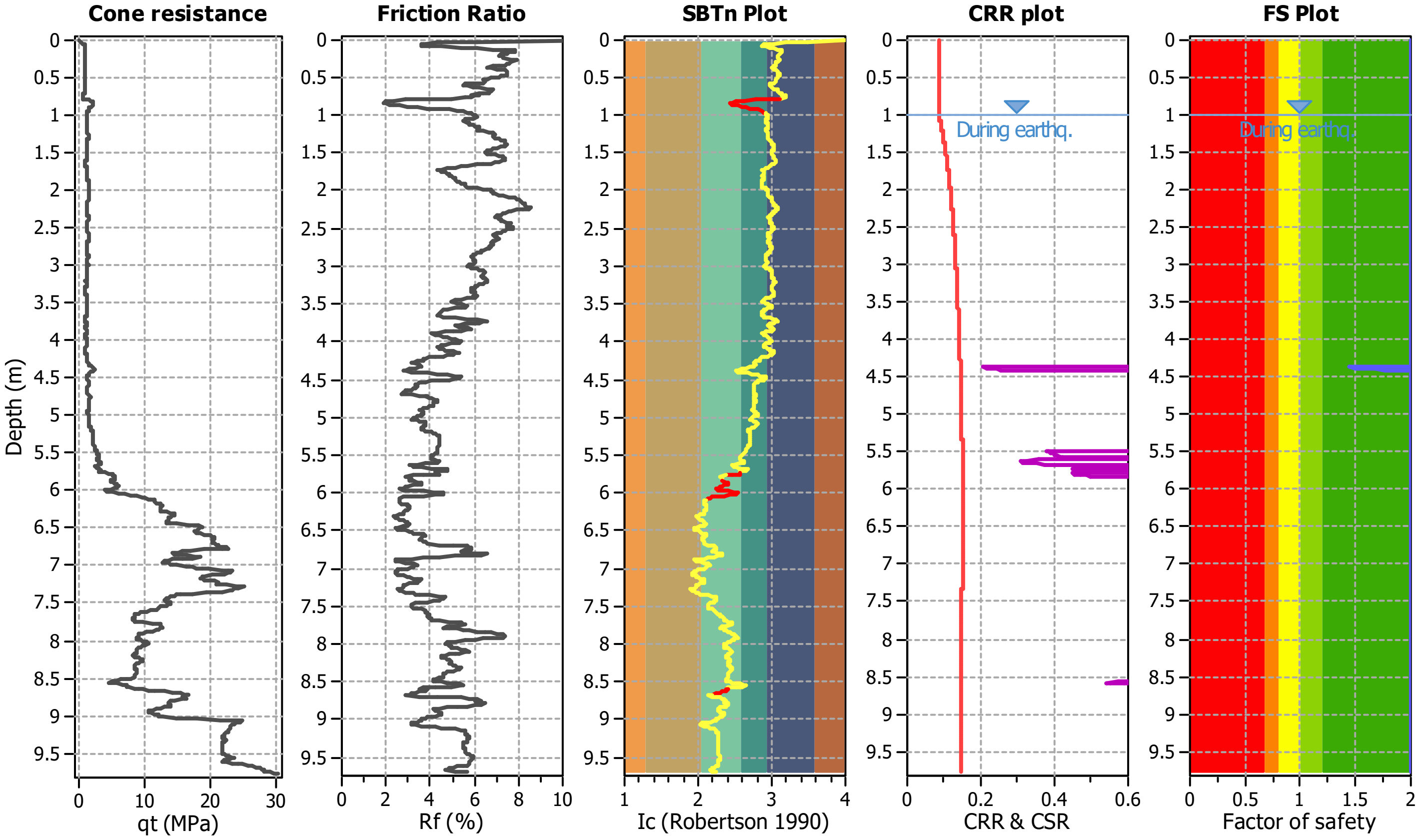
LIQUEFACTION ANALYSIS REPORT

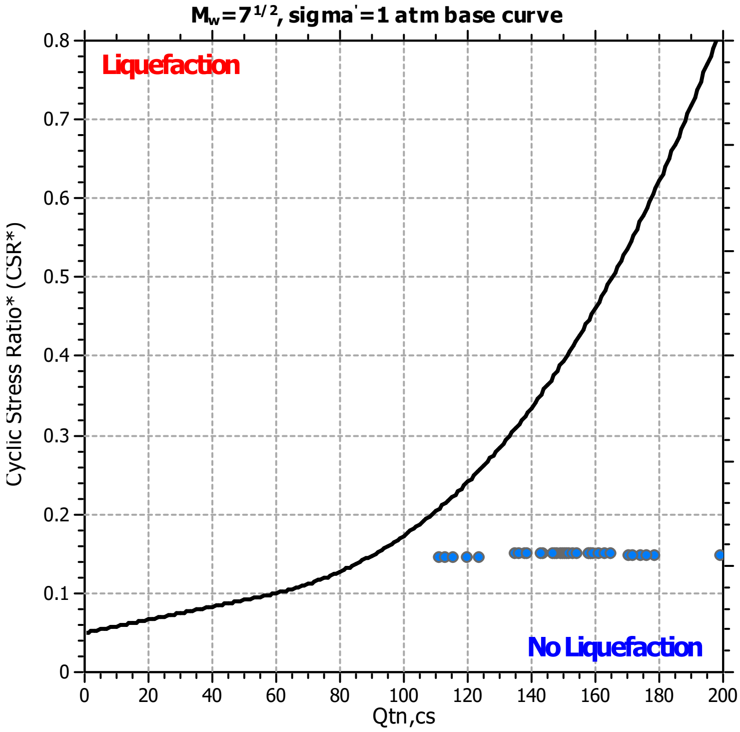
Project title : **Location :**

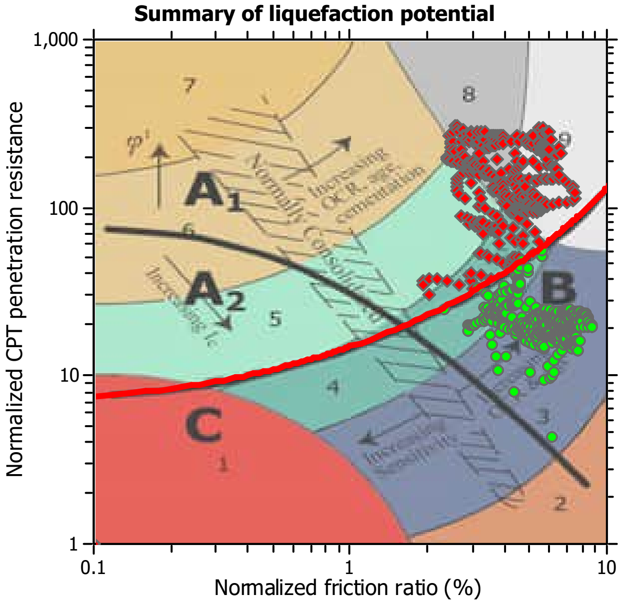
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	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		

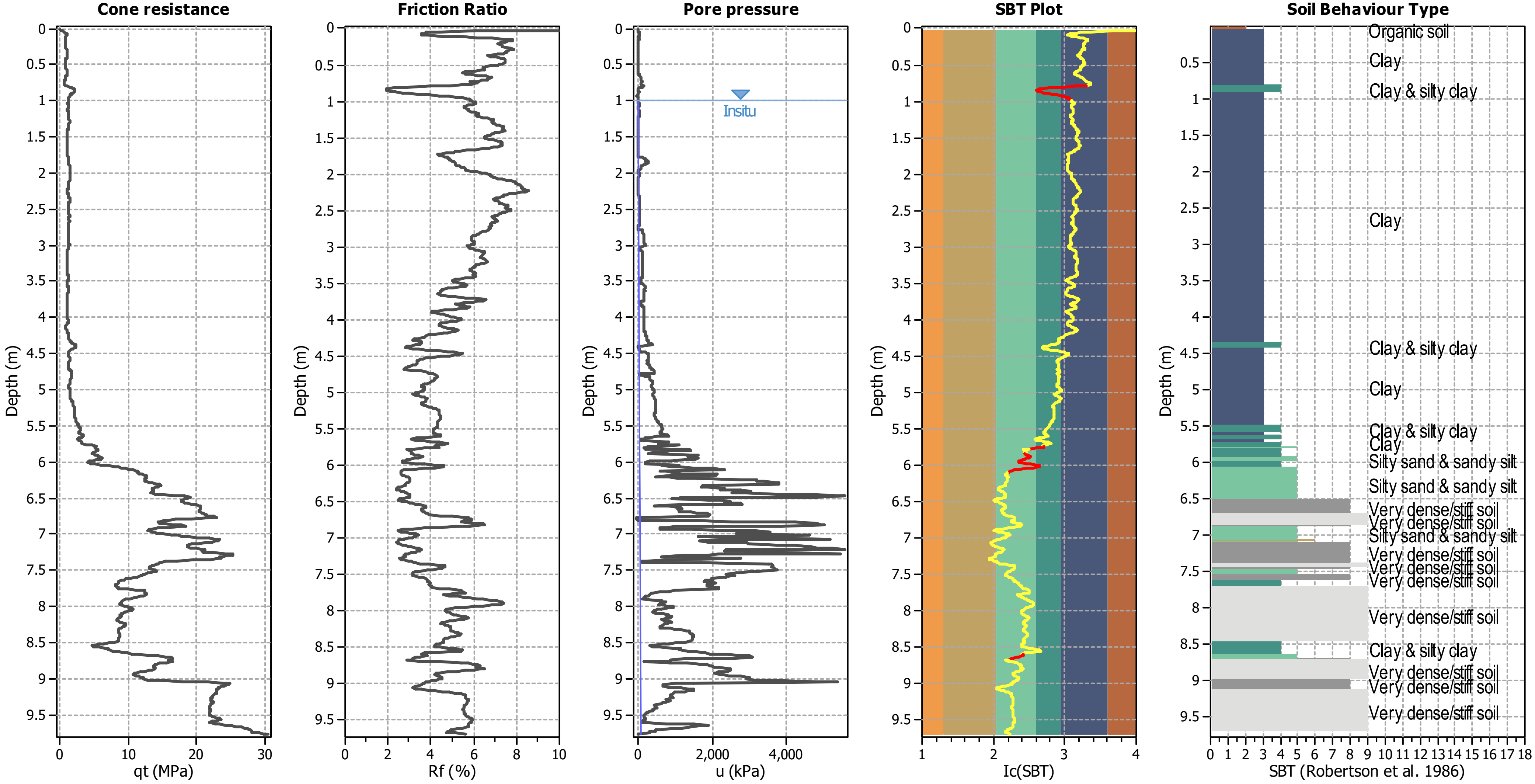






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



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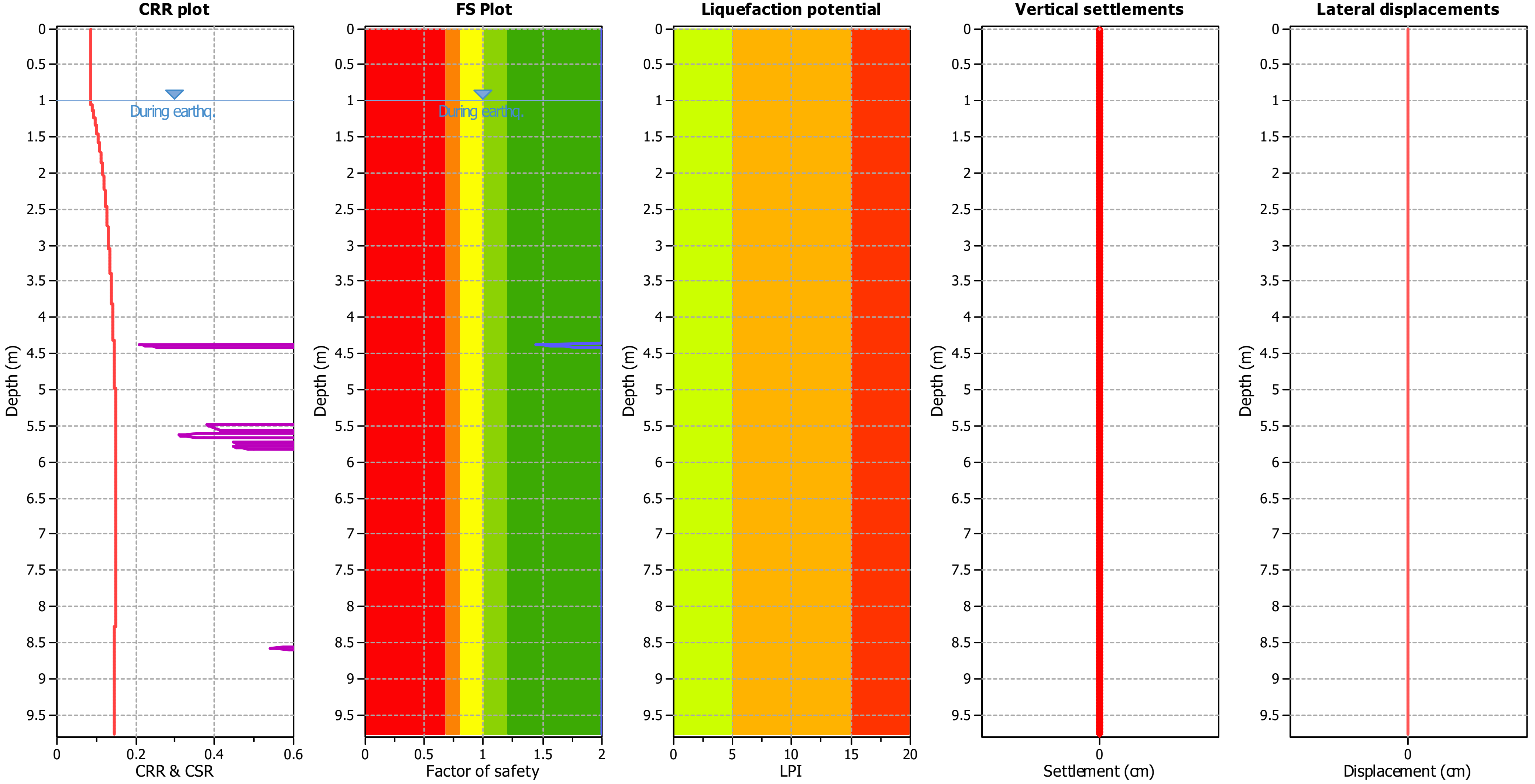
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

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Fines correction method:	NCEER (1998)	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:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

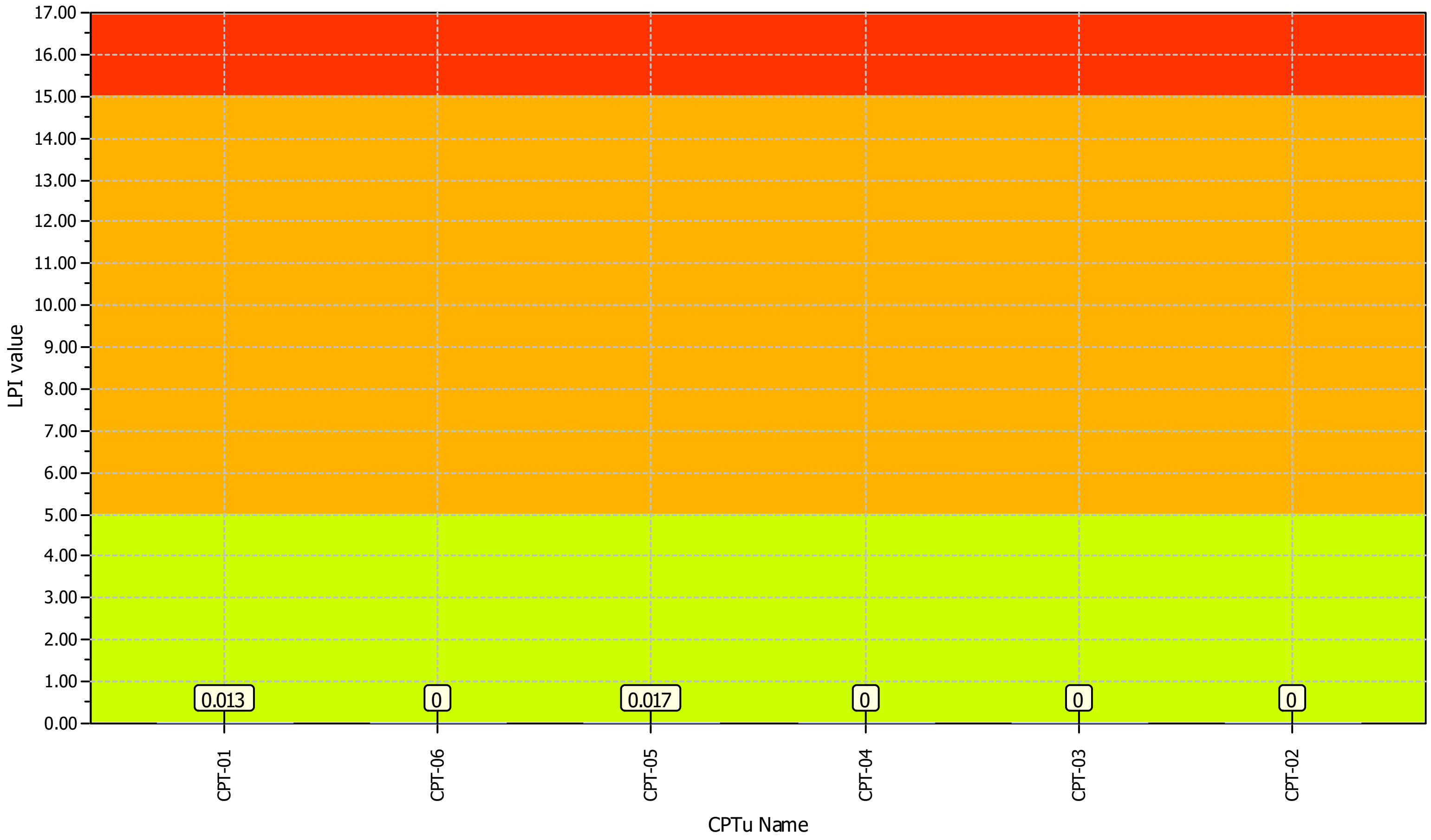
LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

Project title :

Location :

Overall Liquefaction Potential Index report



LPI color scheme

🟥 Very high risk
🟧 High risk
🟩 Low risk

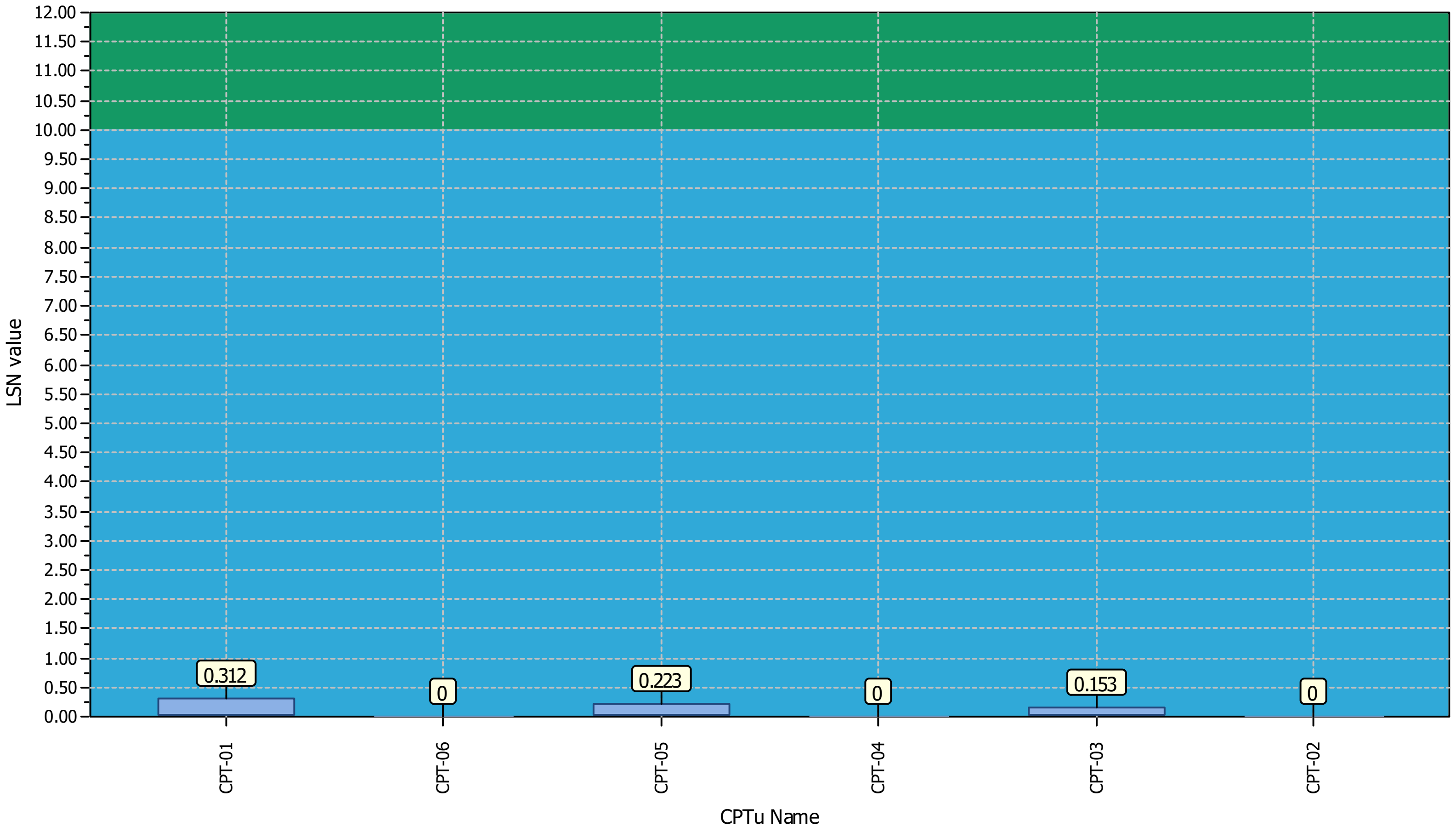
Basic statistics

Total CPT number: 6
100% low risk
0% high risk
0% very high risk

Project title :

Location :

Overall Liquefaction Severity Number report



LSN color scheme

- 🟥 Severe damage
- 🟥 Major expression of liquefaction
- 🟧 Moderate to severe exp. of liquefaction
- 🟨 Moderate expression of liquefaction
- 🟩 Minor expression of liquefaction
- 🟦 Little to no expression of liquefaction

Basic statistics

Total CPT number: 6

100% little liquefaction

0% minor liquefaction

0% moderate liquefaction

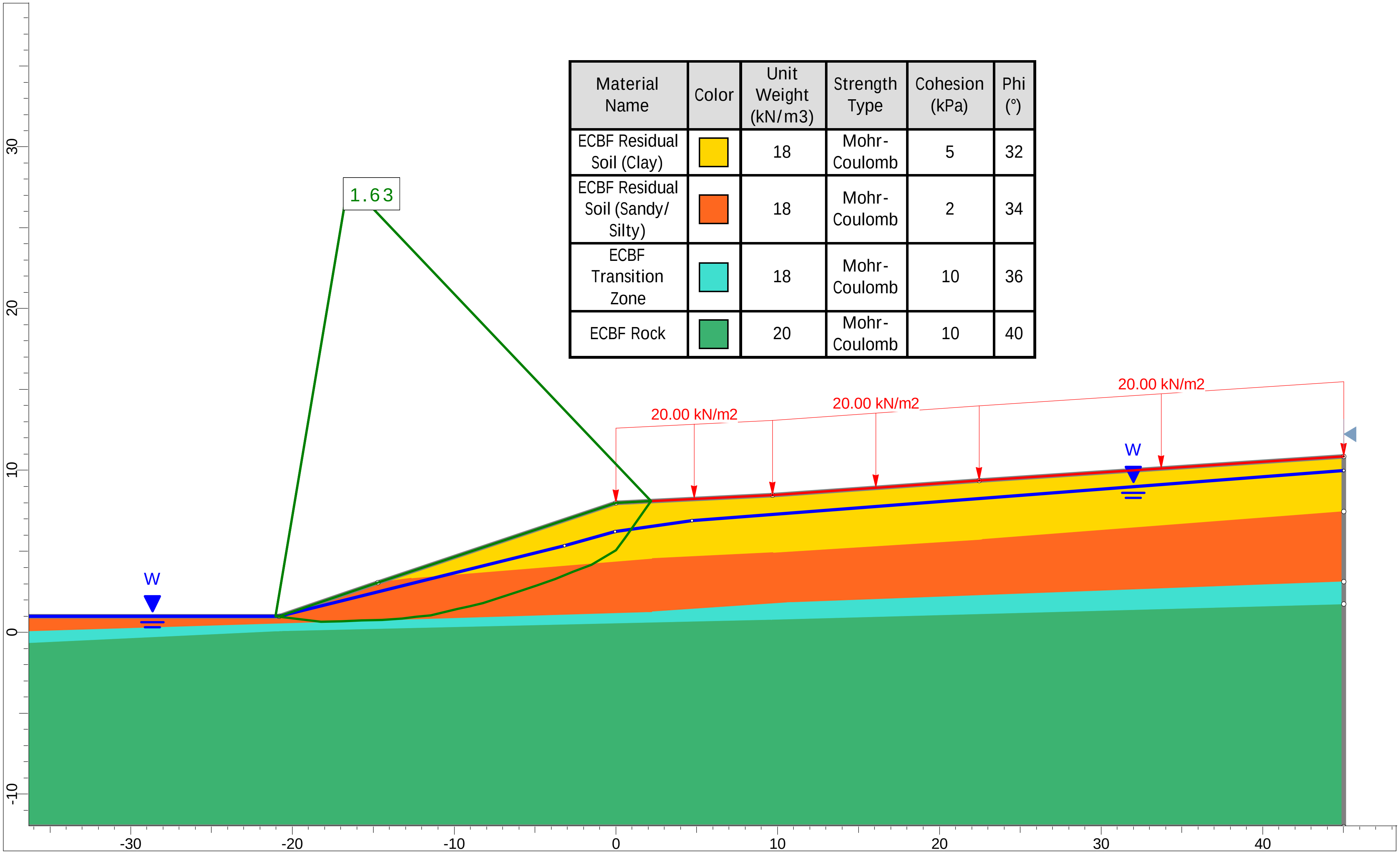
0% moderate to major liquefaction

0% major liquefaction

0% severe liquefaction

APPENDIX 6:

Slope Stability Analyses



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
ECBF Residual Soil (Clay)	🟨	18	Mohr-Coulomb	5	32
ECBF Residual Soil (Sandy/ Silty)	🟧	18	Mohr-Coulomb	2	34
ECBF Transition Zone	🟦	18	Mohr-Coulomb	10	36
ECBF Rock	🟩	20	Mohr-Coulomb	10	40

rocscience

SLIDEINTERPRET 9.030

Project 14 Sinton Road, Hobsonville

Group 14 Sinton Road

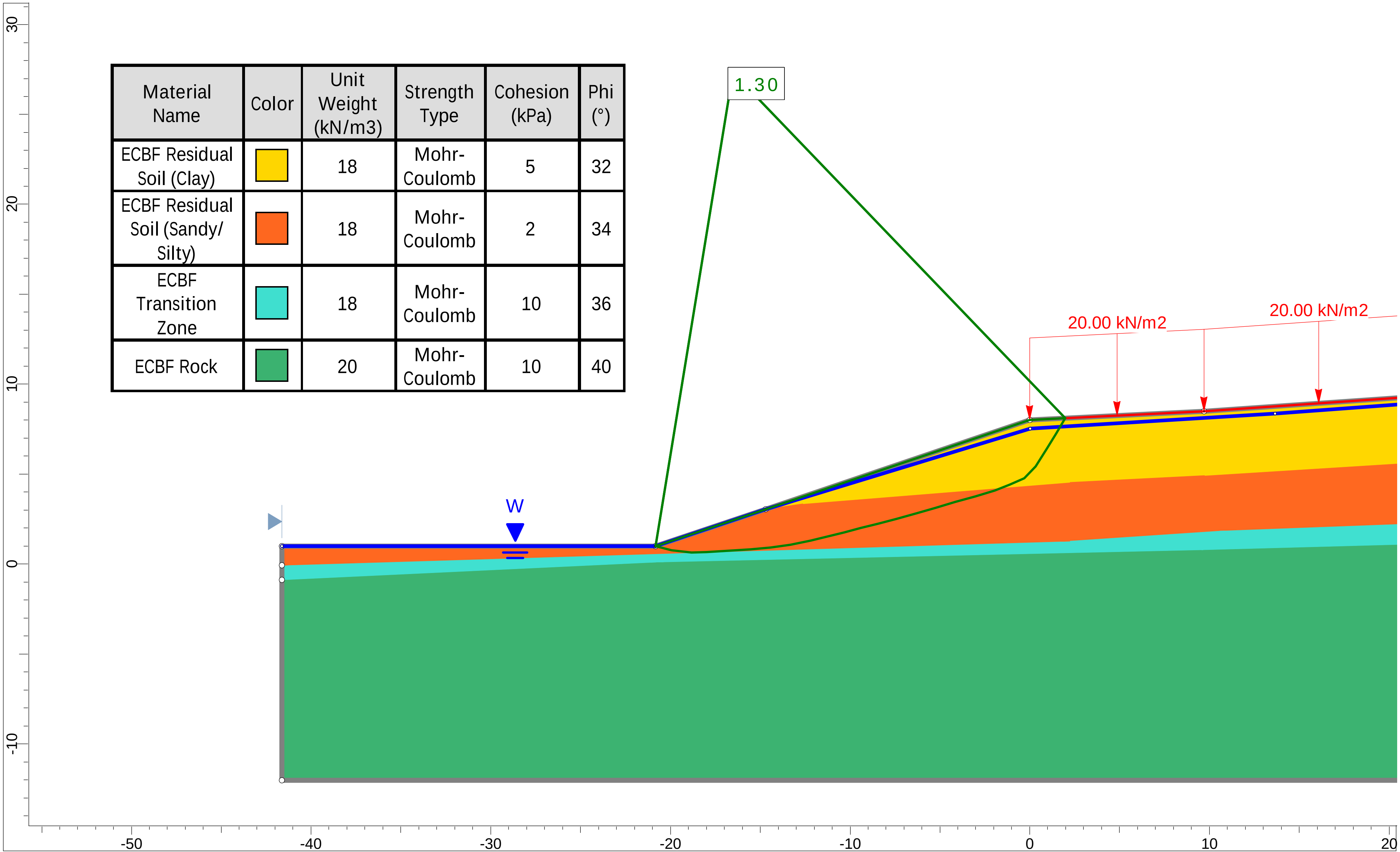
Scenario Long Term

Drawn By JT

Company ENGEO Ltd.

Date 1/25/2024, 9:13:39 AM

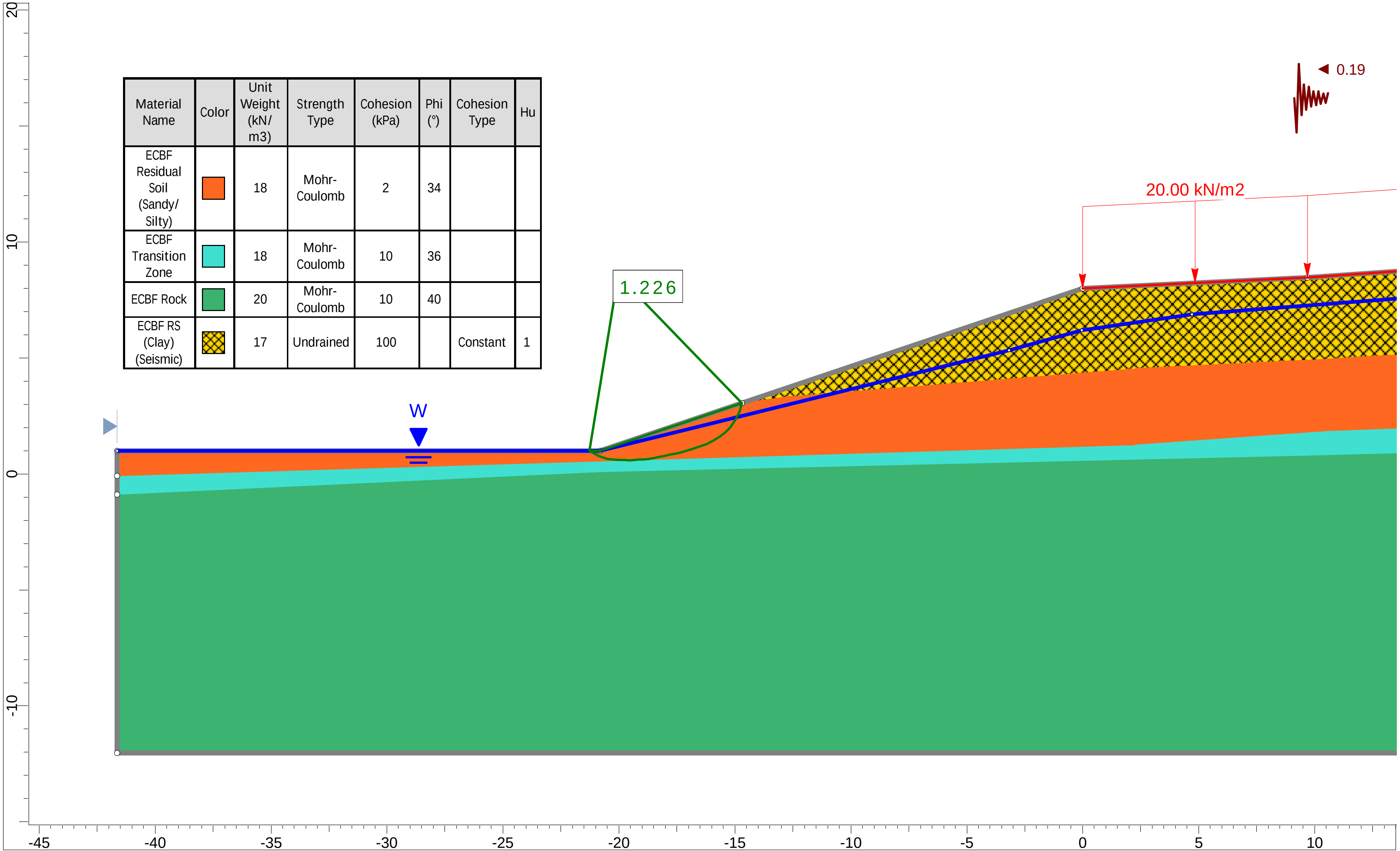
File Name Slope Stability JT.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
ECBF Residual Soil (Clay)	🟨	18	Mohr-Coulomb	5	32
ECBF Residual Soil (Sandy/ Silty)	🟧	18	Mohr-Coulomb	2	34
ECBF Transition Zone	🟦	18	Mohr-Coulomb	10	36
ECBF Rock	🟩	20	Mohr-Coulomb	10	40

SLIDEINTERPRET 9.030

Project	14 Sinton Road, Hobsonville		
Group	14 Sinton Road	*Scenario*	Elevated GW
Drawn By	JT	*Company*	ENGEO Ltd.
Date	1/25/2024, 9:13:39 AM	*File Name*	Slope Stability JT.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Cohesion Type	Hu
ECBF Residual Soil (Sandy/Silty)	🟧	18	Mohr-Coulomb	2	34		
ECBF Transition Zone	🟦	18	Mohr-Coulomb	10	36		
ECBF Rock	🟩	20	Mohr-Coulomb	10	40		
ECBF RS (Clay) (Seismic)	▨	17	Undrained	100		Constant	1

Project	14 Sinton Road, Hobsonville		
Group	14 Sinton Road	*Scenario*	Seismic
Drawn By	JT	*Company*	ENGEO Ltd.
Date	1/25/2024, 9:13:39 AM	*File Name*	Slope Stability JT.slmd

SLIDEINTERPRET 9.030

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Project Number 23849.000.003

Geotechnical Investigation

16 Sinton Road, Hobsonville, Auckland

Submitted to:
Cabra Developments Limited
Unit 9b, 30 Foundry Road
Silverdale
Auckland 0932

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Figures

Appendices

ENGEO Document Control:

Report Title	Geotechnical Investigation - 16 Sinton Road, Hobsonville		
Project No.	23849.000.003	**Doc ID**	04
Client	Cabra Developments Limited	**Client Contact**	Duncan Unsworth
Distribution (PDF)	Duncan Unsworth (Cabra) Tom Lemon (Capture Land Development Consultants)		

Date	Revision	Description	Author	Reviewer	WP
10/11/2023	0	Issued to Client	LM/JL	PF	JT
23/04/2024	1	Issued to Client	LM/JL	HL	JT

1 Introduction

ENGEO Limited was requested by Cabra Developments Limited to undertake a geotechnical investigation of the property at 16 Sinton Road, Hobsonville, Auckland (herein referred to as ‘the site’; shown in Figure 1). The purpose of this assessment was to support a Resource Consent application for the proposed redevelopment of the site. This work has been carried out in accordance with our signed agreement dated 31 July 2023.

We have been provided with an unnumbered draft masterplan of the site by Forme Planning Limited.

Our scope of works included:

- Desktop review of existing geotechnical reports and drawings for the site, a review of publicly available geological and geotechnical data and aerial photographs.
- Undertake a site walkover to assess current site conditions and observe for geomorphological evidence of land disturbance, active and historical slope instability.
- Drill up to eight hand auger boreholes to a target depth of 5.0 m below ground level (bgl) with associated strength tests across the site to provide geotechnical data on the shallow soil profile.
- Undertake two Cone Penetration Tests (CPTs) to target depths of 15.0 m bgl to support a liquefaction assessment for the alluvial soils.
- Recover a representative soil sample from near surface soils for laboratory expansive soils classification testing.
- Undertake a liquefaction assessment using CPT data.
- Preparation of this Geotechnical Investigation Report presenting the findings of our investigation and geohazard assessments to support the Resource Consent application.

To support a Resource Consent application this report is required to reflect the earthworks proposals, particularly with respect to the slope stability assessment, which have not yet been developed. A supplementary assessment will be required to address the development proposals when available.

2 Site Description

The site comprises a 2.8758 ha parcel of joint residential and pastoral land legally described as LOT 9 DP 57408, located on an elevated coastal terrace bordered to the north by a narrow tidal creek; the Waiarohia Inlet. The site is accessed via a private driveway directly off Sinton Road in the eastern corner of the site. The site is comprised of grassed fields with hedging and contains a residential dwelling and sheds. Site features are shown on the plan in Figure 1.

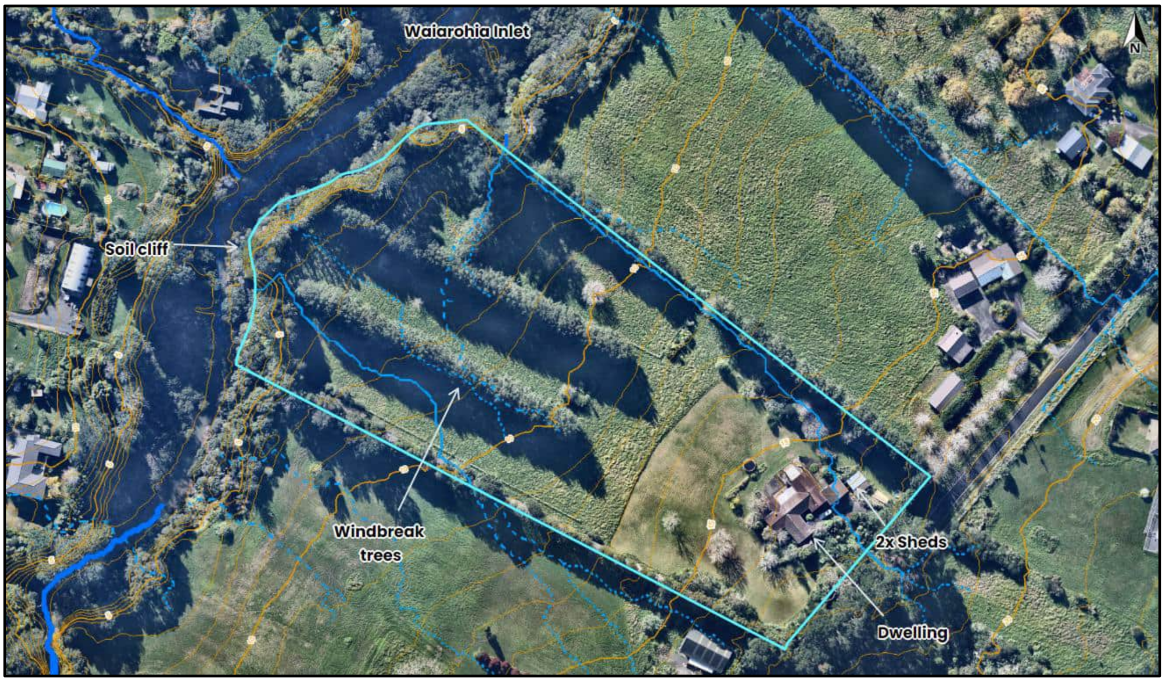
The residential dwelling is located within the eastern corner of the site along with two garages, other storage type structures and minor vegetation. The western portion of the site contains landscaped grassed fields and hedges with three rows of windbreak trees in the western half of the site. The site is bound by Sinton Road to the southeast, lifestyle blocks and residential dwellings to the north and south. The northwest of the site is bordered by a tree lined 6 m high soil cliff, where the terrace landform falls towards the Waiarohia Inlet.

There are multiple overland flow paths that cross the site, draining towards the Waiarohia Inlet. There is a flow path that runs along the northern boundary of the site and cuts through the residential property in the eastern corner. Two other flow paths originate from the centre of the site, one running north and linking up with the northern boundary flow path, the other running northwest to the Waiarohia Inlet. The other flow path runs from the centre of the southern boundary towards the northwest boundary. Mapped contours indicate that small erosional channels have formed on the edge of the soil cliff where the overland flow paths drain into the inlet.

There are no existing public services that run through the site. There is a private septic tank located to the northwest of the residential property.

The site falls from Sinton Road in the east down to the north-western boundary as a gentle slope of approximately three degrees. Minor changes in elevation can be noted along the alignment of the overland flow paths throughout the site. A site plan is presented in Appendix 1.

Figure 1: Site Features Plan



NTS. Aerial imagery from Nearmaps. Site boundary shown in light blue. Overland flow paths shown in dark blue. Contours shown in orange.

3 Proposed Development

We have been provided with an excerpt of a draft masterplan of the proposed development by Forme Planning Limited which shows that the development will comprise an 82 Lot residential subdivision. The proposed lots are to be a range of typologies.

Lots along the north-western boundary will contain detached dwellings while the remainder of the lots on-site will be occupied by terraced town houses. An approximately 20 m wide esplanade will separate the large north-western lots from the soil cliff and Waiarohia Inlet. It is not apparent from the masterplan whether the location of the esplanade reserve is measured in relation to the soil cliff that forms the north-western boundary or in relation to another datum. In places the masterplan shows the edge of the esplanade falling at both the toe and crest of the slope when overlayed on LINZ contour data.

No earthworks plans or proposed contours have been provided to ENGEO at the time of writing. A modified copy of the masterplan is presented in Figure 2.

Figure 2: Lot Layout

NTS. Imagery from LINZ and modified from Forme developments ltd masterplan.

4 Desktop Study

4.1 Published Geology

The site is regionally mapped (1:250,000) by GNS Science[1] as spanning the geological boundary between the East Coast Bays Formation to the north and the Puketoka Formation of the Tauranga Group Alluvium.

Puketoka Formation soils typically comprise pumiceous mud, sand and gravel with muddy peat and lignite. The alluvial nature of the soils means that it may commonly include sediment from a range of eroded sources and reworked material from underlying stratigraphic units including the East Coast Bays Formation.

---

[1] https://data.gns.cri.nz/geology/

East Coast Bays Formation comprises of alternating sandstone and mudstone with variable volcanic content and interbedded volcaniclastic grits. East Coast Bays Formation residual soils are generally described as orange and grey silts and clays with varying sand contents.

The boundary between these two units is mapped as inferred and plotted within the south-eastern portion of the site close to the end of the shelter belt trees. Based on the scale of the regional mapping and the inferred nature of the contact it should be considered that the mapped boundary may not reflect the exact location of the geological contact within the site.

4.2 Historical Aerial Photography Review

Aerial photographs of the site dating from 1940 to 2023 have been accessed from Auckland Council GeoMaps, Retrolens, Nearmaps and Google Earth Pro and these photos have been reviewed under the context of understanding past site use and to identify evidence of historical landform modifications. Table 1 provides a summary of our review findings. Aerial images are presented in Appendix 2.

Table 1: Summary of Historical Aerial Photographs

Date	Description
1940	The site and surrounding area comprise agricultural land; the site itself appears to be used for grazing. The northwest end of the site is vegetated and forms the edge of the Waiarohia Inlet. A darker area in the eastern corner of the site may represent a change in elevation.
1950	With the exception of a small area of bare ground in the eastern corner of the site, no significant changes to the site or surrounding area are noted.
1959	Some of the vegetation at the northwest end of the site has been cleared. A drainage channel appears to be located along the darker area previously observed in the eastern corner of the site. No significant changes to the surrounding area are observed.
1963	No significant changes to the site are observed. Horticultural activity is observed on land on the opposite side of the inlet.
1968	Image quality is too poor to assess details, however the site appears to have been separated into paddocks. Neighbouring land to the north has an area which has been separated into smaller plots which may represent cropping.
1972	The site and surrounding area comprise agricultural land (appears to be used for grazing). The potential crops observed previously are not identified.
1975	A building has been constructed at the southeast end of the site, more or less in the same position as the existing dwelling (but smaller in size). A brighter area in the northern corner may represent bare ground. Buildings (likely residential) have been construction on land to the north.

Date	Description
1978	A small rectangular feature is observed to the northeast of the building on-site; based on the shadow is not likely to be tall enough to be a building. Three square, fenced off areas are noted at the centre of the site. A line of shadow along the northwest extent indicates that land drops steeply down towards the inlet.
1988	The majority of the site is subject to horticultural activity, and areas of crop appear to be separated by shelterbelts. The building in the southeast has been extended and two additional smaller buildings constructed in the vicinity. The smaller feature to the northeast is no longer observed. Land to the north and southeast is subject to horticultural activity.
1996	The shelterbelts remain on-site; however, the land appears to be grassed again. Additional extension of the building in the southeast may have occurred, and potentially an additional smaller building to the northeast; however, details are not clear due to a poor-quality image. Horticultural activity on surrounding land may also have ceased.
2000	A small building has been constructed to the northeast of the main building, on the northern boundary. No significant changes are observed across the balance of the site. No significant changes to the surrounding area are observed.
2006	The main building has been extended and is more or less the size and shape currently observed. A small shed / garage has been constructed to the east of the main building. A small garden plot has been planted to the north of the building. A large building has been constructed on land to the south of the site, no other significant changes to the surrounding area are observed.
2008	The square fenced areas at the centre of the site appear to have been planted. A grassed circular feature is observed to the south of the main building. Land to the north has been developed, or currently under construction.
2017	With the exception of the circular feature now comprising bare ground, no other significant observations are noted. No significant changes to the surrounding area are observed.
2020	The small building constructed in 2000 has been demolished, building debris is observed in the footprint. No significant changes to the surrounding area are observed.
2023	No significant changes to the site or surrounding area are observed.

No significant earthworks or landscape modification and no major slope instability is evident from these photos.

4.3 New Zealand Geotechnical Database

The New Zealand Geotechnical Database (NZGD) reveals that there have been a handful of past intrusive investigations in the vicinity of this site. These investigations comprise:

- Four hand auger boreholes along Sinton Road with the nearest two adjacent to the southern and eastern corner of the site extending approximately 200 m up the road (NZGD ID: HA_96977, HA_96979, HA_96981, HA_96984):
    - These boreholes were drilled by Maunsell Ltd in November 2005 to depths of 5.0 m bgl.
    - Materials encountered:
        - Topsoil between 0.0 and 0.3 m bgl.
        - Puketoka Formation clays and silts with varying sand contents between 0.0 and 5.0 m bgl, with measured shear strengths between 53 and 185+ kPa.
        - East Coast Bays Formation silts and sands with varying clay contents between 3.4 and 5.0 m bgl, with measured shear strengths between 99 and 185+ kPa.
- Four machine boreholes along the Upper Harbour Motorway located approximately 200 m south of the site (NZGD ID: BH_205768, BH_205769, BH_205770, BH_205772).
    - These boreholes were drilled by Tonkin & Taylor between April and June 2021 to depths between 30.0 and 30.1 m bgl.
    - Materials encountered:
        - Topsoil between 0.0 and 0.9 m bgl.
        - Silt, clay and gravel fill between 0.0 and 3.0 m bgl.
        - Puketoka Formation clays and silts with varying sand contents between 0.9 and 10.08 m bgl, with measured shear strengths between 40 and 107 kPa.
        - East Coast Bays Formation silts and sands with varying clay contents between 4.45 and 13.13 m bgl, with measured shear strengths between 33 and 149 kPa.
        - East Coast Bays Formation sandstone and siltstone between 8.25 and 30.01 m bgl, with measured N values of 17 and 50+ kPa.

4.4 Auckland Council GeoMaps

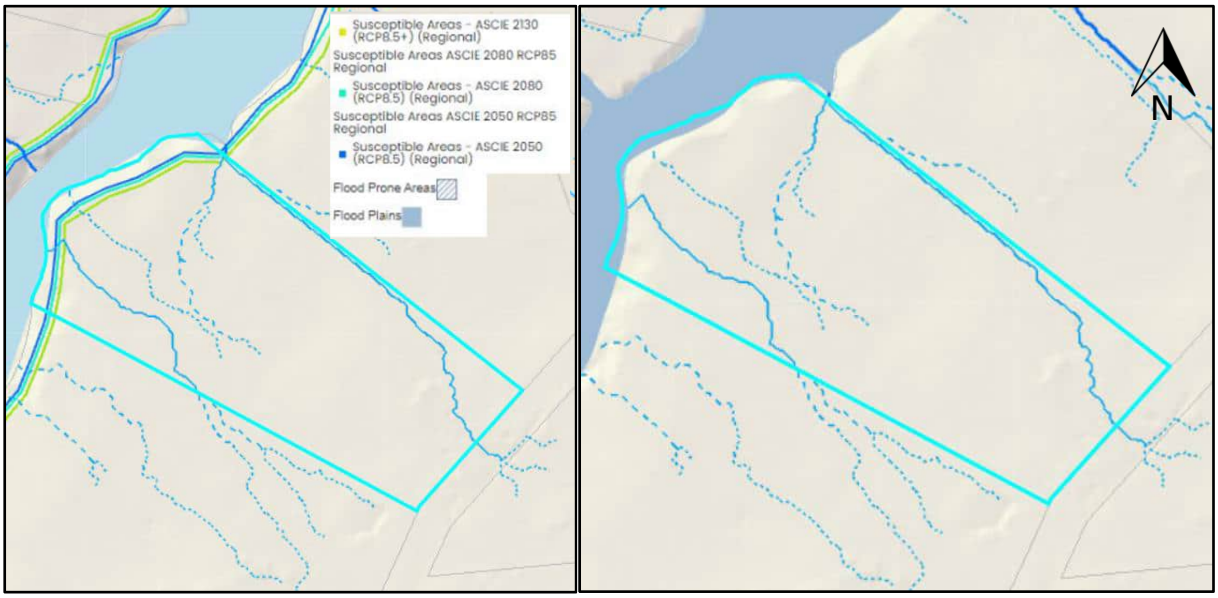
4.4.1 Coastal Instability and Erosion

The Auckland Council GeoMaps layer ‘Areas Susceptible to Coastal Instability and Erosion’ identifies areas of coastline in Auckland that could be affected by coastal erosion and instability under a range of climate change scenarios and timeframes. The potential regression lines for 2050, 2080 and 2130 for this site are shown in Figure 3. These areas are limited to the northern slopes, along the Waiarohia Inlet.

4.4.2 Flood Plains & Prone Areas

The Auckland Council GeoMaps layer ‘Flood Plains & Flood Prone Areas’ identifies areas of land in Auckland that could be affected by flooding during and / or following periods of heavy rain. Portions of the site labelled as flood prone or flood plains are shown in Figure 3 and are limited to areas adjacent to Waiarohia Inlet.

Figure 3: Auckland Council Hazard Map



5 Site Investigation

5.1 Site Walkover

ENGEO visited site on 16 August and 6 October 2023 to complete a site walkover, assess current site conditions and identify evidence of potential geohazards. During these site walkovers, we made the following observations (refer to Figure 4 for site photographs):

- The site predominantly comprises grass paddocks, with the paddocks to the northwest of the site lined with trees (Photographs 1 and 2).
- The majority of the trees are coniferous and are estimated to be approximately 15 m high (Photographs 1 and 2).
- Some smaller trees, approximately 5 to 10 m high are located within the paddocks (Photographs 1 and 2).
- The steep slope along the north-western site boundary shows minor scarps and overturning trees indicative of shallow soil failure (Photographs 3 and 4).
- No evidence of more severe instability was observed within the site itself, although subtle signs may have been obscured by long grass and other vegetation.

Figure 4: Site Photographs

Photo 1: Site facing northwest

Photo 2: Site facing west

Photo 3: Overturning tree on north-western slope

Photo 4: Shallow failure scarp on north-western slope

5.2 Subsurface Investigations

ENGEO attended site on 16 August 2023 to complete a subsurface investigation. This investigation comprised eight hand auger boreholes, HA01 through HA08, drilled to depths of between 2.6 and 5.0 m bgl across the site. Test locations are shown on the Investigation Location Plan in Appendix 1.

Hand auger boreholes HA04, HA06, HA07, and HA08 were drilled to target depths of 5.0 m bgl. Boreholes HA02 and HA03 reached practical refusal on hard material at 4.7 and 3.3 m bgl respectively. HA01 and HA05 were terminated at 2.6 and 3.0 m bgl respectively due to practical refusal through hole collapse. Full hand auger borehole logs are presented in Appendix 3. Logs have been prepared in general accordance with the New Zealand Geotechnical Society Guideline for the field classification and description of soil and rock for engineering purposes (NZGS, 2005).

5.3 Investigation Findings

Ground conditions encountered across the site are summarised as follows:

- Topsoil was encountered to depths of up to 0.35 m bgl across the site within all hand auger borehole locations.

- Native Puketoka Formation soils were encountered below the topsoil, at all borehole locations. These soils were observed to comprise grey mottled orange and brown clays and silts with variable sand content. These stiff to hard soils returned shear strengths between 56 and 220+ kPa and presented variations in plasticity. Local horizons of organic clay and peat typically 0.2 to 0.3 m thick were encountered in these soils.

- Native East Coast Bays Formation soils were encountered below Puketoka Formation soils within HA02 and HA03. These soils were observed to comprise dark grey hard silts with variable sand content. These hard soils returned shear strengths above 201 kPa (the upper measurement limit of the hand shear vane used) or were unable to be penetrated by the shear vane blade.

- Based on a distinct increase in cone resistance (qc) from 3-4 MPa to > 15 MPa the results of the CPTs indicate that East Coast Bays is encountered in CPT01 at approximately 5 m depth and in CPT02 at 11.5 m depth indicating that the stratum may dip to the southeast away from the Waiarohia Inlet.

5.3.1 Groundwater

Groundwater was measured at various levels between 0.3 and 4.8 m bgl when the boreholes were dipped at the conclusion of the drilling. These levels should be considered indicative only as they were recorded on the day of drilling and may not represent longer term levels.

5.4 Laboratory Testing

A soil sample was collected from borehole HA06 (Appendix 2) for Atterberg Limits and Linear Shrinkage testing. This testing was undertaken in accordance with NZS4402:1986. Full results can be found in Appendix 4 and are summarised in Table 2.

Table 2: Atterberg Limits Testing

Sample ID	Sample Depth (m)	Water Content	Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage (%)
HA06	0.50 – 1.00	44.9	68	30	38	17

Expansive soils are classified in NZS 3604 as soils with a liquid limit of greater than 50% and a linear shrinkage greater than 15%.

6 Geohazard and Geotechnical Assessment

6.1 Soil Classification

Based on the findings of our desktop and subsurface investigation, as well as our experience of regional ground conditions, we consider the preliminary seismic site classification to be ‘Class C – Shallow Soil Sites’ in line with NZS 1170.5:2004[2] for the purpose of seismic design.

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[2] Standards New Zealand. (2004). Structural design actions – Part 5: Earthquake actions – New Zealand. Published 21/12/04.

6.2 Seismic Hazards

Potential seismic hazards resulting from nearby moderate to major earthquakes can be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, ground lurching, regional subsidence or uplift, soil liquefaction, lateral spreading, landslides, tsunamis, flooding, or seiches. Based on topographic and lithologic data, risk from earthquake-induced regional subsidence / uplift, ground lurching, and seiches are considered negligible at the site. The following sections present a discussion of ground rupture, liquefaction risk, and other geohazards as they apply to the site.

6.2.1 Ground Rupture

There are no known active faults located within the site. Based on regional mapping, and the results of our field observations, it is our opinion that fault-related ground rupture is unlikely at the subject property.

6.2.2 Landslides

Landslides, while primarily found to occur during or following high rainfall events, can be triggered by earthquakes. Ground accelerations produced by earthquakes can significantly reduce the stability of inclined masses of soil, particularly where the soil is vulnerable to strain softening.

As the proposed lots are within the vicinity of sloping ground and historical landslides (at 10 Sinton Road), consideration must be given to the effects of earthquake loading on the stability of these features. We have considered these factors in our slope stability analyses; see Section 6.6.

6.2.3 Ground Shaking

Ground shaking and subsequent effects on structures, infrastructure and engineering systems can be extensive. The intensity, frequency and duration of ground shaking drives the effect of earthquake loading on structures, while the severity of ground shaking drives the level of ground deformation.

The level of ground shaking to which a building must be designed to withstand is dependent on the building's Importance Level as described in clause A3 of the Building Code. As the planned development is residential, we have assumed all buildings will be Importance Level 2 or lower. According to NZS 1170.5:2004, Importance Level 2 buildings are required to retain their structural integrity and not collapse or endanger life during an earthquake with a 500-year return period; the Ultimate Limit State (ULS) design seismic loading. They are further required to sustain little or no structural damage during an earthquake with a 25-year return period; the Serviceability Limit State (SLS) design seismic loading.

Peak horizontal ground accelerations (a_{max}) in accordance with NZGS Earthquake Geotechnical Engineering Practice Module 1, Appendix A1[3] are 0.19 g (ULS) and 0.05 g (SLS).

6.2.4 Liquefaction Analysis

We have assessed the potential of liquefaction triggering and liquefaction induced settlement occurring at the site by performing liquefaction analyses on the CPT data.

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[3] New Zealand Geotechnical Society (NZGS) and Ministry of Business, Innovation and Employment (MBIE) (2021). Earthquake geotechnical engineering practice Module 1: Overview of the guidelines, Version 1, November 2021.

Soil liquefaction and lateral spreading results from the loss of strength during cyclic loading, such as that imposed by earthquakes. Soils most susceptible to liquefaction are typically identified as clean, loose, saturated, cohesionless materials. Empirical evidence indicates that some silty sands, low plasticity silts and low plasticity clays are also potentially liquefiable or may be subject to strain softening. Lateral spreading occurs as a result of liquefied material moving toward a sloping area or free face. This is most common in sloping ground, backfills behind retaining walls, open stormwater channels and water frontage areas. Thin layers, particularly those that are not laterally extensive, are unlikely to liquefy if they are surrounded by non-liquefiable soils.

Liquefaction Methodology

We have assessed the potential of liquefaction triggering and liquefaction induced settlement occurring at the site by performing liquefaction analyses on the CPT data based on the liquefaction triggering methodologies presented by Boulanger and Idriss[4] and using the proprietary software CLiq v.2.3.1.15.

Our analysis included the following assumptions and inputs:

- Ground motion parameters as outlined in Section 6.2.3.
- A maximum earthquake magnitude groundwater level of 1.6 m to reflect the shallowest groundwater level observed within the hand auger boreholes.
- The Zhang and Brachman[5] (2002) procedure for estimating volumetric strain and vertical settlement for the CPT settlement.
- The Boulanger and Idriss relationship between fines content and Soil Behaviour Type (Ic) with a fitting parameter (CFC) of 0.0 for the CPT analysis (no soil laboratory testing available for calibration of the parameter.

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[4] Boulanger, R.W. and Idriss, I.M. (2014). CPT and SPT based liquefaction triggering procedures. Centre for Geotechnical Modeling. Department of Civil & Environmental Engineering, University of California. Report No. UCD/CGM-14/01. April 2014.

[5] Zhang, G.; Robertson, P.K.; and Brachman, R.W.I. (2002). Estimating liquefaction-induced ground.

settlements from CPT for level ground. Canadian Geotechnical Journal 39: 1168–1180. DOI: 10.1139/T02-047.

Liquefaction Discussion

Full results of our analyses are presented in Appendix 5, a summary only is presented in Table 3 below:

Table 3: Ultimate Limit State LSN, LPI and Calculated Vertical Settlement

CPT	LPI	LSN	Calculated Vertical Index Settlement (SLS)	Calculated Vertical Index Settlement (ULS)	MBIE Module 3 Performance Level
CPT01	Negligible	2	Negligible	7 mm	L_0
CPT02	Negligible	1	Negligible	9 mm	L_0

Our analysis indicates that under SLS conditions liquefaction is not predicted to occur at site. Under ULS conditions, liquefaction is predicted to occur in several isolated, sandy horizons that are typically less than 0.5 m thick. These horizons are found at variable depths within the CPTs and considering the difference in elevation between the two tests, it is not likely that they are laterally continuous.

Based on the distribution and size of the liquefiable layers, and the low Liquefaction Potential Index (LPI) and Liquefaction Severity Number (LSN), we anticipate the surface effects of ULS liquefaction to be minor with settlements within building code tolerances.

Table 5.1 of MBIE / NZGS Module 3[6] indicates that the ULS liquefaction induced settlements on this site are within the insignificant category (L_0). The consequences are described as ‘No significant excess pore water pressures (no liquefaction)’.

6.3 Expansive Soils

Expansive soils shrink and swell as a result of seasonal fluctuation in moisture content. This can cause heaving and cracking of on-grade slabs, pavements, and structures founded on shallow foundations.

Building damage due to volume changes associated with expansive soils can be reduced through proper foundation design. Successful performance of structures on expansive soils requires special attention during design and construction. It is imperative that exposed soils be kept moist prior to placement of concrete for foundation construction. It is extremely difficult to re-moisturise clayey soils without excavation, moisture conditioning, and re-compaction.

Based on our laboratory assessment of the near surface soils and our experience with similar soils within the region, we consider a preliminary soil classification of M (moderately) expansive with respect to NZS 3604 (from Section 3.2 of B1/AS1 November 2019 Amendment) is suitable for this site.

It is considered that this preliminary recommendation may be refined with further site-specific testing at the Geotechnical Completion Report stage, following earthworks.

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[6] New Zealand Geotechnical Society (NZGS) and Ministry of Business, Innovation and Employment (MBIE) (2021). Earthquake geotechnical engineering practice Module 3: Identification, assessment and mitigation of liquefaction hazards, November 2021.

The site was found to contain a number of large coniferous trees. The root systems of large trees extract moisture from the soil, which can cause shrinkage and desiccation of expansive soils. When trees are removed during site development the desiccated soils will begin to return to equilibrium moisture content causing them to swell and resulting in ground heave. Movement resulting from the swelling of desiccated soils may last for several years or even decades in high volume change potential soils.

Coniferous trees are typically high water demand species and therefore it is likely soils within the vicinity of the shelter belts have experienced some degree of desiccation, likely commensurate with the depth of root penetration.

6.4 Coastal Regression Hazard

The northern boundary of the site has been identified by Auckland Council as being potentially susceptible to coastal instability and erosion. The potential regression lines for 2050, 2080 and 2130 are mapped within the proposed council esplanade area and are shown in Figure 3. As such, a site-specific coastal hazard assessment undertaken by a Coastal Engineer will be required to support a Resource Consent Application.

6.5 Settlement

Deposits of the Puketoka formation were found to locally contain thin horizons of peat and organic clay up to 0.25 m thick. The Puketoka formation comprises alluvial sediments; In alluvial environments, peat forms in areas with low sediment input, typically on the margins on small, slow flowing channels. These become buried beneath sediment as the channel migrates subsequently forming a peat containing paleo-channel. Based on the spacing of our investigations the presence of one or more paleo-channels on-site cannot be ruled out and the maximum potential depth of peat on-site may not have been encountered in our boreholes.

Peat is considered an unacceptable bearing stratum for foundations as it is highly susceptible to consolidation due to its high-water content (peat may contain ten times its own weight in water). Under the load of fill and building foundations, peat can reduce its volume by up to 75% resulting in significant vertical settlement. Peat is also vulnerable to wasting where it is found above the groundwater table as oxidation of the biomass results in the peat decaying / decomposing. Primary settlement of peat may take days whereas secondary creep consolidation settlement behaviour due to the decay of organic material may continue over 50+ years.

Additional investigations should be undertaken prior to building consent in order to better characterise the extent of the peat on-site and, where peat is located below proposed structures, carry out detailed settlement analysis. Where settlements caused by peat are found to be beyond building code tolerances; suitable solutions may include undercutting and replacing the peat with engineered fill or piled foundations extending below the peat.

6.6 Slope Stability

As described in Section 2, the site is bordered to the northwest by a steep soil slope approximately 5 m in height with a gradient of 1V:1H. The slope appears to be locally unstable with shallow soil scarps and overturning trees observed on the face of the slope. No evidence of more significant global failures were observed within the elevated, gently sloping portions of the site.

Based on these observations we consider the observed instability to be a result of small shallow soil failures on an over steepened slope. However, it should be noted that elsewhere along the crest of the slope that borders the inlet (outside of this site), evidence of much larger rotational failures can be seen. The results of our investigation indicate that the East Coast Bays Formation is encountered at shallow depths close to the slope and dips away from the inlet which may provide a structural control preventing failures of a similar nature from occurring at this site.

Notwithstanding the above, as more significant failures are evidently possible within this geomorphological setting, we suggest that a building restriction line (BRL) be imposed based on a 1V:4H regression line plotted from the toe of the slope. Based on the current development plans and landform this restriction line only encroaches into areas marked for residential development to a minor extent and should not significantly affect development plans.

This BRL may be refined following additional deep geotechnical testing along the slope crest and quantitative slope stability analysis.

6.7 RMA Section 106 Assessment and Development Suitability

Section 106 of the Resource Management Act (RMA) states that a consent authority may refuse to grant a Subdivision Consent, or may grant a consent subject to specific consent conditions if it considers that:

- There is significant risk from natural hazards; or
- Sufficient provision has not been made for legal or physical access to each allotment to be created by the subdivision.

An assessment of the risk from natural hazards as required by the RMA includes the following:

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

We have assessed the risk of natural hazards at the site in accordance with Section 106 of the Resource Management Act (RMA) and considered the risk to the site from erosion, rockfall, inundation (debris), slope stability, subsidence, flooding and tsunami.

Based on our investigation, assessment and site observations, we consider it is unlikely for the site to be subject to the aforementioned natural hazards providing suitable engineering measures are included in the site development (as discussed in Section 7). As such, the site is considered to be conditionally suitable for the proposed residential development from a geotechnical perspective.

7 Geotechnical Recommendations

Based on the results of our geotechnical investigation and subsequent assessment, we consider the site to be generally suitable for the proposed development subject to our geotechnical recommendations being followed.

However, as mentioned in Section 6 the site is at risk from a number of identified geohazards including the following:

- Instability of the over steepened north-western slope bordering Waiarohia Inlet.

- Portions of the site may be vulnerable to settlement due to the potential presence of compressible alluvial soils.

- Shallow site soils are moderately expansive and may be susceptible to shrinkage and heave.

7.1 Foundations

Based on the draft masterplan provided, it is likely that building foundations will found within stiff to hard silts and clays of the Puketoka Formation or East Coast Bays Formation. We consider these deposits to be generally suitable as a foundation subgrade.

Notwithstanding the above, where the Puketoka Formation is found to contain layers of peat, shallow foundations may be vulnerable to intolerable differential settlement as a result of long term consolidation and wasting of the peat. Peat soils are likely to require undercut and replacement with engineering fill or alternatively the use of piled foundations

It is our preliminary recommendation that the site soils following earthworks will likely be suitable for a geotechnical ultimate bearing capacity of 300 kPa for shallow foundations constructed on identified competent natural ground beneath topsoil and existing non-engineered fill or on engineer certified fill.

For piled foundations an ultimate end bearing capacity of 300 kPa may be adopted, taking no account of skin friction, either positive contributions from competent soils or down drag from peat soils or undocumented fills on-site.

This preliminary recommendation will be revisited in the geotechnical completion report to be issued for the site following the satisfactory completion of the proposed earthworks.

It is considered likely that the soils on-site may be M (moderately) expansive with respect to NZS 3604 (from Section 3.2 of B1/AS1 November 2019 Amendment). This will be reassessed as part of the completion reporting for this site.

7.2 Earthworks

7.2.1 General

- All topsoil and undocumented fill shall be removed from all building platforms or areas to receive fill. Where required, all organic soils / peat shall be undercut and replaced up to finished level with suitably compacted engineered fill.

- Excavations and temporary cuts should not exceed a batter angle of 1V:2H up to 2 m in height and should not be left unsupported for longer than two weeks. Cuts beyond this height should be referred to the Geotechnical Engineer for stability assessment.

- Where vertical and subvertical faces higher than 1.0 m are required, we recommend that this is done in shortened sections (< 5 m) and the faces are left unsupported for a minimal time period (i.e. one week) or temporarily shored.

- All temporary cuts and batters proximal to boundaries should take into account the potential surcharge and risk of undermining neighbouring property.

- Suitable drainage channels must be put in place to divert surface water from unsupported cut faces. Subsurface drains should also be considered for the toe of the long-term slopes.

- If any permanent cuts have a batter steeper than 1V:4H and are to be higher than 1.5 m, they should be supported with a specifically designed retaining wall (approved by a chartered Geotechnical Engineer) or be referred back to the Geotechnical Engineer for stability assessment and specific batter design.

- All cuts and batters should be in line with the WorkSafe Good Practice Guidelines for Excavation Safety (July 2016). Permanent fill batters should not exceed 1V:3H and should be reviewed by the Geotechnical Engineer as part of the site development and earthworks proposal review. Fill batters exceeding 1V:3H will require specific geotechnical assessment.

- All excavations should be inspected by ENGEO (or a suitably qualified Geotechnical professional), prior to constructing foundation elements to verify founding conditions are as anticipated.

- Suitable underfill drainage should be considered for any filling on slopes, within stream gully features and wherever seepage is observed within the stripped surface.

- All engineered or structural fill should be placed in ≤ 200 mm compacted lifts and be compacted to a minimum of 95% of maximum dry density, at no less than optimum moisture content. Maximum dry density for granular fill materials may be obtained from the source quarry, a geotechnical laboratory or from plateau testing undertaken on-site. Compaction should be achieved using standard plant and methodology suitable for the imported material. A water source should be maintained on-site for moisture control.

- All excavated soil should be removed from site or placed in an engineer approved stockpile to avoid unfavorable loading on construction or preconstruction slope batters.

- To prevent triggering global failure of the north-western slope, with the exception of replacement filling to replace undercut unsuitable soils, no fill should be placed within the large north-western lots without prior consultation and approval with a suitably qualified geotechnical professional.

7.2.2 Material Suitability

With the exception of topsoil peat and organic clay, we consider site won native soils to be suitable for reuse as compacted engineered fill provided that all unsuitable organics can be separated and appropriate moisture content be maintained. Moisture contents will increase with depth in the cut areas and are likely to be higher in lower lying areas. Material conditioning and compaction can likely be achieved with standard earthworks machinery.

Our experience with the types of native soils present on this site indicates that when they are exposed to the weather their strengths may be significantly reduced. We therefore recommend that trafficked areas and building platforms are only trimmed to final levels immediately prior to placing hardfill / topsoil and that at all times the site is shaped to avoid water ponding during rain, thereby limiting the need for additional undercuts. On no account should areas of trimmed subgrade be left exposed to allow the ingress of water, nor should subgrade areas be trafficked prior to drying out after rain.

7.2.3 Unsuitables

Topsoil and organic soils are not suitable for bearing foundations or for reworking and re-use as engineered fill and should be undercut and stockpiled away from the earthworks area.

7.3 Building Restriction Line

A building restriction line is required to protect future housing and infrastructure from slope instability hazards associated with the north-western slope. The BRL location should be determined by projecting a 1V:4H line of regression from the toe of the slopes to where the line daylights behind the slope crest. At the critical section of the slope this infers a maximum slope setback of approximately 25 m. An approximate building restriction zone is shown on the plan in Appendix 6.

7.4 Service Lines

The construction and installation of new services lines within alluvial material may intercept flowable sands and organic / peat layers. Particular attention should be paid to drainage and stability of trench walls under such circumstances.

Where the base of service line trenches encounters weak, flowable sands and / or organic soils, increased bedding depths of up to 70% and undercuts of approximately 300 mm plus geotextile wrapping of the bedding may be required to provide adequate support to the services and limit the chance of differential settlement along low gradient service alignment. Specific bedding modifications are best prescribed when the trenches are excavated and the material at invert level are examined in detail by a geotechnical professional.

Construction of services during the winter months may pose a risk of trench wall collapse within soft alluvial soils partly due to raised groundwater, leading to the need for additional support, alternative construction methodology and / or dewatering. This should be allowed for on-site by the contractors. Methods to deal with this could be, but not limited to, trench shields to support service trench walls, benching or excavations to a safe temporary works angle (e.g., 1):H): 1(V)).

Should flowable sands and / or organic soil layers be encountered during service line trenching, the contractor shall contact ENGEO.

7.5 Stormwater and Effluent Disposal

ENGEO has not been provided with plans showing the preferred methods of stormwater and wastewater disposal.

Based on the preliminary plans that have been provided we anticipate that waste-water will be disposed of via reticulated council services.

Due to the proximity of the steep and unstable slopes to the proposed development, we do not recommend in-ground soakage systems are adopted for the site. All stormwater collecting from hard standing areas and roofing should be collected and reticulated to council services.

Overland flows should be directed away from existing slopes to reduce the risk of ponding and erosion exacerbating slope instability concerns.

7.6 Retaining Walls

Currently, there are no retaining structures shown on the development plans. Any future retaining should be designed to accommodate for the soils encountered on-site. Based on our subsurface investigations, we expect the proposed retaining structures will generally support native Puketoka Formation or East Coast Bays Formation.

7.6.1 Preliminary Retaining Wall Parameters

Based on the results of our investigation and ground conditions at the site, future retaining walls should be designed using the following geotechnical parameters:

Table 4: Soil Parameters for Retaining Wall Design

Material Type	Unit Weight	Friction angle (°)	Effective Cohesion c' (kPa)	Undrained Shear Strength Su (kPa)
Puketoka Formation (Stiff to very stiff)	18	28	3	80
East Coast Bays Formation (residual soil)	18	32	5	100
Cohesive Engineered Fill	18	32	5	100
Granular Engineered Fill	20	38	0	-

The retaining wall design should include appropriate drainage which must outlet into an approved stormwater disposal system.

We recommend that design of retaining walls should be carried out in line with Module 6 of the Ministry of Business, Innovation and Employment Guidance.

7.7 Surface Water Management

During construction, appropriate measures shall be undertaken to control and treat stormwater runoff, with silt and erosion controls complying with Auckland Council Guidance for Erosion & Sediment Control (GD05).

This is particularly relevant for the site due to the proximity to a stormwater receptor, being the inlet to the north. Surface cut-off drains or appropriate stormwater flow paths should be maintained outside of the proposed development area, both during and following construction. These drains and impervious surfaces will divert water away from any buildings and minimise possible movement in sensitive soils during and post construction.

Stormwater from paved areas shall be taken in a piped system and disposed of into an approved stormwater system. Uncontrolled discharge onto land or uncontrolled disposal via in-ground systems must be avoided.

All service trenches should be capped with low permeability materials, so that excavations do not become points of entry for surface run-off.

7.8 Pavement Subgrade CBR

Inferred CBRs of approximately 3% may be adopted for native soils and 6% for cohesive engineered fill areas are considered to be suitable for preliminary design purposes. These values are derived from the soils encountered in our hand auger boreholes and our knowledge of the soil type on-site.

It should be noted that actual CBR values can be highly affected by moisture content (i.e., exposure to the elements) and trafficking.

A programme of CBR testing should be carried out on the stripped subgrade level within roading corridors to confirm actual values.

8 Future Work

We recommend ENGEO's involvement in the following activities:

- Review of earthworks proposals when completed, and additional investigations where necessary to identify the extent of peat relative to the proposed earthworks if likely to be affected by earthworks and building development.
- Design Plan Review or Detailed Design to support Resource Consent & Building Consent (walls, structures, etc).
- Preparation of a Geotechnical earthworks specification.
- Observation and certification of earthworks and retaining walls including all stripping and undercuts and engineered fill in accordance with the earthworks and retaining wall specifications.
- Geotechnical Completion Reporting/Producer Statements.

9 Limitations

i. We have prepared this report in accordance with the brief as provided. This report has been prepared for the use of our client, Cabra Developments Limited, their professional advisers and the relevant Territorial Authorities in relation to the specified project brief described in this report. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity.

ii. The recommendations in this report are based on the ground conditions indicated from published sources, site assessments and subsurface investigations described in this report based on accepted normal methods of site investigations. Only a limited amount of information has been collected to meet the specific 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 judgement and it should be appreciated that actual conditions could vary from the assumed model.

iii. 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.

iv. This Limitation should be read in conjunction with the Engineering NZ/ACENZ Standard Terms of Engagement.

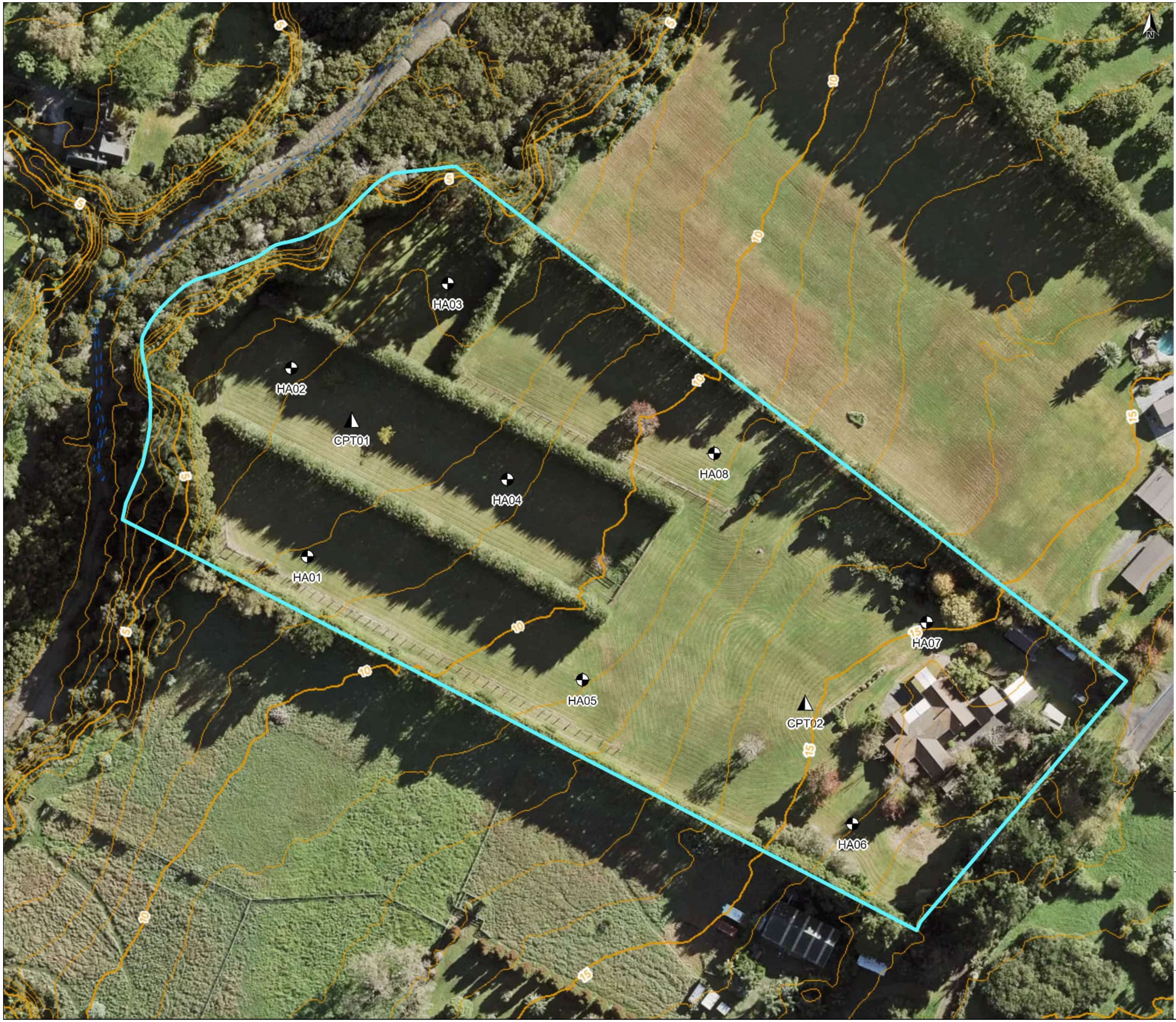
v. This report is not to be reproduced either wholly or in part without our prior written permission.

We trust that this information meets your current requirements. Please do not hesitate to contact the undersigned on (09) 972 2205 if you require any further information.

Report prepared by	Report reviewed by
Jamie Lott	**Paul Fletcher, CMEngNZ (CPEng)**
Engineering Geologist	Principal Geotechnical Engineer
Jamie Thomas	**Heather Lyons, CMEngNZ (PEngGeol)**
Geotechnical Engineer	Associate Engineering Geologist

APPENDIX 1:

Investigation Location Plan



Legend

- 🟦 Site Boundary
- ⊕ ENGEO Hand Auger Boreholes (August 2023)
- ▲ ENGEO CPT (August 2023)

0 10 m 20 m

LINZ CC BY 4.0 © Imagery Basemap contributors, , public

ENGEO

Produced by **Datanest.earth**

Title: Investigation Location Plan

Client:
Cabra Developments Limited

Project:
16 Sinton Road, Whenupai

Drawn:
JL

Appendix No.:
1

Size: A3

Date:
30-10-2023

Checked: PF

Proj No.:
23849.000.001_04

Scale:
1:1000

Version: 1

APPENDIX 2:

Historical Aerial Photographs

1940 (Retrolens NZ)

1950 (Retrolens NZ)

1959 (Auckland Council GeoMaps)

1968 (Retrolens NZ)

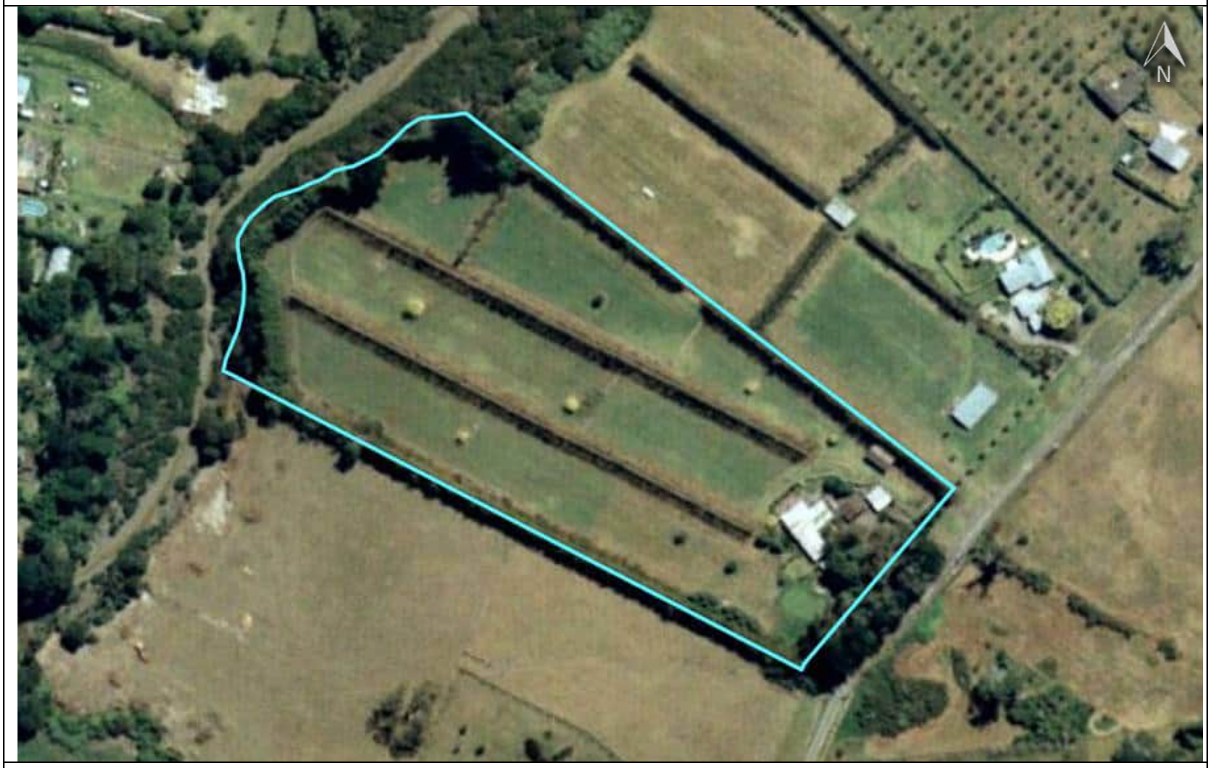
1972 (Retrolens NZ)

1975 (Retrolens NZ)

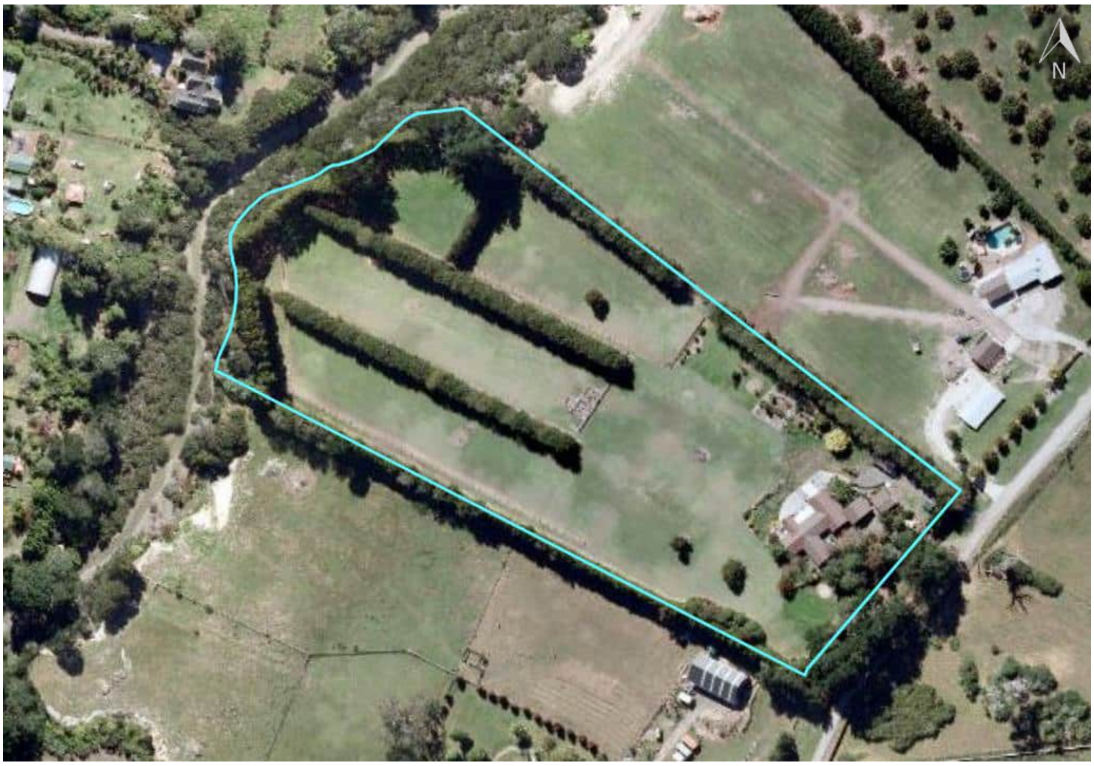
1978 (Retrolens NZ)

1988 (Retrolens NZ)

1996 (Auckland Council GeoMaps)



2000 (Auckland Council GeoMaps)



2008 (Auckland Council GeoMaps)

2017 (Nearmaps)

2020 (Nearmaps)

2023 (Nearmaps)

APPENDIX 3:

Geotechnical Logs

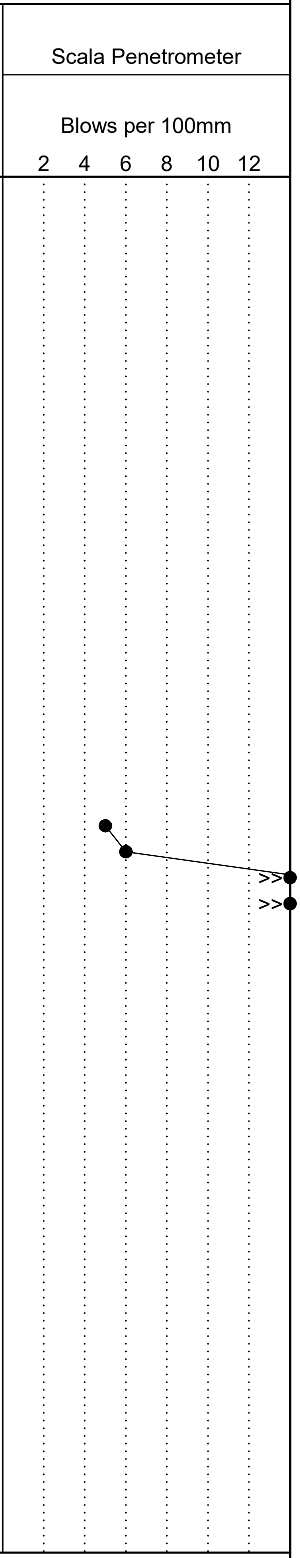
LOG OF AUGER HA01

Geotechnical Investigation
16 Sinton Road
16 Sinton Road, Whenuapai
23849.000.001_04

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_04
Date : 16-08-2023
Hole Depth : 2.6 m
Hole Diameter : 50 mm
Shear Vane No : 3840
Logged By : KE
Reviewed By : JL
Latitude : -36.7961829
Longitude : 174.6387979

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
0.0 – 0.25	TS	OL	TOPSOIL				W	NA	
0.25 – 1.3	PUKETOKA FORMATION	CH	Silty CLAY; light grey with orange brown mottling. High plasticity.		7 (at 1.0)		M	St - H	91/20 97/25 140/31 220+
1.3 – 2.15		CH	CLAY; light grey to white. High plasticity. 1.4 m: Becomes wet.		6 (at 2.0)		W	St	82/39 78/31 82/38
2.15 – 2.6		ML	Fine sandy SILT; light grey. Low plasticity. 2.2 m: Encountered standing groundwater. Becomes saturated.			▼ (2.2)	S	St - VSt	129/47

End of Hole Depth: 2.6 m
Termination Condition: Practical refusal



Hand Auger met practical refusal at 2.6 m depth. due to hole collapse
Scala Penetrometer met practical refusal at 2.8 m depth.
Dip test showed standing water at 2.2 m depth.
Coordinates obtained via handheld GPS.

Elevation obtained via Auckland Council GIS.
TS = Topsoil; NA = Not Assessed.

LOG OF AUGER HA02

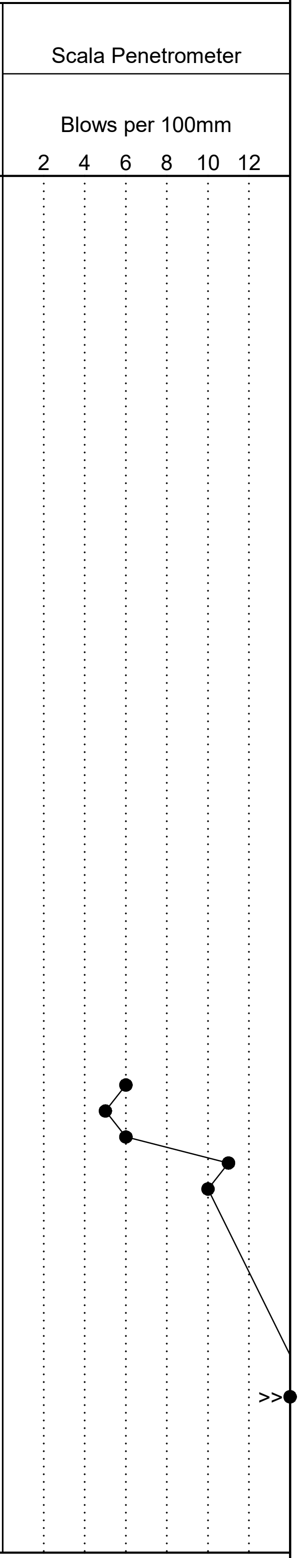
Geotechnical Investigation
16 Sinton Road
16 Sinton Road, Whenuapai
23849.000.001_04

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_04
Date : 16-08-2023
Hole Depth : 4.7 m
Hole Diameter : 50 mm

Shear Vane No : 2853
Logged By : LM
Reviewed By : JL
Latitude : -36.7957157
Longitude : 174.6387477

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
0.0 – 0.3	TS	OL	TOPSOIL			W	NA	
0.3 – 2.3	EAST COAST BAYS FORMATION	CH	Silty CLAY with trace fine sand; grey with orange and yellow mottling. High plasticity. 1.6 m: Becomes light grey with orange streaks.	6 5		M	VSt	142/59 132/49 125/46 106/60 118/62 103/57
2.3 – 3.4		ML	Fine sandy SILT with trace clay; grey with orange mottling. Low plasticity. 2.6 m: Encountered standing groundwater. Becomes saturated.	4	▼	M S	VSt - H	175/52 181/67 201+ 134/33
3.4 – 4.0		SM	Silty fine SAND; brownish grey with orange yellow mottling. Poorly graded.	3		S	MD D	
4.0 – 4.7		ML	Fine sandy SILT; dark grey. Low plasticity. 4.7 m: Encountered dark brownish black fibrous organic streak.			S	H	UTP UTP

End of Hole Depth: 4.7 m
Termination Condition: Practical refusal



Hand Auger met practical refusal at 4.7 m depth. on hard material
Scala Penetrometer met practical refusal at 4.7 m depth.
Dip test showed standing water at 2.6 m depth.
Coordinates obtained via handheld GPS.

Elevation obtained via Auckland Council GIS.
TS = Topsoil; NA = Not Assessed; UTP = Unable To Penetrate.

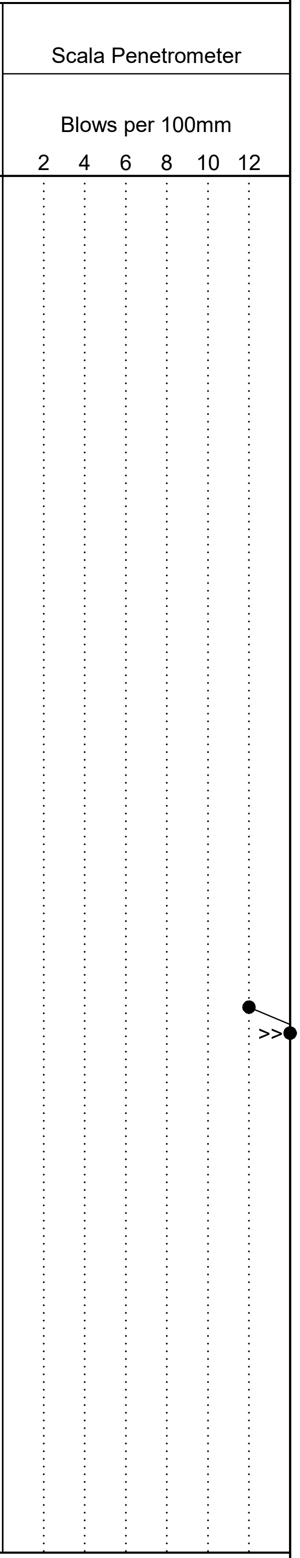
LOG OF AUGER HA03

Geotechnical Investigation
16 Sinton Road
16 Sinton Road, Whenuapai
23849.000.001_04

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_04
Date : 16-08-2023
Hole Depth : 3.3 m
Hole Diameter : 50 mm

Shear Vane No : 2853
Logged By : LM
Reviewed By : JL
Latitude : -36.795506
Longitude : 174.6392327

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
	TS	OL	TOPSOIL				W	NA	
0.5 1.0 1.5	PUKETOKA FORMATION	CH	Silty CLAY; grey. High plasticity. 1.2 m: Encountered standing groundwater. Becomes saturated.		6	▼	W S	St	62/19 57/1 73/1 63/13
2.0	EAST COAST BAYS FORMATION	ML	Fine sandy SILT with trace clay; light grey with orange yellow mottling. Low plasticity.		5		S	VSt	139/47 136/52
2.5 3.0		ML	Clayey SILT with some fine sand; dark grey with occasional black streaks. Low plasticity.		4		S	H	201+ 201+ 201+ 201+
3.5 4.0 4.5 5.0			End of Hole Depth: 3.3 m Termination Condition: Practical refusal						



Hand Auger met practical refusal at 3.3 m depth. on hard material
Scala Penetrometer met practical refusal at 3.3 m depth.
Dip test showed standing water at 1.2 m depth.
Coordinates obtained via handheld GPS.

Elevation obtained via Auckland Council GIS.
TS = Topsoil; NA = Not Assessed.

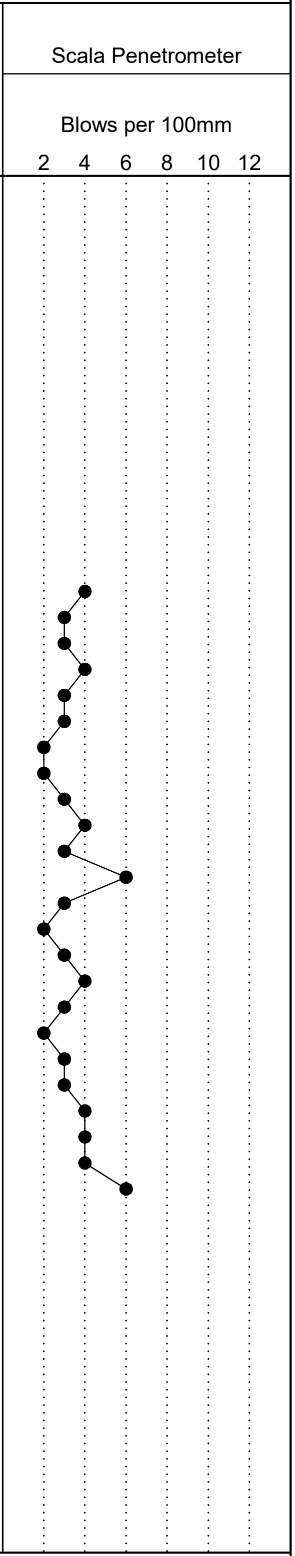
LOG OF AUGER HA04

Geotechnical Investigation
16 Sinton Road
16 Sinton Road, Whenuapai
23849.000.001_04

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_04
Date : 16-08-2023
Hole Depth : 5 m
Hole Diameter : 50 mm
Shear Vane No : 2853
Logged By : LM
Reviewed By : JL
Latitude : -36.7959917
Longitude : 174.6394167

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
	TS	OL	TOPSOIL				M	NA	
0.5 1.0	PUKETOKA FORMATION	ML	Clayey SILT with minor fine sand; brownish grey with orange mottling. Low plasticity. 0.7 m: Becomes light grey with yellow and orange brown streaks. 1.0 m: Becomes light grey with yellow mottling.		8		M	St - VSt	85/17 134/47 125/47 131/65
1.5		SM	Silty fine SAND with some clay; light grey with yellow mottling. Poorly graded. 1.5 m: Encountered standing groundwater. Becomes saturated.		7	▼	S	MD	
2.0		SP	Silty fine to medium SAND; greyish brown. Poorly graded.				S	MD	
2.5 3.0 3.5		SW	Silty fine to coarse SAND with trace organics; dark, brown, orange and grey intermixed. Well graded. Poor recovery.		6 5		S	L - MD	
4.0 4.5 5.0		CH	Silty CLAY with trace fine sand; grey. High plasticity.		4		S	St - VSt	79/53 149/76 112/69

End of Hole Depth: 5 m
Termination Condition: Target depth



Hand Auger met target depth at 5 m.
Scala Penetrometer met target depth at 3.9 m.
Dip test showed standing water at 1.5 m depth,
Coordinates obtained via handheld GPS.

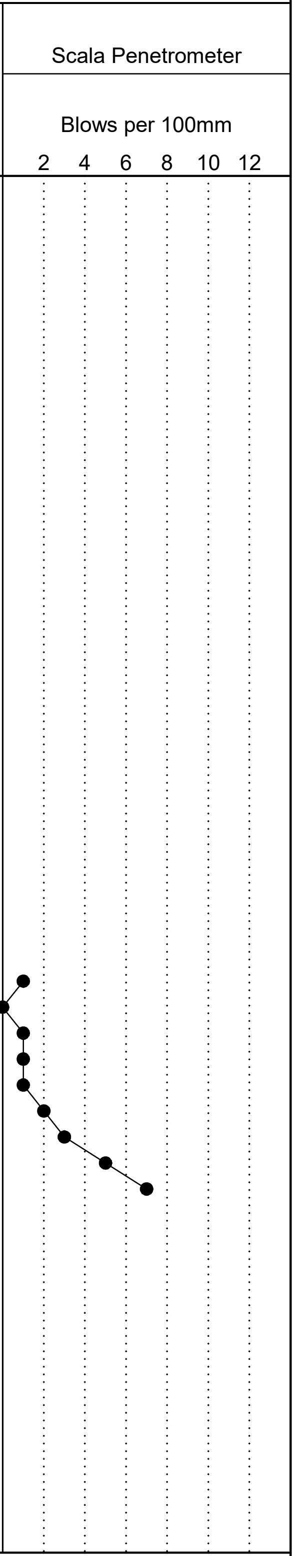
Elevation obtained via Auckland Council GIS.
TS = Topsoil; NA = Not Assessed.

LOG OF AUGER HA05

Geotechnical Investigation
16 Sinton Road
16 Sinton Road, Whenuapai
23849.000.001_04

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_04
Date : 16-08-2023
Hole Depth : 3 m
Hole Diameter : 50 mm
Shear Vane No : 3840
Logged By : KE
Reviewed By : JL
Latitude : -36.7964889
Longitude : 174.6396495

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded
	TS	OL	TOPSOIL			W	NA	
0.5 1.0 1.5	PUKETOKA FORMATION	CH	Silty CLAY; light grey with orange brown streaks. High plasticity. 0.45 m: Becomes very stiff.	10		W	St VSt	67/8 169/78 188/88 151/66 140/61
		OH	Organic CLAY; dark grey with black and grey streaks. High plasticity.			W	VSt	140/72
2.0 2.5		CH	CLAY; light grey. High plasticity. 1.95 m: Becomes stiff. 2.0 m: Encountered standing groundwater. Becomes saturated.	9	▼	S	VSt St	86/34 75/19
		CH	Silty CLAY with minor fine to medium sand; dark greyish brown. High plasticity.			S	St	
			No Recovery.				NA	
3.0		CH	CLAY; grey to light grey. High plasticity.	8		S	St	60/33
3.5 4.0 4.5 5.0			End of Hole Depth: 3 m Termination Condition: Practical refusal					



Hand Auger met practical refusal at 3 m depth. due to hole collapse
Scala Penetrometer met target depth at 3.7 m.
Dip test showed standing water at 2.0 m depth.
Coordinates obtained via handheld GPS.

Elevation obtained via Auckland Council GIS.
TS = Topsoil; NA = Not Assessed.

LOG OF AUGER HA06

Geotechnical Investigation
16 Sinton Road
16 Sinton Road, Whenuapai
23849.000.001_04

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_04
Date : 16-08-2023
Hole Depth : 5 m
Hole Diameter : 50 mm

Shear Vane No : 3840
Logged By : KE
Reviewed By : JL
Latitude : -36.7968457
Longitude : 174.640486

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm 2 4 6 8 10 12
	TS	OL	TOPSOIL				W	NA		
0.5	PUKETOKA FORMATION	ML	SILT with some clay; light brown with orange brown streaks. Low plasticity. 0.4 m: Becomes moist.				W M	St - VSt	100/30 97/31	
1.0 1.5 2.0 2.5 3.0		ML	Clayey SILT; light grey with orange brown streaks. Low plasticity. 3.1 m: Becomes wet.		15 14 13		M W	VSt - H	132/61 172/107 207/108 220+ 220+ 191/125 190/118 151/88	
3.5 4.0 4.5 5.0		CH	CLAY; dark grey with grey mottling. High plasticity. 4.8 m: Encountered standing groundwater. Becomes saturated.		12 11	▼	W S	VSt	136/66 102/56 122/63 133/63 132/75 108/80	
			End of Hole Depth: 5 m Termination Condition: Target depth							

Hand Auger met target depth at 5 m.
Dip test showed standing water at 4.8 m depth.
Coordinates obtained via handheld GPS.
Elevation obtained via Auckland Council GIS.

TS = Topsoil; NA = Not Assessed.

ENGEO

LOG OF AUGER HA07

Geotechnical Investigation
16 Sinton Road
16 Sinton Road, Whenuapai
23849.000.001_04

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_04
Date : 16-08-2023
Hole Depth : 5 m
Hole Diameter : 50 mm

Shear Vane No : 3840
Logged By : KE
Reviewed By : JL
Latitude : -36.7963462
Longitude : 174.640716

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm 2 4 6 8 10 12
0.0	TS	OL	TOPSOIL		▼	W	NA		
0.3	PUKETOKA FORMATION	CH	Silty CLAY with trace sand; light grey with orange brown streaks. High plasticity.	14		S	VSt	71/19 172/88 176/72	
1.2			1.2 m: Becomes with minor fine sand.					191/58	
1.35			1.35 m: Becomes hard.	13			H	220+ 220+ 220+	
2.25			2.25 m: Becomes very stiff.				VSt	110/61 136/44	
2.8			2.8 m: Becomes with some fine sand.	12				107/50	
3.15			3.15 m: Becomes stiff.				St	96/53	
3.5			3.5 m: Becomes orange brown with light grey mottling.					72/38	
3.8		CH	CLAY; dark grey with grey streaks. High plasticity.	11		S	St - VSt	97/50 144/80	
4.5		CH	CLAY with some fine to medium sand and minor organics; dark grey. High plasticity. Organics comprise approximately 1-2 mm woody chunks.	10		S	St - H	220+ 88/56	
5.0			End of Hole Depth: 5 m Termination Condition: Target depth						

Hand Auger met target depth at 5 m.
Dip test showed standing water at 0.3 m depth.
Coordinates obtained via handheld GPS.
Elevation obtained via Auckland Council GIS.

TS = Topsoil; NA = Not Assessed.

LOG OF AUGER HA08

ENGEO

Geotechnical Investigation
16 Sinton Road
16 Sinton Road, Whenuapai
23849.000.001_04

Client : Cabra Developments Ltd
Client Ref. : 23849.000.001_04
Date : 16-08-2023
Hole Depth : 5 m
Hole Diameter : 50 mm
Shear Vane No : 2853
Logged By : LM
Reviewed By : JL
Latitude : -36.7959273
Longitude : 174.6400577

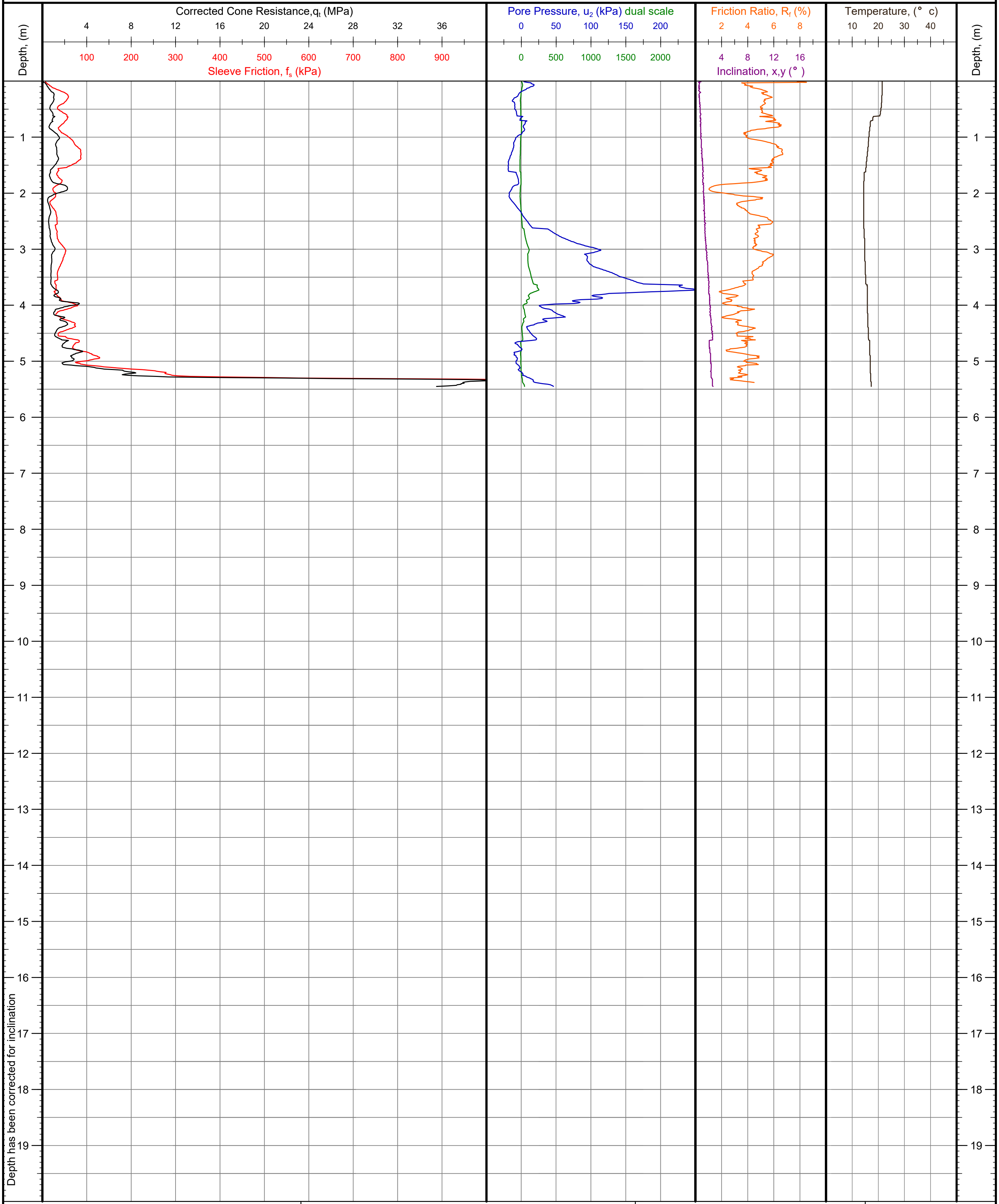
Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm 2 4 6 8 10 12
	TS	OL	TOPSOIL				M	NA		
0.5 1.0 1.5	PUKETOKA FORMATION	CH	Silty CLAY with trace fine sand; light grey with orange yellow mottling. High plasticity.		10		M	VSt	142/59 136/59 158/50 151/67	
2.0 2.5		CH	CLAY with minor silt; light grey with yellow, purple and orange brown mottling. High plasticity. 1.95 m: Becomes stiff. 2.7 m: Encountered standing groundwater. Becomes saturated.		9 8	▼	M S	VSt St	132/67 76/37 56/24	
3.0		OH	Organic CLAY; purplish grey. High plasticity.				S	St	65/20	
		PT	Amorphous PEAT				S	NA		
3.5 4.0 4.5		CH	Silty CLAY with minor organics and trace fine sand; dark grey with black inclusions. High plasticity. Organics are decomposed wood fragments. 3.75 m: Becomes very stiff.		7		S	St VSt	88/63 146/78 145/57 172/42	
5.0		ML	Clayey SILT with minor fine sand and trace organics; dark grey with black mottling. Low plasticity.		6		S	H	201+	

End of Hole Depth: 5 m
Termination Condition: Target depth

Hand Auger met target depth at 5 m.
Dip test showed standing water at 2.7 m depth.
Coordinates obtained via handheld GPS.
Elevation obtained via Auckland Council GIS.

TS = Topsoil; NA = Not Assessed.

CONE PENETRATION TEST (CPT) LOG



Project: 10 & 16 Sinton Rd & 15 Clarks Ln

Location: Hobsonville, Auckland

NZTM 2000 N, E (m): 5926522.52, 1746222.77

WGS84 (deg): -36.795840,174.638959

Location Method: Handheld GPS

Elevation (m): Unknown

Comments: Tested at 16 Sinton Road.

Operator: Cesar Etchevarne

Cone Ref: MKJ311

Cone Type: 10cm^2 Compression

Date of Test: 16/08/2023

Depth (m): 5.55

Termination Reason: High friction resistance

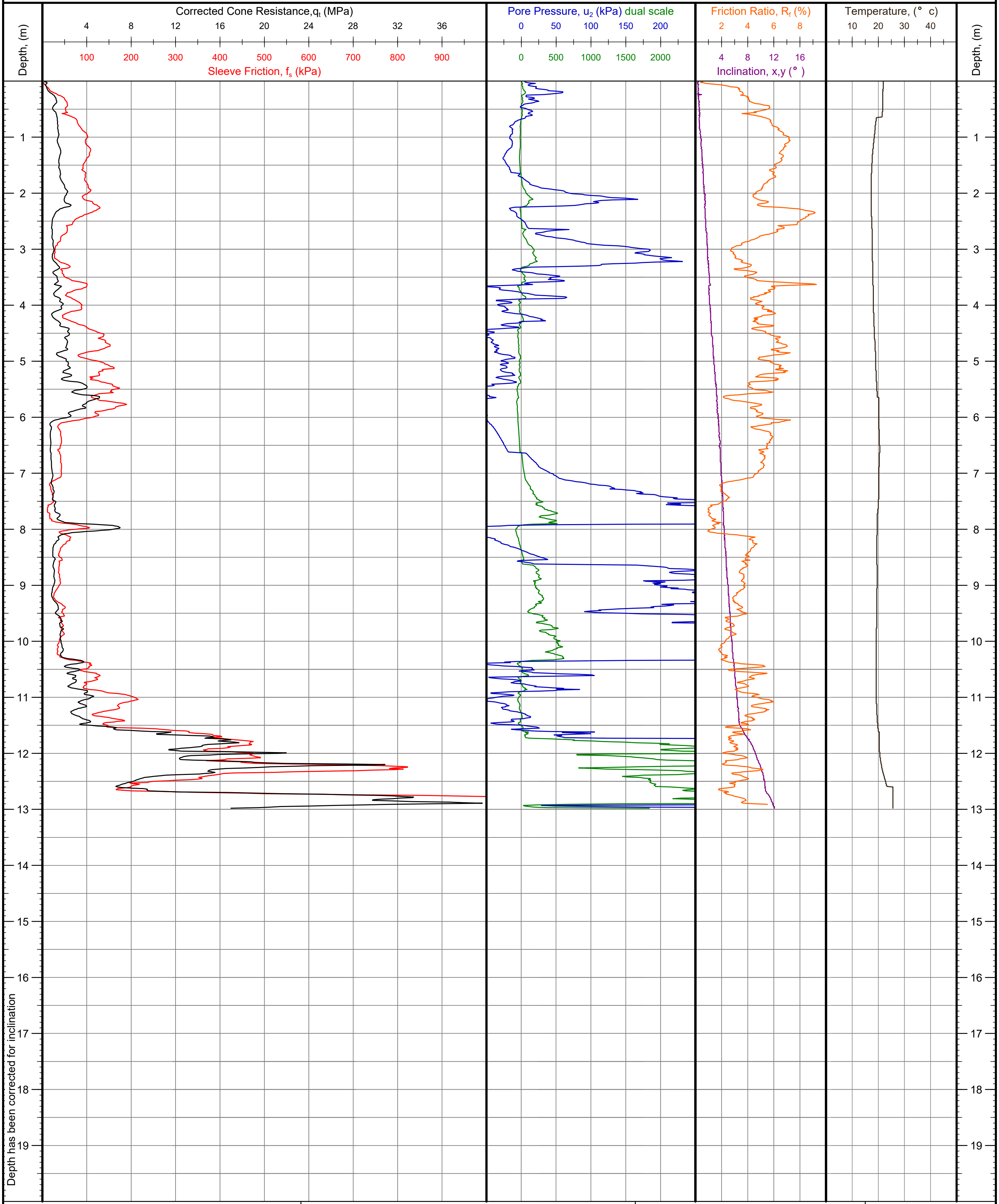
Test Number:

CPT01-23849.000.003

G.I. Job Ref:
230738

Where possible GWL is measured after testing, or estimated in the office. This may not represent the true GWL

CONE PENETRATION TEST (CPT) LOG



Project: 10 & 16 Sinton Rd & 15 Clarks Ln

Location: Hobsonville, Auckland

NZTM 2000 N, E (m): 5926439.05, 1746346.74

WGS84 (deg): -36.796573,174.640364

Location Method: Handheld GPS

Elevation (m): Unknown

Operator: Cesar Etchevarne

Cone Ref: MKJ311

Cone Type: 10cm^2 Compression

Date of Test: 16/08/2023

Depth (m): 13.21

Test Number: CPT02-23849.000.003

G.I. Job Ref: 230738

Comments: Tested at 16 Sinton Road.

Termination Reason: High friction resistance

Where possible GWL is measured after testing, or estimated in the office. This may not represent the true GWL

APPENDIX 4:

Laboratory Test Results

BGL
Babbage Geotechnical Laboratory

Babbage Geotechnical Laboratory
Level 4
68 Beach Road
Auckland 1010
Telephone
E-mail

P O Box 2027
New Zealand
64-9-367 4954
wec@babbage.co.nz

Please reply to: W.E. Campton

ENGEO LTD.
PO Box 33-1527
Takapuna
Auckland 0740

Attention: **HEATHER LYONS**

Job Number: 66273#L
BGL Registration Number: 3064
Checked by: WEC

30th August 2023

ATTERBERG LIMITS & LINEAR SHRINKAGE TESTING

Dear Heather,

Re: 16 SINTON ROAD, HOBSONVILLE

Your Reference: 23849.000.003
Report Number: 66273#L/AL 16 Sinton Rd

The following report presents the results of Atterberg Limits & Linear Shrinkage testing at BGL of a soil sample delivered to this laboratory on the 21st of August 2023. Test results are summarised below, with page 3 showing where the sample plots on the Unified Soil Classification System (Casagrande) Chart. Test standards used were:

Water Content:	NZS4402:1986:Test 2.1
Liquid Limit:	NZS4402:1986:Test 2.2
Plastic Limit:	NZS4402:1986:Test 2.3
Plasticity Index:	NZS4402:1986:Test 2.4
Linear Shrinkage:	NZS4402:1986:Test 2.6

Borehole Number	Sample Number	Depth (m)	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage (%)*
HA06	**Sample 1**	**0.50 – 1.00**	**44.9**	**68**	**30**	**38**	**17**

The amount of shrinkage of the sample as a percentage of the original sample length.

The whole soil was used for the water content test (the soil was in a natural state), and for the liquid limit, plastic limit and linear shrinkage tests. The soil was wet up and dried where required for the liquid limit, plastic limit and linear shrinkage tests.

[redacted]

As per the reporting requirements of NZS4402: 1986: Test 2.1: water content is reported to two significant figures for values below 10%, and to three significant figures for values of 10% or greater. Test 2.2: liquid limit, test 2.3: plastic limit, and test 2.6: linear shrinkage are reported to the nearest whole number.

Please note that the test results relate only to the sample as-received, and relate only to the sample under test.

Thank you for the opportunity to carry out this testing. If you have any queries regarding the content of this report please contact the person authorising this report below at your convenience.

Yours faithfully,

Justin Franklin
Key Technical Person
Assistant Laboratory Manager
Babbage Geotechnical Laboratory

ACCREDITED
IANZ
TESTING LABORATORY
№ 126

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation. This report may not be reproduced except in full & with written approval from BGL.

BGL
Babbage Geotechnical Laboratory

Job Number:	**66273#L**	**Sheet 1 of 1**	**Page 3 of 3**
Reg. Number:	**3064**	**Version No:**	**7**
Report No:	**66273#L/AL 16 Sinton Rd**	**Version Date:**	**July 2022**
Project:	**16 SINTON ROAD, HOBSONVILLE**		

DETERMINATION OF THE LIQUID LIMIT, PLASTIC LIMIT & THE PLASTICITY INDEX

Test Methods: NZS4402: 1986: Test 2.2, Test 2.3 and Test 2.4

Tested By:	JL	August 2023
Compiled By:	JF	30/08/2023
Checked By:	JF	30/08/2023

SUMMARY OF TESTING

Borehole Number	Sample Number	Depth (m)	Liquid Limit	Plastic Limit	Plasticity Index	Soil Classification Based on USCS Chart Below
HA06	**Sample 1**	**0.50 - 1.00**	68	30	**38**	CH

The chart below & soil classification terminology is taken from ASTM D2487-17^{e1} "Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)", April 2020, & is based on the classification scheme developed by A. Casagrande in the 1940's (Casagrande, A., 1948: Classification and identification of soil. Transactions of the American Society of Civil Engineers, v. 113, p. 901-930). The chart below & the soil classification given in the table above are included for your information only, and are not included in the IANZ endorsement for this report.

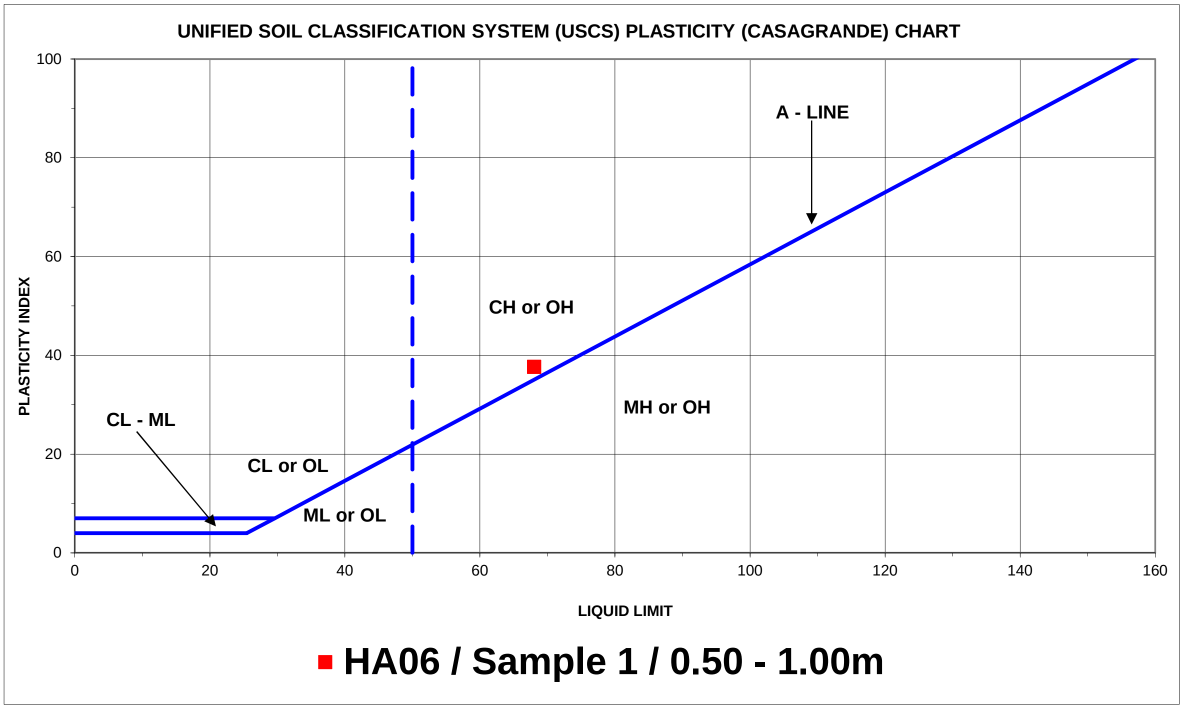


CHART LEGEND

CL = CLAY, low plasticity ('lean' clay)
OL = ORGANIC CLAY or ORGANIC SILT, low liquid limit
ML = SILT, low liquid limit
CL - ML = SILTY CLAY
CH = CLAY, high plasticity ('fat' clay)
OH = ORGANIC CLAY or ORGANIC SILT, high liquid limit
MH = SILT, high liquid limit ('elastic silt')

APPENDIX 5:

Liquefaction Analysis Outputs

ENGEO Limited
1/314 Maunganui Road, Mt Maunganui 3116
+64 7 777 0209
http://www.engeo.co.nz

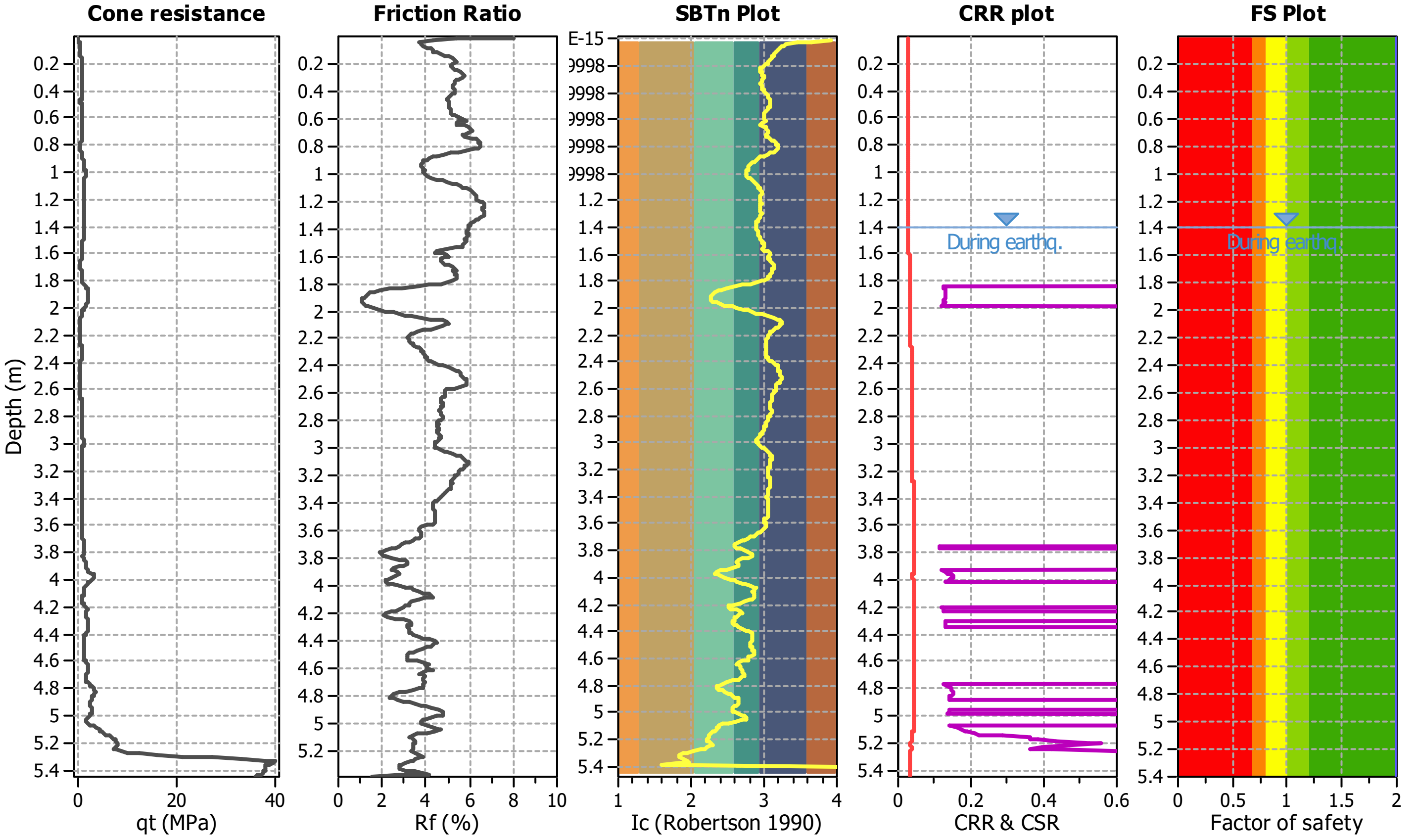
LIQUEFACTION ANALYSIS REPORT

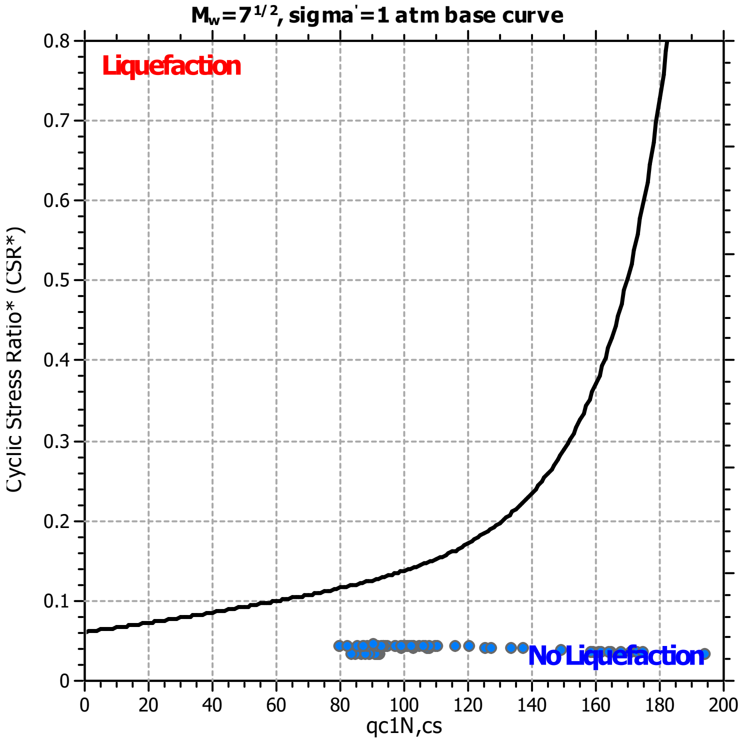
Project title : 16 Sinton Road **Location : Whenuapai**

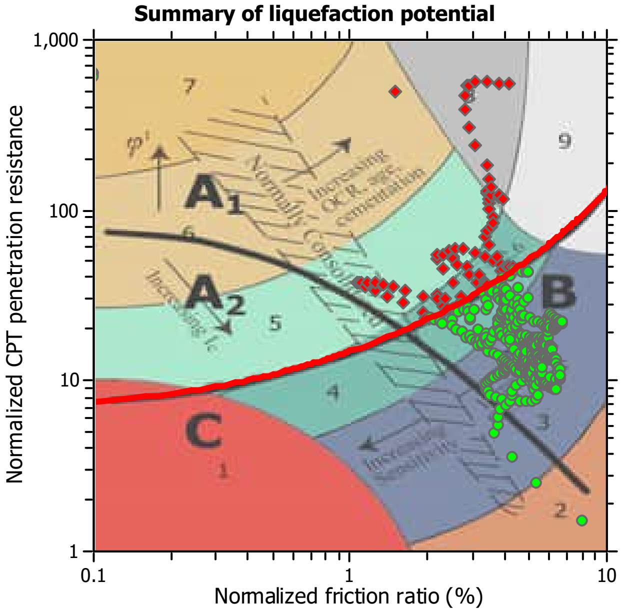
CPT file : CPT01

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.40 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.40 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.05	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

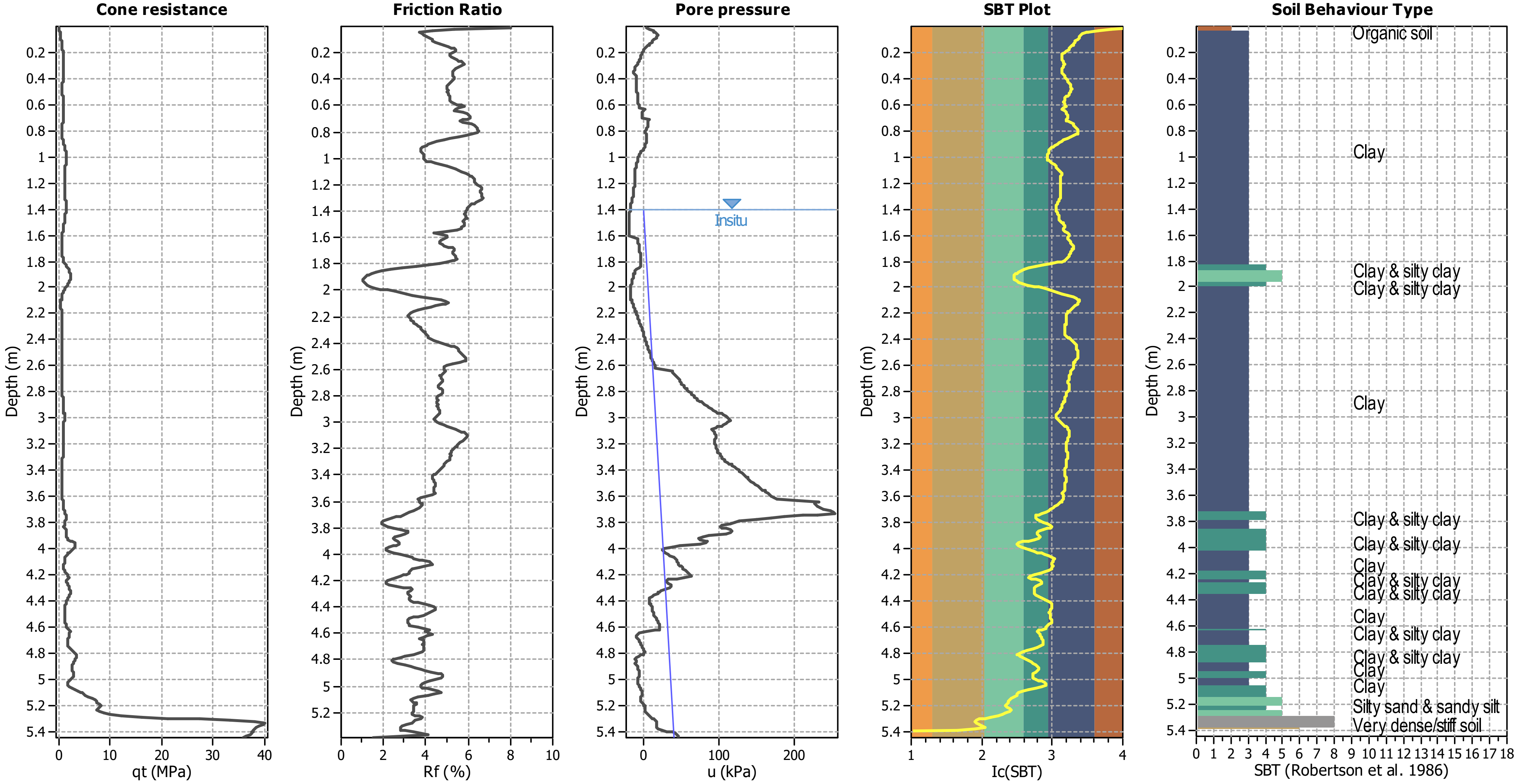






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



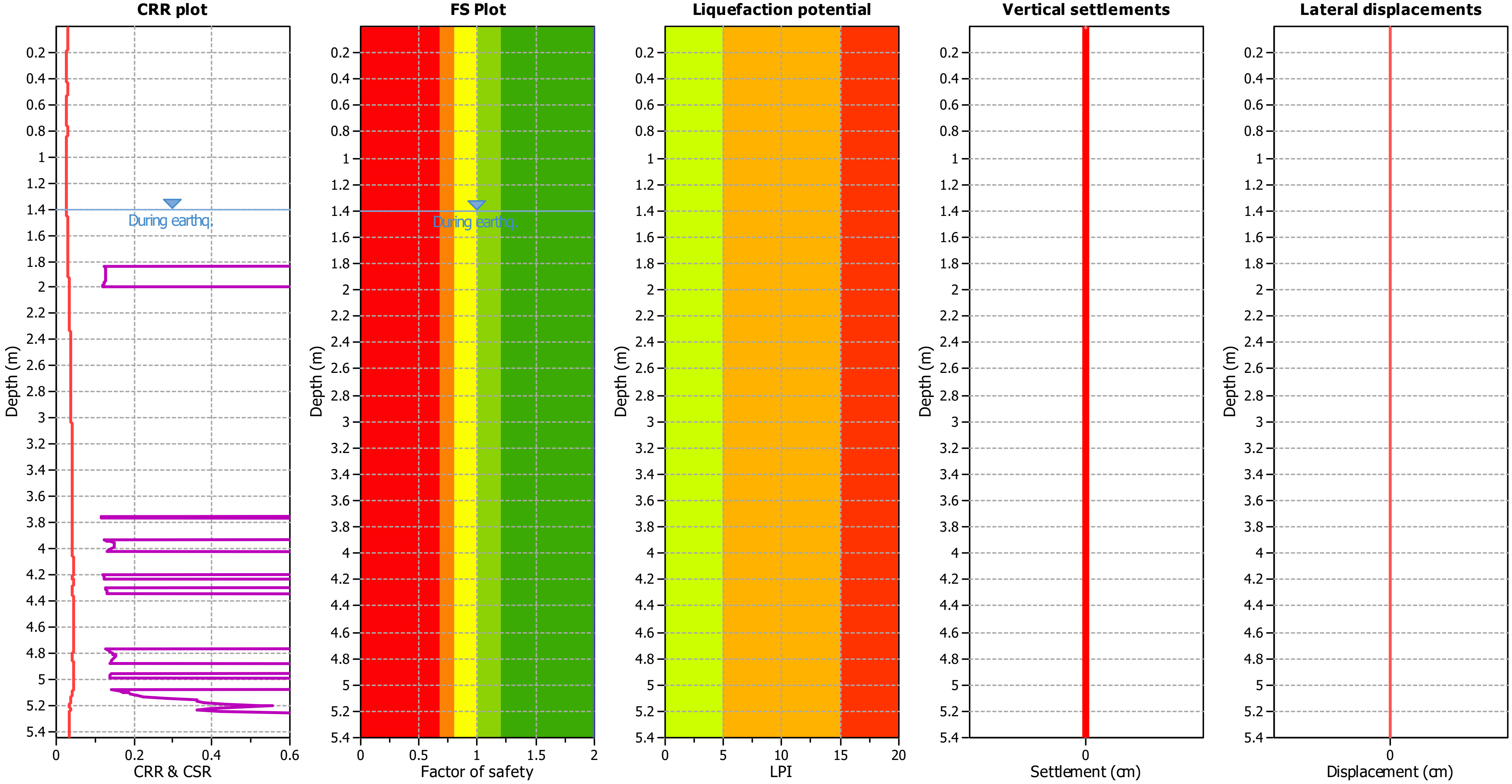
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.40 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.40 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



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: 6.50
Peak ground acceleration: 0.05
Depth to water table (insitu): 1.40 m

Depth to GWT (erthq.): 1.40 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: No
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A

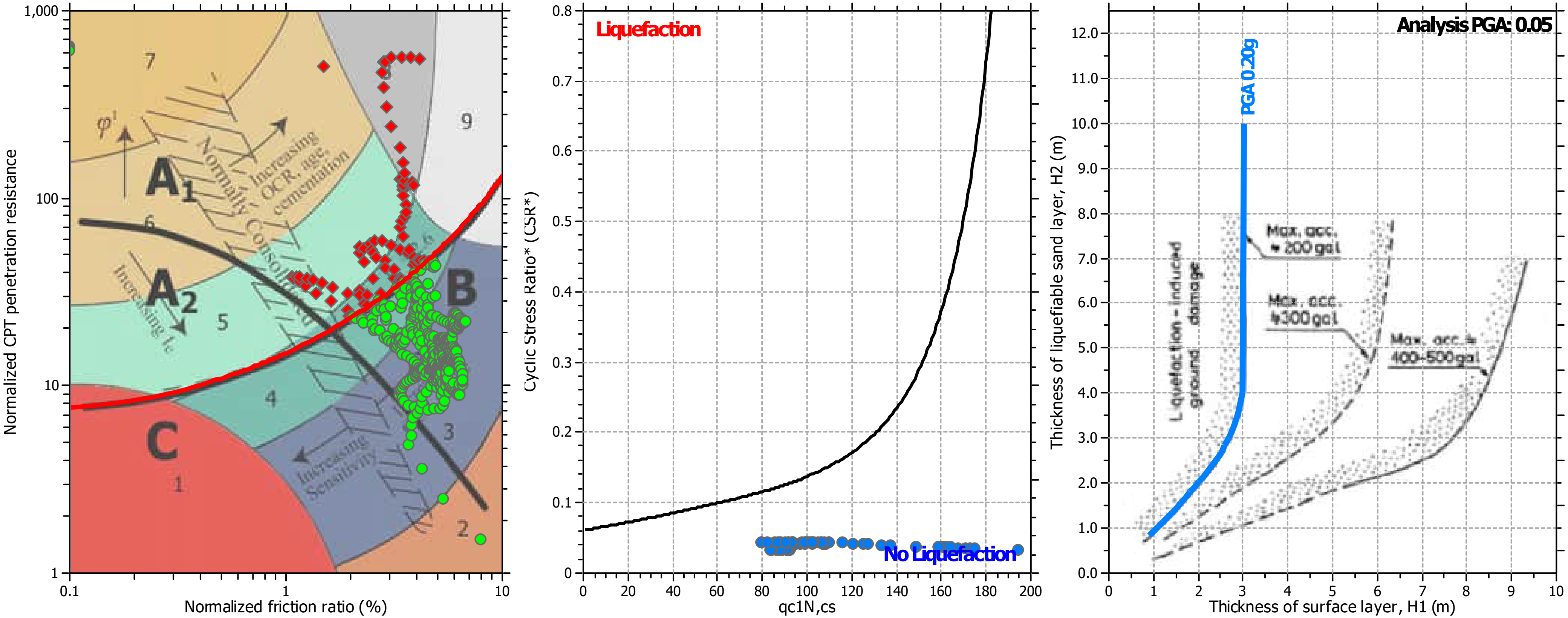
F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.40 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.40 m	Fill height:	N/A	Limit depth:	N/A

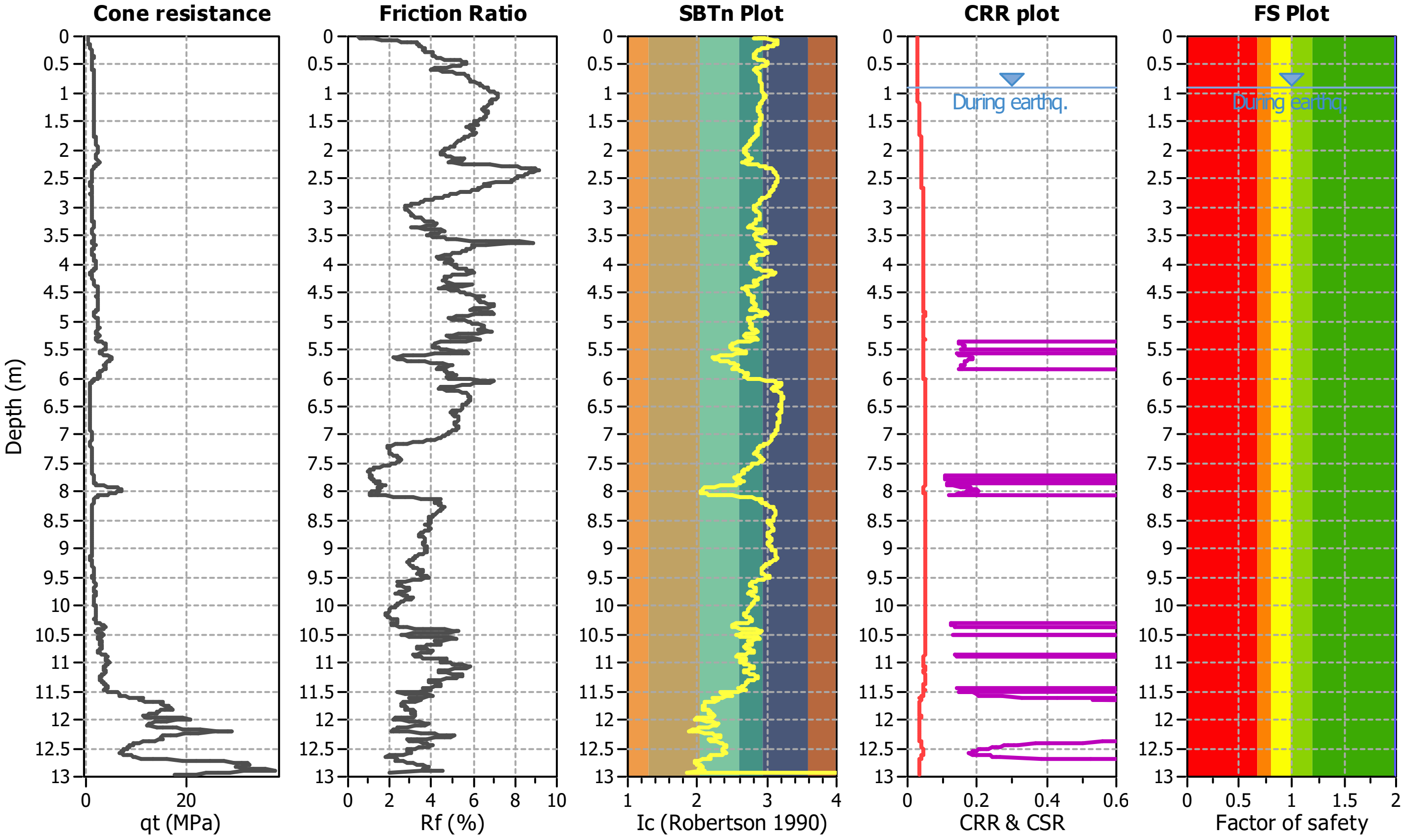
LIQUEFACTION ANALYSIS REPORT

Project title : 16 Sinton Road **Location : Whenuapai**

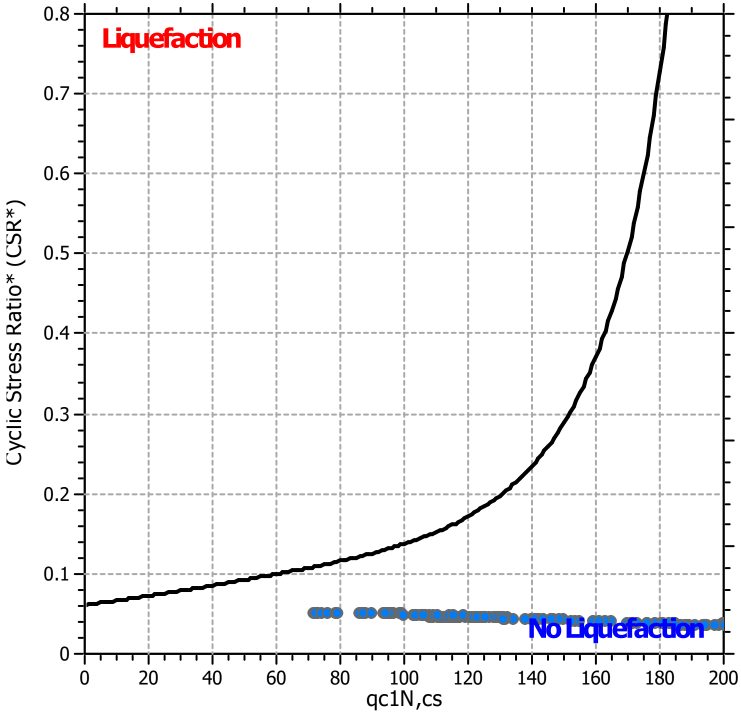
CPT file : CPT02

Input parameters and analysis data

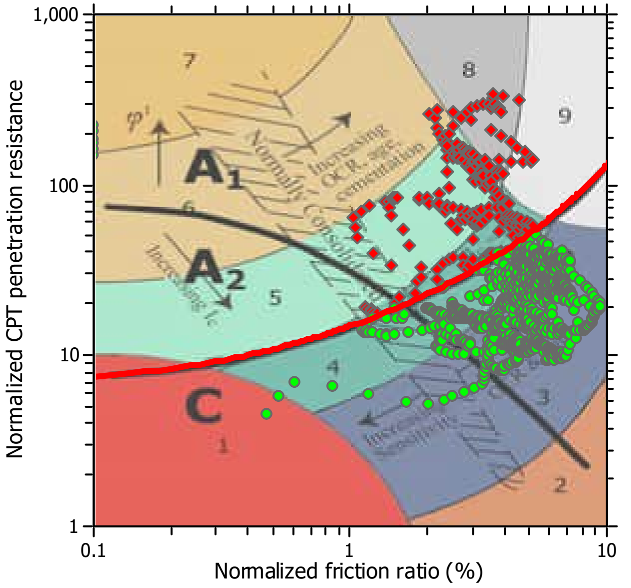
Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.90 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.90 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.05	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



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

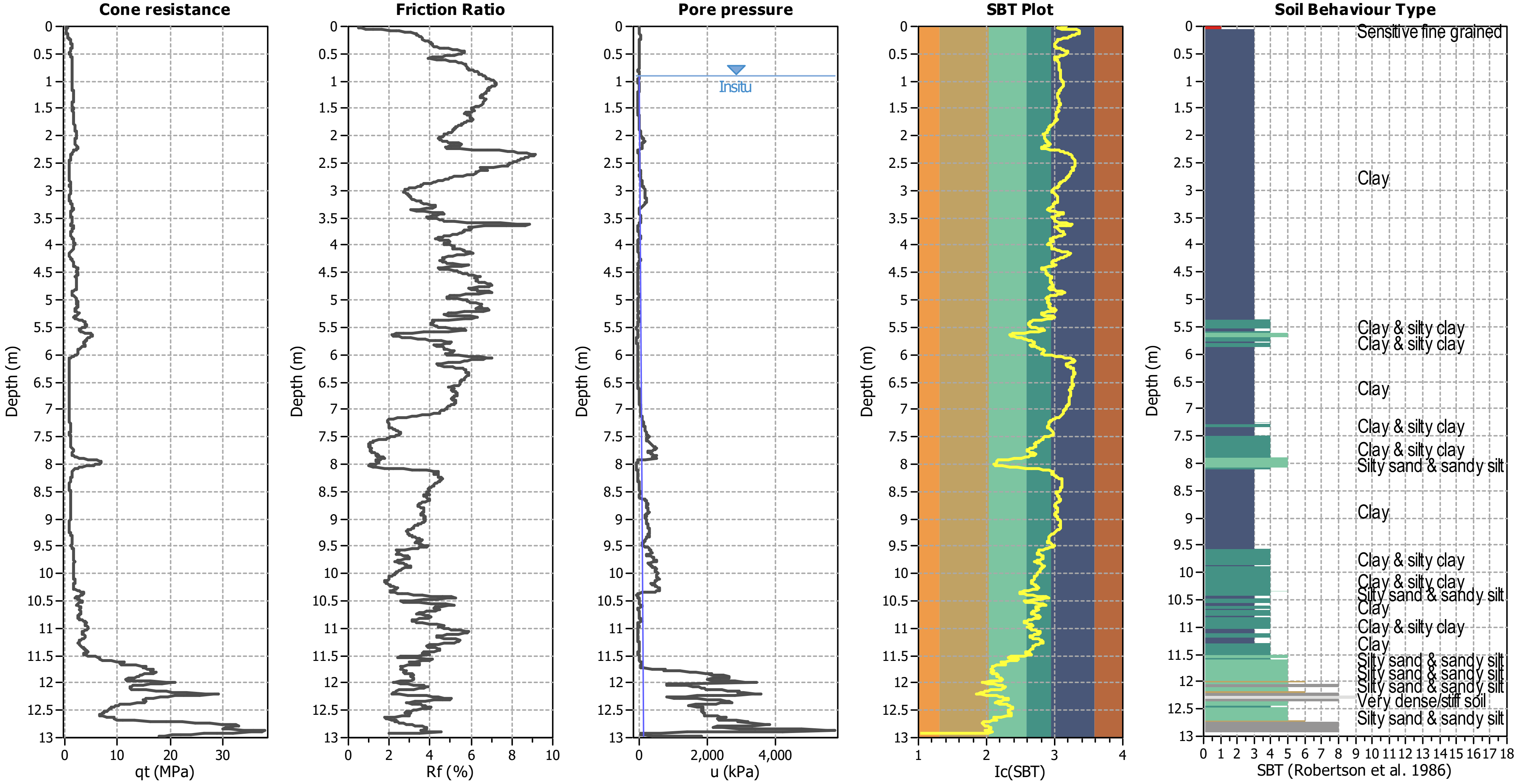


Summary of liquefaction potential



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

CPT basic interpretation plots



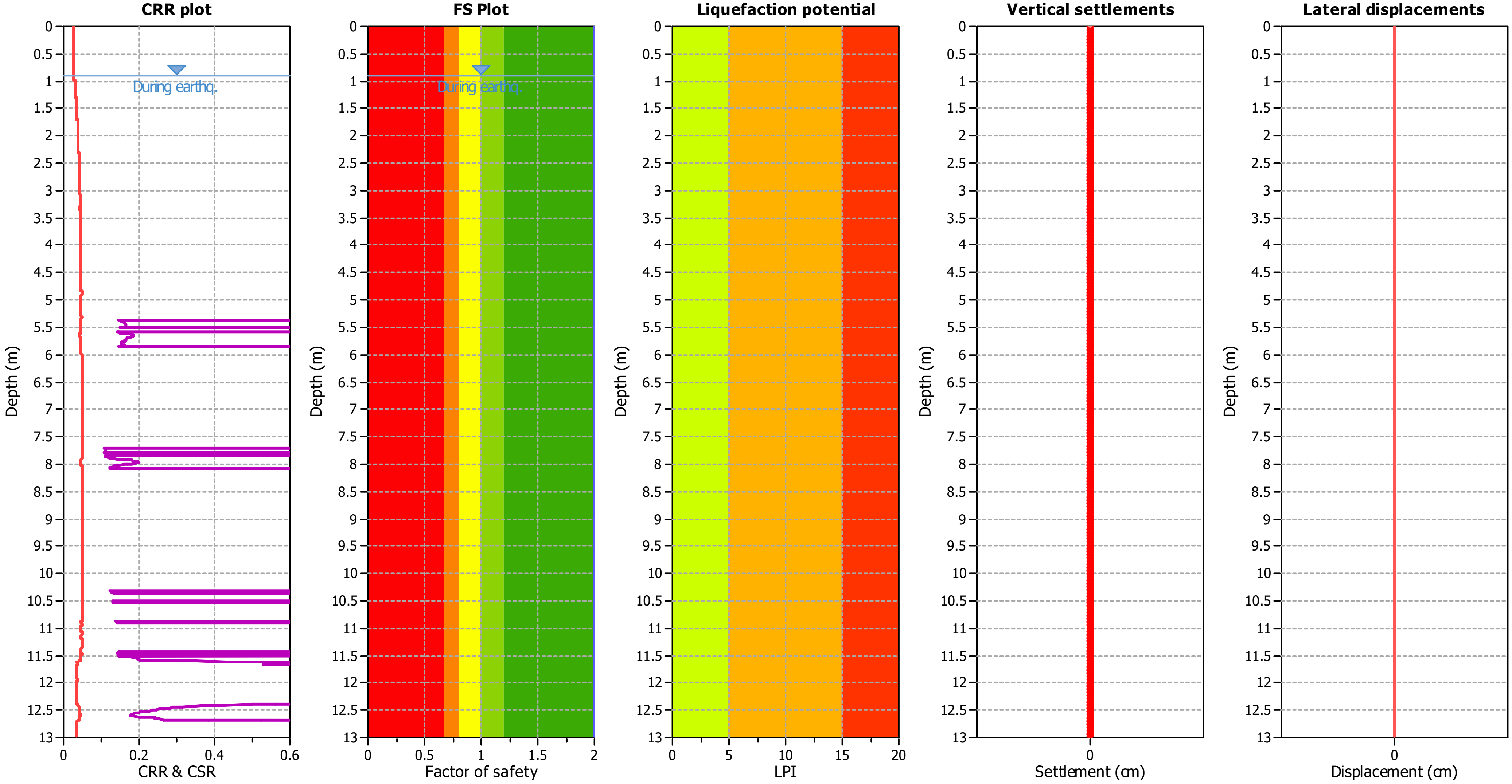
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.90 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	0.90 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.90 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	0.90 m	Fill height:	N/A	Limit depth:	N/A

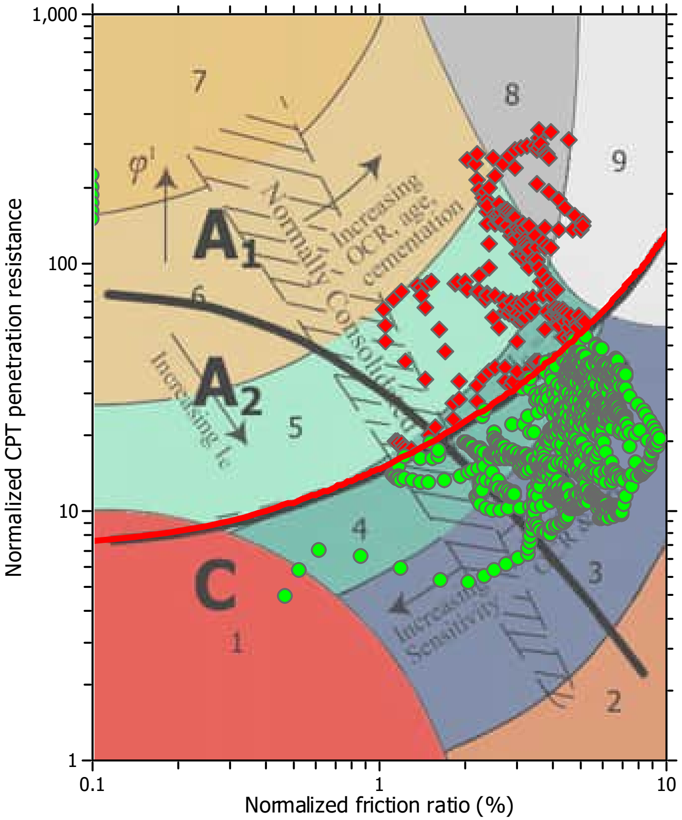
F.S. color scheme

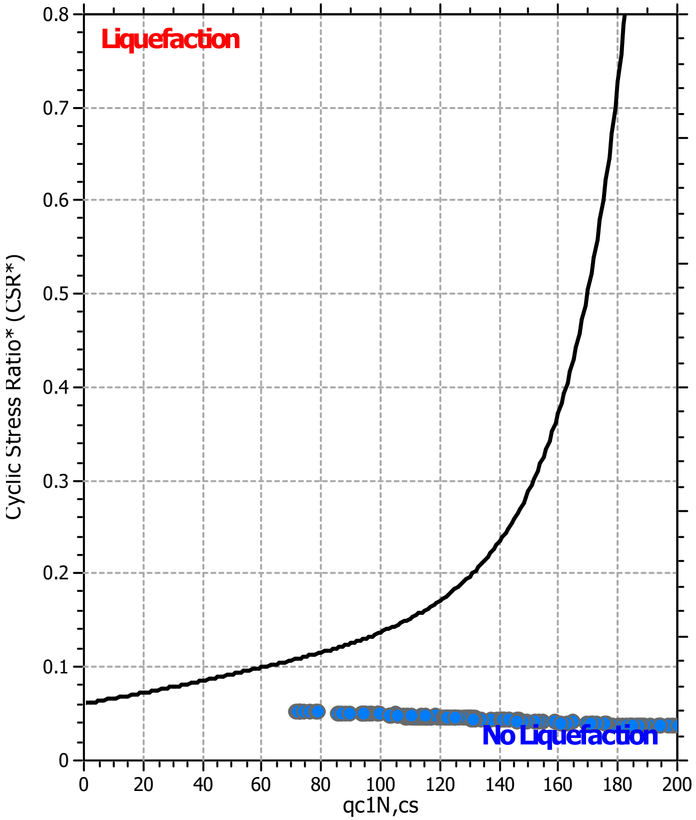
🟥 Almost certain it will liquefy
🟧 Very likely to liquefy
🟨 Liquefaction and no liq. are equally likely
🟩 Unlike to liquefy
🟩 Almost certain it will not liquefy

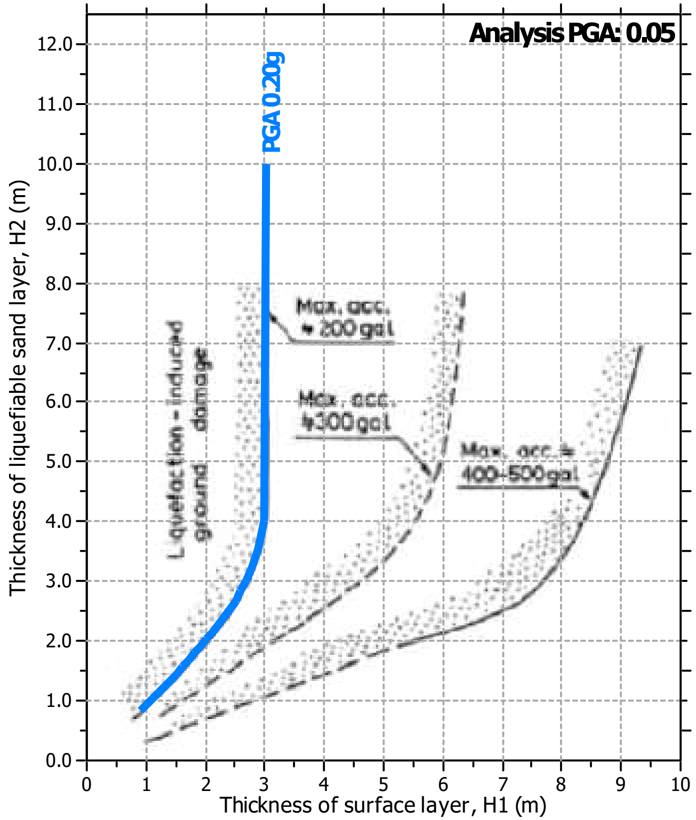
LPI color scheme

🟥 Very high risk
🟧 High risk
🟩 Low risk

Liquefaction analysis summary plots







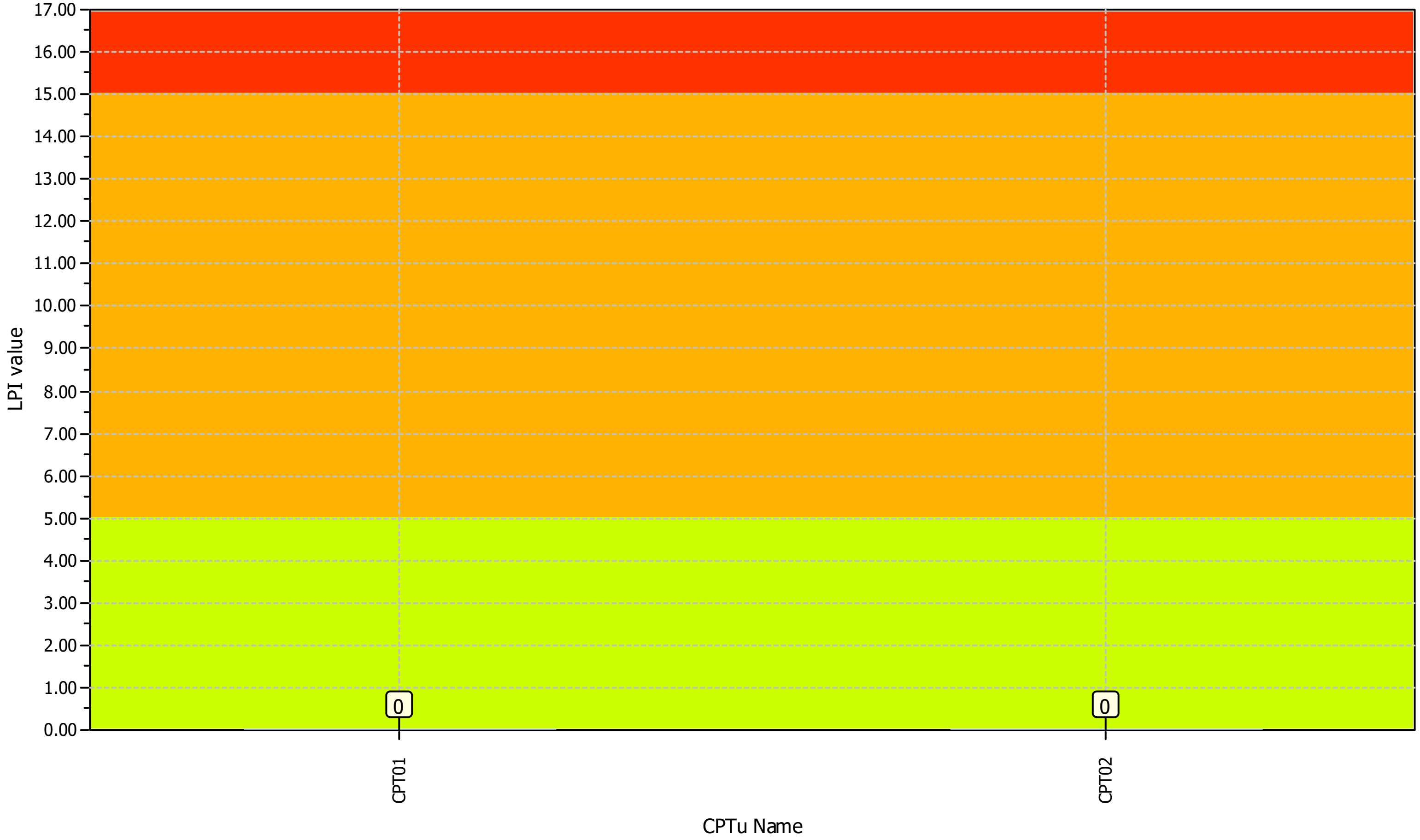
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.90 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.05	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	0.90 m	Fill height:	N/A	Limit depth:	N/A

Project title : 16 Sinton Road

Location : Whenuapai

Overall Liquefaction Potential Index report



LPI color scheme

🟥 Very high risk
🟧 High risk
🟩 Low risk

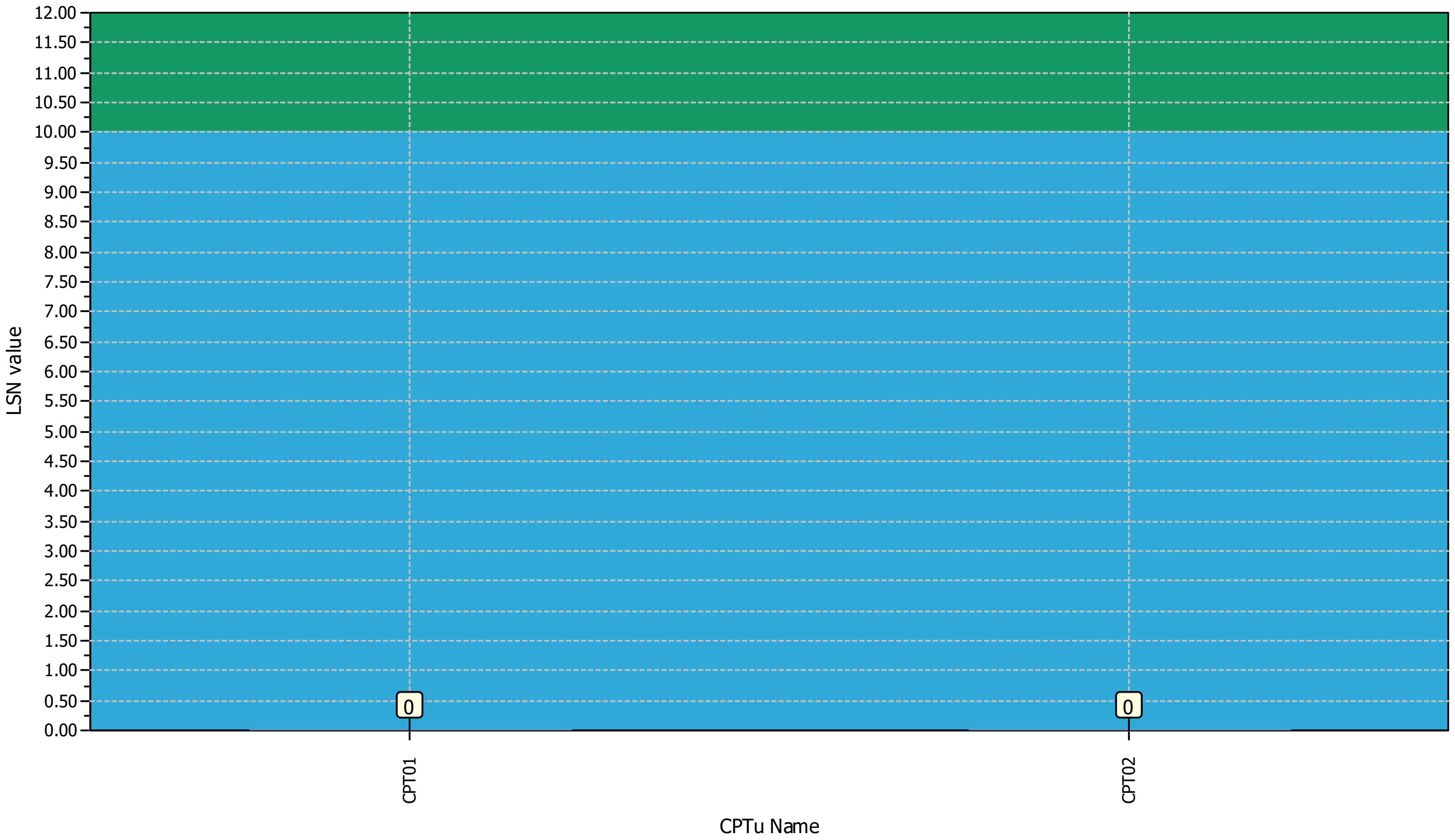
Basic statistics

Total CPT number: 2
100% low risk
0% high risk
0% very high risk

Project title : 16 Sinton Road

Location : Whenuapai

Overall Liquefaction Severity Number report



LSN color scheme

🟥 Severe damage
🟥 Major expression of liquefaction
🟧 Moderate to severe exp. of liquefaction
🟨 Moderate expression of liquefaction
🟩 Minor expression of liquefaction
🟦 Little to no expression of liquefaction

Basic statistics

Total CPT number: 2

100% little liquefaction

0% minor liquefaction

0% moderate liquefaction

0% moderate to major liquefaction

0% major liquefaction

0% severe liquefaction

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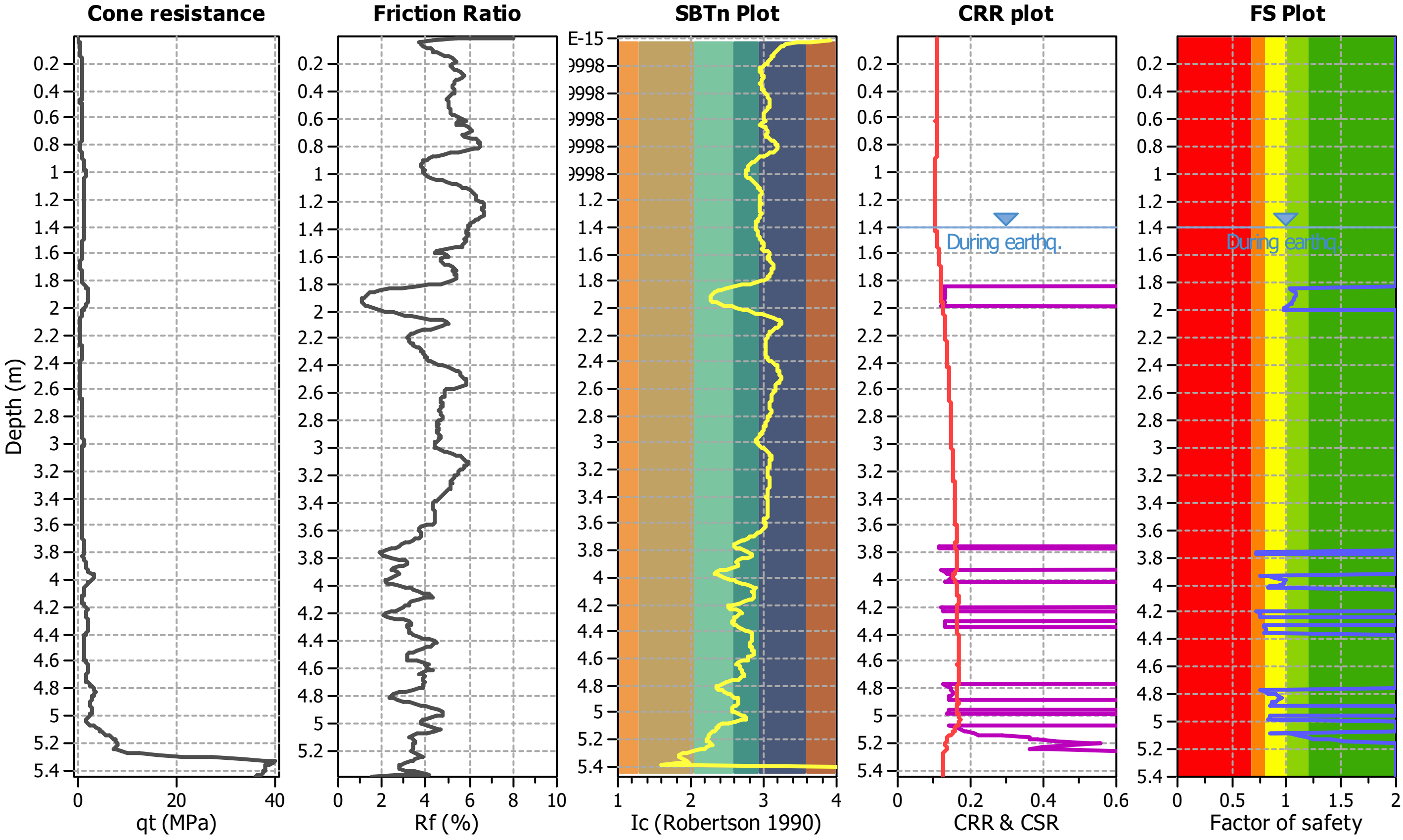
LIQUEFACTION ANALYSIS REPORT

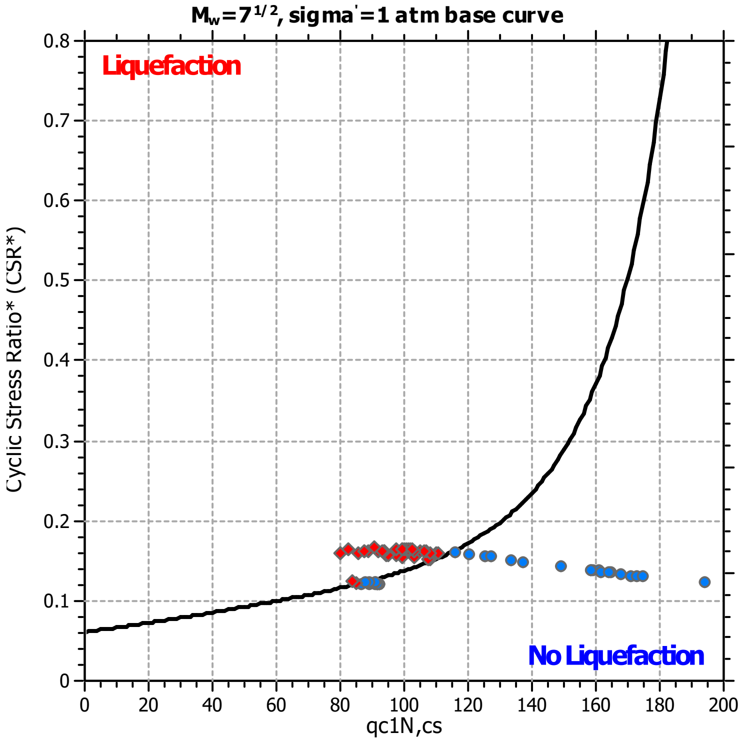
Project title : 16 Sinton Road **Location : Whenuapai**

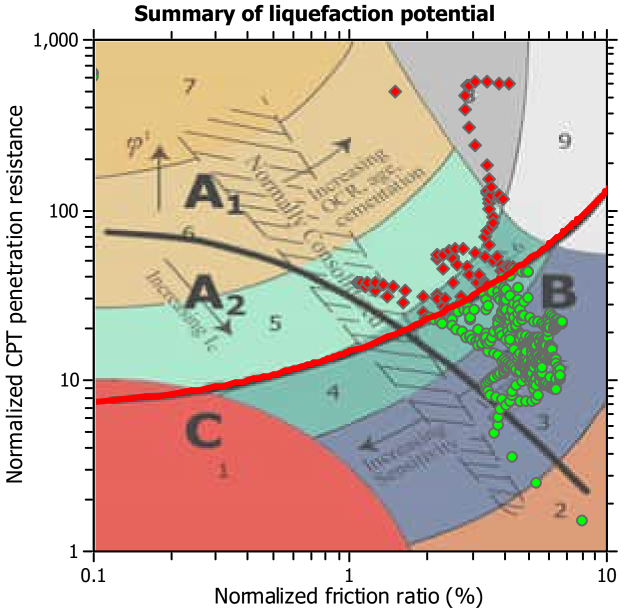
CPT file : CPT01

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.40 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.40 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

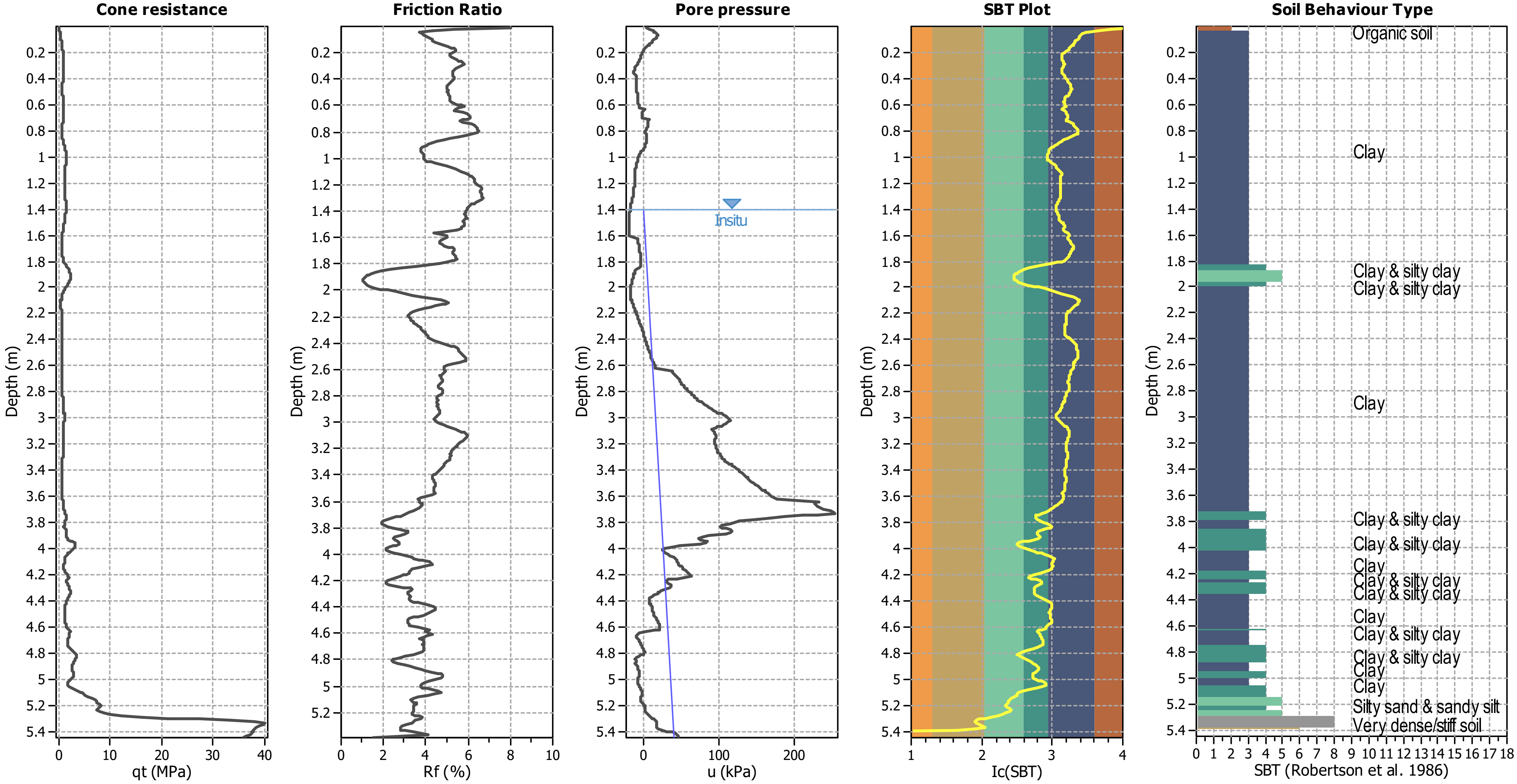






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



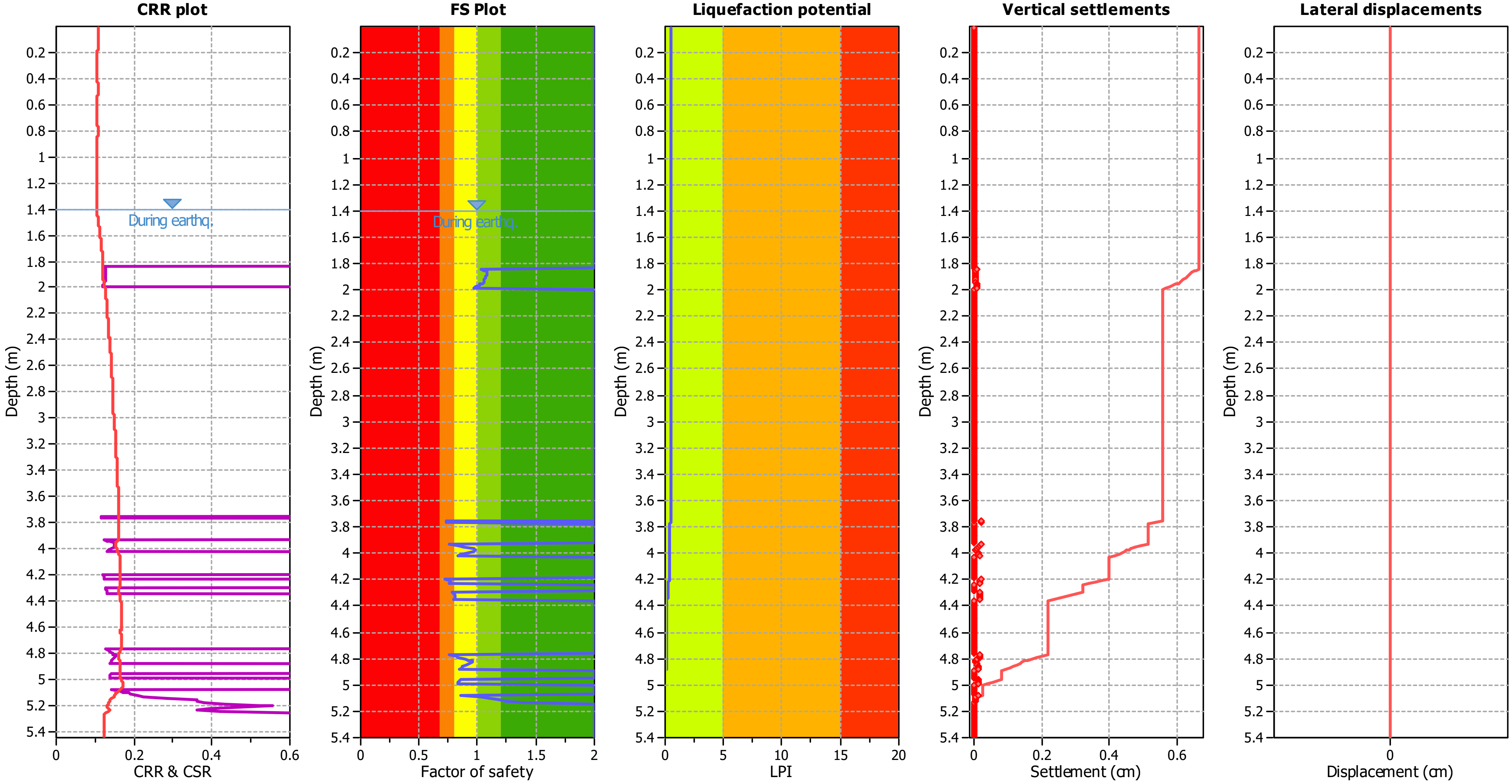
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.40 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.40 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.40 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.40 m	Fill height:	N/A	Limit depth:	N/A

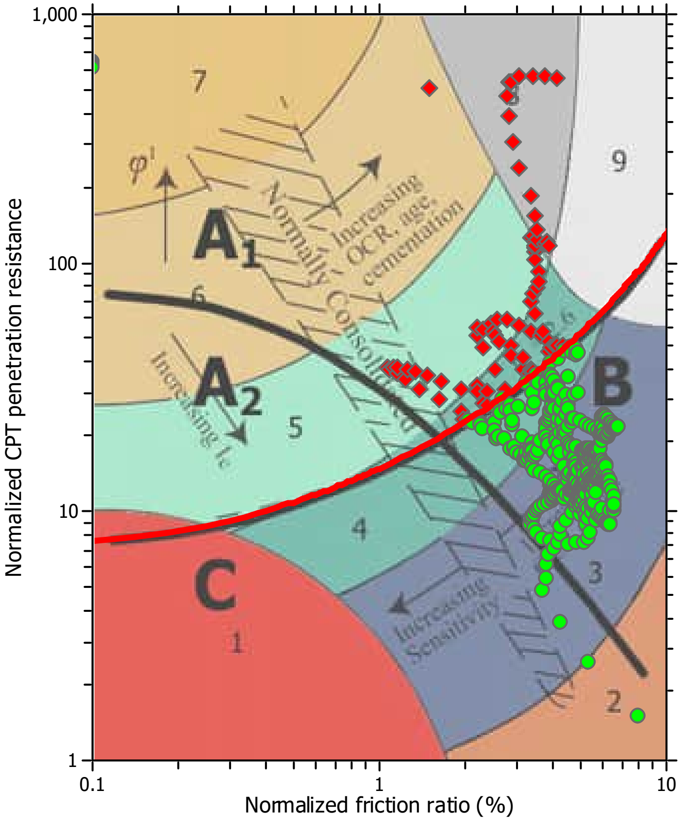
F.S. color scheme

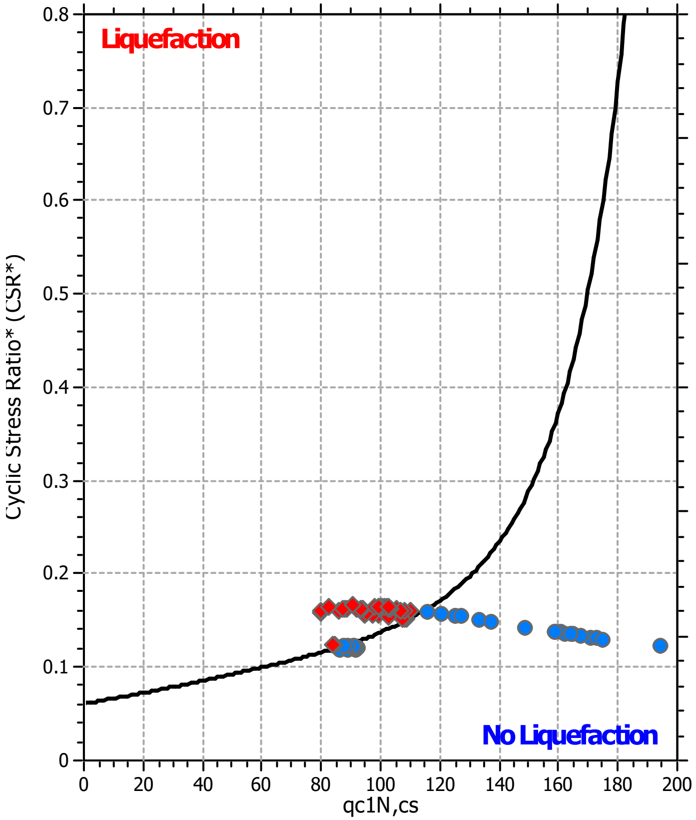
- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

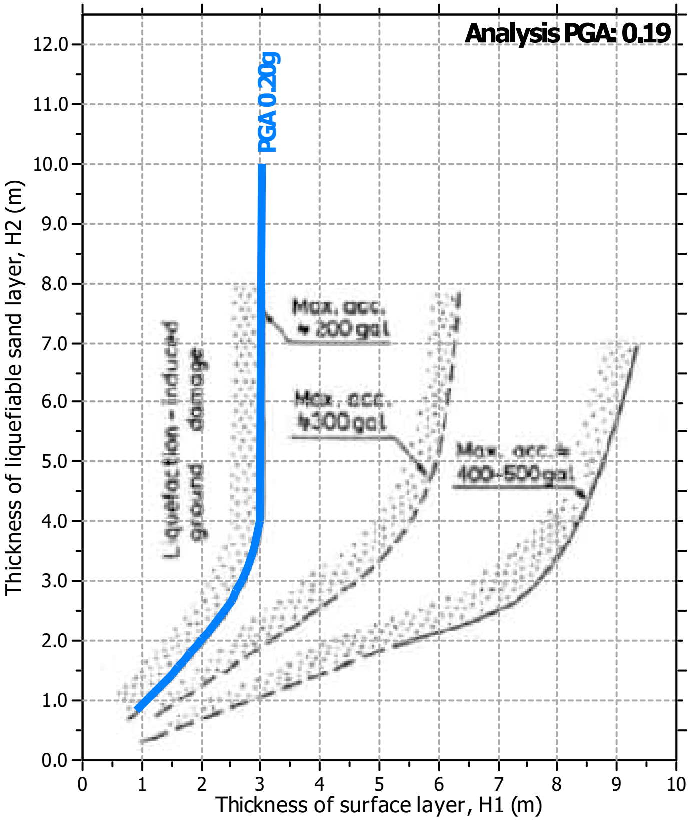
LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟩 Low risk

Liquefaction analysis summary plots







Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.40 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.40 m	Fill height:	N/A	Limit depth:	N/A

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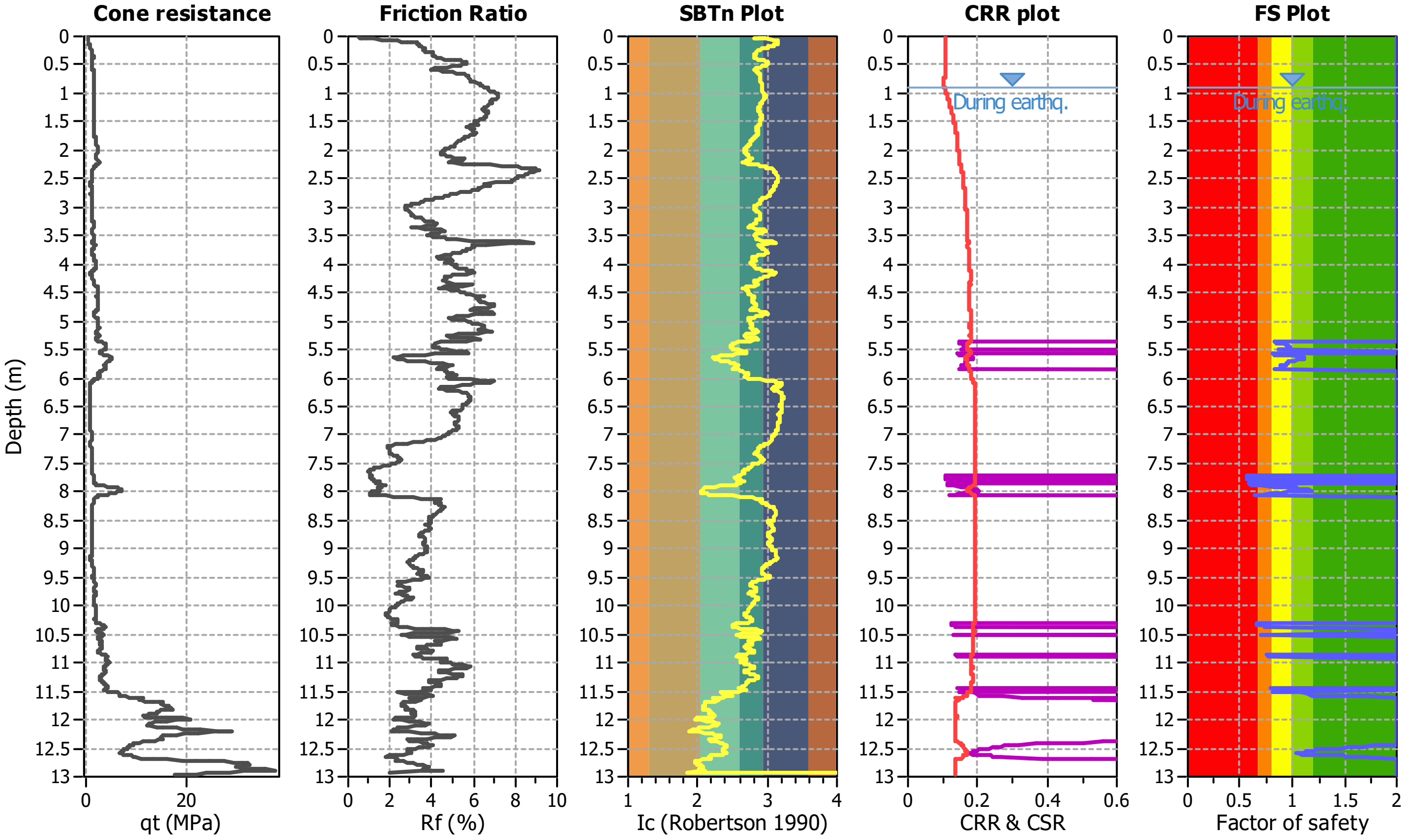
LIQUEFACTION ANALYSIS REPORT

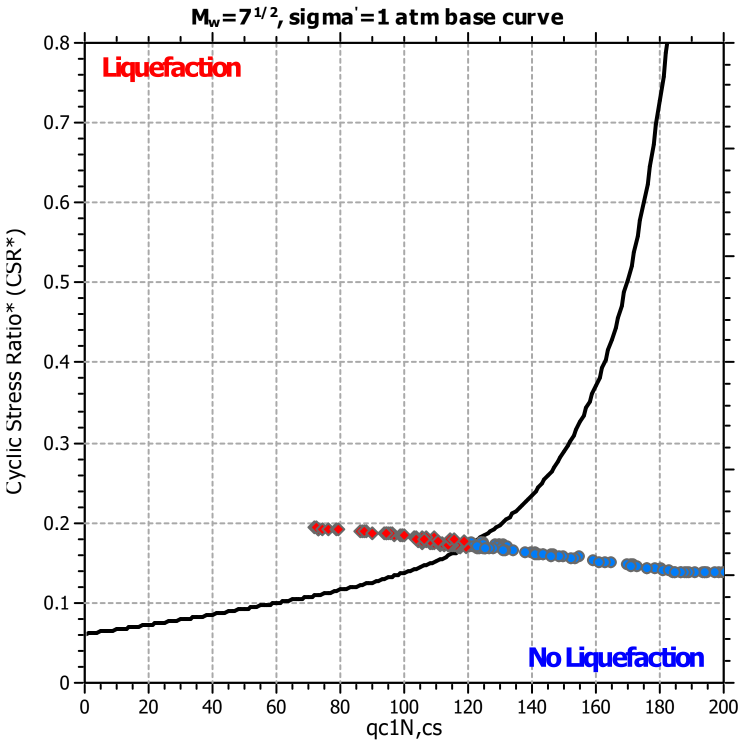
Project title : 16 Sinton Road **Location : Whenuapai**

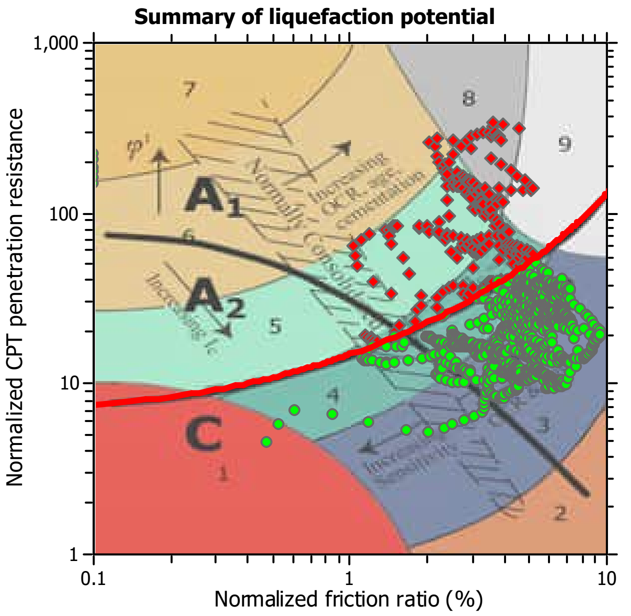
CPT file : CPT02

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.90 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.90 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

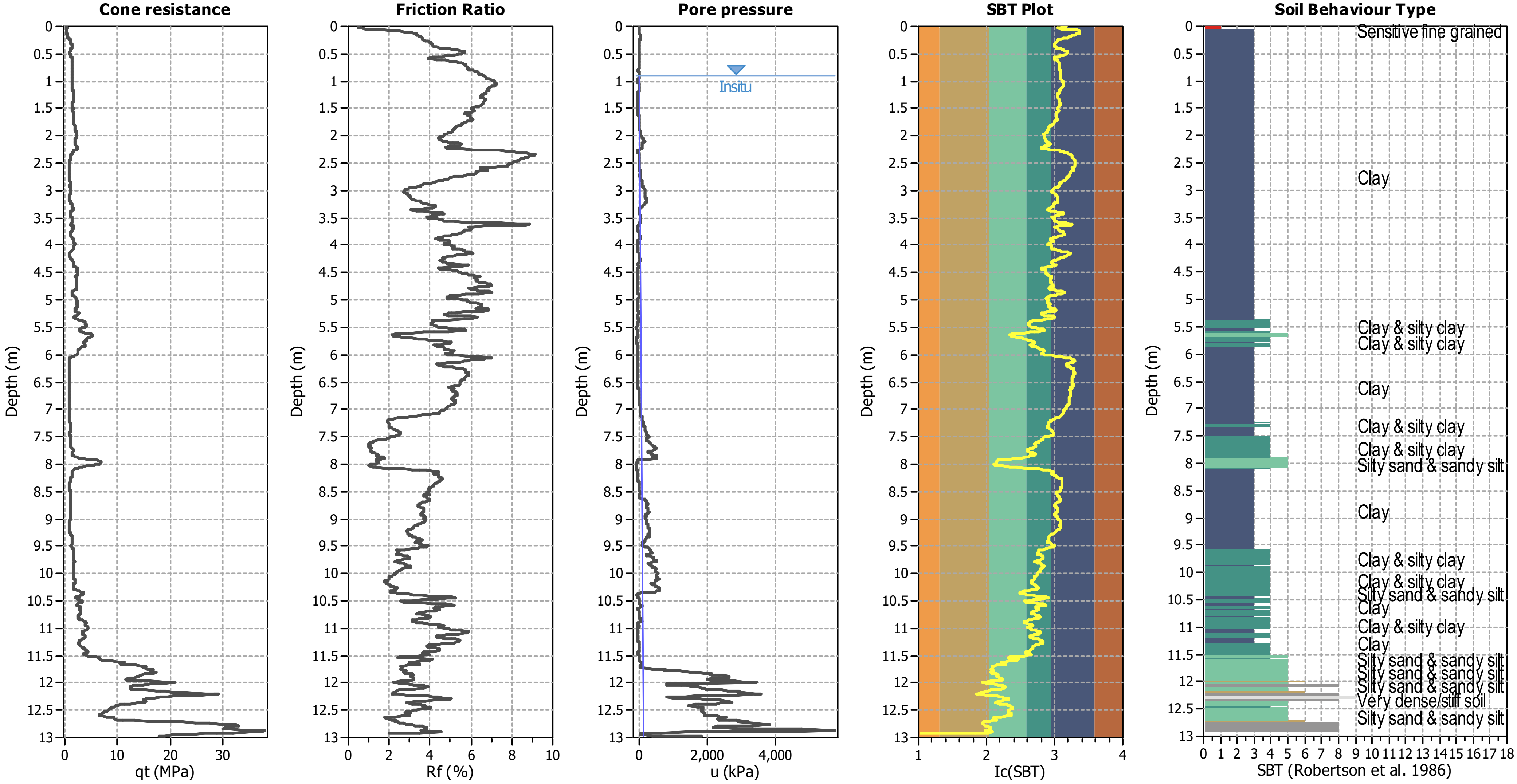






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



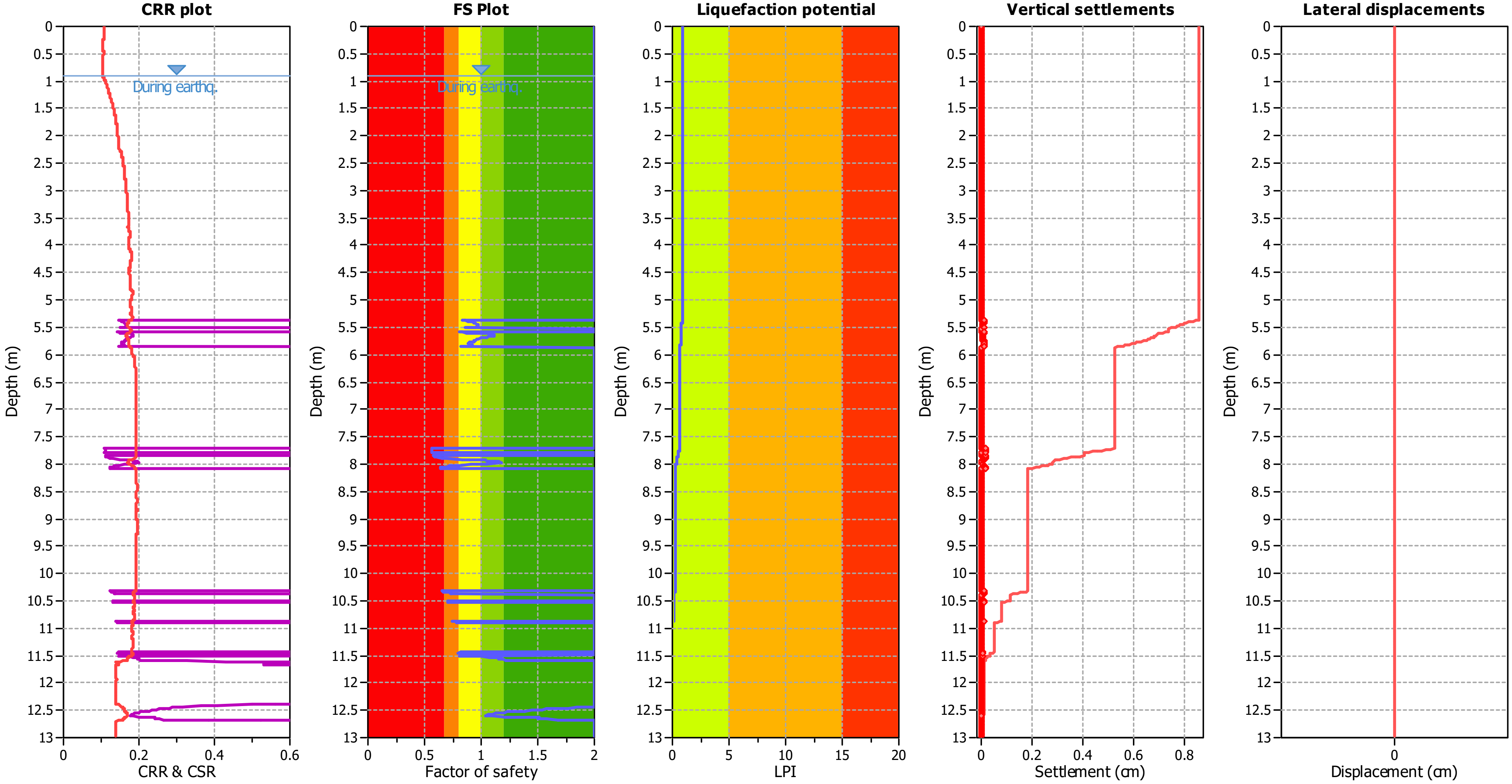
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.90 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	0.90 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



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: 6.50
Peak ground acceleration: 0.19
Depth to water table (insitu): 0.90 m

Depth to GWT (erthq.): 0.90 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: No
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A

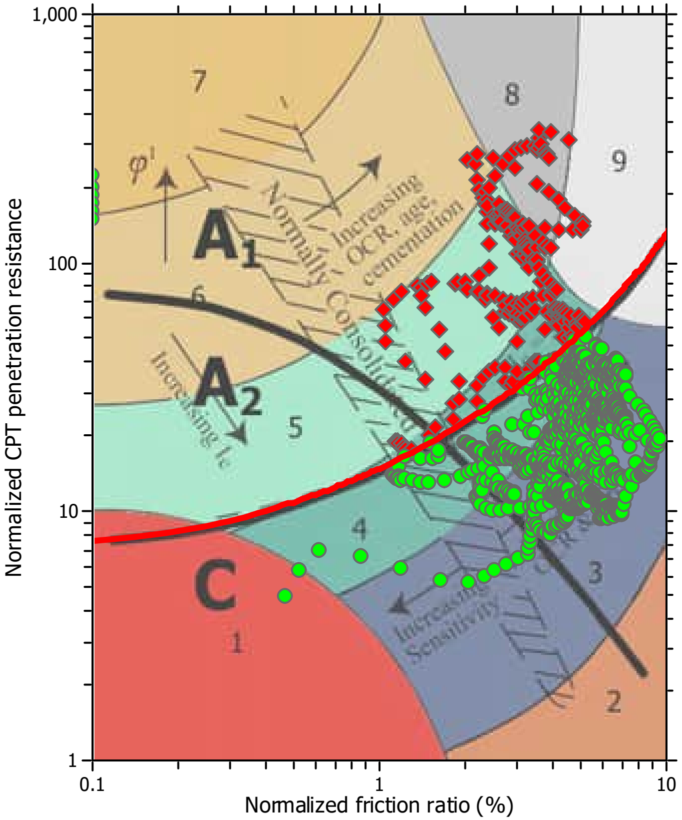
F.S. color scheme

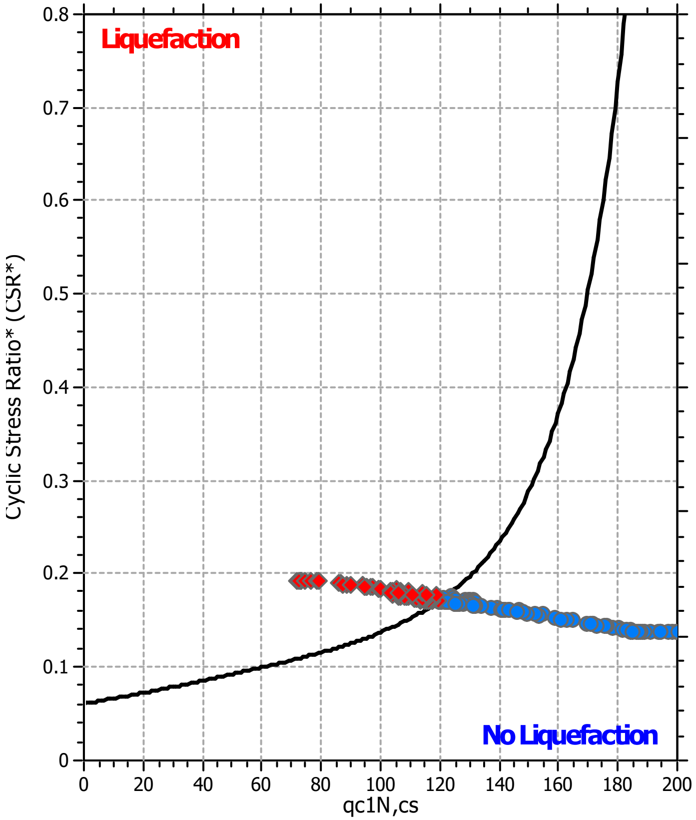
- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

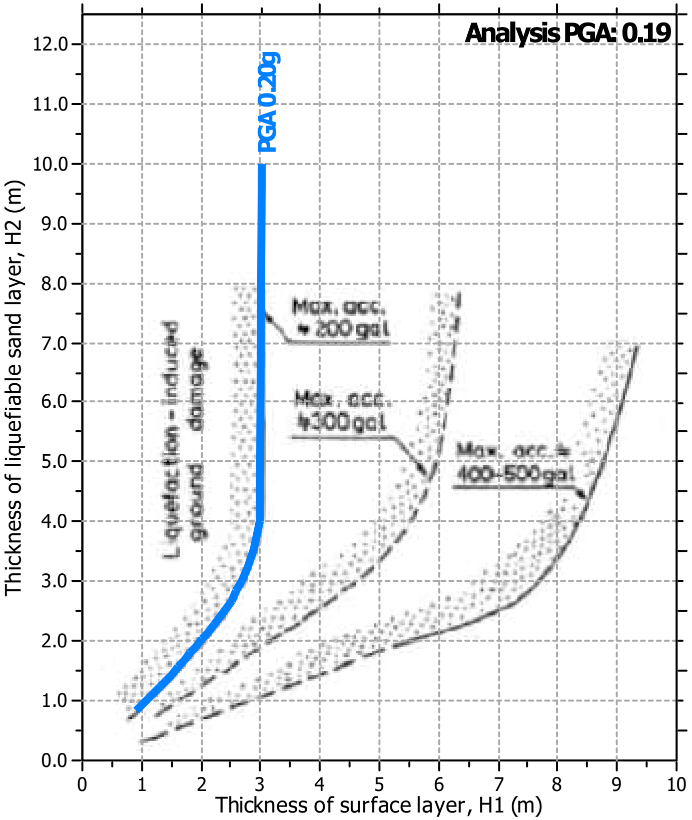
LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟩 Low risk

Liquefaction analysis summary plots







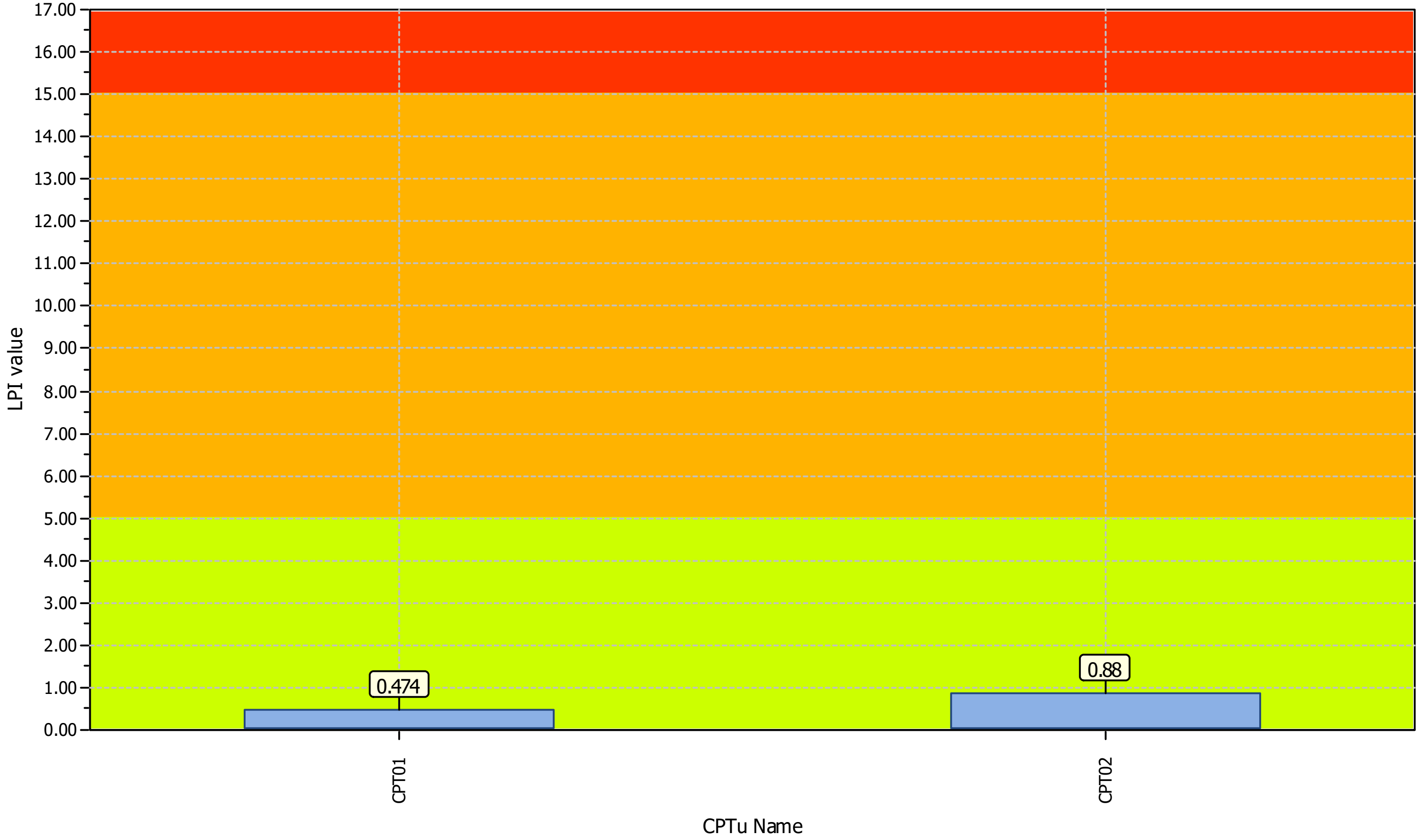
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	0.90 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.19	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	0.90 m	Fill height:	N/A	Limit depth:	N/A

Project title : 16 Sinton Road

Location : Whenuapai

Overall Liquefaction Potential Index report



LPI color scheme

🟥 Very high risk
🟧 High risk
🟩 Low risk

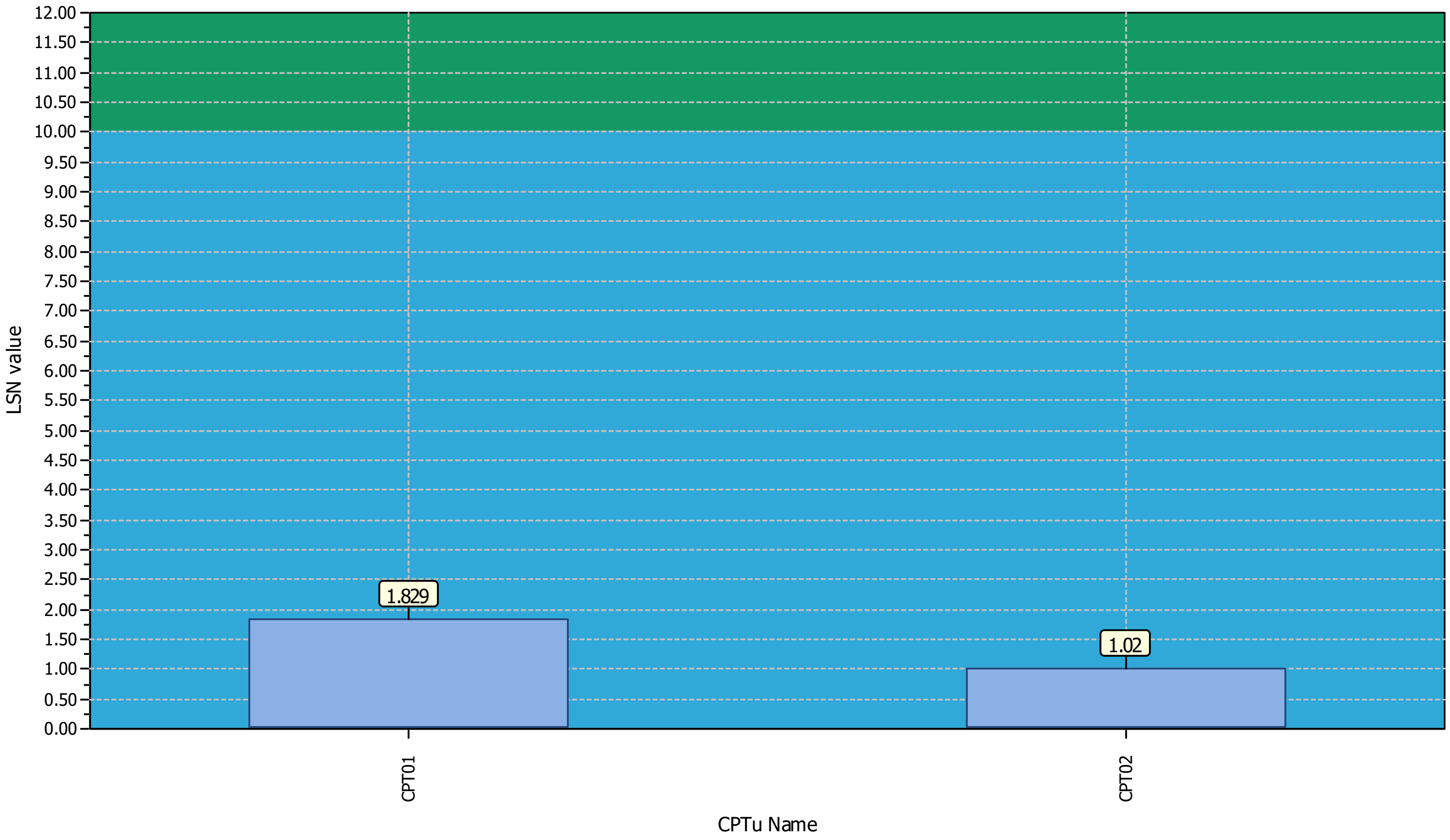
Basic statistics

Total CPT number: 2
100% low risk
0% high risk
0% very high risk

Project title : 16 Sinton Road

Location : Whenuapai

Overall Liquefaction Severity Number report



LSN color scheme

🟥 Severe damage
🟥 Major expression of liquefaction
🟧 Moderate to severe exp. of liquefaction
🟨 Moderate expression of liquefaction
🟩 Minor expression of liquefaction
🟦 Little to no expression of liquefaction

Basic statistics

Total CPT number: 2

100% little liquefaction

0% minor liquefaction

0% moderate liquefaction

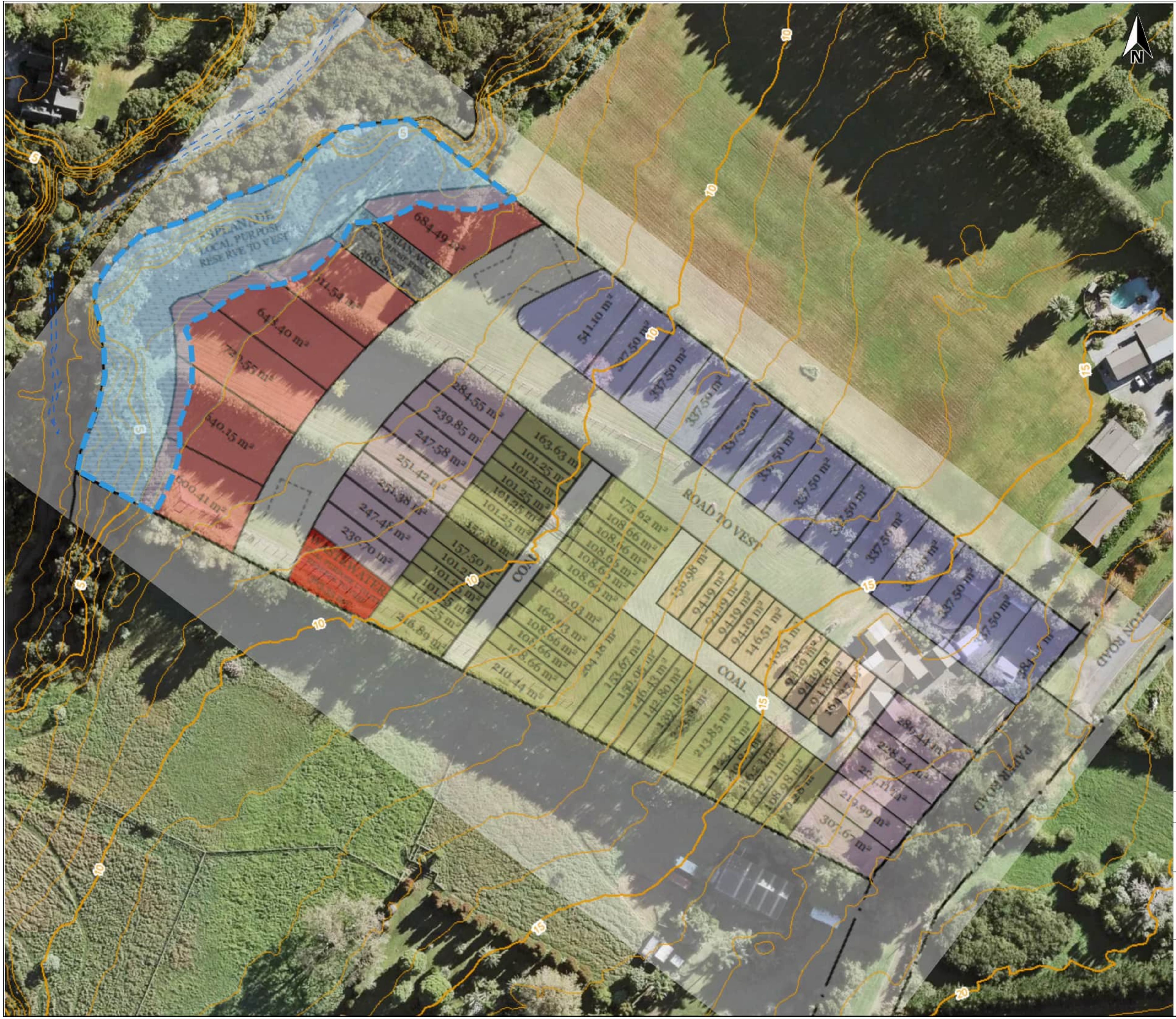
0% moderate to major liquefaction

0% major liquefaction

0% severe liquefaction

APPENDIX 6:

Building Restriction Zone Plan



Legend

▤ Building Restriction Zone

LINZ CC BY 4.0 © Imagery Basemap contributors,

ENGEO

Produced by **Datanest.earth**

Title: Building Restriction Zone

Client:
Cabra Developments Limited

Project:
16 Sinton Road,
Whenupai

Drawn:
JL

Appendix No.:
6

Size: A3

Date:
12-10-2023

Checked: PF

Proj No.:
23849.000.001_04

Scale:
1:1000

Version: 1.0