ASP BEI LAND CO

AUCKLAND SURF PARK COMMUNITY – STAGE 2 FAST TRACK CONSENT

GEOTECHNICAL ASSESSMENT REPORT

INITIA REF P-001537-2 REV D

JUNE 2026

Your Report Summary

This summary outlines the principal geotechnical issues, design considerations and advice for the proposed Auckland Surf Park Community (ASPC) Stage 2 Fast-Track Consent and preliminary design. The summary is intended for our client, ASP Bei Land Co. It is important that all designers and constructors refer to relevant sections in the main body of the report for further detail.

Report Ref	Geotechnical Consideration	Summary Advice/Recommendations
1.2	Report Purpose and Scope	This report has been prepared to support the Stage 2 Substantive Application for the ASPC Project. It provides commentary on all precincts except for the Surf Lagoon which was addressed in the previous Stage 1 reporting and Fast-Track Consent.
66.2	Subsurface Conditions	The site is underlain by topsoil and localised uncontrolled fill over Tauranga Group alluvium, underlain by a weathering profile of Mahurangi Limestone Rock. The depth to Limestone Rock varies across the site but is typically 2 m or less on the elevated hillsides (at ground elevations above approximately RL 56 m) and between 5 – 12 m within the central low-lying 'core'. The alluvium typically comprises firm to stiff SILT and CLAY mixtures, with organic CLAYs and PEAT prevalent at the eastern end of the central low-lying 'core'.
6.4	Groundwater	A complex groundwater regime is present at the site which includes: - Shallow perched groundwater (of limited depth and volume) generally sitting within or marginally below the topsoil in the central core; - Static groundwater sitting within the alluvium in the central core, typically between approximately RL 51 to 52 m, grading down to RL 49 m at the existing creek; and - Inferred percolating groundwater in the fractured rock mass within the hillsides, flowing towards the low-lying central core.
7.2	Site seismicity & liquefaction susceptibility	The Seismic Subsoil Class varies between Site Class B (Rock) and Site Class C (Shallow Soil) across the site, depending on the soil thickness above the Mahurangi Limestone Rock. Liquefaction effects at the site are insignificant and are not expected to influence the design of structures.
7.3	Earthworks	The site-won soils will have variable suitability for re-use as engineered fill. Inorganic alluvium and residual soils should be suitable but will require drying/conditioning/blending prior to placement. Working platforms will likely need to be formed to enable fill compaction above low strength alluvium. Placement of fill above existing ground levels will initiate consolidation settlement. Hold periods will need to be observed between bulk filling and the construction of structures and infrastructure in areas where alluvium is present. Ground improvements, comprising the construction of engineered fill bearing crusts and/or preloading, are likely to be required to enable shallow foundations to be adopted for proposed buildings across the site. It is recommended these specific ground improvements be integrated into the bulk earthworks scope to leverage scale efficiencies and minimise re-work requirements during construction of the buildings.
7.4	Groundwater drawdown considerations	The proposed bulk earthwork cuts will likely intercept perched groundwater and may locally intercept static groundwater. However, due to the low permeability of the soils and significant offset from neighbouring structures and infrastructure, adverse offsite groundwater drawdown settlement effects are expected to be negligible. As groundwater is expected to be intercepted, the works fall outside the definition of a permitted activity in accordance with E7 of the Auckland Unitary Plan. Accordingly, a groundwater consent may be required. The offsite effects associated with groundwater take are assessed as negligible and accordingly no specific groundwater or settlement monitoring is considered necessary as part of the consent conditions.

Report Ref	Geotechnical Consideration	Summary Advice/Recommendations
7.5	Slope stability	Suitable batter slope profiles are recommended in Section 7.5 for cut depths and fill heights in various soil types. Deep cuts into the Mahurangi Limestone in the Northen and Southern Solar Farms should be mapped/monitored by a geologist during construction to verify rock mass stability. The proposed fill embankments within the North-West Neighbourhood Precinct will require specific engineering design to mitigate instability risk. Stabilisation works will likely involve either palisade walls, 'keying-in' of fill and/or geogrid reinforced slopes. Further assessment/design will be required for detailed earthworks design. A landslide hazard assessment has been completed in accordance with Appendix 24 of PC120. It is concluded that the landslide hazard at the site is low, provided the recommendations in this report are adhered to.
7.6	Retaining Walls	Retaining walls will be required to accommodate grade changes about the site and on boundaries. Retaining types are likely to comprise a combination of gravity (e.g. keystone block, MSE wall) and embedded pile (e.g. timber pole, steel UC) walls. Mechanical settlement effects on neighbouring properties and existing buildings and infrastructure are expected to be negligible.
7.7	Civil Infrastructure	Construction of civil infrastructure (road pavements, buried services etc) will need to be programmed with consideration for fill induced settlements and hold periods. Trench subgrades in the alluvium are expected to be low strength. Subgrade improvement layers are likely to be required below bedding materials. Excavation of deep trenches in the alluvium may be unstable. Buried infrastructure depths should be limited as much as possible.
7.8	Pavements	Engineered fill, residual soils and Mahurangi Limestone rock will likely form suitable pavement subgrades. However, these materials are sensitive to trafficking and wet weather. Subgrades should therefore be prepared during dry weather periods and protected with sacrificial layers. Subgrades formed in alluvial soils will likely require improvement comprising dig-out-and-replacement or possibly lime/cement stabilisation.
7.9	Building Foundations	Shallow foundations (likely raft foundations) are expected to be appropriate for most light-weight structures. However, ground improvements will be required in some areas of the site to provide sufficient shallow bearing strength and to mitigate settlements where thick layers of alluvium are present. Ground improvements will likely comprise the construction of a engineered fill crust of specified thickness and preloading. Heavier structures (e.g. Data Centres and Hotel) will require piled foundations bearing in Mahurangi Limestone Rock. It may also be cost-effective to pile other structures in lieu of completing ground improvements to enable shallow foundations.
7.10	Further Work	Further geotechnical investigation, analysis and reporting will be required to design the engineered fill embankments in the North-West Neighbourhood Precinct, design retaining walls and design ground improvements and foundations for future structures. It is recommended that ground improvement design is completed ahead of bulk earthworks so that ground improvements for future buildings can be integrated into the bulk earthworks scope.

Contents

1. Introduction

1.1 General

This Geotechnical Assessment Report (GAR) has been prepared to support the Auckland Surf Park Community (ASPC) Stage 2 Substantive Application at 1320 and 1350 Dairy Flat Highway, 89 and 105 Lascelles Drive, 237 and 253 Postman Road, Dairy Flat. The application is being progressed under the Fast Track Approvals Act 2024 (FTAA) and comprises both:

- The expansion of the ASPC to include a hyperscale artificial intelligence data centre campus, three residential developments, subdivision, a village centre, work-live precinct, and ancillary activities.

- Section 127 variations to Stage 1 of the development consented under the COVID-19 Fast-track Consenting Act 2020.

The advice in this report has been prepared with reference to the development plans referenced below. If development layouts, levels or grades change, Initia should be given the opportunity to review and update the advice in this report accordingly:

- Development Masterplan: Studio Pacific Architecture Illustrative Masterplan[1]. Layout ID RC_1.01. Dated 14/04/2026.
- Civil Earthworks Drawings: McKenzie & Co. Earthworks Final Contour Overall Plan[2]. Rev B. Dated 8/12/2025.

A statement of the author's qualifications and experience is presented in Appendix H.

1.2 Scope of Reporting

Initia previously prepared a Geotechnical Interpretative Report[3] (dated June 2023) to support the Fast-Track Resource Consent and preliminary design of Stage 1 of the ASPC. The 2023 report provides geotechnical advice for the surf lagoon, amenity and accommodation buildings, northern solar farm, and associated civil works at 1350 Dairy Flat Highway (this site has since been subdivided).

A review of the updated Stage 2 Masterplan[1] and Stage 2 Draft Earthworks Drawings[2] indicates the general location, extent and levels for the proposed surf lagoon remains generally consistent with the existing Stage 1 Consent. The site levels and development layout for the remainder of the project (including the buildings from the Stage 1 Masterplan) has been amended.

On this basis, this GAR provides updated and supplementary advice to support Fast-Track Consenting and preliminary design for all precincts in the Stage 2 Masterplan, <u>excluding the surf lagoon and associated bulkhead structures</u>. Designers should refer to the Stage 1 Geotechnical Interpretative Report[3] for preliminary geotechnical advice in relation to the lagoon.

Further geotechnical investigation, analysis, and reporting will be required to support detailed design of all structures, retaining walls, infrastructure, stabilised embankments, and ground improvements.

This report should be read in conjunction with the Initia Geotechnical Factual Report[4] which presents all factual geotechnical investigation data for the site.

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[1] Studio Pacific Architecture (14/04/2026). *Auckland Surf Park Community. Masterplan. Illustrative Masterplan.* Layout ID RC_1.01 Job No. 4663.
[2] McKenzie & Co (8/12/2025). *Earthworks Final Contour Overall Plan.* Dwg No. 3325-2-2000. Rev B.
[3] Initia Ltd (June 2023). *AW Holdings 2021. Auckland Surf Park Community. Geotechnical Interpretative Report.* Ref P-001537. Rev B.
[4] Initia Ltd (19/12/2025). *Auckland Surf Park Community. Geotechnical Factual Report.* Ref: P-001537. Rev F.

2. Site Description

The project site of approximately 54 Ha encompasses the properties at 1320 & 1350 Dairy Flat Highway, 237 & 253 Postman Road and 89 & 105 Lascelles Drive, Dairy Flat. The site is bounded by Dairy Flat Highway to the west, Postman Road to the east and rural-residential properties to the north and south.

A low-lying, typically flat 'core', with ground levels typically between RL 54–55 m, is present in the centre of the site. This core is flanked by northern, eastern, and southern wings that rise gently at gradients of approximately 1V:5H to 1V:20H. Maximum elevations occur near the site boundaries with levels of about RL 66 m to the north, RL 65 m to the east, and RL 60 m to the south.

A primary creek traverses the northern end of the central core, flowing east to west with invert levels ranging from RL 53 m (east) to RL 49 m (west) before discharging through a culvert beneath Dairy Flat Highway. A secondary creek runs approximately north–south along the western edge of the northern wing, joining the main creek. Historical aerial imagery indicates this channel was modified or cut as a farm drain. The creek banks are generally gently sloping (<1V:4H) but steepen up to approximately 1V:2H in local areas above the secondary channel.

The site is currently grassed farmland with associated farm infrastructure and with six residential dwellings and associated on-site effluent disposal systems. Several sheds and shelters are also present. A horse arena at the base of the northern wing has been formed on fill. Elsewhere, historic earthworks are inferred to be minor, limited to small cuts and fills for platforms, access tracks, and farm drains or ponds.

Bulk earthworks for the surf park were commenced under the Stage 1 Consent late 2024 and involved topsoil stripping and filling within the central core and bulk cutting in the northern wing.

Figure 2-1 below shows the site boundary, original ground contours and pertinent site features.

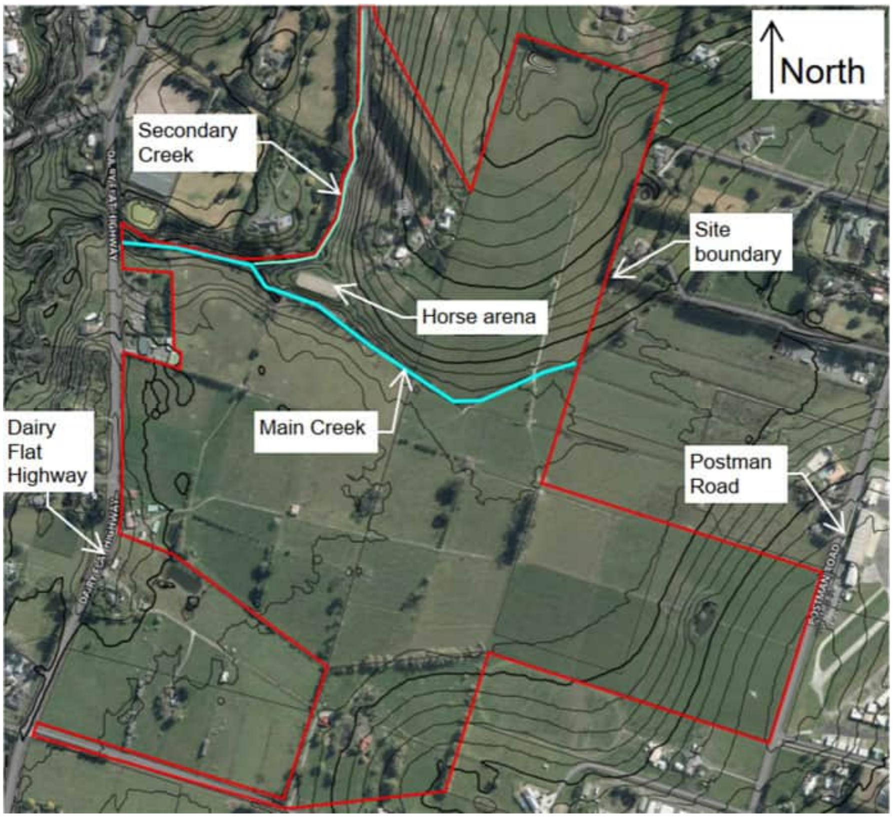

Figure 2-1: Site boundary and pertinent features (1 m contours shown)

3. Proposed Development

The Masterplan for the ASPC Development is presented in Figure 3-1 below. It shows the development will include the following core elements across the 54 Ha development:

1. Surf Lagoon: A proposed 2.3 Ha, 1 to 3 m deep body of water arrayed around a curved 'beach'. The lagoon will be formed largely below existing ground levels and will include a central bulkhead structure and a servicing building at the northern end of the lagoon. <u>The Surf Lagoon is outside of the scope of this GAR.</u>

2. Surf Amenity Precinct: The Surf Amenity Precinct supports the Surf Lagoon operations and includes 5 No. single storey buildings and 1 No. two storey restaurant located to the south and west of the surf lagoon. An 8 storey hotel building is also proposed to the south-east of the lagoon. On-grade carparking will be provided around the southern and western perimeters of the precinct.

3. Accommodation Precinct: The accommodation precinct is located to the north of the lagoon and includes a two storey clubhouse building and approximately 60 No. single storey accommodation villas. The villas are proposed to include terraced units, duplexes and standalone units. The northern most villas are proposed to be elevated on posts above a low-lying flood storage area. The remaining buildings are proposed to be founded on-grade on higher ground.

4. Village Centre Precinct: The Village Centre is proposed to include 3 No. four storey apartment buildings, 4 No. three storey apartment buildings and single storey market, food & beverage, wellness centre and early childhood buildings. The precinct also includes on-grade parking and internal roading.

5. Neighbourhood Precincts (North-West, North-East and South): Three separate residential neighbourhood precincts are proposed, totalling approximately 13ha. It is understood the residential neighbourhoods will include 1 – 3 storey standalone and duplex dwellings with associated roading and green space.

6. Live-Work Precinct: The Live-Work Precinct is to include approximately 25 No. two storey terraced units with both home-office and living spaces. On-grade parking is also proposed. The southern end of the Live-Work precinct is reserved for Water and Wastewater infrastructure to support the wider development.

7. Solar Farms (North, Centre and South): Three separate solar farms are proposed in disparate locations, with a total area of approximately 8.5ha.

8. Data Centres: Two separate Data Centre Buildings are proposed at the eastern end of the project. It is understood these will likely comprise Importance Level 4 structures.

9. Stage 5A – Vacant Lot Subdivision: Vacant, undeveloped subdivision, east of the Data Centre buildings. Future development type not determined.

10. Stream Park: A green space including walking trails, stormwater ponds and flood storage areas is proposed through the centres of the site. A vehicle bridge is to provide access across the existing creek.

Figure 3-1: Stage 2 Masterplan Summary (Source: Studio Pacific Architecture 06/2026)

Civil earthworks plans[5] indicate that, apart from the lagoon, bulk earthworks are expected to comprise filling of up to 3 m in the central core of the site to lift levels to between RL 55 and 57 m (c.f. existing typical site levels of RL 54 – 55 m). Bulk cuts of up to 6.5 m below existing ground level are proposed within the elevated northern and southern solar farms and northern neighbourhood precincts to win fill materials. Filling up to approximately 4 m in height is proposed across the western edge of the North-West Neighbourhood Precinct to re-grade the existing sloping ground.

A series of retaining walls, up to approximately 4 m in height, are proposed across the site including abutments for the proposed vehicle bridge.

[5] McKenzie & Co (8/12/2025). *Earthworks Final Contour Overall Plan.* Dwg No. 3325-2-2000. Rev B.

4. Published Geology

The published geological map for the area is presented on Figure 4-1 below and indicates that the existing central core of the site is underlain by Tauranga Group, Middle Pleistocene to Late Pleistocene river and hill slope deposits. These soils are typically described as pumiceous sand, silt, mud and clay with interbedded gravel and peat.

Mahurangi Limestone (Motatau Complex) of the Northland Allochthon, is mapped as the surficial unit on the elevated areas in the northern, eastern and southern wings. Mahurangi Limestone is typically described as blue-grey to white, micritic, coccolith foraminiferal, muddy limestone, commonly with thin glauconitic sand.

Hukerenui Mudstone (Mangakahia Complex) in Northland Allochthon is mapped approximately 400 m north-west, 500 m west and 1,500 m east of the site.

The site-specific investigations are typically consistent with the published geology, with Tauranga Group alluvial deposits of varying thickness across the full extent of the site. The Tauranga Group alluvium was underlain by weathered Mahurangi Limestone soils and rock which was proven in some areas (via test pits or machine boreholes) or inferred via hand auger, Scala or CPT refusal. No other materials of the Northland Allochthon were visually encountered or proven at the site.

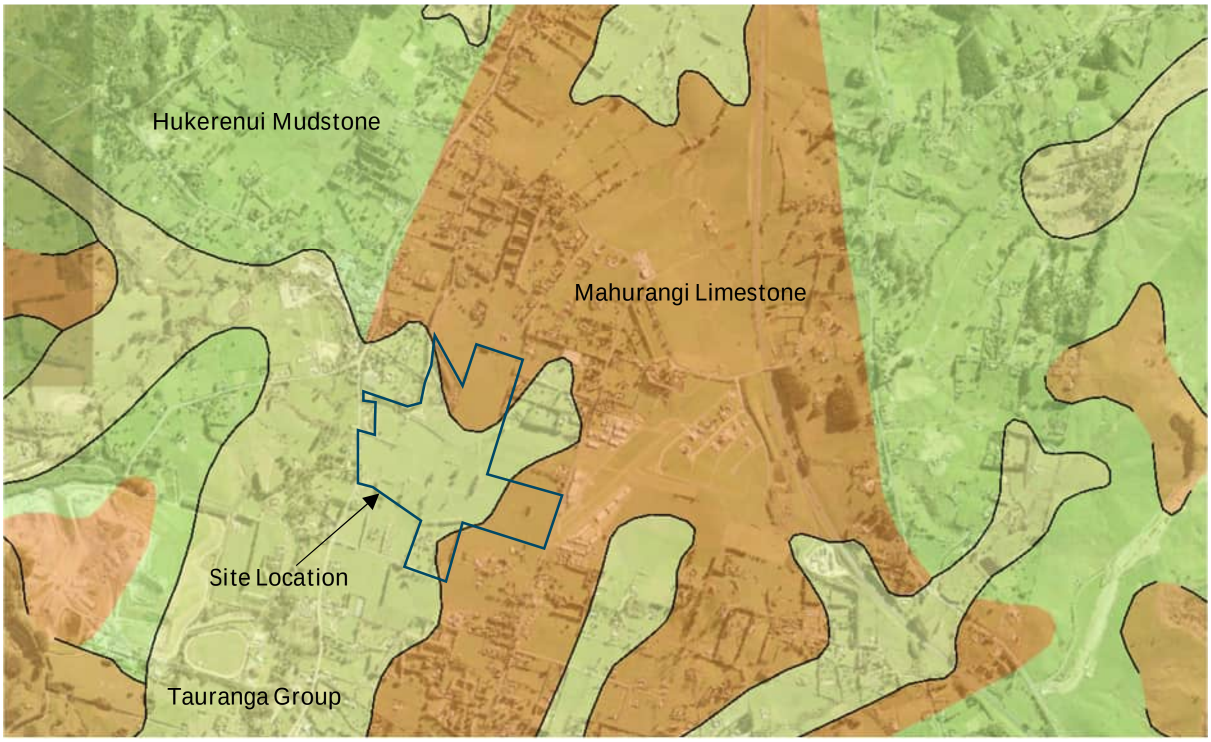

Figure 4-1: Geological Map of the Area. Source (GNS Science)

5. Geotechnical Investigations

Four stages of geotechnical investigations have been undertaken by Initia and others across the development site. All investigation details and logs are presented separately in the site-wide Initia Geotechnical Factual Report[6]. A brief description of the purpose and scope of each stage of geotechnical investigation is presented below.

All investigation locations are presented on the Initia site investigation plans in Appendix A.

Additional investigations will be required to support the developed and detailed design stages.

Tonkin & Taylor Pre-Purchase Assessment – 2022

A geotechnical investigation was undertaken by Tonkin & Taylor[7] (T+T) in May 2022 as part of a pre-purchase assessment for due diligence of 1350 Dairy Flat Highway, 237 Postman Road and 253 Postman Road. The geotechnical investigations included:

- 23 No. hand auger boreholes with Scala Penetrometer (DCP) testing advanced from the base of the holes; and
- 6 No. machine excavated test pits

Standpipe piezometers were installed within 6 No. of the hand auger boreholes. These piezometers have either been destroyed or are unable to be located.

Aurecon Data Centre Investigations - 2023

A geotechnical investigation was undertaken by Aurecon[8] in April 2023 to support the design of the proposed western Spark Data Centre at 237 Postman Road and 253 Postman Road. The investigations included:

- 15 No. Cone Penetration Tests
- 3 No. Seismic Dilatometer Tests
- 8 No. Hand Auger Boreholes; and
- 8 No. Machine Drilled Boreholes.

Standpipe piezometers were installed within 6 No. of the machine boreholes. A suite of laboratory tests were undertaken including Atterberg Limits, Linear Shrinkage, Particle Size Distribution, Shrink-Swell Index, Soaked CBR, CU Triaxial and One Dimensional Consolidation.

Initia Stage 1 Investigations - 2023

Geotechnical investigations were undertaken by Initia between 20th March 2023 and 5th April 2023 to support the preliminary design and Fast Track Consent of the "Stage 1" Surf Park Community Masterplan. The 2023 field investigations (referred to herein as the Initia Stage 1 investigations) included:

- 5 No. Machine Boreholes;
- 25 No. Cone Penetration Tests;
- 11 No. Hand Auger Boreholes; and
- 31 No. Machine Excavated Test Pits.

Standpipe piezometers were installed within 5 No. machine boreholes and were monitored using level loggers between 21/04/2023 to 7/06/2023. A suite of laboratory tests were undertaken including

[6] Initia Ltd (19/12/2025). *Auckland Surf Park Community. Geotechnical Factual Report.* Ref: P-001537. Rev F.
[7] Tonkin & Taylor, June 2022 – 1350 Dairy Flat Highway – Geotechnical report to support Due Diligence & Concept Design. Rev 1, Job Number: 1016502.1000.v01
[8] Aurecon (1 June 2023). *Spark Dairy Flat Campus. Geotechnical Investigation report.* Ref: 523573. Revision A.

Standard Compaction, Soaked CBR, Atterberg Limits, Linear Shrinkage and One Dimensional Consolidation.

Initia Stage 2 Investigations – 2025

Supplementary geotechnical investigations were undertaken by Initia between 5 August 2025 and 22 August 2025 to support the preliminary design and Fast Track Consent of the “Stage 2” Masterplan. The purpose of the investigations was to:

- Investigate the newly acquired land parcels, namely 1320 Dairy Flat Highway, 89 Lascelles Drive and 105 Lascelles Drive; and
- Investigate existing Stage 1 areas that were previously designated as “green space” but now designated to include buildings under the Stage 2 Masterplan.

The investigations completed in Stage 2 included:

- 43 No. Hand Auger boreholes (of which 41 No. included Scala Penetrometer testing at the base of the hole);
- 39 No. Cone Penetration Tests; and
- 10 No. Machine Excavated Test Pits.

It is noted that, due to stakeholder and access restrictions, investigations within the newly acquired land parcels were limited to hand augers and CPT testing. Additional investigations (likely to be inclusive test pits and/or machine boreholes) should be conducted for future design stages.

Laboratory testing was undertaken on select disturbed samples and included Atterberg Limits, Linear Shrinkage, Standard Compaction, Soaked CBR, Organic Content and Water Content tests.

The Stage 2 investigations were undertaken following the first season of bulk earthworks at the site, undertaken under the Stage 1 consent. Some investigations were undertaken in areas already modified by earthworks. Affected investigations are summarised on Table 5-1 below.

Table 5-1: Summary of Stage 2 investigations completed in areas modified by season 1 earthworks

Type of Earthworks at Investigation Location	Affected CPTs	Affected Hand Augers	Affected Test Pits
Topsoil stripped and engineered site won-fill placed (i.e. investigation completed on a fill platform)	CPT121 CPT122 CPT123 CPT124 CPT125 CPT126 CPT127 CPT133 CPT137	HA142 HA143	TP104 TP105 TP110
Investigation completed on the hardfill temporary haul-road	CPT120 CPT128 CPT132 CPT138	-	-
Investigation completed above stockpiled “unsuitables” (generally stripped topsoil and organic soils)	-	-	TP109 TP107

6. Subsurface Conditions

6.1 General

The geotechnical model presented in this report is based on available information obtained from geotechnical investigations at point locations completed by Initia and others. The nature and continuity of the subsoil conditions away from the investigation locations is inferred and it must be appreciated that the actual soil conditions may vary from the assumed model.

As outlined in Table 5-1 above, earthworks have commenced within the previously consented Stage 1 development area. Several recent (Stage 2) investigations were undertaken in zones already modified by these works. The geological models presented on the stratigraphy tables and geological cross sections in Appendix A are presented relative to the original (pre-earthworks) ground conditions. A drone survey of the site was completed by McKenzie & Co on 3 June 2025 following the 2024–2025 earthworks. The recent survey has been overlain on the Initia geological sections, alongside the original ground surface, for illustrative purposes.

Section 6.2 below presents a summary of the geological units encountered across the site. Below this, stratigraphy tables are presented for each development precinct. Geological sections through each precinct are presented in Appendix A.

6.2 Geological Units

The results of the geotechnical investigations indicate that the site is underlain by the following geological units:

1. Topsoil and localised uncontrolled fill; over
2. Tauranga Group Alluvium; over
3. Weathered Mahurangi Limestone (Motatau Complex) soils and rock of the Northland Allochthon.

An overview of the geological units is summarised below.

6.2.1 Topsoil and Uncontrolled Fill

Topsoil is present throughout the site except in localised paved areas, with thicknesses ranging from 0.1 to 1.0 m. It is generally thickest within the flat, low-lying central areas. The material typically comprises dark brown organic SILT with minor clay content.

Existing fill, associated with past farming and minor development, was encountered locally. The fill composition varies and includes GRAVEL, SILT, and CLAY of varying density and stiffness. Where present, fill is generally less than 1 m thick, except beneath the horse arena at the south-western corner of the North-West Neighbourhood Precinct, where thicker fill (inferred to be 1–3 m) is expected. The composition of this fill is not known.

6.2.2 Tauranga Group Alluvium

Alluvial soils of the Tauranga Group were encountered within all investigations across the site.

The thickness of the alluvium varies with ground surface elevation. On the northern, eastern, and southern wings, where ground levels exceed RL 56 m, the alluvium is typically less than 2 m thick. Within the central low-lying basin, it increases to 5–11 m thick, infilling a paleo-depression.

The alluvium generally comprises firm to stiff CLAY and SILT, locally soft or very stiff, with variable organic content. Organic CLAYs and Peat (fibrous and amorphous) are most prevalent within the eastern portion of the basin—notably across the Data Centre, Solar Farm Central, and Neighbourhood Precinct South areas.

The soils are inferred to generally comprise Middle to Late Pleistocene flood, river and hill deposits. Pockets of shallow younger Holocene Age flood deposits may be locally present. Limited Oedeometer testing (presented in Table 6-17) on samples recovered near the Lagoon indicate the CLAYs in the area are typically lightly to moderately over consolidated. Oedometer testing on a shallow organic CLAY sample from the Data Centre Precinct indicate it is normally consolidated, and may therefore be Holocene Age.

6.2.3 Mahurangi Limestone (Motatau Complex)

Mahurangi Limestone of the Motatau Complex (Northland Allochthon) was encountered in machine boreholes drilled within the Lagoon and Data Centre footprints, and in test pits excavated within the North-Eastern Neighbourhood Precinct and Solar Farm North. Light grey residual soils, inferred to represent weathered limestone, were also identified in hand auger boreholes within the North-Western Neighbourhood Precinct, Live–Work Precinct, and Solar Farm South. Across the remainder of the site, CPT and Scala refusal suggests the limestone extends beneath the alluvium, although this has not been confirmed by direct sampling.

Where present, the limestone exhibits a typical weathering profile comprising residual soil and completely weathered limestone, underlain by highly to moderately weathered limestone.

The residually to completely weathered material consists of light grey, stiff to hard SILT mixtures, generally non-plastic to low-plasticity. This horizon is typically less than 1 m thick, though CPT data indicates localised deep weathering zones up to approximately 7 m.

The underlying highly to moderately weathered limestone has been defined where, either:

- SPT-N values ≥ 50, and/or
- CPT qc ≥ 15 MPa and/or
- Scala Penetrometer >10 blows per 50 mm penetration, were recorded.

Where observed in boreholes and test pits, the limestone comprises very weak to moderately strong light grey rock, highly fractured and often recovered as gravel-sized fragments due to drilling and excavation disturbance. Test pits (12-tonne excavator with rock bucket) typically refused less than 0.5 m into this unit.

The Mahurangi Limestone forms part of the Northland Allochthon, a complex assemblage of sedimentary, volcanic, and ophiolitic rocks emplaced through large-scale allochthonous displacement. These processes have produced highly deformed, faulted, and sheared materials, resulting in significant spatial and vertical variability. Although investigations indicate limestone continuity beneath the alluvium, other allochthonous units (e.g., Hukerenui Mudstone, mapped north, east, and west of the site — see Figure 4-1) could also be locally present.

6.3 Site Stratigraphy and In-situ Testing

The following subsections presents the stratigraphy and in-situ testing for each development precinct (excluding the Lagoon). The precincts can broadly be categorised into:

Elevated areas generally with shallow depths to Mahurangi Limestone rock, including the:

- Stage 5a Vacant Lot Subdivision;
- Neighbourhood Precinct North-East;
- Solar Farm North; and
- Solar Farm South.

Low-lying central "core" areas with thick deposits of Tauranga Group alluvium, including the:

- Accommodation Precinct;
- Surf Amenity Precinct;
- Village Centre Precinct;
- Neighbourhood Precinct South; and
- Solar Farm Central.

Transitional areas with variable thicknesses of Tauranga Group alluvium including the:

- Neighbourhood Precinct North-West;
- Live Work Precinct;
- Stream Park; and
- Data Centres.

6.3.1 Stage 5a Vacant Lot Subdivision

Limited investigations within the Stage 5a Vacant Lot Subdivision encountered topsoil and localised shallow silty CLAY fill, overlying a thin layer of stiff to hard Tauranga Group alluvium (CLAY mixtures with trace organic inclusions). Beneath this, residually to completely weathered Mahurangi Limestone soils are present.

Highly to moderately weathered limestone rock was not directly proven but is inferred from Scala and CPT resistance data at depths of approximately 1.7–2.1 m below existing ground level.

Table 6-1 below summarises the site stratigraphy and in-situ testing. A geological section through the Stage 5a Vacant Lot Subdivision is presented on Figure 1537-1-G14 in Appendix A.

Table 6-1: Summary of Site Stratigraphy – Stage 5a Vacant Lot Subdivision

Geological Unit	Soil/Rock Type	Depth to Top of Unit (m, bgl) [Typical]	Elevation of top of unit (m RL) (AUCKHT1946)	Layer Thickness (m) [Typical]	In Situ Strength Parameters Typical Range[1] [Typical value]		
					Su (kPa)	Scala (blows per 50 mm)	CPT qt[2] (MPa)
Topsoil / Fill	*Organic SILT*	0	60.6 – 63.3	0.2 – 0.3 [0.25]	-		
Existing Uncontrolled Fill	*Very Stiff Silty CLAY with trace sand and gravel*	0.2	62.1	0 – 0.4 [4]	106	-	-
Tauranga Group Alluvium	*Stiff to Hard CLAYs with trace organic inclusions*	0.2 – 0.6 [0.3]	60.3 – 63.0	0.5 – 0.9 [0.7]	74 – 225+ [90]	-	0.5 – 1.5 [0.8]
Mahurangi Limestone	*Hard SILTs and Extremely Weak Limestone* *[Residually Weathered to Completely Weathered Limestone]*	0.8 – 1.2 [1.0]	59.5 – 62.1	0.8 – 0.9 [0.9]	225+	5 – 12 [8]	2 – 15 [8]
	Inferred Very Weak to Moderately Strong Limestone [3] *[Inferred Highly Weathered to Moderately Weathered Limestone]* [3]	1.7 – 2.1 [2]	58.6 – 61.2	Not proven	-	15+	14 – 40+ [20]

Note [1]: Typical range typically includes 5th to 95th percentile values and may exclude outliers.
Note [2]: Only corrected Cone Resistance values, qt, were presented on the available Aurecon CPT logs.
Note [3]: Unit inferred only based on Scala Penetrometer and CPT resistances.
Note [4]: Fill only encountered within Aurecon HA08.

6.3.2 Neighbourhood Precinct North-East

Test pits completed in 2023 within the North-East Neighbourhood Precinct encountered highly to moderately weathered Mahurangi Limestone rock at depths of 1.3–2.0 m below original ground level. This was overlain by a mantle of topsoil, stiff to very stiff Tauranga Group alluvium (inorganic SILT and CLAY), and hard residually to completely weathered Mahurangi Limestone soils.

Earthworks completed during the 2024/2025 season have subsequently excavated into the highly to moderately weathered limestone across most of this precinct.

Table 6-2 below summarises the site stratigraphy (relative to original ground levels – before the 2024/2025 earthworks) and in-situ testing. A geological section through the precinct is presented on Figure 1537-1-G13 in Appendix A. The section also shows the drone surveyed ground surface at the completion of the 2024/2025 season earthworks.

Table 6-2: Summary of Site Stratigraphy – Neighbourhood Precinct North East

Geological Unit	Soil/Rock Type	Depth to Top of Unit (m, bgl) [Typical]	Elevation of top of unit (m RL) (AUCKHT1946)	Layer Thickness (m) [Typical]	In Situ Strength Parameters Typical Range[1] [Typical value]	
					Su (kPa)	Scala (blows per 50 mm)
Topsoil	*Organic SILT*	0	57.7 – 66.6	0.2 – 0.5 [0.3]	-	-
Tauranga Group Alluvium	*Stiff to Very Stiff CLAYs and SILTs with trace organics*	0.2 – 0.5	57.5 – 66.4	0.3 – 1.0 [0.7]	57 – 174 [80]	-
Mahurangi Limestone	*Hard SILTs and Extremely Weak Limestone* *[Residually Weathered to Completely Weathered Limestone]*	0.8 – 1.2	56.6 – 65.8	0 – 1.2 [0.5]	210 – 250+ [250]	6 – 15 [8]
	Very Weak to Moderately Strong Limestone, typically highly fractured. *[Highly Weathered to Moderately Weathered Limestone]*	1.3 – 2	56.3 – 65.2	Not proven	-	15+

Note [1]: Typical range typically includes 5th to 95th percentile values and may exclude outliers.

6.3.3 Solar Farm North

Test pits within the Northern Solar Farm footprint encountered highly to moderately weathered Mahurangi Limestone rock at depths of 1.5–2.1 m below original ground level. Overlying materials comprise topsoil, firm to stiff Tauranga Group alluvium (inorganic SILT and CLAY), and stiff to hard residually to completely weathered Mahurangi Limestone soils.

Table 6-3 below summarises the site stratigraphy and in-situ testing. A geological section through the Northern Solar Farm is presented on Figure 1537-1-G15 in Appendix A.

Table 6-3: Summary of Site Stratigraphy – Solar Farm North

Geological Unit	Soil/Rock Type	Depth to Top of Unit (m, bgl) [Typical]	Elevation of top of unit (m RL) (AUCKHT1946)	Layer Thickness (m) [Typical]	In Situ Strength Parameters Typical Range[1] [Typical value]	
					Su (kPa)	Scala (blows per 50 mm)
Topsoil	*Organic SILT*	0	56.7 – 64.4	0.2 – 0.3 [0.25]	-	-
Tauranga Group Alluvium	*Firm to Stiff CLAYs and SILTs with trace organics*	0.2 – 0.3	56.5 – 64.2	0.3 – 1.9 [0.6]	31 – 110 [50]	-
Mahurangi Limestone	*Stiff to Hard SILTs and Extremely Weak Limestone* *[Residually Weathered to Completely Weathered Limestone]*	0.6 – 1.2 [0.9]	57.7 - 63.2	0 [2] – 0.9 [0.8]	72 – 200+ [150]	6 – 15 [8]
	Very Weak to Moderately Strong Limestone, typically highly fractured. *[Highly Weathered to Moderately Weathered Limestone]*	1.5 – 2.1 [1.8]	54.6 - 62.9	Not proven	-	30 +

Note [1]: Typical range typically includes 5th to 95th percentile values and may exclude outliers.
Note [2]: Highly weathered Limestone was logged directly beneath 1.9 m thickness of Tauranga Group Alluvium in TP08.

6.3.4 Solar Farm South

Hand auger investigations within the Southern Solar Farm encountered topsoil or shallow organic fill, overlying firm to stiff Tauranga Group alluvium (typically inorganic SILT and CLAY), underlain by very stiff to hard residually to completely weathered Mahurangi Limestone soils. Highly to moderately weathered limestone rock was not directly proven but is inferred from Scalas and CPTs at typical depths of 1.5–2.2 m below existing ground level, increasing to 4.3 m at the north-eastern corner.

Table 6-4 below summarises the site stratigraphy and in-situ testing. A geological section through the southern Solar Farm is presented on Figure 1537-1-G15 in Appendix A.

Table 6-4: Summary of Site Stratigraphy – Solar Farm South

Geological Unit	Soil/Rock Type	Depth to Top of Unit (m, bgl) [Typical]	Elevation of top of unit (m RL) (AUCKHT1946)	Layer Thickness (m) [Typical]	In Situ Strength Parameters Typical Range[1] [Typical value]		
					Su (kPa)	Scala (blows per 50 mm)	CPT qt[2] (MPa)
Topsoil / Uncontrolled Fill	*Organic SILT and Clayey SILT*	0	54.5 – 60.3	0.15 – 0.5 [0.3]	-	-	-
Tauranga Group Alluvium	*Firm to Stiff CLAYs and SILTs with occasional minor organics*	0.15 – 0.5 [0.3]	54.2 – 60.0	0.4 – 3.7 [0.8]	26 – 84 [45]	0 – 2 [1]	0.35 – 1 [0.45]
Mahurangi Limestone	*Very Stiff to Hard SILTs and Extremely Weak Limestone* *[Residually Weathered to Completely Weathered Limestone]*	0.8 – 4.0 [1.1]	50.5 – 59.5	0.3 – 1.0 [0.8]	132 – 214+	4 – 20 [6]	0.8 – 15 [5]

Geological Unit	Soil/Rock Type	Depth to Top of Unit (m, bgl) [Typical]	Elevation of top of unit (m RL) (AUCKHT1946)	Layer Thickness (m) [Typical]	In Situ Strength Parameters Typical Range[1] [Typical value]		
					Su (kPa)	Scala (blows per 50 mm)	CPT qt[2] (MPa)
	Inferred Very Weak to Moderately Strong Limestone [2] *[Inferred Highly Weathered to Moderately Weathered Limestone] [2]*	1.5 - 4.3 [1.8]	50.2 – 58.6	Not proven	-	20+	15+

Note [1]: Typical range typically includes 5th to 95th percentile values and may exclude outliers.
Note [2] : Unit inferred only based on Scala Penetrometer and CPT resistances

6.3.5 Accommodation Precinct

Test pit and hand auger investigations within the Accommodation Precinct encountered topsoil and localised uncontrolled fill, overlying firm to very stiff Tauranga Group alluvium (CLAY and SILT mixtures with minor organic inclusions). CPT traces indicate the alluvium extends to depths of approximately 6.1–11 m and is underlain by weathered Mahurangi Limestone.

Table 6-5 below summarises the site stratigraphy and in-situ testing. A geological cross secction through the Accommodation Precinct is presented on Figure 1537-1-G12 in Appendix A.

Table 6-5: Summary of Site Stratigraphy – Accommodation Precinct

Geological Unit	Soil/Rock Type	Depth to Top of Unit (m, bgl) [Typical]	Elevation of top of unit (m RL) (AUCKHT1946)	Layer Thickness (m) [Typical]	In Situ Strength Parameters Typical Range[1] [Typical value]		
					Su (kPa)	Scala (blows per 50 mm)	CPT qc (MPa)
Topsoil	*Organic SILT*	0	50.6 – 55.0	0.1 – 0.9 [0.4]	-	-	-
Existing Uncontrolled Fill	*Firm to Very Stiff Clayey SILT (Uncontrolled Fill)*	0.3 – 0.4[3]	50.3 – 54.6[3]	0 – 0.5[3]	47 – 120 [60]	-	-
Tauranga Group Alluvium	*Firm to Very Stiff CLAY and SILT mixtures, with occasional minor organics*	0.1 – 0.9 [0.4]	49.8 – 54.6	5.8 – 10.5 [8]	25 – 142 [50]	0 – 3 [1]	0.25 – 1.2 [0.7]
Mahurangi Limestone	*Inferred Very Stiff to Hard SILTs and Extremely Weak Limestone [2]* *[Inferred Residually Weathered to Completely Weathered Limestone] [2]*	6.1 – 11 [8.5]	44.0 – 45.1	0.3 – 6.2 [1.5]	-	-	1.5 – 18 [8]
	Inferred Very Weak to Moderately Strong Limestone [2] *[Inferred Highly Weathered to Moderately Weathered Limestone] [2]*	7.9 – 12.3 [10]	38.9 – 44.1	Not proven	-	-	13 – 35+ [20]

Note [1]: Typical range typically includes 5th to 95th percentile values and may exclude outliers.
Note [2] : Unit inferred only based on Scala Penetrometer and CPT resistances.
Note [3] : Existing fill was only encountered within TP14 and HA139.

6.3.6 Surf Amenity Precinct (excluding Lagoon)

Investigations within the Surf Amenity Precinct encountered topsoil and localised uncontrolled fill, overlying Tauranga Group alluvium and a weathering profile of Mahurangi Limestone.

The localised uncontrolled fill comprised 0.6 m thick firm SILT at HA136 and 1.0 m thick loosely packed silty GRAVEL with concrete rubble at TP110. The fill in TP110 is inferred to be associated with an existing farm track.

The Tauranga Group alluvium generally comprises CLAY and SILT mixtures with trace to some organic inclusions. Organic CLAY was locally observed in TP110 below 2.5 m depth and additional isolated pockets are expected based on recent bulk earthworks observations. The alluvium typically includes an upper firm to stiff layer, underlain by a stiff to very stiff soils at depth.

Weathered Mahurangi Limestone soils and rock were proven in MBH01 and inferred from CPT traces, with the weathering profile varying across the precinct.

Table 6-6 below presents site stratigraphy and in-situ testing. A geological section through the Surf Amenity Precinct is presented on Figure 1537-1-G11 in Appendix A.

Table 6-6: Summary of Site Stratigraphy – Surf Amenity Precinct

Geological Unit	Soil/Rock Type	Depth to Top of Unit (m, bgl) [Typical]	Elevation of top of unit (m RL) (AUCKHT1946)	Layer Thickness (m) [Typical]	In Situ Strength Parameters Typical Range[1] [Typical value]		
					Su (kPa)	CPT qc (MPa)	SPT N
Topsoil	*Organic SILT*	0	54.6 – 56.8	0 – 0.9 [0.4]	-	-	-
Existing Uncontrolled Fill	*Firm SILTs and loosely packed Gravels (Uncontrolled Fill)*	0 – 0.1[2]	55.1 – 55.6[2]	0.6 – 1.0[2]	44	-	-
Buried Topsoil	*Organic SILT*	1.0[3]	54.6[3]	0.3[3]	-	-	-
Tauranga Group Alluvium	*Unit 1: Firm to Stiff (locally very stiff) CLAY and SILT mixtures, with trace to some organics & local organic CLAY*	0.15 – 1.3 [0.4]	53.8 – 56.6	2.8 – 5.3 [4.0]	28 – 146 [60]	0.2 – 0.75 [0.4]	-
	Unit 2: Stiff to very stiff CLAY and SILT mixtures	3.6 – 6.0 [4.5]	48.8 – 51.8	2.2 – 6.6 [4.0]	-	0.4 – 1.4 [0.7]	4
Mahurangi Limestone	*Very Stiff to Hard SILTs and Extremely Weak Limestone* *[Residually Weathered to Completely Weathered Limestone]*	5.8 – 11.6 [8.5]	43.0 – 49.6	0.15 – 4.1+ [2.0]	-	1.4 – 18 [7]	-
	Very Weak to Moderately Strong Limestone *[Highly Weathered to Moderately Weathered Limestone]*	7.2 – 12.5 [10.5]	42.3 – 47.8	Not proven	-	13 – 40+ [18]	50+

Note [1]: Typical range typically includes 5th to 95th percentile values and may exclude outliers.
Note [2]: Existing uncontrolled fill was only encountered within HA136 and TP111.
Note [3]: Buried topsoil was only encountered in TP111.

6.3.7 Village Centre Precinct

Investigations within the Village Centre Precinct encountered topsoil overlying Tauranga Group alluvium. CPT traces infer weathered Mahurangi Limestone at depth.

Topsoil thickness ranges from 0.3–1.0 m, typically about 0.7 m. The underlying alluvium generally comprises firm to stiff CLAY and SILT with trace to some organic inclusions. Firm organic CLAY was locally recorded in HA142, extending from 3.5 m depth to beyond the termination depth (4.0 m).

Table 6-7 below presents the site stratigraphy and in situ testing. A geological section through the precinct is presented on Figure 1537-1-G11 in Appendix A.

Table 6-7: Summary of Site Stratigraphy –Village Centre Precinct

Geological Unit	Soil/Rock Type	Depth to Top of Unit (m, bgl) [Typical]	Elevation of top of unit (m RL) (AUCKHT1946)	Layer Thickness (m) [Typical]	In Situ Strength Parameters Typical Range[1] [Typical value]	
					Su (kPa)	CPT qc (MPa)
Topsoil	*Organic SILT*	0	53.3 – 54.8	0.3 – 1.0 [0.7]	-	-
Tauranga Group Alluvium	*Firm to Stiff CLAY and SILT with trace to some organic inclusions and occasional organic CLAY*	0.3 – 1.0 [0.7]	52.9 – 54.2	5.1 – 10.9 [9.5]	29 – 92 [50]	0.2 – 0.9 [0.5]
Mahurangi Limestone	*Inferred Very Stiff to Hard SILTs and Extremely Weak Limestone [2]* *[Inferred Residually Weathered to Completely Weathered Limestone] [2]*	5.6 – 11.8 [10.5]	42.7 – 48.7	0.3 – 3.4 [0.5]	-	0.8 – 25 [8]
	Inferred Very Weak to Moderately Strong Limestone [2] *[Inferred Highly Weathered to Moderately Weathered Limestone] [2]*	6.2 – 12.4 [11]	42.4 – 48.2	Not proven	-	12 – 40+ [20]

Note [1]: Typical range typically includes 5th to 95th percentile values and may exclude outliers.
Note [2] : Unit inferred only based CPT resistances.

6.3.8 Neighbourhood Precinct - South

Investigations within the Southern Neighbourhood Precinct encountered topsoil overlying Tauranga Group alluvium. CPT traces infer weathered Mahurangi Limestone at depth.

The Tauranga Group alluvium typically comprises firm to stiff organic CLAY interbedded with inorganic CLAY and SILT. A shallow layer of clayey fibrous and amorphous PEAT, up to 1.65 m thick, was recorded in T+T_TP03 and T+T_HA11P. Soft organic CLAY with undrained shear strengths of 20–23 kPa was also logged in T+T_HA11P.

The total alluvium thickness ranges from 8 to 10.8 m across the southern half of the precinct, reducing to 4.5 to 8 m in the northern half.

Table 6-8 below presents the site stratigraphy and in-situ testing. A geological section through the precinct is presented on Figure 1537-1-G12 in Appendix A.

Table 6-8: Summary of Site Stratigraphy – Neighbourhood Precinct - South

Geological Unit	Soil/Rock Type	Depth to Top of Unit (m, bgl) [Typical]	Elevation of top of unit (m RL) (AUCKHT19 46)	Layer Thickness (m) [Typical]	In Situ Strength Parameters Typical Range[1] [Typical value]	
					Su (kPa)	CPT qc (MPa)
Topsoil	*Organic SILT*	0	53.8 – 54.4	0.2 – 0.8 [0.5]	-	-
Tauranga Group Alluvium	*Firm to Stiff (locally soft) organic CLAY, inorganic CLAY, inorganic SILT and clayey PEAT*	0.2 – 0.8 [0.5]	53.0 – 54.0	4.5 – 10.8 [8]	18 – 99 [40]	0.15 – 0.8 [0.4]
Mahurangi Limestone	*Inferred Very Stiff to Hard SILTs and Extremely Weak Limestone* [2] *[Inferred Residually Weathered to Completely Weathered Limestone]* [2]	4.9 – 11.2 [8.5]	42.9 – 49.5	0.4 – 6.8 [1.0]	-	1 – 20 [9]
	Inferred Very Weak to Moderately Strong Limestone [2] *[Inferred Highly Weathered to Moderately Weathered Limestone]* [2]	7.7 – 12.3 [9.5]	42.5 – 46.1	Not proven	-	12.5 – 40+ [20]

Note [1]: Typical range typically includes 5th to 95th percentile values and may exclude outliers.
Note [2] : Unit inferred only based CPT resistances.

6.3.9 Solar Farm Central

Investigations within the Central Solar Farm (HA105 and TP109) encountered topsoil overlying Tauranga Group alluvium comprising firm to stiff organic CLAY.

Based on results from adjacent precincts, the Tauranga Group alluvium is inferred to extend to approximately 5–10 m depth, underlain by weathered Mahurangi Limestone. The alluvium likely contains interbedded soft to firm fibrous PEAT layers, consistent with materials recorded within the Data Centre Precinct to the east, below the termination depths of HA105 and TP109.

Table 6-9 below summarises the site stratigraphy and in-situ testing. A geological section through the central Solar Farm is presented on Figure 1537-1-G15 in Appendix A.

Table 6-9: Summary of Site Stratigraphy – Solar Farm Central

Geological Unit	Soil/Rock Type	Depth to Top of Unit (m, bgl)	Elevation of top of unit (m RL) (AUCKHT19 46)	Layer Thickness (m)	In Situ Strength Parameters Typical Range[1] [Typical value]	
					Su (kPa)	Scala (blows per 50 mm)
Topsoil	*Organic SILT*	0	53.4 – 53.6	0.1 – 0.2	-	-
Tauranga Group Alluvium	*Firm to Stiff organic CLAY*	0.1 – 0.2	53.3 - 53.4	> 2 proven [2]	32 – 73 [50]	0 – 2 [3] [1]

Note [1]: Typical range typically includes 5th to 95th percentile values and may exclude outliers.
Note [2] : Base of unit not proven.
Note [3] : Higher Scala blow counts were recorded at depth, however, these counts are not considered to reliably represent the strata strength due to an accumulation of rod friction with depth.

6.3.10 Neighbourhood Precinct North-West

Hand auger investigations within the Neighbourhood Precinct North-West encountered topsoil and localised uncontrolled fill, overlying Tauranga Group alluvium and residually to completely weathered Mahurangi Limestone soils. Highly to moderately weathered limestone rock was not directly proven but is inferred from Scala and CPT resistances.

Uncontrolled fill was logged in six hand auger locations, distributed across the precinct. It typically comprises firm to stiff CLAY and SILT between 0.2 and 0.35 m thick. At the south-western boundary, approximately 1-3 m thickness of existing fill is inferred below an existing horse arena.

The Tauranga Group alluvium generally consists of firm to very stiff CLAY and SILT with trace to some organic inclusions. Organic CLAY was locally recorded in TP110 at the south-eastern corner. The alluvium thickness is typically ≤1.5 m but locally increases to 2.9–4.7 m in the south-eastern and south-western corners.

Table 6-10 below summarises the site stratigraphy and in-situ testing. A geological cross section through the precinct is shown on Figure 1537-1-G13 in Appendix A.

Table 6-10: Summary of Site Stratigraphy – Neighbourhood Precinct North-West

Geological Unit	Soil/Rock Type	Depth to Top of Unit (m, bgl) [Typical]	Elevation of top of unit (m RL) (AUCKHT1946)	Layer Thickness (m) [Typical]	In Situ Strength Parameters Typical Range[1] [Typical value]		
					Su (kPa)	CPT qc (MPa)	SPT N
Topsoil	*Organic SILT*	0	51.4 – 62.8	0 – 0.3 [0.25]	-	-	-
Existing Uncontrolled Fill	*Firm to Stiff CLAY and SILT mixtures (Uncontrolled Fill)*	0 – 0.15[3]	52 – 63.7[3]	0 – 0.35[3]	-	-	-
Buried Topsoil	*Organic SILT*	0.3[4]	63.4[4]	0.1[4]	-	-	-
Tauranga Group Alluvium	*Firm to Very Stiff CLAYs and SILTs with trace to some organic inclusions [5]*	0.1 – 0.5 [0.3]	51.1 – 63.3	0.4 – 4.7 [1.2]	25 – 199 [50]	-	0.2 – 0.8 [0.5]
Mahurangi Limestone	*Stiff to Hard SILTs and Extremely Weak Limestone* *[Residually Weathered to Completely Weathered Limestone]*	0.6 – 4.8 [1.4]	48.8 – 62.5	0.3 – 2.4 [0.7]	65 – 205+	2 – 15 [8]	1 – 15 [4]
	Inferred Very Weak to Moderately Strong Limestone [2] *[Inferred Highly Weathered to Moderately Weathered Limestone] [2]*	1.3 – 7.2 [2]	46.4 – 61.9	Not proven	-	15+	15 – 40+ [20}

Note [1]: Typical range typically includes 5th to 95th percentile values and may exclude outliers.
Note [2] : Unit inferred only based on Scala Penetrometer and CPT resistances.
Note [3] : Existing fill was locally found within only 6 No. hand auger boreholes. Thicker fill is inferred beneath the existing horse arena but was not proven by the investigations.
Note [4] : Encountered only within HA104.
Note [5] : Organic clay was also locally encountered only in TP110.

6.3.11 Live Work Precinct

Hand auger investigations within the Live–Work Precinct encountered topsoil and localised uncontrolled fill, overlying Tauranga Group alluvium and residually to completely weathered Mahurangi Limestone soils. Highly to moderately weathered limestone rock was not directly proven but is inferred from Scala and CPT resistances.

Uncontrolled fill was recorded in HA123, HA127, and HA128, ranging from 0.3–1.0 m thick. It generally comprises firm to very stiff CLAY. Additional fill is likely present near existing building platforms and the effluent disposal fields.

The Tauranga Group alluvium typically comprises firm to stiff CLAY mixtures, with soft materials locally encountered in HA127. Trace to some organic content was commonly observed, particularly at the northern end of the precinct. Alluvium thickness ranges from 0.4–3.7 m, with thicker deposits (>1.5 m) present in the lower-lying northern and western areas.

Table 6-11 below summarises the site stratigraphy and in-situ testing. A geological cross section through the Live Work Precinct is presented on Figure 1537-1-G14 in Appendix A.

Table 6-11: Summary of Site Stratigraphy- Live Work Precinct

Geological Unit	Soil/Rock Type	Depth to Top of Unit (m, bgl) [Typical]	Elevation of top of unit (m RL) (AUCKHT1946)	Layer Thickness (m) [Typical]	In Situ Strength Parameters Typical Range[1] [Typical value]		
					Su (kPa)	Scala (blows per 50 mm)	CPT qc (MPa)
Topsoil	*Organic SILT*	0	53.9 – 56.1	0.1 – 0.5 [0.3]	-	-	-
Existing Uncontrolled Fill	*Firm to Stiff CLAY mixtures (Uncontrolled Fill)*	0.1 – 0.2[3]	54 – 55.7[3]	0 – 1[3]	26 – 184 [50]	-	-
Buried Topsoil	*Organic SILT*	1.1[4]	53[4]	0 – 0.2[4]	-	-	-
Tauranga Group Alluvium	*Soft to Stiff CLAYs with trace to some organic inclusions*	0.2 – 1.3 [0.3]	52.8 – 55.9	0.4 – 3.7 [1.5]	14 – 106 [45]	0 – 2 [1]	0.2 – 1 [0.35]
Mahurangi Limestone	*Hard SILTs and Extremely Weak Limestone* *[Residually Weathered to Completely Weathered Limestone]*	1.0 – 4.2 [1.8]	50.0 – 55.0	0.4 – 5.9 [1.5]	214+	2 – 10 [8]	1.5 – 15 [8]
	Inferred Very Weak to Moderately Strong Limestone [2] *[Inferred Highly Weathered to Moderately Weathered Limestone] [2]*	1.6 – 8 [3.5]	45.9 - 54.5	Not proven	-	10+	15 – 40+ [20]

Note [1]: Typical range typically includes 5th to 95th percentile values and may exclude outliers.
Note [2] : Unit inferred only based on Scala Penetrometer and CPT resistances
Note [3] : Only encountered within HA123, HA127 and HA128
Note [4] : Only encountered within HA127

6.3.12 Stream Park

The Stream Park occupies a transitional zone at the base of the northern hillside, where the thickness and composition of the Tauranga Group alluvium is highly variable. Alluvium thickness ranges from 0.6–10.1 m, generally increasing southward.

At the western end, the alluvium generally comprises firm to very stiff CLAY and SILT with low organic content, while at the eastern end, firm to stiff organic CLAYs and clayey PEAT is present.

Table 6-12 below summarises the stratigraphy and in-situ testing. Geological sections passing through various locations of the Stream Park are presented on Figures 1537-1-G13 and 1537-1-G15 in Appendix A.

Table 6-12: Summary of Site Stratigraphy – Stream Park

Geological Unit	Soil/Rock Type	Depth to Top of Unit (m, bgl) [Typical]	Elevation of top of unit (m RL) (AUCKHT1946)	Layer Thickness (m) [Typical]	In Situ Strength Parameters Typical Range[1] [Typical value]	
					Su (kPa)	CPT q_c (MPa)
Topsoil	*Organic SILT*	0	50.9 – 57.7	0.1 – 0.6 [0.3]	-	-
Tauranga Group Alluvium	*Firm to Very Stiff CLAYs, SILTs and Organic CLAYS, with occasional PEAT*	0.1 – 0.6 [0.3]	50.5 – 57.5	0.6 – 10.1 [6]	25 – 199 [45]	0.2 – 1 [0.4]
Mahurangi Limestone	*Stiff to Hard SILTs and Extremely Weak Limestone* *[Residually Weathered to Completely Weathered Limestone]*	0.8 – 10.7 [6.3]	44.0 – 56.7	0.2 – 6.2 [1.2]	-	1 – 15 [8]
	Very Weak to Moderately Strong Limestone, typically highly fractured. *[Highly Weathered to Moderately Weathered Limestone]*	1.4 – 12.3 [7.5]	38.9 – 56.3	Not proven	-	12 – 30+ [20]

Note [1]: Typical range typically includes 5th to 95th percentile values and may exclude outliers.

6.3.13 Data Centres

The western data centre footprint was investigated by Aurecon (2023). This platform is underlain by topsoil, over 5.3–9.8 m of Tauranga Group alluvium, underlain by a weathering profile of Mahurangi Limestone. The alluvium typically comprises firm to stiff interbedded CLAY, organic CLAY, SILT, and amorphous PEAT. A soft to firm fibrous PEAT layer, 1.0–2.3 m thick, was recorded in all machine boreholes, with upper surfaces at 1.2–2.3 m depth.

The eastern data centre footprint was not included in Aurecon's investigation; however, a T+T test pit and hand auger borehole was completed at the eastern (upslope) end of the building footprint. At this eastern end, highly to moderately weathered Mahurangi Limestone was encountered at approximately 2.4 m depth. It is inferred the alluvium thickness and subsequent rock depth will increase rapidly towards the west over the building platform.

Table 6-13 below summarises the site stratigraphy and in-situ testing. A geological cross section through the Data Centres is presented on Figure 1357-1-G14 in Appendix A.

Table 6-13: Summary of Site Stratigraphy – Data Centres

Geological Unit	Soil/Rock Type	Depth to Top of Unit (m, bgl) [Typical]	Elevation of top of unit (m RL) (AUCKHT1946)	Layer Thickness (m) [Typical]	In Situ Strength Parameters Typical Range[1] [Typical value]		
					Su (kPa) [3]	CPT qt [4] (MPa)	SPT N
Topsoil	*Organic SILT*	0	53.6 - 58.9	0 – 0.4 [0.3]	-	-	-
Existing Uncontrolled Fill	*SAND, SILT and GRAVEL (Uncontrolled Fill)*	0 – 0.1[2]	58.8 – 59.1 [2]	0 – 0.9 [2]	81 – 137 [100]	-	-
Tauranga Group Alluvium	*Unit 1: Firm to Very Stiff CLAY, SILT and Organic CLAY*	0.15 – 0.9 [0.3]	53.3 – 58.3	0.9 – 2.1 [1.5]	29 – 199 [50]	< 1 [5]	0 – 4 [0]
	Unit 2: Soft to Firm Fibrous PEAT	1.2 – 2.3 [6] [1.8]	51.7 – 53.0 [6]	0 – 2.3 [6] [1.5]	12 – 47 [25]	< 1 [5]	0 – 1 [0]
	Unit 3: Firm to Very Stiff CLAY, SILT, Organic CLAY and PEAT	3.0 – 3.5 [6] [3.2]	50.2 – 51.4 [6]	0 – 6.8 [6] [5.5]	80 - 145	< 1[5]	0 – 3 [0]
Mahurangi Limestone	*Very Stiff to Hard SILTs and Extremely Weak Limestone* *[Residually Weathered to Completely Weathered Limestone]*	1.5 – 10 [8.7]	43.9 - 57.4	0.2 – 1.0 [0.5]	165 – 171	1 – 15 [5]	5
	Very Weak to Moderately Strong Limestone *[Highly Weathered to Slightly Weathered Limestone]*	2.4 – 10.4 [9.2]	43.7 - 56.7	Not proven	-	-	50+

Note [1] : Typical range typically includes 5th to 95th percentile values and may exclude outliers.
Note [2] : Fill was only logged within T+T_TP04 and T+T_HA14 below the eastern data centre.
Note [3] : Only includes down-hole shear vane measurements. Shear vane measurements at the end of the core barrel have been excluded as these are generally considered to be unreliable.
Note [4] : Only corrected Cone Resistances, qt, were presented on the available Aurecon CPT logs.
Note [5] : Only PDF CPT logs were made available and the scale on the logs does not allow for accurate qt ranges to be interpreted.
Note [6] : Tauranga Group Alluvium Units 2 and 3 not encountered within T+T_TP04 and T+T_HA14 below the eastern data centre.

6.4 Groundwater

Groundwater data has been collected from measurements in standpipe piezometers installed by Initia, Tonkin & Taylor, and Aurecon. Groundwater inflows were also recorded, where encountered, during test pit, hand auger, and borehole investigations. All groundwater data is presented in the Initia Geotechnical Factual Report.

Surface water was also observed at the invert of existing creek along the western boundary of the Neighbourhood Precinct North-West and through Stream Park, at levels typically between RL 49 and 52 m.

The groundwater monitoring and observations indicate that groundwater conditions at the site are complex and involve multiple different groundwater regimes. The groundwater regimes across various parts of the site are summarised in the subsections below.

6.4.1 Low Lying Central "Core"

Within the low-lying central core of the site (generally where existing ground surface level is at or below RL 56 m) groundwater conditions comprise:

1. Shallow perched groundwater, typically within the upper topsoil.
2. Static groundwater, within the Tauranga Group alluvium; and
3. Potential Artesian Conditions within the Mahurangi Limestone at depth.

Shallow Perched Groundwater:
Shallow perched groundwater, typically between 0 – 1 m deep was generally encountered within the boreholes and hand auger investigations. This groundwater appears to sit typically near the base of the topsoil layer (which is thick in areas) or at levels coincident with the surface of the Tauranga Group alluvium (i.e. water is present within higher permeability soils perched above the underlying low permeability alluvial clays). Test pit observations showed the perched groundwater has limited depth and volume. Inflows were initially rapid but attenuated quickly (within minutes) and groundwater levels stabilised at greater depths. The perched groundwater level is likely to fluctuate significantly and may not be present at all following a dry summer.

Static Groundwater:
Based on test pit groundwater observations, the static groundwater level, below which hydrostatic pressure conditions are present, is inferred from a typical depth of between 3 and 4 m below existing ground level (generally between approximately RL 51 to 52 m, grading down to approximately RL 49 m at the western end of the creek). The static groundwater level is likely controlled by the water levels in the creek channel, with a gentle upslope gradient.

Artesian Groundwater Conditions:
Artesian conditions were locally encountered following the drilling of MBH01 which was drilled into Mahurangi Limestone rock. Artesian levels were recorded in the piezometer to a height of 0.6 m above ground level. This level reduced to approximately 1 m below ground level the next day. Artesian conditions could be encountered within the Limestone Rock across other areas of the site.

6.4.2 Elevated Hillsides

The hand auger and test pit investigations that were undertaken on the elevated northern, eastern and southern hillsides at ground elevations greater than RL 56 m generally did not encounter groundwater. Given these investigations typically terminated near the surface of Highly to Moderately Weathered Mahurangi Limestone, it is inferred that groundwater sits within the fractured Mahurangi Limestone rock and likely percolates through the rock mass towards the low-lying central area of the site.

Shallow perched groundwater may be present within the topsoil or on the rock surface following heavy rainfall events, however, this groundwater likely drains down towards the creek and central core of the site meaning shallow groundwater is generally not present during dry weather periods.

6.5 Summary of Laboratory Test Results

Geotechnical laboratory tests have been undertaken by both Initia and Aurecon on select samples obtained from machine boreholes, hand augers and test pits. Laboratory tests conducted by Initia have included Atterberg Limits, Linear Shrinkage, Soaked CBR, Standard Compaction, Organic Content tests and Oedometer (1D consolidation tests).

A summary of the laboratory test results is presented in Table 6-14, Table 6-15, Table 6-16 and Table 6-17. It is noted that this includes samples obtained from within the surf lagoon footprint.

Full laboratory test results and reports are presented in the Initia Geotechnical Factual report.

Table 6-14: Statistical Summary of Atterberg Limit, Linear Shrinkage & Organic Content Tests

	Laboratory Test Results Range & [Median Value]			
Sample Geology	Residually Weathered Mahurangi Limestone SILT, minor sand Very Stiff to Hard	Tauranga Group Fibrous PEAT Soft to Firm	Tauranga Group Organic CLAY & CLAY with some Organics Firm	Tauranga Group CLAY, Silty CLAY and Clayey SILT, with none to minor orgaincs Firm to Very Stiff
Range of Sample Depths	1.1 – 1.4	1.5 – 3.5	0.5 – 2.5	0.3 – 2.5
Sample Locations	Neighbourhood Precinct North-West, Solar Farm South	Data Centres	Neighbourhood Precinct South	Surf Lagoon, Surf Amenity Precinct, Accommodation Precinct, Village Centre Precinct, Neighbourhood Precinct North-west, Neighbourhood Precinct South, Live Work Precinct, Data Centres, Solar Farm South
Natural Water Content (%)	32 – 47 [36]	113 – 263	88 – 127	28 – 93 [45]
Plastic Limit	19 – 21 [20]	44 – 99	35 – 53	17 – 44 [28]
Liquid Limit	66 – 88 [75]	170 – 176	99 – 148	39 – 146 [83]
Plasticity Index	45 – 68 [55]	71 – 132	64 – 95	21 – 117 [55]
Liquidity Index	24 – 39 [32]	52 – 231	78 – 83	9 – 76 [30]
Linear Shrinkage (%)	16 – 22 [18]	31	18 - 22	9 – 28 [17]
Organic Content (%)	Not tested	Not tested	10 – 13 [12]	Not tested

Table 6-15: Statistical Summary of Standard Compaction Tests

	Standard Compaction Results Range & [Median Value]
Sample Geology	Tauranga Group Alluvium Inorganic CLAY and Silty CLAY Firm to Very Stiff (typically Stiff)
Range of Sample Depths	0.5 – 2.0
Sample Locations	Surf Lagoon, Accommodation Precinct
Natural Water Content (%)	35 – 61 [51]
Optimum Water Content (%)	27 – 36 [34]
Difference Between Natural Water Content and Optimum Water Content (%)	6 – 27 [16]
Maximum Dry Density (t/m^3)	1.26 – 1.44 [1.3]

Table 6-16: Statistical Summary of Soaked CBR Tests

	Soaked CBR Results Range
Sample Geology	Tauranga Group Alluvium Silty CLAY and CLAY, no to trace organics Firm to Very Stiff (typically Stiff)
Range of Sample Depths	1 – 2 m
Sample Locations	Accommodation Precinct, Surf Amenity Precinct, Village Centre Precinct, Neighbourhood Precinct South
Soaked CBR. No stabiliser added	1 – 3 %
Soaked CBR. 2% Lime & 2% Cement	13 – 30 %
Soaked CBR. 1% Lime & 3% Cement	30 %
Soaked CBR. 3% Lime & 1% Cement	20%

Table 6-17: Summary of One Dimensional Consolidation (Oedometer) Test Results

Sample ID, Depth and Location	Sample Description[1]	m_v across working stress range[2] (MPa^{-1})	P'c (kPa)	OCR	C_{ce}	C_{re}
MBH01 4.5 – 5.0 m Surf Amenity Precinct	CLAY, with trace organics. Stiff, high plasticity.	0.18	260	5	0.19	0.06
MBH02 3.0 – 3.5 m Surf Lagoon	CLAY, with minor organics. Stiff, high plasticity	0.38	160	4	0.25	0.06
MBH02 6.0 – 6.5 m Surf Lagoon	CLAY, with trace organics. Stiff, high plasticity.	0.13	270	4	0.22	0.05
MBH02 7.5 – 8.0 m Surf Lagoon	CLAY, with trace organics. Stiff, high plasticity.	0.095	280	3.5	0.20	0.06
MBH03 1.5 – 2.0 m Surf Lagoon	Silty CLAY, organic stained. Stiff, high plasticity.	0.092	300	10	0.20	0.07
MBH04 4.5 – 5.0 m Surf Lagoon	Silty CLAY, with trace organics. Firm, high plasticity.	0.52	90	1.7	0.17	0.05
Aurecon_BH07 4.5 – 5.0 m Data Centres	CLAY, trace organics. Stiff, high plasticity.	0.15	180	3.5	0.2	0.05
Aurecon_BH08 3.0 – 3.45 m Data Centres	Organic CLAY. Firm, non-plastic.	2.2	35	1	0.27	0.06

Notes :
[1] All tested samples comprise Tauranga Group Alluvium
[2] Working stress range defined as between the existing vertical effective stress and approximately 50 kPa above the existing vertical effective stress.

6.6 Preliminary Geotechnical Design Parameters

Geotechnical strength parameters recommended for preliminary stability, retaining wall and foundation assessment are presented in Table 6-18 below. A range of parameters have been provided which reflect the variability in the materials across the site.

During developed and detailed design, parameters should be reviewed and selected based upon a review of the local ground conditions and testing.

The material parameters are based upon the in-situ test data and experience with similar deposits across the Auckland and Northland regions.

Table 6-18: Preliminary Geotechnical Strength and Permeability Parameters

Geological Unit	Bulk unit weight, γ, (kN/m³)	Friction angle, ϕ' (deg)	Effective Cohesion, c' (kPa)	Undrained Shear Strength, su, (kPa)	Deformation Modulus, E' (MPa)[1]	Poisson's Ratio, ν
Existing Uncontrolled Fill	Specific review required					
Soft Tauranga Group Clays and Peat	Specific review required					
Firm to Stiff Tauranga Group Silts and Clays	17.5 – 18	26 – 28	3 – 5	30 – 70	10 – 18	0.35
Stiff to Very Stiff Tauranga Group Silts and Clays	18 – 18.5	28 – 30	4 – 6	50 – 120	15 – 25	0.35
Very Stiff to Hard Residual Mahurangi Limestone Silts	18.5	32	5 – 7	100 – 200+	20 – 30	0.3
Mahurangi Limestone Weathered Rock	Specific review required					
Engineered Cohesive Bulk Fill	18.5	30	5	100 [2]	30	0.3
Engineered Hardfill	20	38	0	-	50	0.3

Note:
[1] Modulus suitable for retaining wall deflection analysis only, not for consolidation settlement estimates
[2] Compaction specification requires average Su ≥ 140 kPa for fill, however, it is recommended this is assumed as the long-term undrained shear strength for the design of structures.

7. Geotechnical Considerations

7.1 General

Based on the ground model outlined in Section 6 and the proposed development, the following geotechnical considerations are expected to be pertinent to the Fast Track Consenting, preliminary design, and earthworks/enabling works for the wider development[9]:

- Site seismicity/site subsoil class;
- Liquefaction and cyclic softening potential and associated effects;
- Earthworks considerations including trafficking, fill suitability, fill conditioning and fill compaction;
- Temporary and permanent stability of cut and fill batters;
- Preliminary retaining wall options and design considerations;
- Groundwater drawdown considerations and effects;
- Consolidation and secondary (creep) settlements beneath fill and structure loads;
- Preliminary foundation options and earthworks-based ground improvement measures (e.g. engineered fill crust and preloading) to support shallow foundations;
- Construction and design considerations for internal civil servicing; and
- Subgrade strengths and protection for slabs and pavements.

These geotechnical considerations are addressed in the following subsections.

7.2 Seismic Considerations

7.2.1 Seismic Subsoil Class

The Seismic Subsoil Class has been assessed based on NZS1170.5:2004. On the basis that:

- The highly to moderately weathered Limestone has an unconfined compressive strength between 1 – 50 MPa;
- The combined thickness of overlying soils (i.e. fill, alluvium and residually to completely weathered Mahurangi Limestone) at the completion of earthworks will be less than 20 m; and
- The soils present at the site have a minimum undrained shear strength of 12.5 kPa (i.e. are not "very soft")

Areas of the site will fall into the classification of either Class B (Rock) or Class C (Shallow Soil). Areas where the post-earthworks combined soil thickness is less than 3 m will be Class B and areas with greater than 3 m combined soil thickness will be Class C.

Figure 7-1 below presents the NZS1170.5 seismic subsoil class, based on the existing depths to rock and proposed earthworks cut/fill depths[10]. If cut/fill depths change (including if rock is sub-excavated and backfilled) this may change the subsoil class. Additional investigation may be required to confirm the site subsoil class in some areas of the site. In the absence of certainty in rock depth versus ground level, designers should refer to Initia or alternatively adopt the more conservative design spectra.

Structures straddling Class B and Class C zones should be designed considering the envelope of seismic loading for both the Classes B and C spectra (the most conservative of the two) or be further investigated to confirm site class.

It is noted that Standards New Zealand are currently drafting a replacement seismic design standard, TS1170.5, that will provide an updated subsoil classification system based on VS_{30} measurements. At present, there is no confirmed timeframe for when TS1170.5 will be cited in the Building Code.

[9] Excluding the Surf Lagoon, which is excluded from this report.
[10] McKenzie & Co (8/12/2025). *Earthworks Cut Fill Overall Plan*. Dwg No. 3325-2-2100 Rev B.

Depending on the development programme and the release timeframe for TS1170.5, the seismic subsoil class may need to be re-assessed at the building design stage in accordance with TS1170.5.

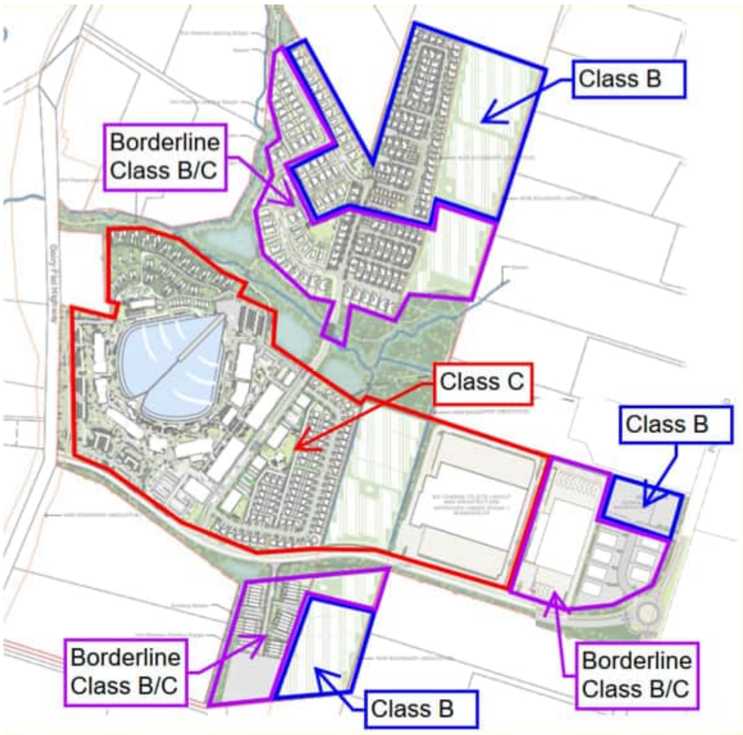

Figure 7-1: Markup of Approximate Seismic Subsoil Class (based on proposed earthworks finished levels)

7.2.2 Geotechnical Seismic Design Parameters

Design peak ground acceleration (PGA) and associated magnitude (M_w) for Serviceability (SLS) and Ultimate (ULS) Limit States has been assessed in accordance with the MBIE Geotechnical Guidelines Module 1, based on the following assumptions:

- Design life 50 years;
- Importance Level 2 (in accordance with NZS1170.0) except for the proposed data centres which are understood to be Importance Level 4 (to be confirmed by designers);
- Site Subsoil Classes B or C (in accordance with NZS1170.5);

The derived geotechnical seismic design parameters are presented in Table 7-1 below.

The building designers should confirm the design life and importance level for specific buildings at the building design stage and re-assess seismic demands if they are not consistent with the assumptions in this report.

Table 7-1: Seismic Design Parameters for Geotechnical Design

Design Life and Importance Level	Seismic Design Case and Annual Probability of Exceedance	Peak Ground Acceleration (PGA)	Effective Earthquake Magnitude (M_w)
50 years – IL2	SLS (1 in 25)	0.05g	5.9
	ULS (1 in 500)	0.19g	6.5
50 years – IL4	SLS1 (1 in 25)	0.05g	5.9
	SLS2 (1 in 500)	0.19g	6.5
	ULS (1 in 2500)	0.28g	5.9

7.2.3 Liquefaction Susceptibility and Triggering

Liquefaction occurs when soil loses shear resistance under cyclic loading (cyclic shear strains). For liquefaction to develop, the following conditions must be present:

- Material with the potential to densify under cyclic loading (usually loose silt and sand mixtures);
- Saturated ground (beneath the groundwater level); and
- Cyclic shear loading (usually a seismic event).

In general, the susceptibility of the soils at the site to liquefaction is considered to be very low. This is due the plasticity of the Tauranga Group alluvium (Plasticity Index > 12%) and the age and stiffness/density of weathered Mahurangi Limestone soils. Notwithstanding this, a CPT-based liquefaction susceptibility and triggering assessment has been undertaken for SLS and ULS seismic loading cases using the Initia CPTs and the following method/assumptions:

- Boulanger & Idriss (2014) method;
- $I_c \leq 2.6$ liquefaction susceptibility cut-off;
- Seismic loading for IL2 structures with a 50 year design life, in accordance with Table 7-1;
- Groundwater depth conservatively applied at 1 m below the assessed ground level;
- Filling has conservatively been neglected and proposed cuts have been modelled.

The liquefaction triggering assessment indicates that under SLS loading (1 in 25 year return period) liquefaction is not triggered. Under ULS loading (1 in 500 year return period), liquefaction triggers in occasional, very thin (maximum 200 mm thick) localised lenses at depths greater than 4.5 m below existing ground level. The assessed effects of liquefaction are “Insignificant – L0” in accordance with Table 5.1 of MBIE Seismic Module 3.

Table 7-2 below summarises the liquefaction triggering assessment and analysis outputs for ULS loading are attached in Appendix B.

Table 7-2: Results Summary from Liquefaction Triggering Assessment

	SLS Loading (1 in 25 year event)	ULS Loading (1 in 500 year event)
LPI Number	0	0 – 1
LSN Number	0	0 – 3
Free Field Settlement	0	0 – 20 mm
Non liquefiable Crust Thickness	N/A	> 4.5 m
MBIE Seismic Performance Level[11]	L0 – Insignificant	L0 – Insignificant

It is noted that a liquefaction triggering assessment has not been undertaken using the Aurecon CPTs for proposed data centre as digitalised CPT data was not made available. A previous liquefaction review by Aurecon[12] noted that the soils were not susceptible to liquefaction.

7.2.4 Seismic Cyclic Softening Susceptibility and Triggering

Plastic silts and clays can be susceptible to strength loss from cyclic strains during a seismic event. Soils which are lower strength, normally consolidated soils, and soils with high natural water contents are generally most susceptible to cyclic softening.

A seismically induced cyclic softening assessment has been undertaken at the site using the Initia CPT data and the Boulanger & Idriss (2007)[13] method. A Cone Factor of $N_{kt} = 12$ has been adopted to

[11] In accordance with Table 5.1 of MBIE Seismic Module 3.

[12] Aurecon (1 June 2023). *Spark Dairy Flat Campus. Geotechnical Investigation Report*. Ref: 523578. Rev A.

[13] Boulanger R. & Idriss I. M. (June 2007). *Evaluation of Cyclic Softening in Silts and Clays.* Journal of Geotechnical and Geoenvironmental Engineering.

estimate existing soil undrained shear strengths using the CPT traces based on calibration against shear vane testing.

Results of the cyclic softening assessment indicate that, under SLS level seismic loading, cyclic softening is not triggered. Under ULS seismic loading (1 in 500 year return period), cyclic softening is generally not triggered, except for some localised lenses of alluvium, up to 0.2 m thick. The lenses were encountered at depths greater than 3.5 m below existing ground levels.

A cyclic softening assessment has not been undertaken by Initia using the Aurecon CPTs for the proposed data centre as digital CPT data was not made available. A previous cyclic softening review by Aurecon[14] concluded that cyclic softening is not expected to occur for a 1 in 2,500 year ULS design earthquake (for IL4).

7.2.5 Design Considerations for Liquefaction and Cyclic Softening Effects

The subsections below provide preliminary design advice in relation to the design of structures for liquefaction and cyclic softening effects. This advice should be reviewed by the Geotechnical Engineers for the buildings, particularly for the Data Centre structures, which may have specific performance criteria.

Shallow Foundation Systems:
Buildings founded on shallow foundations should not require specific detailing in relation to liquefaction or cyclic softening effects due to the presence of a thick non-liquefiable crust and negligible liquefaction induced settlements under ULS seismic loading.

Piled Foundation Systems:
Liquefaction and cyclic softening effects (i.e. down-drag, loss of skin friction and kinematic displacement loading) on deep piles should be reviewed by the building designers but is not expected to govern design.

For lightly loaded piles, terminating within alluvium, it is recommended that end bearing is neglected unless a review is undertaken to check piles are founding on material that is not subject to liquefaction or cyclic softening.

Lateral Spreading:
The risk of lateral spreading towards a free face (e.g. the creek channels) is negligible as only thin, discontinuous lenses of potentially liquefiable soils are present.

7.3 Earthworks Considerations

The following subsections outline general geotechnical considerations for bulk earthworks. Additionally, reference should be made to the following sections of this report which are relevant to earthworks design and construction:

- Sections 7.5 and 7.6 provide advice in relation to batter stability and retaining walls;
- Section 7.8 provides advice in relation to subgrade preparation and protection for pavements;
- Section 7.9 provides advice in relation to recommended ground improvement works for building platforms. It is recommended that ground improvement works, where required, are generally incorporated into the bulk earthworks programme.

7.3.1 Scope of Earthworks

[14] Aurecon (1 June 2023). *Spark Dairy Flat Campus. Geotechnical Investigation Report*. Ref: 523578. Rev A.

The subsections below summarise the proposed scope of earthworks in accordance with the McKenzie and Co. draft earthworks drawings[15] (dated 8/12/2025).

7.3.1.1 Earthworks Completed to Date

Earthworks commenced during the 2024/2025 summer season under the Stage 1 Consent and development plan. Works included topsoil stripping and placement of engineered, site-won fill across some of the Surf Amenity, Village Centre, and Neighbourhood South Precincts, with fill heights up to approximately 2.5 m.

The fill materials, sourced primarily from temporary cuts (up to ~4 m deep) within the North-East Neighbourhood Precinct, comprised blends of inorganic Tauranga Group alluvium, residual Mahurangi Limestone soils, and weathered Mahurangi Limestone rock.

Further detail is provided in the Initia Season 1 Earthworks Progress Report[16]. The post-earthworks surface, captured by drone survey on 3 June 2025, is shown on the geological cross-sections in Appendix A.

7.3.1.2 Proposed Stage 2 Earthworks

The McKenzie & Co. cut/fill plan[17] (presenting cut/fill isopach contours comparing design levels to current site levels, i.e. as at completion of 2024/2025 earthworks) indicate a total cut volume of approximately 276,000 m^3 and total fill of approximately 283,000 m^3. Considering a compaction factor of 0.9, the presence of unsuitable soils (too wet or organic to be placed and compacted as engineered fill), and requirements to sub-excavate (excavate below ground level), there will be a fill deficit. Therefore, importation of fill material will be required. Accordingly, reuse of as much site-won material as possible will be a key consideration for earthworks design and management.

Apart from the lagoon, bulk filling is generally proposed across the central development precincts (namely Surf Amenity Precinct, Village Centre Precinct, Accommodation Precinct South and Data Centres) to raise the site to typical levels between RL 55 – 57 m (c.f. existing typical site levels of RL 54 – 55 m). Bulk filling, up to approximately 3 m in height is also proposed across the western edge (existing low point) of the North-Western Neighbourhood Precinct. Fill up to approximately 2 m is also proposed in the North-Eastern Neighbourhood Precinct which was used as a borrow area during the 2024/2025 earthworks season to source Limestone rock.

Bulk cuts up to approximately 6.5 m deep are proposed within the Northern and Southern Solar Farms to provide a source of weathered Limestone rock for the project. In addition, cuts of up to approximately 2 m are required to re-grade elevated areas within the North-Western Neighbourhood Precinct and the Stage 5a Vacant Lot Subdivision. Cuts up to approximately 2 m deep will be needed within the Stream Park and Accommodation Precinct to provide increased flood storage.

Grade changes across the site are proposed to be achieved by a combination of cut/fill batters and retaining walls. Recommendations for batter slopes and retaining wall design are presented in Sections 7.5 and 7.6.

Specific Earthworks for Ground Improvement

As discussed in Section 7.9.1, the Tauranga Group alluvium generally has insufficient bearing strength for shallow foundations, and building settlements may exceed structural tolerances, particularly for heavier buildings and those positioned in areas of deeper alluvium (central low-lying areas of the site).

[15] McKenzie & Co (8/12/2025). *Earthworks Final Contour Overall Plan.* Dwg No. 3325-2-2000. Rev B.
[16] Initia Ltd (7/11/2025). *Auckland Surf Park – Season 1 Earthworks. Earthworks Progress Report.* Ref: P-001537-1. Rev 0.
[17] McKenzie & Co (8/12/2025). *Earthworks Cut Fill Overall Plan.* Dwg No. 3325-2-2100 Rev B.

To enable shallow foundation systems, ground improvements will likely be required which may include:

- Construction of an engineered fill crust of appropriate thickness to provide a suitable bearing layer for shallow foundations; and
- Preloading to reduce long term static settlements.

Further detail on ground improvement options and design are provided in Section 7.9.1. However, it is strongly recommended that ground improvement works are integrated into the bulk earthworks scope to leverage scale efficiencies and to minimise re-work during building construction. In particular:

- Areas requiring an engineered fill crust that is thicker than the baseline bulk fill thickness should be sub-excavated prior to bulk filling; and
- Unsuitable soils stripped during bulk earthworks should be strategically stockpiled to act temporary preload material.

The required extent of ground improvement works will be dependent on the finalised floor levels, loads and foundation types of the buildings. Where possible, it is recommended that these variables be confirmed ahead of commencing earthworks.

It is noted that fill placed during the 2024/2025 season in the Surf Amenity, Village Centre, and South Neighbourhood Precincts has not specifically accounted for crust thickness requirements beneath proposed buildings as earthworks were completed relative to the Stage 1 Masterplan layout. A specific geotechnical review of these areas is recommended before further earthworks is progressed to minimise potential rework.

The sections below outline the key geotechnical considerations for earthworks cut and fill.

7.3.2 Topsoil, Site Stripping and Subsoil Drainage

Topsoil is present across the whole site, ranging from 0.1 to 1.0 m thick, with the thickest deposits (>0.5 m average) within the Village Centre Precinct and Neighbourhood South Precincts. Localised uncontrolled fill is also present in isolated areas.

Topsoil and vegetation are to be stripped from all pre-fill subgrades, building platforms, and pavement areas. Buried tree stumps are known to be present at shallow depth within the alluvium. Where these are encountered, they should be removed and backfilled with engineered fill, unless otherwise approved by the geotechnical engineer.

Existing uncontrolled material should also be stripped beneath future building platforms and pavements, unless appraised by a geotechnical engineer as having appropriate composition and strength/density for the future development. Depending on its composition, it may be feasible to re-work existing fill materials.

Other unsuitable materials at subgrade level, such as highly organic soils or soft saturated soils (e.g. within existing farm drains), should also be mucked out at the commencement of earthworks.

Within low-lying areas, temporary subsoil drainage may be required to manage surface and subsurface inflows and to maintain conditions suitable for compaction of fill, particularly during shoulder seasons. Drainage systems could comprise perforated Nexus Hiway (or equivalent) pipes surrounded by drainage aggregate and a non-woven geotextile wrap or sand/gravel filter. Final drainage locations are to be confirmed by the geotechnical engineer during earthworks.

Permanent underfill drainage may also be required beneath fill embankments in the North-Western Neighbourhood Precinct (refer Section 7.5.3).

7.3.3 Excavatability of Mahurangi Limestone

Based on the earthworks cut and fill plan[18], bulk cuts within the following areas are expected to extend into highly to moderately weathered Mahurangi Limestone rock:

- Solar Farm North;
- Solar Farm South;
- Neighbourhood Precinct North-East;
- Neighbourhood Precinct North-West; and
- Stage 5a Vacant Lot Subdivision

Earthworks completed to date have included excavation of limestone rock within the Neighbourhood Precinct North-East. Large excavators (≥25 t) fitted with rock buckets were sufficient to excavate the material, despite the high strength of the rock, due to the close fracture spacing of the rock mass. Similar methods are expected to be effective across other areas; however, blocky zones with wider fracture spacing may require specialised tooling (e.g., single-tyne rippers or pneumatic breakers) for efficient excavation.

7.3.4 Site Trafficking and Commencement of Filling

Across areas designated for bulk filling, the pre-fill subgrade typically comprises firm to stiff Tauranga Group alluvium, including CLAY and organic CLAY. These soils generally have insufficient strength to support heavy earthworks plant (e.g. Moxys, sheepsfoot rollers) or achieve effective compaction. They also soften rapidly under trafficking, particularly in wet conditions. Therefore, a "working layer" is likely to be required on the surface of the alluvium to enable compaction of bulk fill over. Based on earthworks completed to date, an effective working layer generally comprises a 300 to 500 mm thick layer of coarse rock fill (e.g. GAP150 or a similarly graded blue-brown rock or site-won Limestone material) placed directly on the alluvium and static rolled, allowing it to "punch in" to the alluvial soils. This should generally be overlain by a heavy geotextile and bi-axial geogrid (e.g. a composite Gridtex product), placed prior to compacting engineered bulk fill above. A working layer thickness of approximately 300 mm has proved effective in the inorganic soils, however, an increased layer thickness (e.g. up to approximately 500 mm) may be required in areas with organic soils or during periods of marginal weather.

Where pre-fill surfaces are sloping, they should be benched to allow the fill to be keyed in. Consideration for stability will also be required as discussed in Section 7.5.3..

A preliminary compaction specification for the working layer is provided in Section 7.3.7 below.

7.3.5 Site Won Fill Suitability

Based on the proposed cut/fill plan, site-won materials obtained from bulk cut earthworks are expected to include:

- Tauranga Group alluvium across all areas of cut, typically comprising firm to stiff CLAY and Silty CLAY, with trace to minor organics. However, organic CLAY will be widespread within the Neighbourhood Precinct South, Solar Farm Central and Data Centre areas; and
- Residually to completely weathered Mahurangi Limestone Soils and Highly to moderately weathered Mahurangi Limestone Rock in the areas outlined in Section 7.3.3 above.

An assessment of soil re-useability for engineered fill has been undertaken for the broad material types in relation to the preliminary engineered fill specification outlined in Section 7.3.7 below.

[18] McKenzie & Co (8/12/2025). *Earthworks Cut Fill Overall Plan.* Dwg No. 3325-2-2100 Rev B.

Tauranga Group Alluvium – Organic Clays

Laboratory testing on shallow organic clays (sourced within the South Neighbourhood Precinct) indicate the materials typically have an organic content greater than 10% and a liquidity index of >75%. Therefore, these materials will generally not be suitable for re-use as engineered fill.

Tauranga Group Alluvium – CLAY and Silty CLAY, with no to minor organics

Laboratory standard compaction testing of Tauranga Group CLAY mixtures measured the in-situ water content of the soils as being 6 – 24 % (typical 16%) higher than optimum for compaction. The Atterberg Limit tests also measured typical liquidity indexes of about 30 – 40% which are higher than typical maximum liquidity indexes of 10 – 20% required for engineered fill placement. Based on this, site-won alluvium will generally need to be dried by approximately 5 – 25% (reduction in water content) with an average of about 10 to 15%. This level of drying is very high and unusual for Auckland earthworks projects. On this basis the alluvial soils on site can be considered marginal with respect to use as engineered fill. Conditioning of these materials will be required to enable their re-use, as outlined below. Some wetter materials may be impractical to re-use.

Residually to Completely Weathered Mahurangi Limestone Soils

Atterberg Limits testing on a limited number of residual soil samples indicated the natural water content of the soils was 11 – 26% above the plastic limit (liquidity indexes of 24 – 39%). This indicates the soils may generally be wet of optimum water content and may require some conditioning (drying) to be suitable for re-use during the summer earthworks season.

Mahurangi Limestone Rock (Lime-rock)

Highly to moderately weathered Mahurangi Limestone sourced from the North-Eastern Neighbourhood Precinct during the 2024/2025 earthworks season was generally recovered as a relatively well graded gravel material, attributable to its close fracture spacing. Where larger particles were recovered, these can be broken under heavy compaction plant (e.g. heavy sheepsfoot rollers). Therefore, during the summer earthworks season, lime-rock should generally suitable for use as both a lower “working layer” and a bulk fill material (often blended with cohesive soils). The lime-rock is not a suitable “all-season” fill as it can weather break down into a cohesive soil in wet conditions.

Recommendations for maximising site-won material re-use:

As outlined above, both the alluvium and residual soils at the site are generally wet of optimum water content, particularly the alluvium. The alluvium is therefore likely to require conditioning to enable re-use, involving laying out in thin layers and discing/drying for up to approximately 1 week of fine weather, prior to compaction. Drying periods will likely be slightly less for Residual soils.

Given the marginal and wet nature of the soils, options to mitigate the risks associated with this should be fully considered. These include:

- Programming of bulk earthworks to coincide with the summer peak of the earthworks season (typically January to March);
- Blending of lime-rock (or an alternative imported material such as blue-brown rock) with residual soils/alluvium. Blending of lime-rock and site-won soils was undertaken during the 2024/2025 earthworks season and reduced required drying times;
- Use of lime or cement conditioning. Ideally lime/cement conditioning should be limited for bulk filling due to cost but if drying is not practical or fill needs to be placed in the shoulder seasons this is an option for alluvial soils.

Materials unsuitable for use as engineered fill (e.g. topsoil, organic alluvium, wet alluvium) should be utilised as temporary preload embankment fill (as outlined in Section 7.9.1.3). Preload fill does not require compaction to an engineered standard, however, wet and organic unsuitable soils may be challenging to place and track roll outside of dry weather periods.

7.3.6 Imported Fill Materials

Due to the anticipated fill deficit, imported fill is likely to be required. Suitable fill materials may include soft pit run (SPR), inorganic clay/silt or a well graded hardfill (e.g. GAP65 or a similarly graded crushed concrete). Fill materials should be reviewed and approved by the geotechnical engineer prior to import to site.

7.3.7 Preliminary Compaction Specification

Bulk fill supporting future structures and pavements should be placed and compacted in accordance with NZS 4404:2010 and a project-specific earthworks specification which is reviewed and approved by Initia and the project Civil Engineer.

A preliminary compaction specification is provided below for the following fill types:

- Granular Fill Working Layer (refer Section 7.3.4 above);
- Granular Engineered Bulk Fill Material (e.g. Limerock, GAP65 or similar); and
- Cohesive Engineered Bulk Fill Material (e.g. weathered Limestone soils and blended inorganic alluvium).

The proposed compaction testing should be supplemented by regular proof rolling under heavy compaction plant (e.g. fully loaded Moxy or similar), observed by the project Geotechnical Engineer.

First Lift- Granular Working Layer (GAP150, Lime Rock or similar)

Clegg Hammer:	Minimum CIV:	15
	Average CIV:	18 (5 consecutive tests)

Engineered Well Graded Granular Fill (GAP65, Lime rock or similar)

Clegg Hammer:	Minimum CIV:	22
	Average CIV:	25 (5 consecutive tests)
<u>OR</u>		
Solid density – NDM:	Minimum Solid Density	90%
	Average Solid Density	92%

Site-Won Inorganic Cohesive Engineered Bulk Fill

Undrained shear strength	Minimum Su:	120 kPa
	Average Su:	140 kPa (any 5 consecutive tests)

<u>and</u>

Air voids:	Maximum value	10%
	Average value	8% (any 5 consecutive tests)

7.3.7.1 Compaction and Bulking Factors

A compaction factor of 0.9 (comparison of compacted fill density to natural soil density) may be assumed for earthworks calculations of site-won soils. This value can be further verified during construction by undertaking additional nuclear densometer testing on natural and engineered fill material types as works progress. A bulking factor of 1.4 should be assumed for comparison of in situ loose volume measures for hauling and stockpiling purposes.

7.3.8 Settlement of Fill

The Tauranga Group alluvial soils are moderately to highly compressible and will consolidate (settle) beneath bulk filling. The replacement of sub-excavated topsoil and alluvial soils with denser engineered fill material will also apply a net load with resulting consolidation settlements.

Unless structures/infrastructure are designed to tolerate fill induced settlements (in addition to the settlements induced by the loads applied by the structures) a "hold period" will be required between the completion of filling and the construction of the buildings and infrastructure. The magnitude of settlement and the hold period duration will vary across the site depending on the thickness, compressibility and permeability of the Tauranga Group alluvium.

Existing Fill Settlement Monitoring

Settlements induced from fill placed during the 2024/2025 earthworks season have been monitored using settlement plates that were installed at pre-fill subgrade level, prior to the commencement of filling. Settlement plate locations are presented on Figure 7-2 below.

At each settlement plate monitoring location, the net fill load has been estimated based on the following assumptions:

- Increase in soil density to original ground level resulting from replacement of topsoil/alluvium with engineered fill. This equates to approximately 3 kPa/m thickness;
- Bulk fill load of 19 kPa/m thickness for engineered fill placed above original ground levels. The engineered fill comprised blended site-won materials including alluvium, residual soils and Limestone.

Figure 7-3 below plots both the measured settlements and the net applied load over time.

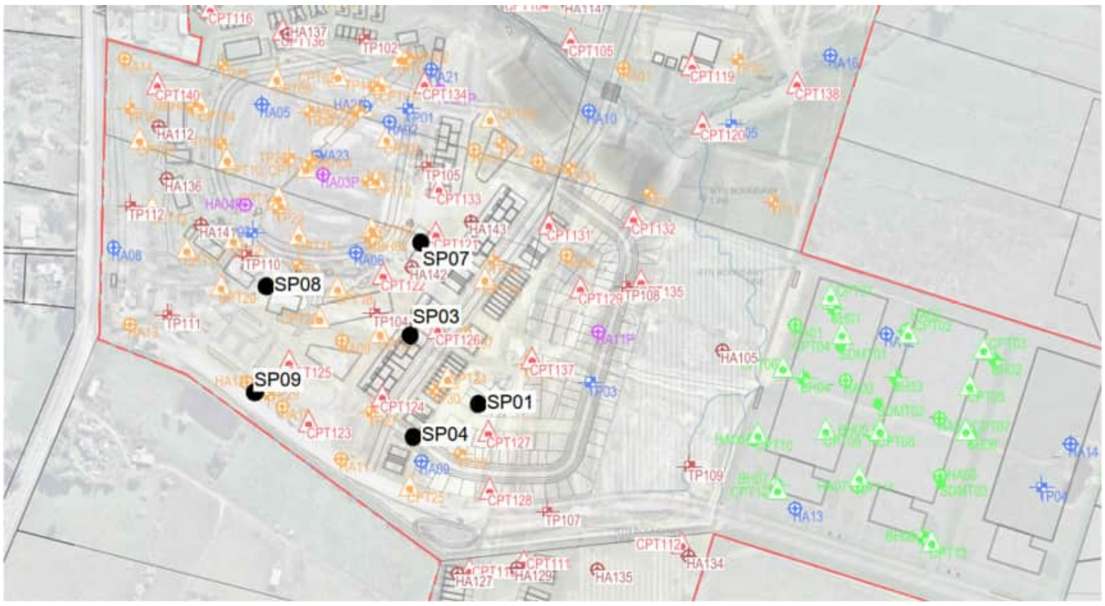

Figure 7-2: Existing settlement plate monitoring locations

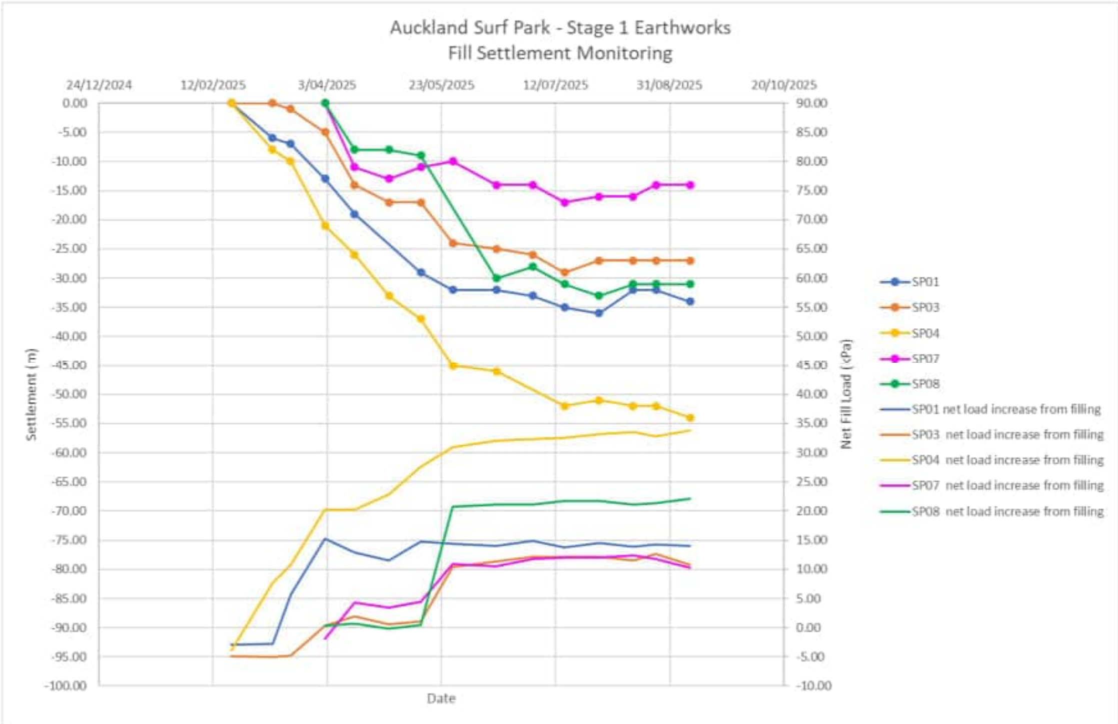

Figure 7-3: Existing settlement plate monitoring data

As shown on Figure 7-3, fill was placed until early April 2025 at the SP01 location and until late May 2025 for the remaining settlement plate locations. Therefore, hold periods of approximately 5 months for SP01 and 3.5 months for the remaining plates were observed between the completion of filling and the latest survey monitoring round on 9/09/2025.

The survey monitoring data indicates that settlement rates attenuated at the monitored locations, inferred to have generally reaching an equivalent "t90", within 3 months or less of filling.

A comparison between the monitored settlement magnitudes and predicted estimates (using the empirical method presented by Robertson & Cabal (2015)[19]) from the nearest CPT investigations indicates monitored settlement magnitudes typically fall within the range predicted by CPTs, typically near the lower end of the range. It is noted though that CPT predictions can vary considerably though between adjacent investigations.

Preliminary Estimates Fill Settlement Magnitudes and Hold Periods

A preliminary estimate of filling induced settlement magnitude within each development precinct has been undertaken using CPT data and is presented in Table 7-3 below. As noted above, based on the fill monitoring, it is expected that actual settlements will likely fall within or below this range.

A qualitative estimate of the hold periods required for fill induced settlements to attenuate is also presented in Table 7-3 below. Across most areas of the site, settlement rates are expected to be comparable to or faster than that measured at the existing settlement plate locations (as the monitored locations are located across the area of maximum alluvium thickness). However, slower settlement rates may occur within the Neighbourhood Precinct South, Solar Farm South and Data Centre areas where higher compressibility organic clay and peat layers are present.

[19] Robertson, P.K. and Cabal, K.L. (2015). Guide to Cone Penetration Testing for Geotechnical Engineering, 6th Edition, Gregg Drilling and Testing.

Table 7-3: Indicative filling induced settlement magnitudes and hold periods

Development Precinct	Preliminary Fill Settlement Magnitude Estimates (mm)			Estimated Hold Period for Settlement Attenuation after Completion of Filling
	1 m fill above existing ground	2 m fill above existing ground	3 m fill above existing ground	
Neighbourhood Precinct North-East & Stage 5a Vacant Lot Subdivision	< 25 mm	< 25 mm	< 25 mm	< 3 months
Neighbourhood Precinct North-West	< 25 mm	Typically < 25 mm Locally up to 50 mm	Typically < 25 mm Locally up to 75 mm	< 3 months
Live-Work Precinct	Typically < 25 mm Locally up to 40 mm	Typically 10 – 40 mm Locally up to 75 mm	Typically 20 – 50 mm Locally up to 100 mm	< 3 months
Accommodation Precinct	30 – 50 mm	60 – 100 mm	80 – 150 mm	2 – 5 months
Surf Amenity Precinct	20 – 75 mm	40 – 140 mm	50 – 200 mm	2 – 5 months
Village Centre Precinct	40 – 100 mm	80 – 200 mm	100 – 300 mm	3 – 6 months
Neighbourhood Precincts (South)	50 – 120 mm	100 – 250 mm	150 – 350 mm	3 – 9 months
Data Centres	Not assessed[1] but expected to be the highest compressibility area of the site			Not assessment but may be > 12 months

[1]: No digitalised CPT data available at present. However, this area is inferred to have the highest compressibility alluvium and greatest subsequent settlement magnitudes within the site.

Monitoring and Programme Recommendations

Given the settlements presented above will generally not be tolerable for buildings and settlement sensitive infrastructure, a post-filling 'hold' period will be required to enable consolidation to 'complete' prior to building and infrastructure construction. Estimated hold periods are presented in Table 7-3. It must be appreciated that settlement rates can be variable and it is recommended that additional contingency is factored into the programme.

Settlements beneath filled areas should be monitored via regular surveying of settlement plates, typically installed in a grid at 20 – 40 m c/c centres. The survey plates must be installed and baselined prior to the commencement of filling. Settlement plate locations should be specified by the geotechnical engineer.

Even if the programme allows for hold periods considerably longer than required, it is recommended that fill induced settlements are still monitored beneath future building platforms as the information can be used for refining and verifying building settlement estimates.

If hold periods need to be reduced to meet programme requirements, this can be achieved either via surcharging or the installation of wick drains. Surcharging involves placing additional temporary fill above the finished bulk fill levels. Wick drains are vertical prefabricated drains which would be installed in a grid pattern to the base of the alluvium. Whilst these can significantly accelerate settlements they have a relatively high cost premium.

7.4 Groundwater Drawdown Considerations

With reference to Section 6.4, the following groundwater regimes are inferred to be present at the site:

- Seasonal shallow perched groundwater, typically sitting within the topsoil and perched above the lower permeability alluvial clays. Perched groundwater is typically present within the low-lying central area;
- Static groundwater, typically between approximately RL 51 – 52 m, grading down to approximately RL 49 m at the western end of the existing creek; and
- Percolating groundwater within the fractured Mahurangi Limestone rock mass, within the northern, eastern and southern hillsides, flowing down towards the low-lying central area of the site; and
- Potential artesian water pressures within Mahurangi Limestone rock.

The proposed earthworks plan[20] has been reviewed to assess where bulk cuts may intercept and potentially drawdown groundwater and to assess the potential effects of this (predominantly settlement). A summary of the assessment is presented below relative to the various bulk cut areas as presented on Figure 7-4. This figure presents the absolute level difference between the proposed earthworks levels and the original surveyed ground levels (considered most relevant for assessing groundwater drawdown potential effects). This differs from the cut/fill plan prepared by McKenzie & Co which considers topsoil stripping and earthworks already completed to date.

Figure 7-4: Delineation of bulk cut areas > 1m in depth. (Note: Green/yellow/red denotes net cut, turquoise/blue denotes net filling)

[20] McKenzie & Co (8/12/2025). *Earthworks Final Contour Overall Plan.* Dwg No. 3325-2-2000. Rev B.

Areas 1.1, 1.2 and 1.3:

Bulk cuts up to 6.5 m, 3.5 m and 4.5 m deep are proposed for Areas 1.1, 2.2 and 1.3 respectively. The proposed cuts are located on the elevated hillsides where groundwater was generally not encountered above the Mahurangi Limestone rock. The proposed cuts may intercept percolating flows in the Limestone rock mass, however, offsite settlement effects will be negligible due to the age and stiffness of the Limestone materials and because groundwater is not present in the overlying soils.

Area 2:

A cut to approximately RL 53.5 m is proposed in Area 2. This involves internal cut batters up to 4.5 m deep, however, cuts along the eastern boundary are less than 2m deep. Whilst shallow perched groundwater may be encountered during wet periods, the groundwater conditions presented in Table 7-4 indicate the cut will not extend below historic groundwater levels and will therefore not cause offsite settlement effects.

Table 7-4: Groundwater Observations – Area 2

Investigation ID	Investigation Ground Elevation (m RL)	Observed Groundwater Depth (m)	Observed Groundwater Elevation (m RL)
TP07	58.9	> 2 (pit terminated in Limestone and was dry)	< 56.9
TP08	56.7	> 2.3 (pit terminated in Limestone and was dry)	< 54.4
TP10	54.7	> 4 (pit terminated in Alluvium and was dry)	< 50.7
T+T_HA16	54.3	2.2	52.1

Area 3:

Area 3 comprises a cut to RL 51.6 m to form a stormwater pond. The elevation of this cut sits above the ground level on the adjacent site boundary (approximately RL 50 m). The cut will therefore not drawdown groundwater levels beyond the boundary or cause offsite settlement effects.

Area 4:

Area 4 comprises cuts to approximately RL 51 to 52 m to form stormwater ponds and a flood storage area. These cuts will intercept shallow perched groundwater (in the topsoil) and may intercept the static groundwater by up to 1 m. However, due to the low permeability of the alluvium and the minimum 100 m offsite to the site boundary, groundwater drawdown effects will not extend beyond the site boundary.

Area 5:

Area 5 comprises a 1V:3H cut to steepen an existing north-facing slope above the creek which runs along the northern boundary. The cut, up to 2 m below existing ground level, will not extend below the existing creek channel invert. On this basis:

- The cut will not drawdown groundwater beyond the northern boundary; and
- The cut may drawdown groundwater to the south as it may intercept the existing groundwater gradient within the north facing slope.

A conservative estimate of the potential radius of influence of groundwater drawdown towards the south has been undertaken using the empirical Sichardt Formula presented in CIRIA C750. Conservatively assuming a groundwater drawdown of 2 m, a Sichardt coefficient, C, of 2000 and a horizontal hydraulic conductivity[21], $k = 3 \times 10^{-7}$ m/s, the conservative maximum radius of groundwater drawdown is estimated to be 2.5 m. Therefore, groundwater drawdown and associated settlement effects at the nearest existing structures at 1368 Dairy Flat Highway, offset a minimum of 40 m, are

[21] This was the upper bound (therefore conservative) permeability calculated from rising head testing at the site which is detailed in the Stage 1 Geotechnical Interpretative Report.

assessed as negligible.

Area 6:
Area 6 comprises the lagoon which was previously assessed and consented in the Stage 1 Reporting[22]. The previous assessment concluded there will be no offsite groundwater drawdown effects beyond the project boundary. Designers should refer to the Stage 1 Reporting for sequencing considerations within the project site.

7.4.1 Auckland Unitary Plan Considerations

The Auckland Unitary Plan (AUP) generally requires specific consent to be applied for where excavations extend below the natural groundwater level. Based on the groundwater monitoring to date, the groundwater regimes presented in Section 6.4, "natural" groundwater will be intercepted by the proposed works, which includes:

- Shallow seasonal perched groundwater;
- Percolating groundwater within the Mahurangi Limestone rock-mass on the elevated hillsides; and
- Static groundwater, localised near the centre of the site.

On this basis, the proposed development has been assessed against the AUP Groundwater Take and Divert Permitted Activity criteria (Chapter E7). Based on the assessment, attached in Appendix F, the development does not meet the permitted activity criteria for groundwater. Accordingly, a groundwater take/divert resource consent is expected to be required.

In accordance with the groundwater drawdown effects assessment, presented in Section 7.4 above, there will be no offsite groundwater settlement effects relating to the proposed works. Accordingly, no specific groundwater or settlement monitoring is considered to be required as a condition of the groundwater consent.

7.5 Slope Stability Considerations

The proposed development includes a series of permanent cut batters, fill embankments and retaining structures. The subsections below provide preliminary advice and analysis for temporary and long-term site stability of cut batters and fill embankments. Detailed design of retaining walls will also require specific assessment/design for global stability, as discussed in Section 7.6. Further investigations, analysis and design will be required to detail retaining structures and stabilisation measures for embankments, where required.

7.5.1 Existing Site Stability and General Stability Risks

The existing site is generally flat to gently sloping (<1V:5H), except for locally steeper slopes (between 1V:3H to 1V:2H and up to 2–3 m high) above the western creek/farm drain within the North-Western Neighbourhood Precinct.

A site walkover and hill-shade review did not identify evidence of existing instability except for a few areas of localised slumping above the western creek/farm drain. These areas have scarps/tension cracking approximately 5 – 10 m in length, are located less than 10 m upslope of the drain invert. The instability is inferred to comprise shallow slumping within the low-strength alluvium. Fill embankments and retaining walls are proposed along this western boundary and their design will need to consider this existing slumping, as discussed in Section 7.5.3.

[22] Initia Ltd (June 2023). AW Holdings 2021. Auckland Surf Park Community. Geotechnical Interpretative Report. Ref P-001537. Rev B.

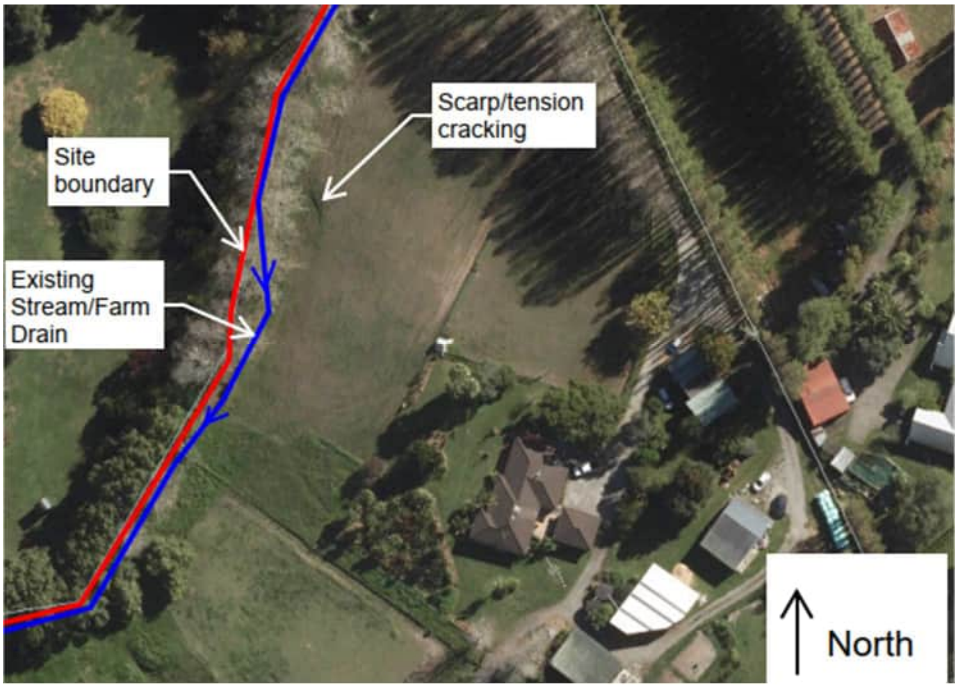

Figure 7-5: Existing slumping observed adjacent to western boundary

In general, the Tauranga Group Alluvial soils across the site are of low strength and therefore pose a risk of instability if cuts into the material, or fill placed above the alluvium are not appropriately battered and formed. Failure mechanisms can include deep seated rotational failures and shallow surficial creep movements.

The Mahurangi Limestone materials at the site form part of the wider Northland Allochthon which, due to its emplacement, can be highly fractured and pervasively sheared. Other subsets of Northland Allochthon (e.g. Hukerenui Mudstone) are known to have a high risks of instability at gentle slope angles (i.e. less than 1V:4H) either within the rock mass or at the interface between weathered soils and rock. However, compared with other Northland Allochthon materials, Mahurangi Limestone typically has a much lower stability risk and natural hills are commonly observed stable at grades of 20^{o} or greater[23]. Steep cuts within Mahurangi Limestone also commonly remain stable as demonstrated by the highwall cuts shown on Figure 7-6 of the former Redvale quarry, located approximately 1km south-west of the project site. The existing site investigations and public investigations across the wider area indicate it is likely the site is underlain by a relatively thick and continuous deposits of Mahurangi Limestone. However, across the wider Auckland/Northland region, rapid spatial lithological heterogeneity within Allochthon does occur due to the chaotic emplacement of the geological unit. It will therefore be important that cuts into this unit are mapped and monitored by a Geologist during excavation/earthworks.

[23] Auckland Council (March 2018). *Geotechnical and Coastal Hazards Topic Report. Warkworth Structure Plan.*

Figure 7-6: Image of Redvale Limestone Quarry. Source: (Whites Aviation, 1966)

7.5.2 Batter Grades for Temporary and Permanent Batters

Table 7-5 below provides recommended maximum temporary and permanent batter grades for cut and fill slopes of up to 3 m height/depth. These may be adopted for embankments, ponds, swales and minor grade changes on the basis that:

- The batters are not surcharged by structures, vehicles, plant or stockpiled materials within 2 m of the batter crests;
- Flat or very gently (< 5 deg) ground is present above and below the batters;
- Any concentrated surface water flows are diverted away from the batters; and
- Batters are observed/monitored by a Geotechnical Engineer during construction.

If batters are surcharged, formed below or above sloping ground (e.g. fill placed above the existing western boundary farm drain) or exceed the heights in Table 7-5 below, they should be specifically assessed and designed by a Geotechnical Engineer, as outlined in Section 7.5.3 below

Table 7-5: Recommended Maximum Minor Batter Grades

Batter Type	Batter Height Range	Maximum Recommended Temporary Grade [1]	Maximum Recommended Permanent Grade [1]
Cut Batter in Firm to Very Stiff Tauranga Group Soils and/or weathered Mahurangi Limestone	0 – 2 m	1H:1.5H	1V:2.5H
	2 – 3 m	1V:2H	1V:3H
Engineered Fill Batter	0 – 2 m	1V:1.25H	1V:2H
	2 – 3 m	1V:1.5H	1V:2.5H

[1] : Assumes no surcharging within 2 m of batter crest and ground above and below the batter is flat.

7.5.3 Preliminary Stability Assessment of Permanent Cut/Fill Batters

A preliminary stability assessment has been undertaken for the proposed development earthworks where batters fall outside the constraints outlined in Section 7.5.2.

The stability analyses have been undertaken using Slide2 limit equilibrium software using the Morgenstern-Price Method of Slices. The analyses have adopted the lower bound Mohr-Coulomb material parameters from Table 7-1. The parameters for residual to completely weathered Mahurangi Limestone soils has also been adopted for the inferred underlying weathered Limestone Rock materials.

The preliminary analyses have been used to assess the stability of the slopes for the following load cases and factors of safety, as specified for roads, buried services and building platforms in the Auckland Council Code of Practice[24]:

- Static with Normal Groundwater level: FoS ≥ 1.5
- Static with Worst Credible Groundwater level: FoS ≥ 1.3
- ULS Seismic Loading: FoS ≥ 1.0 (or acceptable displacements)

The site has been delineated into analysis zones and where applicable a critical slope cross section has been analysed. The analysis zones and sections are shown on Figure 7-7 below. Results of the preliminary analyses and recommendations for any further analysis or detailing is summarised below.

Where retaining walls are proposed, they will also need to be designed with specific assessment/consideration for global stability, as discussed in Section 7.6.

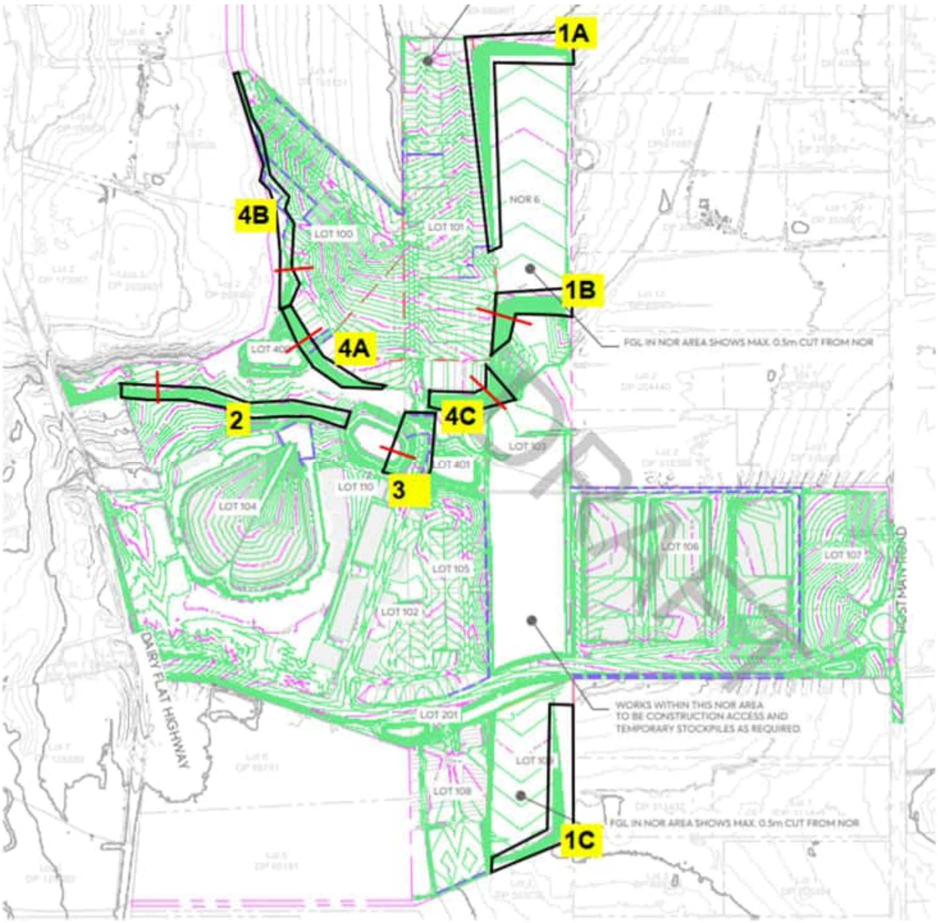

Figure 7-7: Preliminary Stability Analysis Zones (black box) and critical section locations (red lines)

Zones 1A, 1B, 1C:

Zones 1A, 1B and 1C are proposed to comprise cut batters up to 6.5 m in height and between 1V:3H and 1V:4H in grade. Existing investigations indicate cuts within Zones 1A and 1C should extend through Highly to Moderately Weathered Mahurangi Limestone rock below cut depths of 1.5 m and 2 m respectively. The depth to rock in Zone 1B is expected to be greater, with the overlying alluvium thicknesses expected between 1 and 4 m below existing ground.

[24] Auckland Council (May 2023). *The Auckland Code of Practice for Land Development and Subdivision. Chapter 2: Earthworks and Geotechnical.* Version 2.0.

Slope stability analysis has been undertaken for the maximum height (5.5 m), 1V:3H grade cut within Zone 1B, representing the critical case. A 12 kPa surcharge on the slope crest has been considered, accounting for traffic loading and/or 2-storey townhouses in the proposed North-Eastern Neighbourhood Precinct. Analysis outputs are presented in Appendix C and results are summarised below:

- Static Normal GW: FoS = 2.0
- Static Elevated GW: FoS = 1.5
- ULS Seismic: FoS = 1.2

On this basis, the proposed cuts within Zones 1A, 1B, 1C achieve the required factors of safety and should be suitable, provided the following recommendations are adhered to:

- Excavations should be regularly observed (i.e. every 1.5 m depth of cut) by a Geologist or Geotechnical Engineer during construction to map the exposed soils/rock. In the unlikely case that allochthonous Meleange (i.e. highly sheared materials of a mixture of lithologies) is encountered, stabilisation measures (e.g. drainage, shear keys) may be required;
- The design of any building foundations set-back less than 2.5 m from the slope crest should be specifically reviewed by a geotechnical engineer;
- Any concentrated surface water flows are diverted away from the batters. It is also recommended cuts within the limestone are capped with a minimum of 300 mm of clay as the limestone surface can be prone to “rilling” (small-scale erosion in narrow channels, which can be exacerbated when water is slightly acidic).

Zone 2:

A 1V:3H cut batter, up to 3.5 m in height is proposed in Zone 2 to provide increased flood storage capacity. The cut will sit within topsoil/fill and Tauranga Group Alluvium.

Slope stability analysis has been used to assess the stability of the critical cut with a 7 kPa surcharge on the slope crest modelled to represent single storey villas on-grade. It is understood that the northern-most villas will be founded within/below the slope, assumed to be on piles. Stability analysis outputs are presented in Appendix C results and are summarised below:

- Static Normal GW: FoS = 1.6
- Static Elevated GW: FoS = 1.4
- ULS Seismic: FoS = 1.0

On this basis, the proposed 1V:3H cut batter achieves the required factor of safety and should be suitable, provided the following recommendations are adhered to:

- No villas should not be constructed less than 2.5 m set-back from the slope crest without specific assessment;
- Any structural piles sitting within the slope should be specifically assessed and may need to consider lateral loading from shallow creep;
- Any concentrated surface water flows should be diverted away from the slope crest/face;
- The slope face and toe should be appropriately planted/protected to mitigate scour from surface water flows in the flood plain.

Zone 3:

Zone 3 comprises the northern and southern abutments for the proposed vehicle bridge. As shown on the earthworks drawings, all abutments are to be retained except for the western edge of the southern abutment which batters down towards a stormwater pond. A stability assessment has been undertaken for this proposed western batter. A 12 kPa vehicle surcharge was modelled at the batter crest. Stability outputs are presented in Appendix C and results are summarised below:

- Static Normal GW: FoS = 2.3
- Static Elevated GW: FoS = 1.9
- ULS Seismic: FoS = 1.1

On this basis, the proposed western batter achieves the required factor of safety and should be suitable, provided that:

- The fill embankment is constructed using engineered fill; and
- Any concentrated surface water flows are diverted away from the slope crest/face.

The detailed design of retention structures surrounding the remainder of the abutments shall be designed to mitigate global instability.

Zone 4A:

Zone 4A comprises 1V:3H fill embankments, up to 3 m in height, supporting residential lots in the North-Western Accommodation Precinct. Downslope of the fill embankments, the ground profile is gently sloping (less than 1V:5H) for a minimum horizontal distance of 15 m above the existing east-west creek channel, except for the location of the proposed wetland, which is to be excavated directly below the toe of the fill batter.

A preliminary stability analysis has been undertaken through the fill embankment and wetland pond, representing the critical design geometry. A 10 kPa dwelling surcharge load on the slope crest was modelled. Stability outputs are presented in Appendix C and results are summarised below:

- Static Normal GW: FoS = 1.6
- Static Elevated GW: FoS = 1.4
- ULS Seismic: FoS = 1.0

On this basis, the proposed 1V:3H fill embankments and adjacent wetland pond typography achieves the required factor of safety and should be suitable, provided that:

- The fill embankments are constructed using engineered fill (tested and certified);
- Any existing uncontrolled fill materials are excavated and replaced with engineered fill, unless reviewed by a geotechnical engineer and approved as suitable;
- Any concentrated surface water flows should be diverted away from the slope crest/face; and;
- The design of any building foundations set-back less than 2.5 m from the slope crest should be specifically reviewed by a geotechnical engineer.

Zone 4B:

Zone 4B comprises filling, typically 2 – 4 m in height supporting residential lots and roads within the North-Western Accommodation Precinct. Downslope of the fill, the ground slopes down toward the existing creek/farm drain, typically between 1V:4H to 1V:3H but locally up to 1V:2H near the northern end of the precinct. As noted in Section 7.5.1, existing shallow slumping has been observed above this farm drain in the steeper areas near the northern end of the precinct. The existing investigations indicate ground conditions in this zone typically comprise low-strength alluvium underlain by weathered Mahurangi Limestone, with rock inferred to typically be 1.5 – 2 m below existing ground levels.

At the northern end of Zone 4B, fill will be supported by retaining walls. Specific design of the retaining walls will be required, with consideration for potential downslope instability, noting that the retaining walls are positioned upslope of the existing slumping above the farm drain. Retaining walls may comprise either embedded piled walls socketed into the rock or Mechanically Stabilised earth walls which are benched into the slope and sitting on weathered rock materials, subject to specific design.

At the southern end of Zone 4B the fill is proposed to be battered at grades between 1V:2H and 1V:1H. Preliminary slope stability analysis of a critical section comprising a 3.5 m high 1V:1H fill embankment supporting the proposed road has been modelled. The stability assessment has projected a 1V:4H grade from the farm drain up to the toe of the proposed works, to consider the shallow slumping observed for over-steepened areas above the drain.

The preliminary analysis indicates that specific stabilisation measures will be required here and are likely to include:

1. Measures to mitigate instability through the underlying low-strength alluvium (within which existing downslope slumping is present). This is likely to comprise:
    a. Underfill drainage; and
        i. Over-excavation of alluvium at the fill toe to key into the Mahurangi Limestone rock. This may need to include installation of shear keys into the rock; or
        ii. Installation of palisade piles below the fill embankment, socketed into the Mahurangi Limestone rock.

2. In addition to the above stabilisation measures to address instability through the alluvium, any fill batters battered steeper than 1V:2H will likely need to be constructed as a specially designed, mechanically stabilised earth slopes with layers of geogrid.

A preliminary stability assessment output is presented in Appendix C. The preliminary assessment modelled 4 m long geogrids within the fill and palisade piles with an unfactored 50 kN/m shear resistance. Results are summarised below:

- Static Normal GW: FoS = 1.6 *(with specifically designed stabilisation measures)*
- Static Elevated GW: FoS = 1.5 *(with specifically designed stabilisation measures)*
- ULS Seismic: FoS = 1.0 *(with specifically designed stabilisation measures)*

This indicates the proposed earthworks will be feasible, however, specific detailed design of stabilisation measures will be required.

Zone 4C:

Zone 4C comprises embankments to support the 6 No. Neighbourhood Precinct North-West lots located to the east of the vehicle bridge. The proposed embankments are 1V:3H in grade and up to approximately 6 m in height. Downslope of the fill embankments, the proposed ground is near flat, except for the proposed stream diversion channel which is less than 1 m deep. A stability assessment has been undertaken for the critical section which has also considered the diversion channel directly below the fill embankment. A 10 kPa surcharge has been modelled setback 1 m from the batter crest to represent the proposed lightweight 1-2 storey dwellings. Stability outputs are presented in Appendix C and results are summarised below:

- Static Normal GW: FoS = 1.5
- Static Elevated GW: FoS = 1.3
- ULS Seismic: FoS = 1.0

On this basis, the proposed 1V:3H embankments achieve the required factor of safety, however, marginally. The proposed geometry is considered suitable, provided that:

- The fill embankments are constructed using engineered fill (certified by a geotechnical engineer);
- Underfill drainage is detailed below the fill embankments, unless otherwise instructed by the geotechnical engineer;
- Any concentrated surface water flows are diverted away from the slope crest/face;
- Scour protection measures are appropriately detailed in the proposed stream channel diversion to mitigate erosion in the channel and slope; and
- The design of any building foundations set-back less than 2.5 m from the slope crest should be specifically reviewed by a geotechnical engineer.

It is also recommended that where the stream channel diversion is positioned within 5 m laterally from the embankment toe, the embankment toe is over-excavated to the channel invert (or lower) and built back up using engineered fill. This is to provide toe-buttressing to the slope.

Additional undercuts may also be warranted if low strength soils are encountered below the embankment footprint.

7.5.4 PC120 Landslide Susceptibility Review

Although this report outlines a detailed assessment of land stability and documents the required stabilisation works or mitigation measures required to demonstrate suitable stability for future development, a separate desk study assessment has been completed to satisfy the regulatory requirements of Plan Change 120. As part of this assessment, the Auckland Council GIS Landslide Susceptibility Maps have been reviewed. The full extent of the site is mapped as having “very low” susceptibility to deep landslides and land within 150 m of the site boundary is mapped as either “very low” or “low” susceptibility to deep landslides.

The site and area within 150 m of the site boundary also is also mapped as having either “very low” or “low” susceptibility to shallow landslides (predominantly very low) except in a small number of localised “zones” mapped as “moderate” to “very high” susceptibility to shallow landslides.

Accordingly, a landslide risk assessment was completed in accordance with Appendix 24 of PC120, with specific review for each “zones” mapped as “moderate” to “very high” susceptibility to shallow landslides. The landslide risk assessment is presented in Appendix G.

With reference to the PC120 desk study assessment presented in Appendix G, the landslide risk is assessed as low (acceptable) provided that the advice presented in this report is adhered to.

7.6 Retaining Wall Considerations

As shown on Figure 7-8 below, retaining walls are proposed to support grade changes across the site. Table 7-6 presents an appraisal of suitable wall types. Preliminary design considerations are outlined in the subsections below.

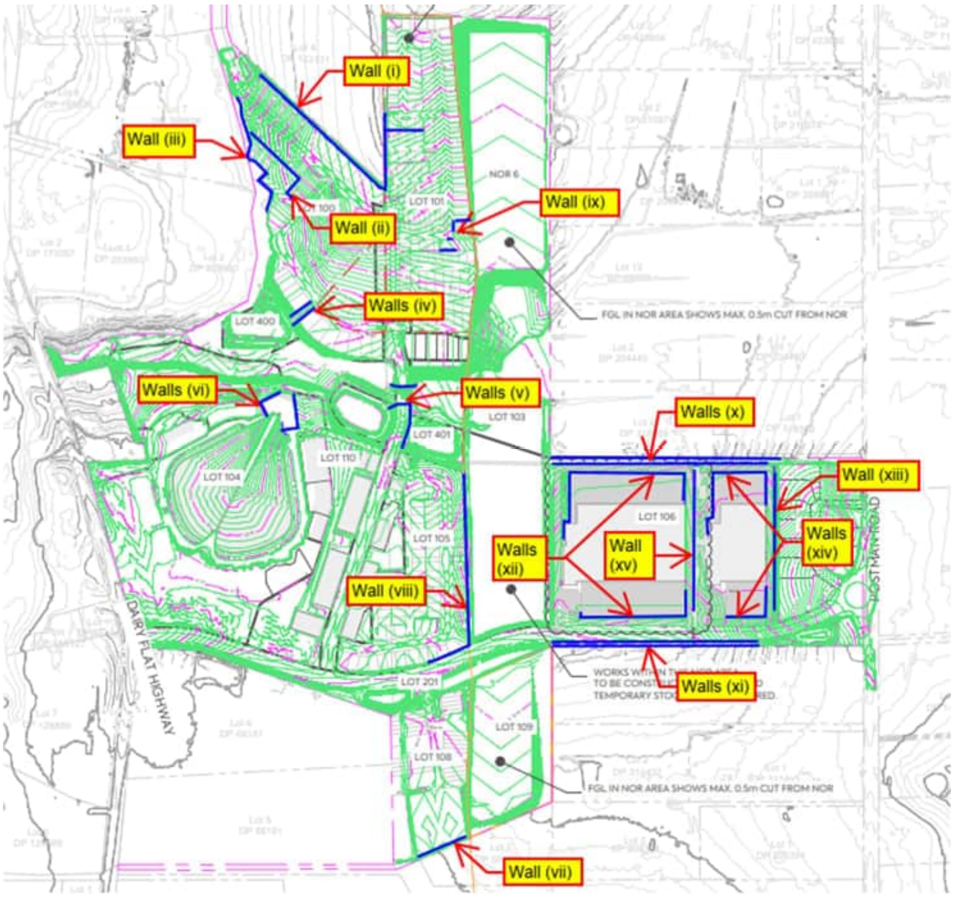

Figure 7-8: Proposed retaining wall locations

Table 7-6: Preliminary Appraisal of Retaining Wall Options

Wall Reference	Retained Height	Wall Supporting	Potential Wall Types and Key Considerations
Wall (i)	0.5 – 1.5 m	Cut	- Embedded timber pole wall (poles expected to socket into Limestone rock); or - Gravity block wall (e.g. Keystone)
Wall (ii)	≤ 1.5 m	Fill	- Embedded timber pole wall; or - Gravity block wall (e.g. Keystone)
Wall (iii)	≤ 3.5 m	Fill	- Embedded pile wall (e.g. steel UC sections) socketed into Limestone rock; or - Mechanically stabilised earth wall, likely founding on Limestone rock. *Specific geotechnical design will be required for global stability towards the farm drain, as outlined in Section 7.5.3.*
Wall (iv)	≤ 2.8 m	Fill	- Mechanically stabilised earth wall; or - Embedded pile wall (e.g. steel UC sections and/or timber poles)

Wall Reference	Retained Height	Wall Supporting	Potential Wall Types and Key Considerations
Wall (v) (Bridge abutments)	≤ 4.4 m	Fill overlying cut	- Mechanically stabilised earth wall with sufficient embedment/ground improvement to mitigate global instability and provide a bearing crust; or - Embedded piled wall (likely reinforced concrete piles, socketed into Limestone Rock) *Retaining wall design will need to consider settlement effects from fill loading and may need to consider allowance for some shallow slumping below the wall in the stream channel.*
Wall (vi)	≤ 4 m	Cut	- Embedded pile wall (likely reinforced concrete piles or steel UC sections). Lateral support (either dead-mans, anchors or propping) may be required.
Wall (vii)	≤ 2 m	Fill	- Embedded timber pole wall (poles expected to socket into Limestone rock); or - Gravity block wall (e.g. Keystone) *Stormwater channel is proposed below the wall so appropriate scour protection will be required.*
Wall (viii)	≤ 2 m	Cut and fill	- Embedded timber pole wall; or - Gravity block wall (e.g. Keystone) on a ground improved crust *Stormwater channel is proposed below the wall so appropriate scour protection will be required.*
Wall (ix)	≤ 0.7 m	Fill	- Embedded timber pole wall; or - Gravity block wall (e.g. Keystone)
Wall (x)	≤ 1 m	Cut and fill	- Embedded timber pole wall; or - Gravity block wall (e.g. Keystone) on a ground improved crust *Stormwater channel is proposed below the wall so appropriate scour protection will be required.*
Wall (xi)	≤ 1.9 m	Cut and fill	- Embedded timber pole wall; or - Gravity wall (e.g. Magnumstone) on a ground improved crust *Stormwater channel is proposed below the wall so appropriate scour protection will be required.* *Design will need to consider settlement effects from fill loading.*
Wall (xii)	≤ 1.9 m	Fill	- Piled wall (e.g. steel UCs and/or timber poles); or - Gravity wall (e.g. Magnumstone) on a ground improved crust *Design will need to consider settlement effects from fill loading and consider global stability.*
Wall (xiii)	≤ 1.2 m	Cut and fill	- Piled wall (e.g. steel UCs and/or timber poles) likely required with consideration of global stability of upslope batter
Walls (xiv)	≤ 2.4 m	Cut and fill	- Piled wall (e.g. steel UCs and/or timber poles); or - Gravity wall (e.g. Magnumstone) on a ground improved crust and specific consideration for global stability. *Design will need to consider settlement effects from fill loading and consider global stability.*
Wall (xv)	≤ 4 m	Cut and fill	- Piled wall (likely steel UC and/or RC concrete piles, socketed into Mahurangi Limestone rock); or - MSE wall with specific ground improvement/design to provide appropriate bearing and global stability. In-ground piles may be required beneath the wall as part of the global stability design. *Design will need to consider settlement effects from fill loading and consider global stability*

7.6.1 Preliminary Retaining Wall Design Considerations

Preliminary design of retaining structures can be completed using the geotechnical parameters in Table 6-18, however, these should be specifically reviewed by a geotechnical engineer for detailed design. Supplementary geotechnical investigations are expected to be required to support the design of Wall Types (iii), (iv), (v), (vi) and possibly some retaining walls in the Data Centre Precinct. Detailed geotechnical design will also be required for the retaining walls, particularly where sloping ground is present above or below the walls.

All walls should be detailed with drainage. For piled or block walls this should generally comprise a minimum 300 mm wide layer of granular drainage aggregate (e.g. 20/7) with a perforated Novacoil subsoil drain at the base. An impermeable "cap" should be detailed above the aggregate to mitigate surface water inflows. Wall designs should also consider elevated groundwater cases.

Gravity retaining walls founding on alluvial materials will likely require over-excavation to form a fill bearing crust, similar to building foundations, as outlined in Section 7.9.1.2. Global stability analyses will generally also be required for ground bearing retaining walls, particularly for larger walls such as Wall (iii), Wall (v), Wall (vi) and the Data Centre Precinct retaining walls. Specific ground improvement may be required to achieve acceptable global stability which may involve over-excavation and replacement with engineered fill (e.g. shear key), in-situ stabilisation beneath the gravity wall or in-ground palisade piles sitting beneath the gravity wall.

Retaining walls supporting bulk fill should consider filling induced settlements, as outlined in Section 7.3.8. Flexible wall systems may be required to tolerate settlements in some locations.

Any buildings positioned above retaining walls should be detailed so that the building foundations do not sit above a 45° zone of influence line, drawn from the base of the retaining wall. This may require local deepening of foundations.

Where open stormwater drains are proposed below retaining walls, the drains should be appropriately lined/protected to mitigate scour unless the retaining walls are specifically designed to accommodate scour downslope of the wall.

Wall types (i), (viii), (x) and (xi) will support cuts near the site boundary, up to 1.5 m deep. No existing structures or services are present within 20 m upslope of these walls (based on a Before u Dig and Auckland Council Geomaps review). Therefore, mechanical settlements due to retaining wall deflections will not have adverse effects on neighbouring properties.

7.6.2 Retaining Wall Construction Considerations

To enable the construction of "bottom up" retaining walls in cut areas, temporary batters/benching will be required, as outlined in Section 7.5.2. If insufficient space is available to form safe batters (e.g. boundary retaining walls), the retaining walls may need to comprise soldier piled walls installed top-down (i.e. piles are drilled and installed before excavating in-front of the piles).

Similarly, for retaining walls supporting fill, the construction sequencing will need to be agreed with Initia such that local and global stability is maintained during the construction process. No unsupported cut/fill batters should be formed at retaining wall locations without the approval from Initia.

If piled walls are adopted for Walls (iv), (v) and (vi), (viii), (xi), (xii), (xiv) and (xv) the piles will extend through deep alluvium and likely below groundwater level. Therefore, there will likely be a high risk of pile hole collapse for unsupported bored pile holes. Casing or support fluids could be allowed for or alternative pile types (e.g. driven piles) could be considered. However, driven piles may not penetrate the limestone rock, where it is present. Piles for the remaining retaining walls may extend through shallow alluvium (typically less than 2 m deep) or likely require relatively shallow pile embedments due to the limited retained heights. If holes are drilled hit and miss and poured the same day, hole collapse is expected to likely be avoided, however, provision for temporary stub casing should be included if soft ground is encountered.

It is expected that most of the retaining wall piles extend into Highly to Moderately Weathered Mahurangi Limestone rock. Experience on other projects indicates that generally the highly fractured Limestone can be drilled into using standard auger tooling, albeit at slow rates. If blocky areas are encountered, specialised tooling may be required.

7.7 Geotechnical Considerations for Civil Infrastructure

It is understood that civil infrastructure at the development is expected to include the following:

1. Stormwater Infrastructure:
    a. Construction of a series of wetlands and stormwater retention ponds;
    b. Construction of swale drains to collect and convey surface water;
    c. Installation of buried stormwater pipes (of unknown diameter and depth) to service buildings.
    d. Installation of stormwater detention tanks.

Stormwater infrastructure will also include re-alignment of the existing creek and installation of a subsoil drainage system for the lagoon. Both these items were consented under the Stage 1 Fast-Track consent and no further comment is provided in this report.

2. Wastewater Infrastructure:
    a. Construction of an on-site treatment plant and pump station at the southern end of the Live Work Precinct;
    b. Construction of a buried sewer pipe network (of unknown diameter and depth) to service buildings.
    c. Onsite wastewater disposal fields.

3. Water Supply and Utilities:
    a. Installation of a water supply network including water bore supplies and heat transfer pipes connecting to the proposed data centre;
    b. Installation of power and telecommunication utilities.

General key geotechnical considerations for ponds, drains and buried services are outlined in the subsections below. Preliminary considerations for above ground structures is provided in Section 7.9.

Further geotechnical design input is expected to be required, particularly for any infrastructure requiring deep excavation.

7.7.1 Stormwater Retention Ponds and Swale Drains

Stormwater ponds and open drains should be formed in accordance with the maximum batter grade recommendations outlined in Section 7.5.

Drains and channels conveying flows should be appropriately detailed to mitigate against scour. It is noted that Limestone materials can be prone to “rilling” which comprises the small-scale surface erosion in narrow channels, which can be exacerbated when rainwater is slightly acidic. Subsequently, it is recommended that any channels that extend into Limestone materials are lined with a minimum 300 mm thick clay cap.

7.7.2 Buried Services

Geotechnical considerations relevant to the design and installation of services are expected to include:

- Settlements beneath filling and tanks;
- Subgrade strengths beneath manholes, tanks and and pipe bedding; and

- Temporary excavation stability/methodology and groundwater control.

These considerations are summarised below.

Settlements Beneath Filling:
As outlined in Section 7.3.8, consolidation settlement will occur where the site is being raised above existing levels. Unless buried services are specifically detailed to tolerate settlements in accordance with Table 7-3, works should be sequenced so that the recommended hold periods in Table 7-3 are observed between the completion of filling and installation of buried services.

Given the potential risks of some long-term settlements (e.g. creep) in some areas of the site, it is also recommended that infrastructure is designed to be resilient to some minor long term settlement. This may involve using flexible pipe systems and providing reasonable falls on gravity-fed systems.

Settlements may also occur beneath any above ground tanks. Settlements should be reviewed at detailed design in relation to the tolerances of tanks.

Subgrade strengths beneath manholes, pipe bedding and tanks:
Where trench subgrades are within Tauranga Group alluvium, they will have a low strength (i.e. CBR of 1 – 2 %). These soils may not be suitable for compacting bedding material against, and subgrade improvement layers are likely to be required beneath pipe bedding, manholes and tanks. Subgrade/bedding improvement options should be investigated via site trials; however, potential options could include:

- Placement of a coarse aggregate such as imported blue-brown rock or limestone rock;
- Cement stabilised Woodhill Black sand, which is self-compacting under the application of water. Black sand is commonly wrapped in geotextile to minimise the risk of internal erosion (piping); or
- Application of geotextiles and/or geogrids followed by lightly compacted hardfill.

Temporary Excavation Stability/Methodology and Groundwater Control:
All temporary excavations should be either appropriately battered/benched in accordance with advice presented in Section 7.5 or supported with appropriately detailed trench shields or temporary retention. Unsupported steep excavations have a high risk of collapse, particularly within the low strength Tauranga Group alluvium and where groundwater is present.

Cantilevered temporary retention systems (e.g. driven sheet piles or timber poles) are expected to generally be suitable for supporting temporary cuts in alluvium up to approximately 2.5 – 3 m deep. Deeper cuts will likely require propped systems (e.g. sheet piles with waler beams/props).

Trenchless methods could be considered for any required deep services within the alluvium, however, this would require specific review, as buried stumps have been noted within the alluvium. Trenchless methods are not expected to be practical within the Limestone.

Contractors should allow for some pumping of perched water flows for all trenches, particularly during winter months. If any excavations extend below the static groundwater table (generally inferred to sit between RL 51 – 52 m, grading down to approximately RL 49 m at the creek invert) specific groundwater cut-off systems (e.g. sheet-piling) may be required. This should be assessed by a geotechnical engineer at detailed design. Deep infrastructure should be installed prior to adjacent settlement sensitive structures (e.g. buildings) unless settlement effects and stability is specifically assessed.

Due to the construction challenges associated with deep excavations, buried infrastructure depths should be limited as much as possible.

7.8 Pavement and Subgrade Protection Considerations

Pavements will be required for roads, on-grade parking areas, and light industrial yards. Based on the cut/fill plan[25], subgrade conditions will generally comprise:

1. At least 0.5m of engineered fill below design subgrade level; or
1. Subgrade at or close to the natural Tauranga Group Alluvium soils; or
2. Subgrade on weathered Mahurangi Limestone soils or rock.

Subgrades Underlain by ≥0.5 m of Engineered Fill

Where at least 0.5 m of engineered fill exists beneath subgrade level, a CBR ≈ 4 % should be achievable. Higher CBRs may result if imported hardfill or lime/cement stabilisation is used. Laboratory testing indicates a 4 % total lime/cement dose can raise alluvial fill CBRs to ≥10 % (refer Table 6-16).

Subgrades on Natural Alluvium

Where the subgrade is at or close to the natural Tauranga Group Alluvium soils, subgrade CBRs will likely range between 1 – 2 % and it is recommended that subgrade improvement is allowed for. Subgrade improvement may comprise sub-excavation of approximately 0.5 m below subgrade level and backfilling with a circa 0.2 - 0.3 m thick rock fill working layer overlain with compacted bulk fill. Alternatively, in-situ lime/cement stabilisation could be considered in areas with inorganic alluvium, however, trafficability of stabilisation plant may limit its practicality. A 0.5 m subgrade improvement layer should increase the subgrade CBR to 4%.

Subgrades on Mahurangi Limestone

Pavements within the deeper cut areas of Neighbourhood Precincts North-East and North-West should be founded on Mahurangi Limestone soils or weathered rock. Residual soils should generally achieve a subgrade CBR of ≥ 3 % and highly to moderately weathered rock should generally achieve considerably higher CBRs. However, it is noted that when the weathered rock is exposed to weather and trafficking, it can rapidly weather and reduce in strength. Therefore, a maximum subgrade CBR of 5 % is recommended. Higher design CBRs could be adopted, where there is significant cost benefit, subject to site trials, and testing.

Subgrade Protection and Drainage

All existing materials at the site (including the Mahurangi Limestone) are sensitive and can lose strength, particularly when trafficked under wet conditions. It is recommended that subgrades are protected with a sacrificial layer if left exposed to wet weather. Subgrades should also be graded to prevent ponding of surface water. It is important that permanent subsoil drainage is detailed beneath the subgrade of all pavements.

A summary of the preliminary subgrade CBR recommendations is summarised in Table 7-7 below:

[25] McKenzie & Co (8/12/2025). *Earthworks Cut Fill Overall Plan*. Dwg No. 3325-2-2100 Rev B.

Table 7-7: Summary of preliminary subgrade CBR recommendations

Subgrade Conditions	Recommended Preliminary Subgrade CBR
>0.5m thick engineered bulk fill (below subgrade level)	4 %
Tauranga Group Alluvium (typically firm to stiff clays/silts)	1 – 2 % [Note – subgrade improvement is recommended to improve subgrade CBR. Either undercut and replace or in-situ lime/cement stabilisation (subject to site trafficability constraints)]. A 500 mm subgrade improvement layer of Tauranga Group Alluvium should increase the subgrade CBR to 4%
Mahurangi Limestone residual to completely weathered soils	3%
Mahurangi Limestone highly to moderately weathered rock	5% (possibly higher, subject to validation by site trials and testing)

7.9 Considerations and Ground Improvement for Building Foundations and Floor Slabs

Foundation options across the site will vary depending on building typology and post-earthworks ground conditions. The key constraint governing suitable foundation systems is the strength and compressibility of the Tauranga Group alluvium, which generally has insufficient bearing capacity to directly support shallow foundations. Settlements beneath both distributed (e.g. waffle slabs) and concentrated footing loads may also exceed structural tolerances for some structures, particularly where deep alluvium is present.

Where alluvium is present below the proposed building platforms at the completion of earthworks, foundation systems will generally need to adopt one of the following strategies to provide sufficient foundation capacity and mitigate settlements:

1. Piled foundation systems: which transfer foundation loads to the weathered Mahurangi Limestone materials. Floor slabs may either be fully suspended on piles or may be ground bearing for some structures, on a ground improved platform; or
2. Shallow foundation systems on a ground improved platform: which will involve forming an engineered fill crust of adequate thickness below building foundations to provide sufficient bearing capacity/subgrade reaction. Preloading will also likely be required for some buildings, depending on the loading conditions, the structure's tolerance to settlement and the ground conditions.

Whilst either strategy will likely be feasible for the majority of the light-weight (1 – 3 storey) buildings, structures that are highly settlement sensitive or have high concentrated loads will likely require piling. These structures include, but may not be limited to, the Hotel Building and the Data Centres.

For buildings founded in areas where bulk cuts will extend below the alluvium (i.e. the majority of Neighbourhood Precinct North-East, the north-eastern end of Neighbourhood Precinct North-West and localised areas of the Stage 5a Vacant Lot Subdivision) shallow foundation systems, bearing directly on the weathered Mahurangi Limestone soils/rock will be suitable.

It is recommended that ground improvement works for shallow foundations are integrated into the bulk earthworks phase to maximise efficiency and minimise rework. Further geotechnical assessment should be undertaken ahead of bulk earthworks to confirm specific ground improvement requirements and allow them to be integrated into the earthworks operations.

Preliminary options, design considerations, and advice for piled and shallow foundation systems (including ground improvements) are outlined in the following subsections, and a summary of options by development area is presented in Table 7-13.

7.9.1 Shallow Foundation Systems and Ground Improvement

The key geotechnical considerations for shallow foundation systems include:

- Soil expansivity
- Bearing capacity –i.e. providing a sufficiently thick bearing "crust" below foundations; and
- Settlements below footings and distributed loads (e.g. rafts).

Where Tauranga Group alluvium remains below building platforms, an engineered fill crust will generally be required to provide bearing capacity, and preloading may also be necessary to mitigate long-term settlement.

Shallow foundation systems may comprise strip or pad footings, or raft foundations (e.g., waffle slabs for dwellings, beam grillages for larger buildings). Where structurally feasible, raft systems are preferred in alluvial areas because they:

- Can be embedded shallower, reducing the required engineered fill crust thickness;
- Distribute loads more evenly, which can reduce or eliminate preload requirements; and
- Offer greater resilience to differential settlement.

The following subsections provide preliminary guidance on foundation detailing and ground improvement measures required to address soil expansivity, bearing capacity, and settlement. Further geotechnical assessment will be required to detail ground improvements and foundations.

7.9.1.1 Expansive Soils

Expansive soils undergo appreciable volume change with moisture variation, causing seasonal ground movements that can lead to foundation movement. Moisture variation may also be influenced by mature trees or vegetation with high water demand located near building platforms.

Laboratory testing on a range of near surface samples has been undertaken to assess soil expansivity in accordance with the following standards:

- NZS3604[26] to define whether soils are classified as "expansive". The presence of expansive near surface soils precludes a site from the definition of "good ground"; and
- NHBC guidelines[27] to define the expansivity class. It is noted that NZS3604 and the New Zealand Building Code B1/VM4 notes expansivity class should be defined based on the Shrink-Swell Index Laboratory Test (AS1298 Method 7.1.1). However, recent research[28] has demonstrated the Shrink-Swell Index Laboratory Test is unreliable in New Zealand and has recommended that the NHBC guidelines are utilised in lieu to inform site classification.

The laboratory results and assessments are summarised in Table 7-8. Testing indicates both the Tauranga Group alluvium and residual Mahurangi Limestone soils are "expansive" under NZS3604 and fall within the High Volume Change Potential category under NHBC.

For preliminary design, it is recommended shallow foundations are designed for Expansivity Class H (Highly Expansive) in both areas of cut and fill (assuming site-won soils and lime-rock blends will be used for fill materials). For building design, it is recommended further post-earthworks testing is undertaken to confirm site class, particularly in areas with imported fill materials.

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[26] Standards New Zealand (2011). *New Zealand Standard. Timber-framed buildings.* Ref: NZS 3604:2011.
[27] NHBC, (Land quality – managing ground conditions Chapter 4.1, dated January 2019.
[28] Rogers et al. (June 2020). *The Shrink Swell Test: A Critical Analysis.* NZ Geomechanics News.

Foundations designed for Expansivity Class H can comprise either a specifically detailed raft (e.g. waffle slab detailed in accordance with B1/AS1 or AS 2870:2011, or a specifically designed grillage of interconnected ground beams) <u>or</u> strip/pad footings embedded a minimum of 900 mm.

As outlined above, where platforms are underlain by Tauranga Group Alluvium, it is recommended that raft systems are adopted as they will have a reduced foundation embedment compared with isolated strip/pad footings. This will reduce the required depth of the fill bearing crust, discussed below.

Table 7-8: Soil Expansivity Assessment Summary

Sample ID	Sample Location	Geological Unit	Sample Depth (m)	Linear Shrinkage (%)	Liquid Limit	Plasticity Index	NZS 3604 "Expansive" or "Not Expansive" Classification	NHBC Volume Change Potential
HA03	Surf Amenity Precinct	Tauranga Group	1 – 1.3	14	55	33	Not Expansive	Medium
HA04			1 – 1.4	17	81	45	Expansive	High
TP16			1.0 – 1.5	22	123	79	Expansive	High
TP24			1.0 – 1.5	18	104	70	Expansive	High
MBH02	Lagoon	Tauranga Group	1 – 1.5	16	67	45	Expansive	High
MBH05			1.3 – 1.5	16	93	60	Expansive	High
TP19			0.5 – 1.0	17	65	43	Expansive	High
TP21			0.5 – 1.0	18	106	74	Expansive	High
TP26	Neighbourhood Precinct South	Tauranga Group	1.0 – 1.5	12	43	24	Not Expansive	Medium
TP107			1.0 – 1.5	18	99	64	Expansive	High
TP108			0.5 – 1	22	148	95	Expansive	High
TP27	Village Centre Precinct	Tauranga Group	1.0 – 1.5	14	73	45	Not Expansive	High
TP101	Accommodation Precinct	Tauranga Group	0.5 – 1.0	19	78	53	Expansive	High
HA140			0.3 – 0.5	12	55	22	Not Expansive	Medium
Aurecon_BH01	Data Centres	Tauranga Group	1 – 1.5	31	170	71	Expansive	High
Aurecon_BH02			0.5 – 0.7	9	53	26	Not Expansive	Medium
Aurecon_BH06			0.5 – 1.0	21	68	43	Expansive	High
HA122	Live Work Precinct	Tauranga Group	0.5 – 0.8	23	104	87	Expansive	High
HA128			0.5 – 0.8	14	62	40	Not Expansive	High
HA133	Solar Farm South	Residual Limestone	1.2 – 1.4	16	66	45	Expansive	High
HA102	Neighbourhood Precinct North-West	Residual Limestone	1.2 – 1.4	17	74	55	Expansive	High
HA116			1.1 – 1.4	22	88	68	Expansive	High
HA104		Tauranga Group	0.5 – 0.8	28	146	117	Expansive	High
HA114			0.4 – 0.7	24	144	116	Expansive	High
HA121			0.3 – 0.6	25	131	112	Expansive	High

7.9.1.2 Preliminary Bearing Crust Details for Foundations and Floor Slabs

The Tauranga Group alluvial soils are variable in strength and composition and are generally unsuitable for directly supporting shallow foundation systems. To provide a suitable bearing surface, a sufficiently thick engineered fill crust will need to be formed below the foundation bearing level. Based on the earthworks cut/fill plan[29], bulk fill is proposed beneath the majority of buildings where alluvium will remain. However, the nominal bulk fill thickness (placed following a topsoil strip) may form an insufficiently thick fill crust across some areas of the site.

Accordingly, some areas may require over-excavation to a pre-fill surface elevation <u>below</u> the underside of topsoil (i.e. sub-excavation into the alluvium), so that an adequately thick engineered fill crust can be formed to support shallow foundations. It will generally be more efficient to form this crust during bulk earthworks, rather than undertaking targeted ground improvements during building foundation

[29] McKenzie & Co (8/12/2025). Earthworks Cut Fill Overall Plan. Dwg No. 3325-2-2100 Rev B.

preparation. This is particularly relevant for areas already receiving nominal bulk fill to avoid later rework removing the already placed fill.

Figure 7-9 below presents a typical schematic of the fill crust build up. Table 7-9 summarises preliminary thicknesses of each component within the crust. These thicknesses may be used for planning purposes, however, further specific assessment should be undertaken to confirm appropriate pre-fill surface elevations prior to placing bulk fill beneath building platforms. Fill crusts will need to extend laterally approximately 1 m beyond the perimeter of foundations.

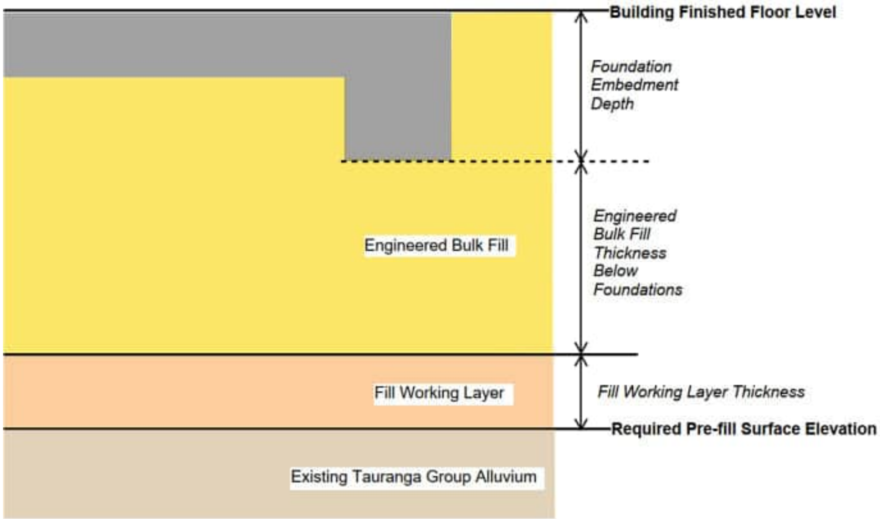


Figure 7-9: Schematic of Shallow Ground Improvement Beneath Shallow Foundations

Table 7-9: Preliminary thicknesses for shallow foundation bearing crust build up

Build-up Component	Foundation System/Building Type			Comments
	Waffle Slab Foundations (Typically 1 – 3 storey residential)	Stiffened Raft (Typically 1 – 4 storey amenity buildings and apartments)	Strip/Pad Footings	
Foundation Embedment Depth (m)	0.3 – 0.5	0.5 – 0.8	0.9 [Due to soil expansivity]	Typical depths presented. Actual depths will depend on structural design.
Engineered bulk fill thickness required beneath underside of foundations (m)	0.5 – 1.0	1.0 – 1.5	1.0 – 1.5	Will depend on foundation loadings and strength of underlying alluvium.
Fill working layer (m)	0.2 – 0.5	0.2 – 0.5	0.2 – 0.5	Will depend on strength of underlying alluvium and weather conditions during earthworks.

The thicknesses presented above are appropriate for firm to stiff Tauranga Group alluvium soils (typically Su = 35 – 60 kPa). If soft to firm soils are encountered (Su = 20 – 40 kPa), further over excavation may be required. Conversely, where stiff to very stiff soils are encountered at subgrade level during construction, the working layer and/or engineered fill crust requirement may be able to be reduced or eliminated at the direction of the geotechnical engineer.

Bearing and lateral capacities of foundation systems will be contingent on the specific ground conditions and ground improvements and should be derived at future design stages.

Ground Bearing Floor Slabs

Structures that are either piled or supported on shallow strip/pad footings are likely to have ground bearing floor slabs. For lightly loaded slabs (i.e. 5-10 kPa floor loading), the following preliminary subgrade CBRs are recommended:

- Slabs bearing on residual Mahurangi Limestone soils should generally achieve a subgrade CBR of 3% (assumes Su ≥ 100 kPa).
- Where alluvium is present, a typical build-up of a working layer, overlain by approximately 0.5 m thickness of engineered fill (assumes Su > 130 kPa) should provide a subgrade CBR of 4%.

7.9.1.3 Building Settlement and Preloading

Oedometer, CPT, and fill settlement monitoring indicate that the Tauranga Group alluvium is moderately compressible, and consolidation settlements will occur beneath both fills and building loads. Settlement magnitude will depend on soil composition, alluvium thickness, and applied loading.

Secondary creep settlements can also gradually occur over the full operational life of a structure. Large-magnitude secondary creep settlements typically occur in young, normally consolidated soils with high organic contents (e.g. Peat and highly organic clay). Specific laboratory testing has not been undertaken to evaluate creep settlements at present, however, it is expected that the slightly-over consolidated Pleistocene-Aged CLAYs and SILTs with low organic contents will have a low risk of creep settlement. Creep settlements may occur within the PEAT and Organic CLAYs (typically encountered within the Data Centre Precinct, Central Solar Farm and Residential Neighbourhood South areas).

A preliminary settlement review has been undertaken using the Oedometer results and CPT data (using the empirical Robertson & Cabal (2015) method). The assessment has considered typical distributed building loads.

The settlement estimates assume that filling induced settlements are complete prior to the commencement of building construction. This will require the observation of a hold period and fill settlement monitoring as detailed in Section 7.3.8.

The results of the preliminary settlement review are summarised in Table 7-10 below. Specific analyses will be required for each building platform/area once design loadings and building forms/layouts and levels are confirmed. Additional investigations may also be required to provide improved certainty in the ground model. It is also noted that review of accurate settlement monitoring data, from fill placement, may enable refinement of the estimated settlement magnitudes under building loads.

Table 7-10: Preliminary Building Settlement Appraisal

Development Precinct	Structure Typology	Assumed Foundation System for Settlement Estimate	Assumed long term distributed SLS load (kPa)	Preliminary Consolidation Settlement Estimates	Preliminary Creep Settlement Appraisal	Settlement ground improvement or piled foundations required?
Surf Amenity Precinct	Academy, Change, Rentals, Retail, Lounge and Administration Buildings (1 storey)	Shallow Raft	10 kPa	10 – 40 mm	Low Magnitude	Possibly
	Restaurant (2 storeys)	Shallow Raft	25 kPa	50 – 110 mm	Low Magnitude	Likely
	Hotel Building (8 storeys)	Fully piled	N/A	N/A	N/A	N/A

Development Precinct	Structure Typology	Assumed Foundation System for Settlement Estimate	Assumed long term distributed SLS load (kPa)	Preliminary Consolidation Settlement Estimates	Preliminary Creep Settlement Appraisal	Settlement ground improvement <u>or</u> piled foundations required?
Accommodation Precinct	Clubhouse (2 storeys)	Shallow Raft	25 kPa	30 – 60 mm	Low Magnitude	Possibly
	On-grade Villas (1 storey)	Waffle Slab	7 kPa	≤ 25 mm	Low Magnitude	Unlikely
Village Centre Precinct	Apartment Types A, B and C (4 storey)	Shallow Raft	40 kPa	40 – 200 mm	Low to Moderate Magnitude	Likely
	Apartment Types D and E (3 storey)	Shallow Raft	30 kPa	30 – 140 mm	Low to Moderate Magnitude	Likely
	Market, Food & Beverage, Wellness, Early Childhood (1 storey)	Shallow Raft	10 kPa	20 – 80 mm	Low to Moderate Magnitude	Possibly
Neighbourhood Precinct South	Standalone Dwellings (2 – 3 storey)	Waffle Slab	15 kPa	20 – 70 mm	Moderate Magnitude	Possibly to Likely
Neighbourhood Precinct North-East	Standalone Dwellings (1 to 2 storey)	Waffle Slab	10 kPa	≤ 25 mm	Very low Magnitude	Not required
Neighbourhood Precinct North-West	Standalone Dwellings (2 storey)	Waffle Slab	10 kPa	≤ 25 mm	Very low Magnitude	Not required
Live-Work Precinct	Work-Live Terraces (2 storey)	Waffle Slab	15 kPa	10 – 35 mm	Low Magnitude	Possibly in local areas
	Water tanks	Slab	Unknown – 40 kPa assumed [2]	≤ 30 mm	Very low Magnitude	Unlikely
Data Centres	Heavily Loaded Data Centre Structures	Assumed to be fully piled	N/A	N/A	N/A	N/A
Stage 5a Vacant Lot Subdivision	Not determined	Slab on Grade Assessed[1]	15 kPa	≤ 25 mm	Very low Magnitude	Not required

[1] Structural assumed to be founded on strip/pad footings. Footing settlements will be contingent on specific founding conditions and have not been assessed.
[2] Tank details not known. Preliminary assessment considered a 20 m diameter tank with 40 kPa applied load. This corresponds to tank height of approximately 4 m.

The preliminary assessment indicates settlements are likely to exceed typically allowable tolerances for a number of the buildings across the site. Where this is the case, either deep piled foundations (refer Section 7.9.2) <u>or</u> ground improvement will likely be required to mitigate settlements.

It is noted that the presented settlement assessment is preliminary only and based upon widely spaced CPT investigations. As presented in Section 7.3.8, fill settlement monitoring has demonstrated that actual settlements have varied from CPT predictions (typically less than predicted). Future testing, monitoring and analysis should therefore be undertaken to develop more accurate settlement predictions at future design stages and optimise ground improvement requirements. This may include:

- Survey monitoring of fill using settlement plates which are strategically placed below future building platforms. *(If the construction programme allows, the monitoring of bulk fill prior to placing preload in marginal areas may be able to verify building settlements would be acceptable without preloading/settlement ground improvement)*;

- Additional CPT and DMT (Flat Plate Dilatometer Testing); and
- Additional machine boreholes with 1D Consolidation laboratory testing.

Where ground improvement is required to address settlements, preloading is likely to be the most practical and cost-effective option. This involves constructing a temporary embankment across a proposed building platform and holding it for a designated hold period to allow settlements to occur prior to the construction of the buildings. Preloads can be formed using any available fill source of reasonable density which may include topsoil/unsuitables.

The specification of preload heights will be contingent on multiple factors, including the density of the preload fill material; however, indicative preload heights relative to building finished floor levels are presented in Table 7-11.

Table 7-11: Preliminary Indicative Preload Heights above Finished Floor Levels

Building Long Term SLS Distributed Load	Preliminary Indicative Preload Height above Finished Floor Level
10 kPa	1 m
15 kPa	1 – 1.5 m
25 kPa	1.5 – 2.5 m
40 kPa	2.5 – 3.5 m

If further investigation identifies a notable secondary creep settlement risk, additional “surcharge” fill may be necessary in some areas which typically can be up to an additional 50% embankment height above FFL, compared with the baseline preload height.

The hold period for preloads will vary across the site but is typically expected to range between 3 – 9 months, as presented in Table 7-3. The hold period commences once the preload is formed to full height. Where programme is constrained, hold periods can be accelerated via surcharging (i.e. increasing the preload embankment height) or via the installation of wick drains, though the latter is unlikely to be cost effective. Preloads shall be monitored via survey monitoring of settlement monitoring plates which are to be installed and baselined prior to the placement of any fill (including bulk fill).

To minimise double handling, it is preferable to place permanent bulk engineered prior to building up the temporary preload embankment. However, if required, preload embankments can be built up from existing ground levels (noting the design elevation is still contingent on building finished floor levels).

To limit total required preload fill volumes, it is recommended that works are programmed to allow preload fill to be “rolled over” between successive development areas.

Preload and “fill crust” design will also need to consider settlements beneath footings where distributed raft-type foundations cannot be adopted.

7.9.2 Preliminary Options and Design Considerations for deep Piled Foundations

Whilst shallow foundation systems will likely be cost-effective the most of the proposed light weight structures; for heavy or settlement sensitive structures (e.g. Hotel and Data Centres), deep piles founding in weathered Mahurangi Limestone materials will likely be required. Piled foundations will also likely be necessary for the northern-most Accommodation Precinct Villas, given they are proposed to be suspended above sloping ground. Piles may also be locally required for any buildings positioned within a 45^o zone of influence line behind retaining walls and for bridging over sewers.

Piled foundations may also be the most cost-effective solution for other specific buildings where significant “extra over” ground improvement would be required to accommodate shallow foundation systems. Examples of this may include:

- Moderately loaded structures in areas underlain by Tauranga Group Alluvium with negligible (or no) bulk earthworks fill proposed;

- Structures in areas where bulk filling has already commenced (under the Stage 1 consent) and the placed fill would provide an insufficiently thick fill crust (noting specific “sub-excavation” was not undertaken during existing earthworks); or
- Structures where the depth to Mahurangi Limestone is relatively shallow and Tauranga Group alluvium will be present at or near design subgrade level (e.g. Live Work Precinct).

Driven piles (either steel or timber) are likely to be the most suitable pile option for most piling applications. Screw piles or bored piles may be considered as an alternative for some applications. The subsections below provide preliminary considerations for optioneering purposes. Further analysis will be required at the design stages for the buildings.

7.9.2.1 Driven Piles

Timber or steel piles driven to effective refusal in weathered Mahurangi Limestone are likely to be the most cost-effective pile type for supporting high compressive axial loads, particularly in areas with deeper alluvium. However, the driven piles are unlikely to penetrate deep into the highly to moderately weathered limestone rock meaning uplift capacities will likely be limited. Should foundation uplift demands be high, relatively heavy steel sections may be required (to enable greater embedment). It is recommended trial piles with PDA testing are considered in this case. Alternatively, vertical ground anchors drilled into the Mahurangi Limestone could be considered for providing uplift resistance.

If driven piles are adopted, careful consideration will need to be given to hammer sizes and drop heights so that pile stresses at the pile toe do not cause damage to the pile when it reaches the Limestone surface. Pile installation should be monitored via PDA testing.

It may be possible for lightly loaded piles (e.g. for the Accommodation Villas) to be designed as friction piles, sitting in the alluvium. However, this will require further assessment.

Vibrations would also need to be reviewed if piles are driven near existing sensitive structures (e.g. surf lagoon) to check vibrations from pile driving do not cause adverse effects on the structures.

Preliminary geotechnical ultimate compressive capacities for a range of driven pile sizes, driven to refusal onto Limestone rock are presented in Table 7-12 below. These should be reviewed by the building designer and may be refined via instrumented trials. Geotechnical ultimate capacities should be reduced by a strength reduction factor of $\Phi_g = 0.5$ (without PDA testing) or $\Phi_g = 0.75$ if a minimum of 10% of production piles are PDA tested.

If piles are proposed in an area of bulk filling and installed prior to the completion of filling induced settlements, the piles should be designed to allow for negative skin friction loads within the fill and alluvium layers. Negative skin friction values should be derived at future design stages. As outlined in Section 7.2.5 it may also be applicable to apply negative skin friction loading in a post-ULS-seismic loading case, however, this is not expected to govern design.

Table 7-12: Preliminary Driven Pile Capacities in Axial Compression (Driven to refusal on Limestone)

Pile Type[1]	Preliminary Geotechnical Ultimate Capacity in Compression (kN)
250UC89	1,600
310UC118	2,100
310UC137	2,400
310UC158	2,800

350SED	600 - 1000 (Test piles recommended to confirm achievable capacities achievable without damaging pile during installation)

Note:
[1] Assumes 300MPa grade steel

7.9.2.2 Screw Piles

Screw piles can be installed with limited noise and vibration so could be considered as an alterative to driven piles if/where noise and vibration constraints present. Similar to driven piles, negligible embedment is likely to be achieved into the highly to moderately weathered Mahurangi Limestone rock, limiting achievable uplift capacities. Lateral capacities will also be limited compared with driven or bored piles.

Preliminary capacities for screw piles can be provided by a local specialist screw piling contractor.

7.9.2.3 Bored and CFA piles

Bored piles could be considered as an alternative to driven piles or screw piles which could provide increased vertical and lateral capacity with relatively low installation noise and vibration. However, bored piles may have a considerable cost premium, due to the following construction challenges:

- Risks of pile shaft collapse during drilling, particularly in the low-lying central area of the site where alluvium thicknesses are greater and groundwater sits within the alluvium. It is likely that casing or support fluids will be required for temporary shaft support; and
- Drilling rates in the Mahurangi Limestone rock are expected to be slow. Experience on other projects indicates that generally the highly fractured Limestone can be drilled into using standard auger tooling, albeit at slow rates. If blocky areas are encountered, specialised tooling may be required.

In areas of deep alluvium, Continuous flight auger (CFA) piles could be considered as this would eliminate the risk of pile hole collapse. However, significant penetration into the Limestone may not be achievable with the CFA piling rigs available in New Zealand. A specialist contractor would need to be consulted to assess early feasibility, should CFA piles be considered.

Parameters for bored pile design should be derived at future design stages. Negative skin friction from filling induced settlement and liquefaction induced settlement should also be considered.

7.9.3 Foundation Options Summary

Table 7-13 below presents a summary of the proposed building/structure typologies and a preliminary appraisal of foundation options and associated ground improvement based on the considerations outlined above. Further geotechnical investigation, analysis and reporting will be required to support building foundation and ground improvement design.

Table 7-13: Preliminary Foundation Options and Ground Improvement Appraisal

Development Precinct	Structure Type	Baseline Earthworks and Ground Conditions Summary	Preliminary Recommendations for Building Foundation and Ground Improvement Options
Surf Amenity Precinct	Academy, Change, Rentals, Retail, Lounge and Administration Buildings (1 storey) & Restaurant (2 storeys)	*Typical Ground conditions:* > 5 m alluvium overlying weathered Mahurangi Limestone *Baseline earthworks:* Bulk Fill > 0.5 – 2.5 m	Shallow raft foundations founded on an engineered fill crust. Sub-excavation may be required to form the crust for some buildings. Preloading likely required for 2 storey restaurant and possibly required for single storey buildings.
	Hotel Building (8 storey)	*Typical Ground conditions:* 8 – 12 m alluvium overlying weathered Mahurangi Limestone *Baseline earthworks:* Bulk fill > 1 m	Structure supported on piles extending to Mahurangi Limestone. Driven steel piles likely preferred. Ground floor slab founded on-grade on engineered fill crust.
Accommodation Precinct	Clubhouse (2 storey)	*Typical Ground conditions:* 10 – 12 m alluvium overlying weathered Mahurangi Limestone *Baseline earthworks:* Bulk fill approximately 1 – 1.5 m	Shallow raft foundations founded on an engineered fill crust. Minor sub-excavation may be required to provide sufficient fill crust thickness. Preloading possibly required.
	On-grade Villas (1 storey)	*Typical Ground conditions:* 8 – 12 m alluvium overlying weathered Mahurangi Limestone *Baseline earthworks:* Bulk cuts up to 1.5 m and bulk filling up to 1.5 m	Waffle Slab Foundations supported on a limited thickness of engineered fill crust. Recommend the crust is formed as part of bulk earthworks in bulk fill areas. In bulk cut areas, the fill crust could be formed during bulk earthworks of during the foundation preparation stage. Preloading unlikely to be required.
	Suspended Villas (1 storey)	*Typical Ground conditions:* 5 – 8 m alluvium overlying weathered Mahurangi Limestone. *Baseline earthworks:* Bulk cut 1 – 2 m. Finished ground surface sloping.	Given the sloping ground, piles likely to be required. Driven timber poles are likely to be suitable. Specific design consideration will be required for sloping ground and surface water flows in the flood storage area.
Village Centre Precinct	Apartment Types A, B, C, D & E (3 to 4 storey)	*Typical Ground Conditions:* 5 – 11 m alluvium (including organic soils) overlying weathered Mahurangi Limestone. *Baseline earthworks:* Bulk fill 0.5 to 2.5 m	Where the baseline bulk fill is greater, shallow raft foundations founded on a specified engineered fill crust may be most economical. Preloading will likely be required. Driven piles (e.g. timber poles) may be preferred where the baseline fill thickness is insufficient to support shallow foundations, particularly any areas where fill has already been placed during the Stage 1 2024/2025 earthworks consent.
	Market, Food & Beverage, Wellness, & Early Childhood Buildings (1 storey)	*Typical Ground Conditions:* 5 – 11 m alluvium (including organic soils) overlying weathered Mahurangi Limestone. *Baseline earthworks:* Bulk fill 0.5 to 2.5 m	Where the baseline bulk fill is greater, shallow raft foundations founded on a specified engineered fill crust may be most economical. Preloading may be required. Strip/pad footings could be considered for larger span structures but with increased engineered fill crust depths and preloading. Driven piles (e.g. timber poles) or screw piles may be preferred for structures not suitable for raft-type foundations (e.g. portal-framed type structures) and where the baseline fill thickness is insufficient to support shallow foundations.
Neighbourhood Precinct South	Standalone Dwellings (2 to 3 storey)	*Typical Ground Conditions:* 4.5 – 11 m alluvium (including organic soils) overlying weathered Mahurangi Limestone. *Baseline earthworks:* Bulk fill 0.5 to 2.5 m	Where the baseline bulk fill is greater, waffle slabs founded on a specified engineered fill crust may be most economical. Preloading may be required due to organic soils and potential creep settlements. Driven piles (e.g. timber poles) may be preferred where the baseline fill thickness is insufficient to support shallow foundations, particularly any areas where fill has already been placed during the Stage 1 2024/2025 earthworks.
Neighbourhood Precinct North-East	Standalone Dwellings (1 to 2 storey)	*Typical Ground Conditions:* ≤ 1 m alluvium overlying weathered Mahurangi Limestone. *Baseline earthworks:* Bulk cuts typically 1 – 3 m below original ground levels. Some areas of the site have already been over-excavated and will require filling to build up to design levels. Negligible cut is proposed at the northern end.	Waffle slabs formed on grade to minimise detailed excavations into the rock (for footings). Preloading will not be required. Undercutting (to form a crust) is generally not expected to be required, except for potential localised undercuts of low-strength alluvium which may remain at the northern end of the precinct following bulk earthworks.

Development Precinct	Structure Type	Baseline Earthworks and Ground Conditions Summary	Preliminary Recommendations for Building Foundation and Ground Improvement Options
Neighbourhood Precinct North-West	Standalone Dwellings (2 storey)	*Typical Ground Conditions:* Typically ≤ 1.5 m alluvium, locally up to 4.7 m at south-western and south-eastern corner, overlying weathered Mahurangi Limestone. *Baseline earthworks:* Bulk cuts up to 2 m at eastern end Bulk filling up to 4 m at northern end, western end and lots to the east of the bridge abutment.	Waffle slabs will likely be most suitable. Localised undercutting to form an engineered fill crust may be required at the cut/fill interface where alluvium is not removed (via bulk cutting) and an insufficiently thick engineered fill crust is formed. Preloading is not expected to be required. Alternatively, suspended floors founded on footings or short timber piles could be preferred at the cut/fill interface to reduce extra-over earthworks for ground improvement. Deepened foundations (e.g. timber poles) may be required locally if buildings sit within a 45º zone of influence line behind retaining walls or near the crests of fill embankments.
Live-Work Precinct	Work-Live Terraces (2 storey)	*Typical Ground Conditions:* 0.5 – 4 m alluvium overlying weathered Mahurangi Limestone. Alluvium is thickest at the north-western corner. *Baseline earthworks:* Bulk cuts up to 1.5 m (eastern end) and bulk fills up to 0.5 m at western end	Waffle slab foundations likely most suitable with the formation of an engineered fill crust at the western end (where alluvium will likely remain). Preloading may locally be required at the north-western corner. Alternatively, footings/short piles extending to weathered Mahurangi Limestone materials could be considered to limit ground improvement earthworks.
	Water/Wastewater Tanks and Pump Stations	*Typical Ground Conditions:* 0.5 – 3 m alluvium overlying weathered Mahurangi Limestone. Alluvium is thickest at the western boundary. *Baseline earthworks:* Bulk cuts up to 1.5 m (eastern end) and bulk fills up to 1.0 m at western end.	Foundation types will depend on the typology of the structures. It may be most efficient to found on weathered Mahurangi Limestone via the use of shallow piles and/or footings.
Data Centres	Heavily loaded and Settlement Sensitive Structures	*Typical Ground Conditions:* - Western Data Centre: 7 – 10 m alluvium (including organic CLAY and Fibrous PEAT), overlying weathered Mahurangi Limestone. - Eastern Data Centre: Inferred variable alluvium thickness from 1 – 7 m, overlying weathered Mahurangi Limestone. *Baseline earthworks:* Bulk cuts up to 2 m and bulk fills up to 4 m	Structures likely to need to be fully piled to Mahurangi Limestone inclusive of suspended floor slabs. Driven steel piles are likely to be most suitable.
Stage 5a Vacant Lot Subdivision	Not determined	*Typical Ground Conditions:* <1 m alluvium overlying weathered Mahurangi Limestone. *Baseline earthworks:* Bulk cuts up to 1.5 m and bulk fills up to 1.5 m	Future structures can likely be supported on footings embedded on weathered Mahurangi Limestone Materials or Engineered Fill. Limited undercutting and replacement may be required to remove low strength alluvium below floor slabs and footings. Further advice required depending on land use.
Solar Farm North, Centre and South	Solar Panel Arrays	*Typical Ground Conditions:* - Northern and Southern Solar farms: 0.5 – 3 m alluvium, overlying weathered Mahurangi Limestone. - Central Solar Farm: Inferred > 7 m alluvium (including organic CLAY and Fibrous PEAT), overlying weathered Mahurangi Limestone. *Baseline earthworks:* Bulk cuts up to 6.5 m (northern), 4 m (southern) and 2 m (central)	Foundation solutions will depend on the propriety systems adopted. However, shallow pads are likely most suitable where bulk cuts extend into to Highly to Moderately Weathered Mahurangi Limestone within the northern and southern solar farms, given the likely difficulty to drill/drive piles into this material. Driven or screw piles likely to be most suitable in Central Solar Farm with low-strength alluvium present at design subgrade level.

7.10 Further Work

7.10.1 Developed and Detailed Design

The scope of geotechnical investigations and assessment for this report is considered appropriate to support preliminary design and the substantive application under the Fast-track Approvals Act (2024). Further geotechnical investigation, analysis and reporting will be required to design the following:

- Design of retaining walls and bridge abutments;
- Design of engineered embankment fill slopes in the North-Western neighbourhood precinct (refer Section 7.5.3);
- Design of temporary works for any deep buried infrastructure;
- Design of ground improvement works beneath building platforms (i.e. preloading and construction of fill crusts). This should be undertaken prior to the commencement of bulk earthworks filling;
- Design of subgrade improvements for pavements;
- Design of building foundations (shallow and deep); and
- Design of the lagoon.

Supplementary investigations, site trials and laboratory testing are also likely to be warranted to optimise earthworks.

It is important that all future civil and structural drawing sets are reviewed by a Geotechnical Engineer to check the recommendations in this report and future reporting are correctly interpreted and implemented.

7.10.2 Construction Monitoring

Given the significant earthworks at the site and challenging ground and groundwater conditions, a high level of geotechnical construction observation and support will be required for the project. Construction stage observations will be confirmed following detailed design stages; however, it is likely that the geotechnical aspects in Table 7-14 will need to be verified.

Table 7-14: Preliminary Geotechnical Construction Observation Requirements

Stage	Observation Point	Specification / Acceptance Criteria
Bulk earthworks	Following stripping of topsoil and unsuitables.	Confirm that topsoil and unsuitables have been removed and appraisal of soft spots. Confirm strength of exposed soils.
	Subgrade level prior to bulk filling	Confirm composition and strength of exposed soils. Appraise undercut requirements and confirm appropriate “engineered crust” thickness.
	“Mucking out” of low-lying areas	Confirm soft, unsuitable materials have been removed
	Bulk excavations	Observe stability of temporary and permanent cut batters, Review excavated soils and appraise suitability and conditioning requirements for re-use as bulk fill.
	Subsoil Drainage	Review subsoil drain layout based on subgrade conditions. Observe subsoil drainage.
	Bulk filling	Review of bulk fill materials. Compaction testing of bulk fill by a geotechnical technician.
	Engineered Embankments	Observe ground improvements (e.g. shear key, drainage). Observe placement of reinforcement (e.g. geogrids) and compaction of fill.
	Preloads and settlement monitoring	Monitoring of settlement monitoring data from preloads and settlements beneath bulk fill. Geotechnical engineer to instruct when preload may be removed and if any adjustments are required to the preload height based upon the monitoring data.

Stage	Observation Point	Specification / Acceptance Criteria
Civil Infrastructure	Excavation retention/batter grades	Observation of batters to appraise stability. Observations of construction of temporary retention, if adopted.
	Subgrade and subgrade improvement	Appraise subgrade strengths. Review and advise on subgrade improvements where required.
	Backfill testing	Compaction testing of backfill materials by a geotechnical technician
Lagoon Liner	Appraisal of subgrade conditions	Confirm in situ strengths at subgrade level and advice on any undercutting or stabilisation requirements.
	Compaction testing and observations of "build up"	Compaction testing and observations on "build up" below the liner, in accordance with the lagoon liner designer's specifications.
Retaining Walls	During retaining wall foundation construction	Observation of either pile holes during drilling (for piled walls) or observation and testing of footing subgrade (for gravity walls).
Shallow Building Foundations	Foundation preparation	Confirm in situ strengths in footing excavations are consistent with those assumed in design. Appraisal of any undercut requirements.
Piled Building Foundations	During installation of piles	Observation during pile drilling for bored piles to confirm ground conditions are consistent with design assumptions. Observation of pile driving and review of pile driving records and PDA testing for driven piles.
Floor Slabs and Pavements	Preparation of pavement subgrade	Confirm CBR is consistent with design and appraisal of any undercut or other subgrade improvement requirements.
	Following placement of basecourse	Compaction testing in accordance with design engineer's requirements.

8. Conclusions

The following key geotechnical conclusions can be made in relation to the preliminary design and substantive application (under the Fast-track Approvals Act 2024) of the proposed Stage 2 ASPC development:

Report Scope and Purpose:

1. This report has been prepared to support proposed Section 127 variations to the existing Stage 1 Resource Consent (BUN60429155) and support the Stage 2 Resource Consent for the expansion of the project to include new land parcels. The report provides advice on all precincts of the development except for the Surf Lagoon and associated bulkhead structures, for which the existing Stage 1 Geotechnical Report should be referred to.

Subsurface Conditions:

3. The site is underlain by topsoil and localised uncontrolled fill, underlain by Tauranga Group Alluvial soils and a weathering profile of Mahurangi Limestone soils and rock. The thickness of alluvial soils and subsequent depth to rock varies across the site and is typically less than 2 m within the elevated northern, eastern and southern hillsides and between 5 – 12 m within the central low-lying core.
4. The Tauranga Group alluvium typically comprises firm to stiff SILT and CLAY mixtures. Organic CLAYS and PEAT are also present, particularly at the eastern end of the low-lying core.
5. A complex groundwater regime is present at the site which generally includes:
    a. Shallow perched groundwater of limited depth and volume, generally sitting within the thick topsoil within the low-lying central core;
    b. Static groundwater, typically sitting within the alluvial soils in the central core between approximately RL 51 – 52 m, grading down towards the existing creek channels;
    c. Inferred groundwater within the fractured Mahurangi Limestone rock on the elevated hillsides, percolating towards the central core; and
    d. Potential artesian conditions within the Mahurangi Limestone.

Seismic Considerations:

6. The NZS1170.5 Seismic Subsoil Class is considered Class B (Rock) where the soil mantle above the Mahurangi Limestone rock is less than 3 m thick and Class C (Shallow Soil) where the soil mantle is greater than 3 m thick.
7. The effects of liquefaction are insignificant under both SLS and ULS design earthquakes. No specific detailing is required for shallow foundation systems in relation to liquefaction effects. Liquefaction effects should be considered in the design of deep foundations but are unlikely to govern design.
8. The risk of seismic cyclic softening is low and it is generally not expected to govern the design of structures.

Earthworks Considerations:

9. Many buildings are expected to require specific ground improvement to enable shallow foundation systems which may involve sub-excavation to form an engineered fill bearing crust and/or preloading. It is strongly recommended the ground improvement requirements are considered and integrated into the bulk earthworks operations to leverage scale efficiencies and minimise re-work.
10. The low-strength alluvial soils will generally not be suitable for directly trafficking or compacting engineered fill against. A working layer comprising a coarse aggregate (e.g. site-won lime rock), geotextile and geogrid will generally be required.

11. The site-won, inorganic alluvial clays and residual soils will generally be wet of optimum water content and require significant drying prior to placement as engineered fill. Blending of these materials with site-won limestone rock or imported fill will reduce the drying periods.
12. Some site-won soils (e.g. organic clays) will not be suitable for re-use as engineered fill, even with conditioning.
13. Site-won fill materials (including Mahurangi Limestone rock) will generally only be suitable for placement during the summer earthworks season as they soften when wet.
14. The placement of fill above existing ground levels will trigger consolidation settlements and a hold period will need to be observed between the completion of filling and construction of both buildings and settlement sensitive infrastructure. Hold periods are generally expected to be less than 5 months but may be longer (potentially up to 12+ months) within the Data Centre, Central Solar Farm and Southern Neighbourhood Precincts where higher compressibility soils are present.

Groundwater Drawdown and Effects:

15. The proposed bulk earthworks will likely intercept groundwater, particularly perched groundwater and percolating groundwater within the Mahurangi Limestone. However, due to the low permeability of the soils, the age/stiffness of the Limestone and the considerable offsets to neighbouring services and structures, groundwater drawdown settlement effects on existing structures or services outside the site boundaries are expected to be negligible.
16. The earthworks fall outside the definition of a permitted activity in accordance with E7 of the Auckland Unitary Plan and a specific groundwater consent may be required. However, since offsite effects are assessed to be negligible, no specific groundwater or settlement monitoring is considered to be required.

Slope Stability Considerations:

17. The proposed 1V:3H cuts of up to 6.5 m depth within the Northern and Southern Solar farms are expected to extend into Mahurangi Limestone Rock. The cuts are expected to be stable but should be mapped and closely monitored by a Geologist during construction to confirm design assumptions.
18. The proposed 1V:3H cut batter (up to approximately 3.5 m in height) in alluvium at the northern end of the accommodation precinct is expected to be suitable. However, the design of foundations above or within the slope should be specifically reviewed.
19. The proposed fill embankments within the North-Western accommodation precinct will require specific geotechnical review and design to mitigate instability. Measures such as keying fill into the rock, palisade piles, underfill drainage and/or geogrid reinforcement may be required.
20. A landslide risk assessment has been completed in accordance with Appendix 24 of Plan Change 120. The assessment demonstrates landslide risk is low (tolerable) provided recommendations in this report are adhered to.

Retaining Wall Considerations:

21. Preliminary options and design considerations for retaining walls are presented in Section 7.6. Depending on the location, and wall height, and whether the walls support cut or fill, gravity walls (e.g. block walls, MSE walls) and/or piled walls (e.g. timber pole, steel UC) will likely be suitable.
22. Boundary retaining walls are proposed to support cuts up to 1.5 m deep. A Before U Dig and aerial imagery review indicates no existing services or structures are located within 20 m upslope of these proposed walls. Therefore, mechanical settlements due to retaining wall deflections will not have adverse effects on neighbouring properties.

Civil Infrastructure Considerations:

23. Sequencing of works will need to consider the effect of fill induced settlements on buried infrastructure;
24. Any pipes and manholes deeper than approximately 3 m below existing ground level are likely to be challenging to install/construct and may require temporary retention including temporary propping systems and groundwater cutoff measures (e.g. sheet piles);
25. Subgrade conditions at the base of service trenches are generally expected to be low strength and will require subgrade improvement prior to placing bedding material.

Pavement Considerations:

26. Where pavement subgrades comprise a minimum thickness of 0.5 m engineered fill, a subgrade CBR of 4% is expected to be achieved. Pavements founded on natural residual soils or weathered rock will likely achieve subgrade CBRs of 3% and 5% respectively;
27. Where alluvial soils are present at or near design subgrade level, specific ground improvement such as dig-out-and-replace or lime/cement stabilisation is likely to be required;
28. The soils at the site are sensitive and loose strength when trafficked or exposed to wet weather. Subgrades should be prepared in dry conditions and be protected against trafficking with sacrificial layers.

Building Foundation Considerations:

29. Shallow foundation systems will likely be suitable for most light-weight structures, however ground improvement will be required in some areas, particularly where deep alluvium is present. Ground improvements will likely include:
    a. Construction of an engineered fill crust to provide adequate bearing; and
    b. Preloading to address consolidation and creep settlements (subject to confirmation of settlement magnitudes by further investigation and analysis and building loads/levels/layouts and sensitivity).
30. The existing soils at the site are expansive. Foundations supported in natural soils or site won fill will likely need design for expansive site Class H (Highly Expansive).
31. Raft foundation systems (in lieu of strip/pad footings) are recommended to reduce the embedment requirement (and subsequent engineered fill crust depth) and provide improved resilience to differential settlement.
32. Heavy structures (e.g. Hotel and Data Centres) will need to be piled. Driven piles (e.g. steel UC sections) are likely to be the most cost-effective solution. Bored piles could be considered but would likely need casing or support fluid to prevent shaft collapse where deep alluvium is present.

Further Work:

33. The scope of geotechnical assessment presented in this report is considered appropriate to support preliminary design and the substantive application under the Fast-track Approvals Act (2024).
34. Additional geotechnical investigations, analysis and reporting will be required to design and detail stabilised embankments, retaining walls, ground improvements and building foundations. Further detail is provided in Section 7.10.1.
35. Given the efficiencies of completing ground improvements (bearing crust construction and preloading) during bulk earthworks, it is recommended that ground improvement design (which will be contingent on building loads, layouts and levels) is undertaken ahead of commencing engineered bulk filling below future building platforms.

9. Applicability

This report has been prepared for our client, ASP Bei Land Co, with respect to the brief provided to us. The advice and recommendations presented in this report should not be applied to any other project or used in any other context without prior written approval from Initia Limited.

Report prepared by:

Kieran Bursell
Geotechnical Engineer

Report reviewed by:

Nick Speight
Senior Geotechnical Engineer, Director

Document Control Record

Report Title		Auckland Surf Park Community – Stage 2 Fast Track Consent Geotechnical Assessment Report			
Initia Project Reference		P-001537-2			
Client		ASP Bei Land Co			
Revision	Date	Revision Detail	Author	Reviewer	Approved By
0	07-11-2025	First Issue	K. Bursell	N. Speight	N. Speight
A	10-12-2025	Updated following minor development plan updates and planner's review	K. Bursell	N. Speight	N. Speight
B	18-12-2025	Minor updates to reflect Data Centre layout amendment	K. Bursell	N. Speight	N. Speight
C	15-04-2026	Minor updates to reflect layout amendment	K. Bursell	N. Speight	N. Speight
D	09-06-2026	Masterplan revision to eastern extent of Surf Park	N. Speight	N. Speight	N. Speight
Current Revision		D			

Appendix A Initia Figures

LEGEND

INITIA INVESTIGATION (AUG 2025)

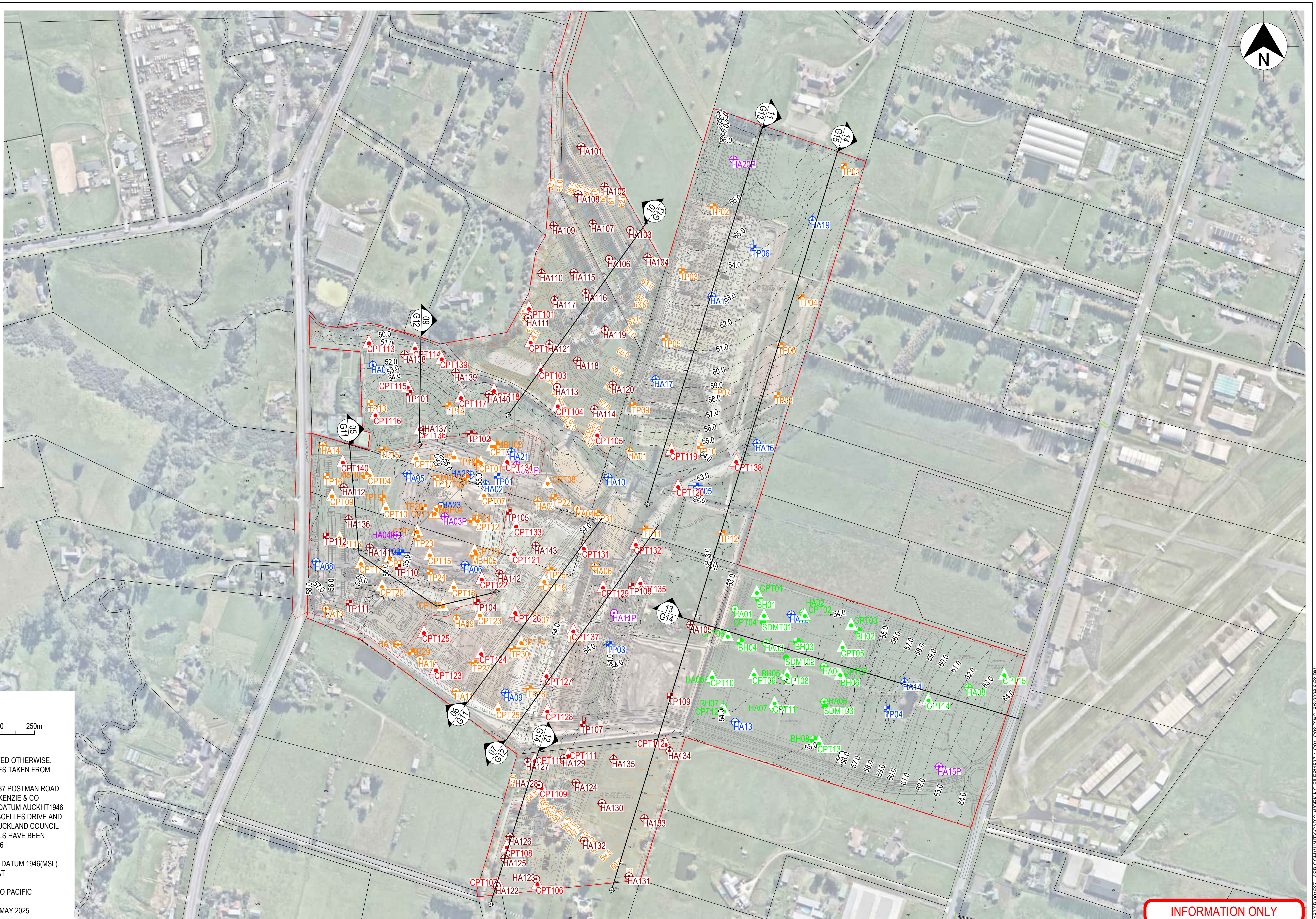
- CPT118 — CONE PENETRATION TEST
- TP101 — TEST PIT
- HA112 — HAND AUGER

INITIA INVESTIGATION (MAR - APR 2023)

- CPT01 — CONE PENETRATION TEST
- MBH01 — MACHINE BOREHOLE
- HA01 — HAND AUGER
- TP01 — TEST PIT

HISTORICAL INVESTIGATION

- HA01P — HAND AUGER WITH PEIZOMETER (T&T - MAY 2022)
- HA10 — HAND AUGER (T&T - MAY 2022)
- TP01 — TEST PIT (T&T - MAY 2022)
- CPT06 — FLAT PLATE DILATOMETER (AURECON - APRIL 2023)
- BH04 — MACHINE BOREHOLE (AURECON - APRIL 2023)
- HA03 — HAND AUGER (AURECON - APRIL 2023)
- SITE BOUNDARY
- 0.5 — EXISTING GROUND CONTOUR (0.5m INTEVAL) TOPOGRAPHICAL SURVEY (DATED 24/03/2023)
- 0.5 — EXISTING GROUND CONTOUR (0.5m INTEVAL) AUCKLAND COUNCIL LIDAR (DATED 2016)



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0 50 100 150 200 250m

NOTES

1. ALL DIMENSIONS ARE IN METRES UNLESS NOTED OTHERWISE.
2. PROPERTY BOUNDARY AND EXISTING SERVICES TAKEN FROM AUCKLAND COUNCIL DATE 2017.
3. CONTOURS FOR 1350 DAIRY FLAT HIGHWAY, 237 POSTMAN ROAD AND 253 POSTMAN ROAD SOURCED FROM MCKENZIE & CO TOPOGRAPHICAL SURVEY, DATED 24/03/2023, DATUM AUCKHT1946
4. CONTOURS FOR 89 LASCELLES DRIVE, 105 LASCELLES DRIVE AND 1320 DAIRY FLAT HIGHWAY SOURCED FROM AUCKLAND COUNCIL LIDAR SURVEY, DATED 2016. CONTOURS LEVELS HAVE BEEN TRANSPOSED FROM NZVD2016 TO AUCKHT1946
5. COORDINATE DATUM: MOUNT EDEN 2000
6. LEVEL ARE IN TERMS OF AUCKLAND VERTICAL DATUM 1946(MSL). ORIGIN OF LEVELS - MARK:BACV DAIRY FLAT RL: 46.107m (LINZ NZVD2016 CONVERSION)
7. SITE DEVELOPMENT PLAN SUPPLIED BY STUDIO PACIFIC ARCHITECTURE. DATED 03/06/2026
8. AERIAL SHOWN TAKEN FROM NEARMAP DATE MAY 2025

INFORMATION ONLY

Rev	Revision Description	Designed	Drawn	Checked
E	UPDATED SITE DEVELOPMENT PLAN (03/06/2026)	NS	GG	NS
D	UPDATED SITE DEVELOPMENT PLAN (15/04/2026)	KB	JG	KB
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ASP BEI LAND CO

AUCKLAND SURF PARK

GEOTECHNICAL INVESTIGATION LOCATION PLAN

Initia Project ref: P-001537

Figure Number: 1537-1-G01

Revision: E

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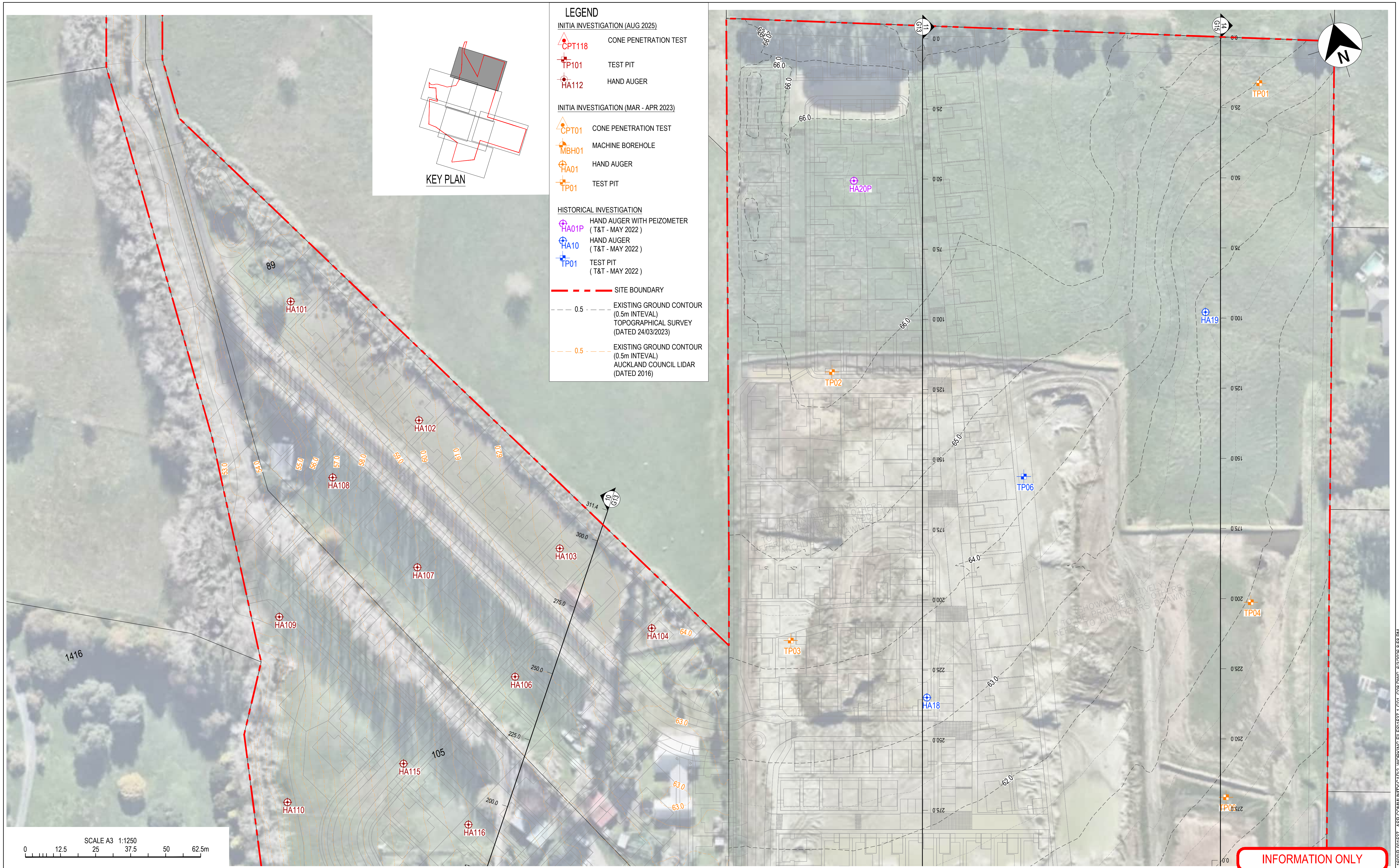

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GEOTECHNICAL INVESTIGATION
LOCATION PLAN
(SHEET 1 OF 7)

Initia Project ref: P-001537

Figure Number: 1537-1-G02

Revision: E

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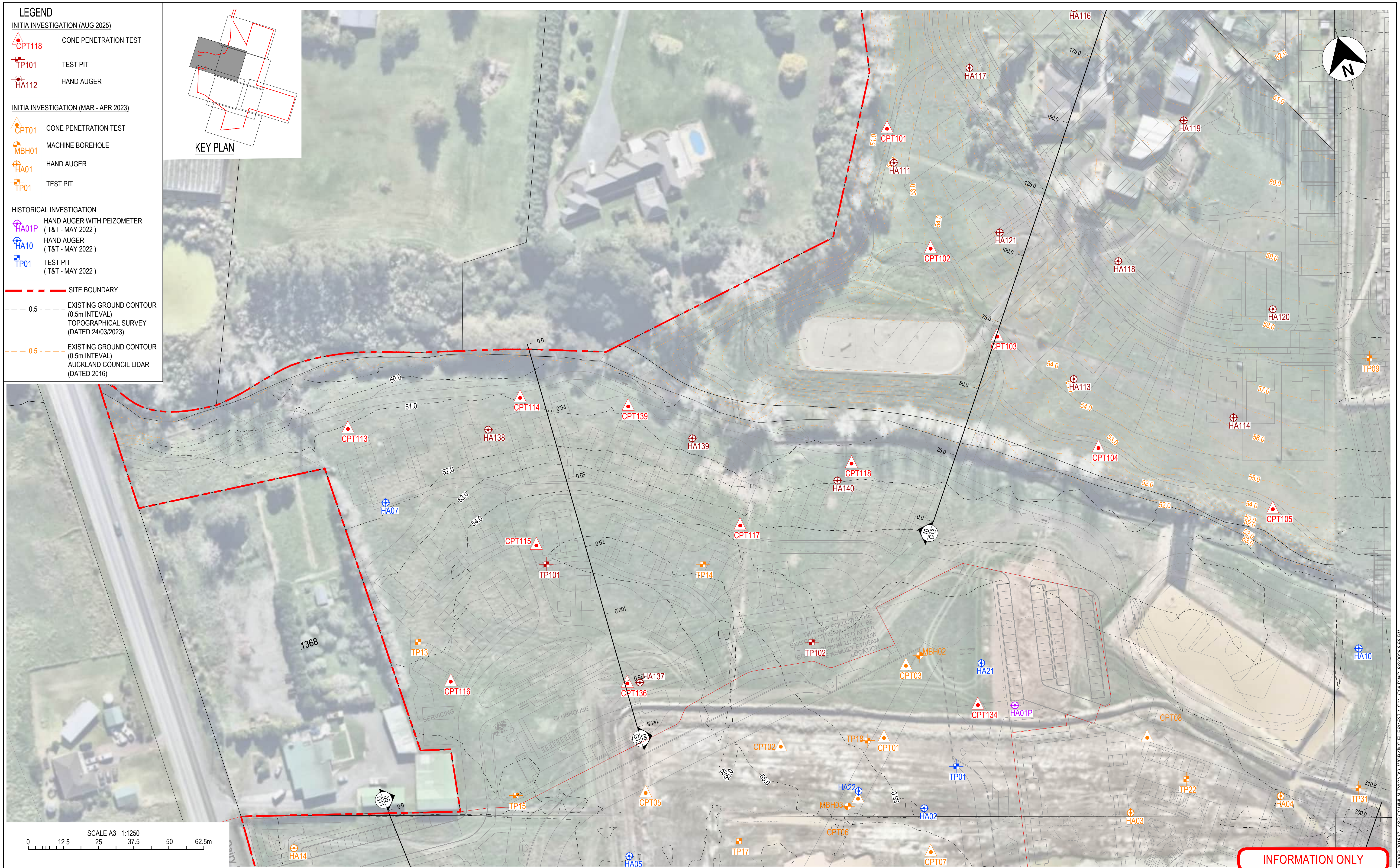

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GEOTECHNICAL INVESTIGATION
LOCATION PLAN
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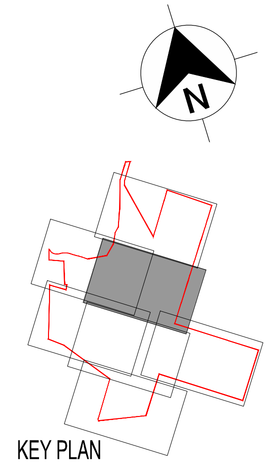
Initia Project ref:	P-001537
Figure Number	1537-1-G03
Revision	E



LEGEND

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- TP101 TEST PIT
- HA112 HAND AUGER

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- MBH01 MACHINE BOREHOLE
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- HA10 HAND AUGER (T&T - MAY 2022)
- TP01 TEST PIT (T&T - MAY 2022)

SITE BOUNDARY

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0.5 EXISTING GROUND CONTOUR (0.5m INTEVAL) AUCKLAND COUNCIL LIDAR (DATED 2016)

CPT01

INFORMATION ONLY

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GEOTECHNICAL INVESTIGATION LOCATION PLAN (SHEET 3 OF 7)

Initia Project ref: P-001537

Figure Number: 1537-1-G04

Revision: E

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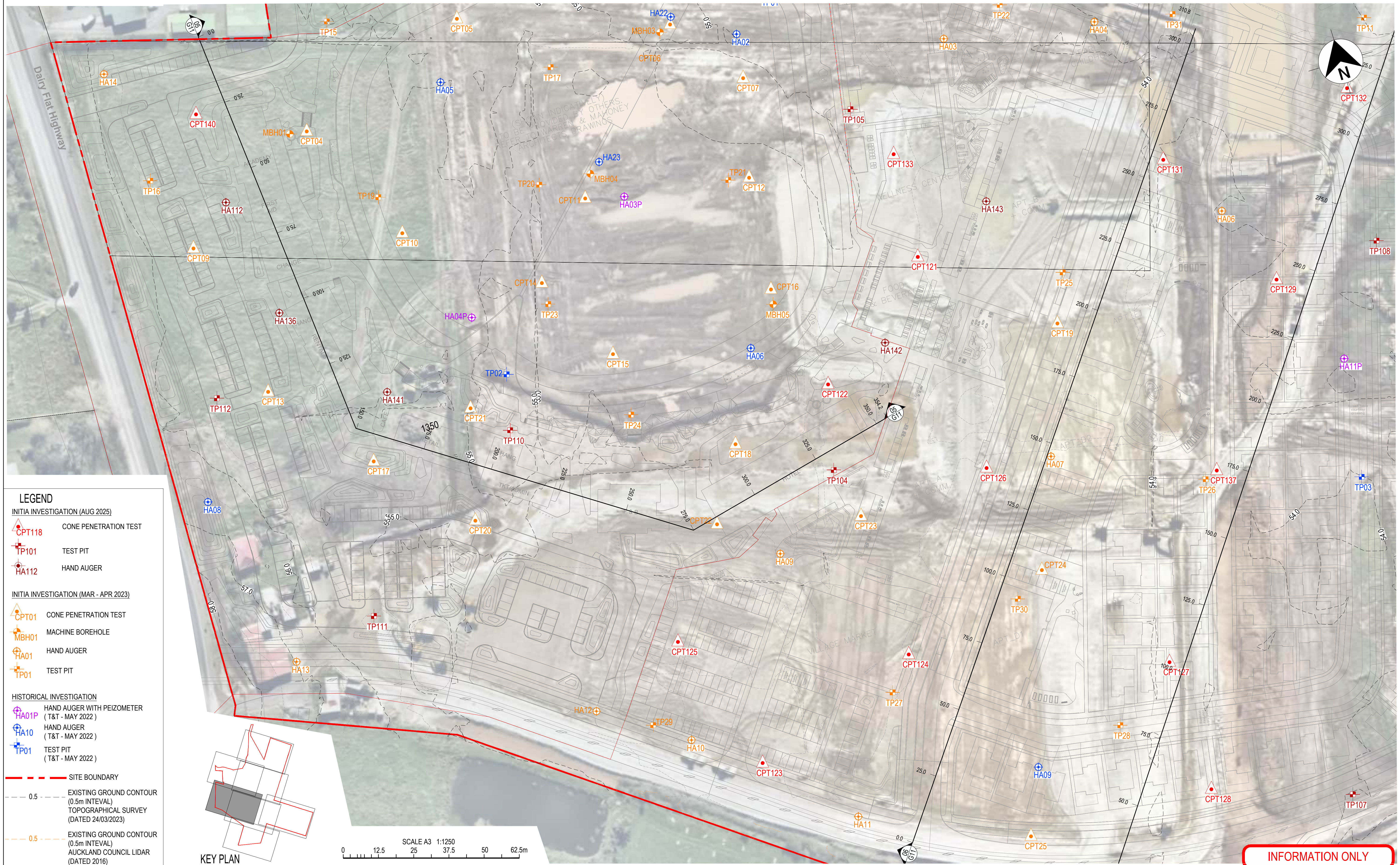

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GEOTECHNICAL INVESTIGATION LOCATION PLAN (SHEET 4 OF 7)

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Figure Number 1537-1-G05
Revision E

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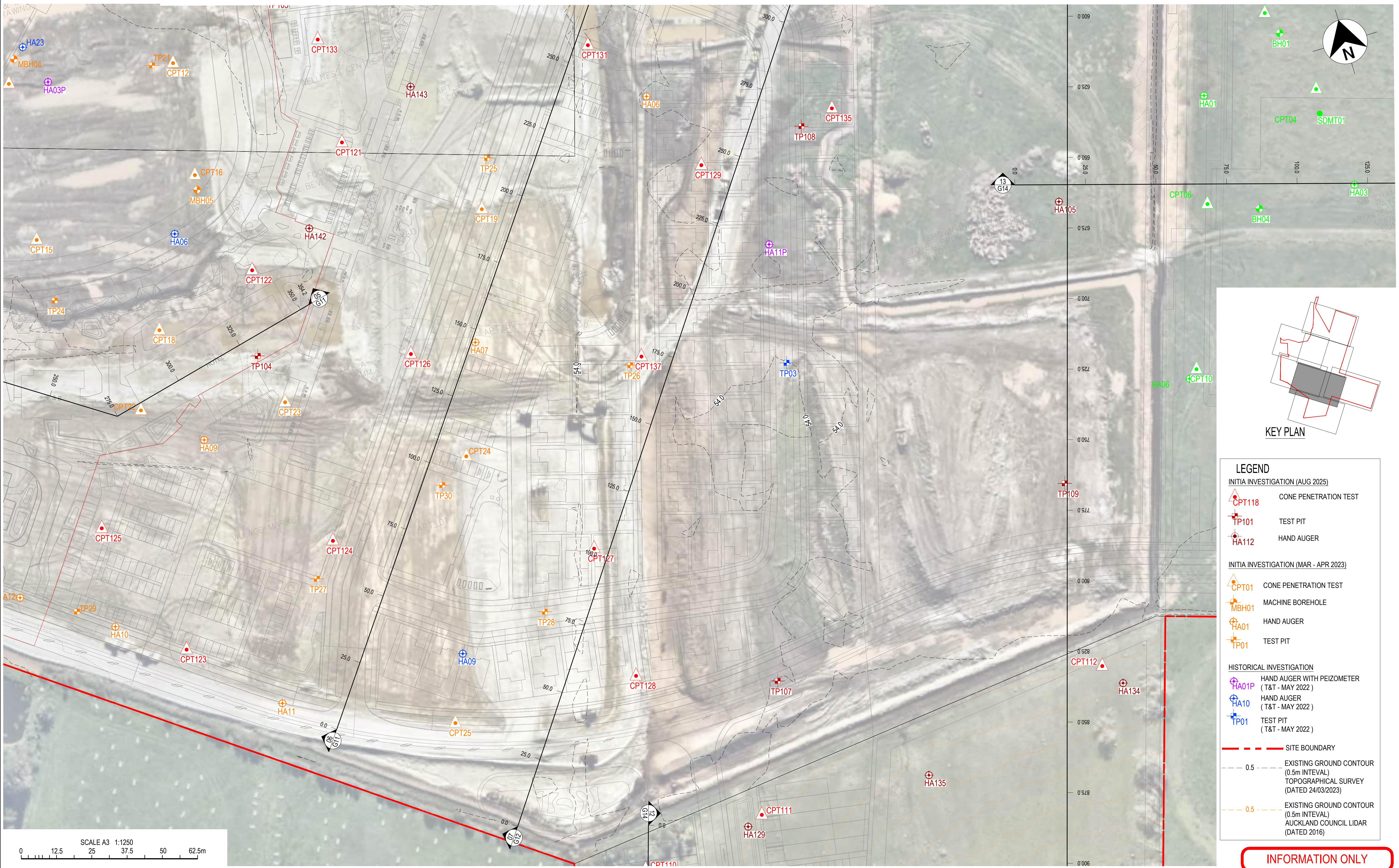

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GEOTECHNICAL INVESTIGATION LOCATION PLAN (SHEET 5 OF 7)

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Figure Number 1537-1-G06

Revision E

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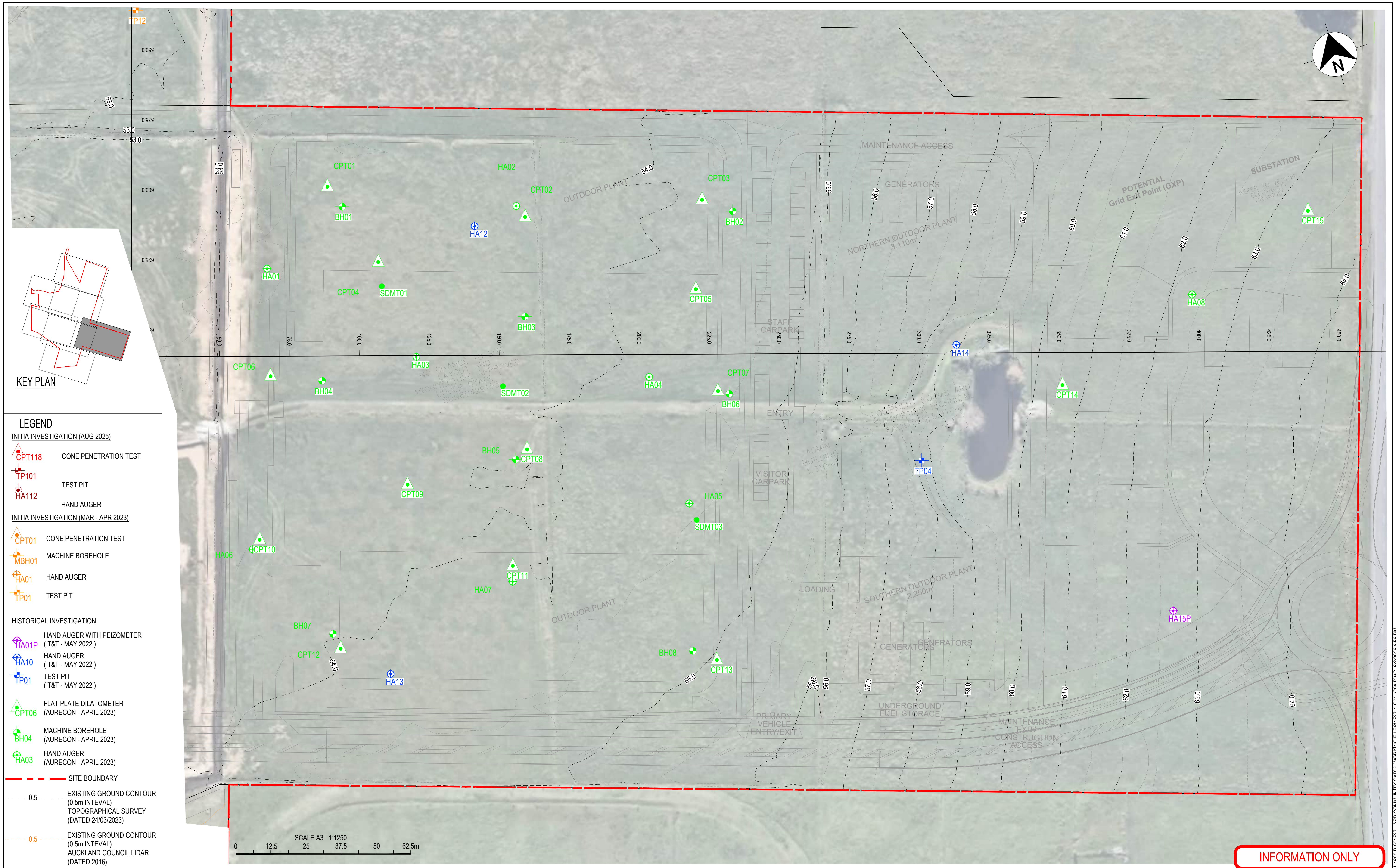

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GEOTECHNICAL INVESTIGATION LOCATION PLAN (SHEET 6 OF 7)

Initia Project ref: P-001537

Figure Number 1537-1-G07

Revision E

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GEOTECHNICAL INVESTIGATION LOCATION PLAN (SHEET 7 OF 7)

Initia Project ref: P-001537
Figure Number: 1537-1-G08
Revision: E

Appendix B ULS Liquefaction Assessment Outputs

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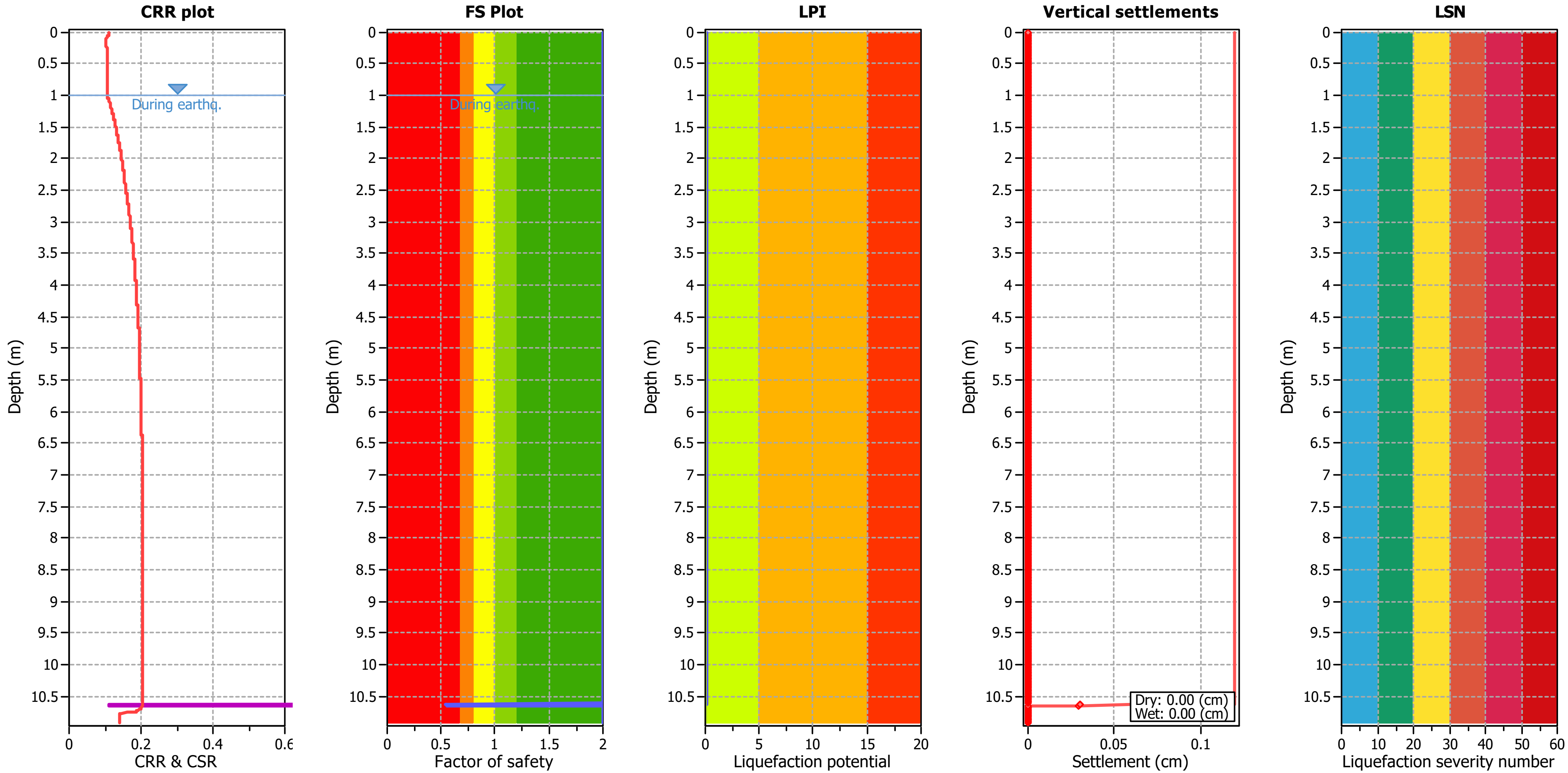
Geotechnical Specialists
114 St. Georges Bay Road, Auckland
www.initia.co.nz

Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT03
Total depth: 10.92 m



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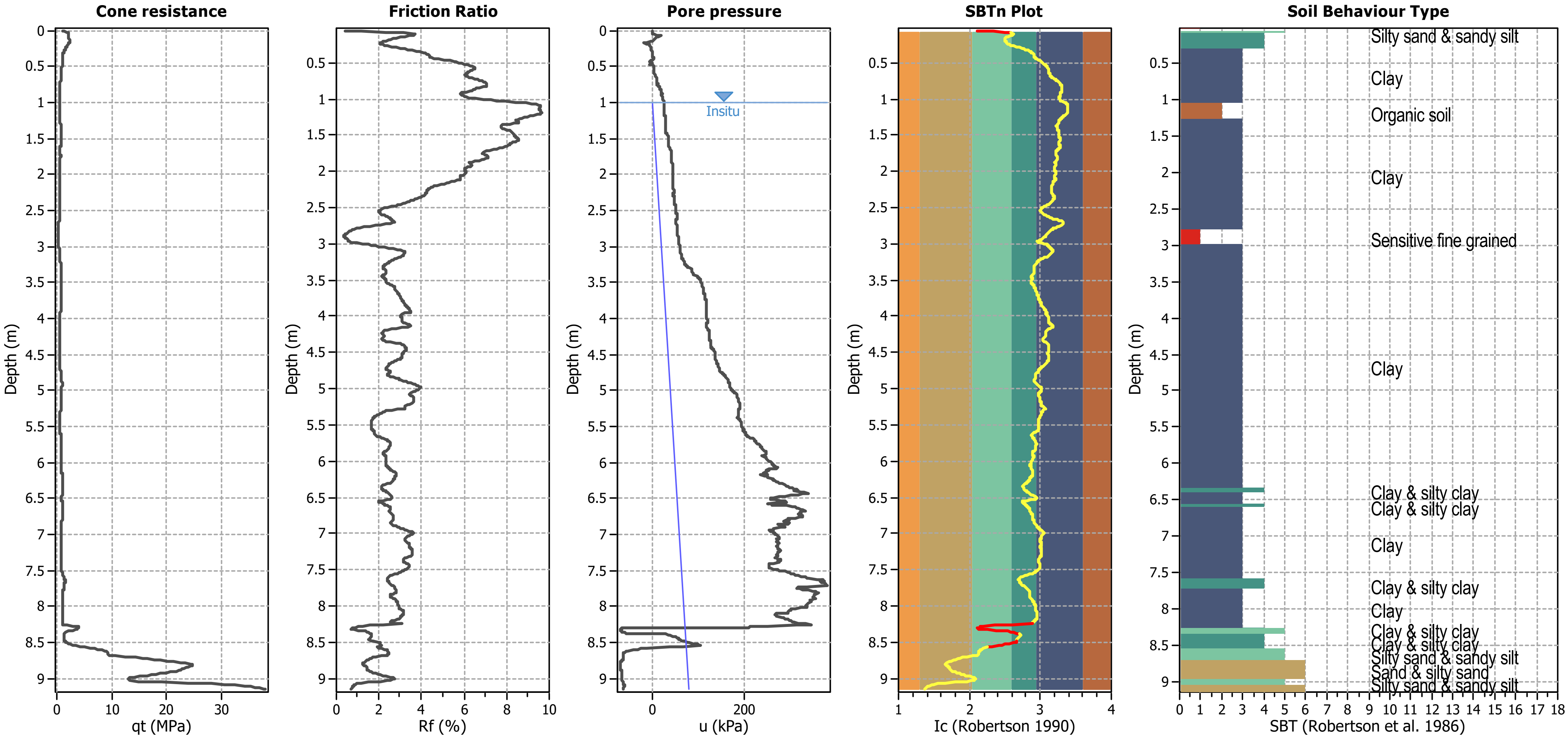
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Initia Ltd
Geotechnical Specialists
114 St. Georges Bay Road, Auckland
www.initia.co.nz

Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT04
Total depth: 9.15 m



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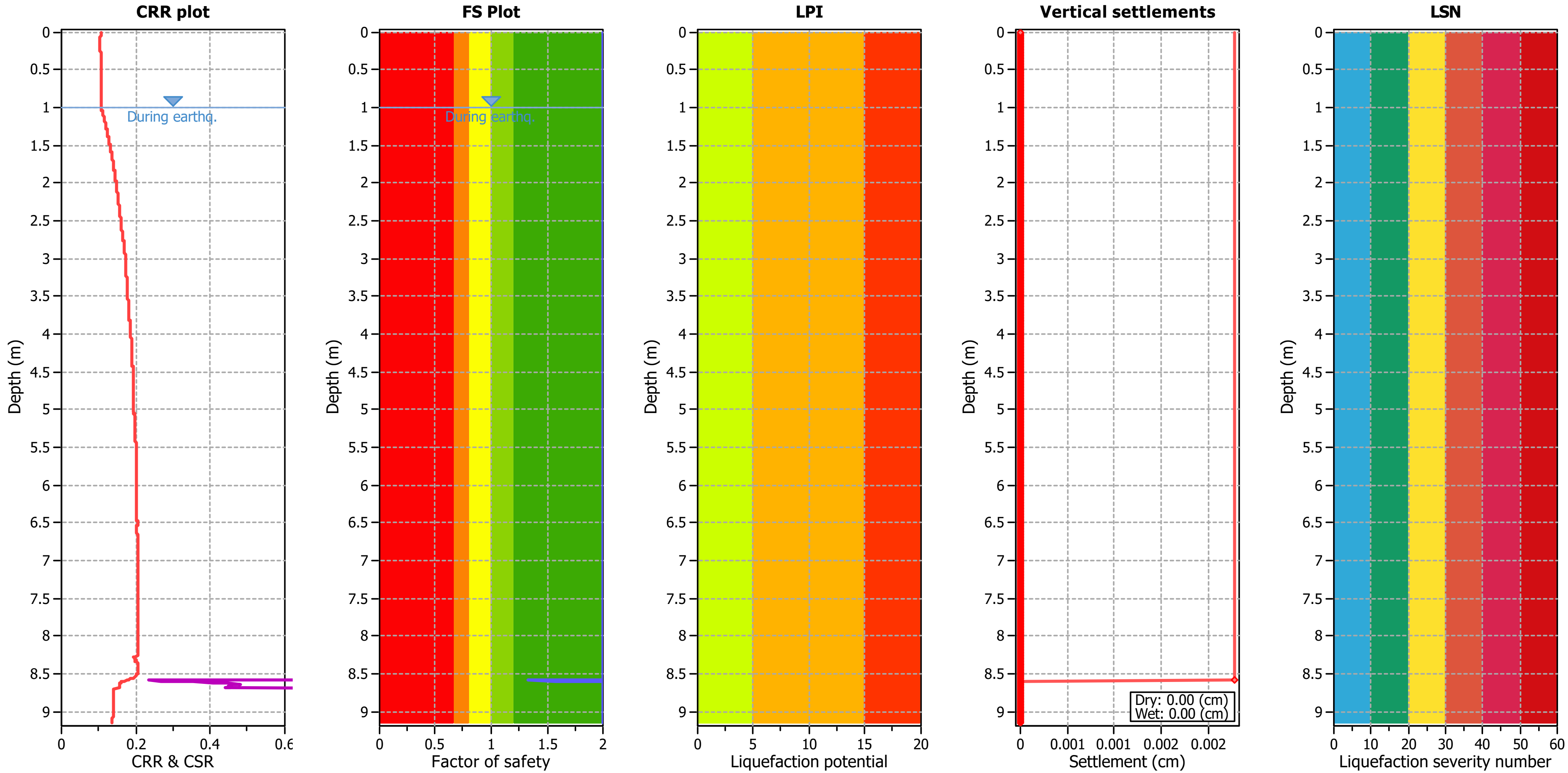
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Initia Ltd
Geotechnical Specialists
114 St. Georges Bay Road, Auckland
www.initia.co.nz

Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT04
Total depth: 9.15 m



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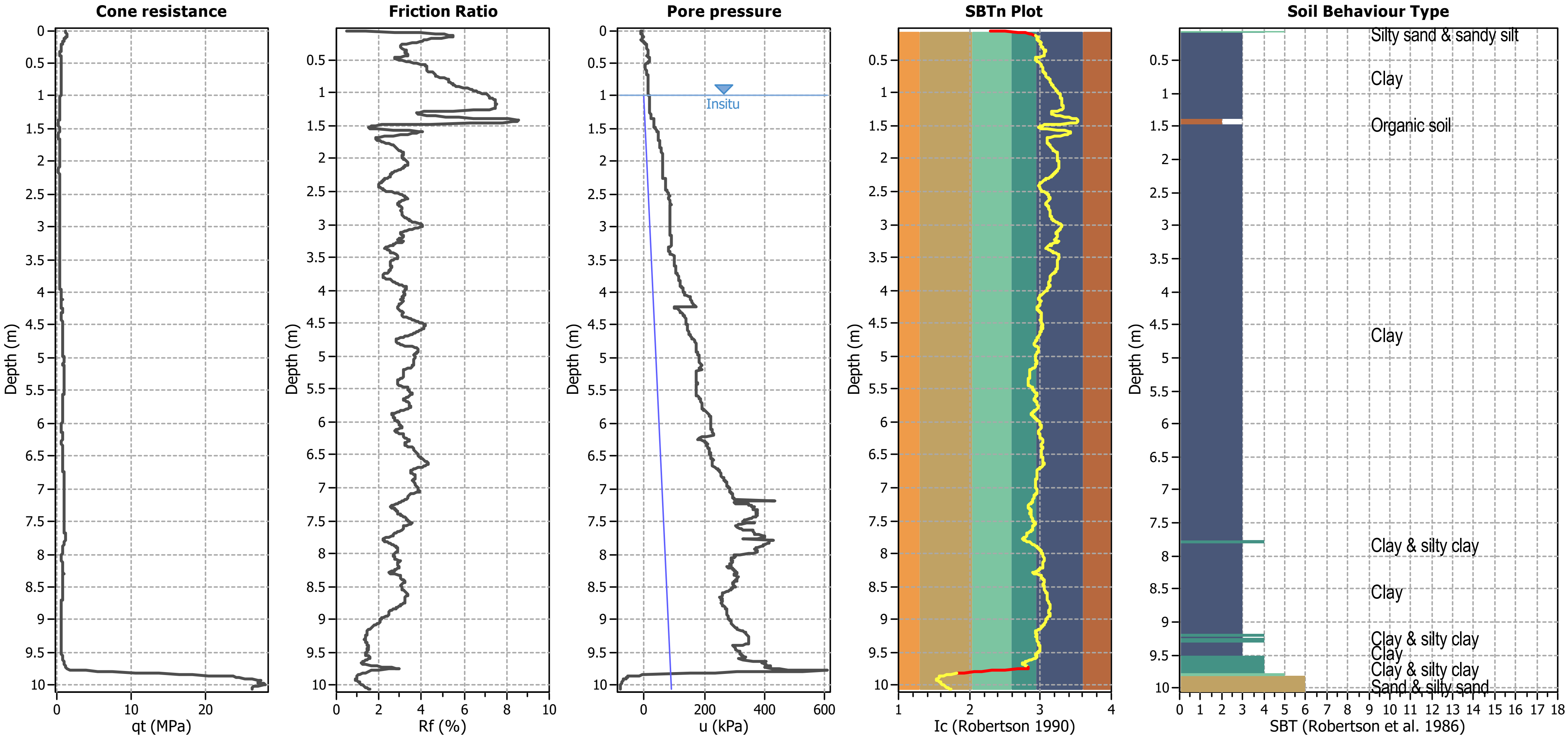
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Initia Ltd
Geotechnical Specialists
114 St. Georges Bay Road, Auckland
www.initia.co.nz

Project: Auckland Surf Park **CPT: CPT05**

Location: Dairy Flat, Auckland Total depth: 10.06 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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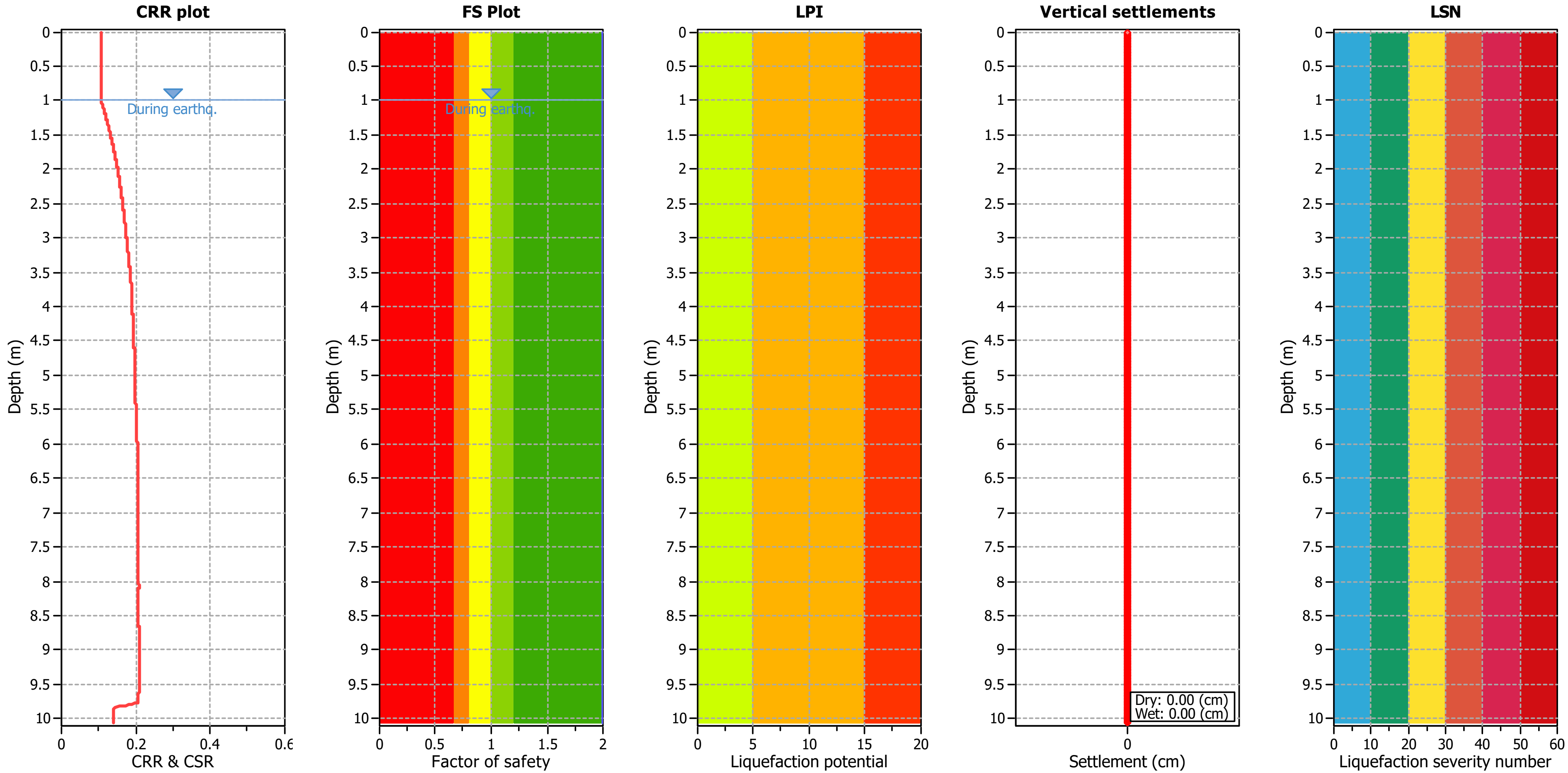
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT05
Total depth: 10.06 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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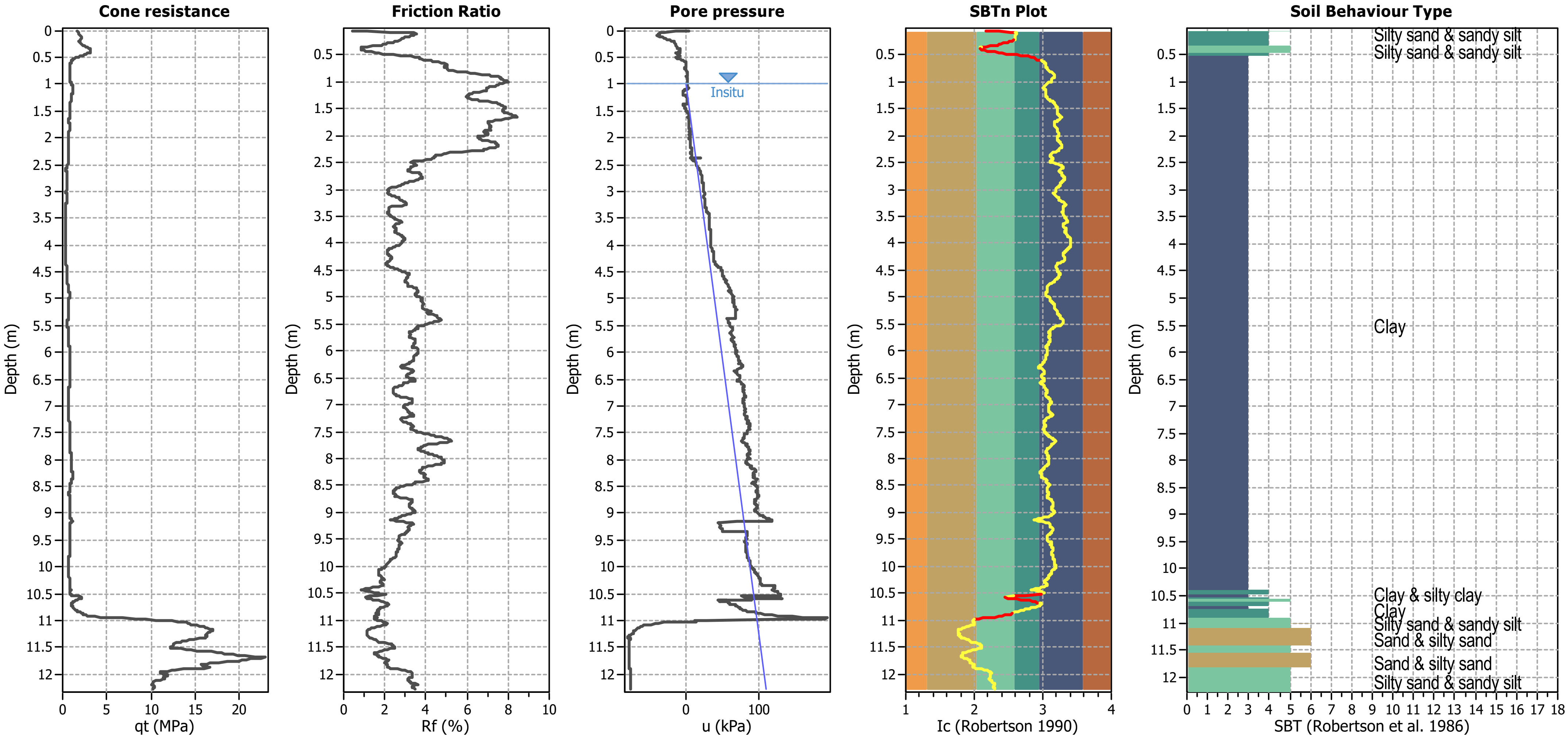
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT06
Total depth: 12.28 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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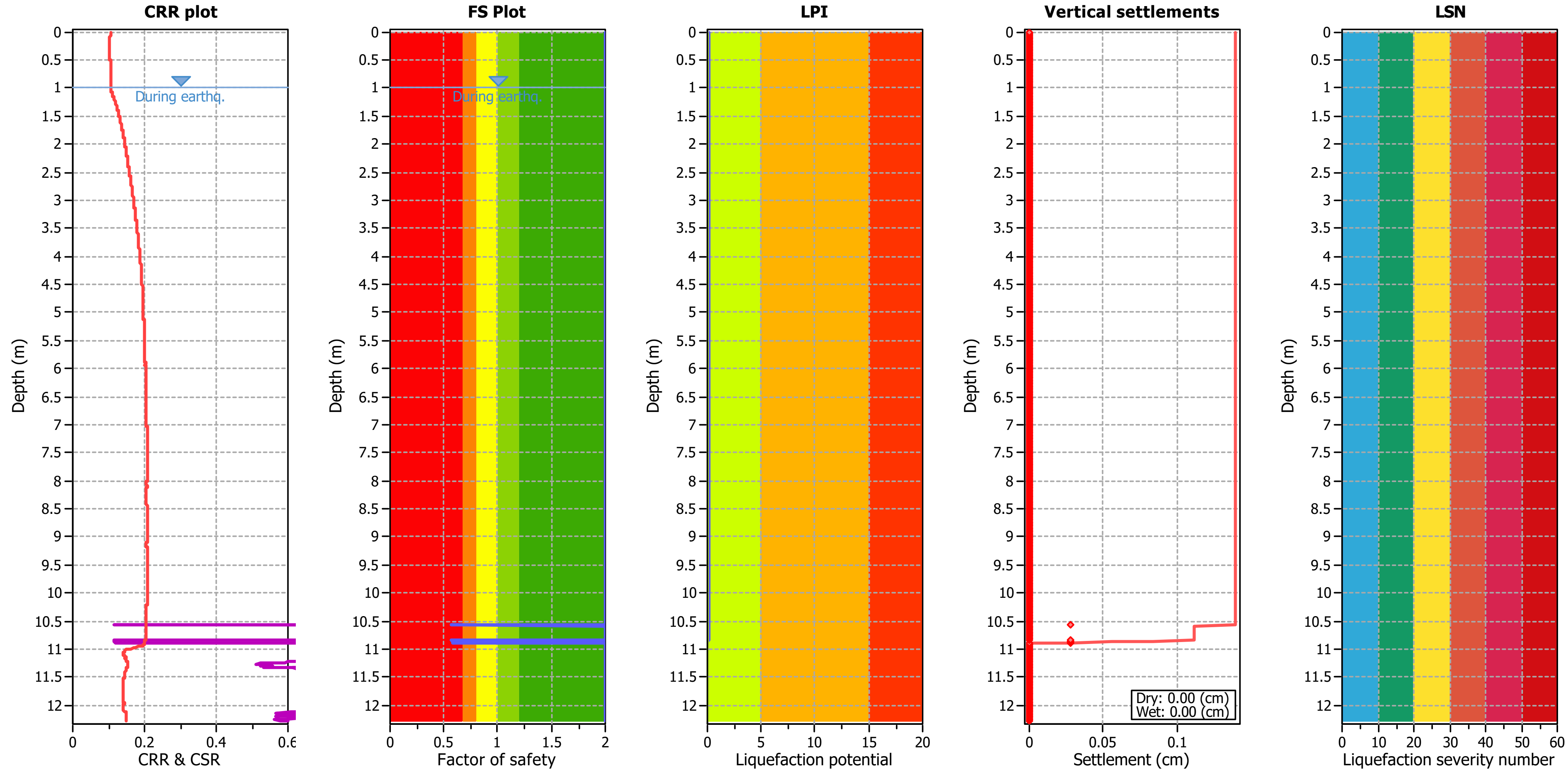
Project: **Auckland Surf Park** | **CPT: CPT06**
Location: **Dairy Flat, Auckland** | Total depth: 12.28 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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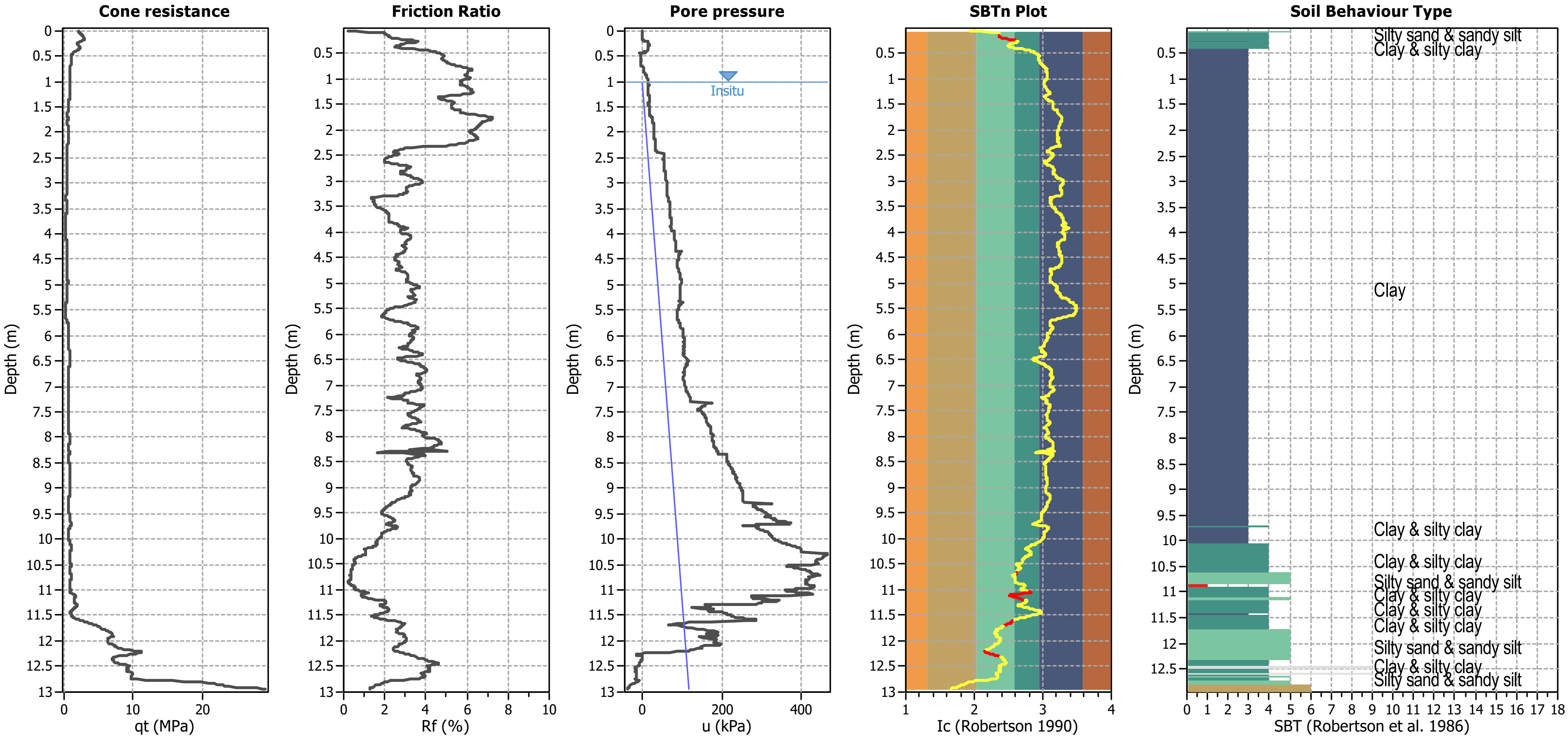
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

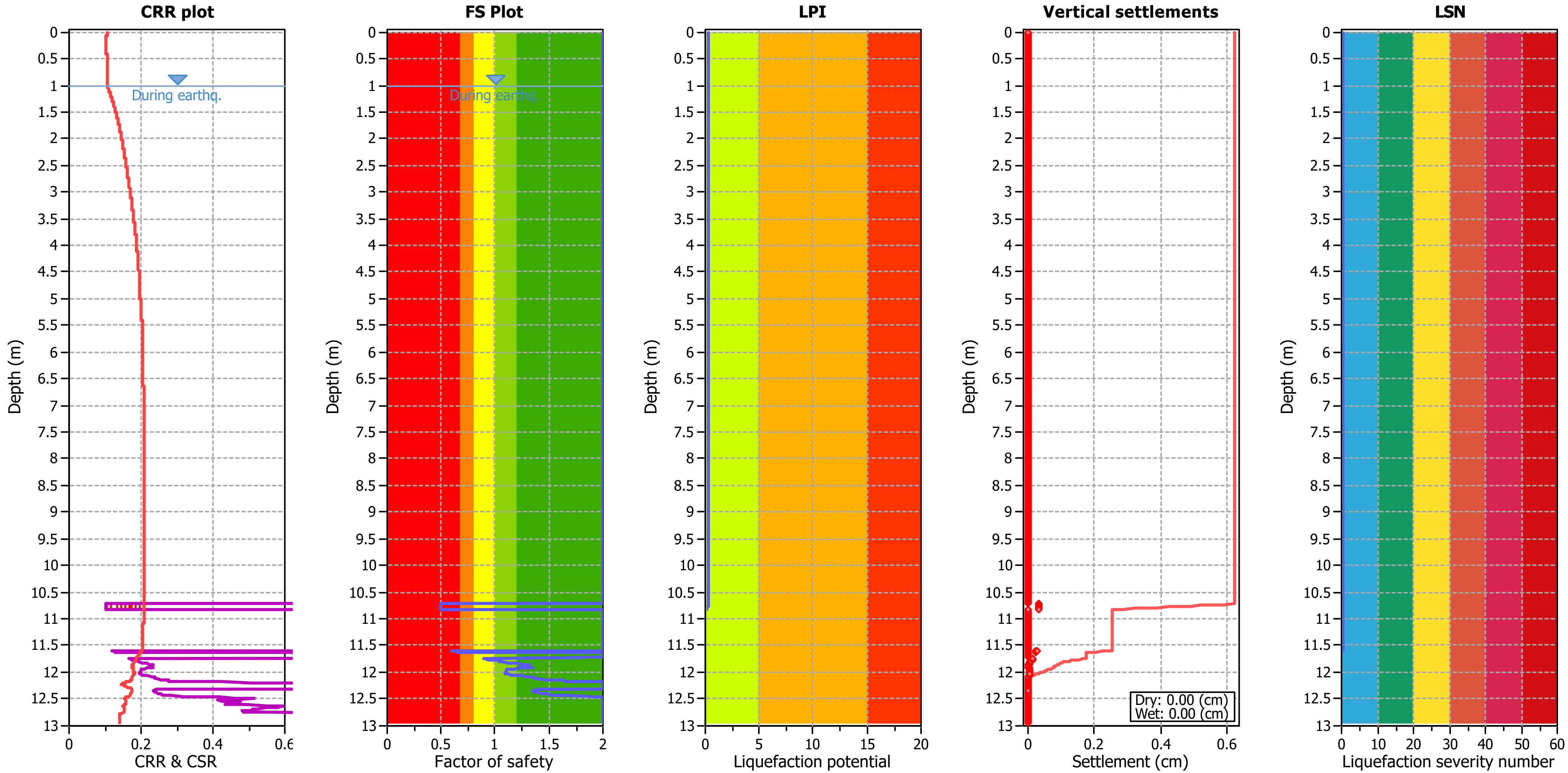
CPT: CPT07
Total depth: 12.95 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT07
Total depth: 12.95 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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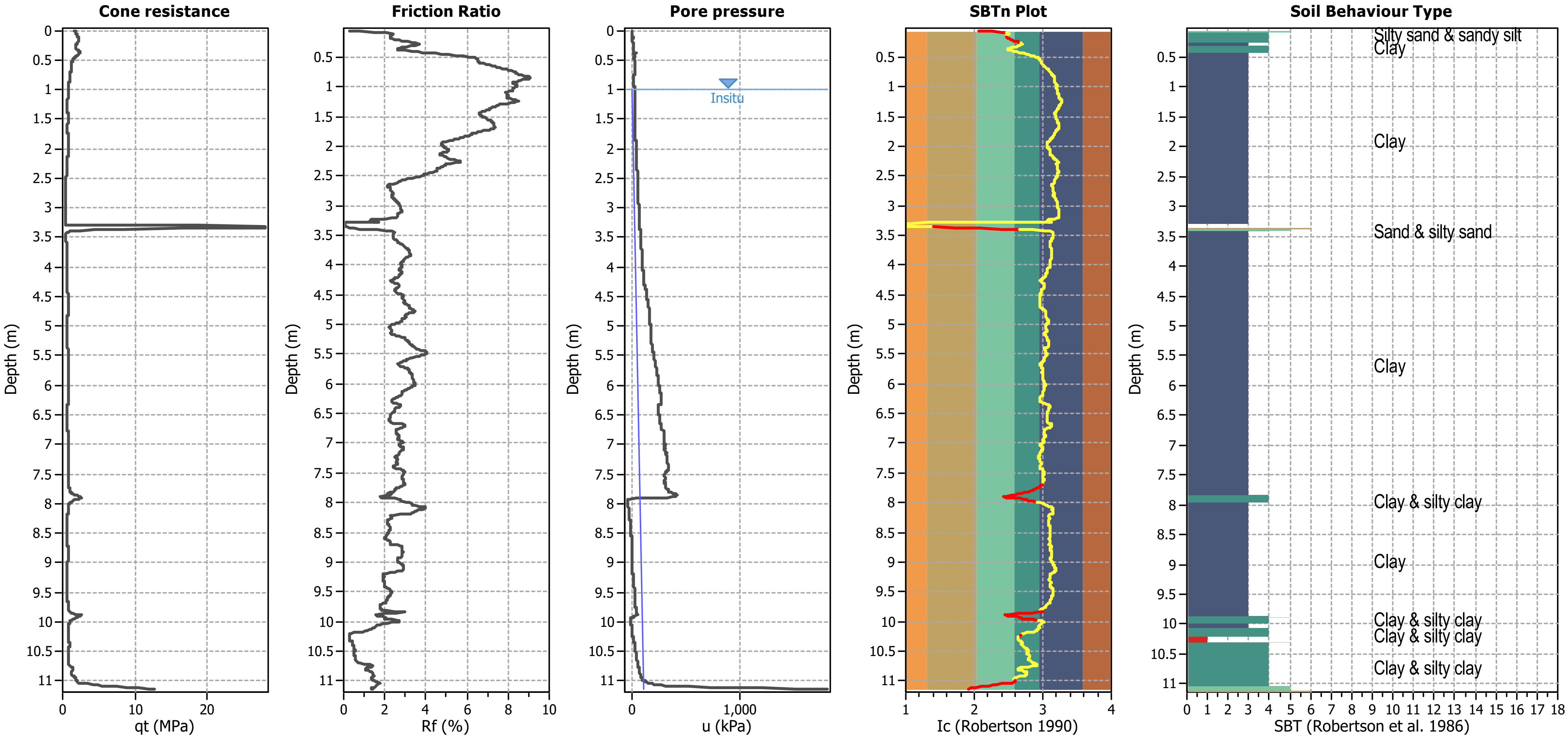
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT08
Total depth: 11.14 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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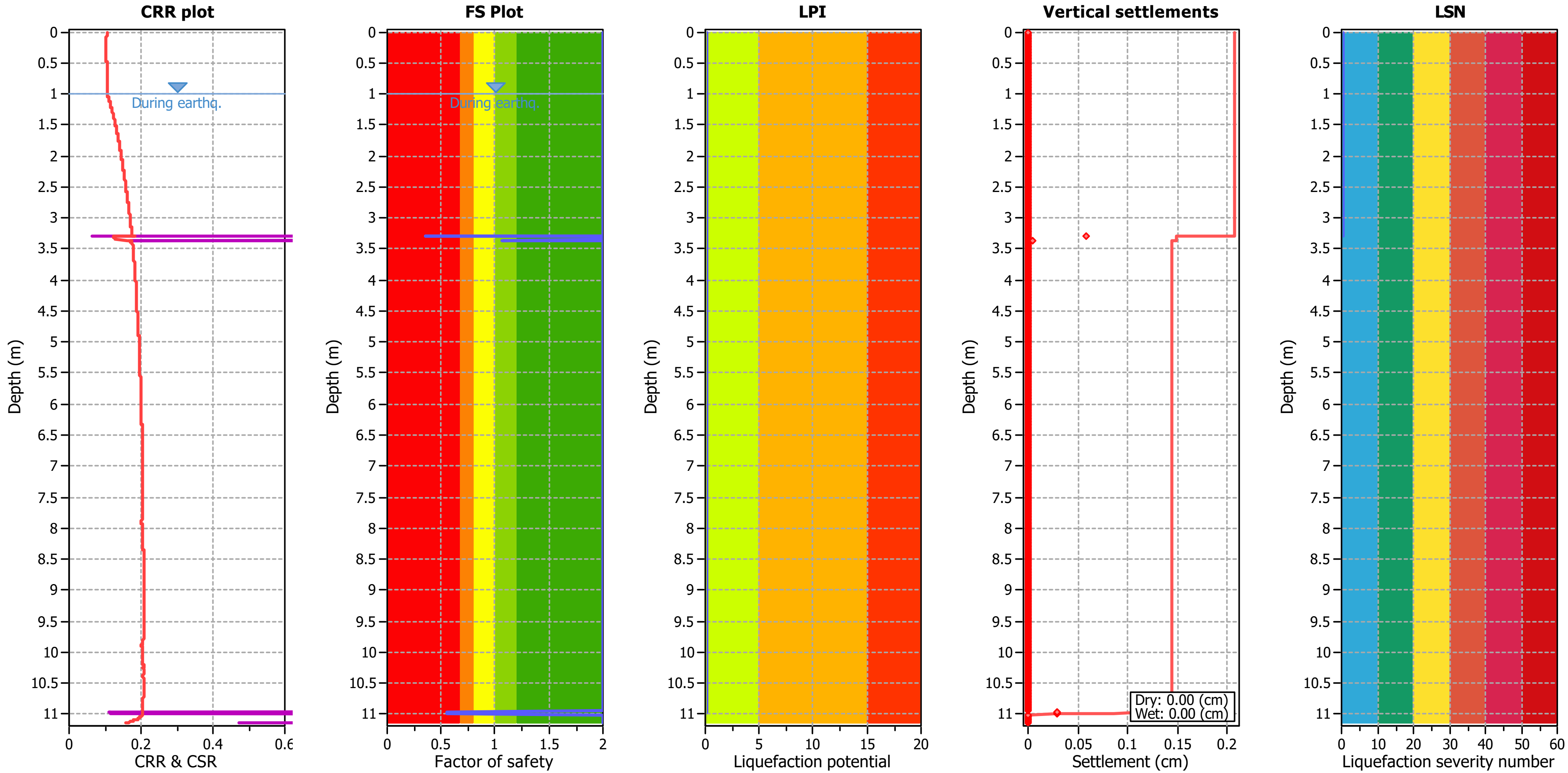
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT08
Total depth: 11.14 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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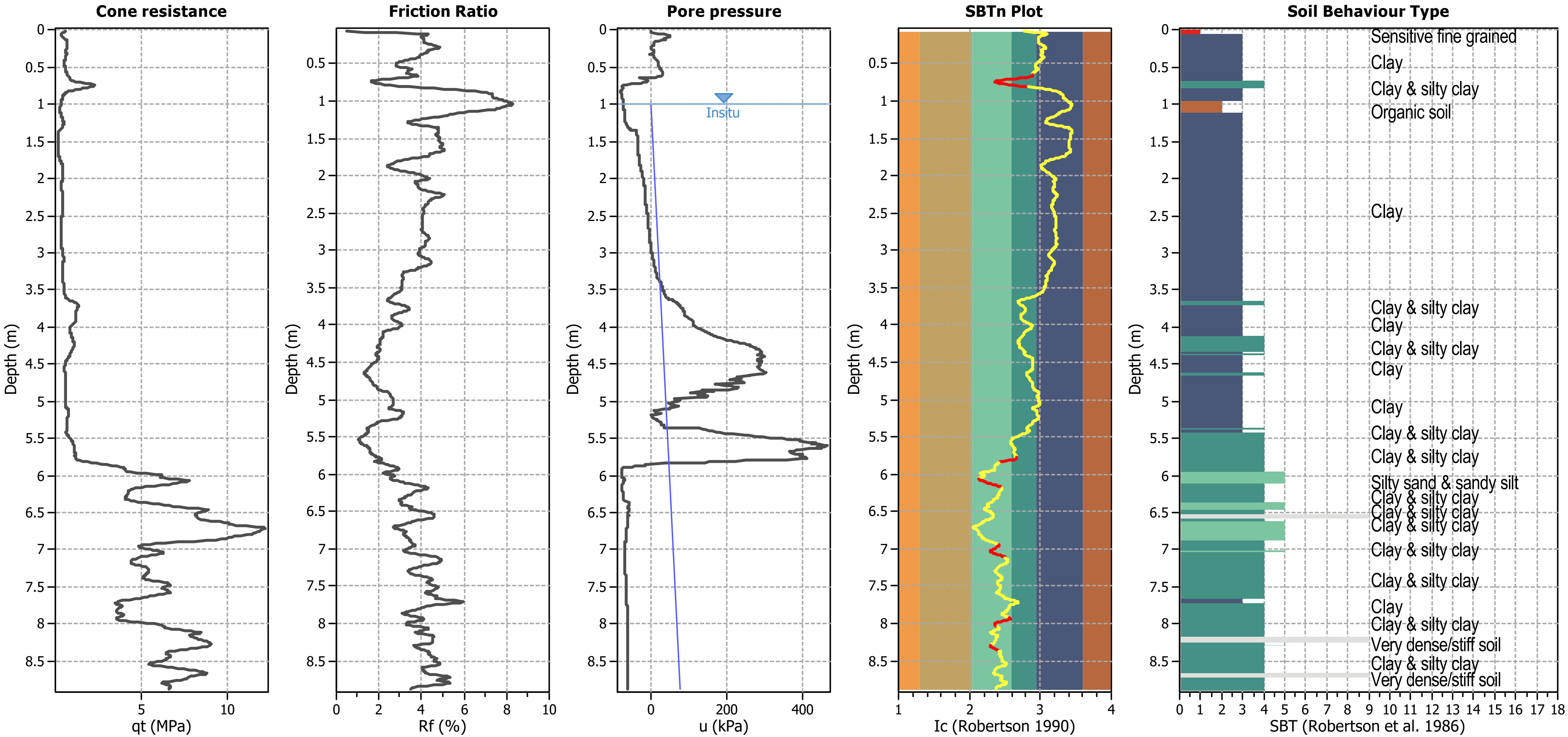
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT09
Total depth: 8.88 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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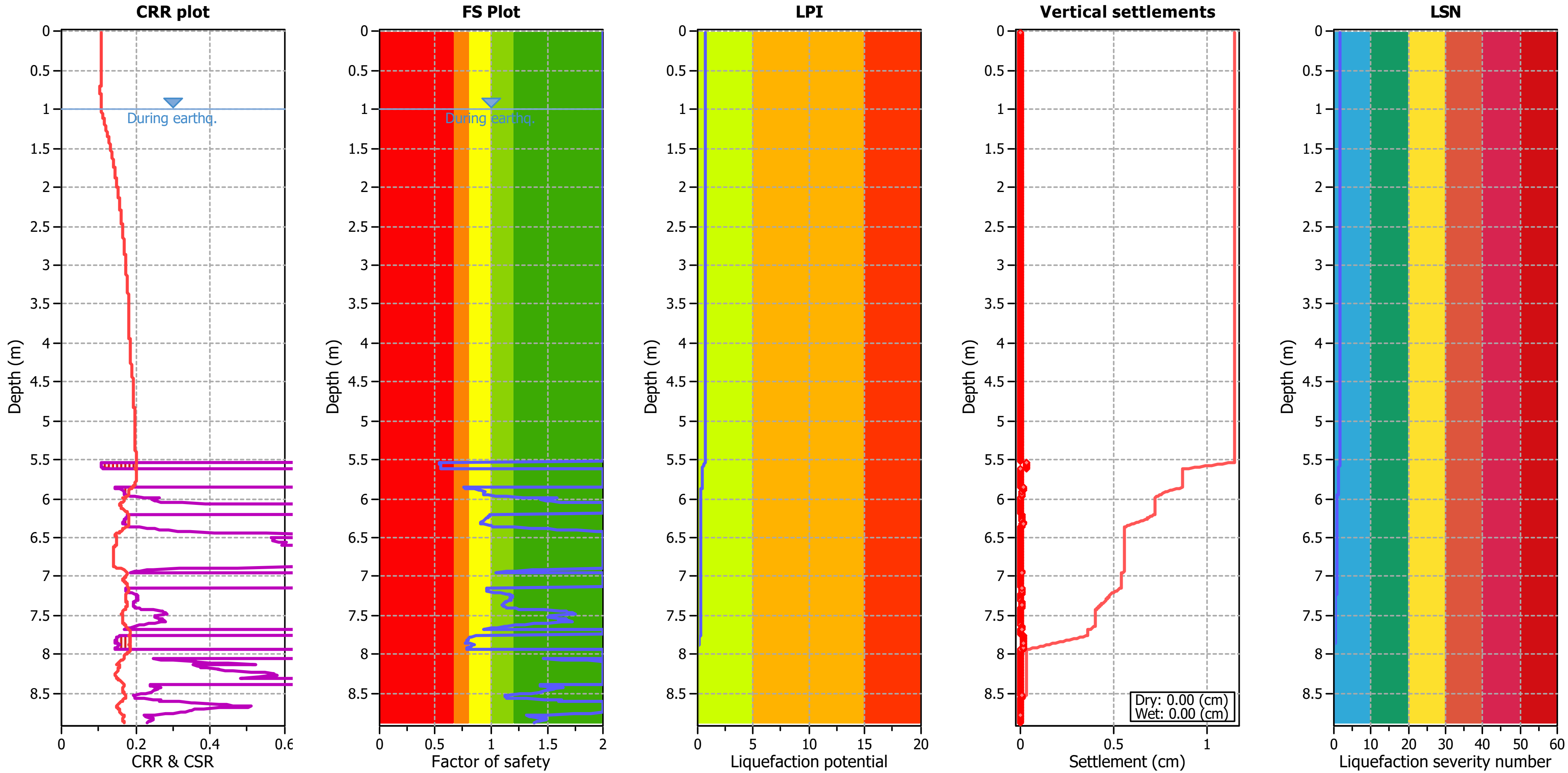
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT09
Total depth: 8.88 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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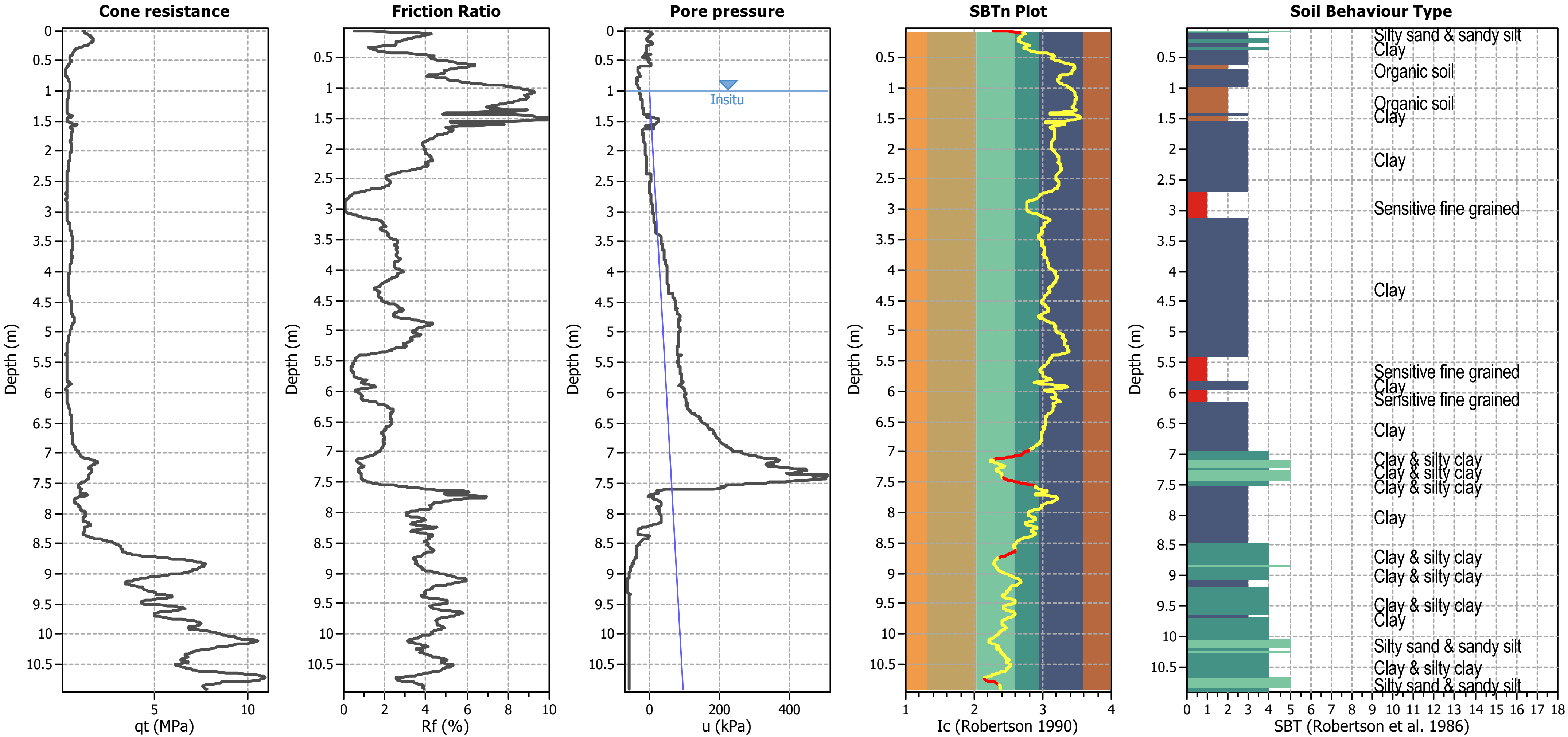
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT10
Total depth: 10.91 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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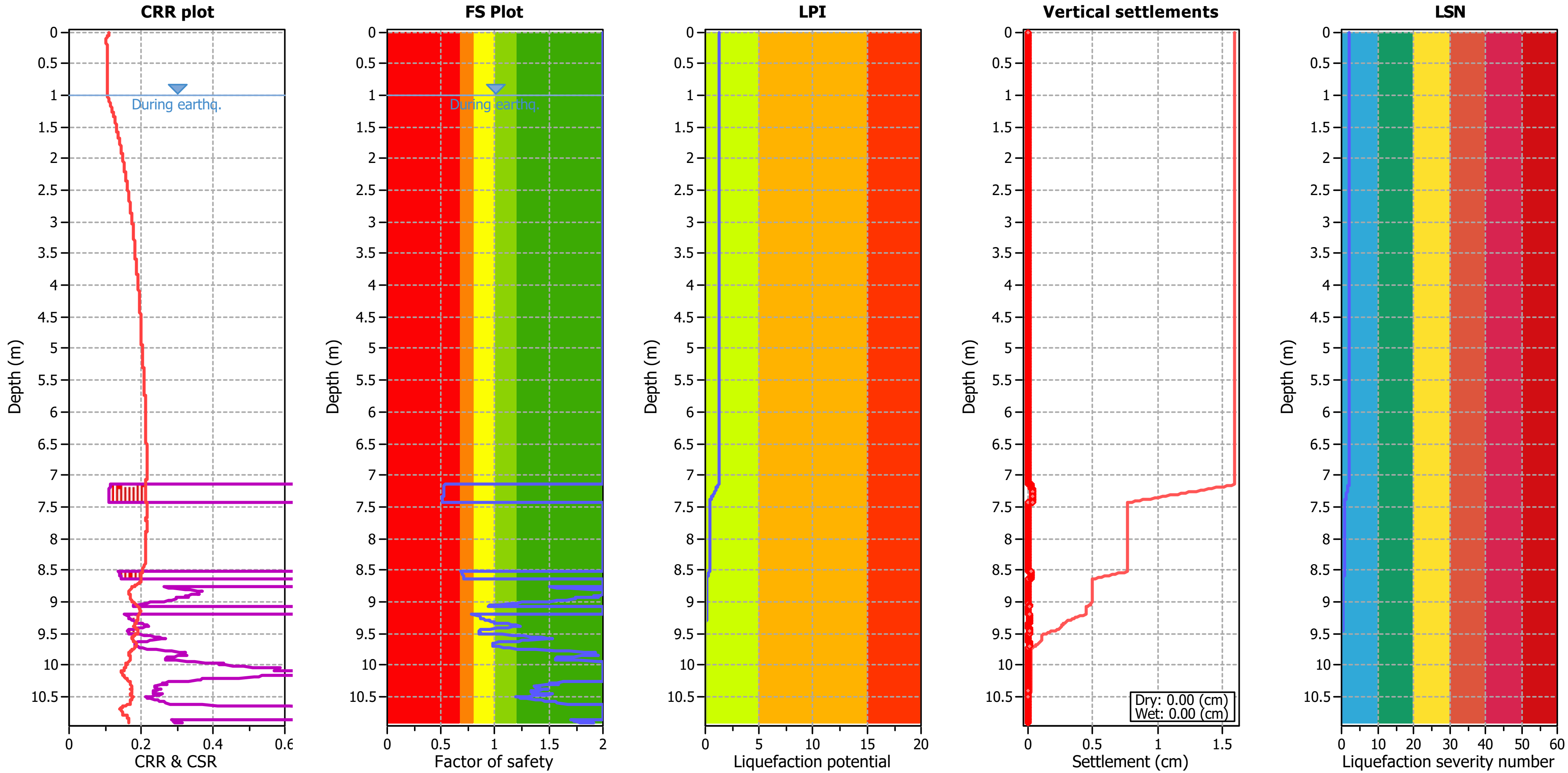
Project: **Auckland Surf Park** | **CPT: CPT10**
Location: **Dairy Flat, Auckland** | Total depth: 10.91 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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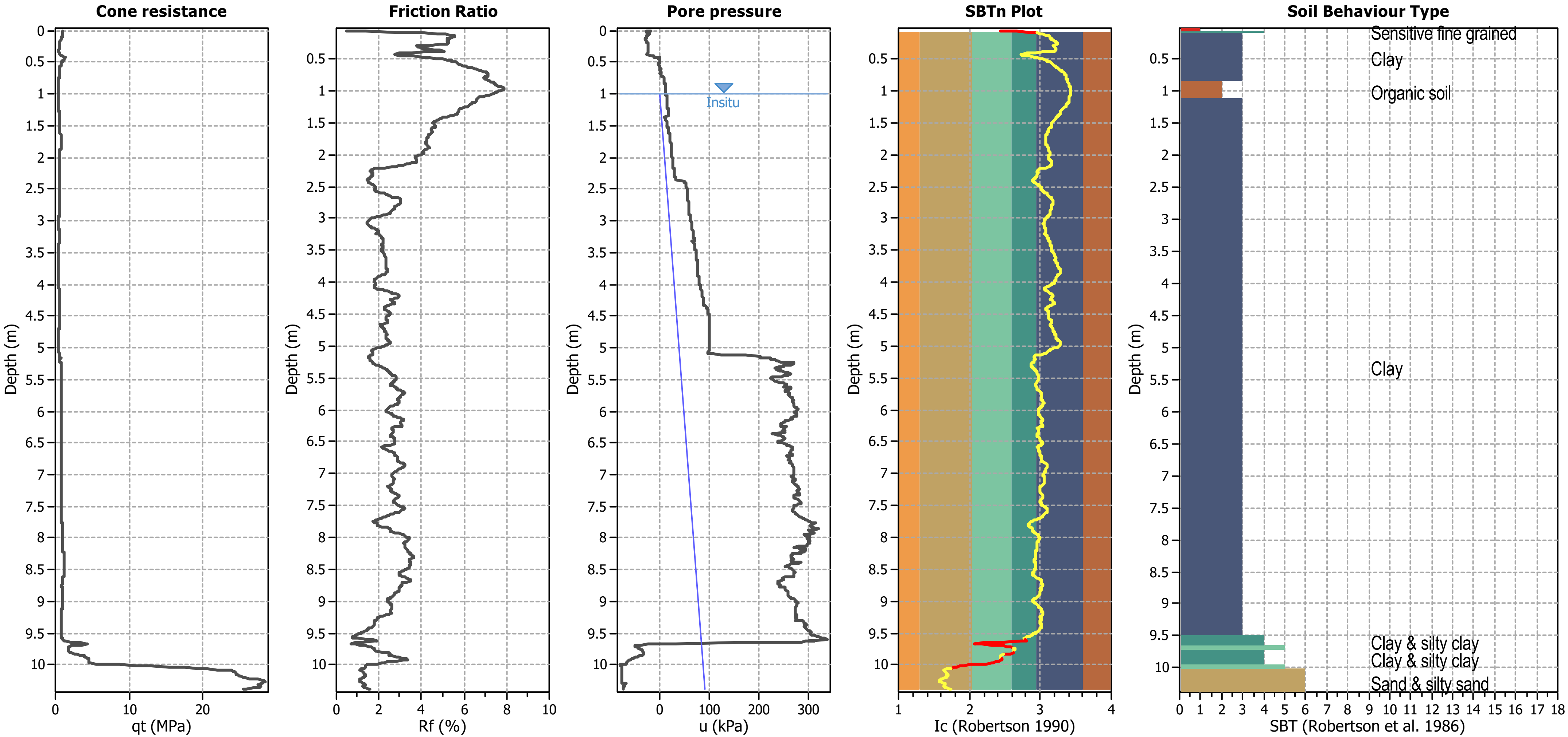
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT11
Total depth: 10.38 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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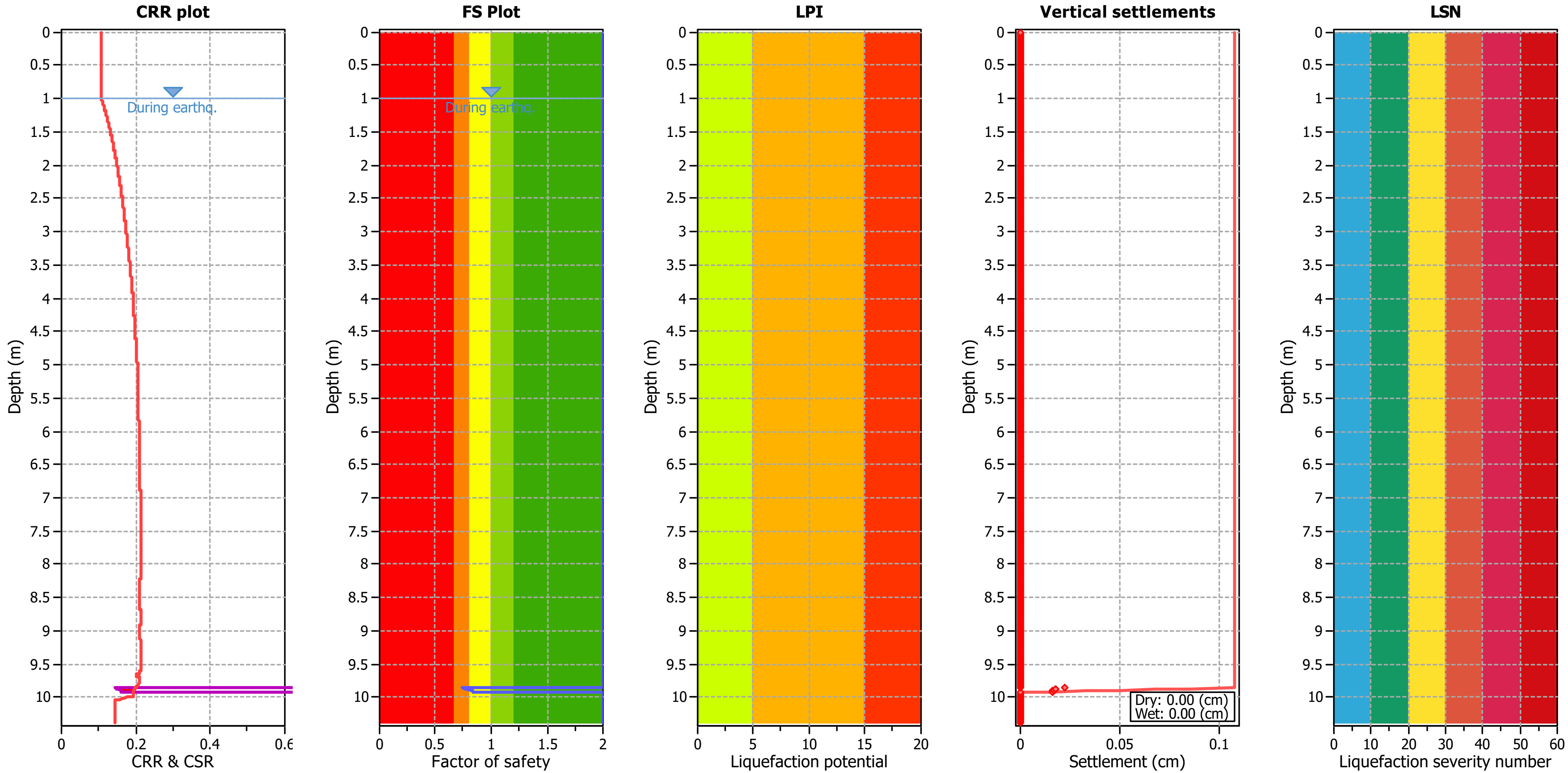
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT11
Total depth: 10.38 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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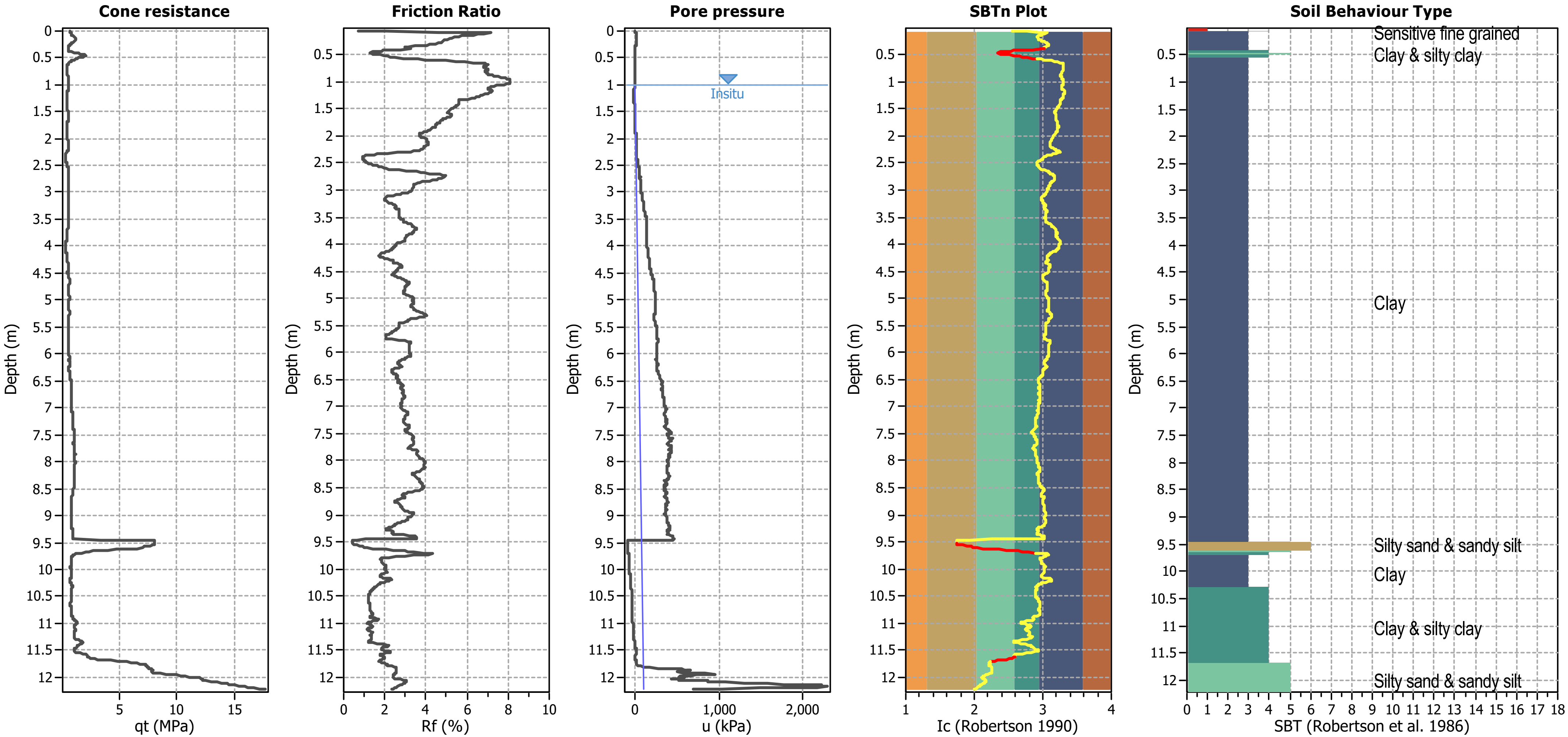
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT12
Total depth: 12.22 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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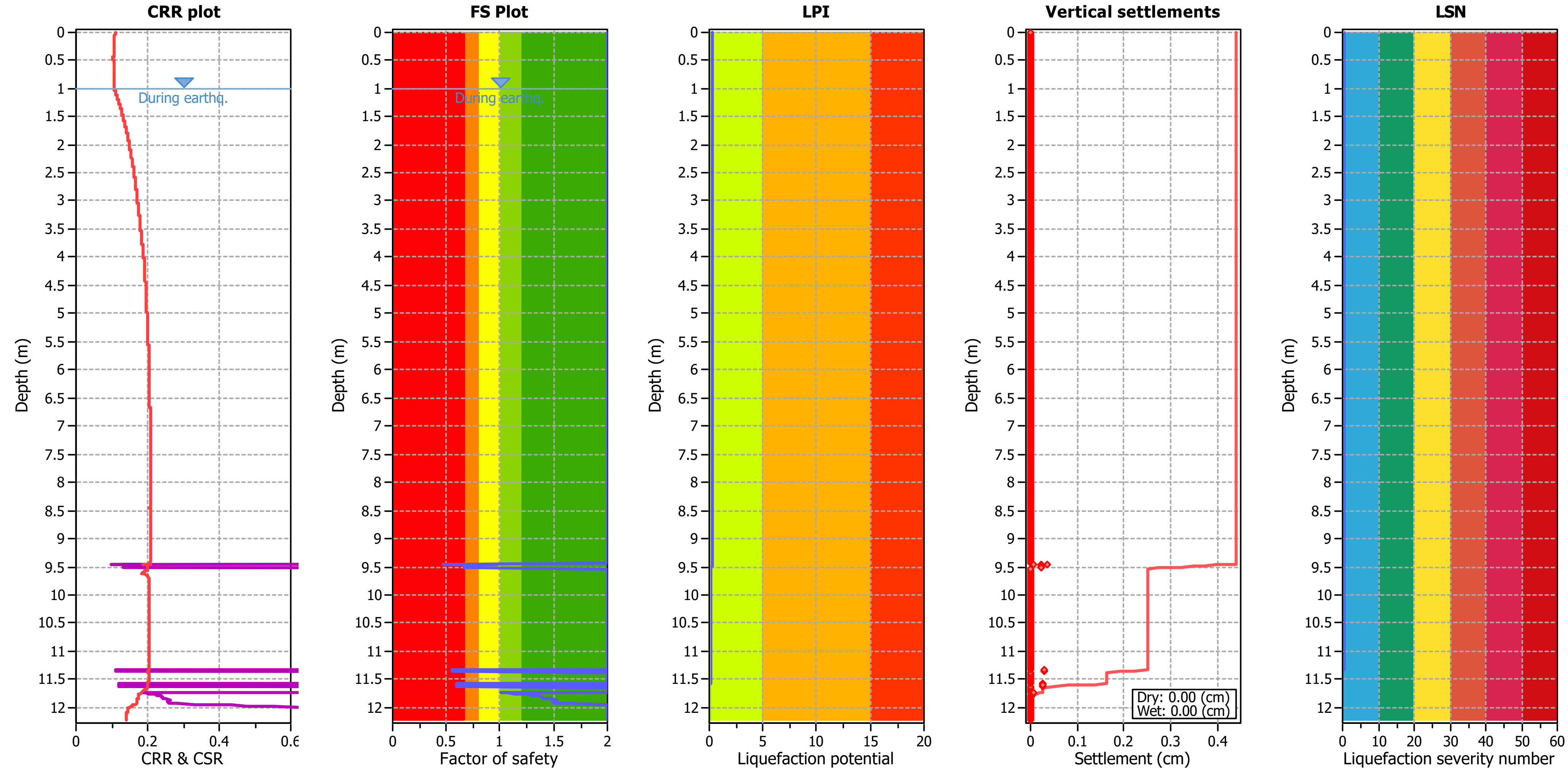
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

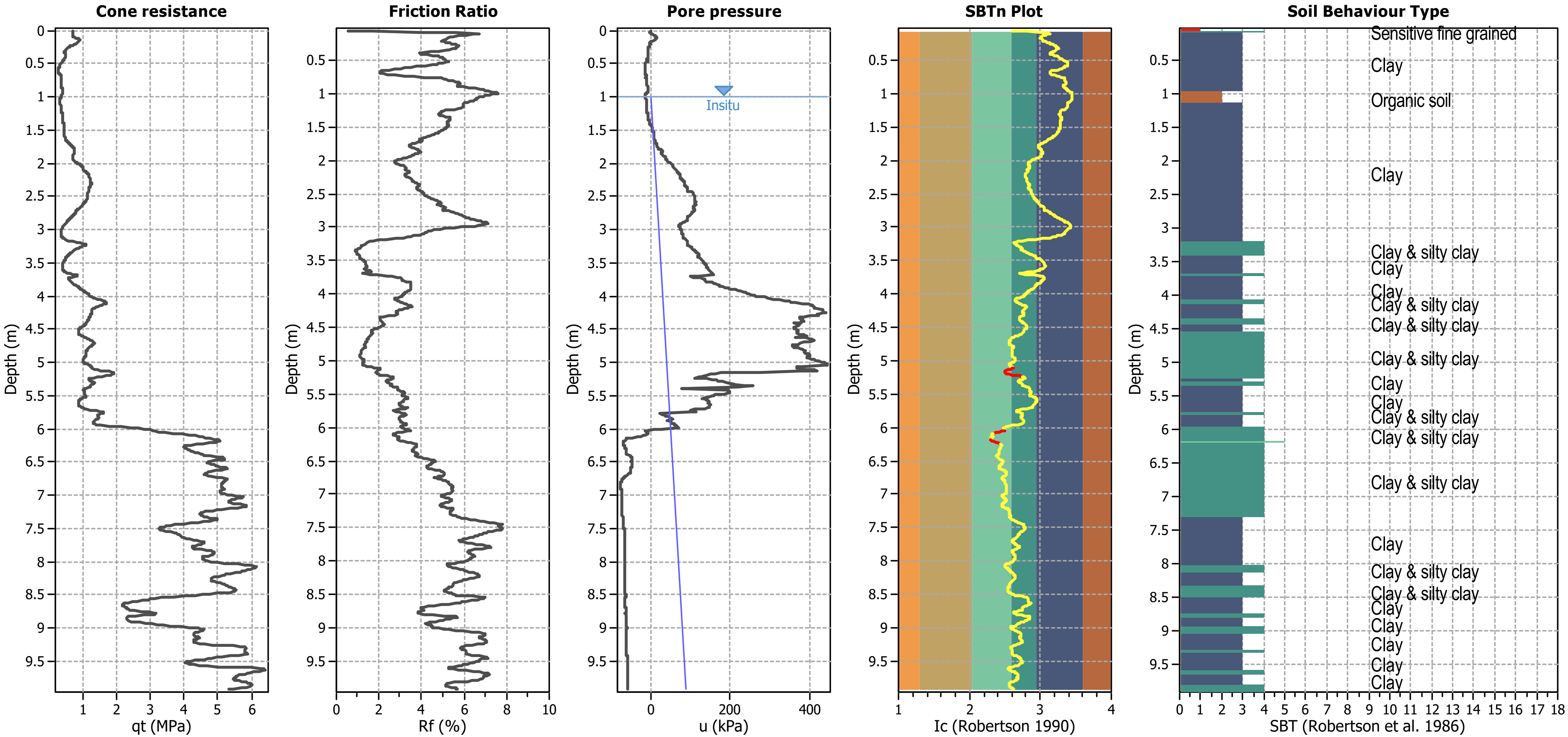
CPT: CPT12
Total depth: 12.22 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT13
Total depth: 9.92 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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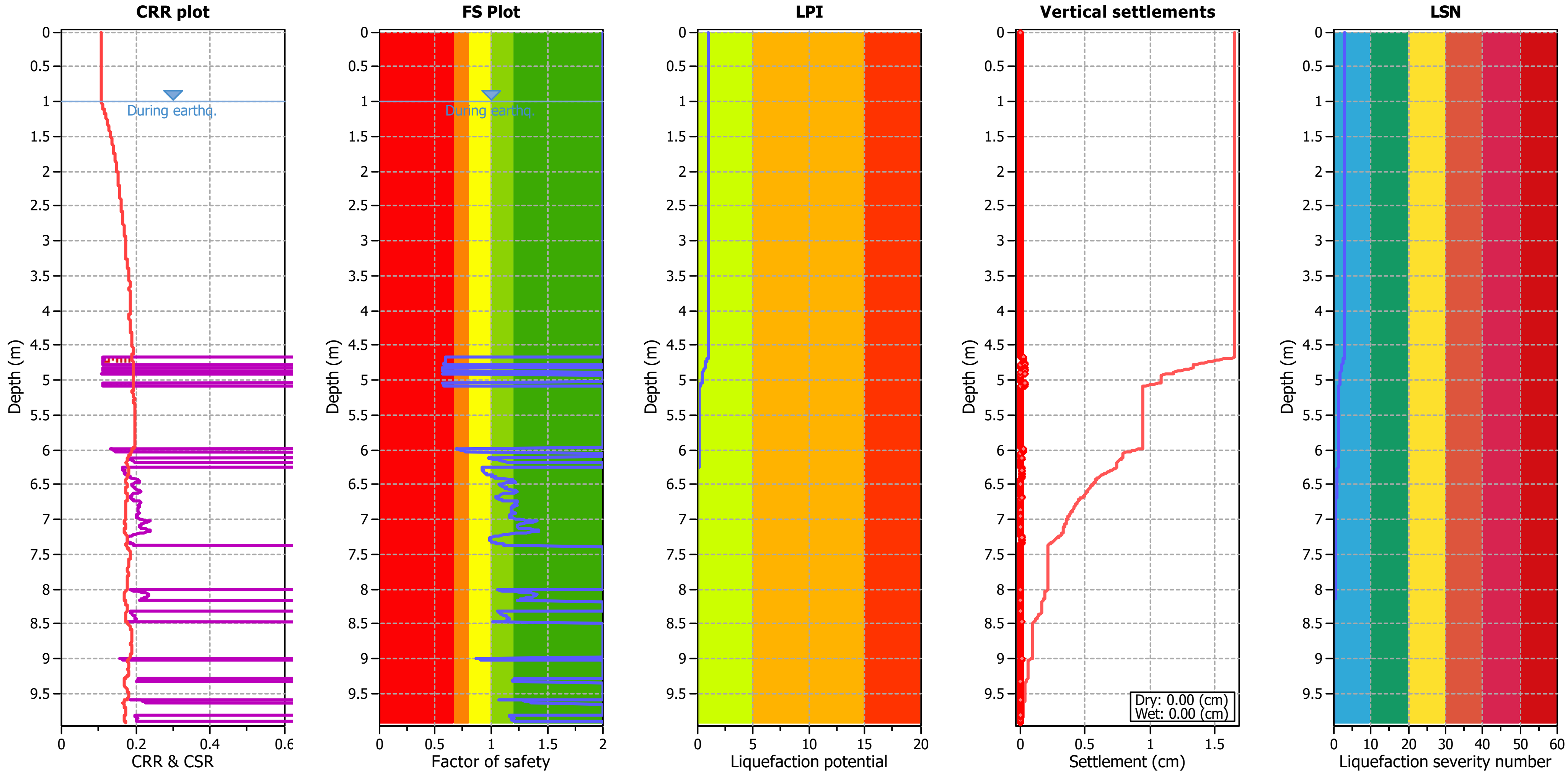
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT13
Total depth: 9.92 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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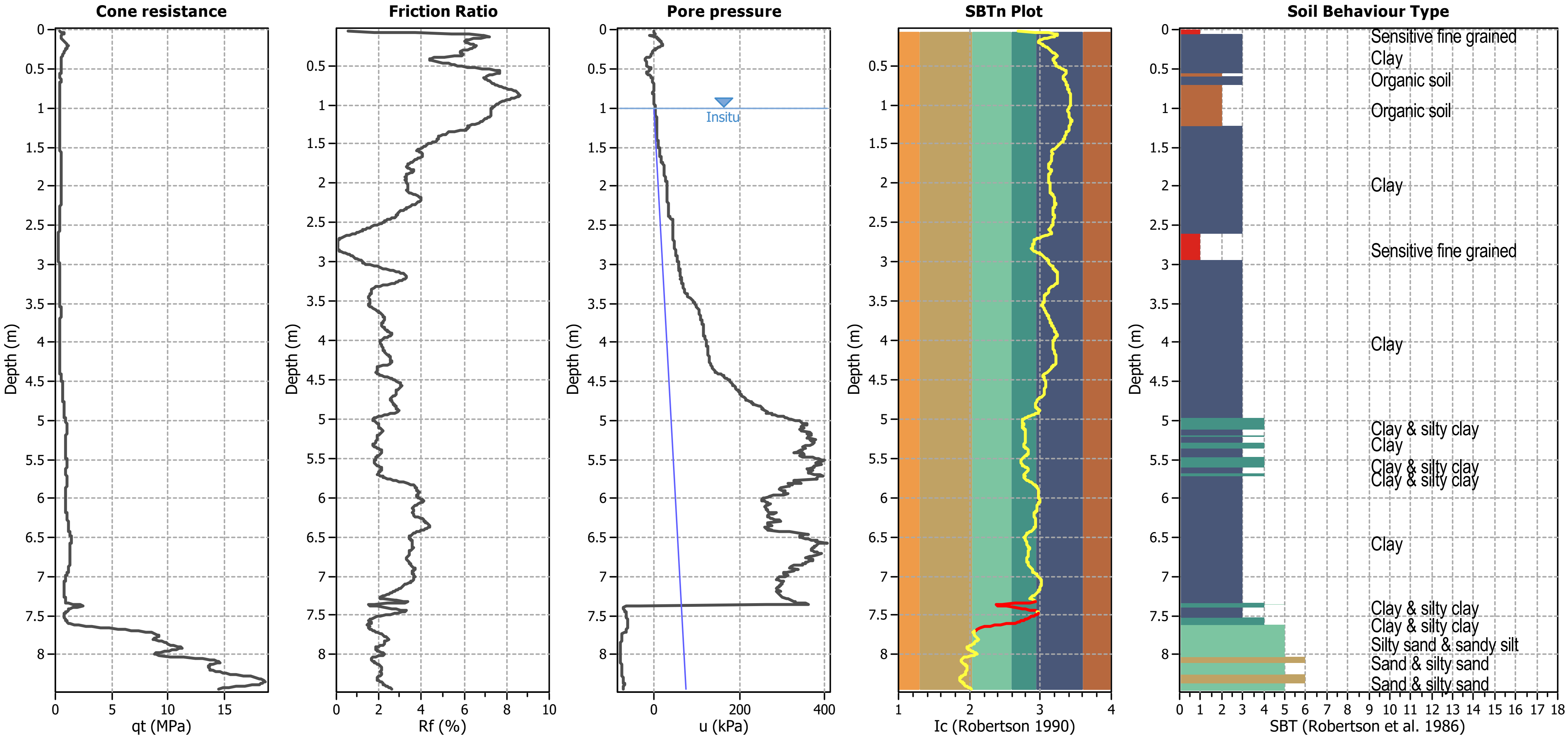
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT14
Total depth: 8.44 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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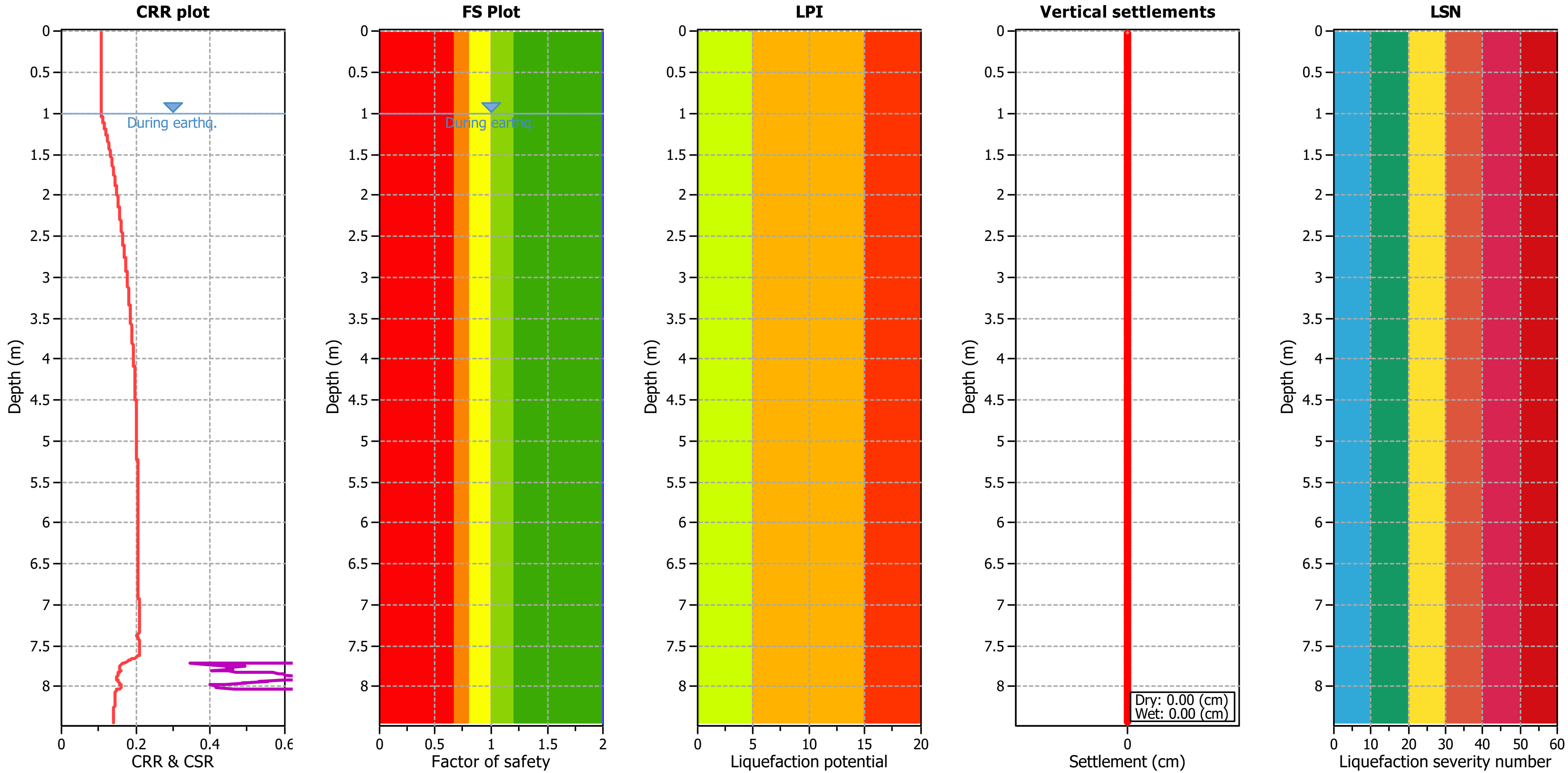
Project: **Auckland Surf Park** | **CPT: CPT14**

Location: **Dairy Flat, Auckland** | Total depth: 8.44 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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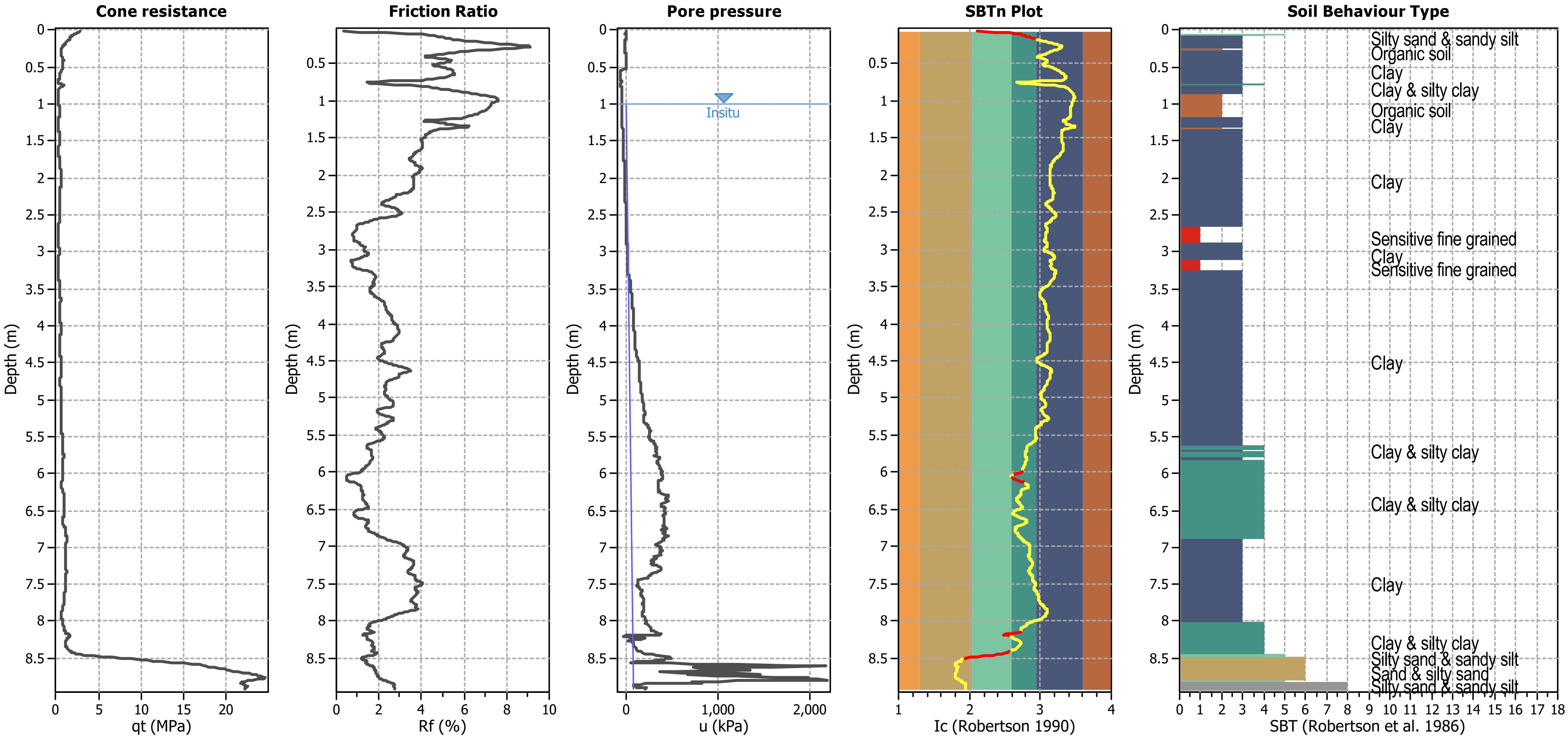
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT15
Total depth: 8.92 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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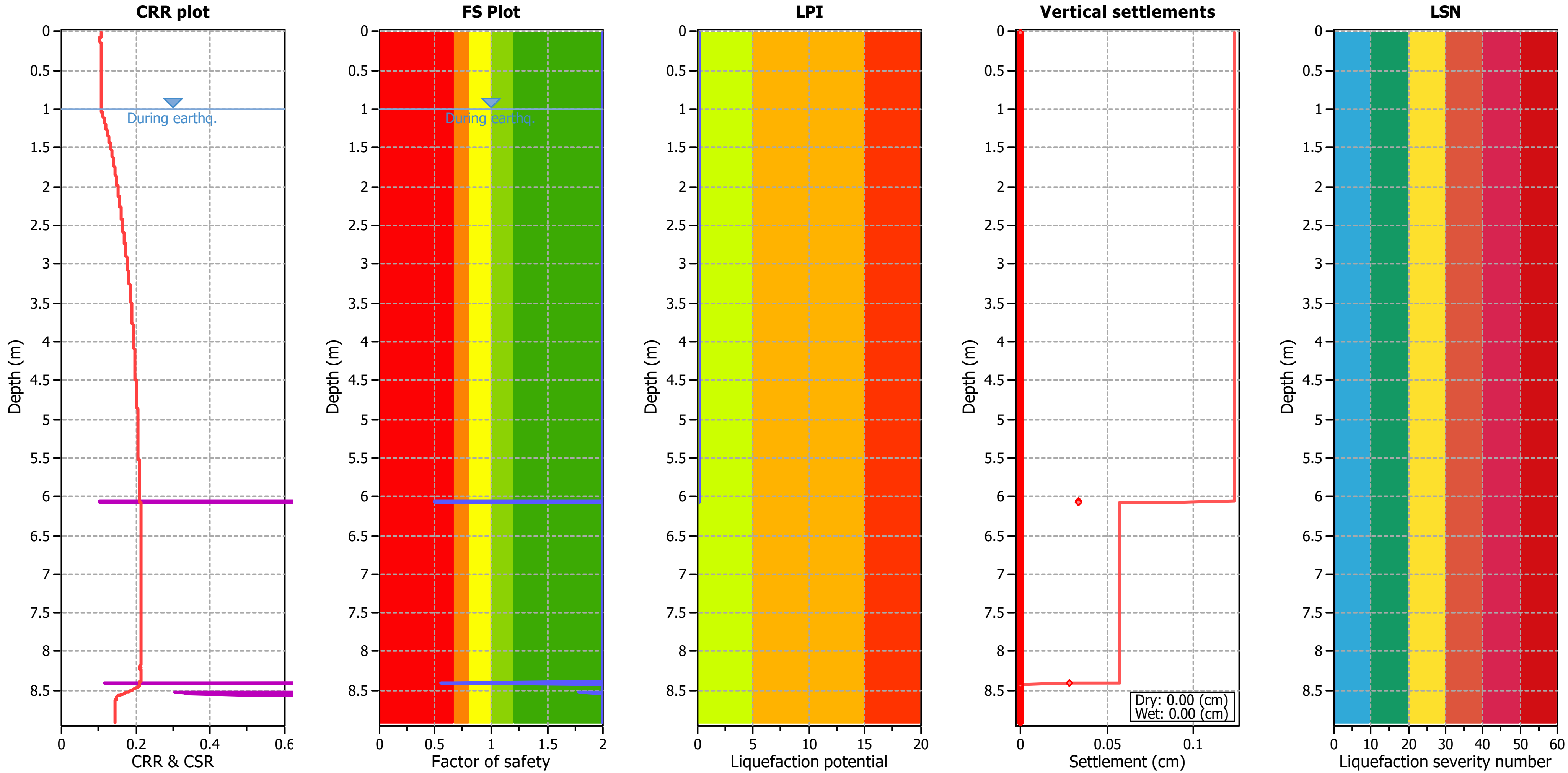
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT15
Total depth: 8.92 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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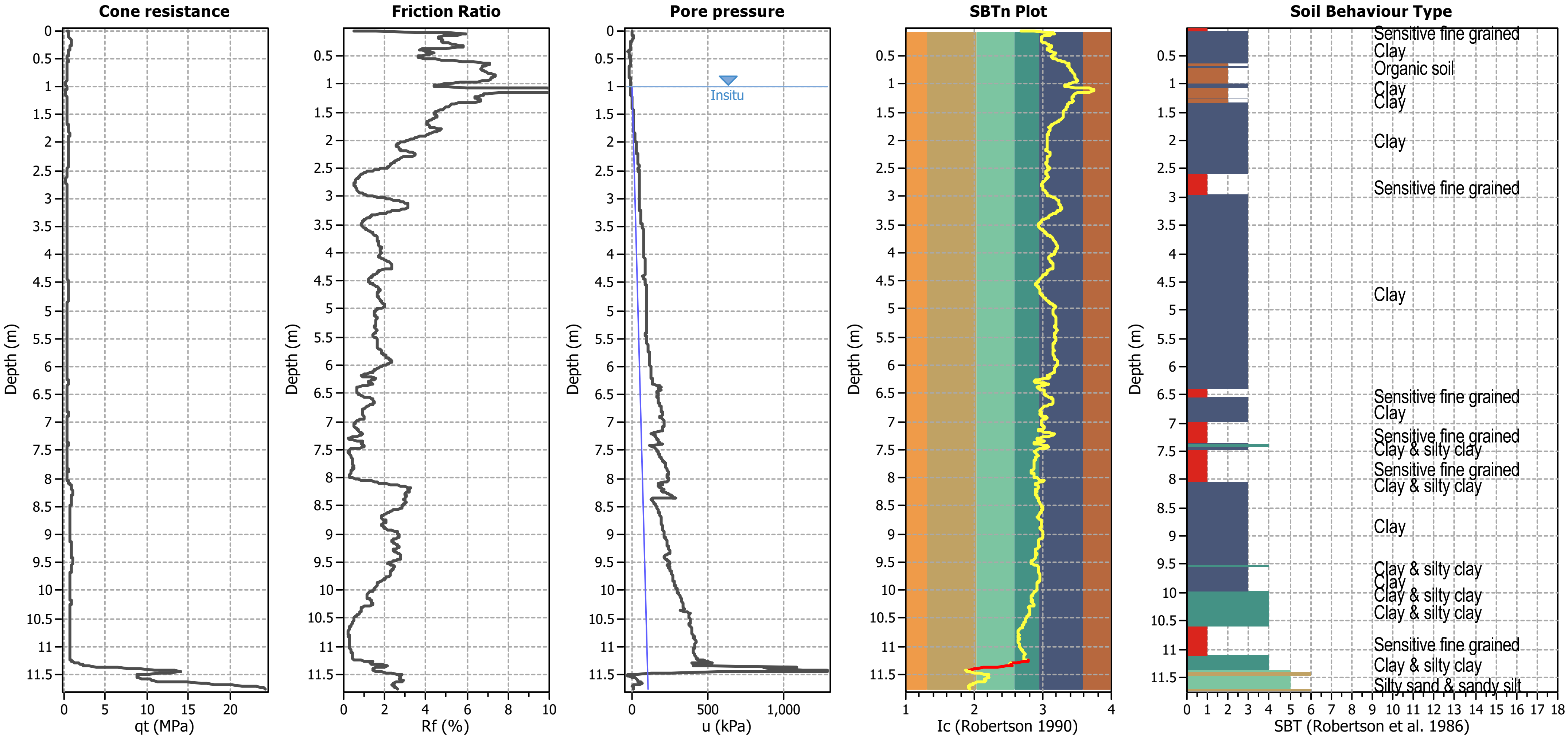
Project: Auckland Surf Park **CPT: CPT16**
Location: Dairy Flat, Auckland Total depth: 11.76 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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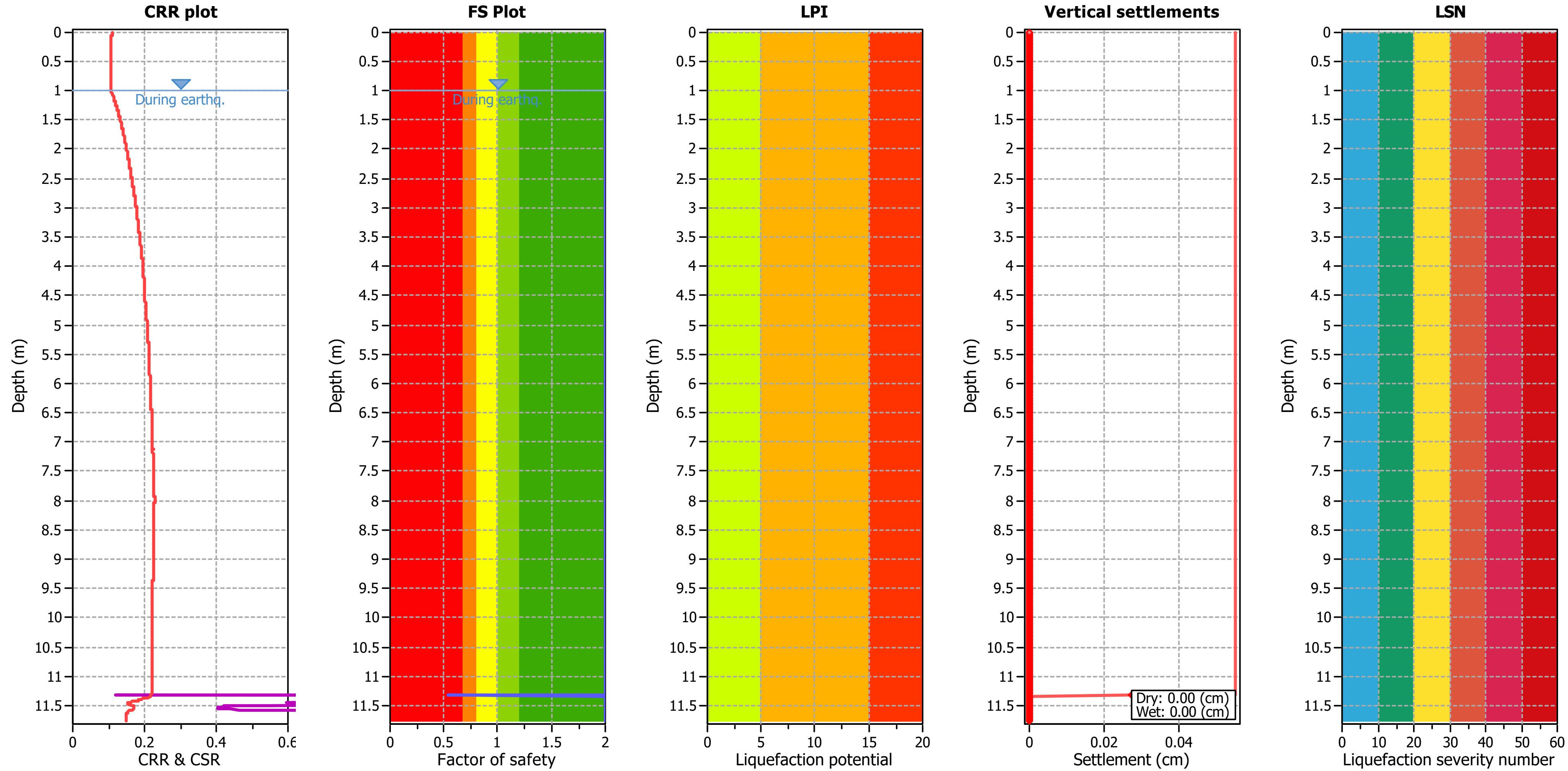
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT16
Total depth: 11.76 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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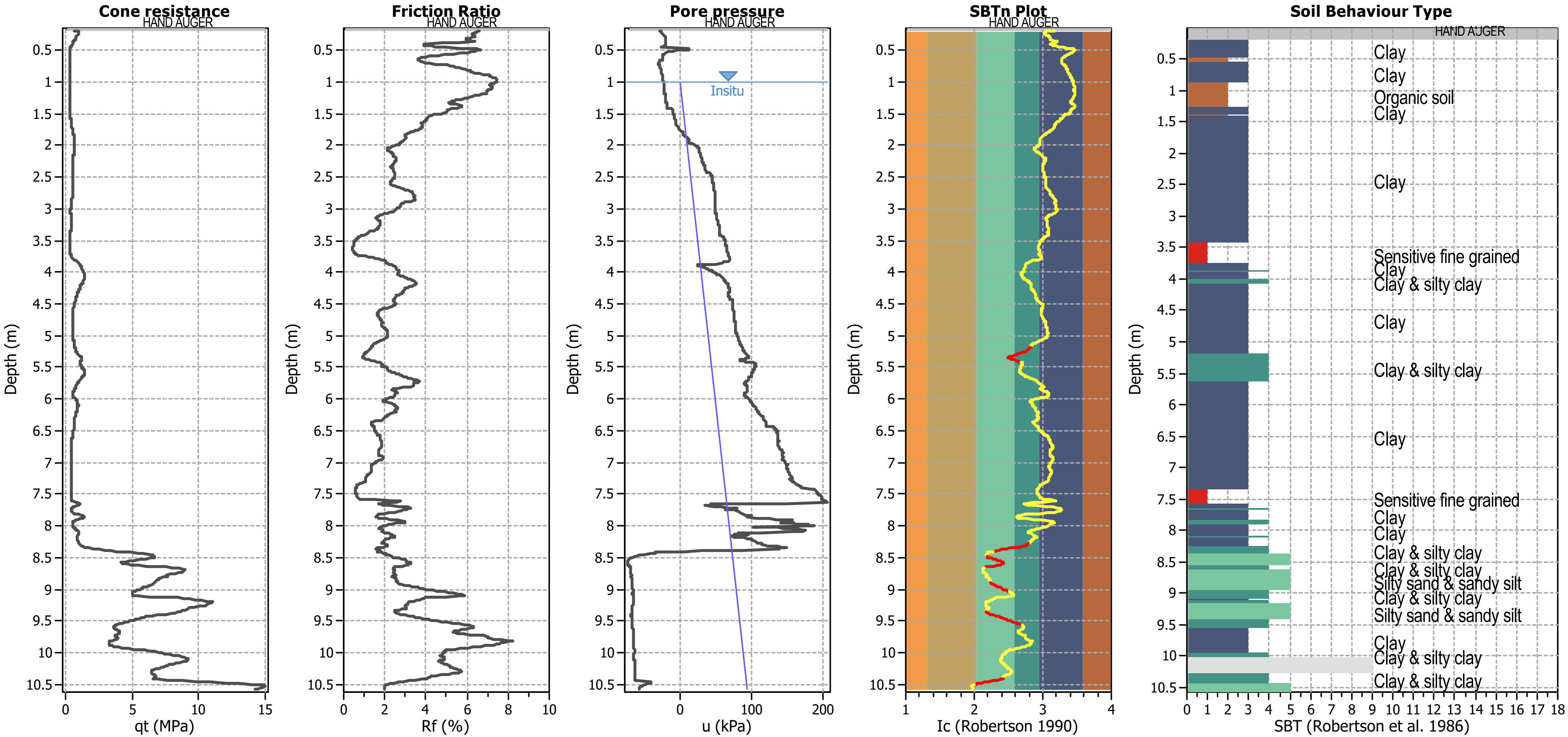
Project: **Auckland Surf Park** | **CPT: CPT17**

Location: **Dairy Flat, Auckland** | Total depth: 10.57 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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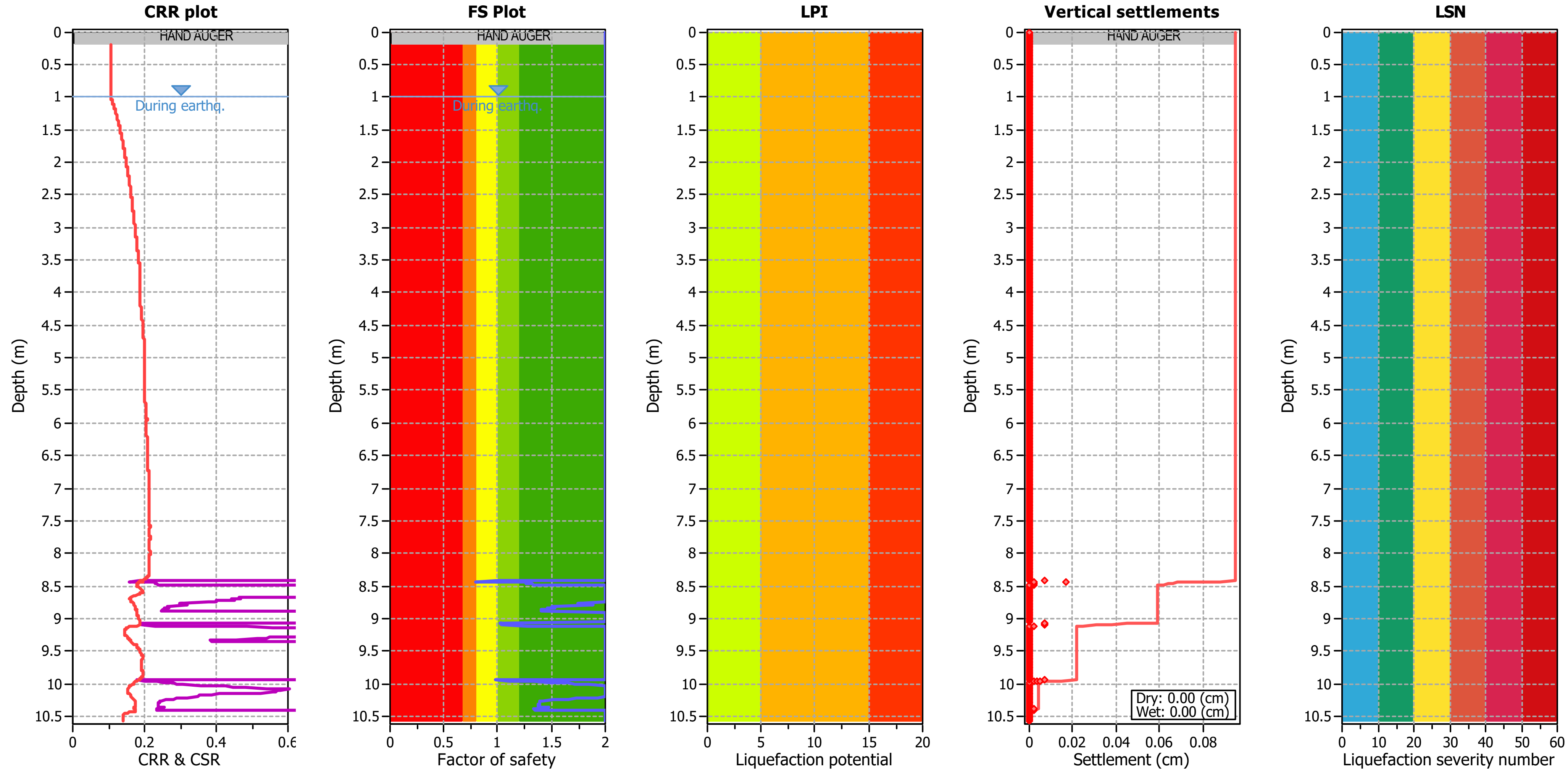
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT17
Total depth: 10.57 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior	
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A	applied:	.
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

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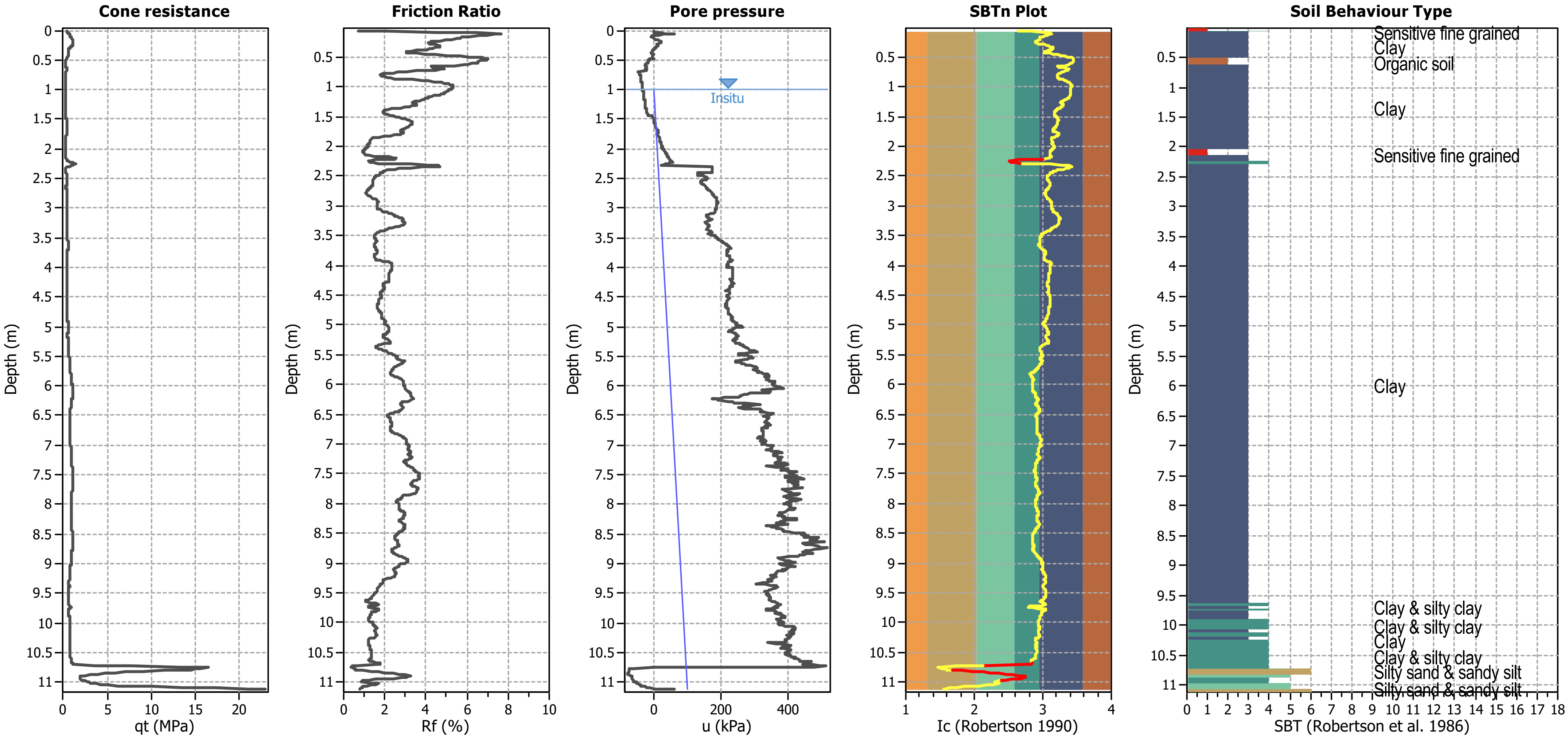
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

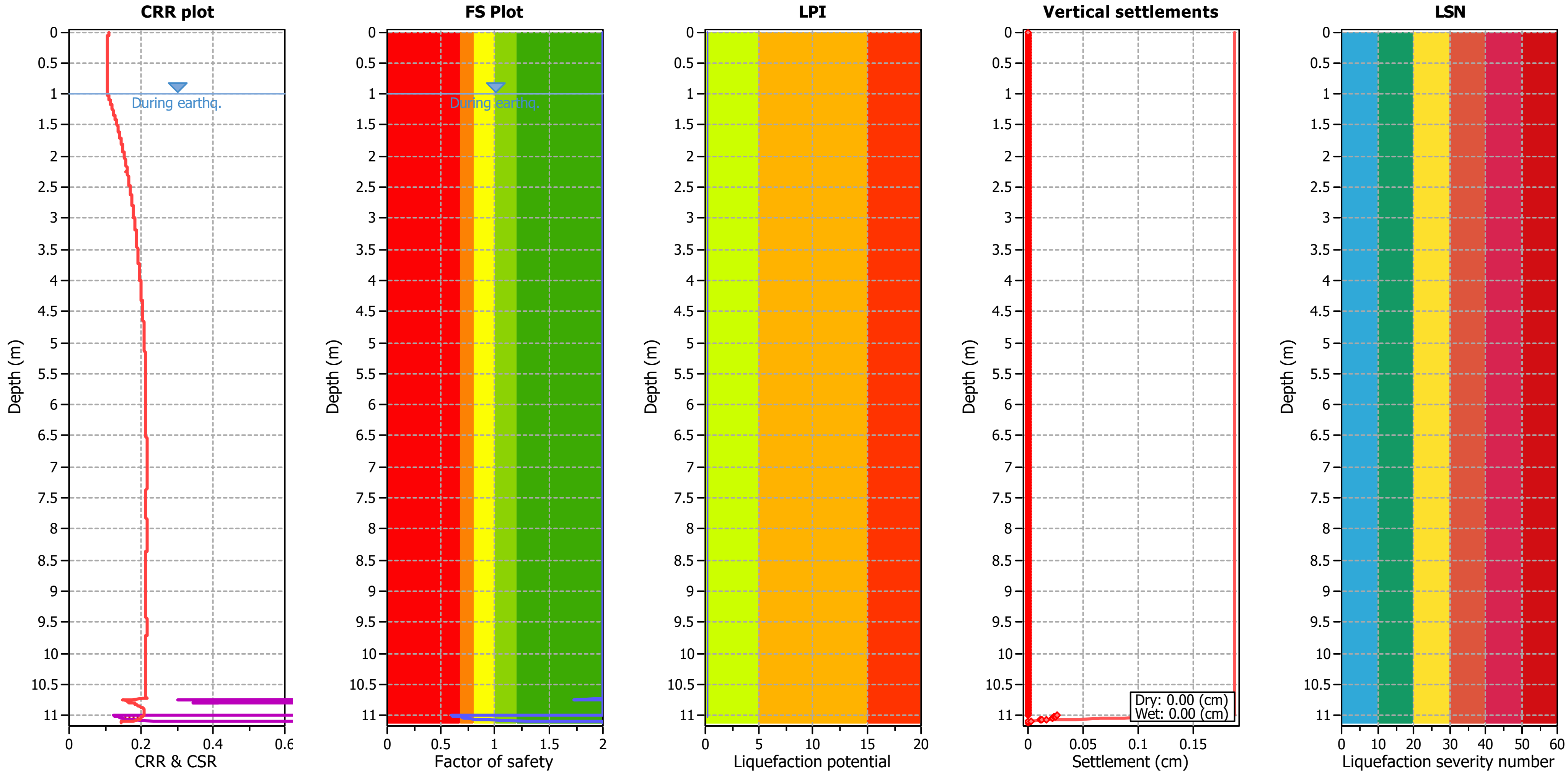
CPT: CPT18
Total depth: 11.12 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT18
Total depth: 11.12 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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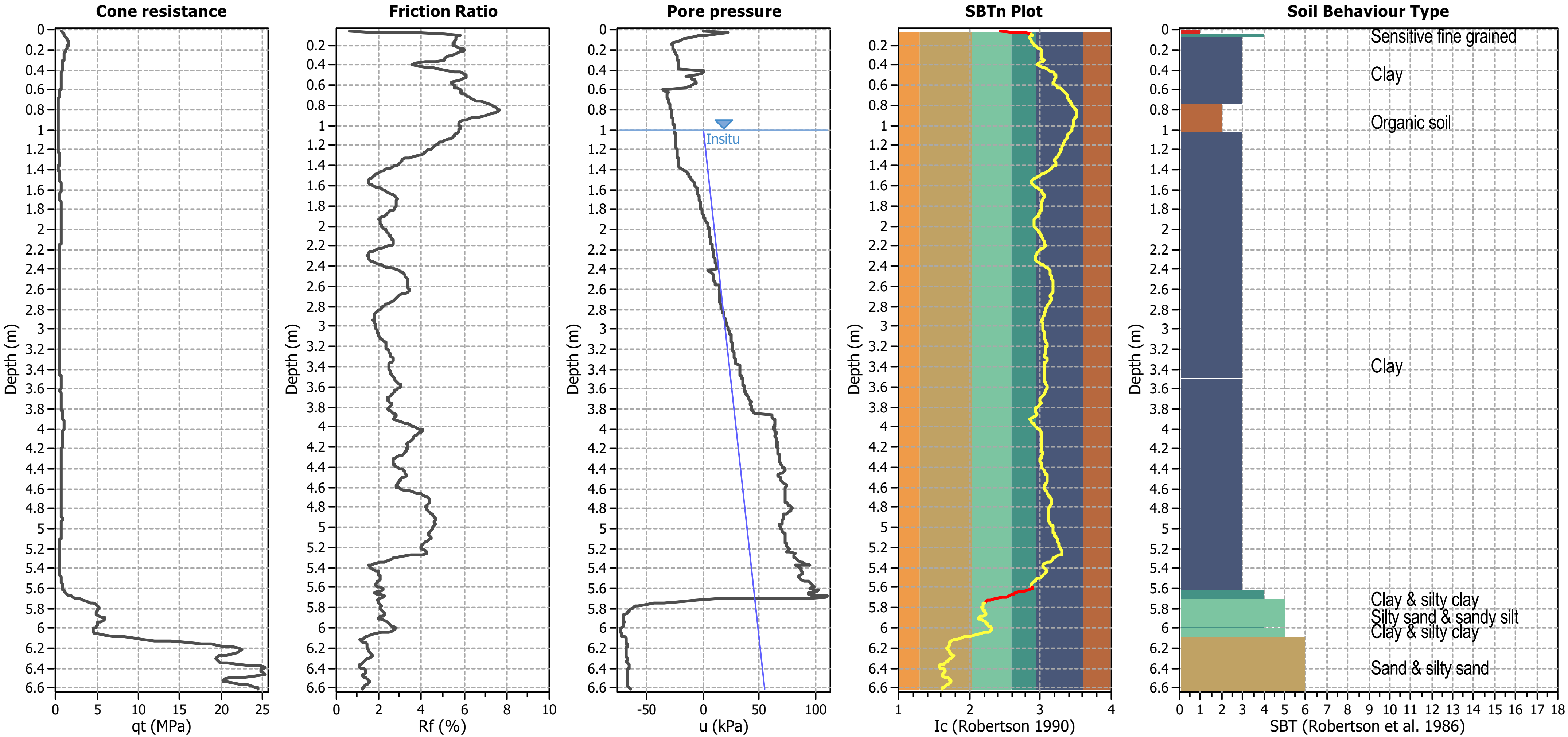
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT19
Total depth: 6.61 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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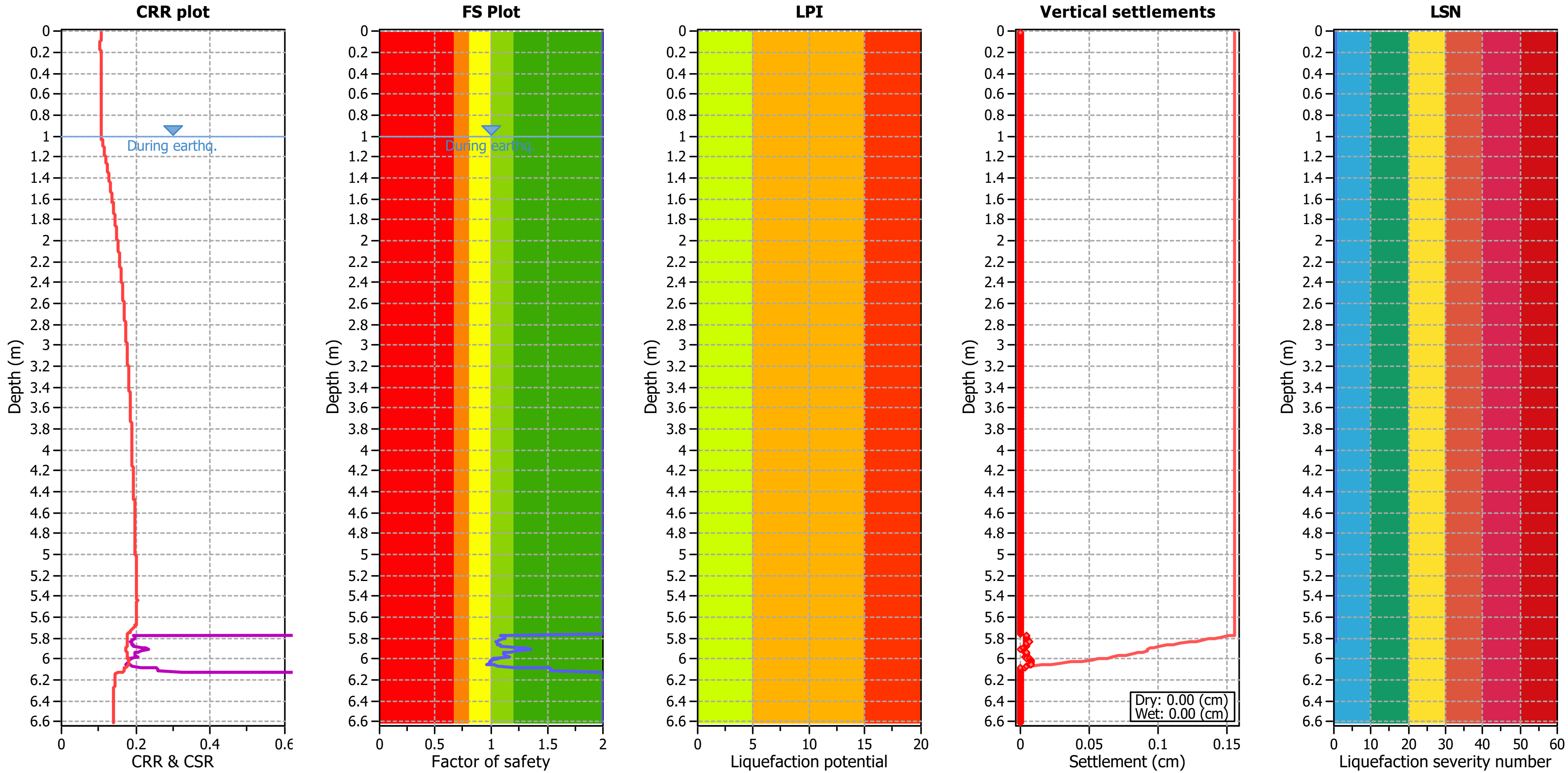
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT19
Total depth: 6.61 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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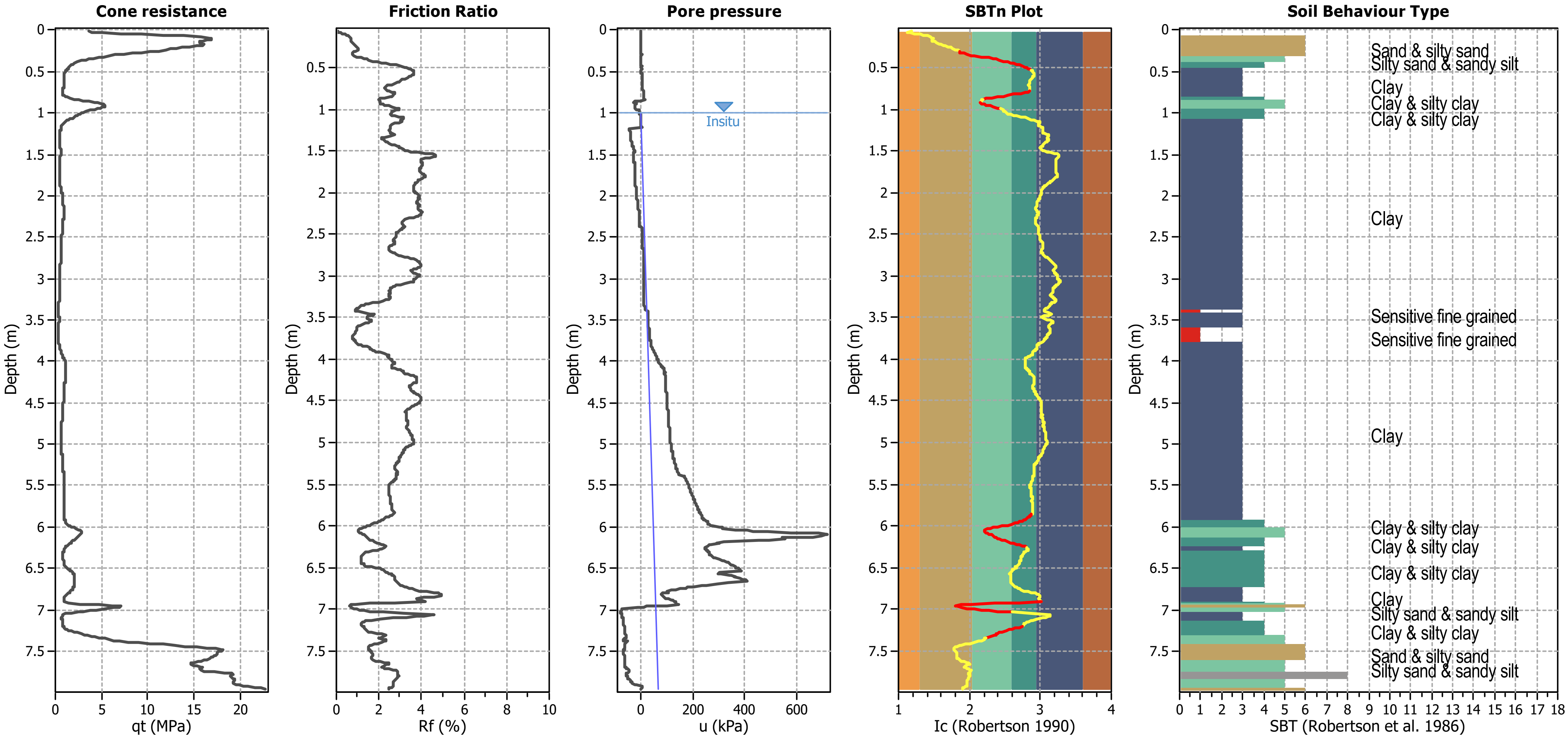
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT20
Total depth: 7.96 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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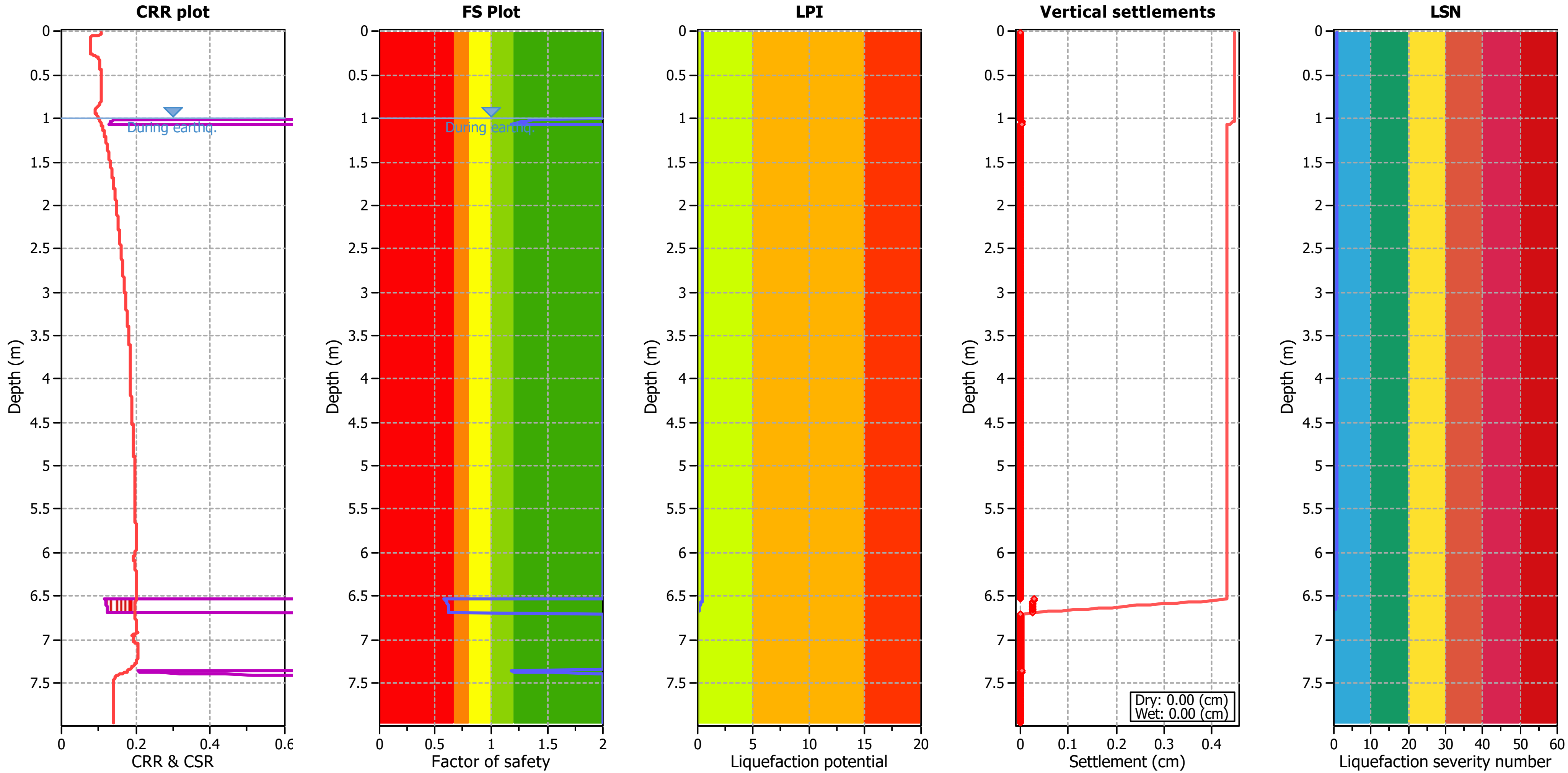
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT20
Total depth: 7.96 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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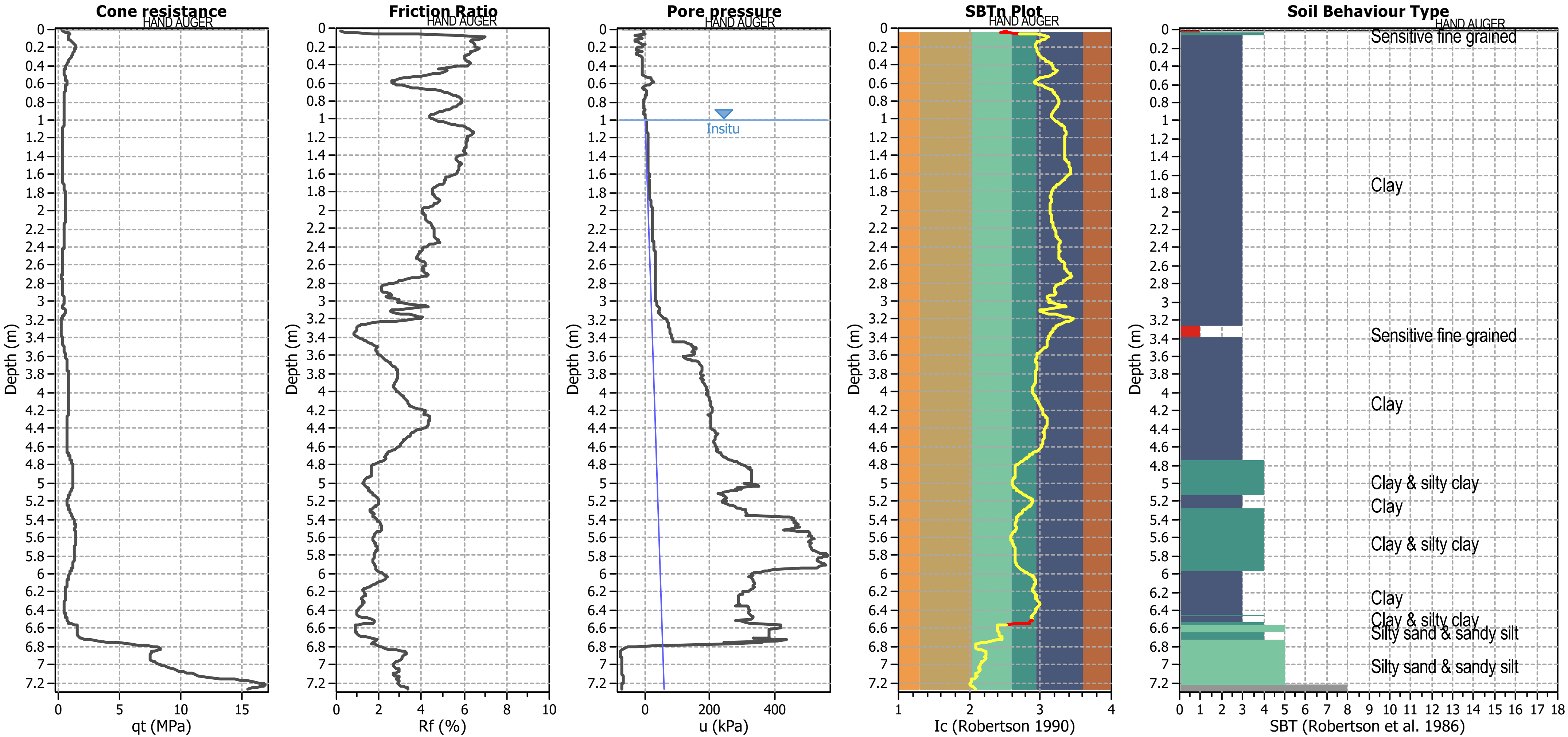
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT21
Total depth: 7.27 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

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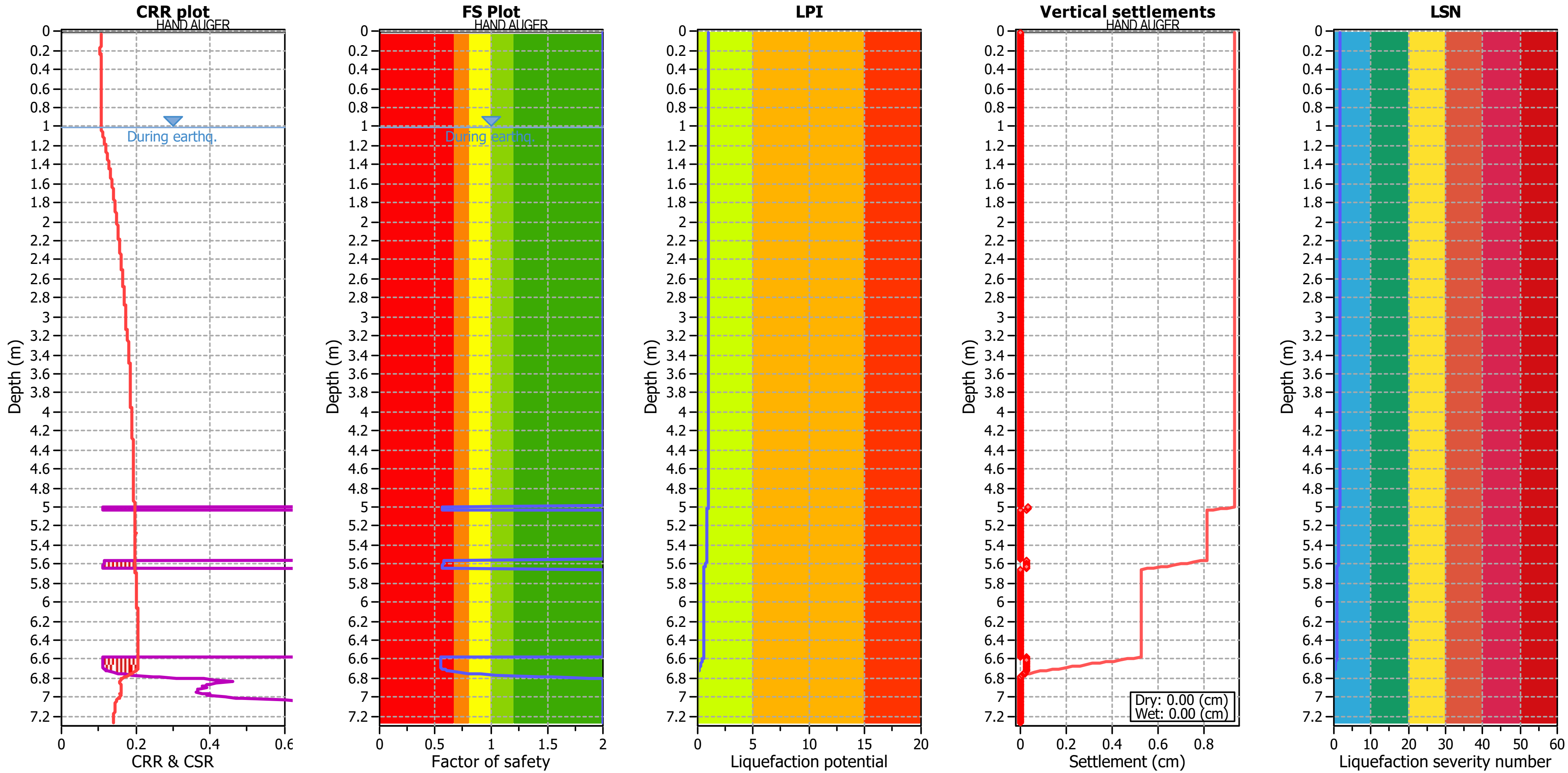
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT21
Total depth: 7.27 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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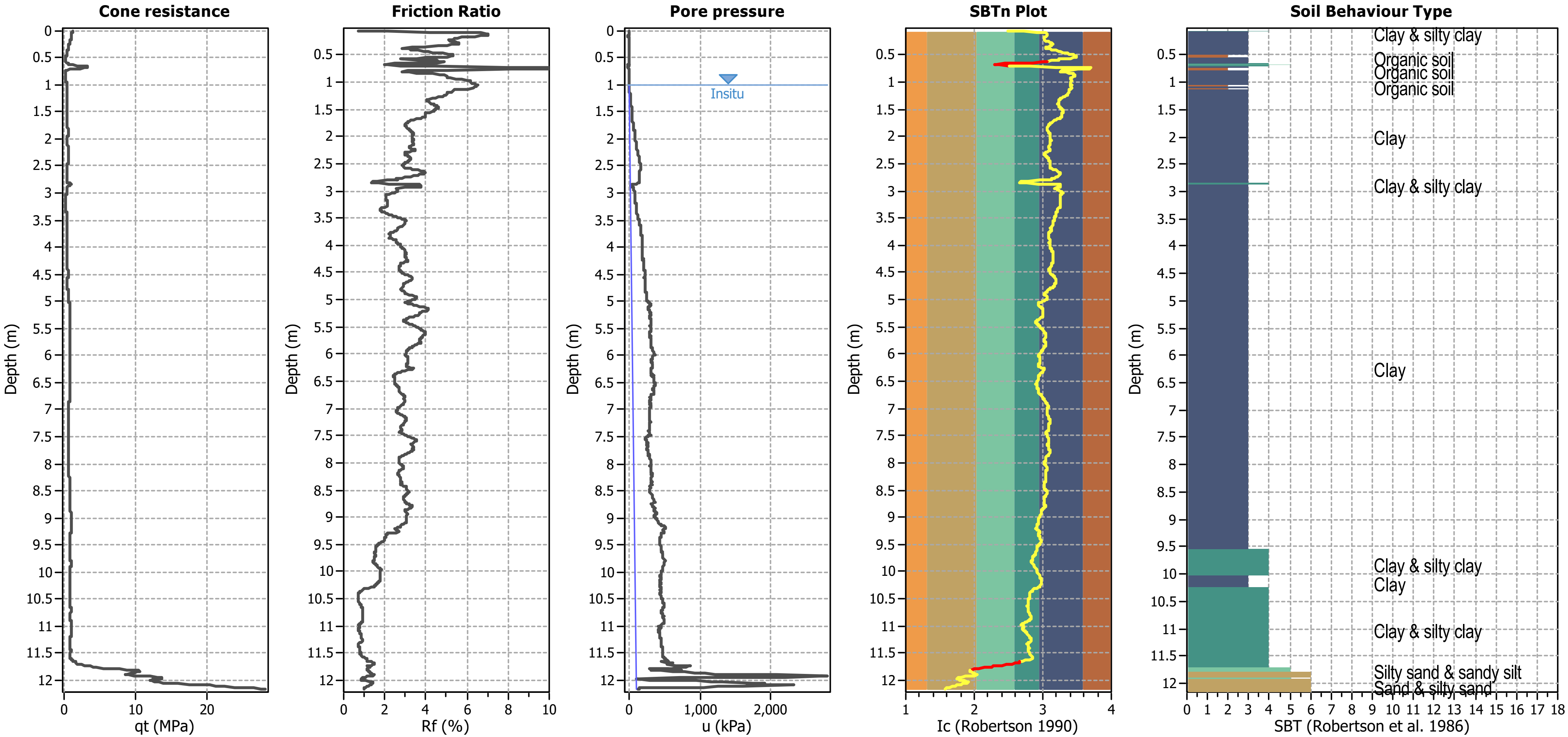
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT22
Total depth: 12.16 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

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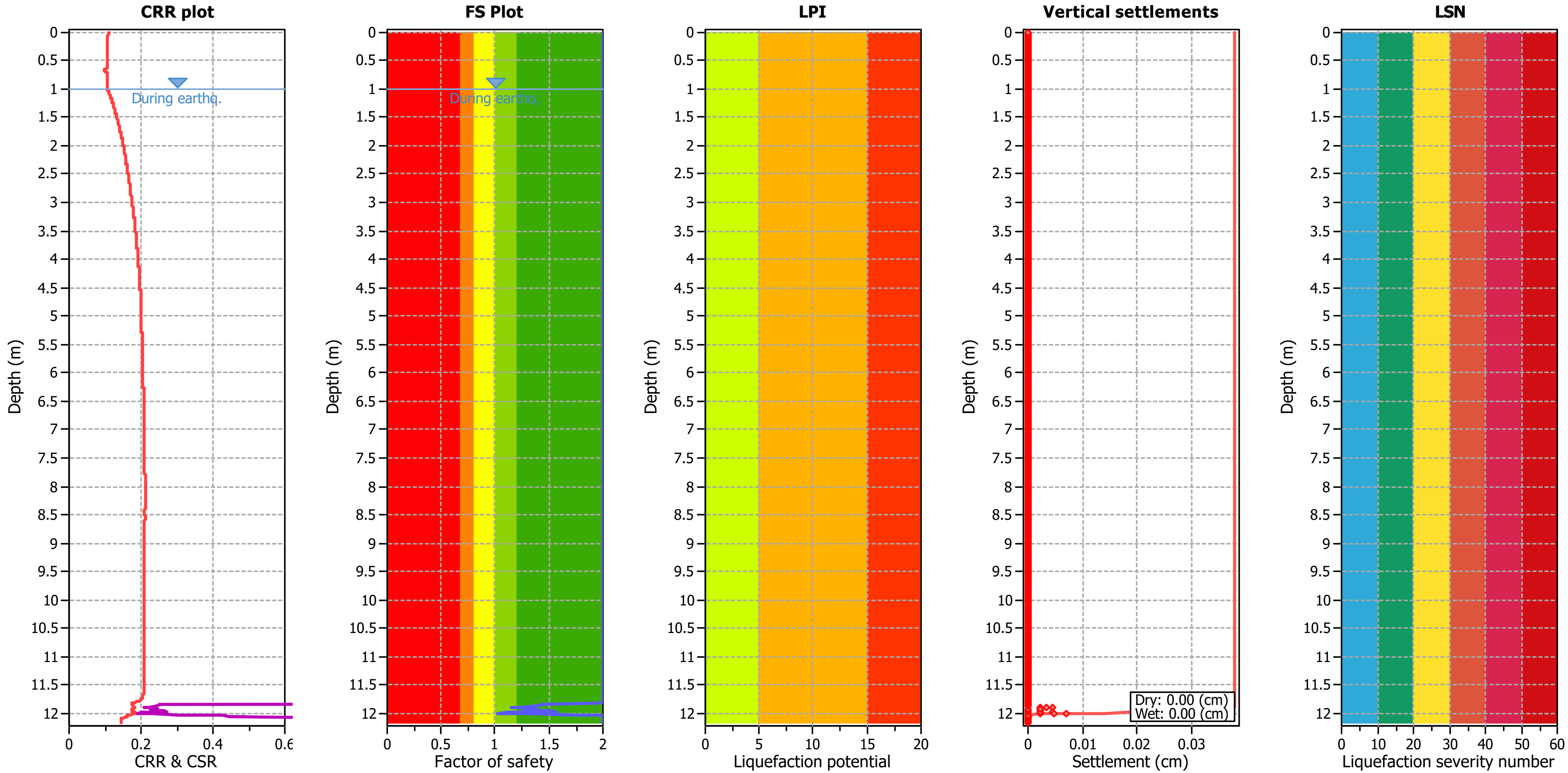
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT22
Total depth: 12.16 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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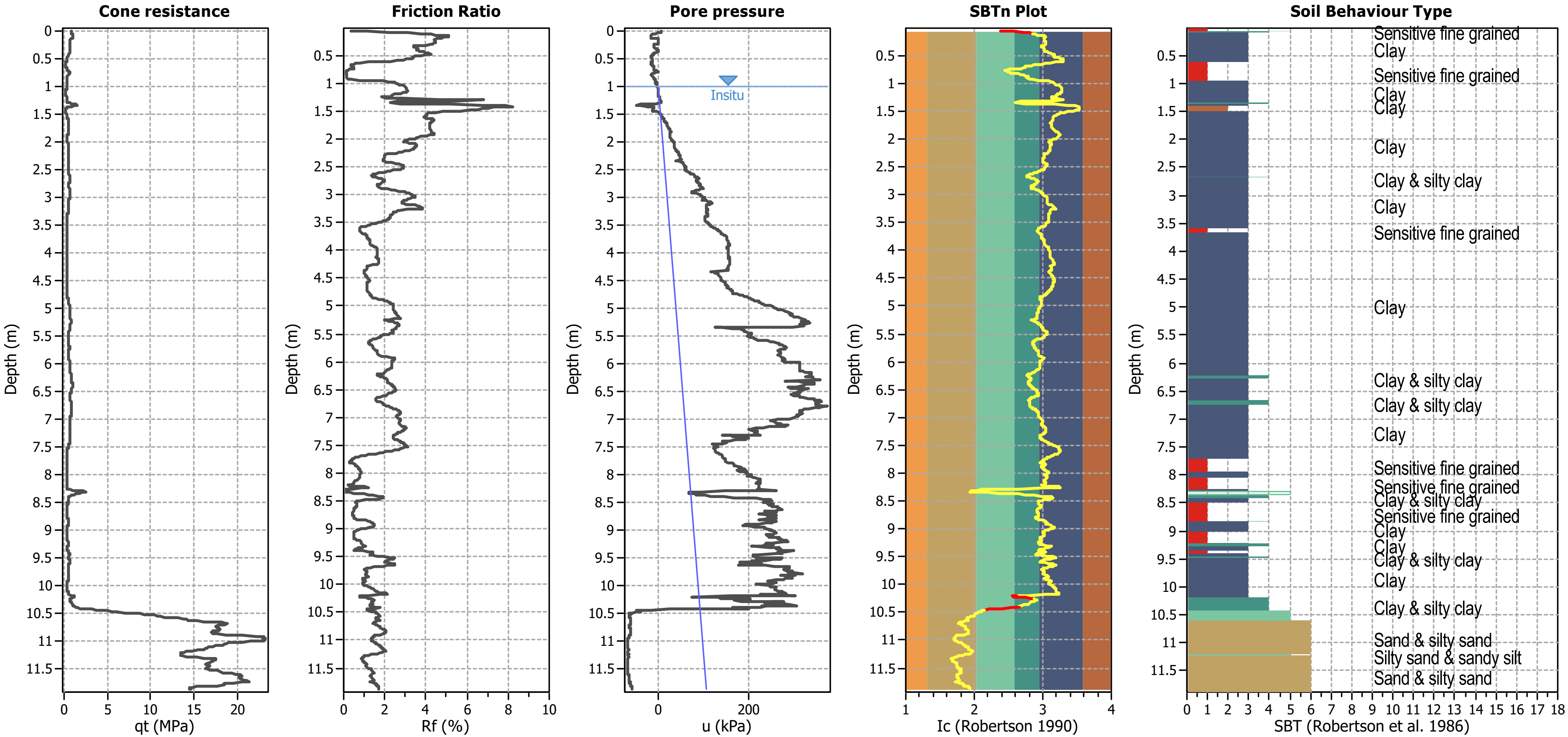
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

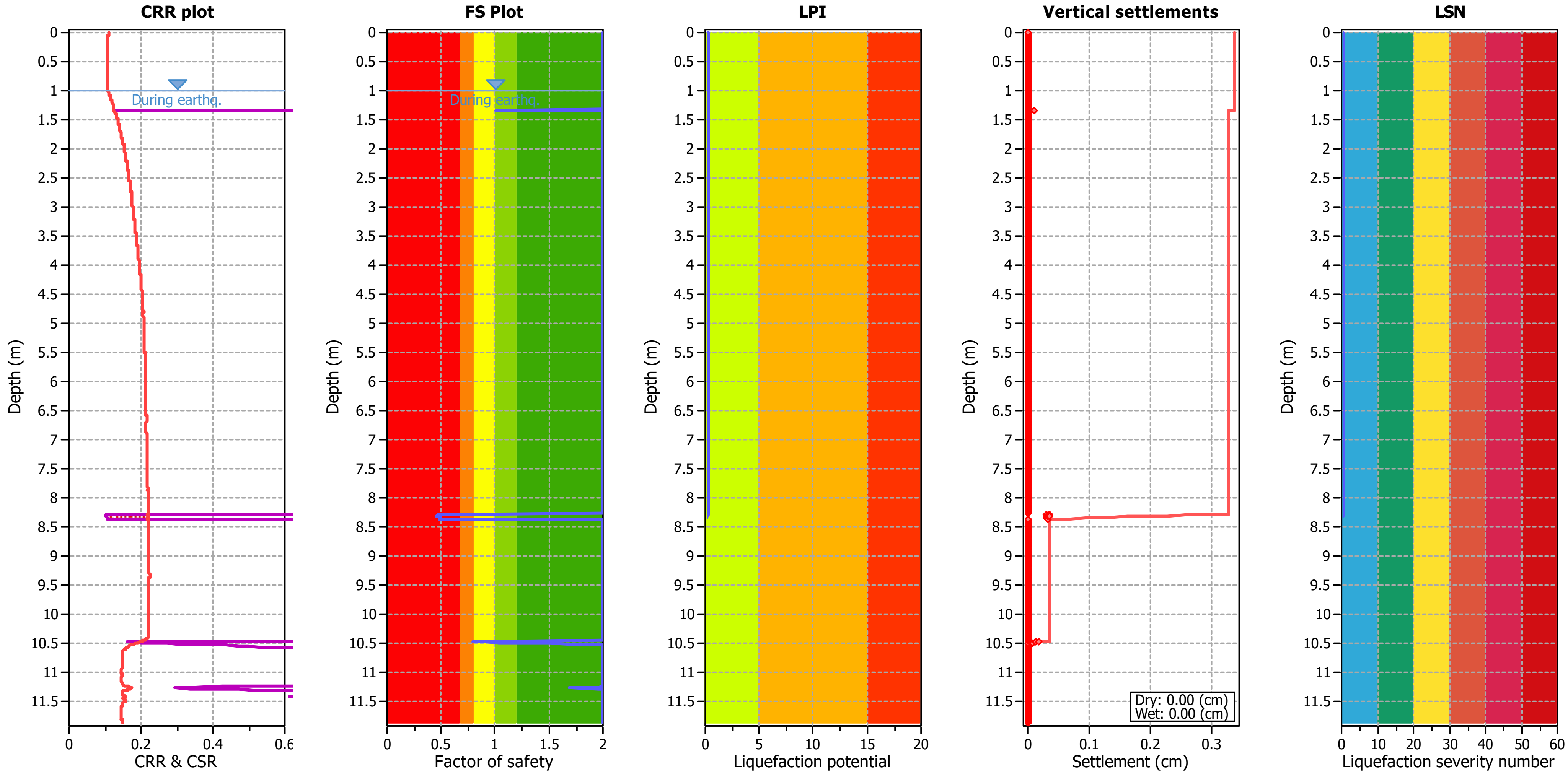
CPT: CPT23
Total depth: 11.88 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT23
Total depth: 11.88 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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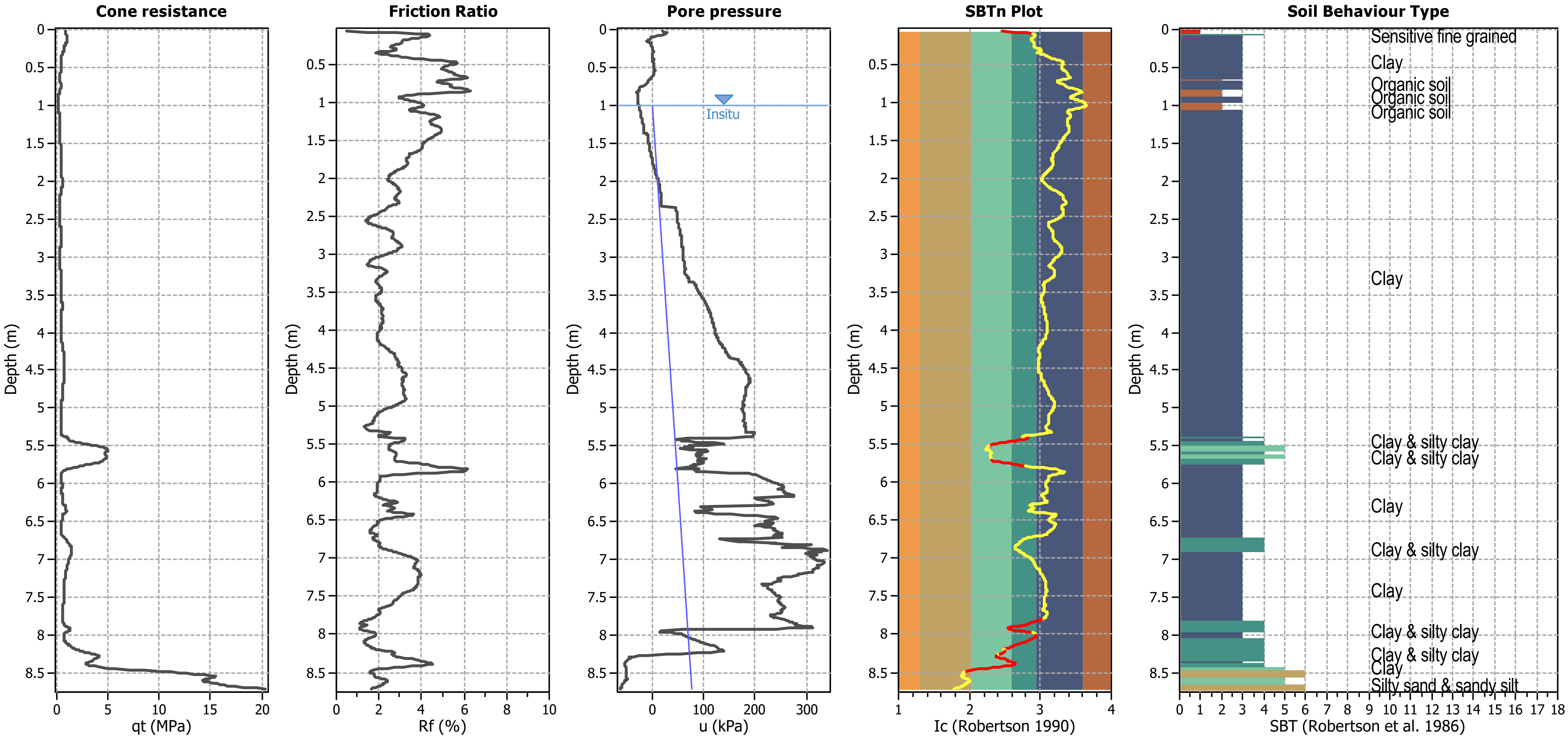
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT24
Total depth: 8.72 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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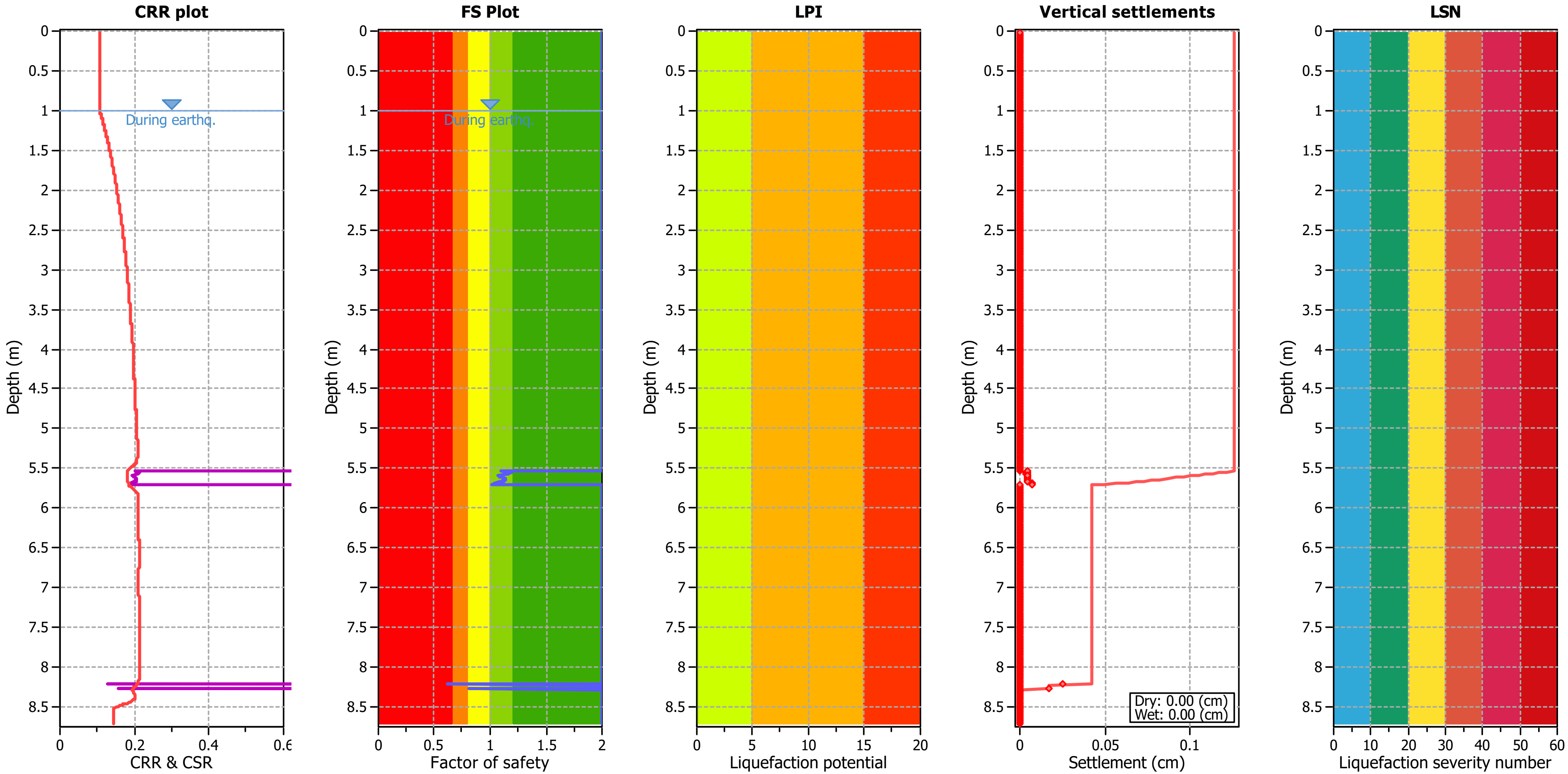
Project: **Auckland Surf Park** | **CPT: CPT24**
Location: **Dairy Flat, Auckland** | Total depth: 8.72 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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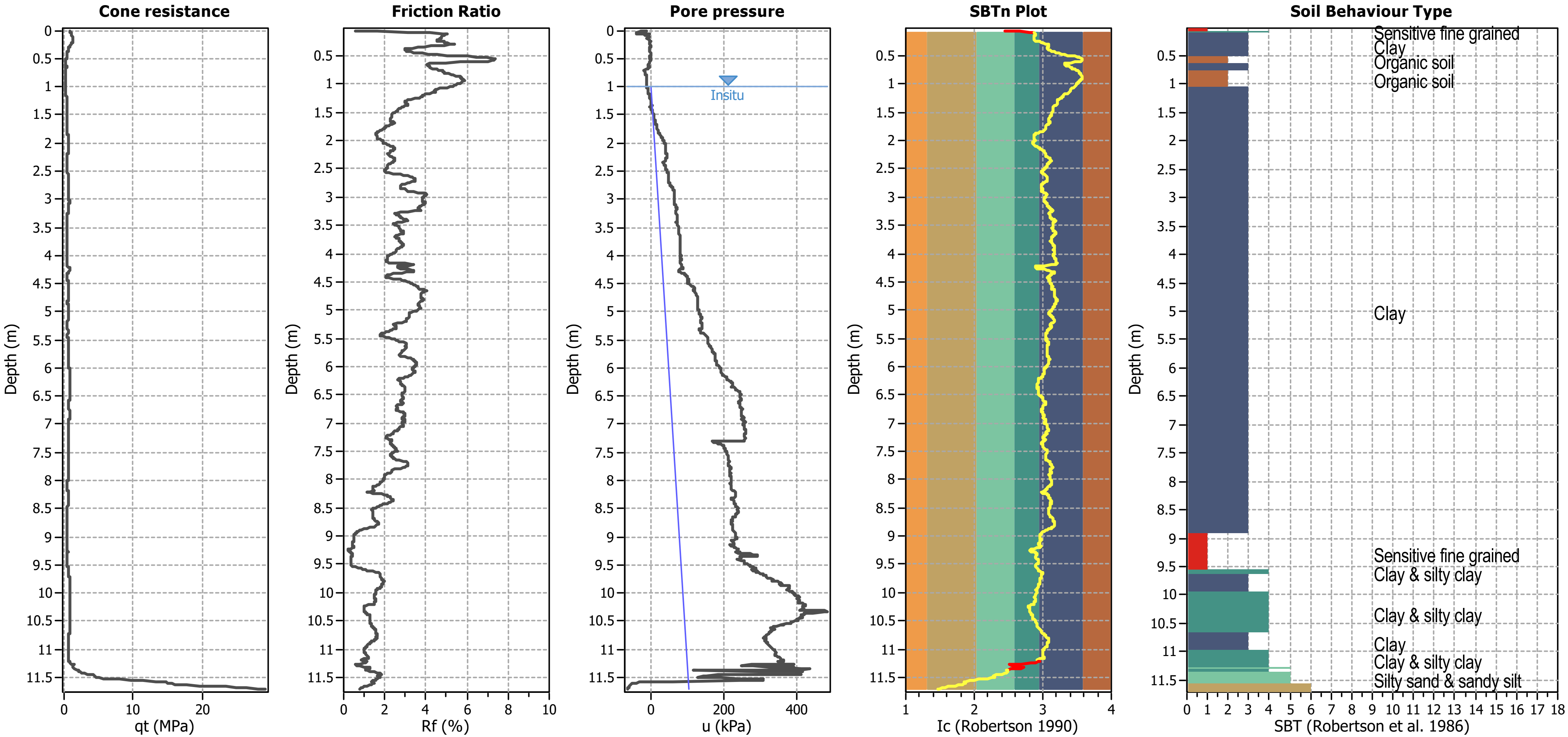
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT25
Total depth: 11.71 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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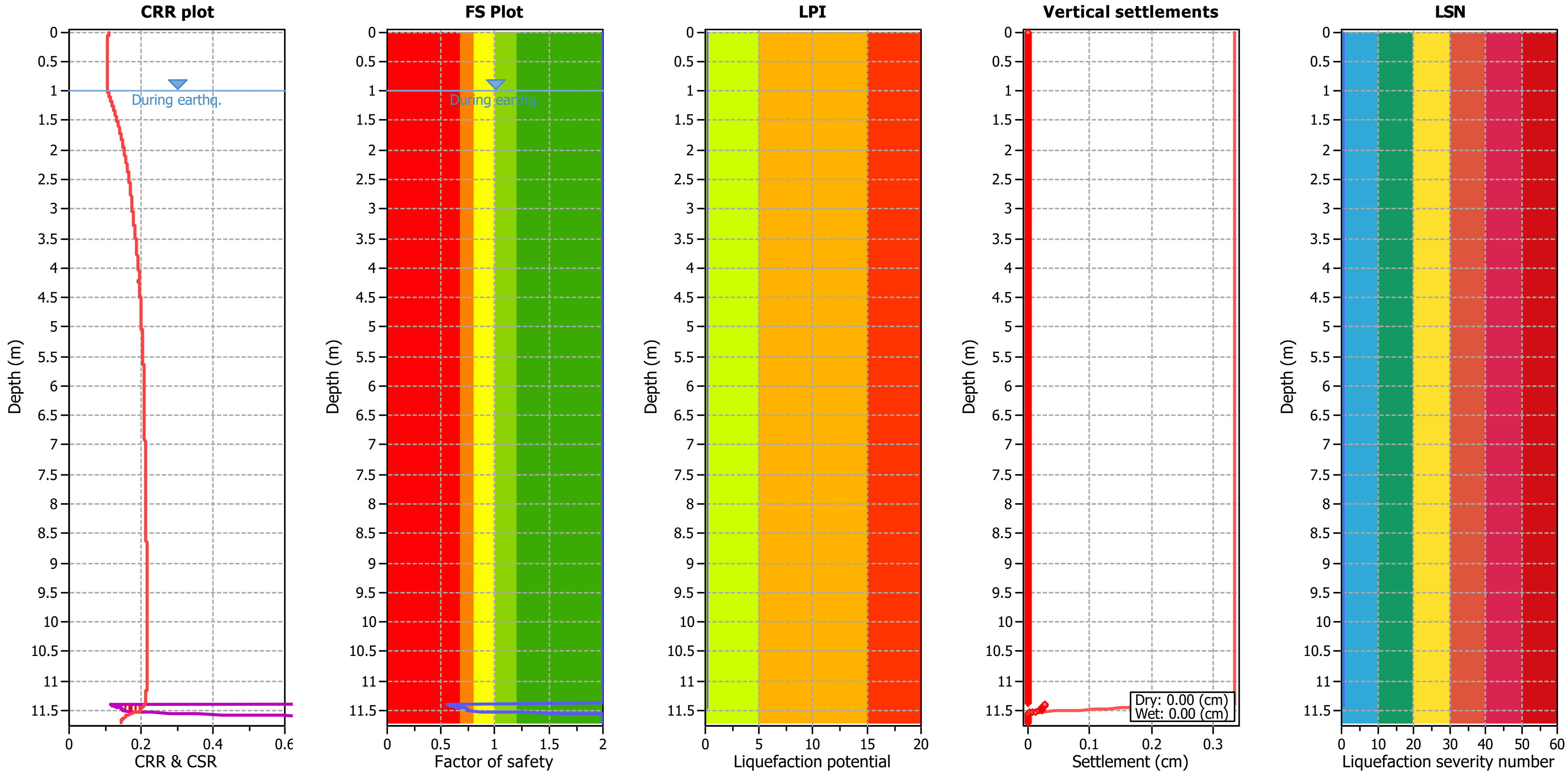
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT25
Total depth: 11.71 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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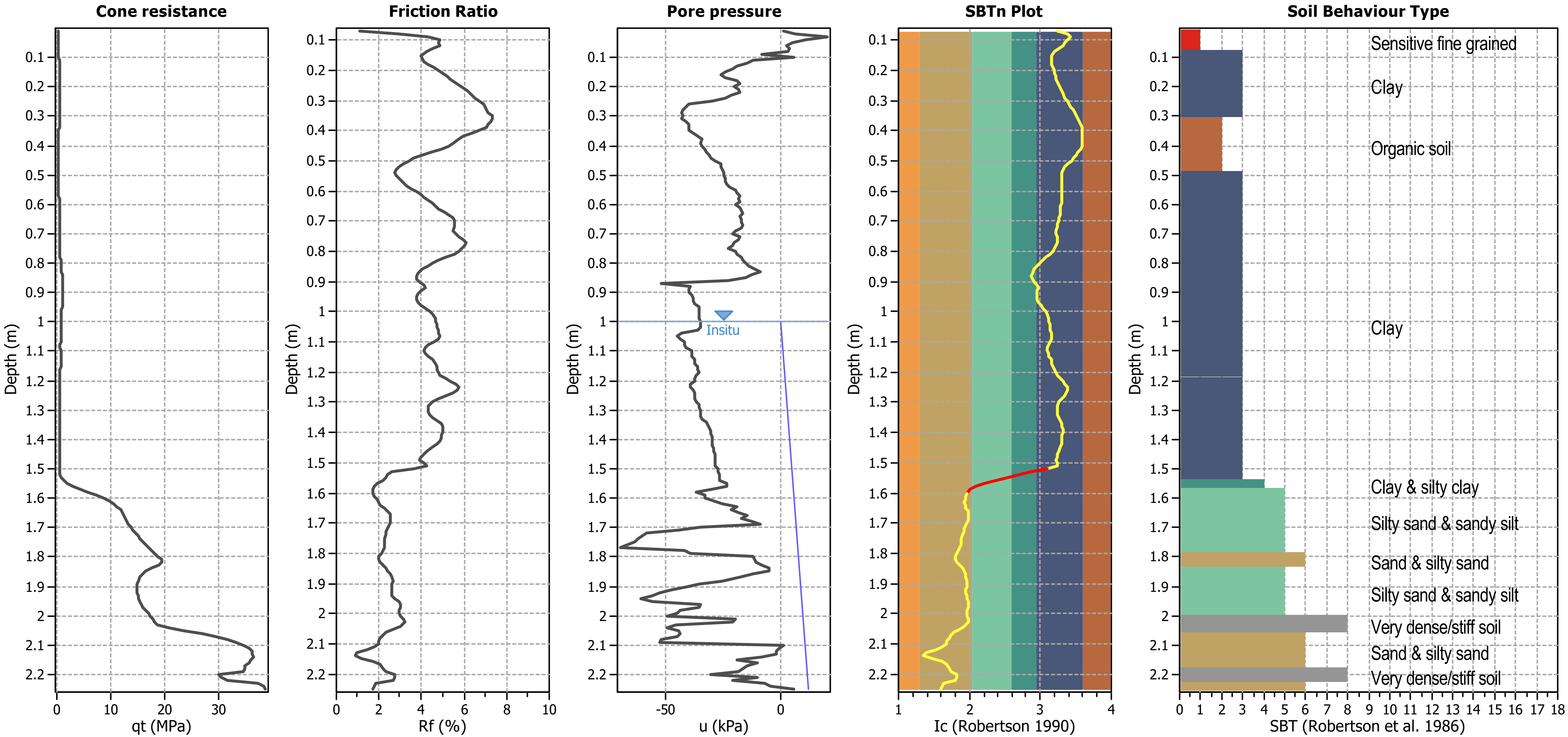
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT101
Total depth: 2.25 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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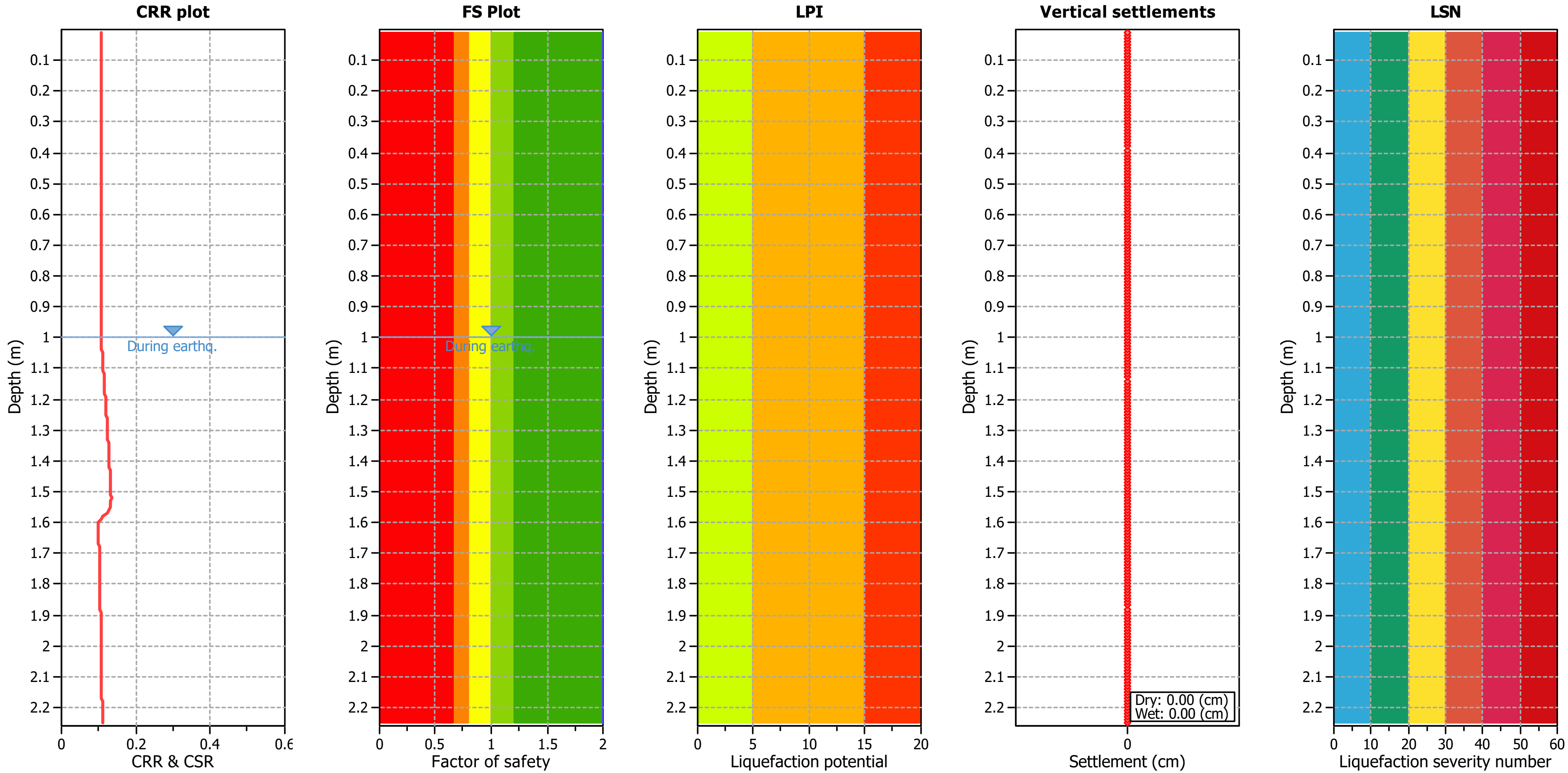
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT101
Total depth: 2.25 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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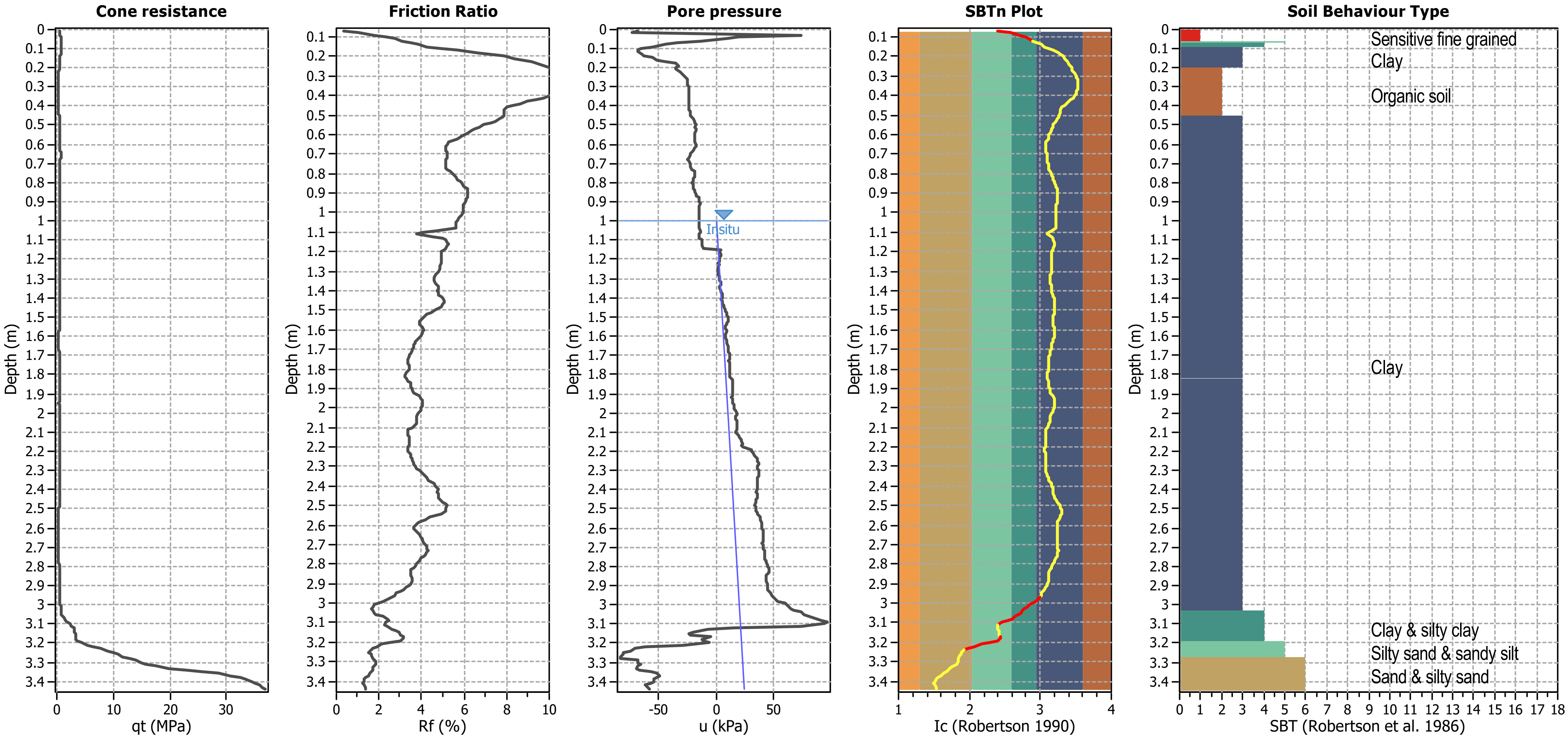
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT102
Total depth: 3.44 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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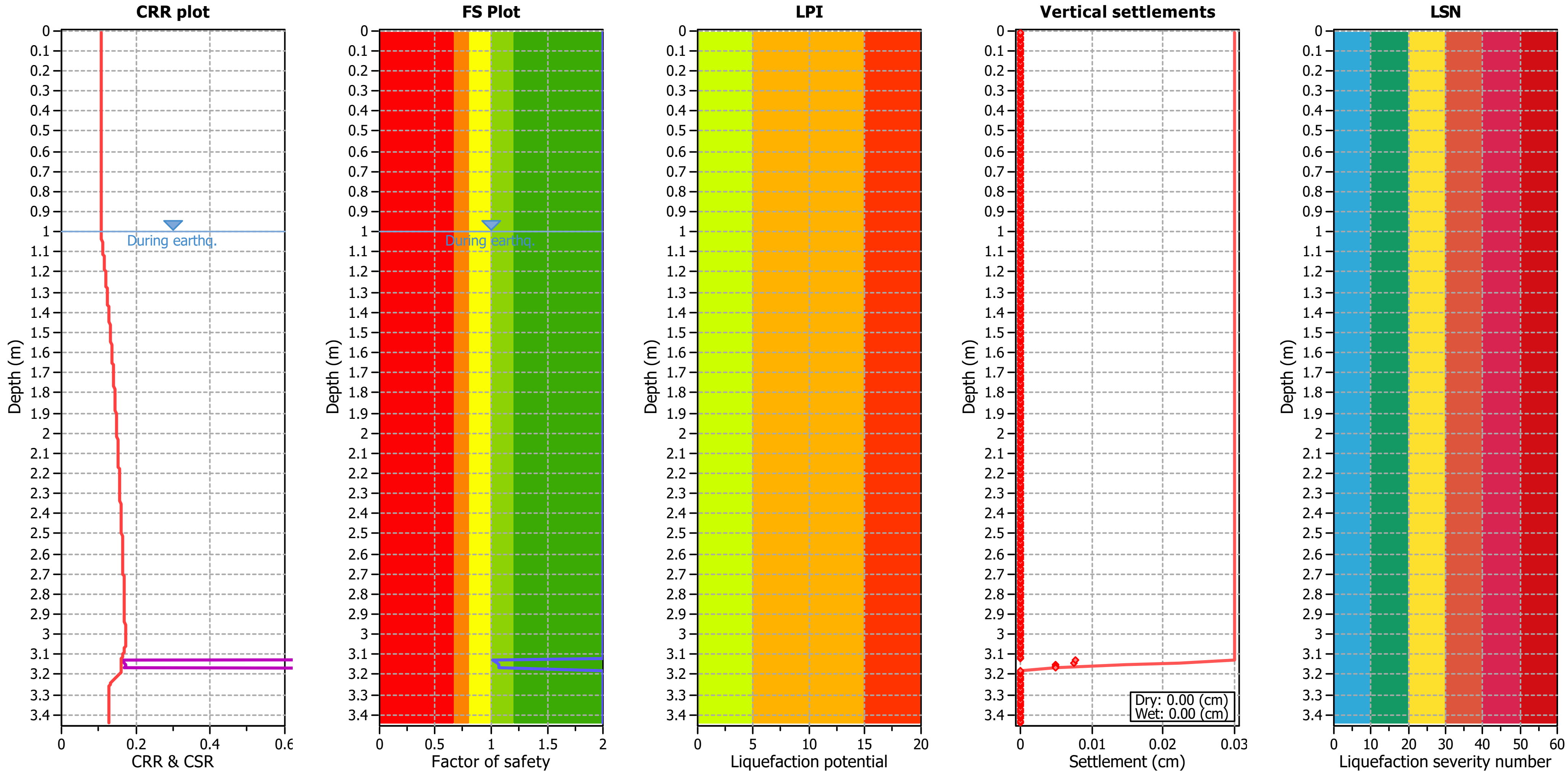
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT102
Total depth: 3.44 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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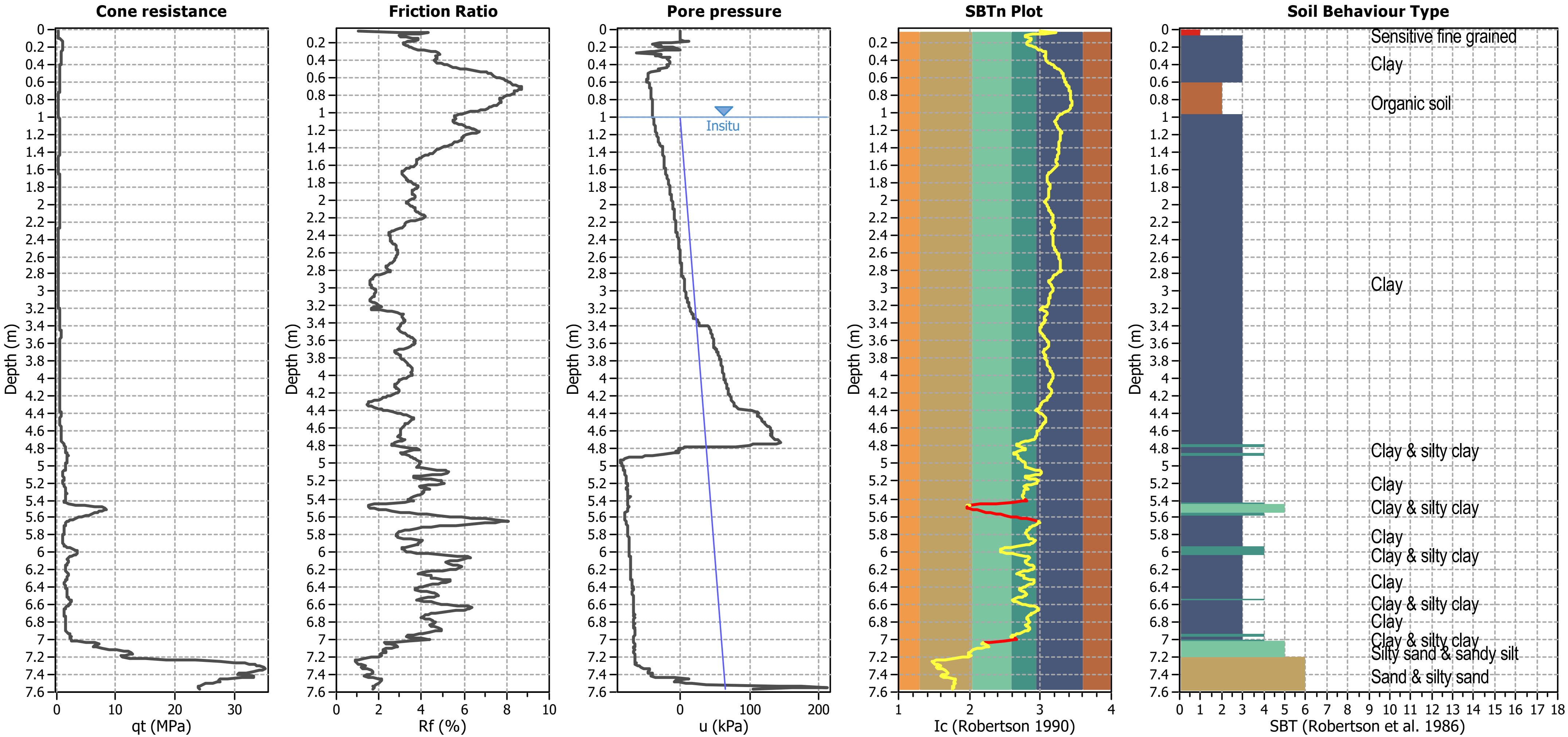
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT103
Total depth: 7.57 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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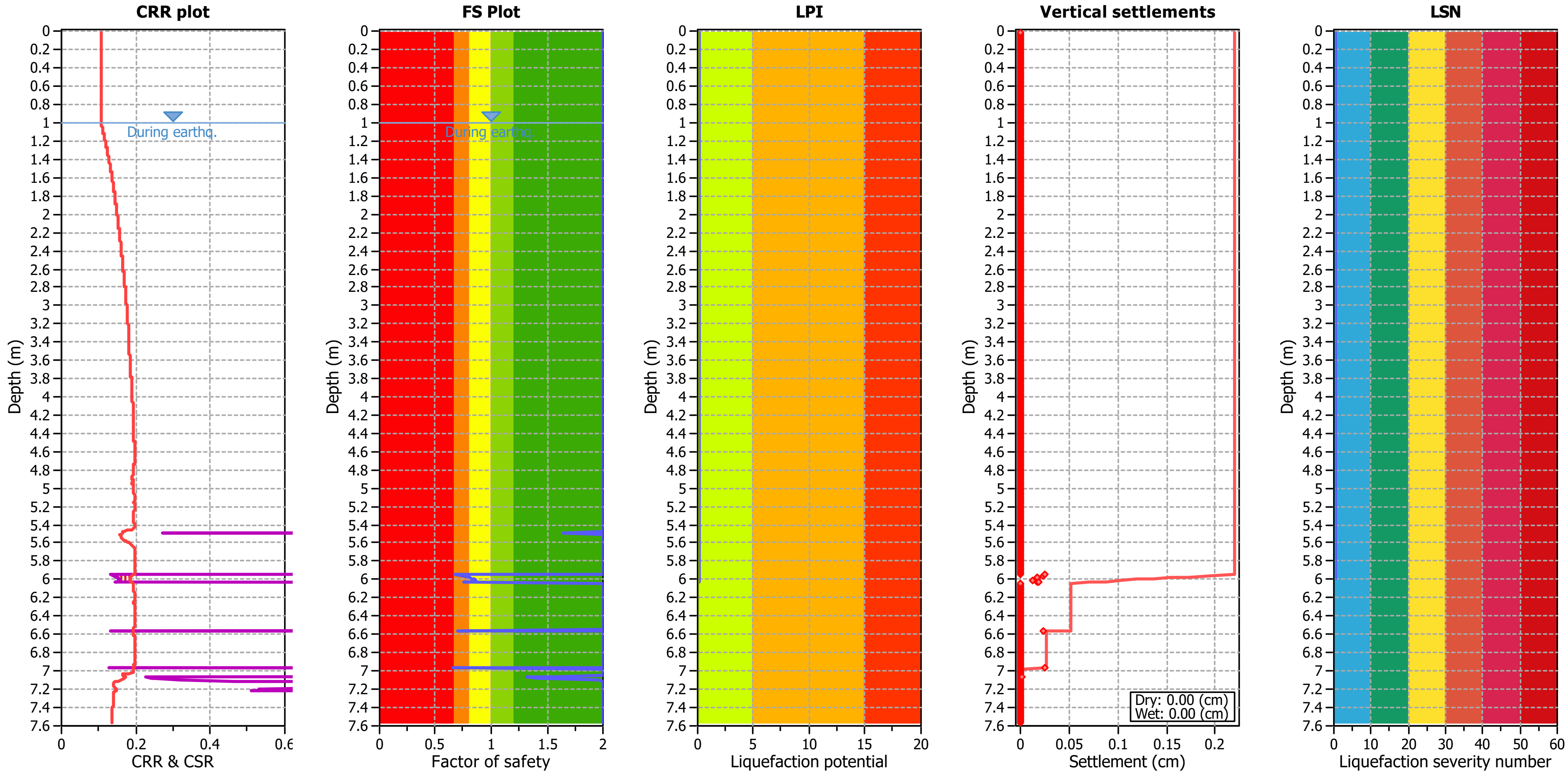
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT103
Total depth: 7.57 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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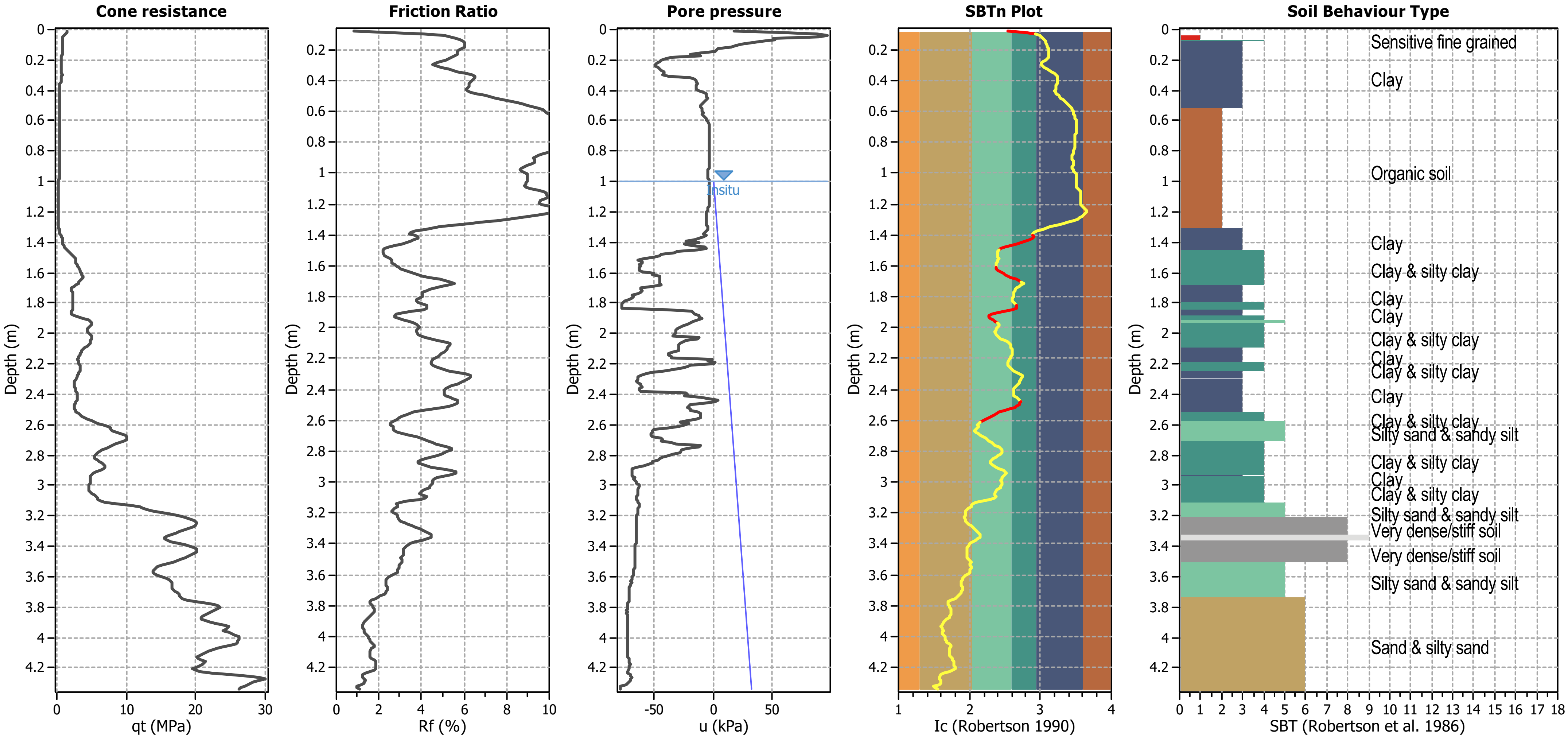
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT104
Total depth: 4.34 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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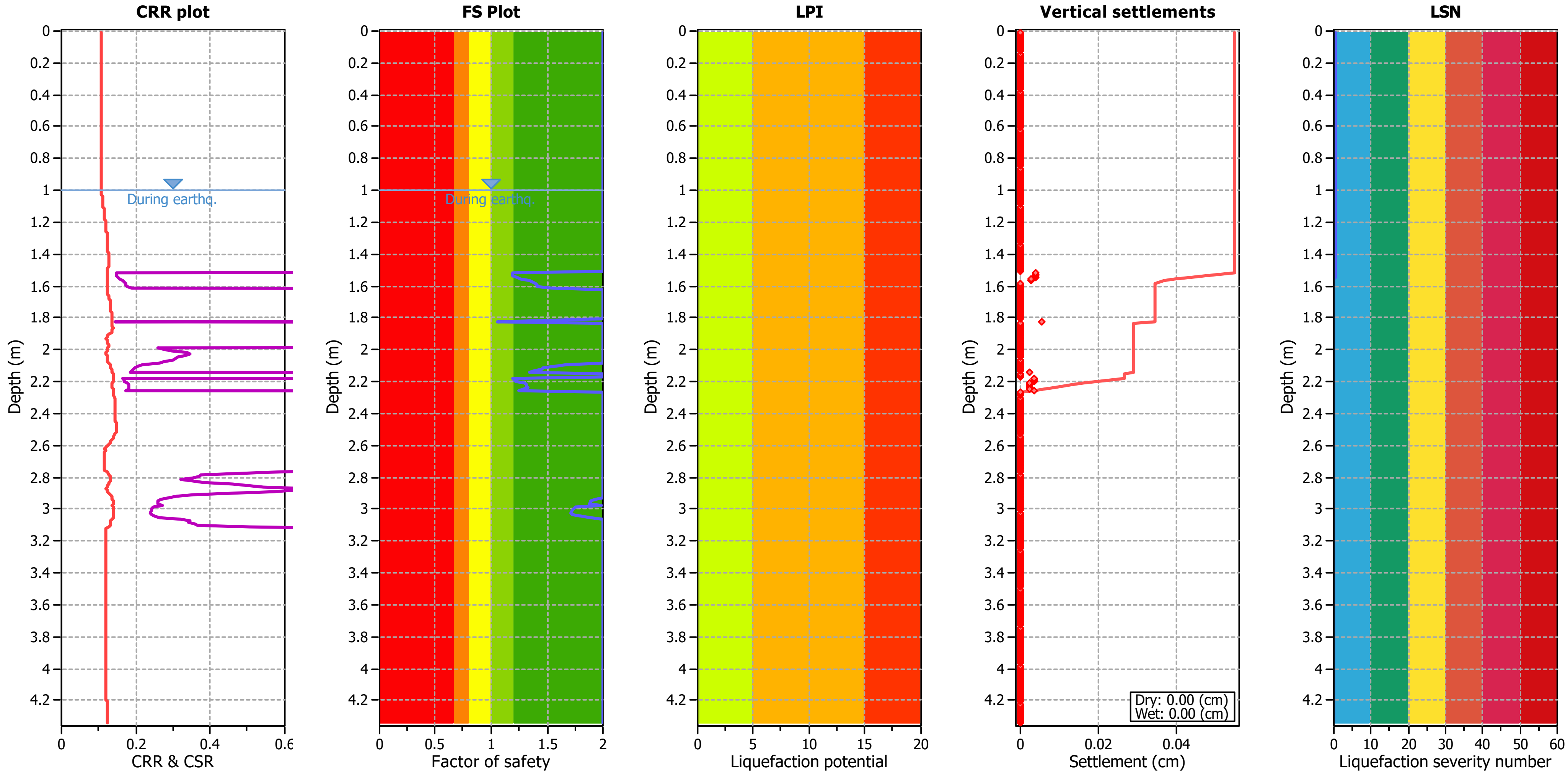
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT104
Total depth: 4.34 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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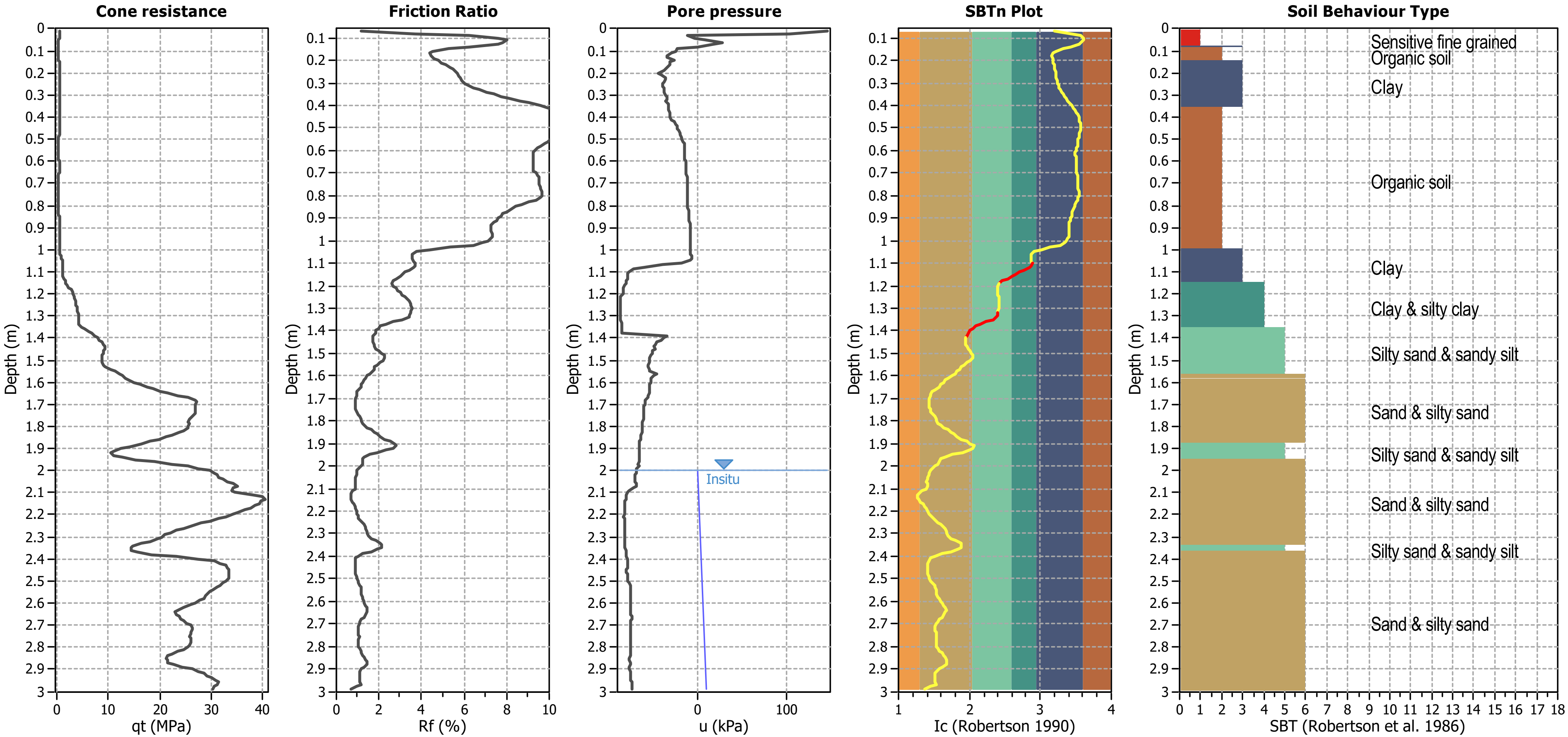
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT106
Total depth: 2.99 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Excavation:	Yes	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Excavation depth:	1.00 m	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.00 kPa	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		

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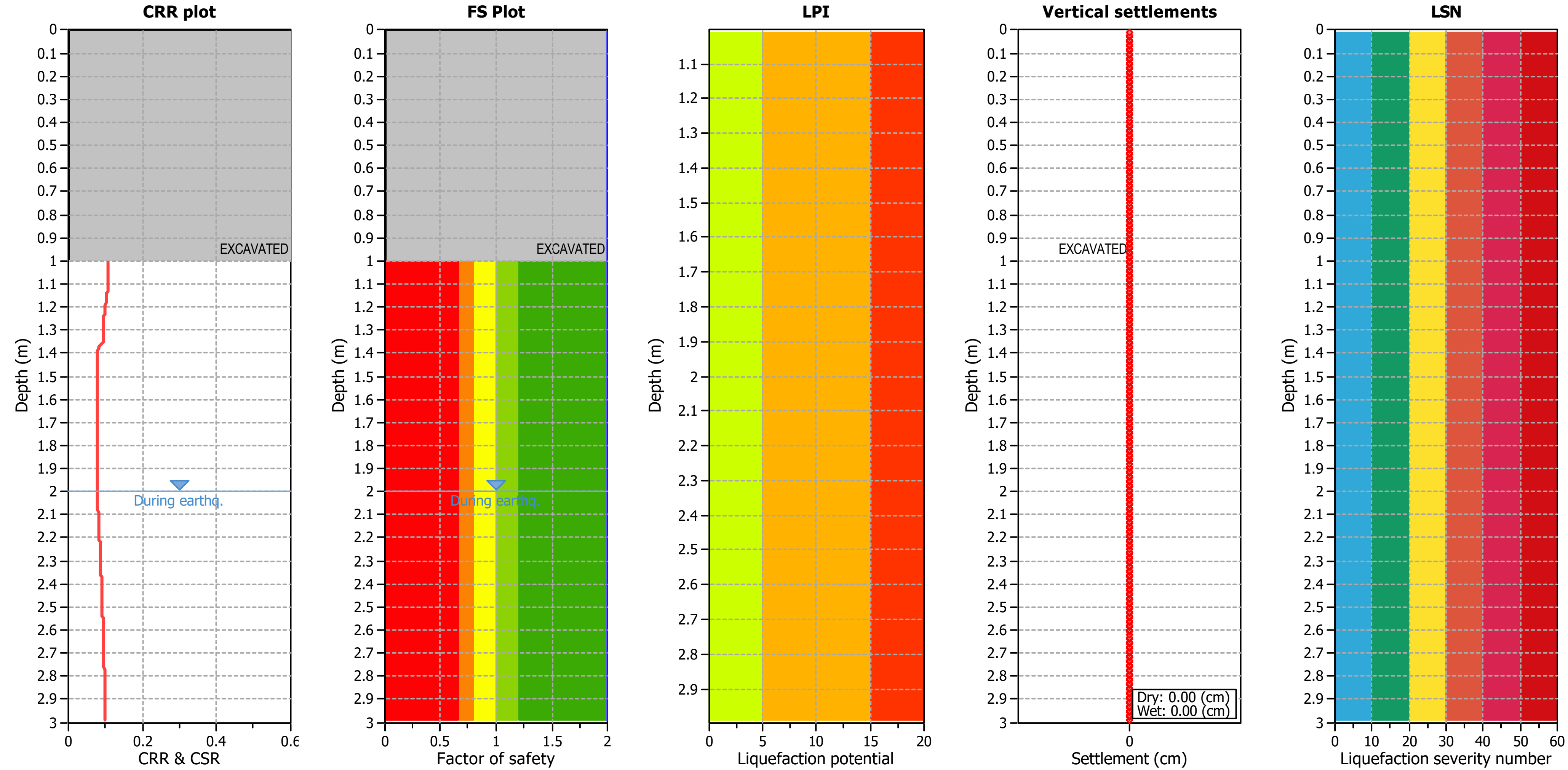
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT106
Total depth: 2.99 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Excavation:	Yes	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Excavation depth:	1.00 m	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.00 kPa	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		

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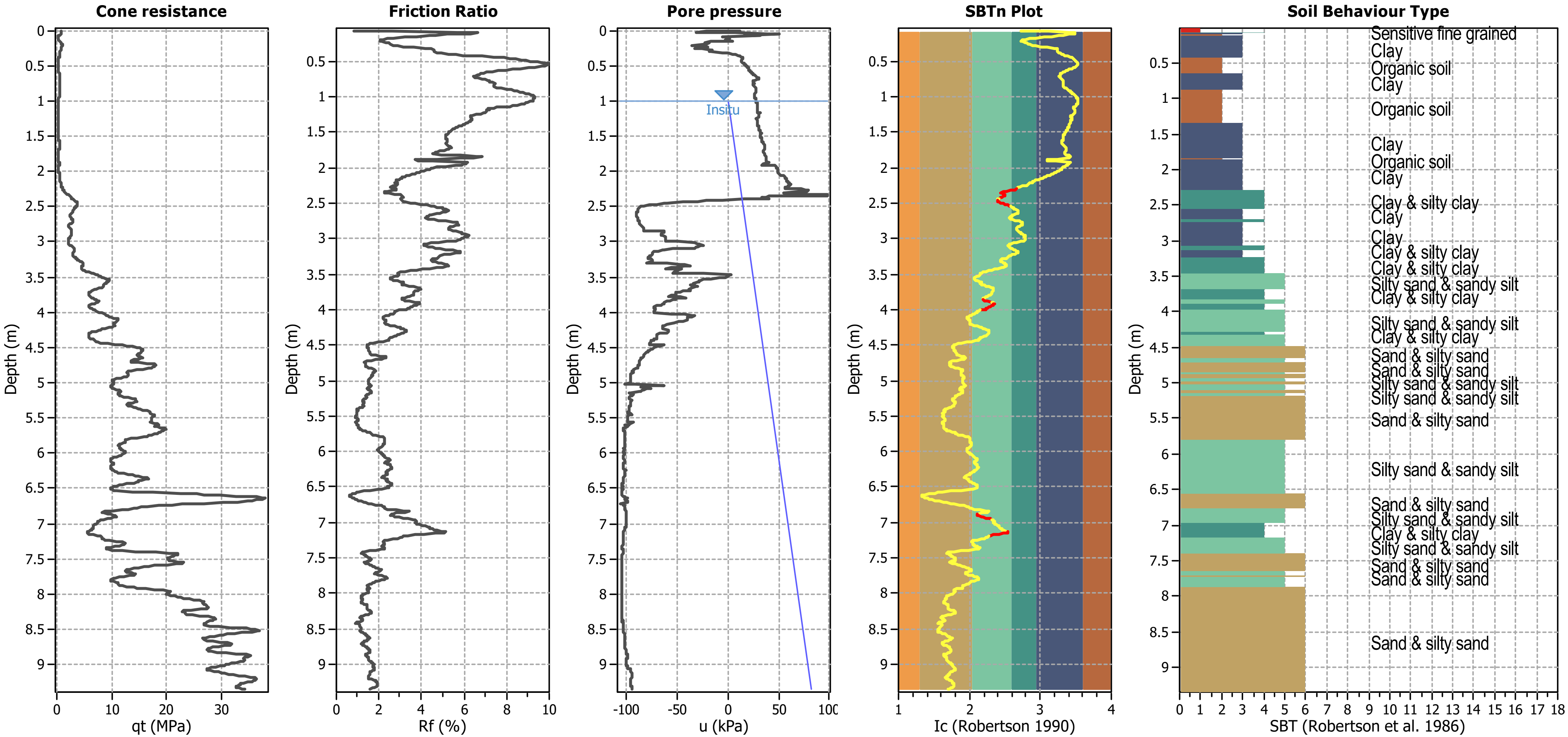
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT107
Total depth: 9.35 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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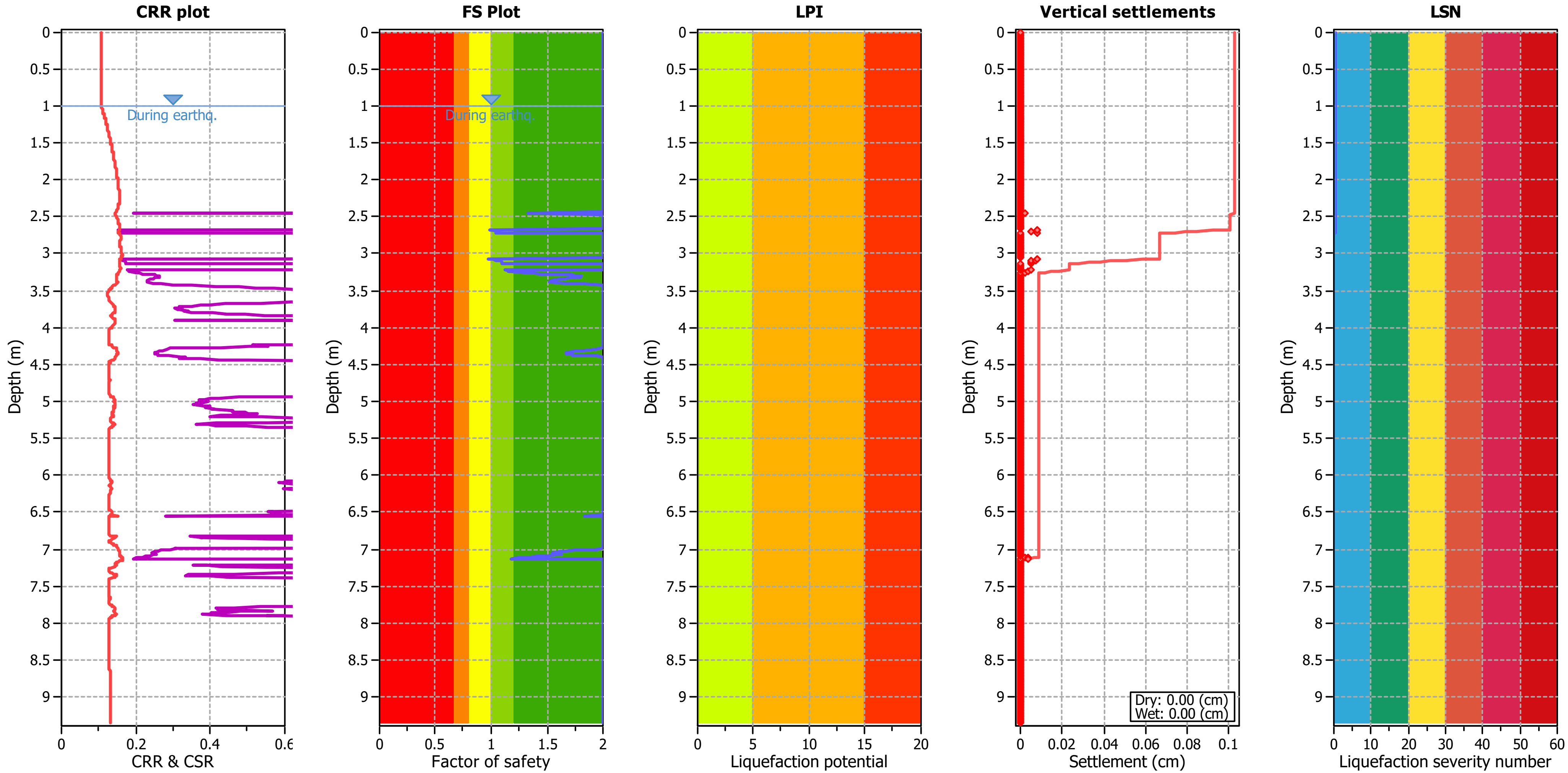
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT107
Total depth: 9.35 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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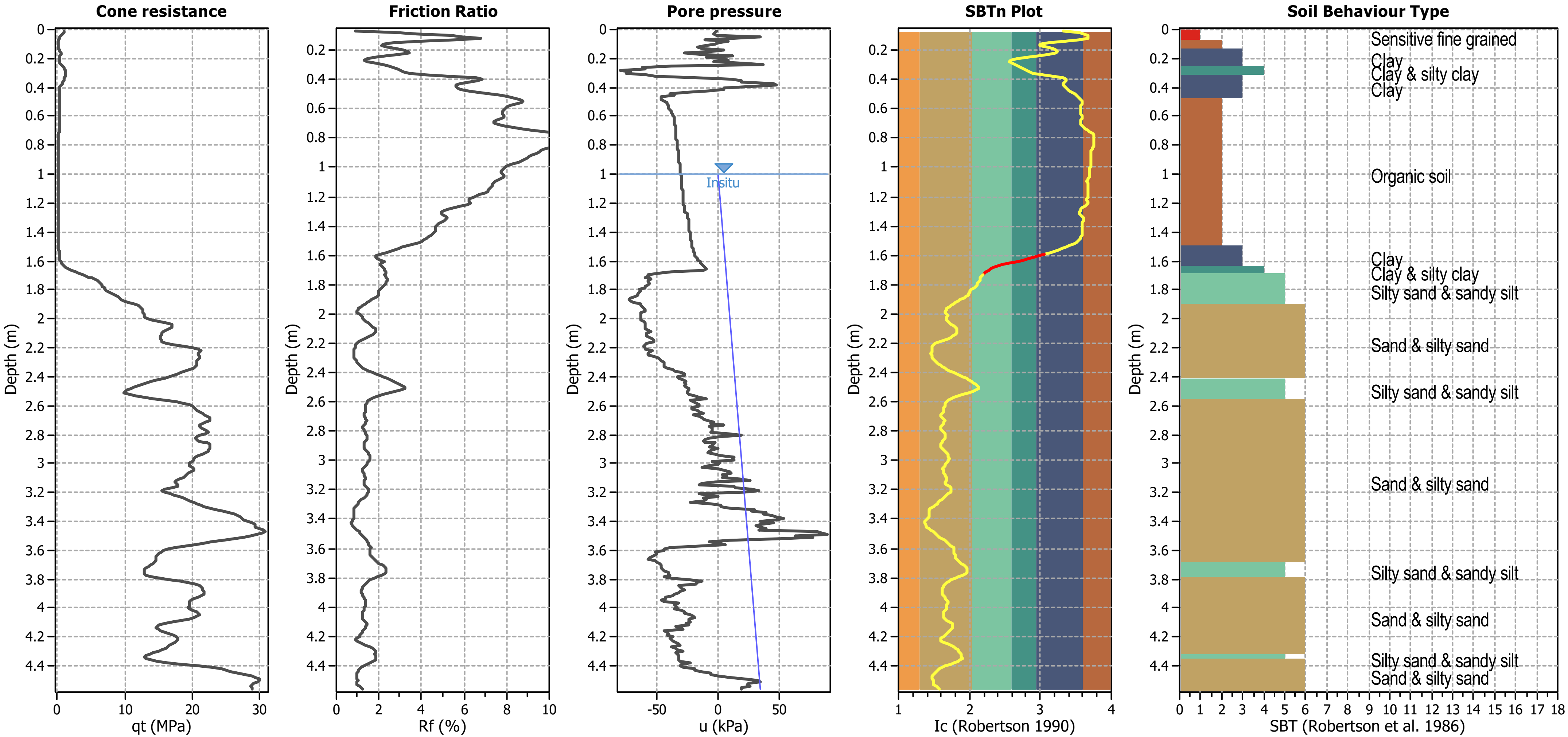
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

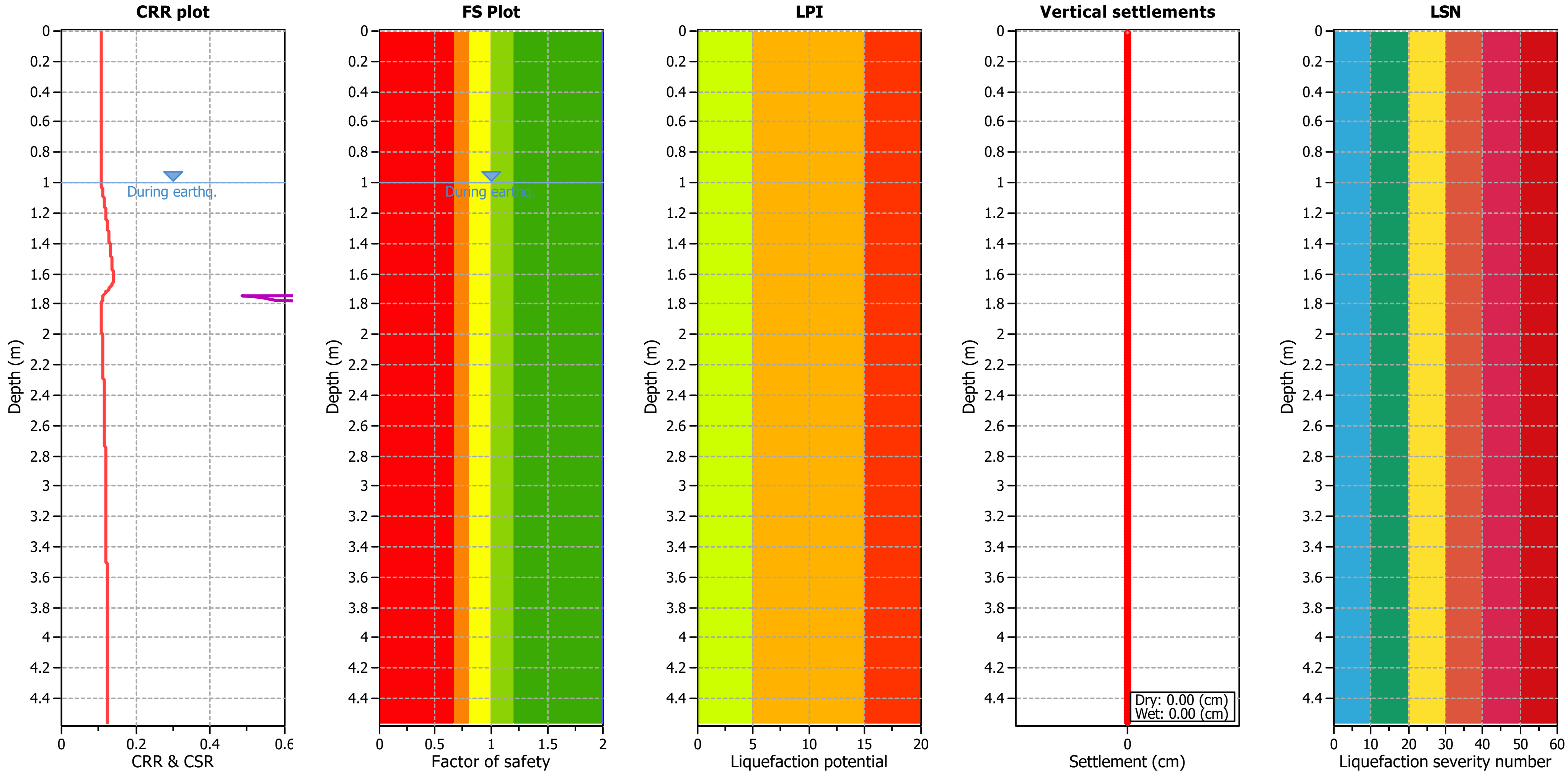
CPT: CPT108
Total depth: 4.56 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT108
Total depth: 4.56 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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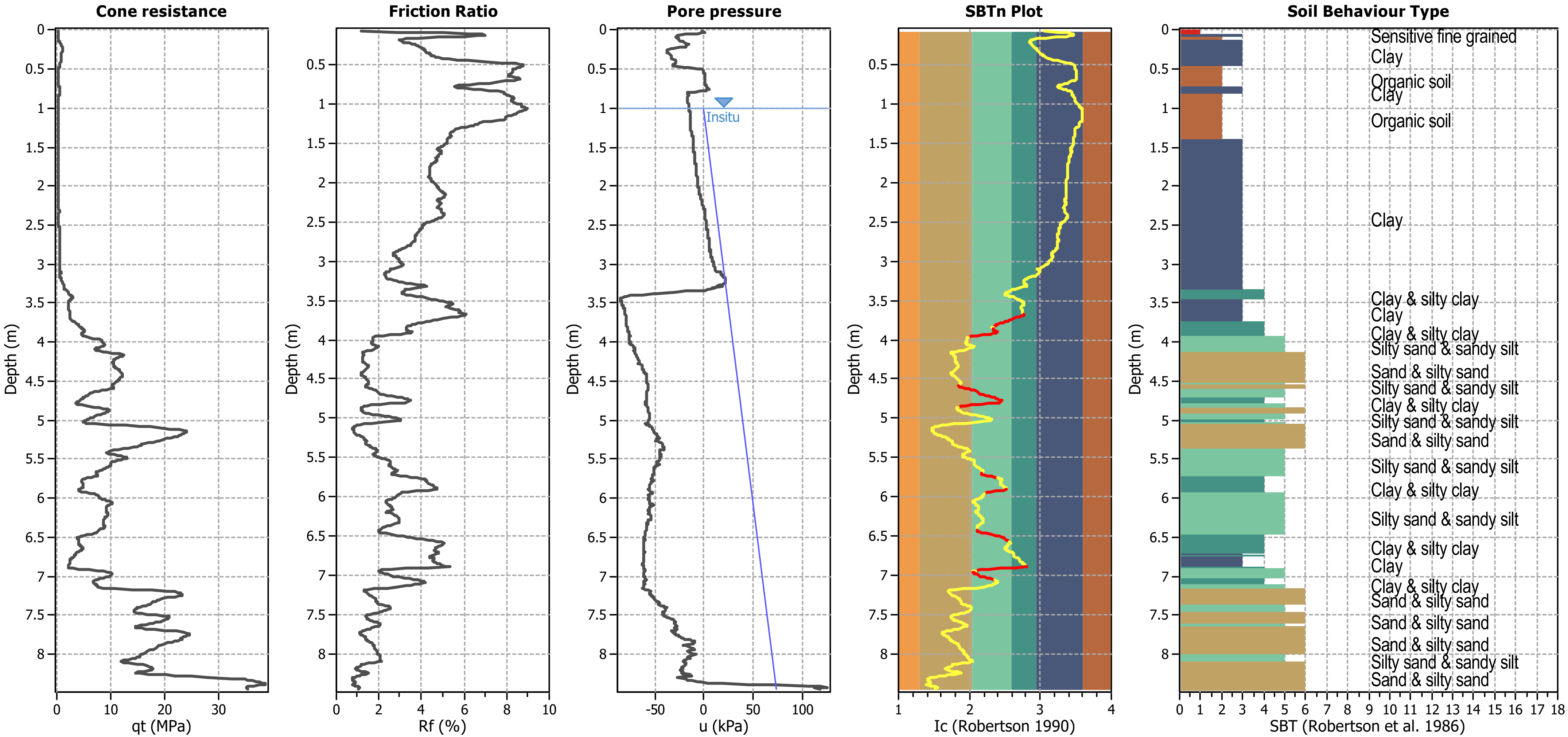
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

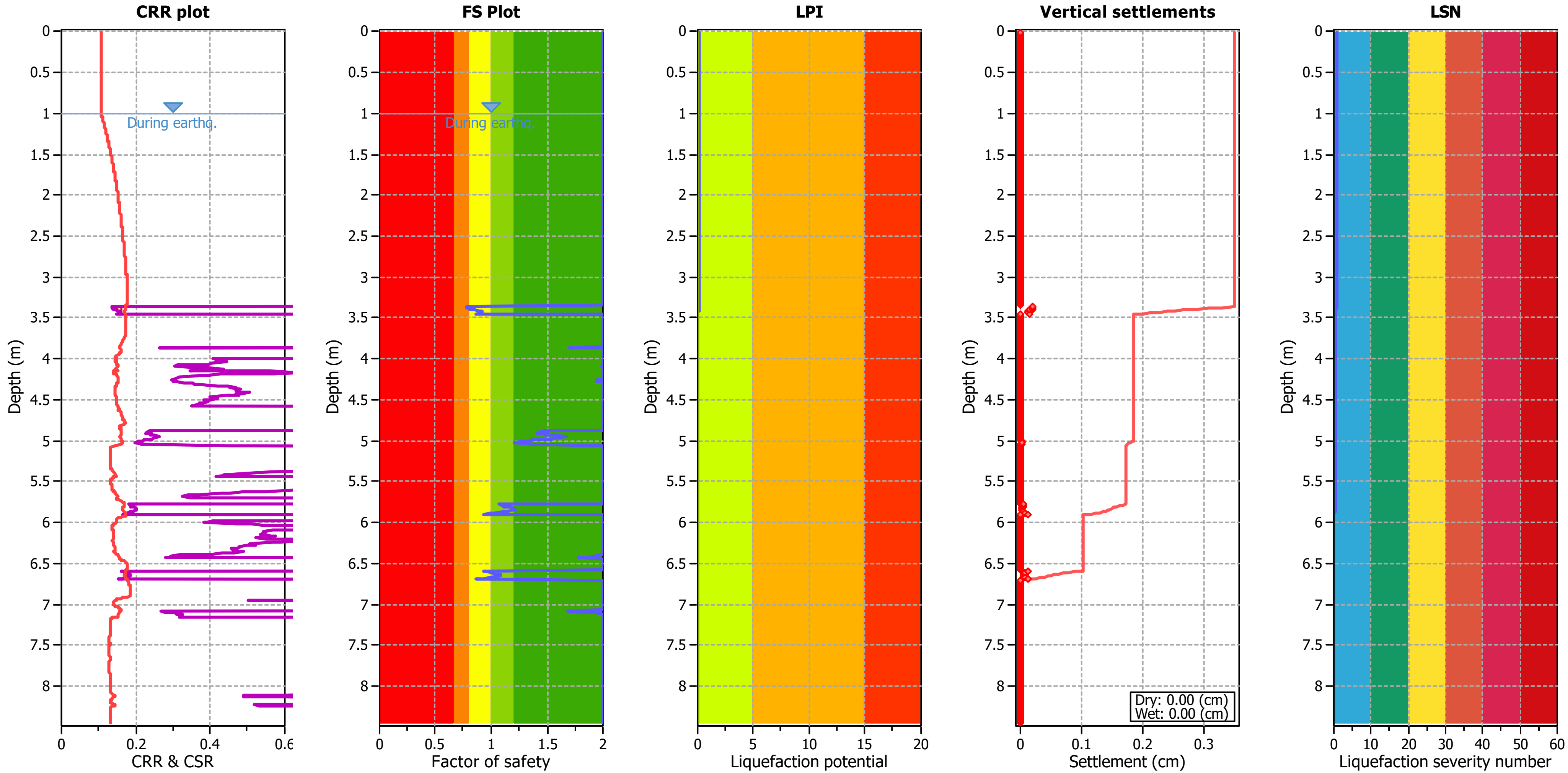
CPT: CPT109
Total depth: 8.45 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT109
Total depth: 8.45 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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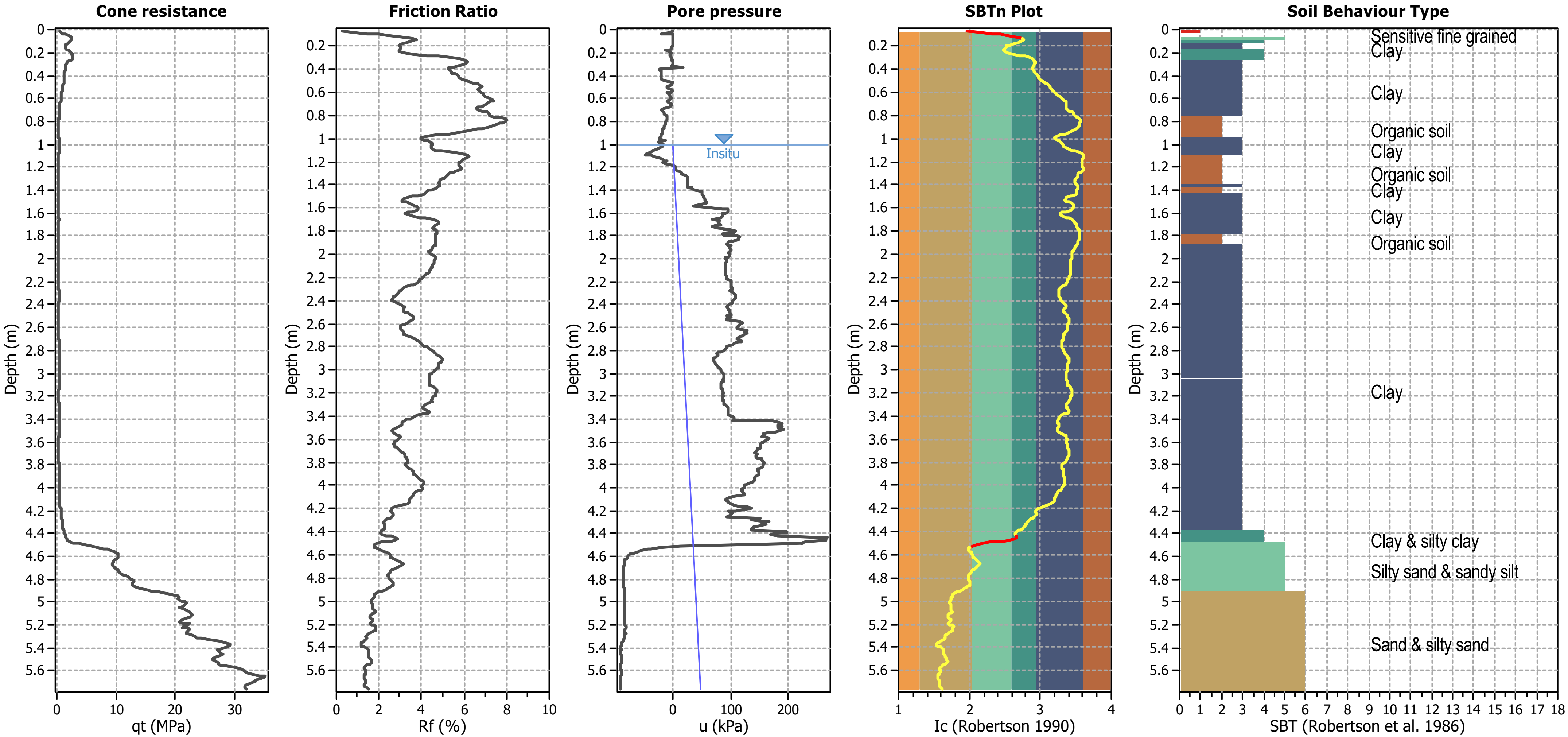
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT110
Total depth: 5.76 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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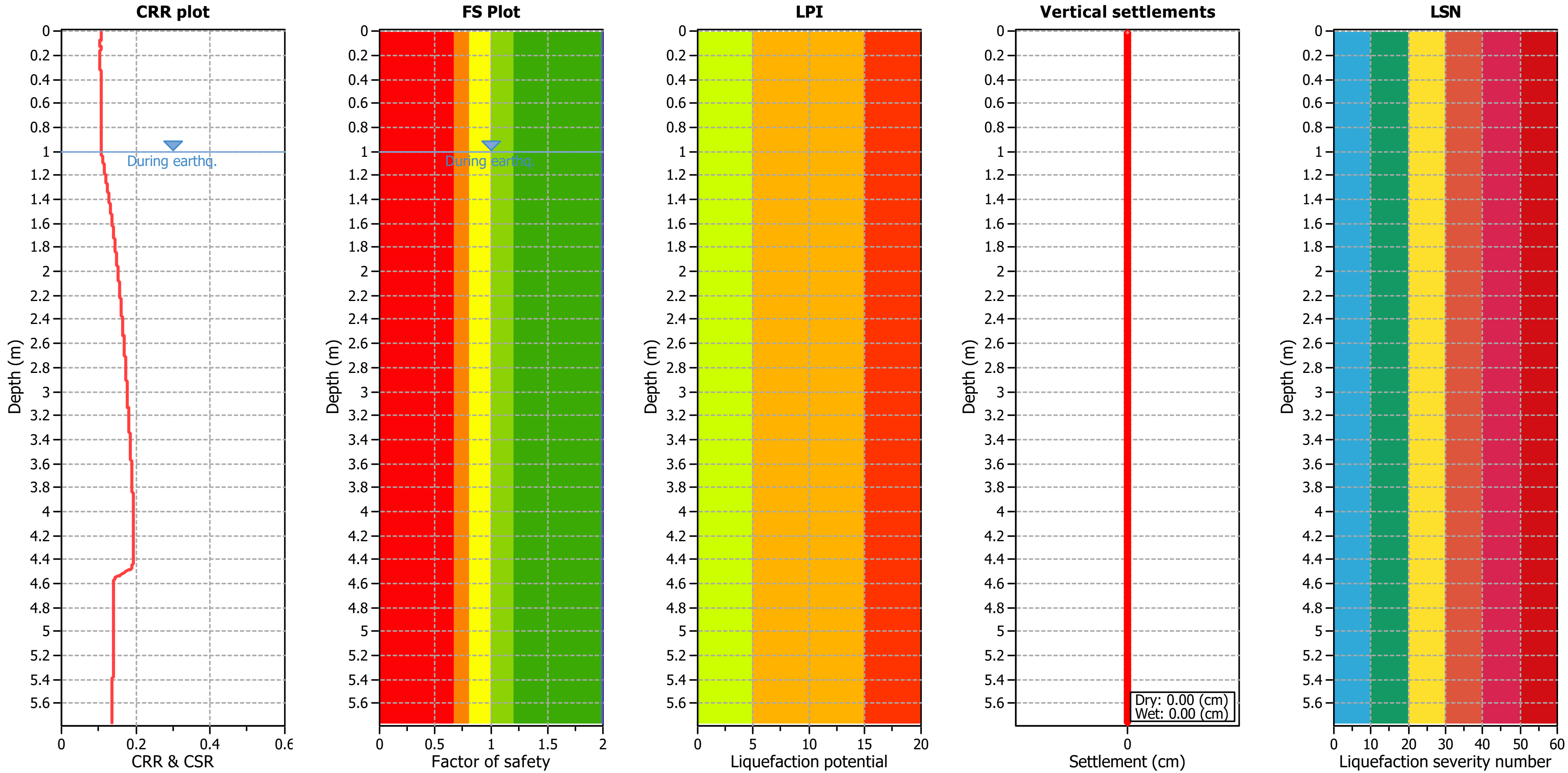
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT110
Total depth: 5.76 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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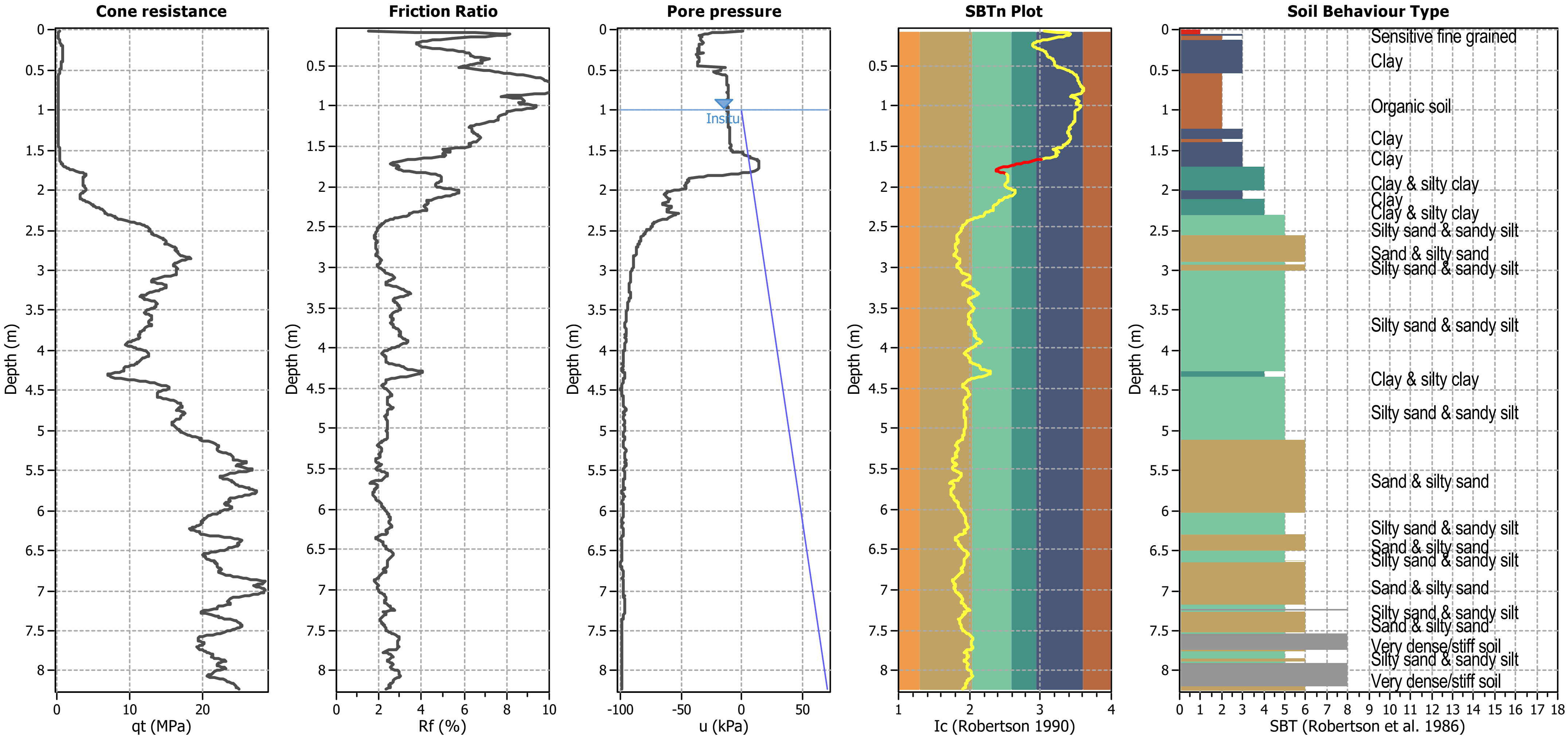
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT111
Total depth: 8.23 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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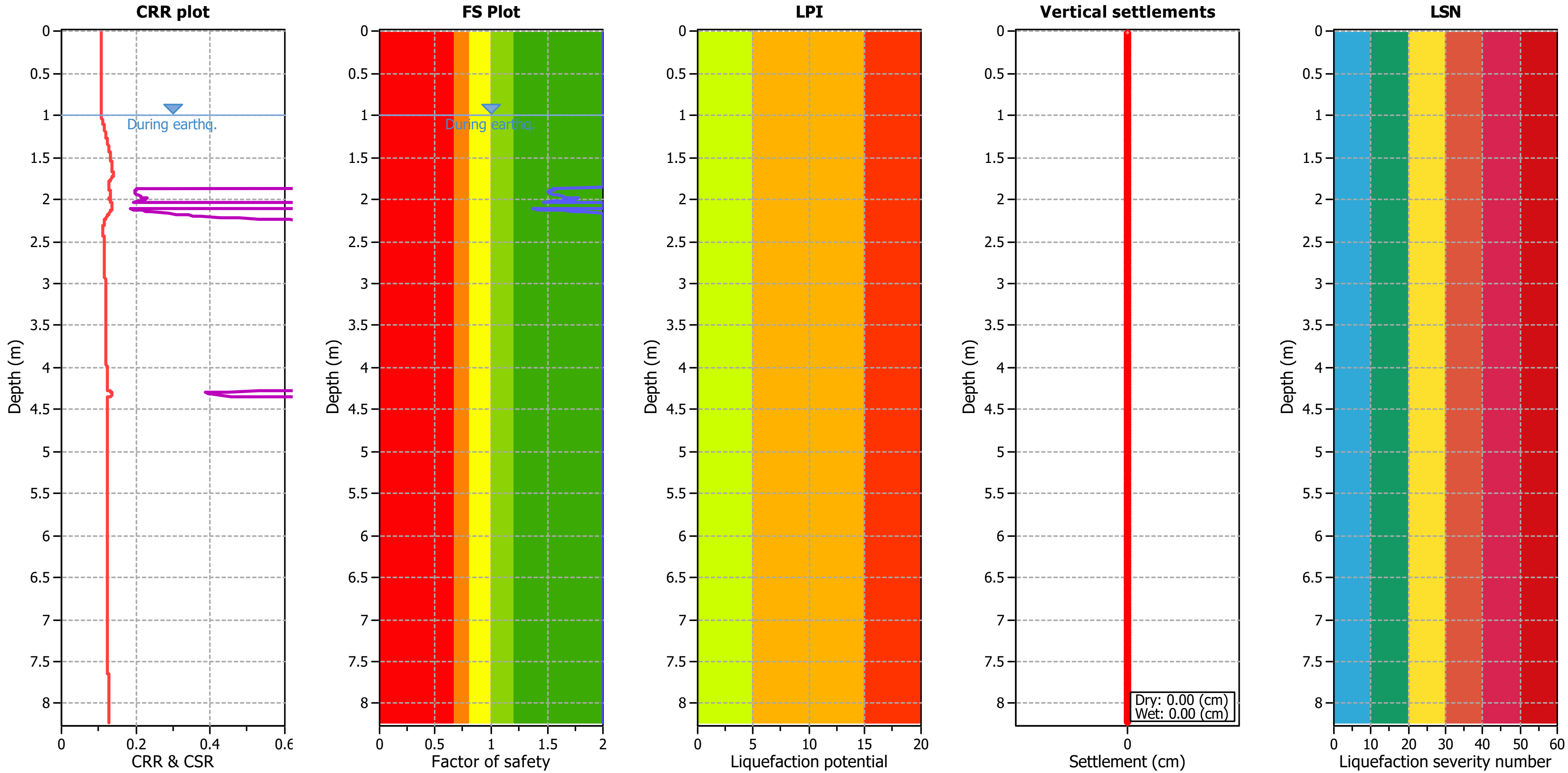
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT111
Total depth: 8.23 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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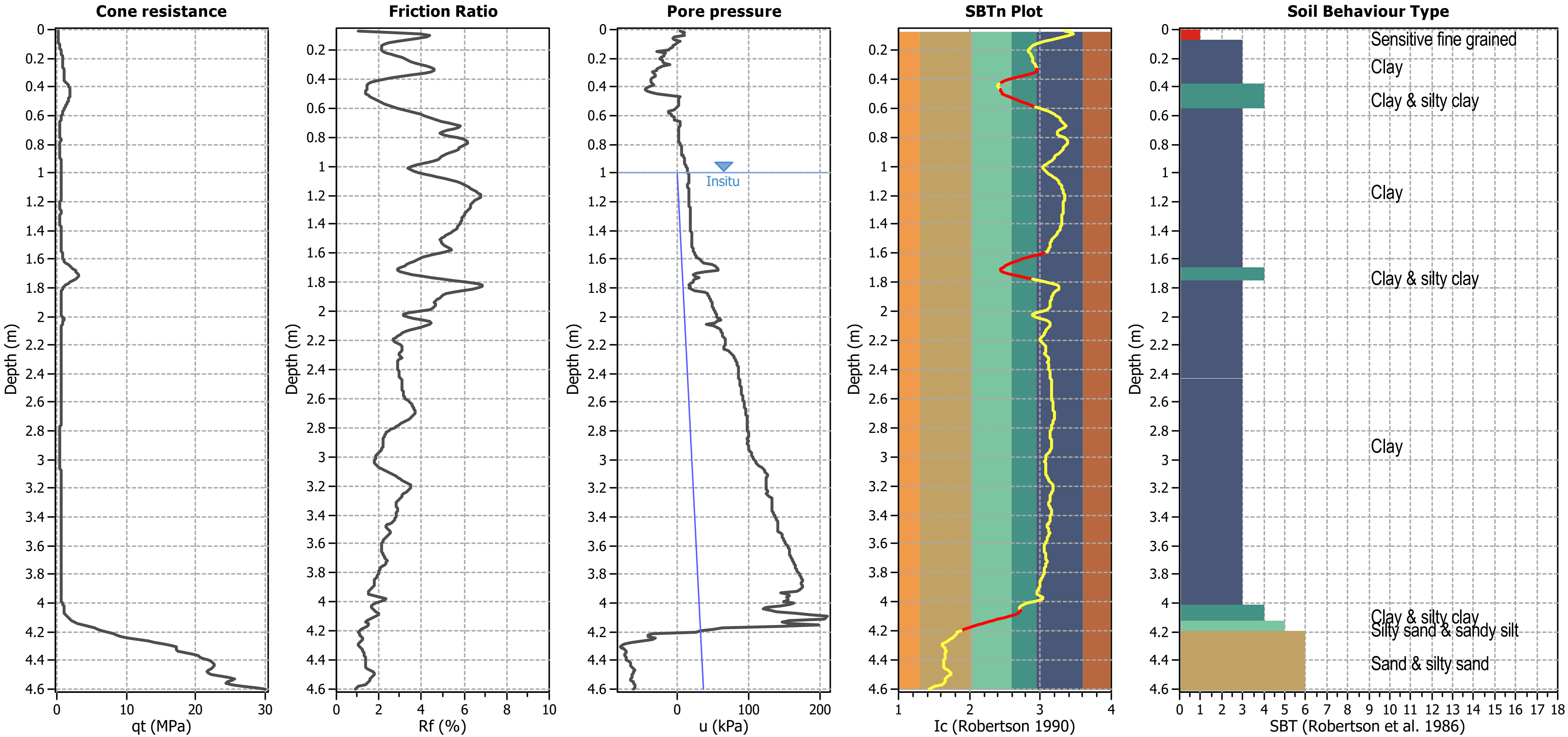
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT112
Total depth: 4.60 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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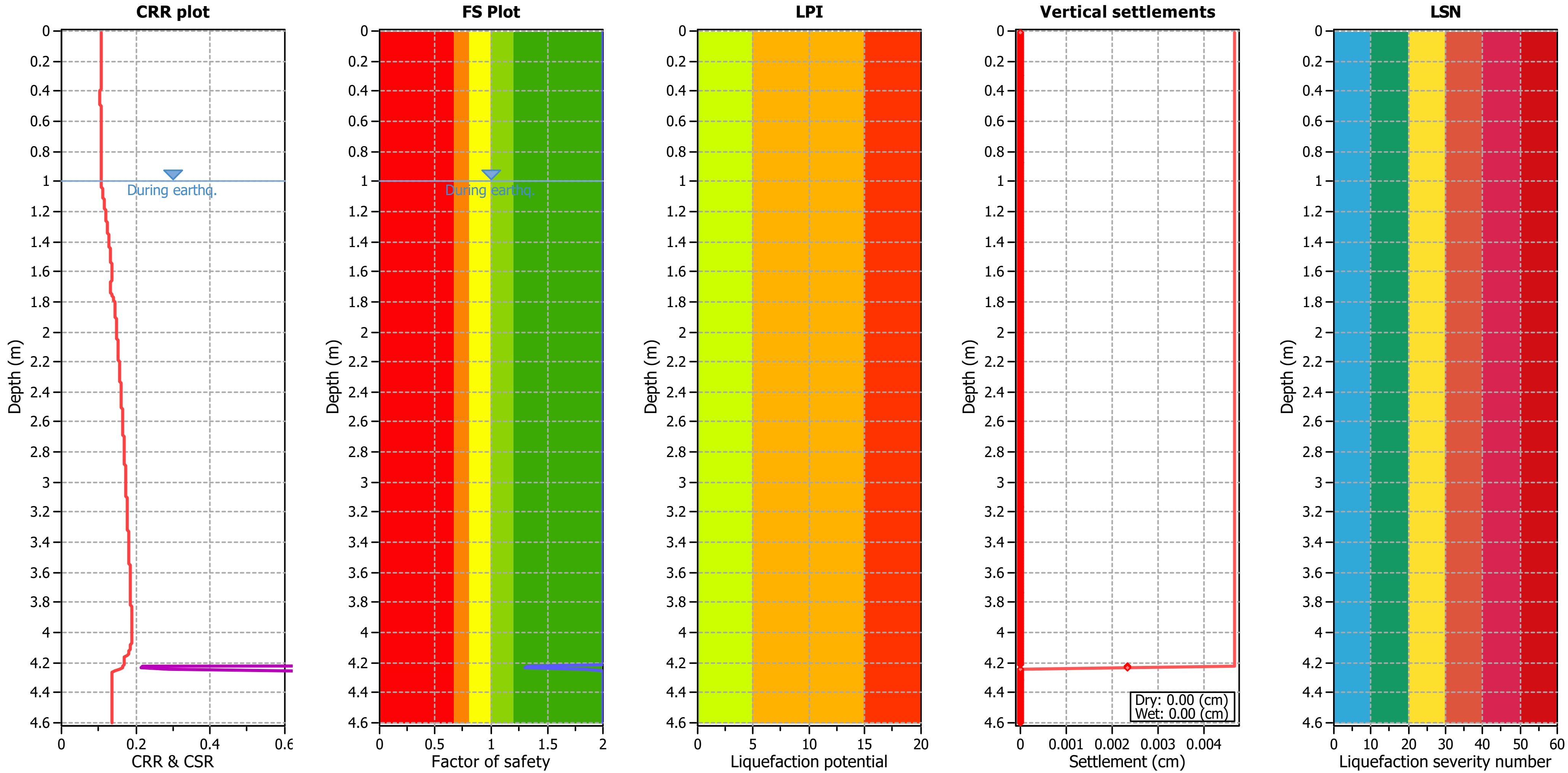
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT112
Total depth: 4.60 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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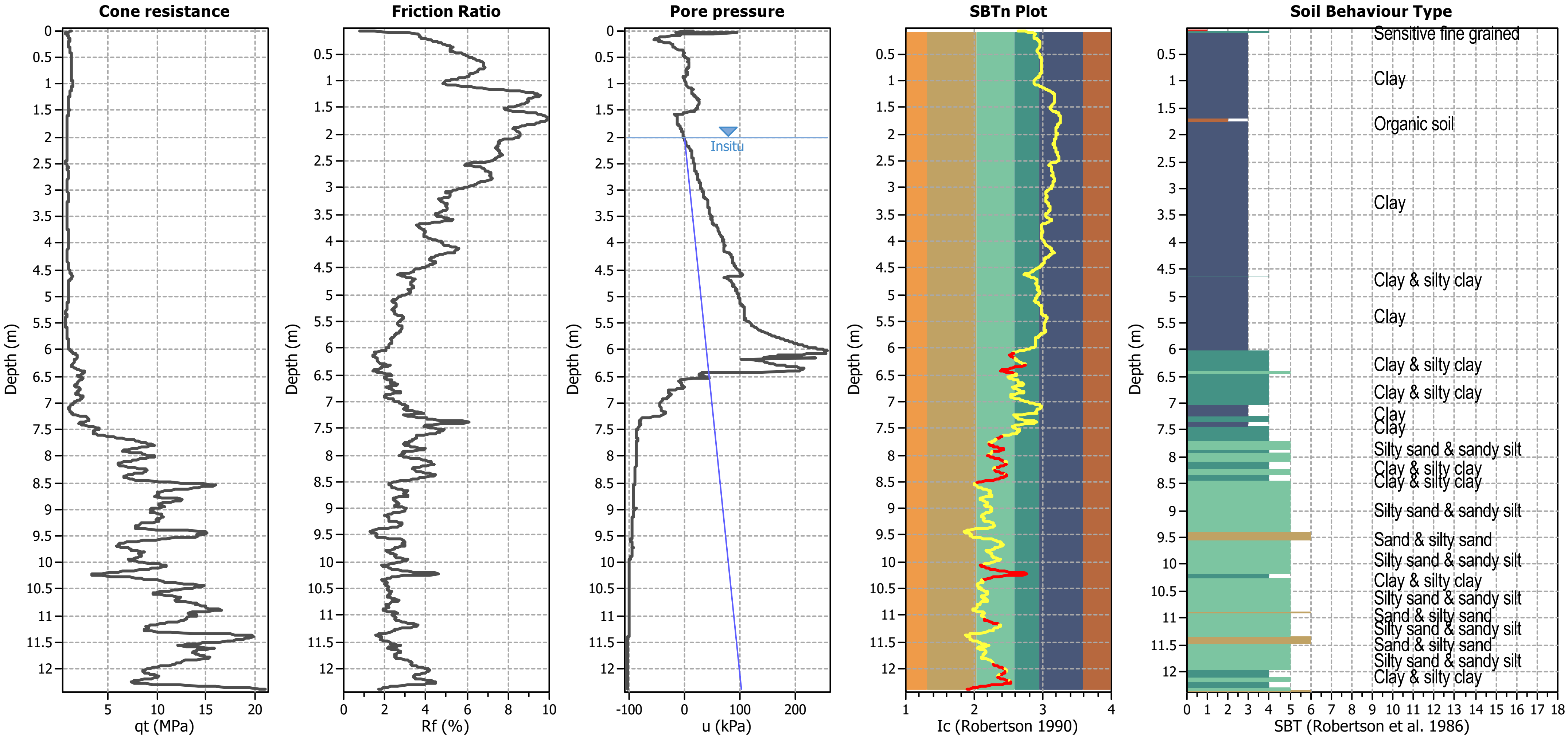
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT113
Total depth: 12.38 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Excavation:	Yes	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Excavation depth:	2.00 m	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.00 kPa	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		

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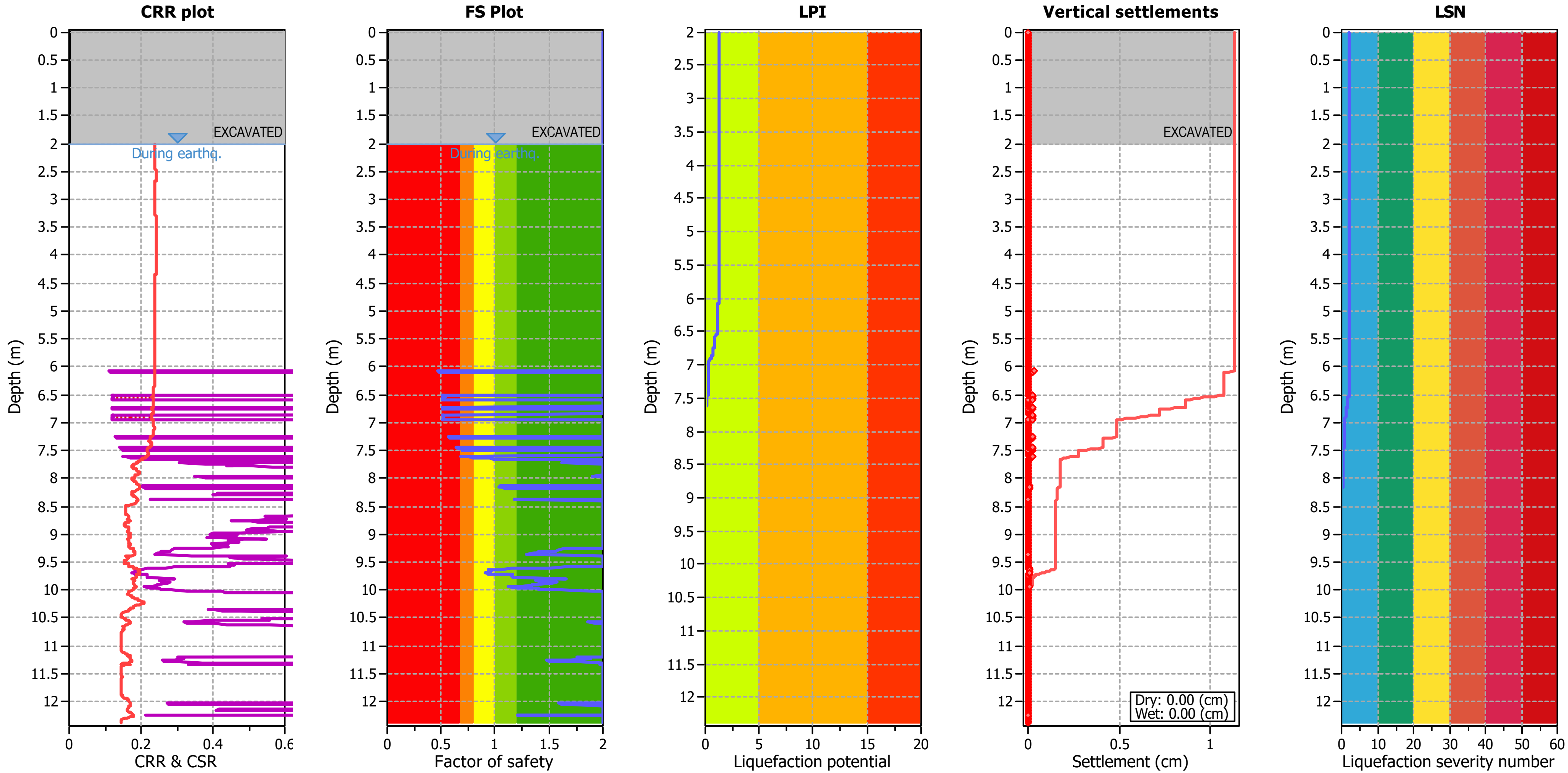
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT113
Total depth: 12.38 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Excavation:	Yes	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Excavation depth:	2.00 m	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.00 kPa	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		

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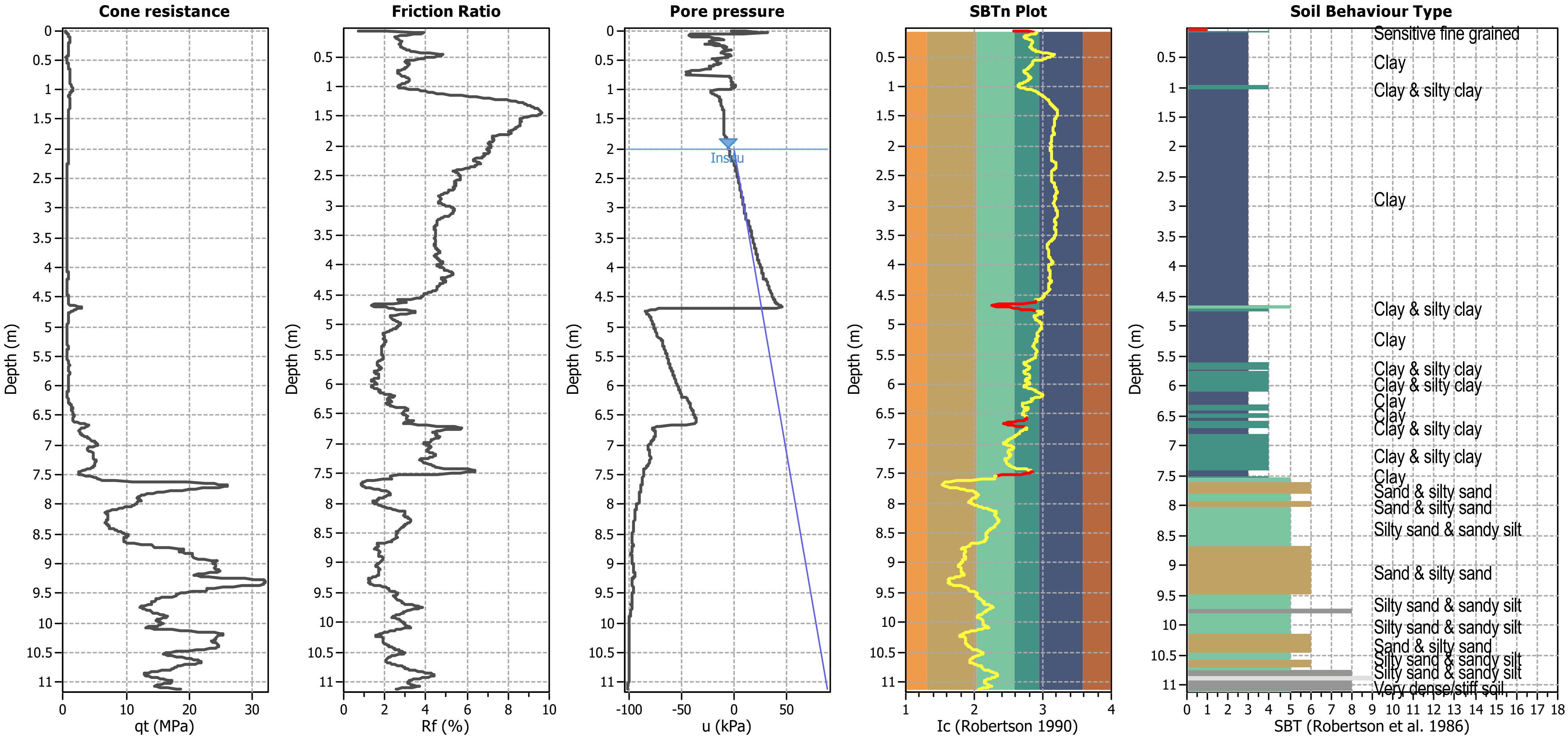
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT114
Total depth: 11.12 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Excavation:	Yes	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Excavation depth:	2.00 m	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.00 kPa	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		

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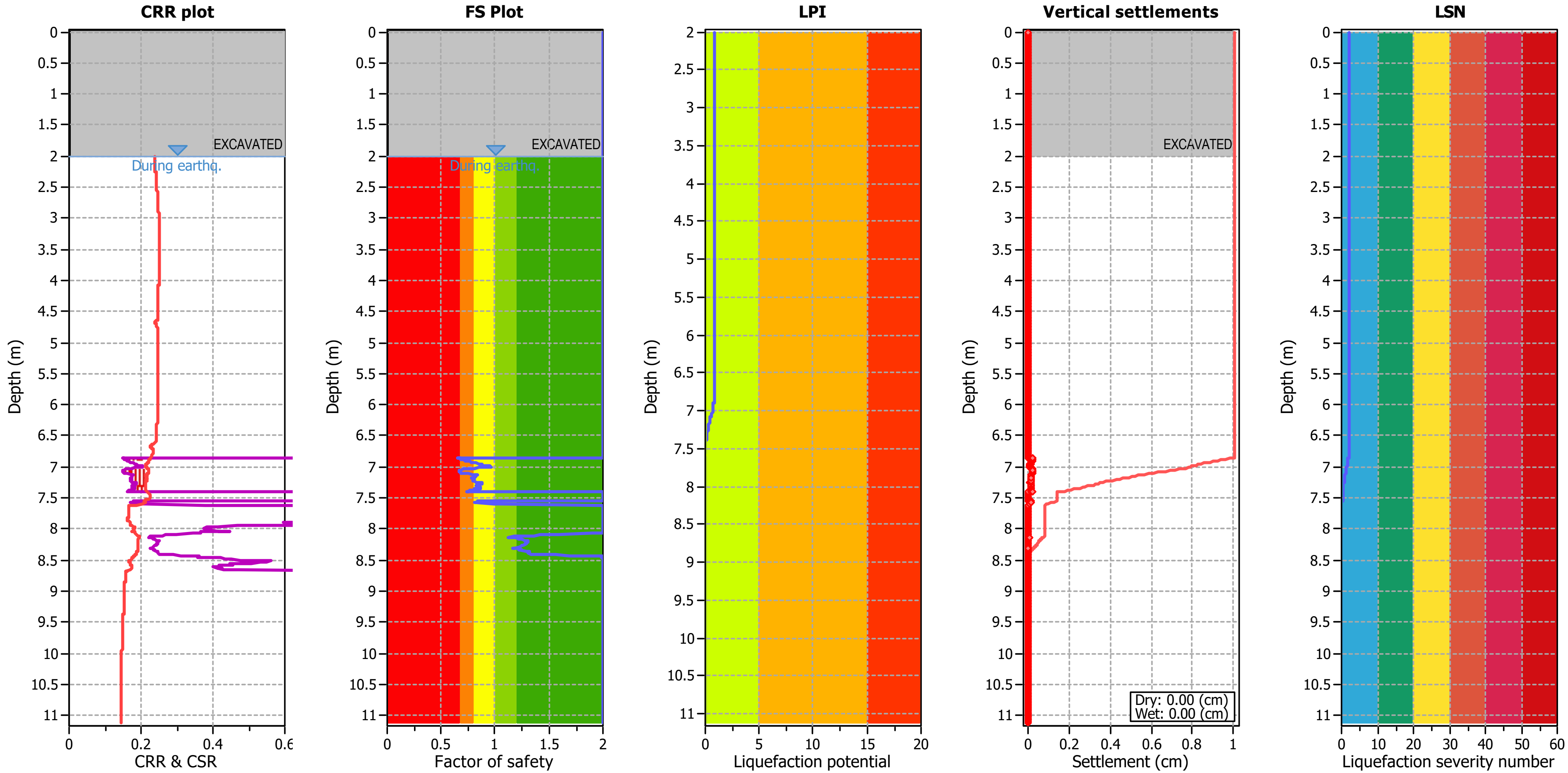
Project: Auckland Surf Park | **CPT: CPT114**
Location: Dairy Flat, Auckland | Total depth: 11.12 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Excavation:	Yes	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Excavation depth:	2.00 m	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.00 kPa	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		

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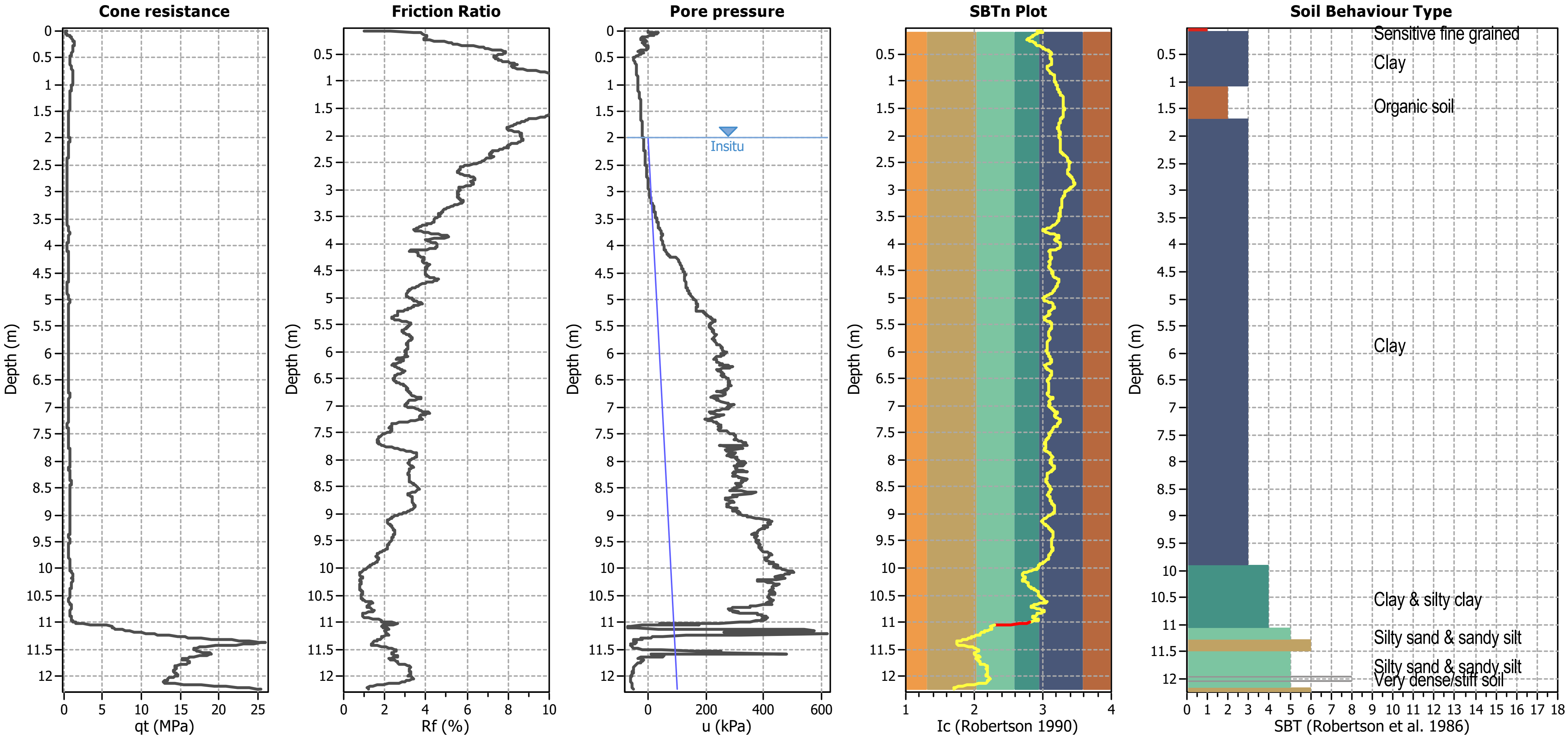
Project: Auckland Surf Park | **CPT: CPT115**
Location: Dairy Flat, Auckland | Total depth: 12.24 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Excavation:	Yes	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Excavation depth:	1.00 m	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.00 kPa	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		

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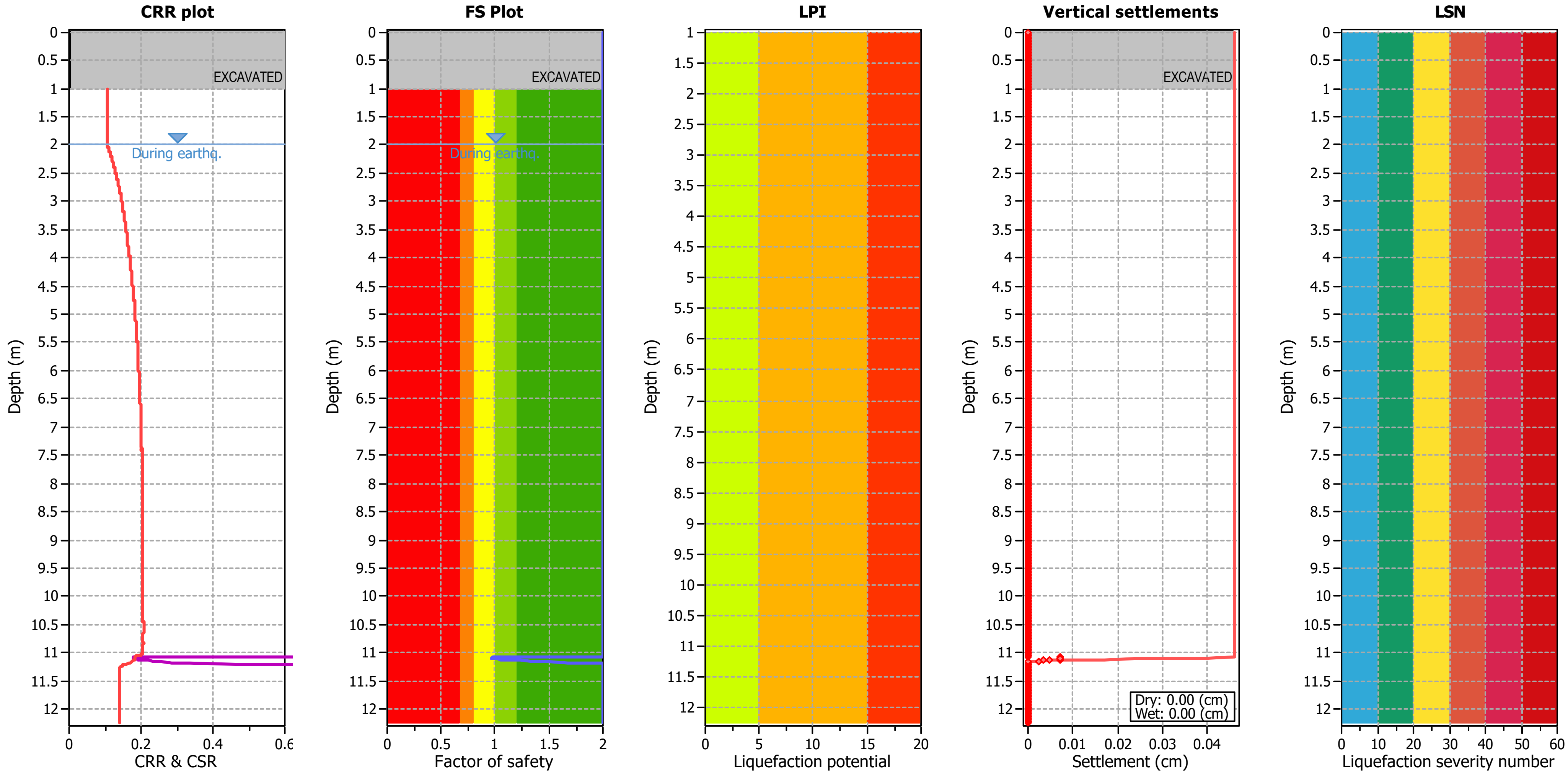
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT115
Total depth: 12.24 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Excavation:	Yes	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Excavation depth:	1.00 m	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.00 kPa	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		

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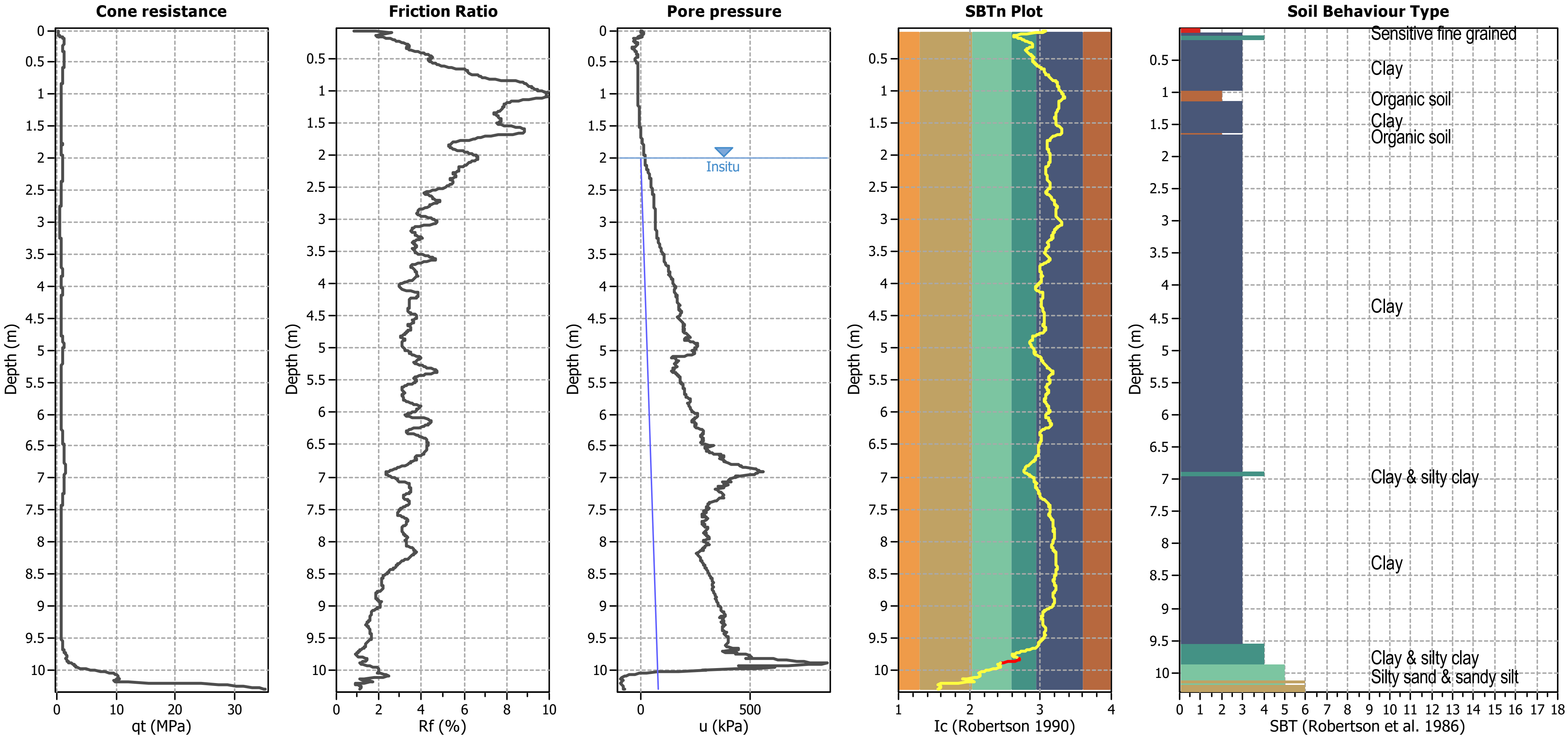
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT116
Total depth: 10.30 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Excavation:	Yes	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Excavation depth:	1.00 m	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.00 kPa	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		

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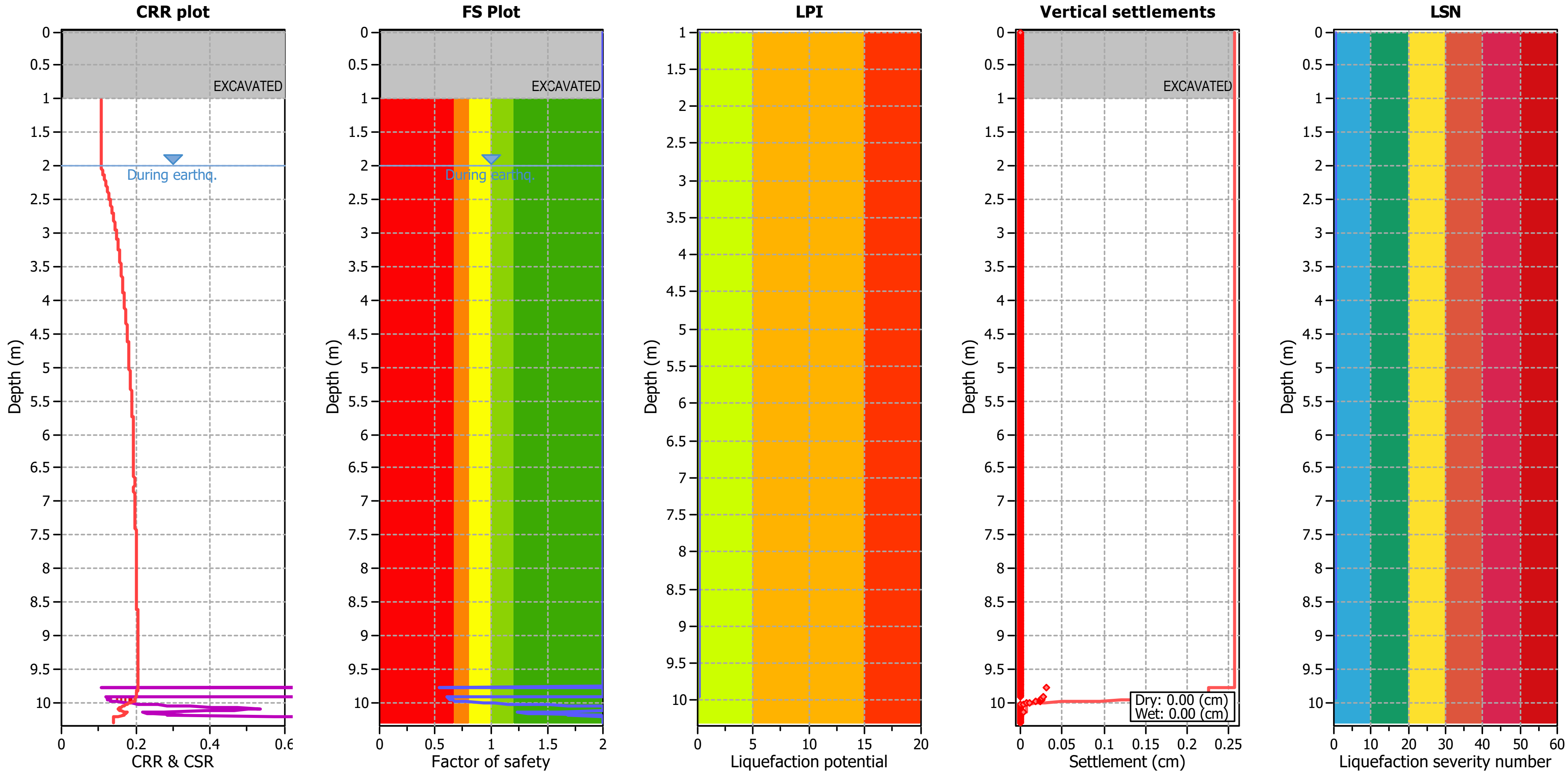
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT116
Total depth: 10.30 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Excavation:	Yes	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Excavation depth:	1.00 m	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.00 kPa	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		

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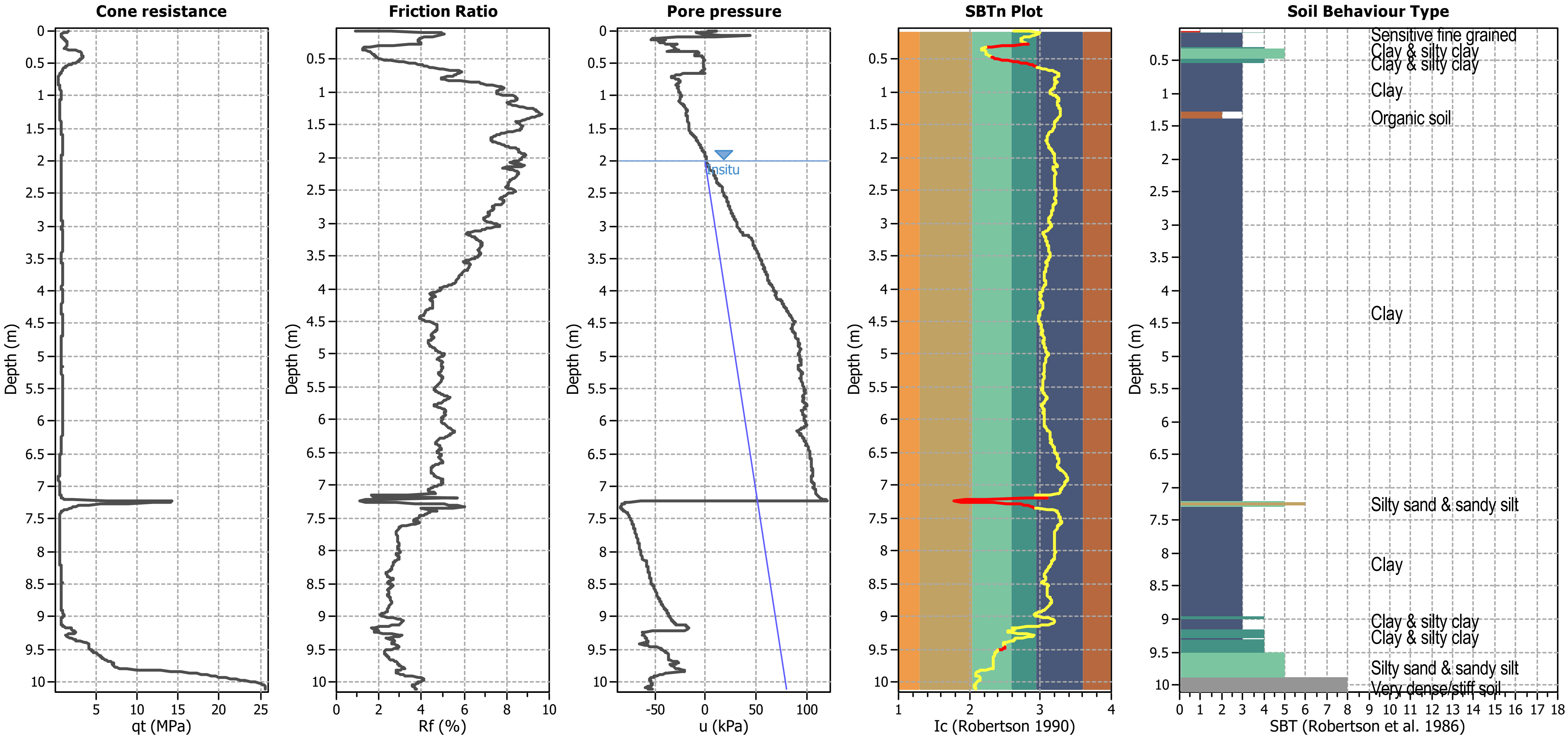
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT117
Total depth: 10.11 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Excavation:	Yes	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Excavation depth:	1.00 m	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.00 kPa	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		

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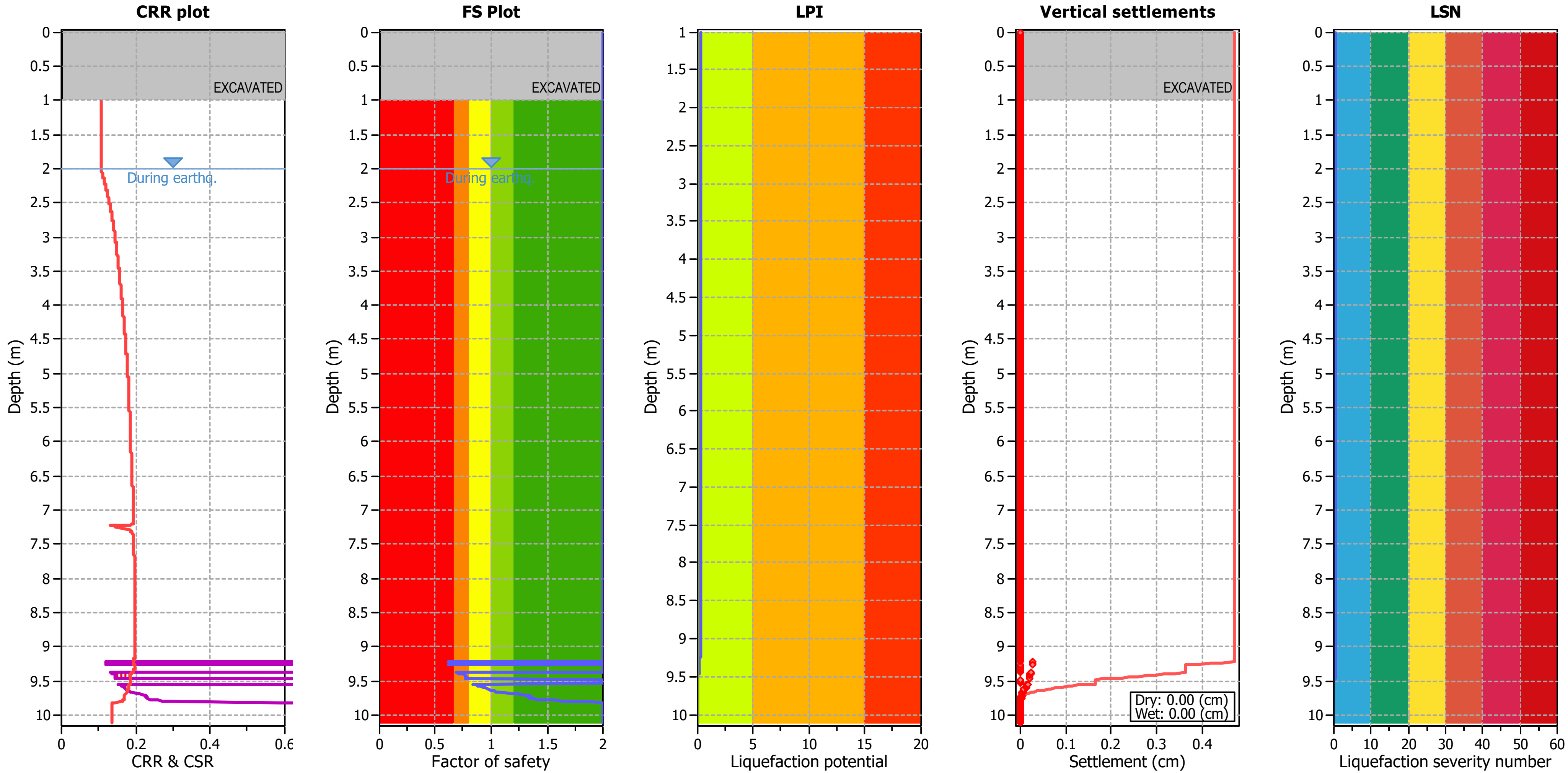
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT117
Total depth: 10.11 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Excavation:	Yes	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Excavation depth:	1.00 m	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.00 kPa	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		

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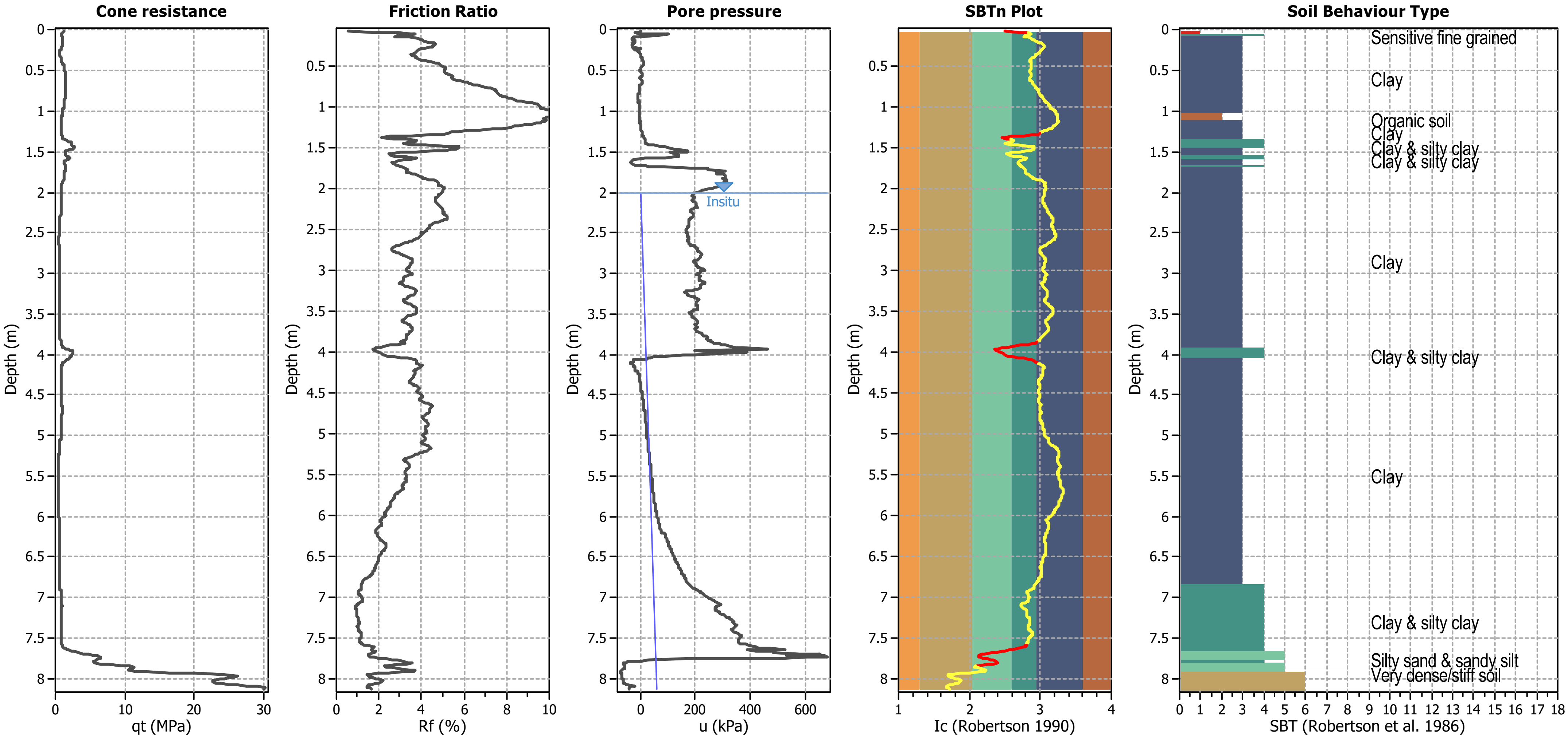
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT118
Total depth: 8.13 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Excavation:	Yes	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Excavation depth:	2.00 m	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.00 kPa	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		

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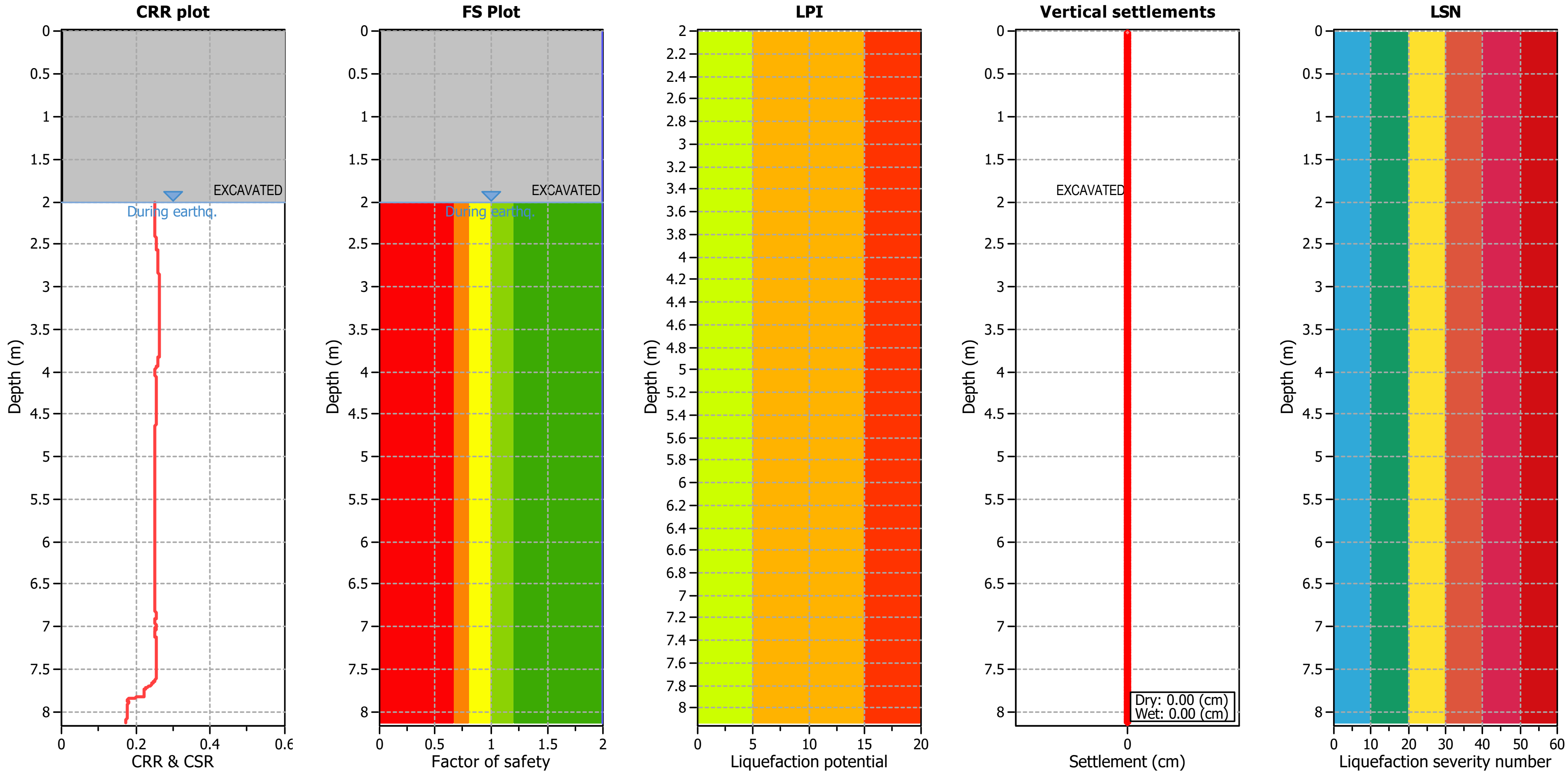
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT118
Total depth: 8.13 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Excavation:	Yes	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Excavation depth:	2.00 m	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.00 kPa	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		

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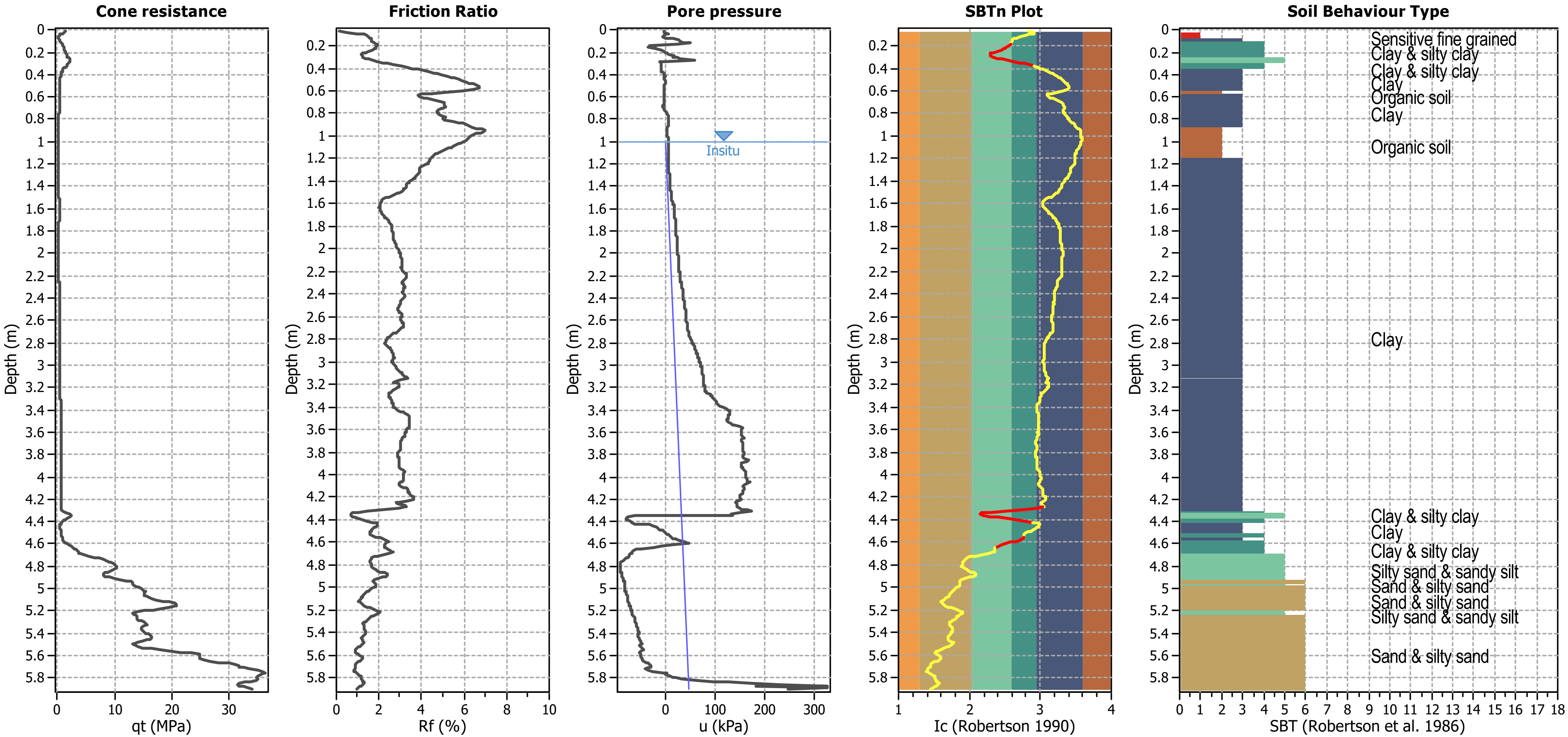
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT119
Total depth: 5.90 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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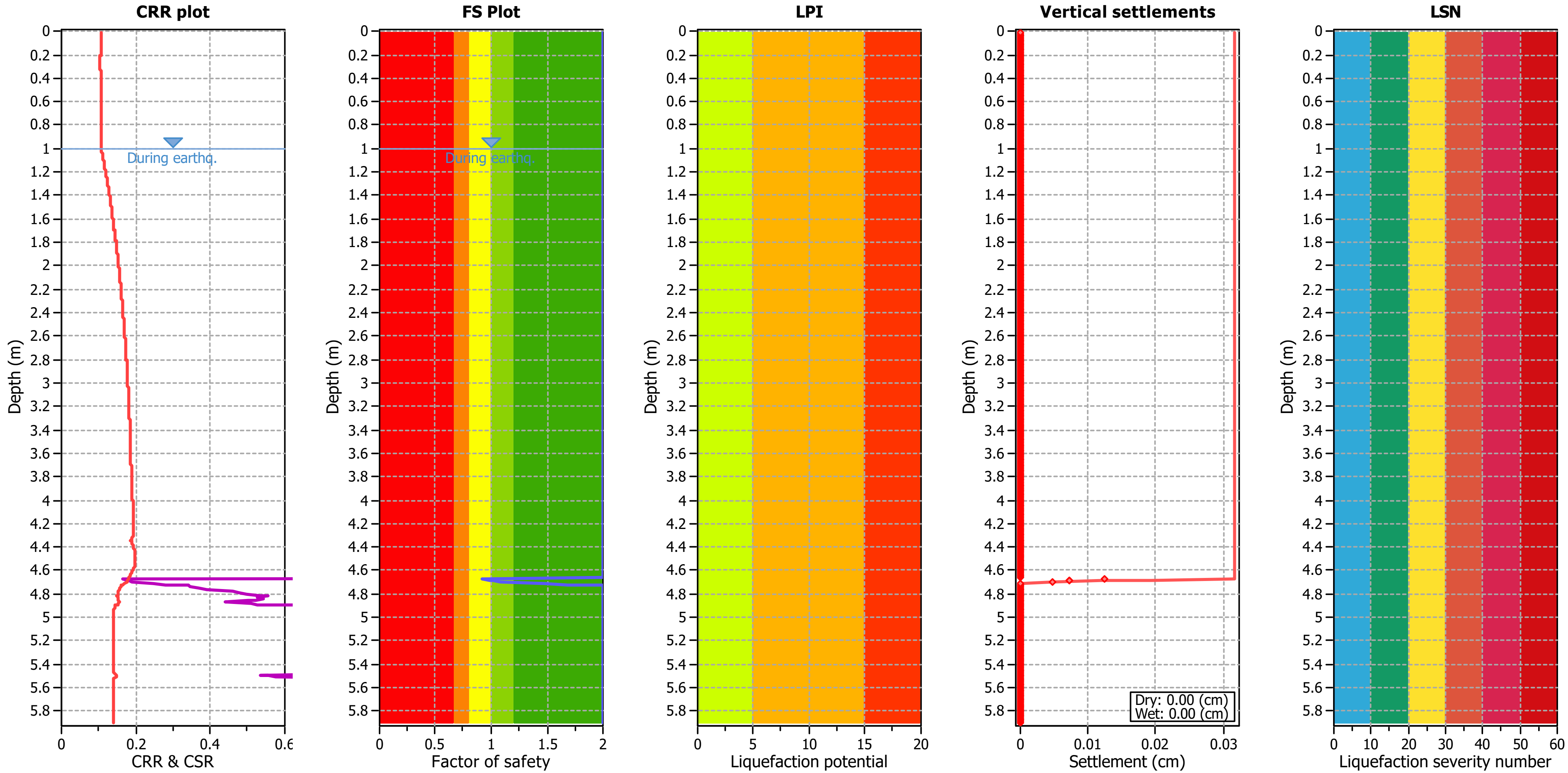
Project: Auckland Surf Park **CPT: CPT119**
Location: Dairy Flat, Auckland Total depth: 5.90 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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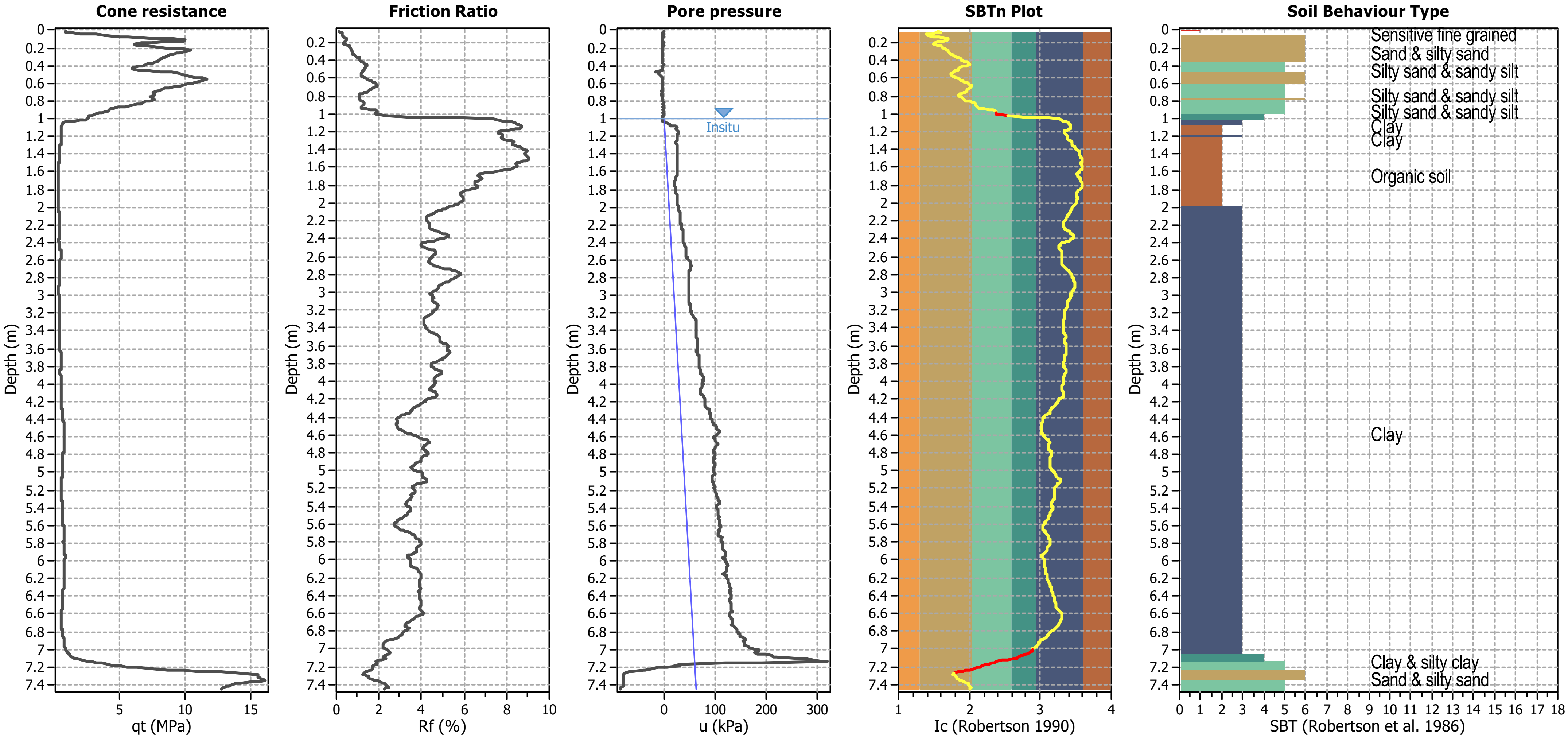
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT120
Total depth: 7.45 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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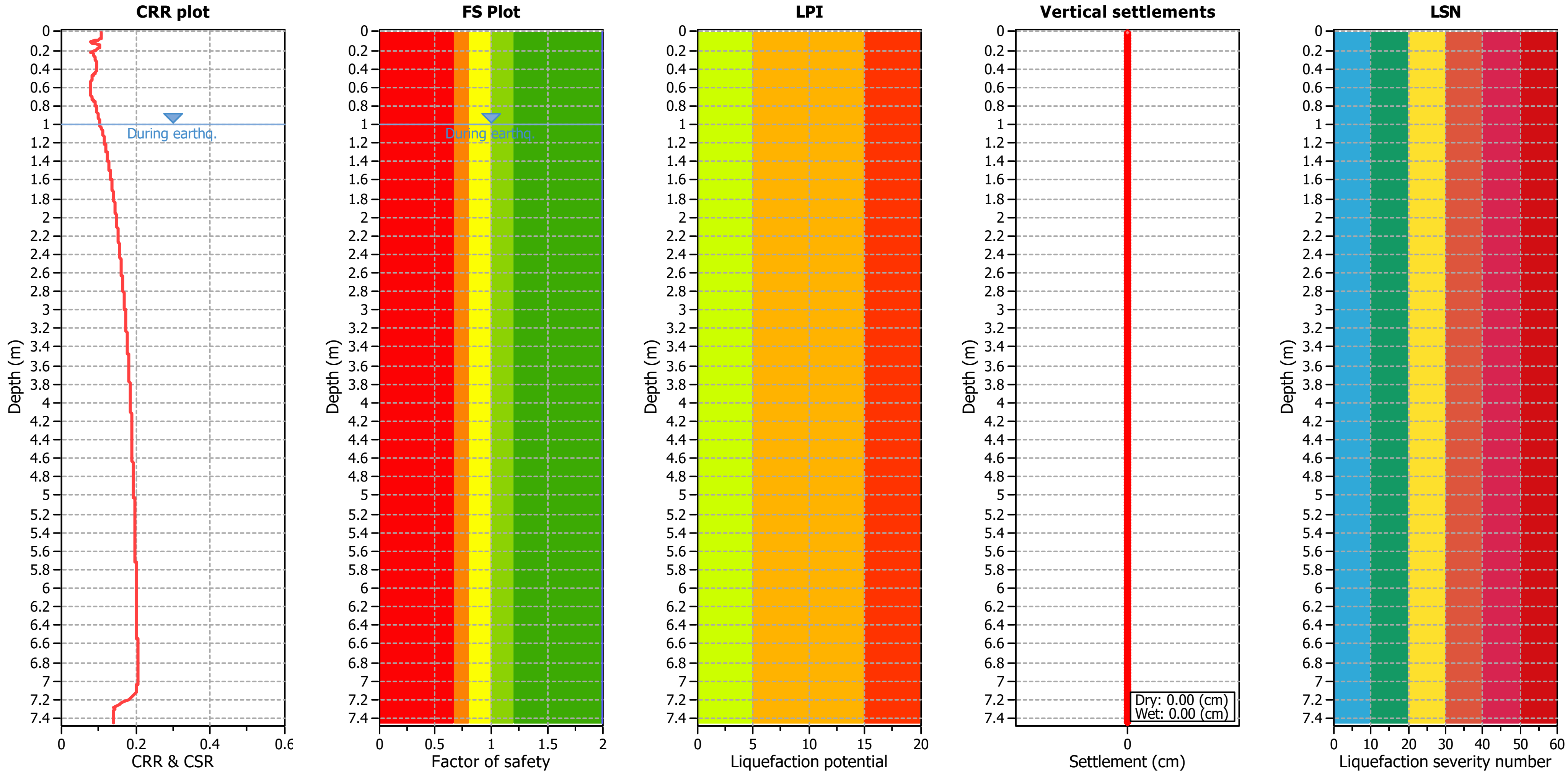
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT120
Total depth: 7.45 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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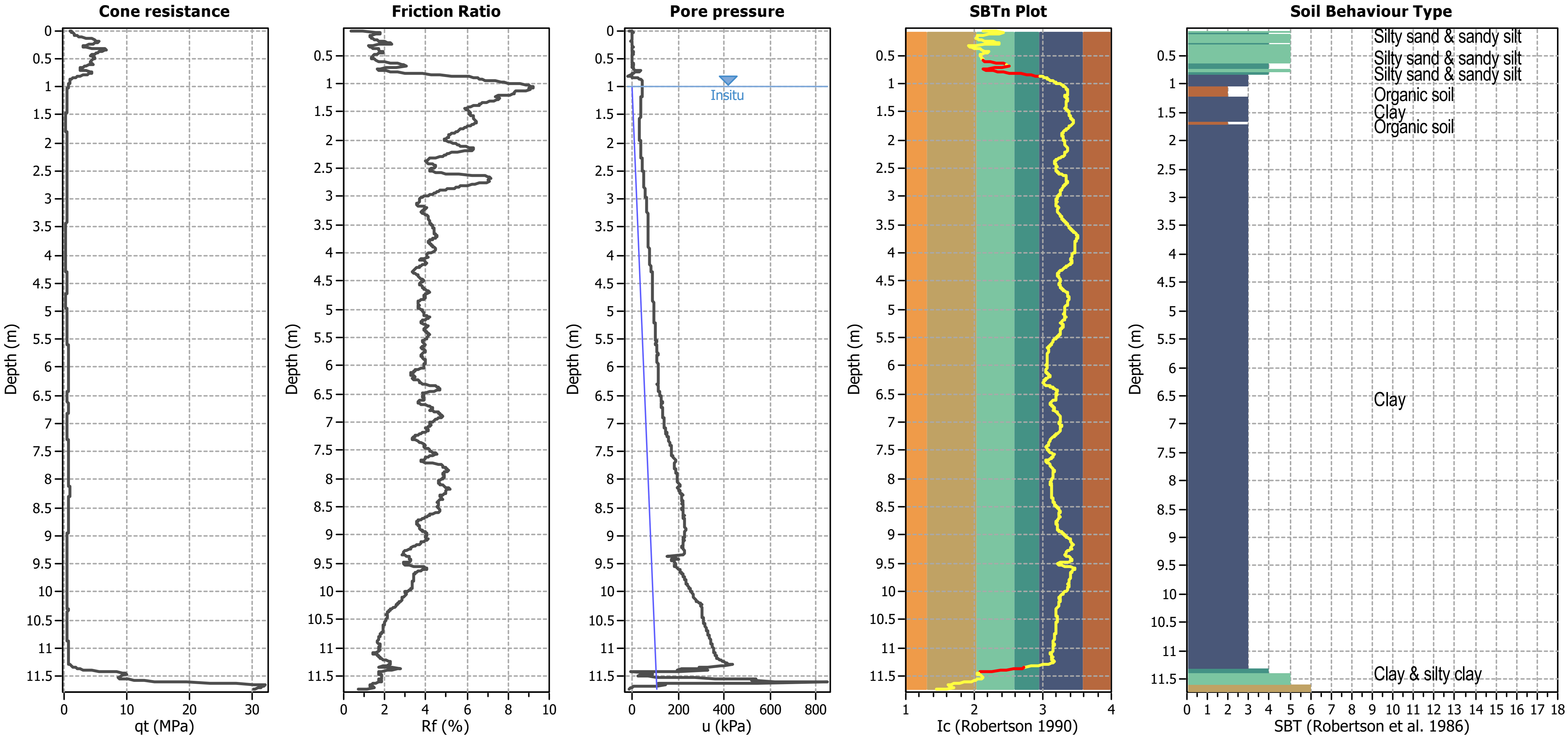
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

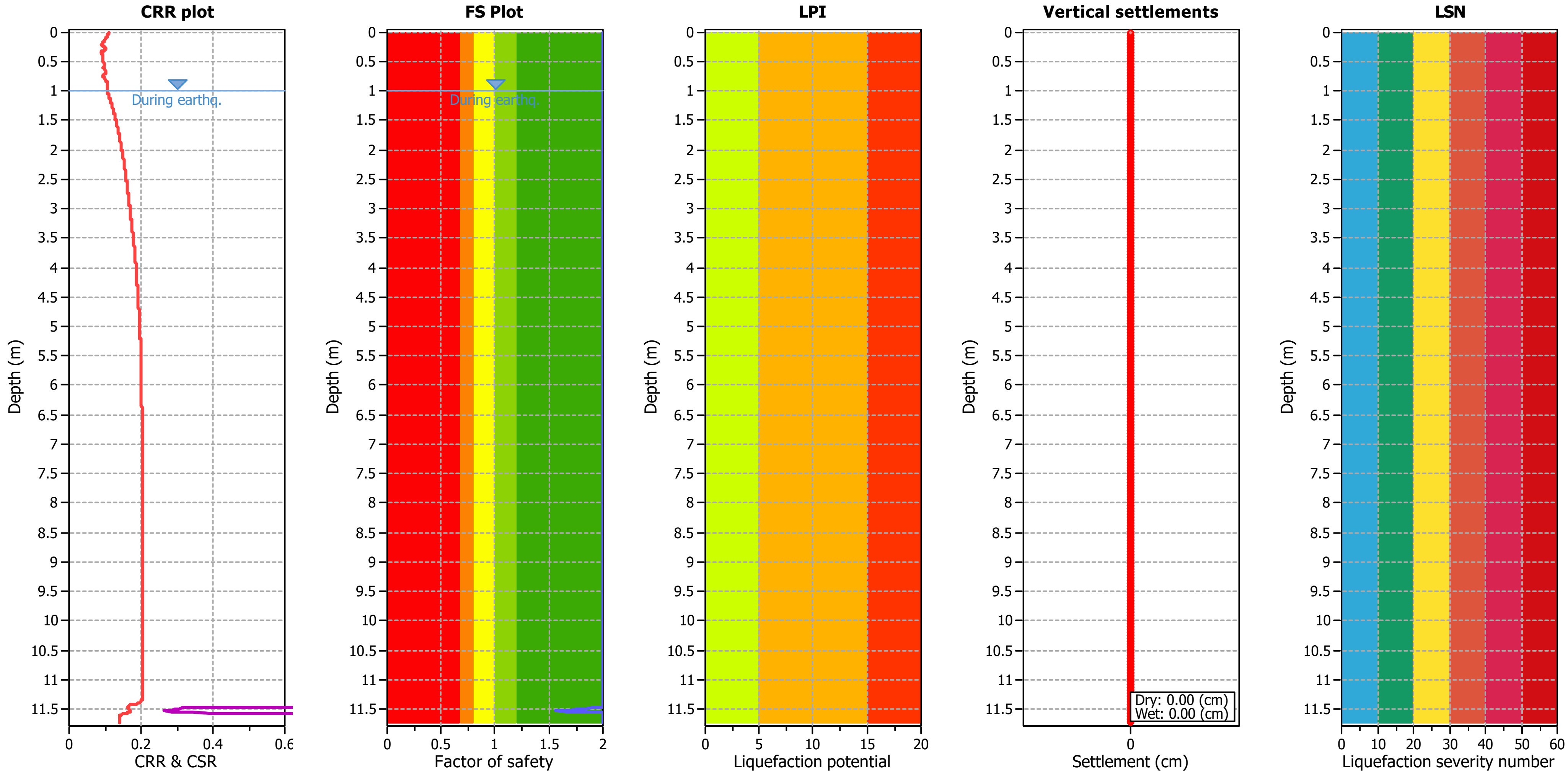
CPT: CPT121
Total depth: 11.73 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT121
Total depth: 11.73 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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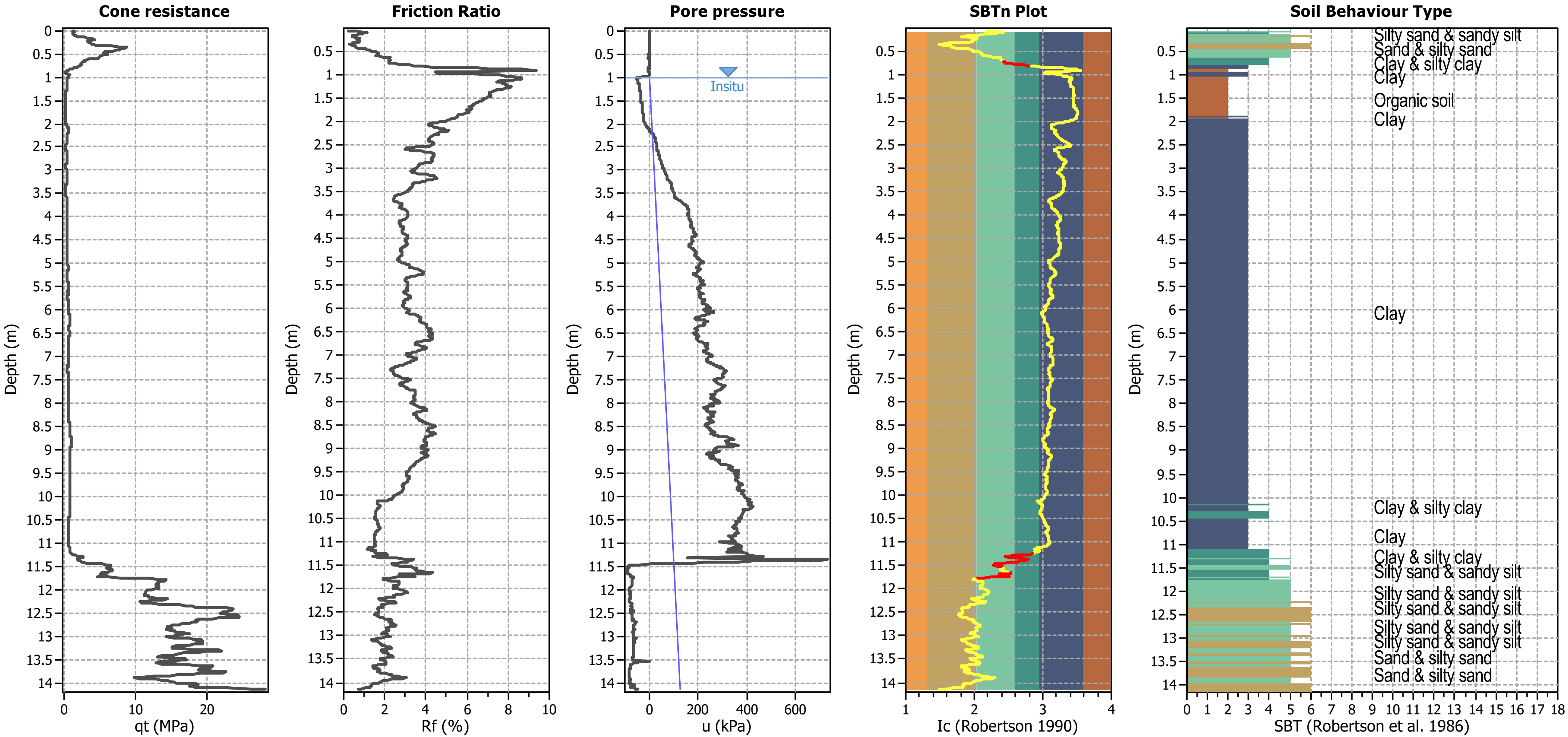
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

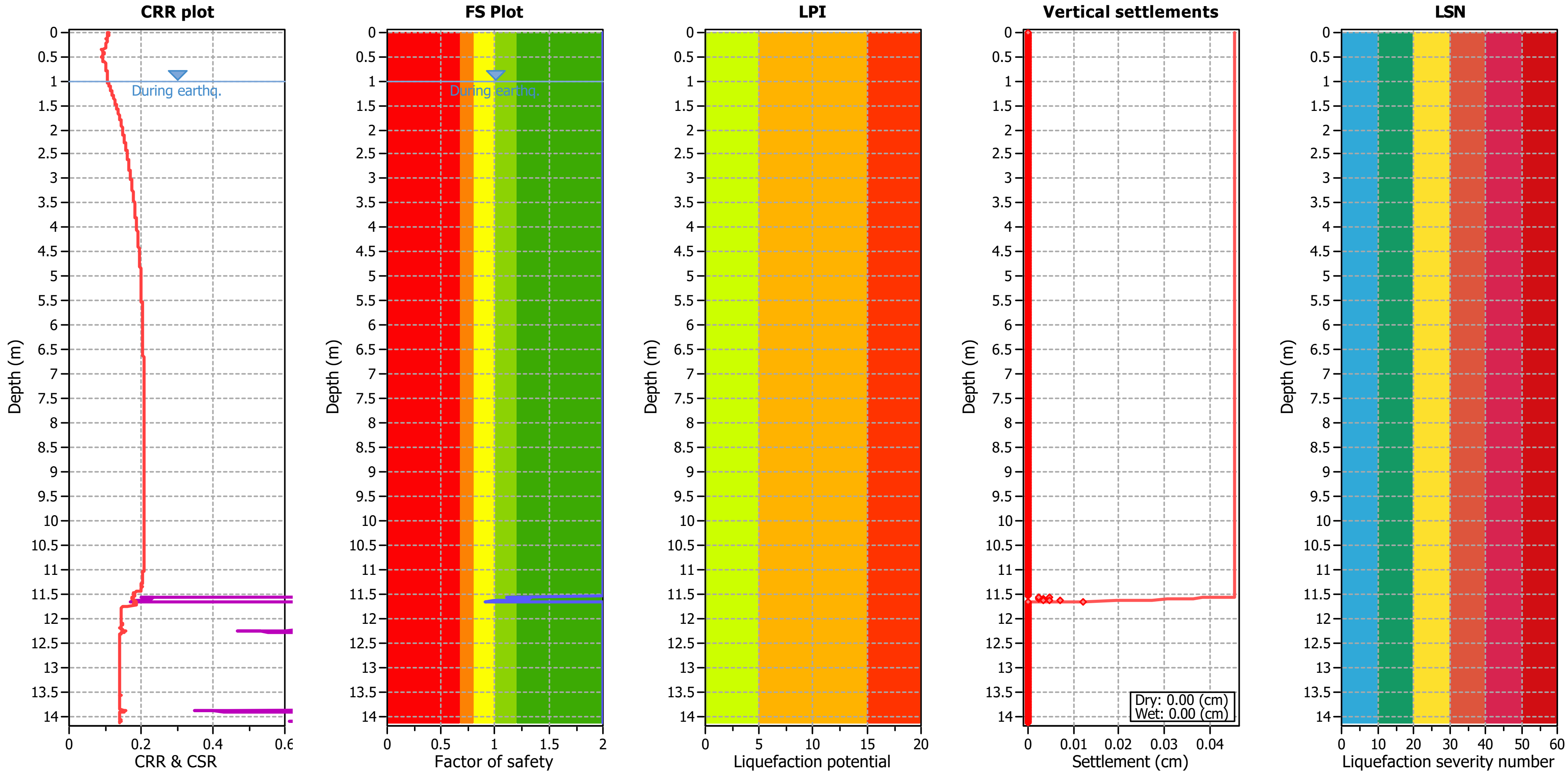
CPT: CPT122
Total depth: 14.14 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

Project: Auckland Surf Park
Location: Dairy Flat, Auckland

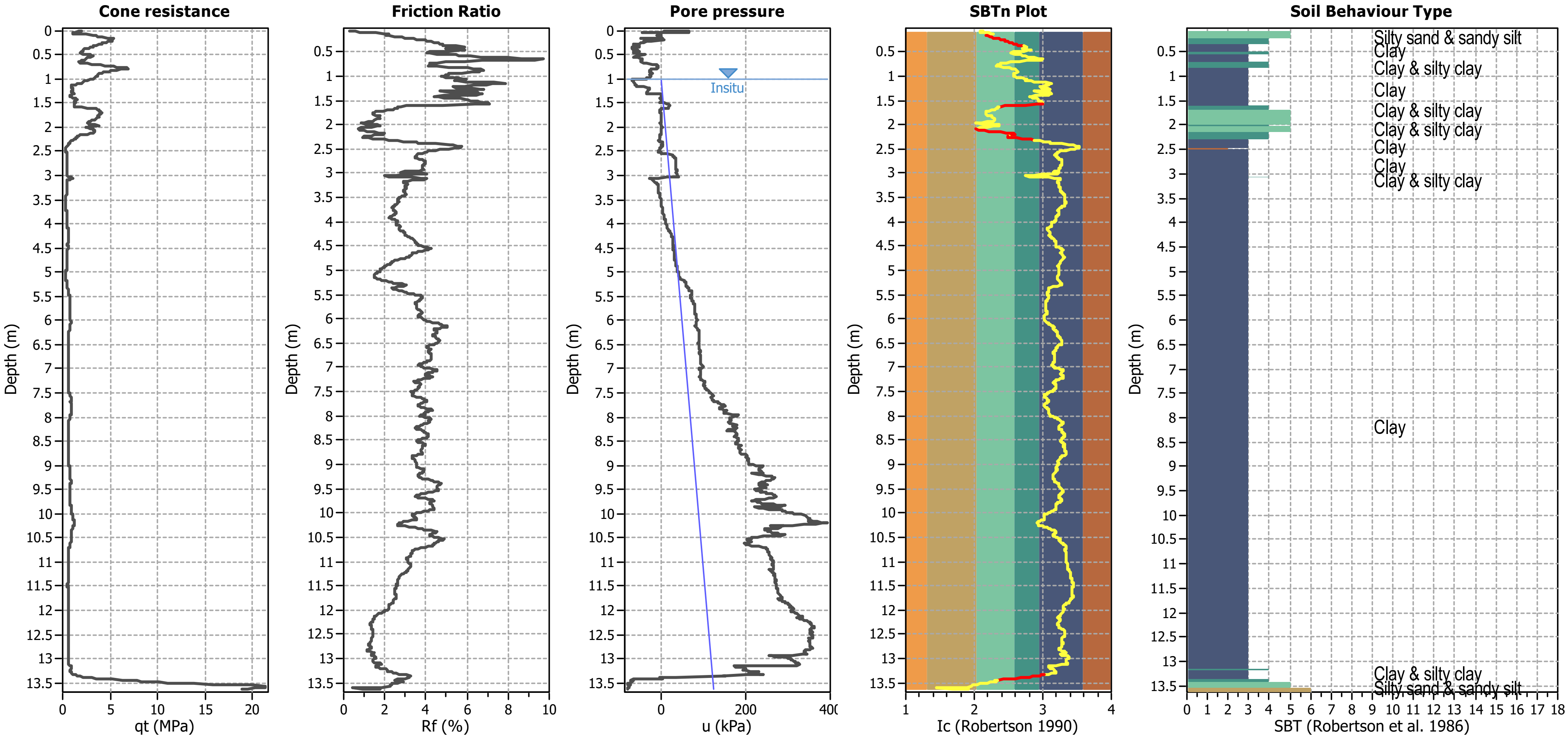
CPT: CPT122
Total depth: 14.14 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT123
Total depth: 13.63 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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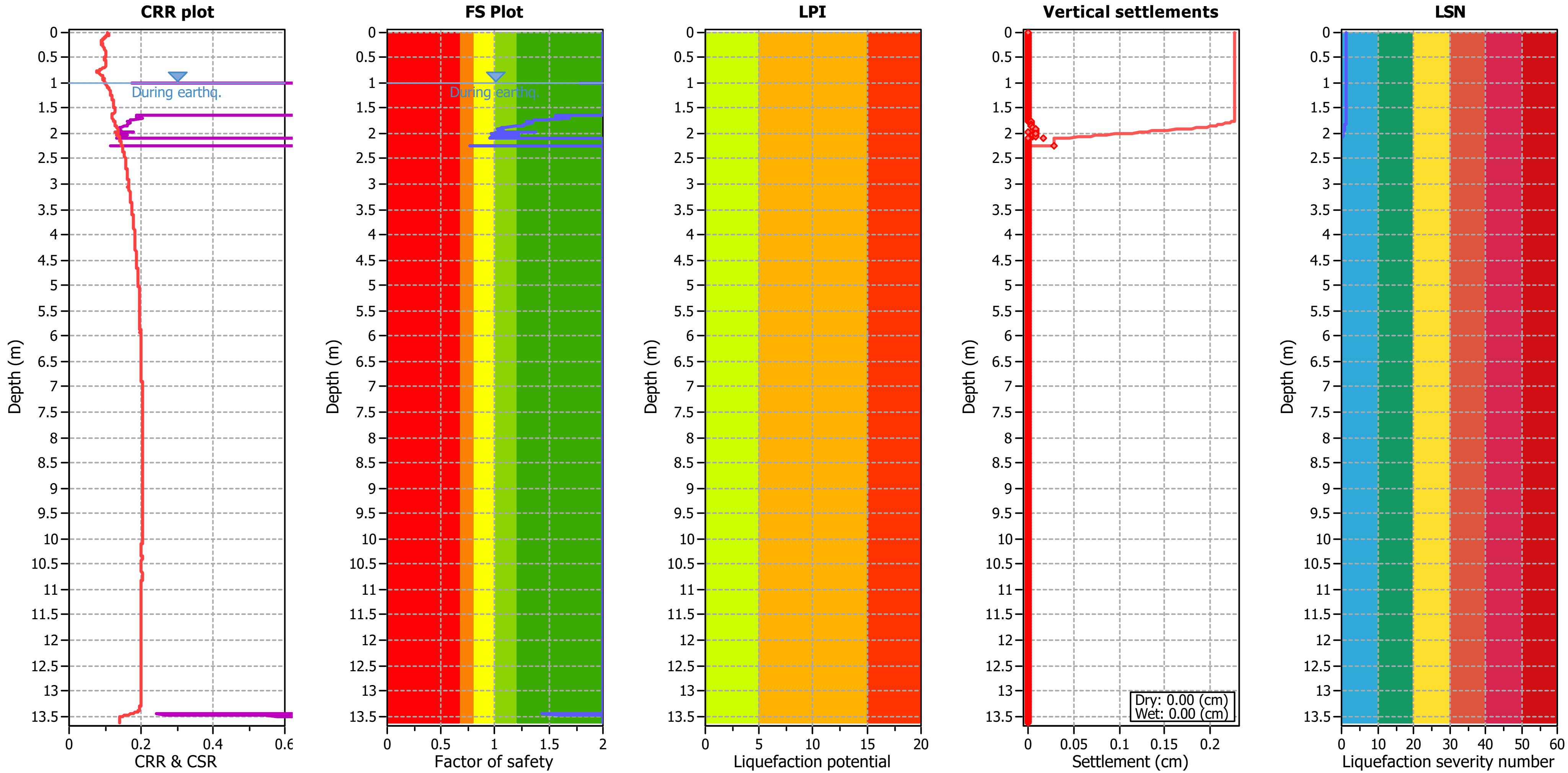
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT123
Total depth: 13.63 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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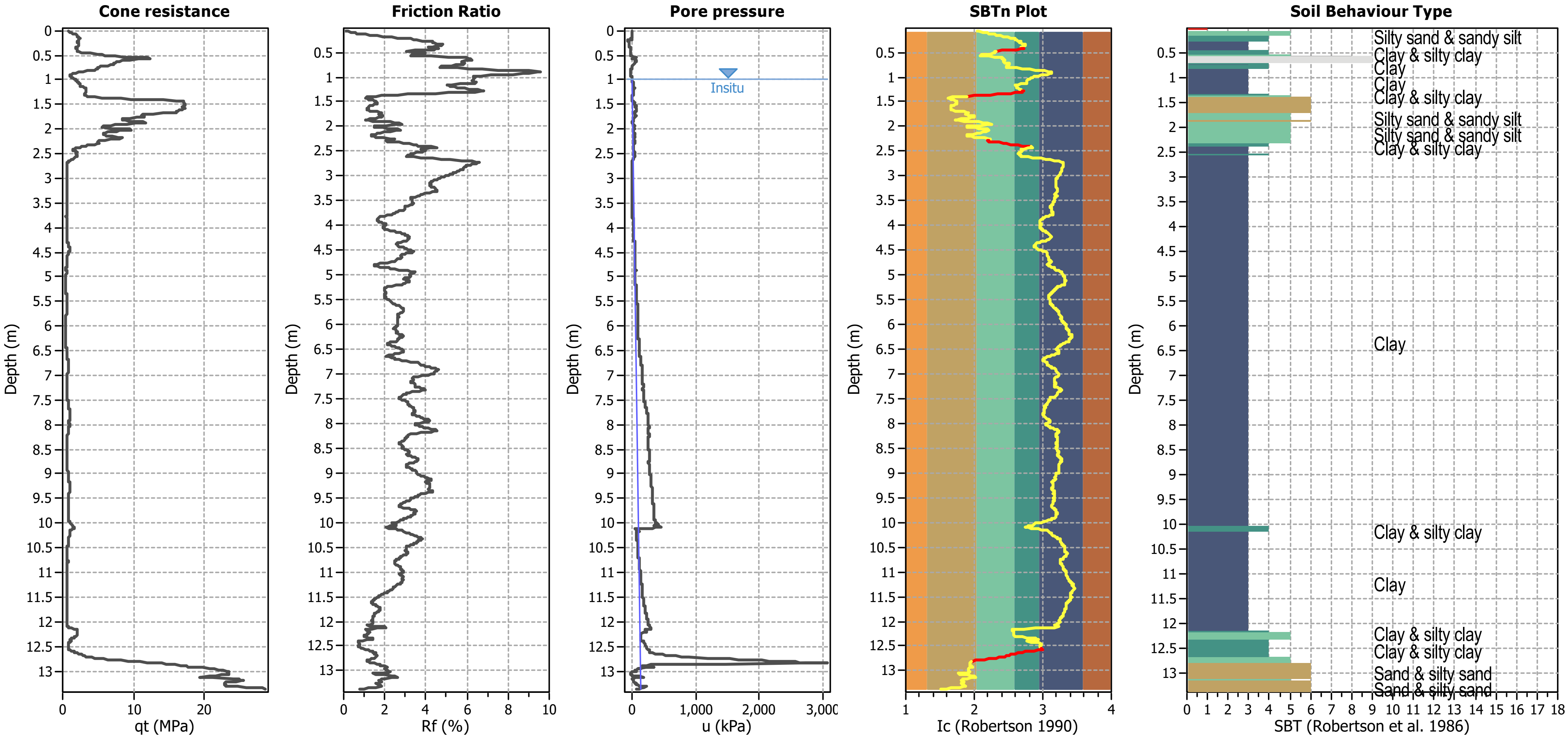
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT124
Total depth: 13.37 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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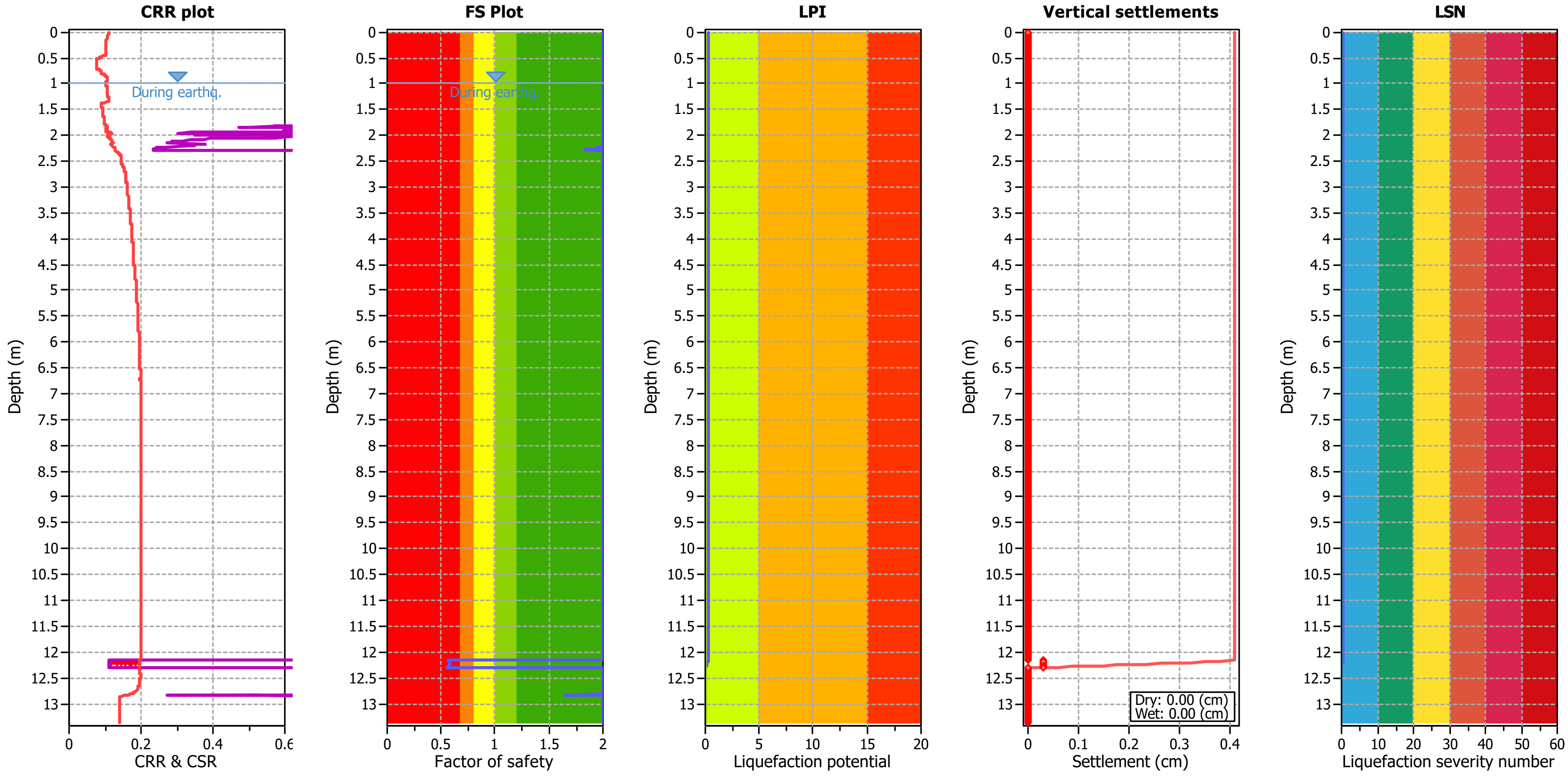
Project: **Auckland Surf Park** **CPT: CPT124**
Location: **Dairy Flat, Auckland** Total depth: 13.37 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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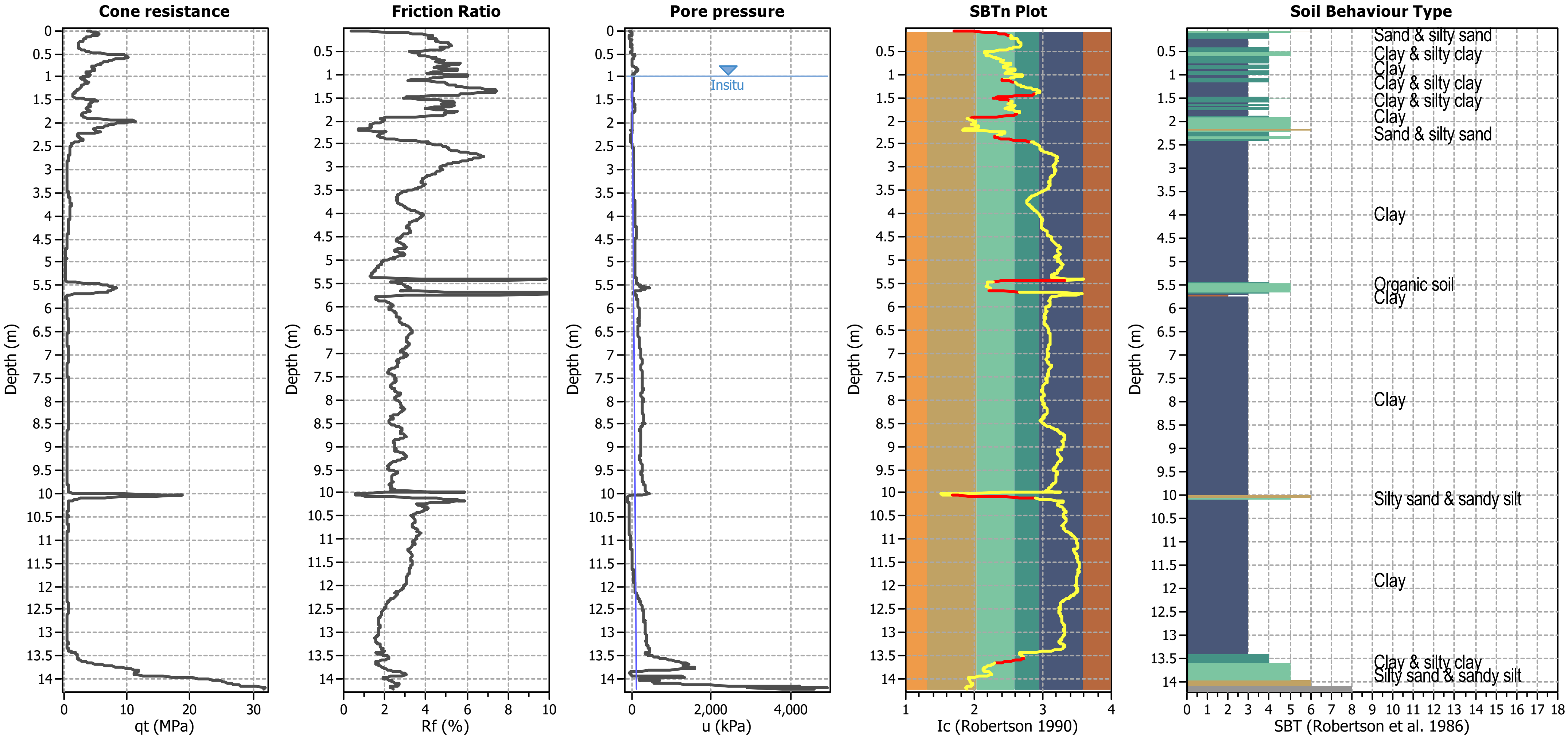
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT125
Total depth: 14.22 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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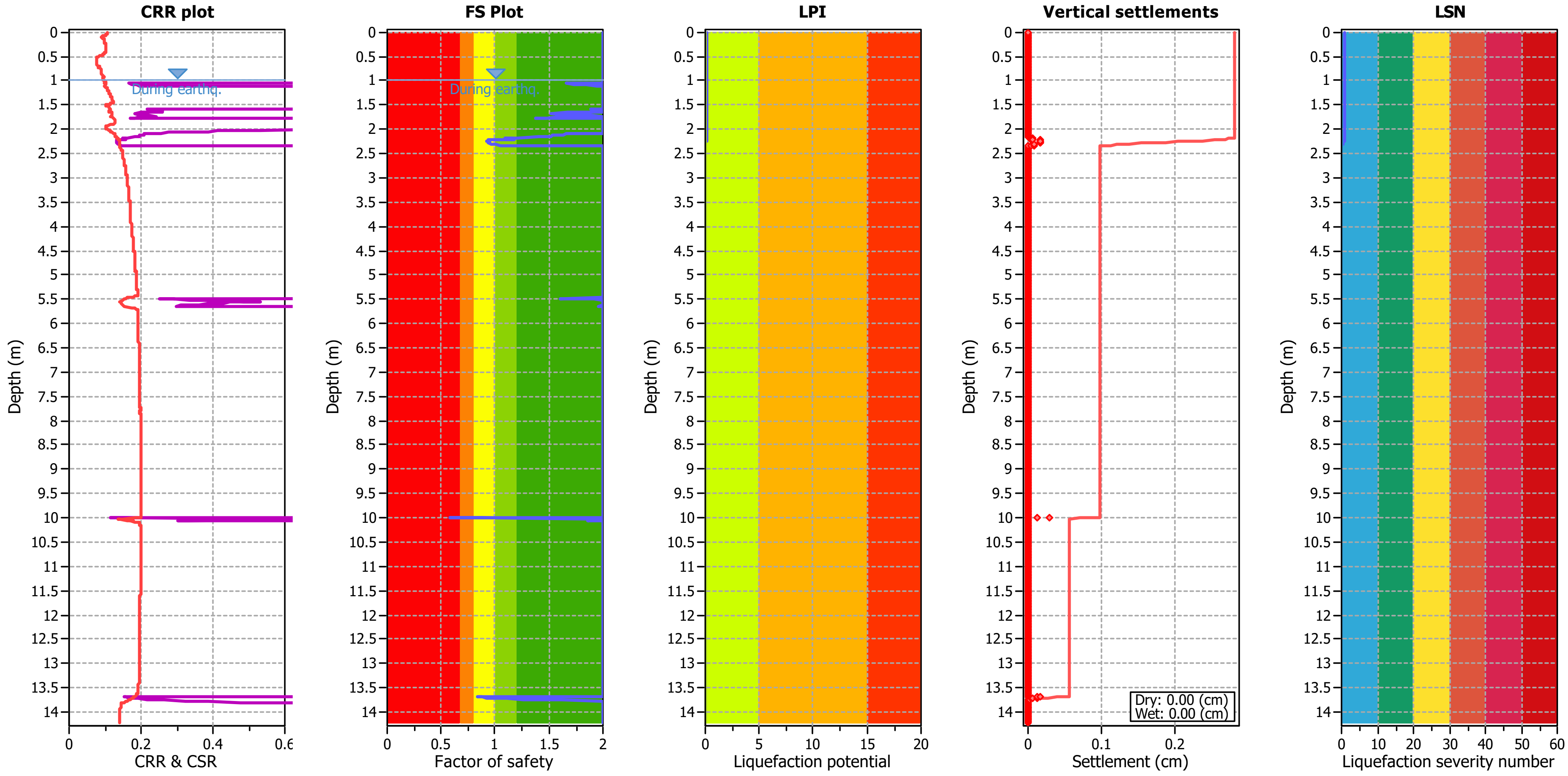
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT125
Total depth: 14.22 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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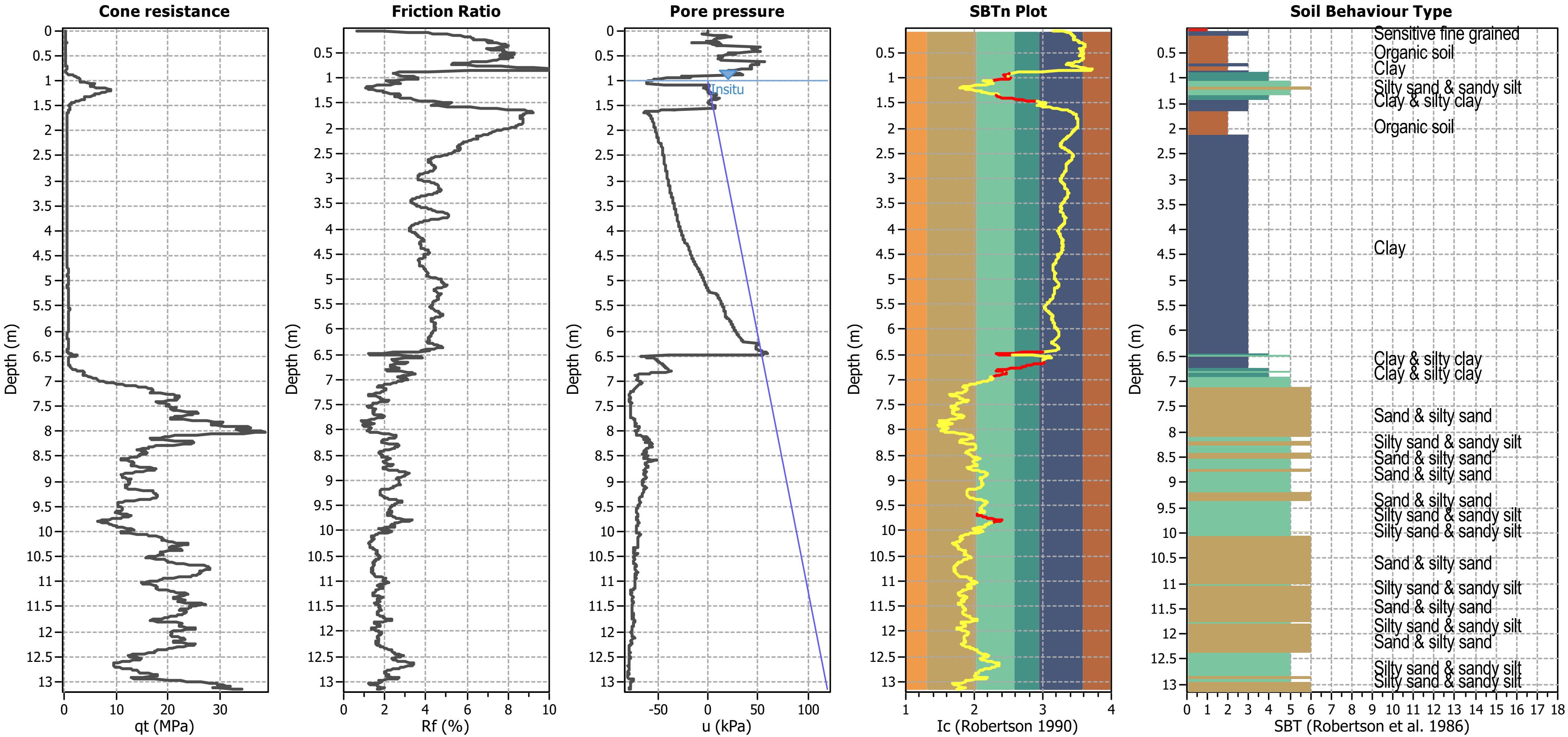
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT126
Total depth: 13.15 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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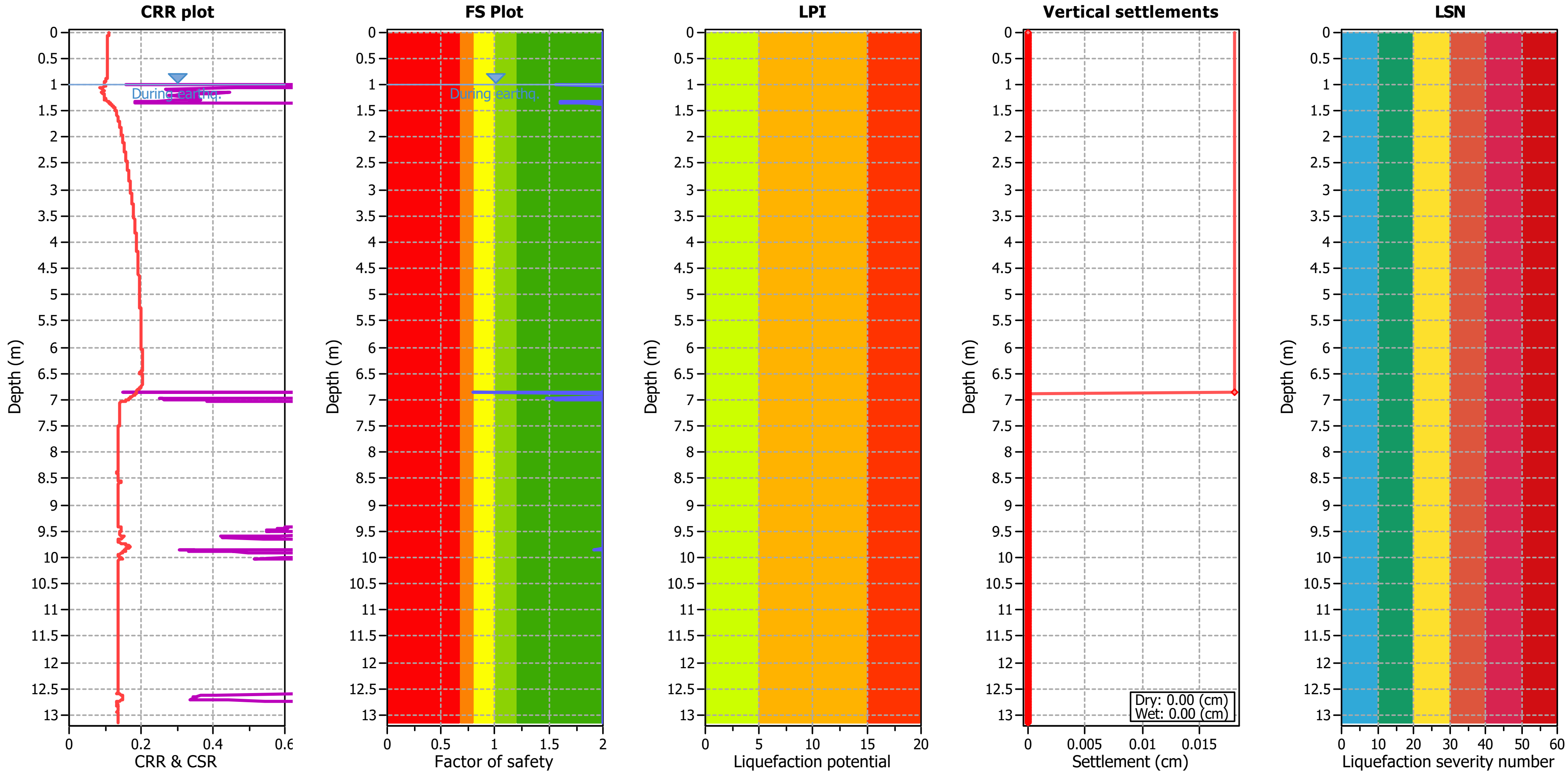
Project: Auckland Surf Park | **CPT: CPT126**

Location: Dairy Flat, Auckland | Total depth: 13.15 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior	
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A	applied:	.
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

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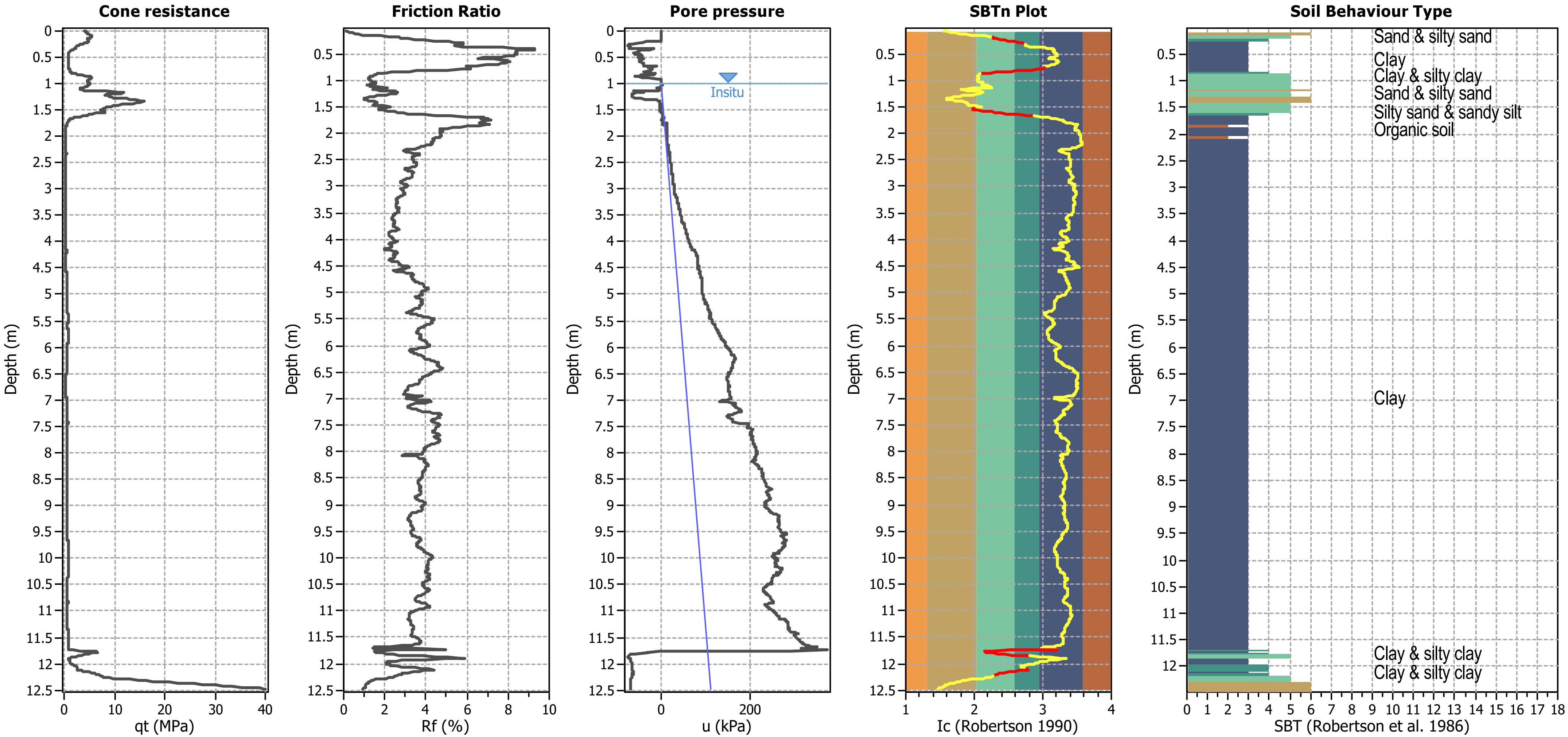
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT127
Total depth: 12.48 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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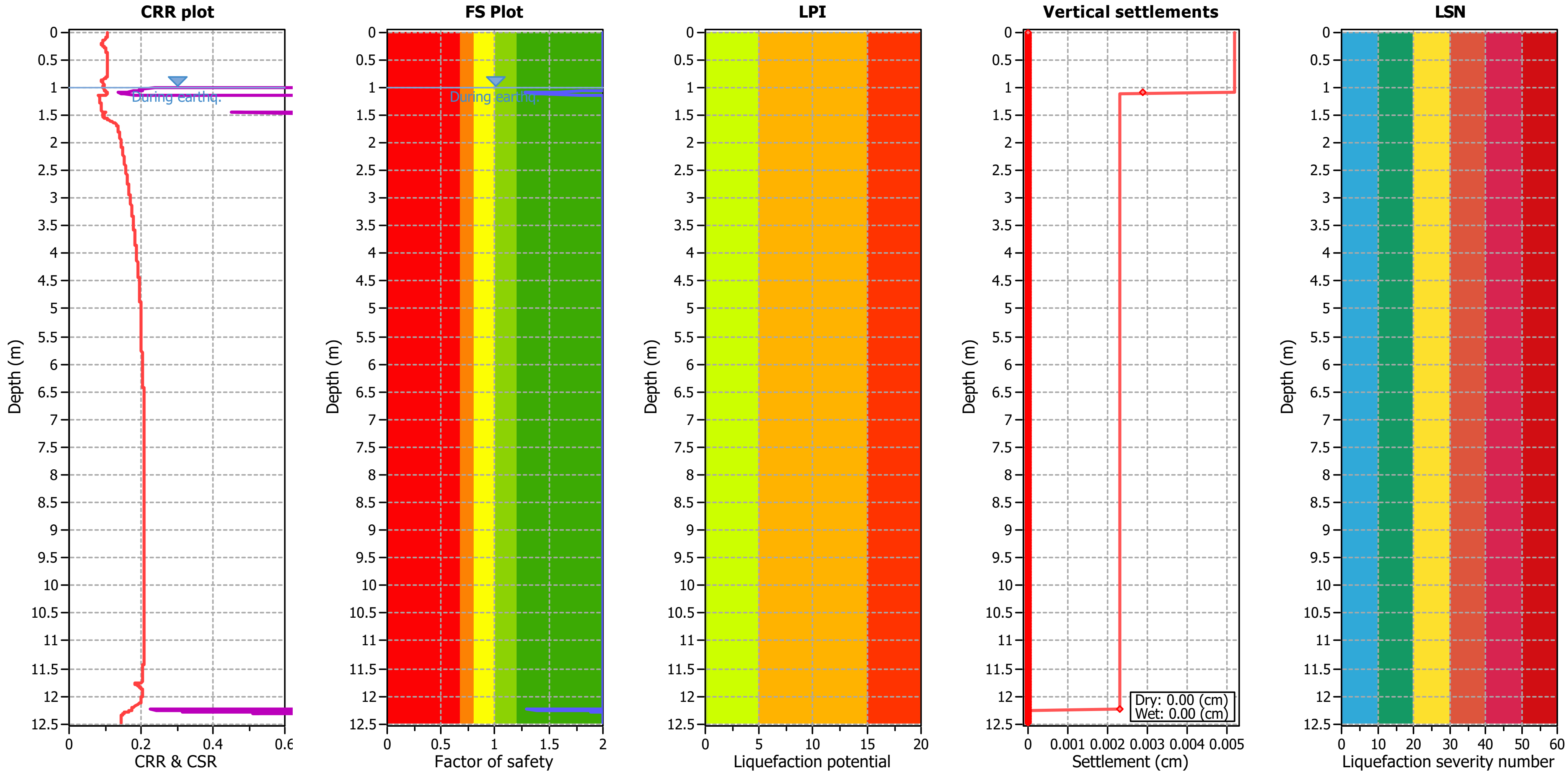
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT127
Total depth: 12.48 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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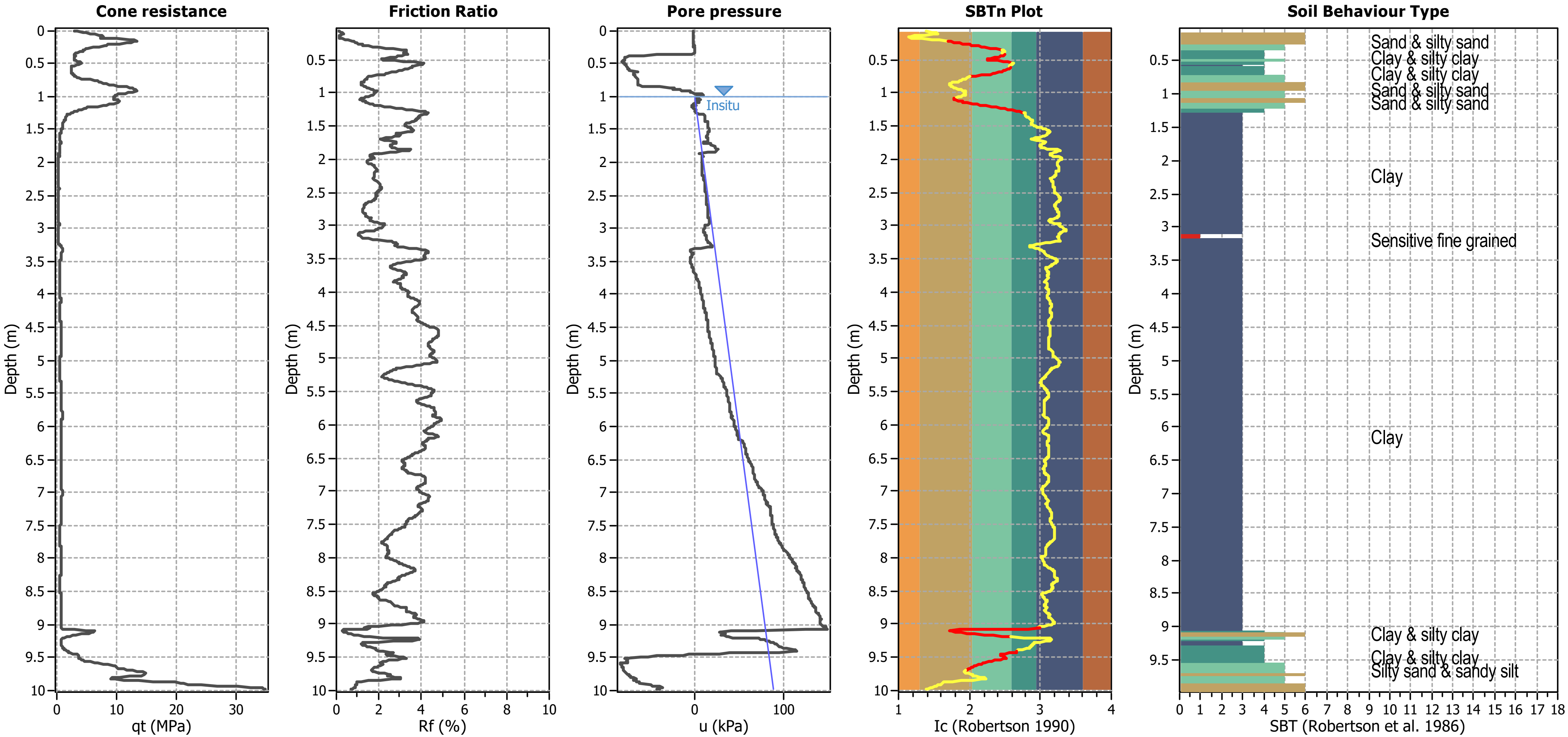
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT128
Total depth: 9.98 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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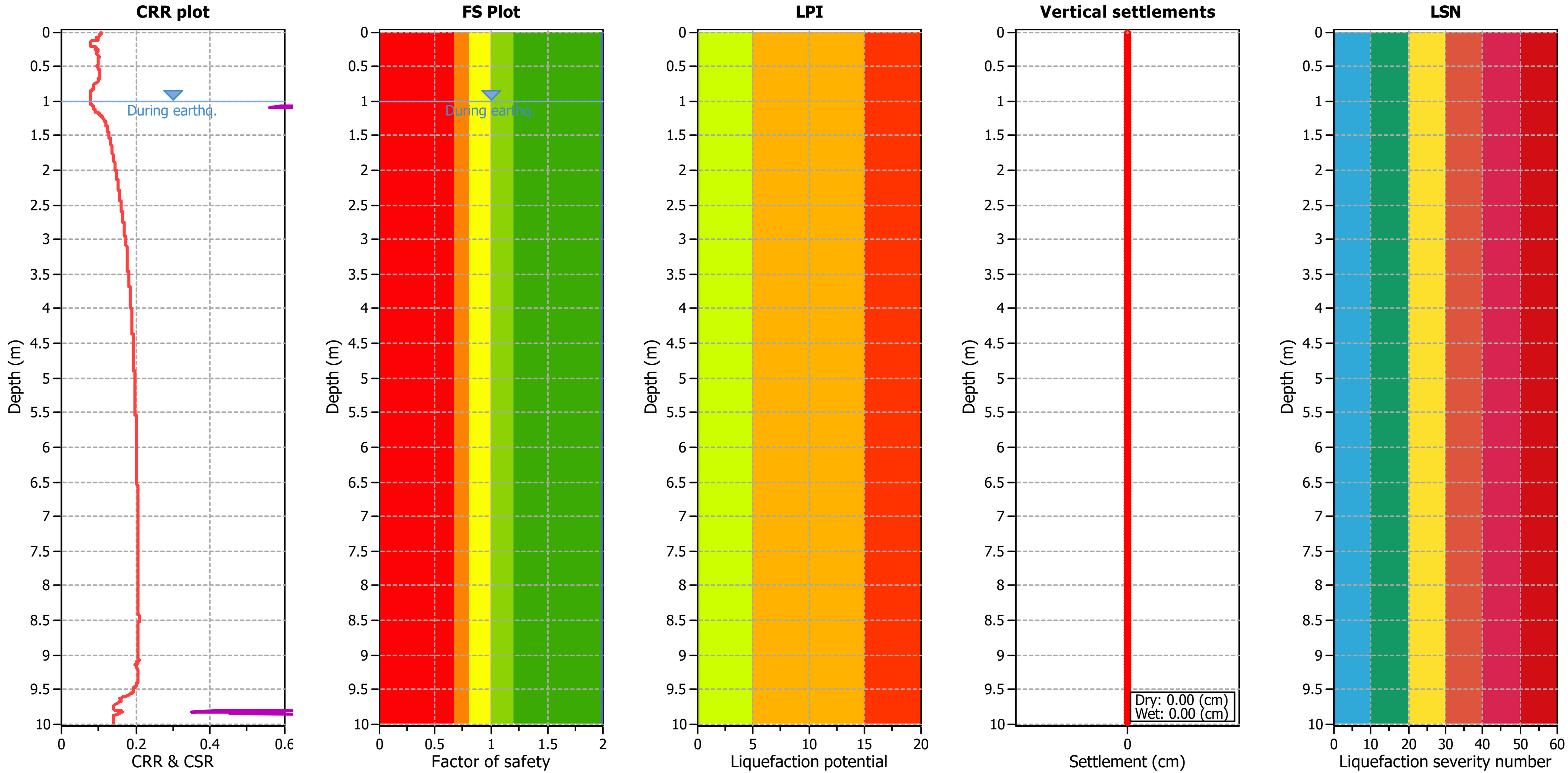
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT128
Total depth: 9.98 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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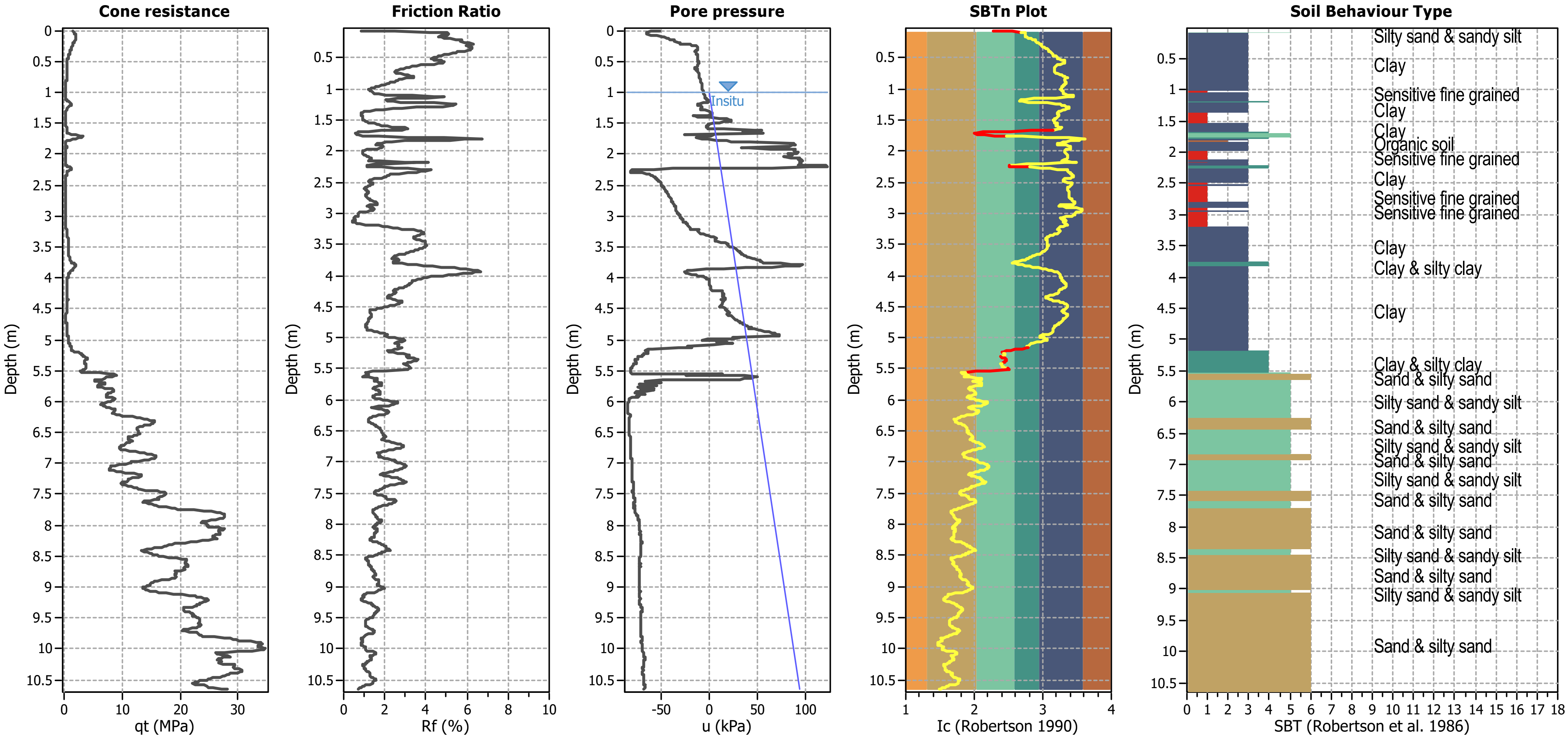
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT129
Total depth: 10.65 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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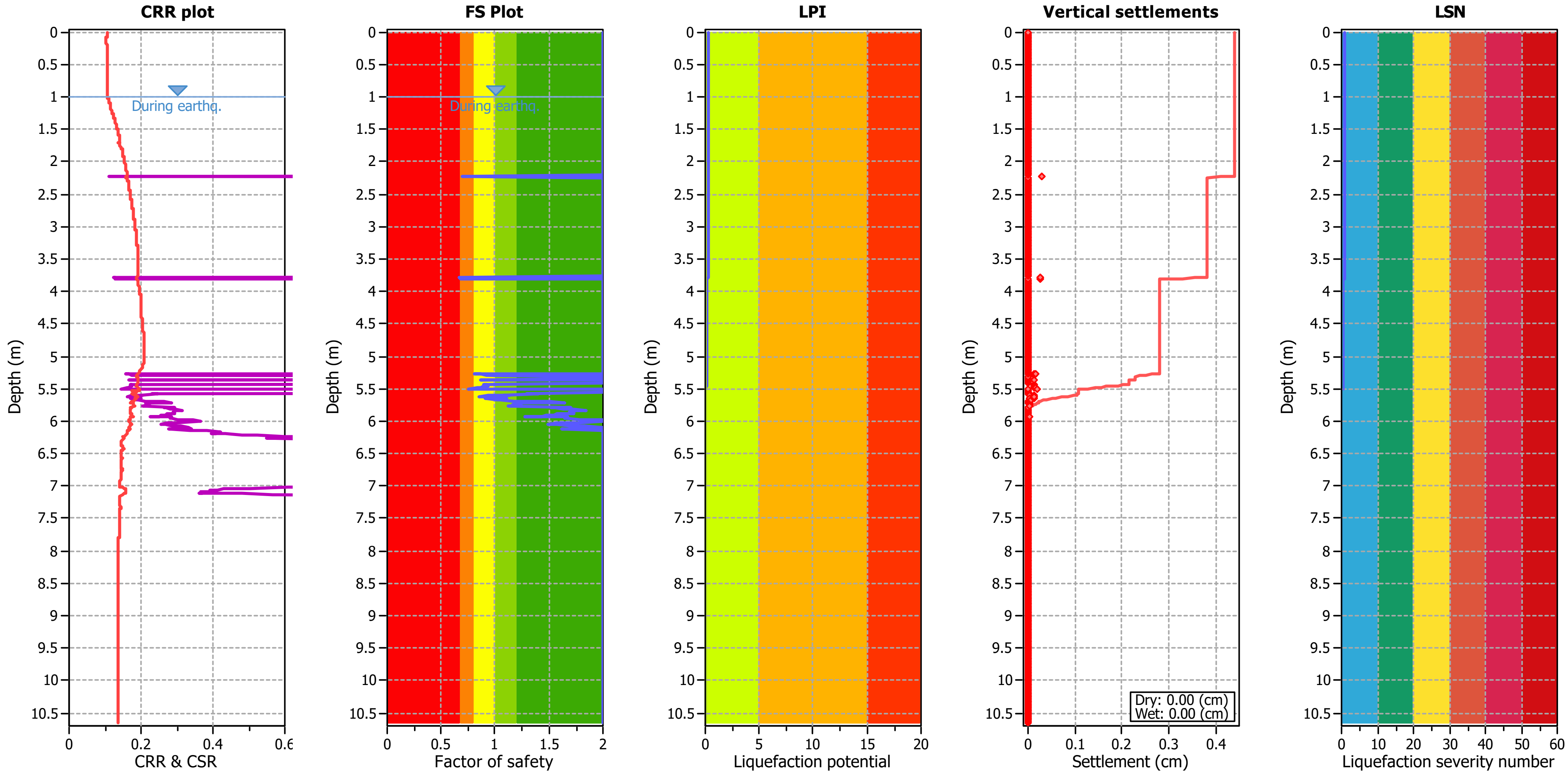
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT129
Total depth: 10.65 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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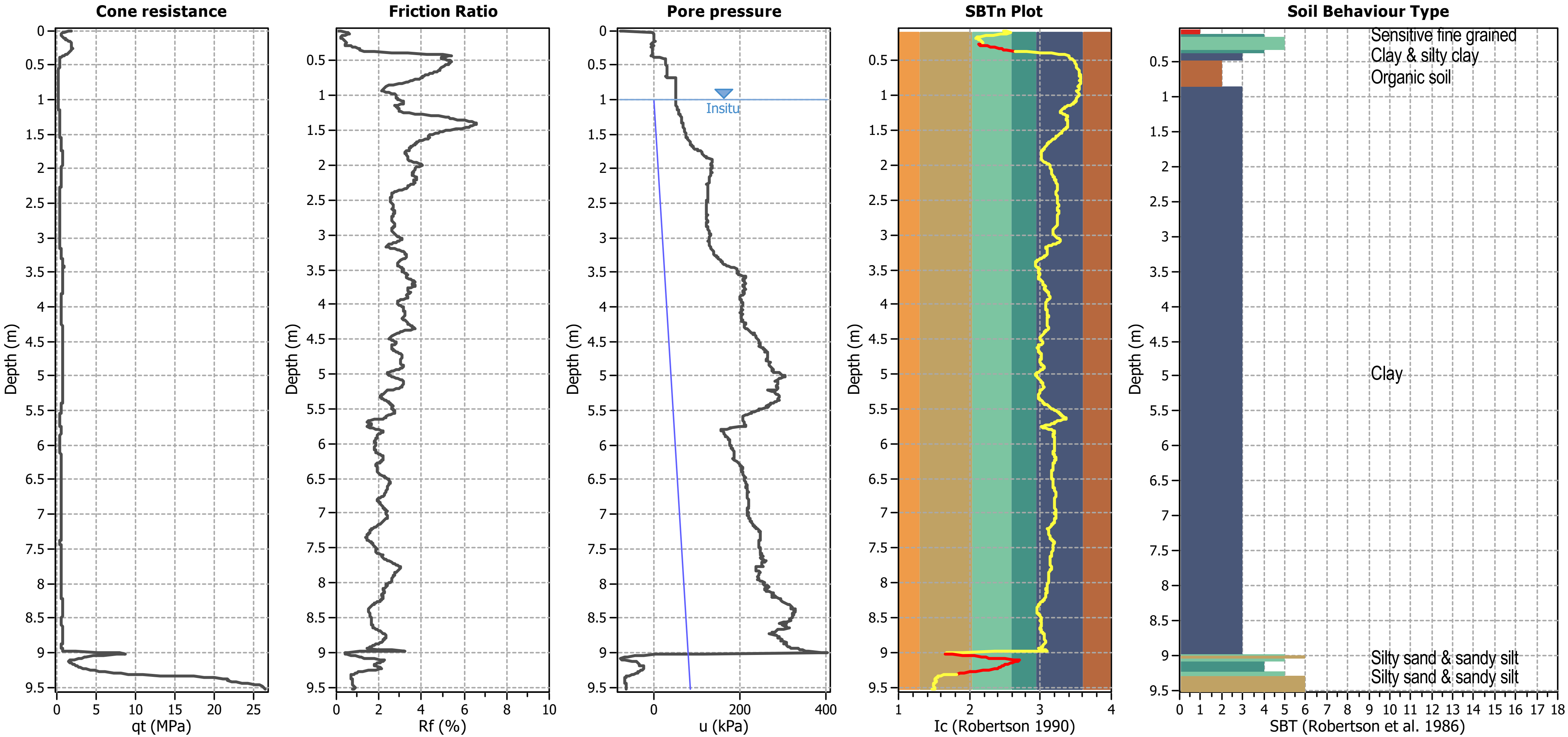
Project: Auckland Surf Park

Location: Dairy Flat, Auckland

CPT: CPT131
Total depth: 9.53 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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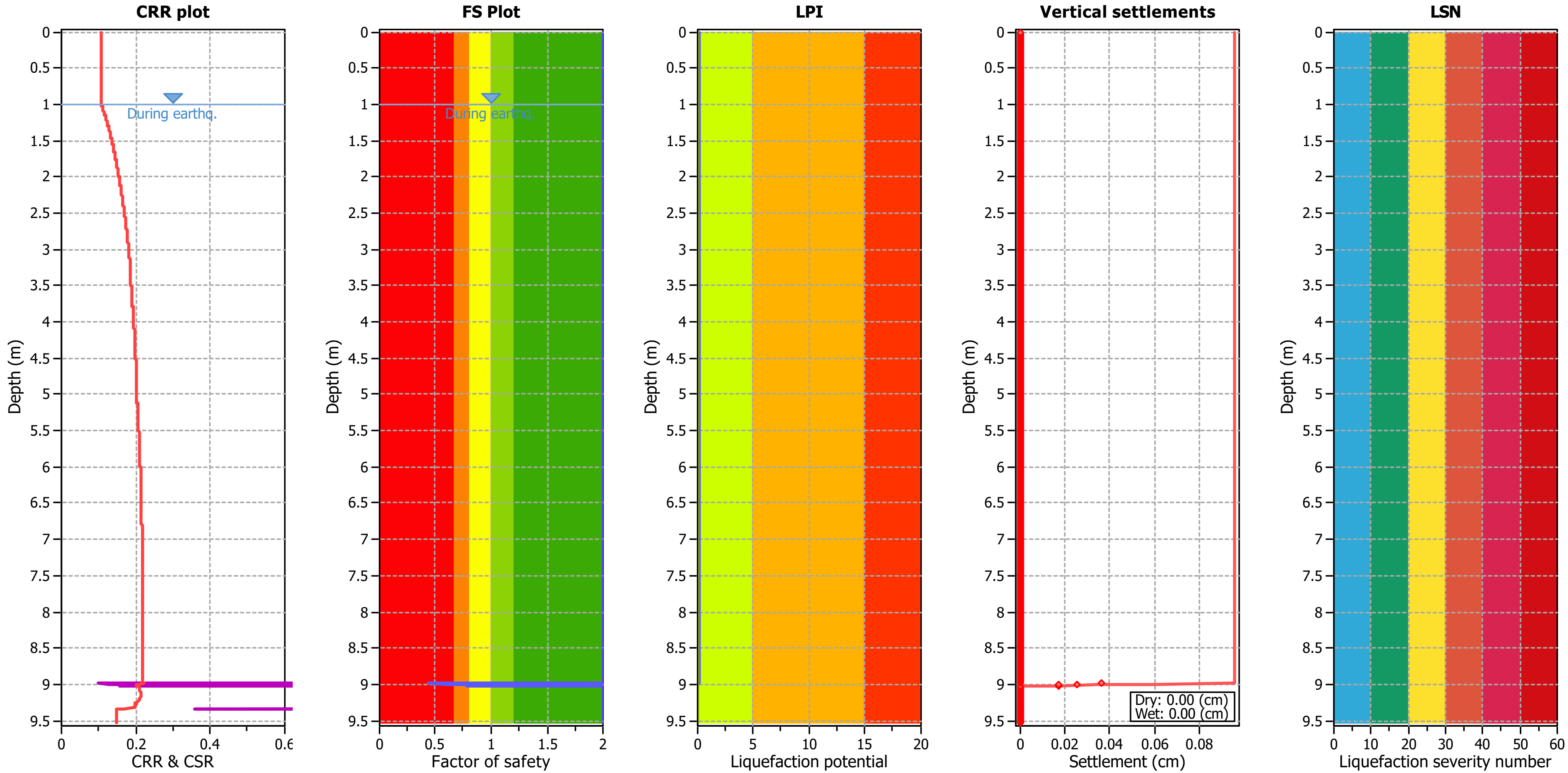
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT131
Total depth: 9.53 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

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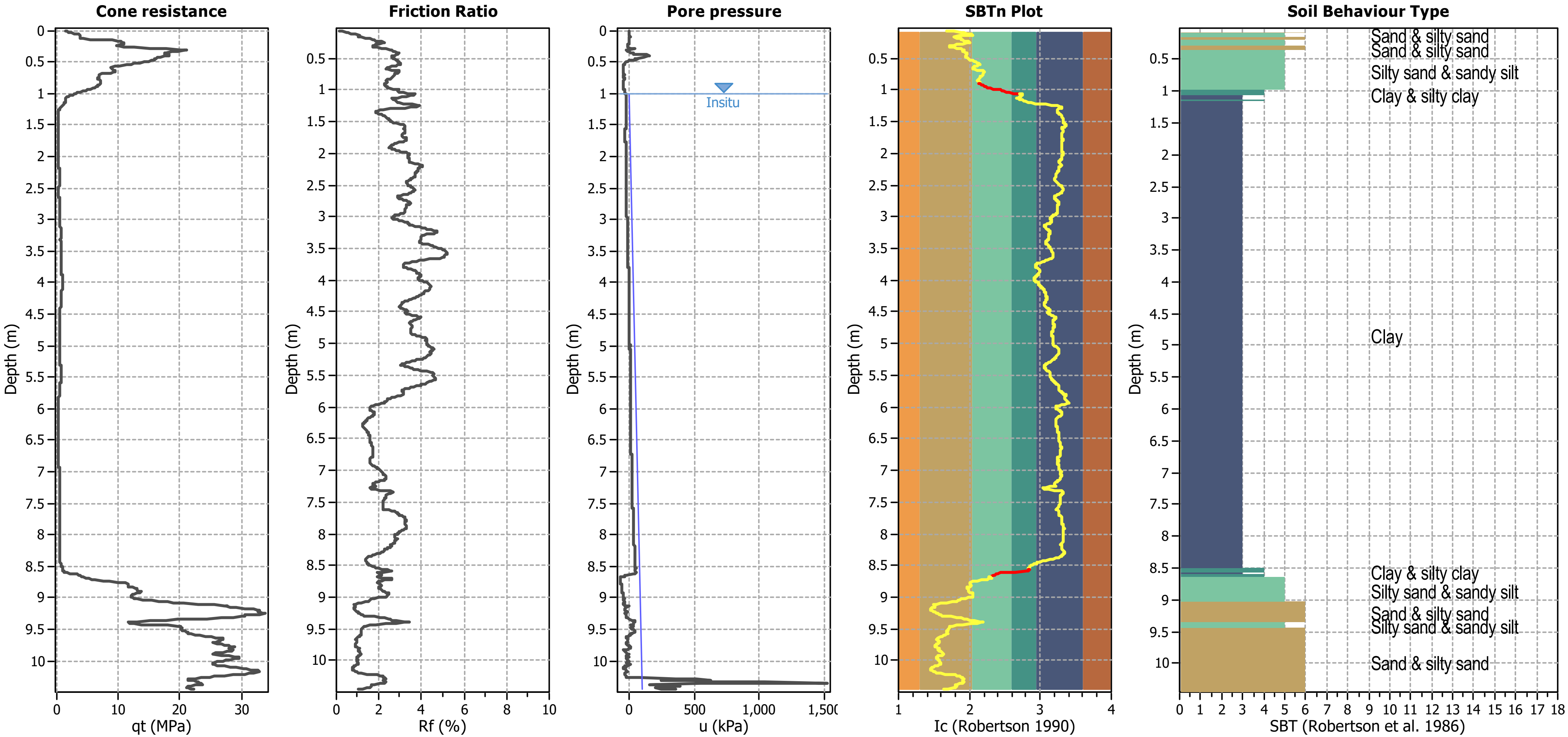
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT132
Total depth: 10.45 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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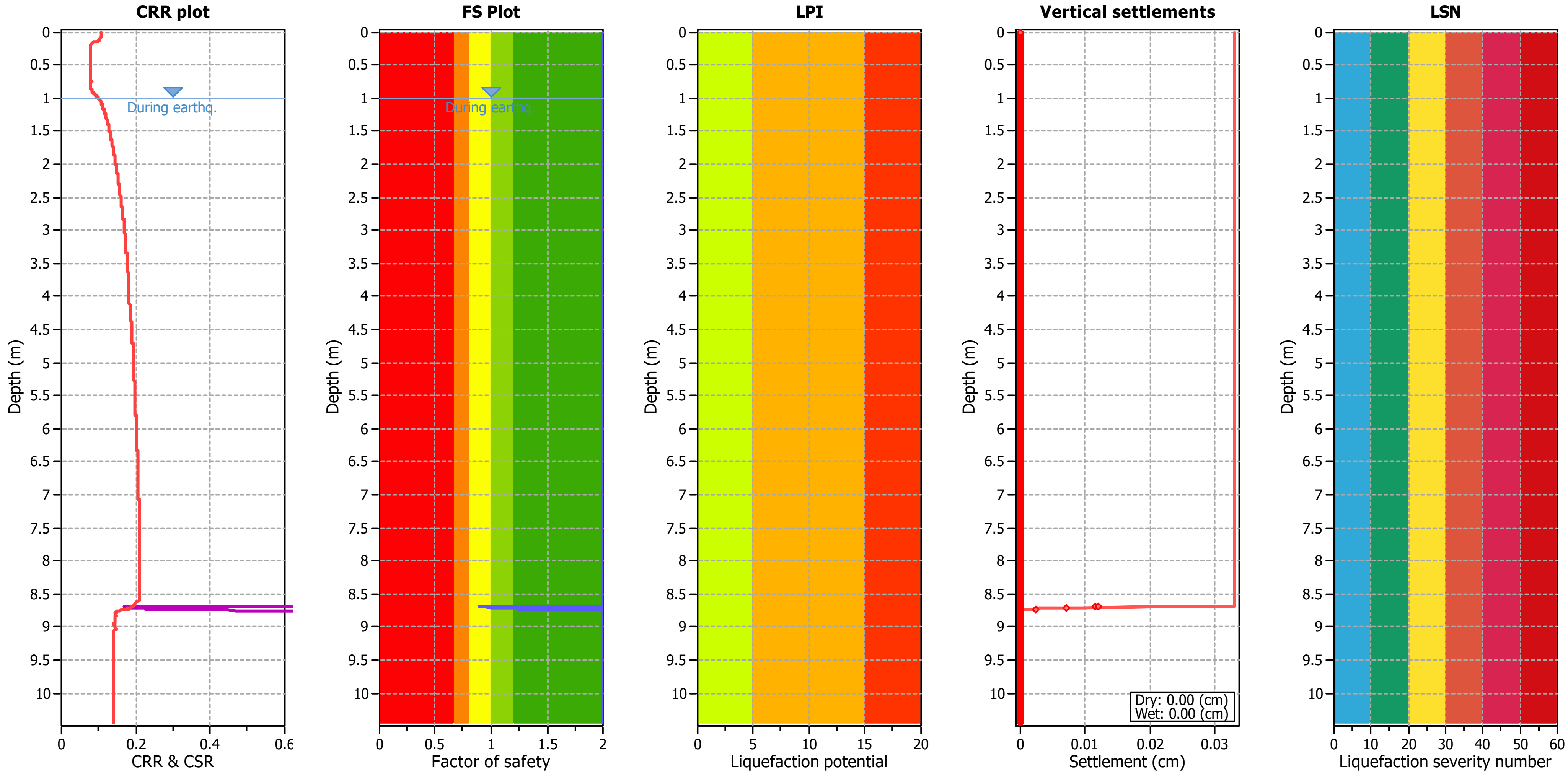
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT132
Total depth: 10.45 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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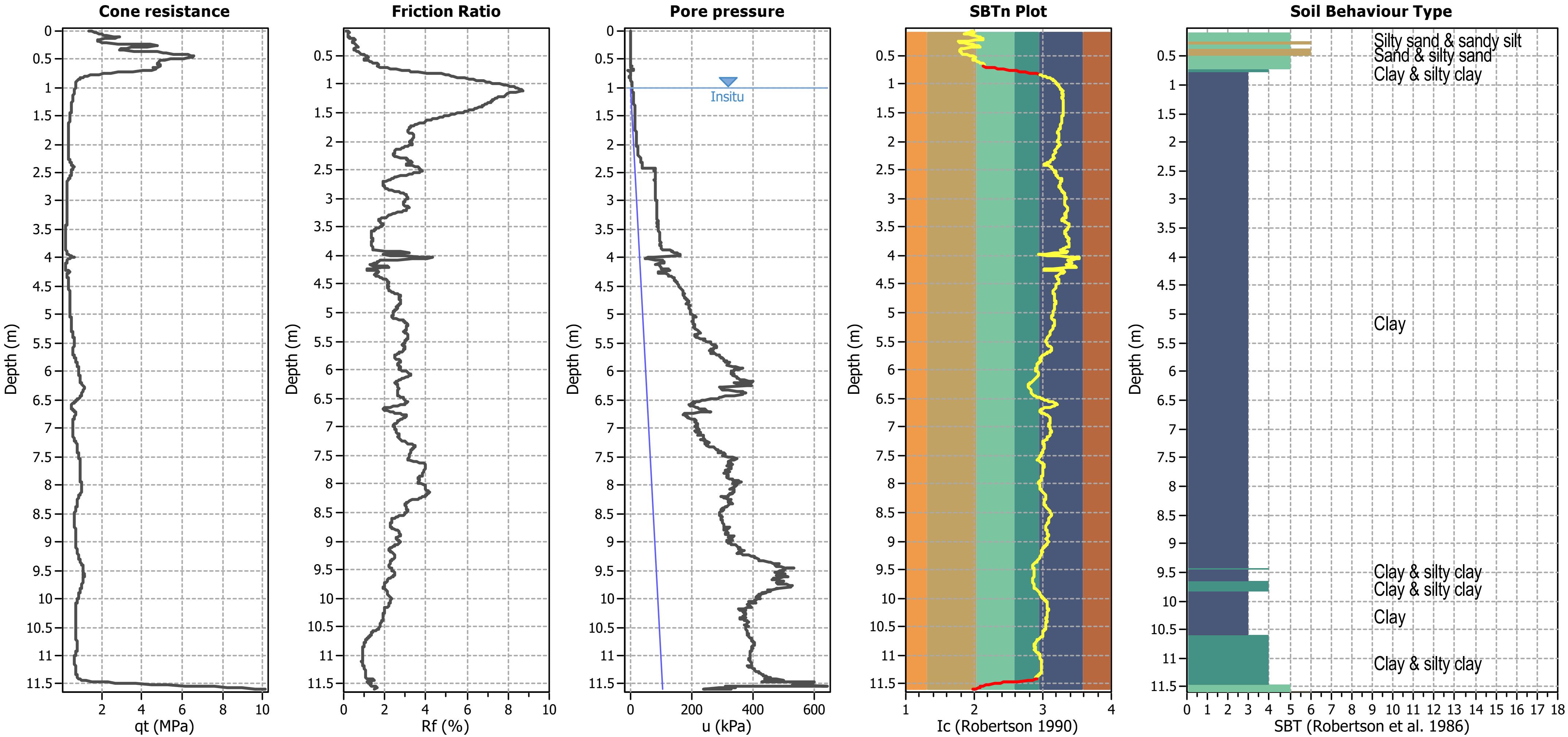
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

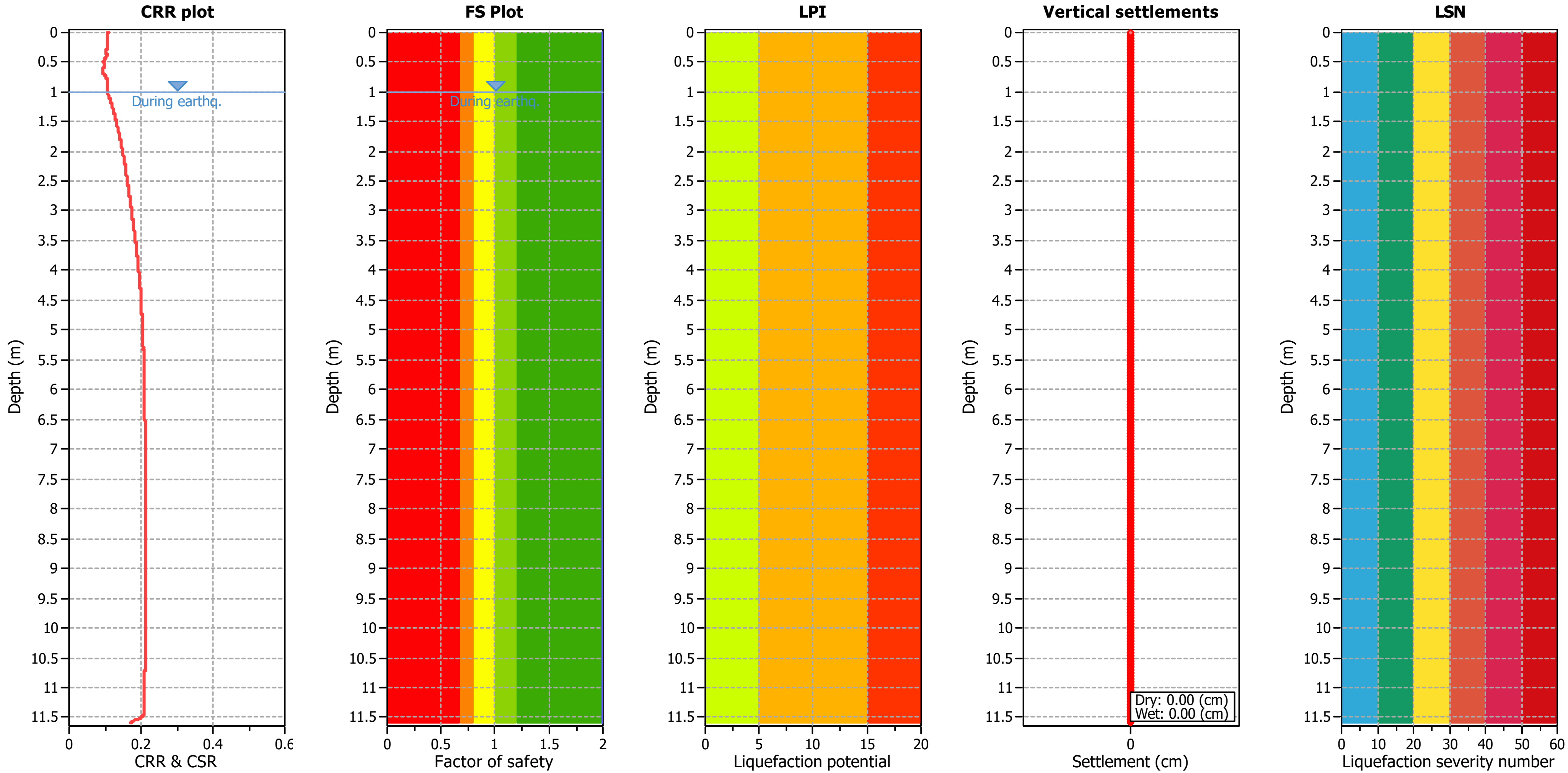
CPT: CPT133
Total depth: 11.59 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT133
Total depth: 11.59 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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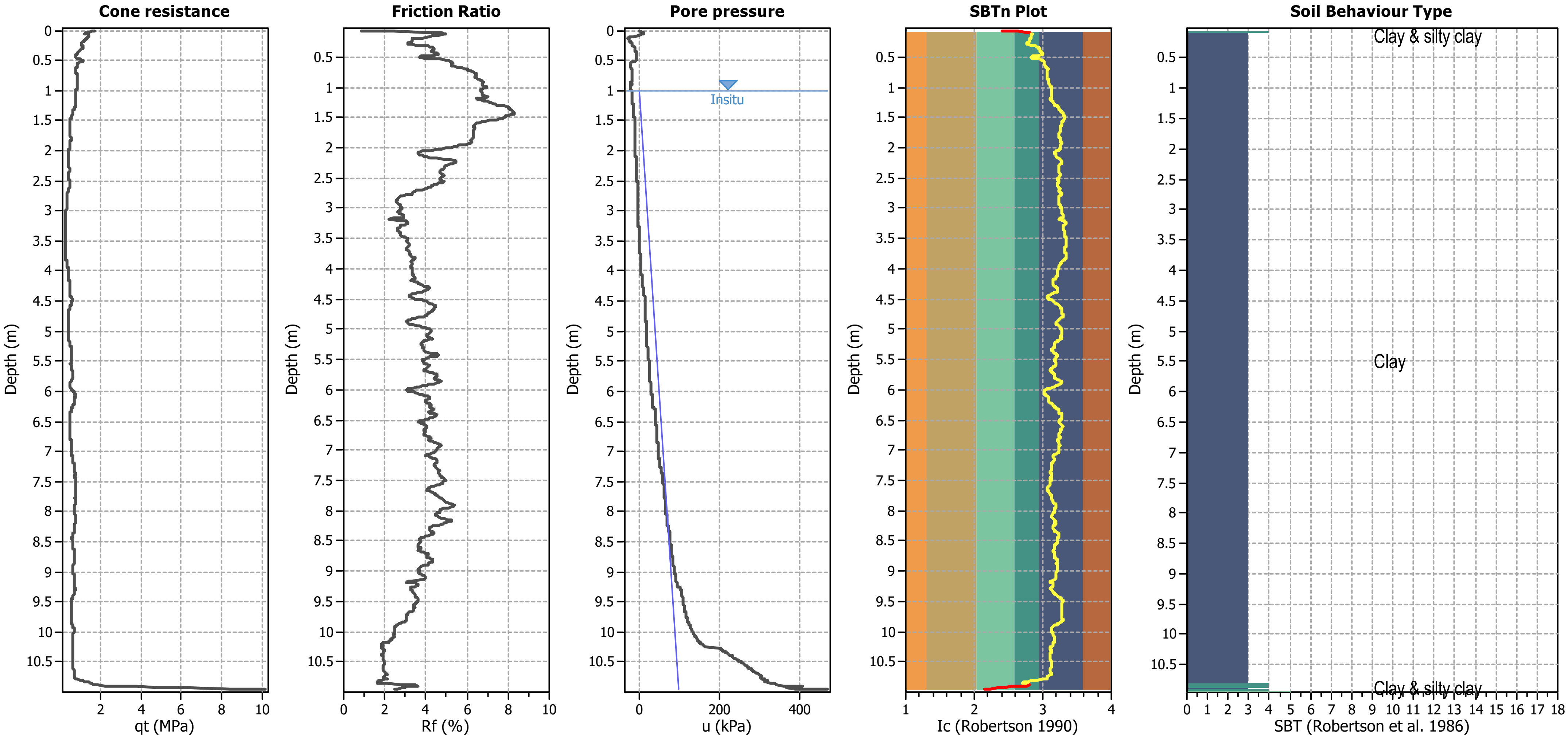
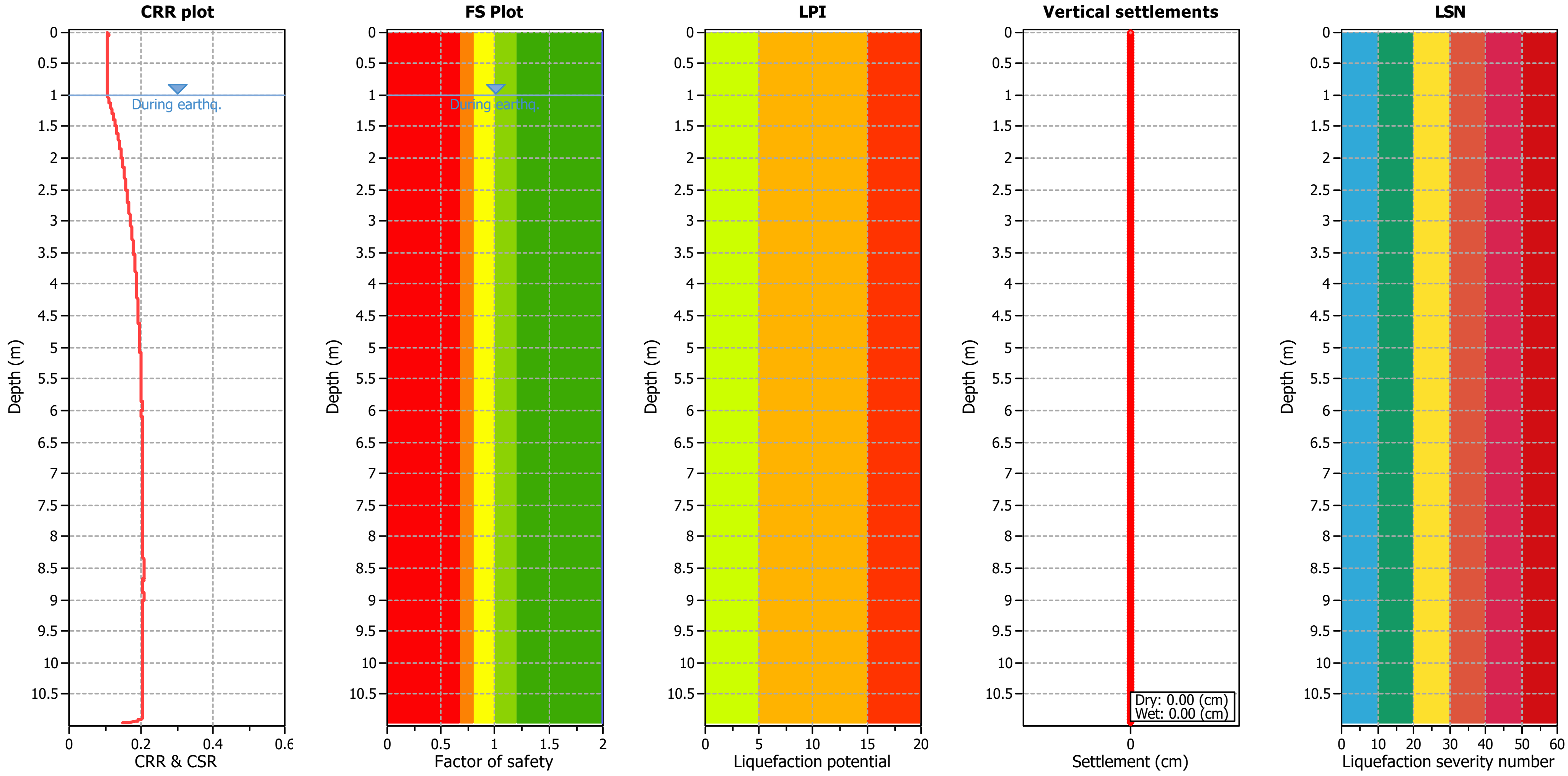
Project: **Auckland Surf Park** | **CPT: CPT134**
Location: **Dairy Flat, Auckland** | Total depth: 10.95 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT134
Total depth: 10.95 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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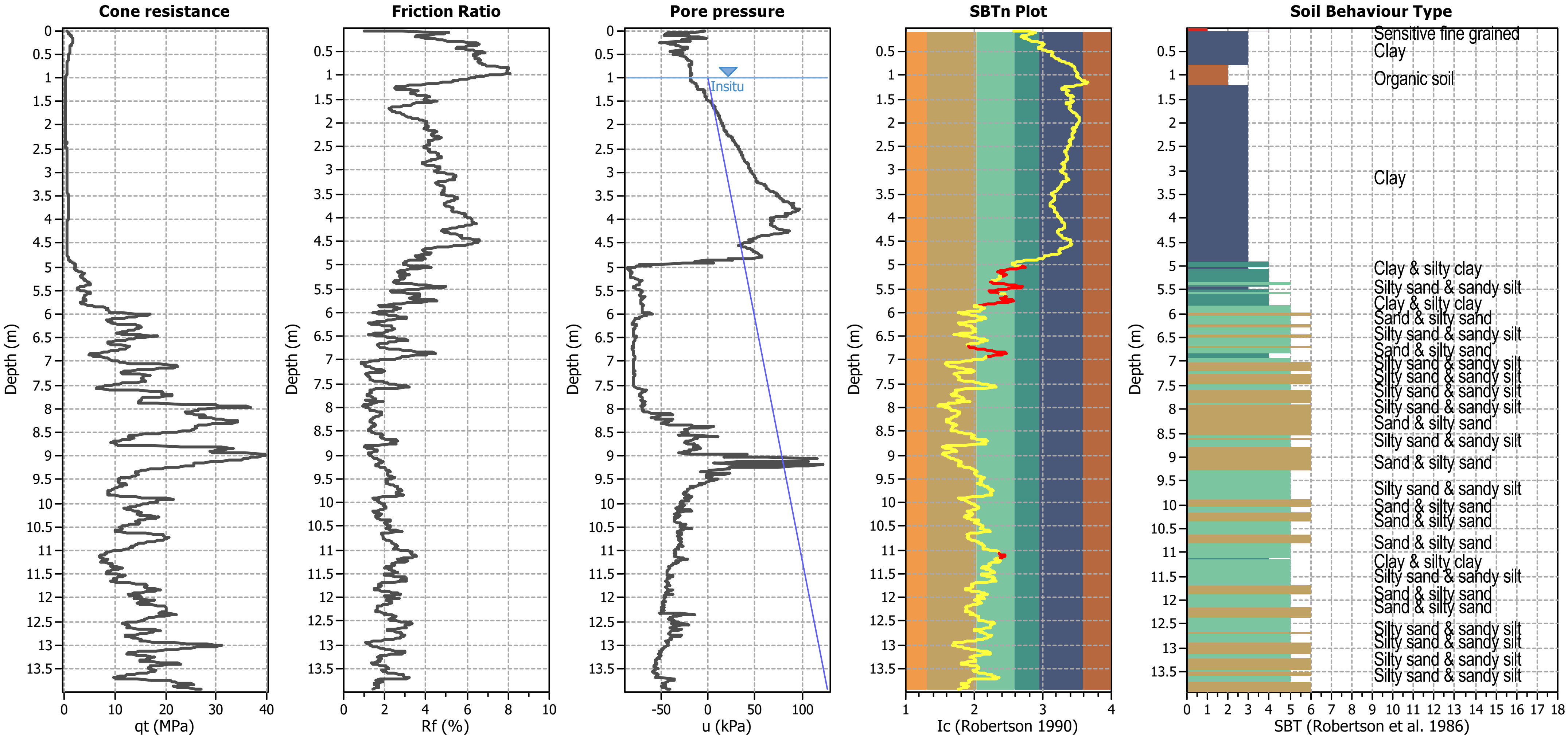
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT135
Total depth: 13.93 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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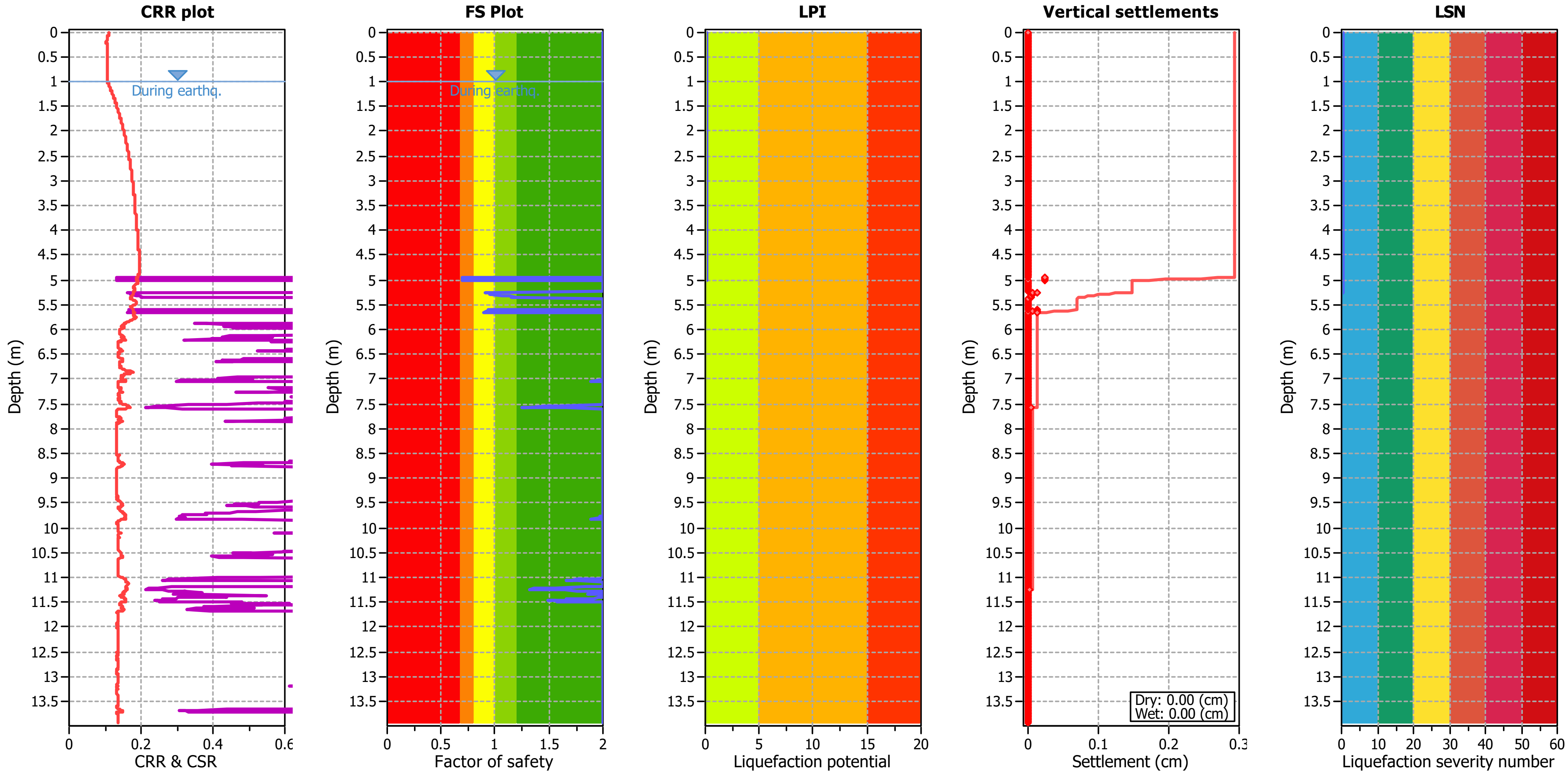
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT135
Total depth: 13.93 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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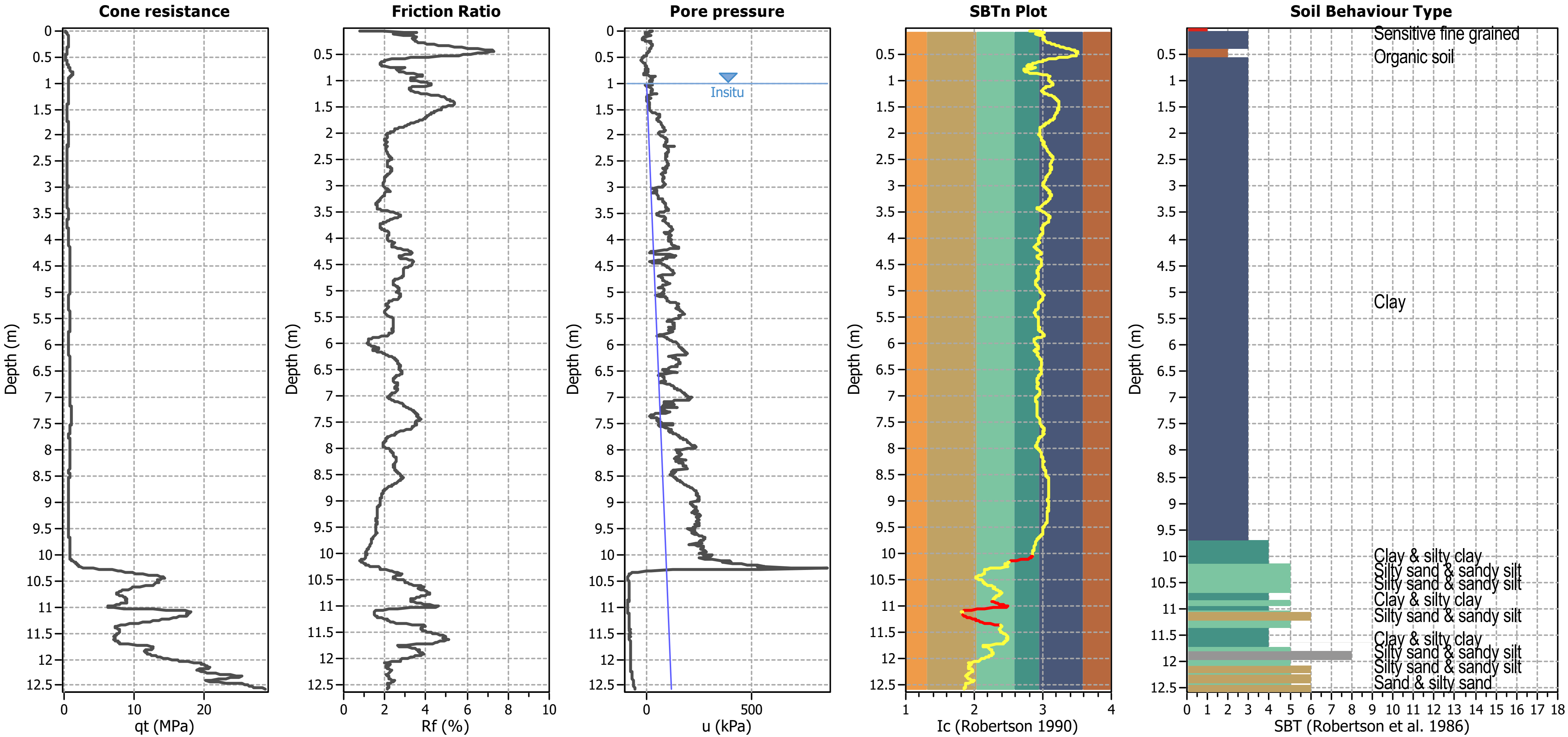
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT136
Total depth: 12.57 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

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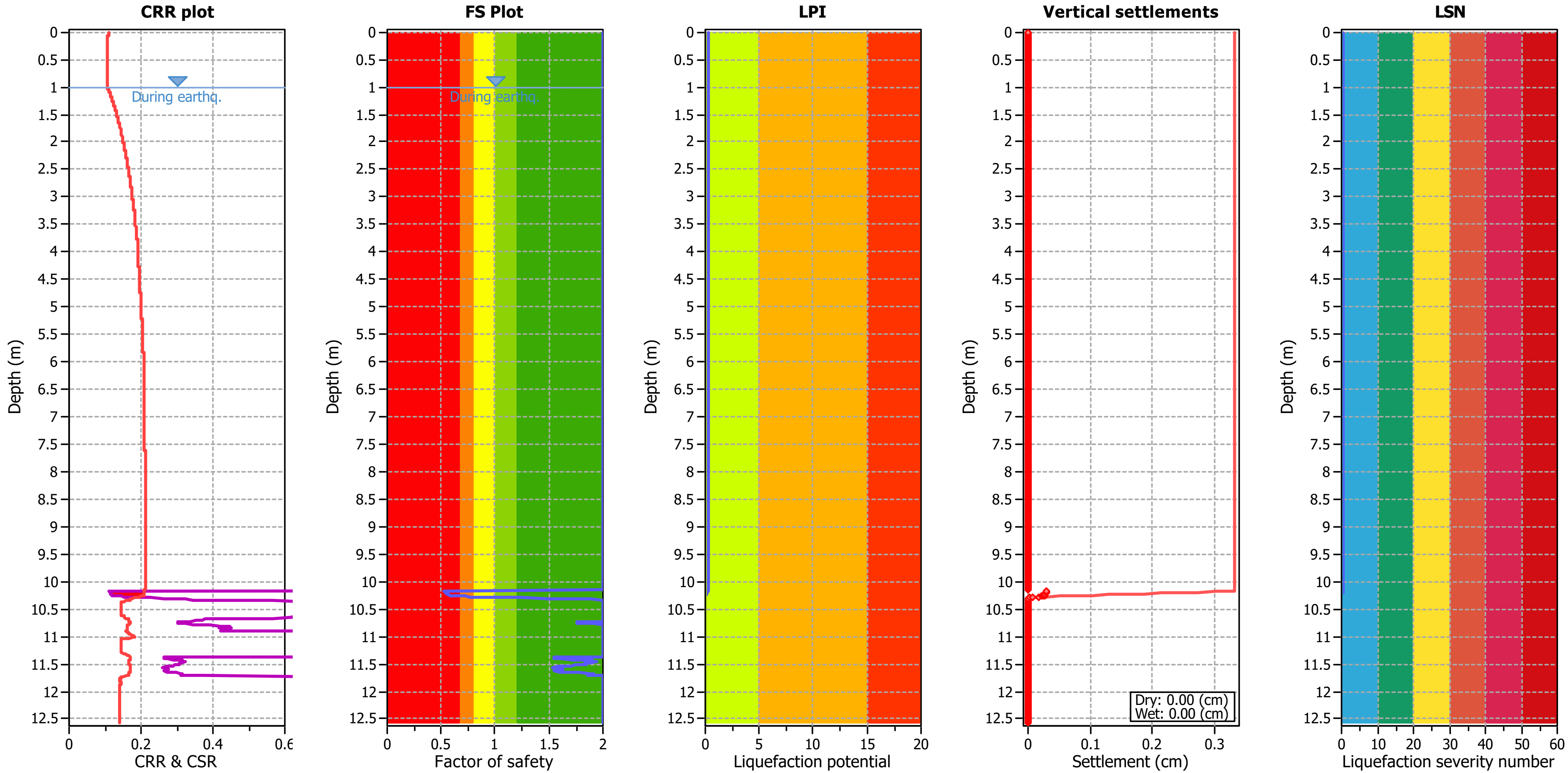
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT136
Total depth: 12.57 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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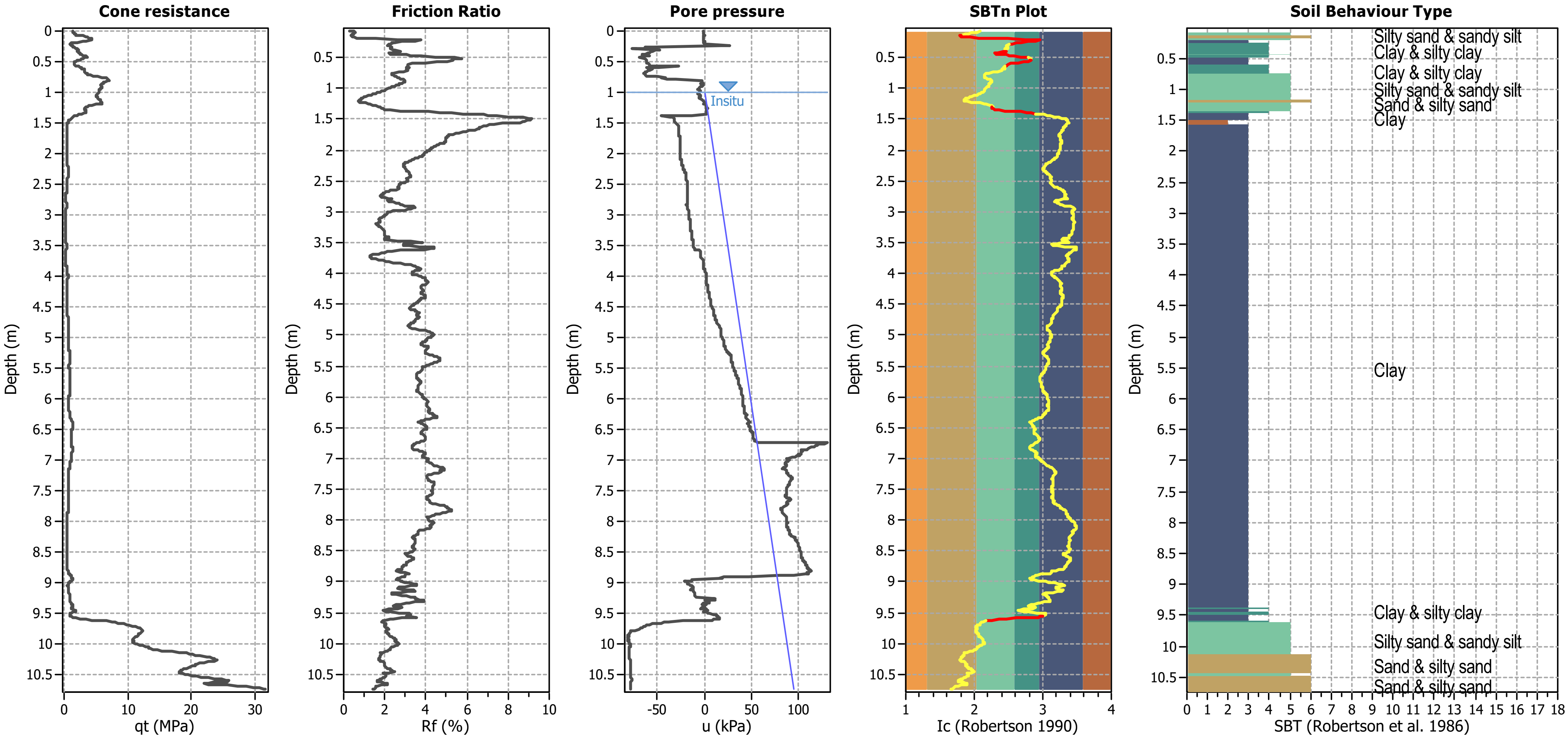
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT137
Total depth: 10.74 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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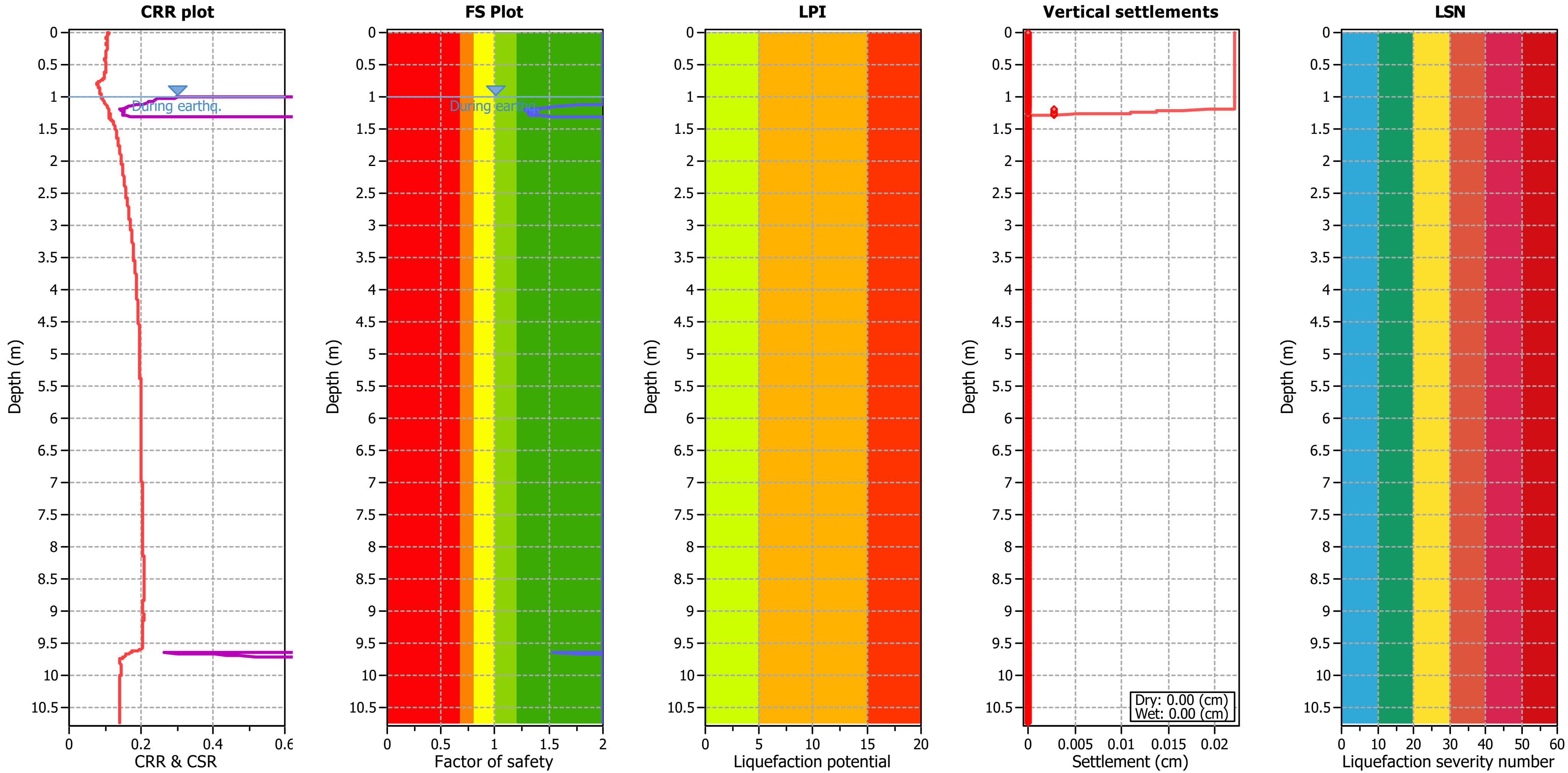
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT137
Total depth: 10.74 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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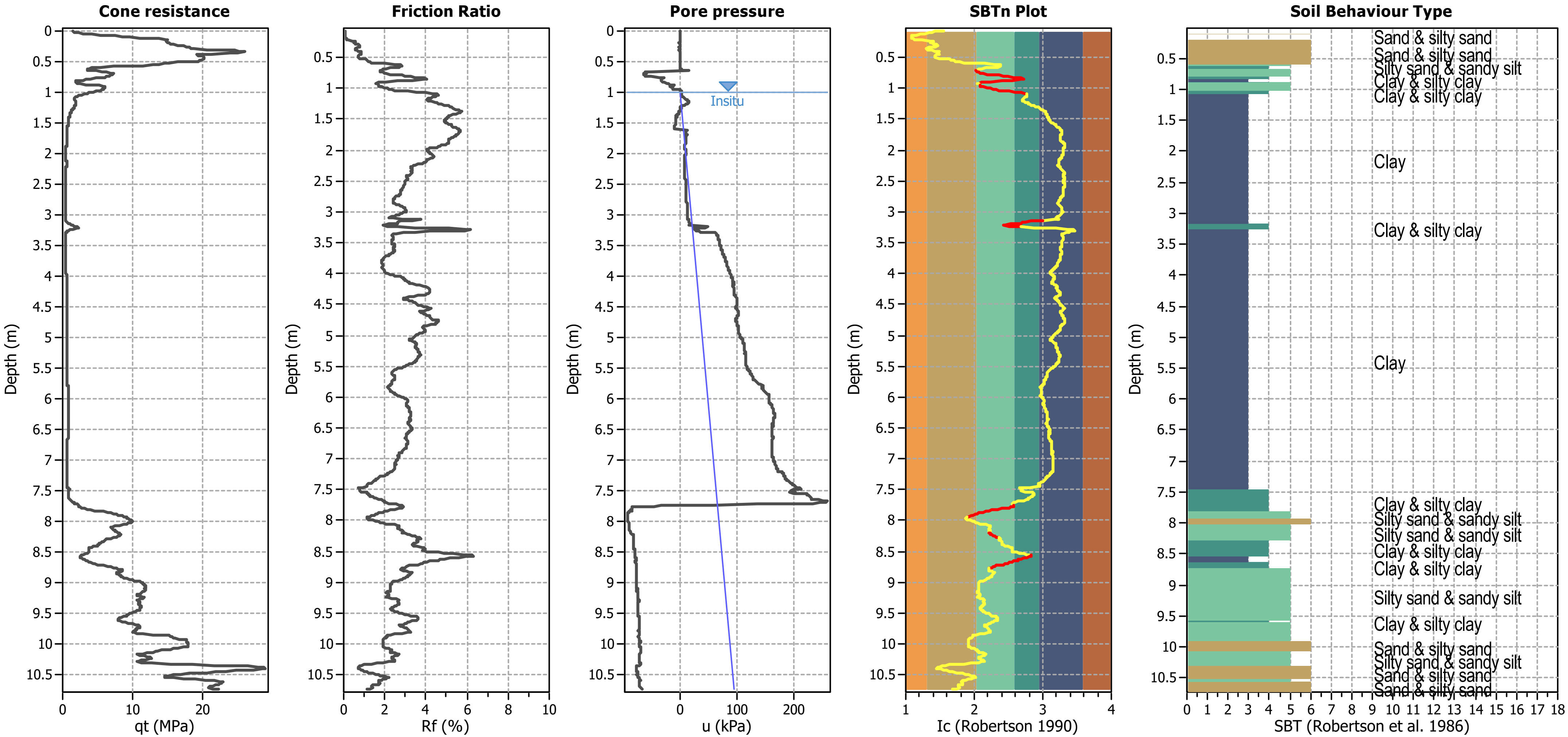
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT138
Total depth: 10.73 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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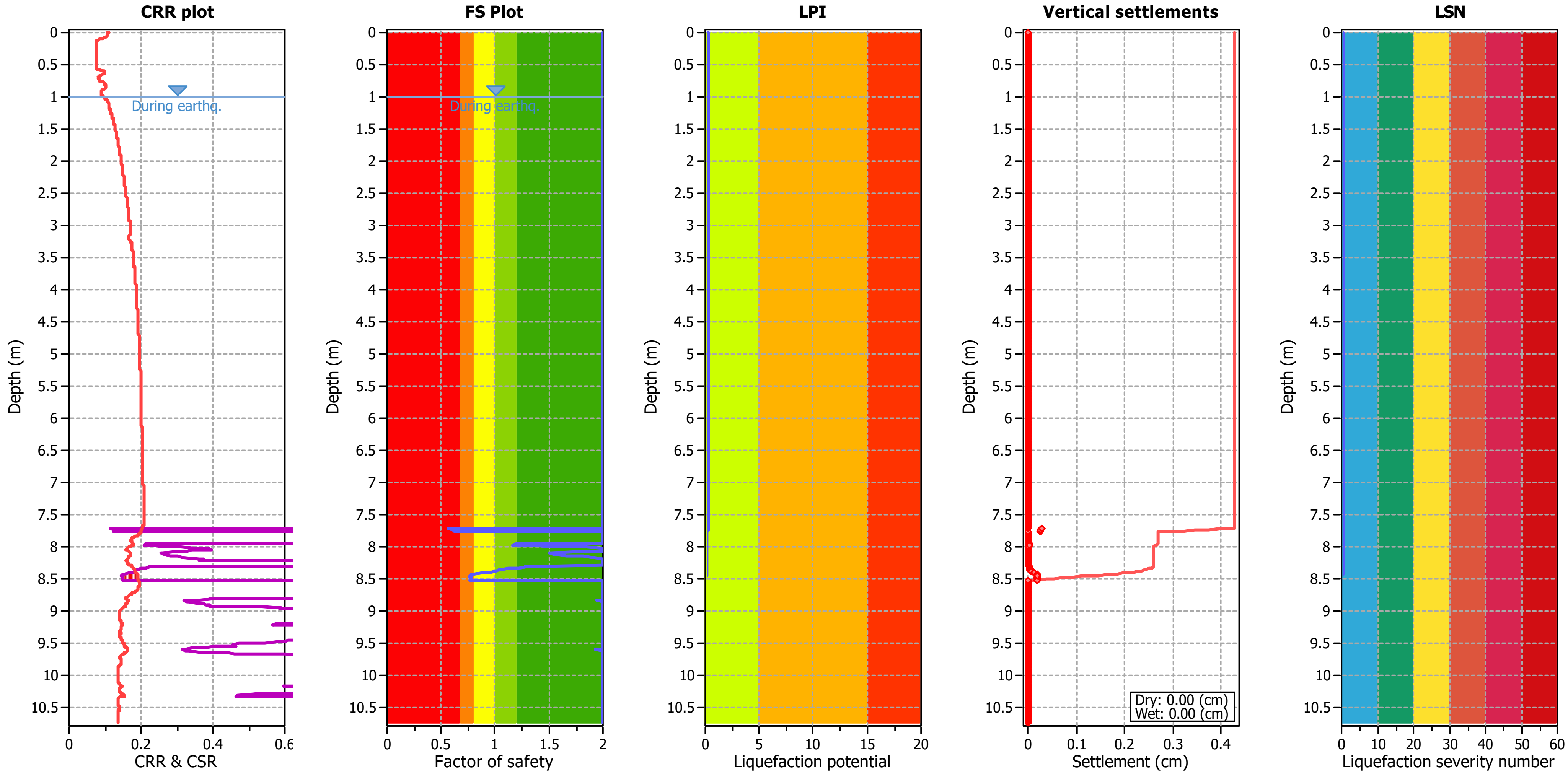
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT138
Total depth: 10.73 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

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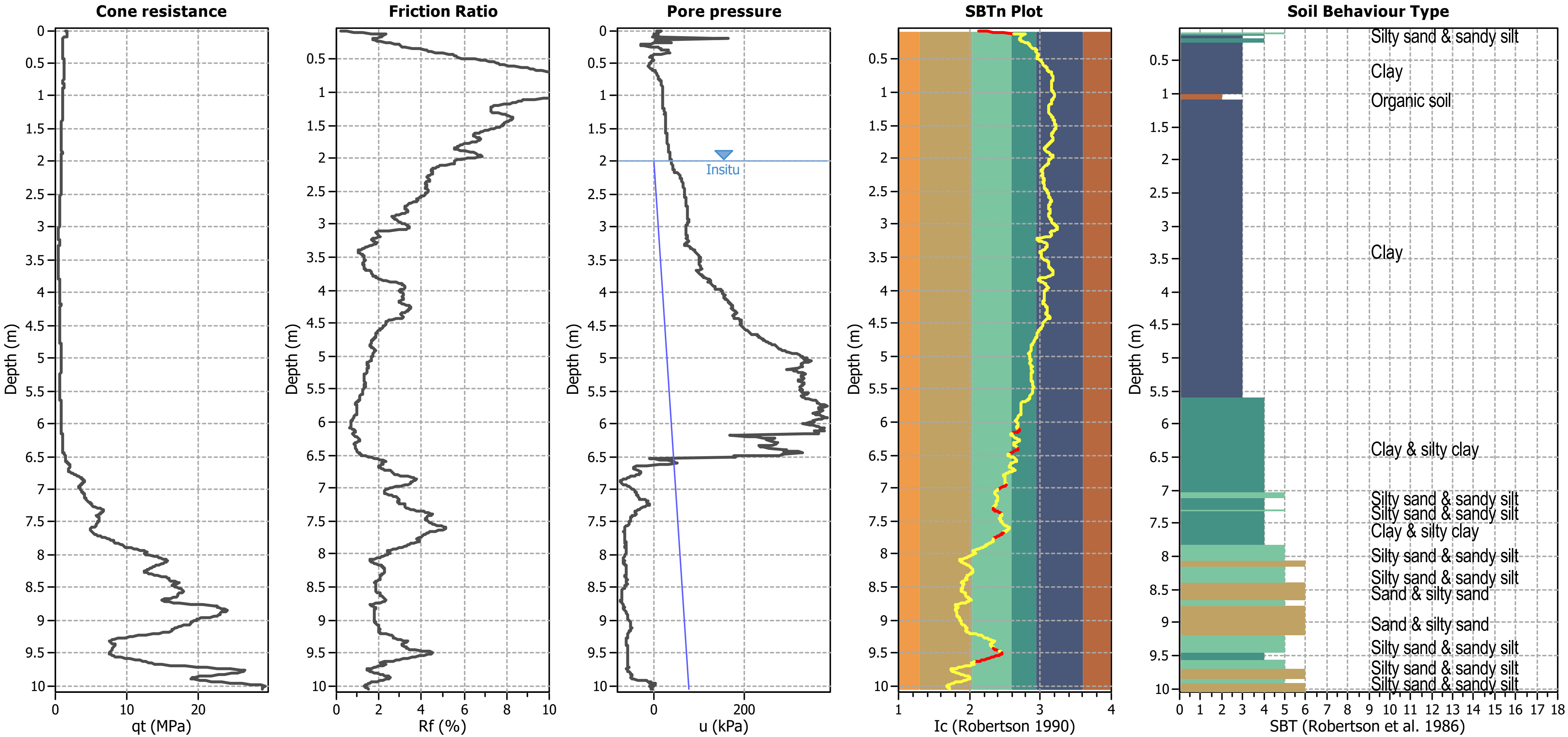
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

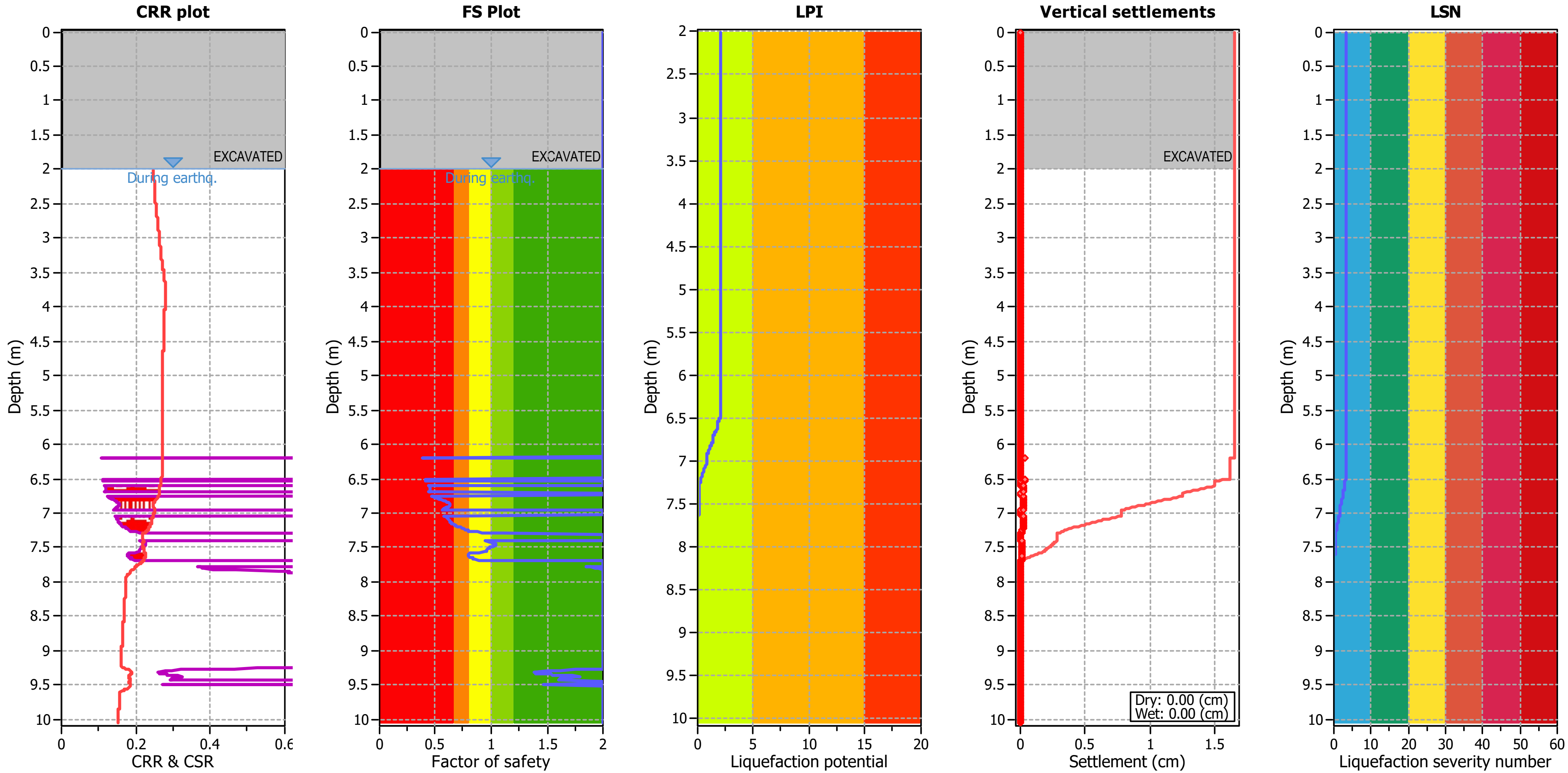
CPT: CPT139
Total depth: 10.05 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Excavation:	Yes	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Excavation depth:	2.00 m	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.00 kPa	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		

Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT139
Total depth: 10.05 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Excavation:	Yes	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Excavation depth:	2.00 m	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	0.00 kPa	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		

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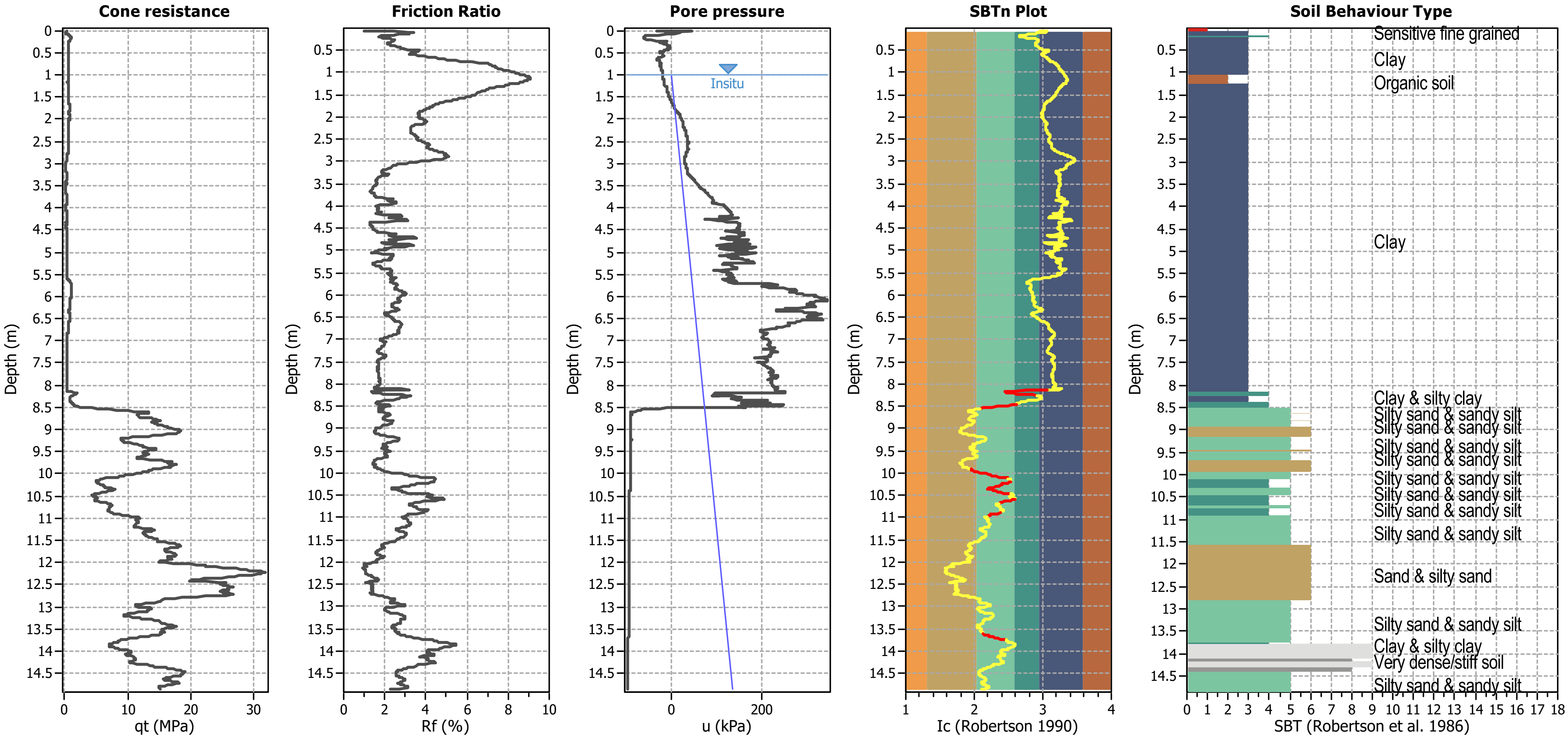
Project: **Auckland Surf Park**
Location: **Dairy Flat, Auckland**

CPT: CPT140
Total depth: 14.86 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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:	6.50	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.19	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

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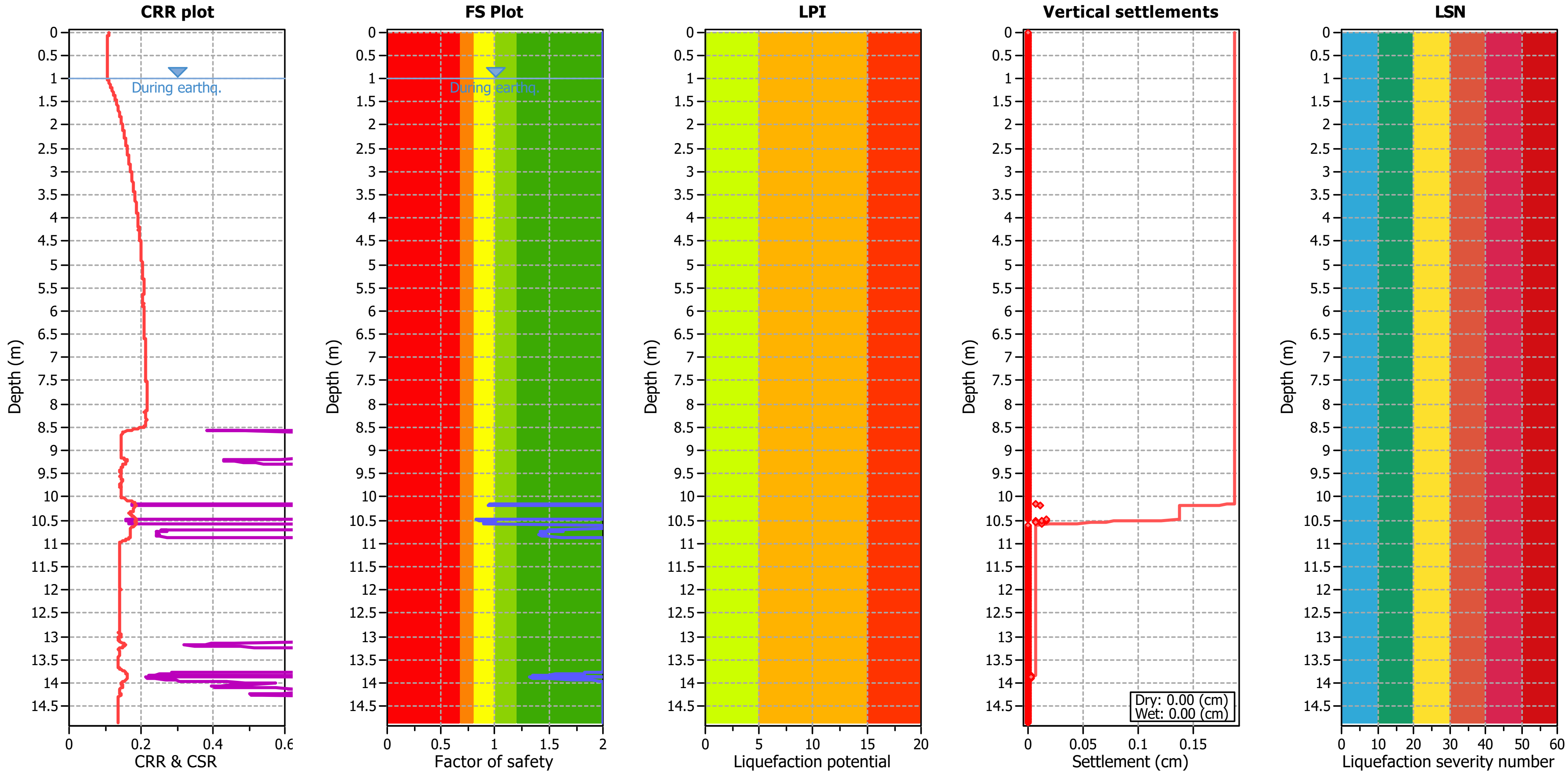
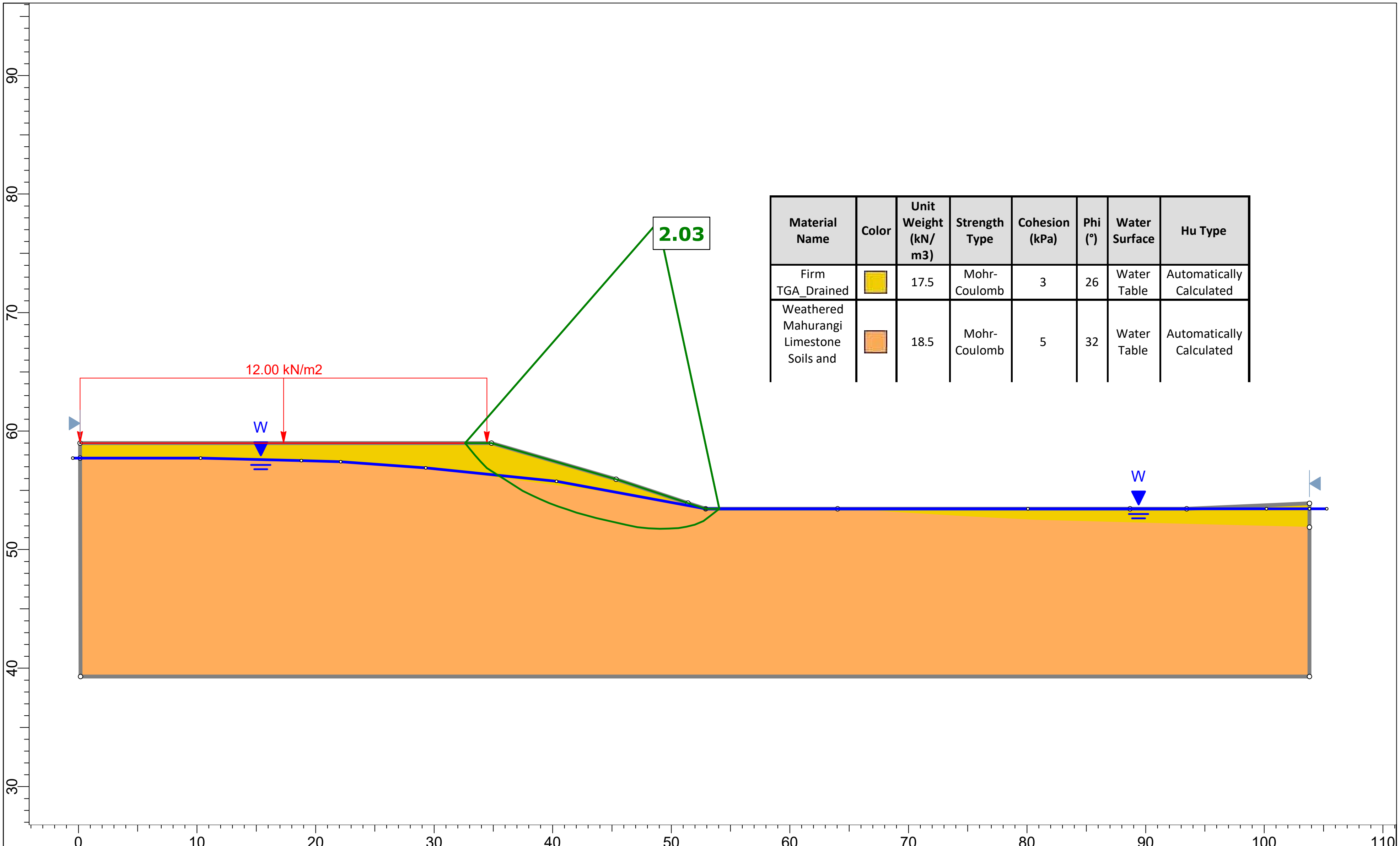
Project: Auckland Surf Park
Location: Dairy Flat, Auckland

CPT: CPT140
Total depth: 14.86 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	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		

Appendix C Preliminary Slope Stability Outputs



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated

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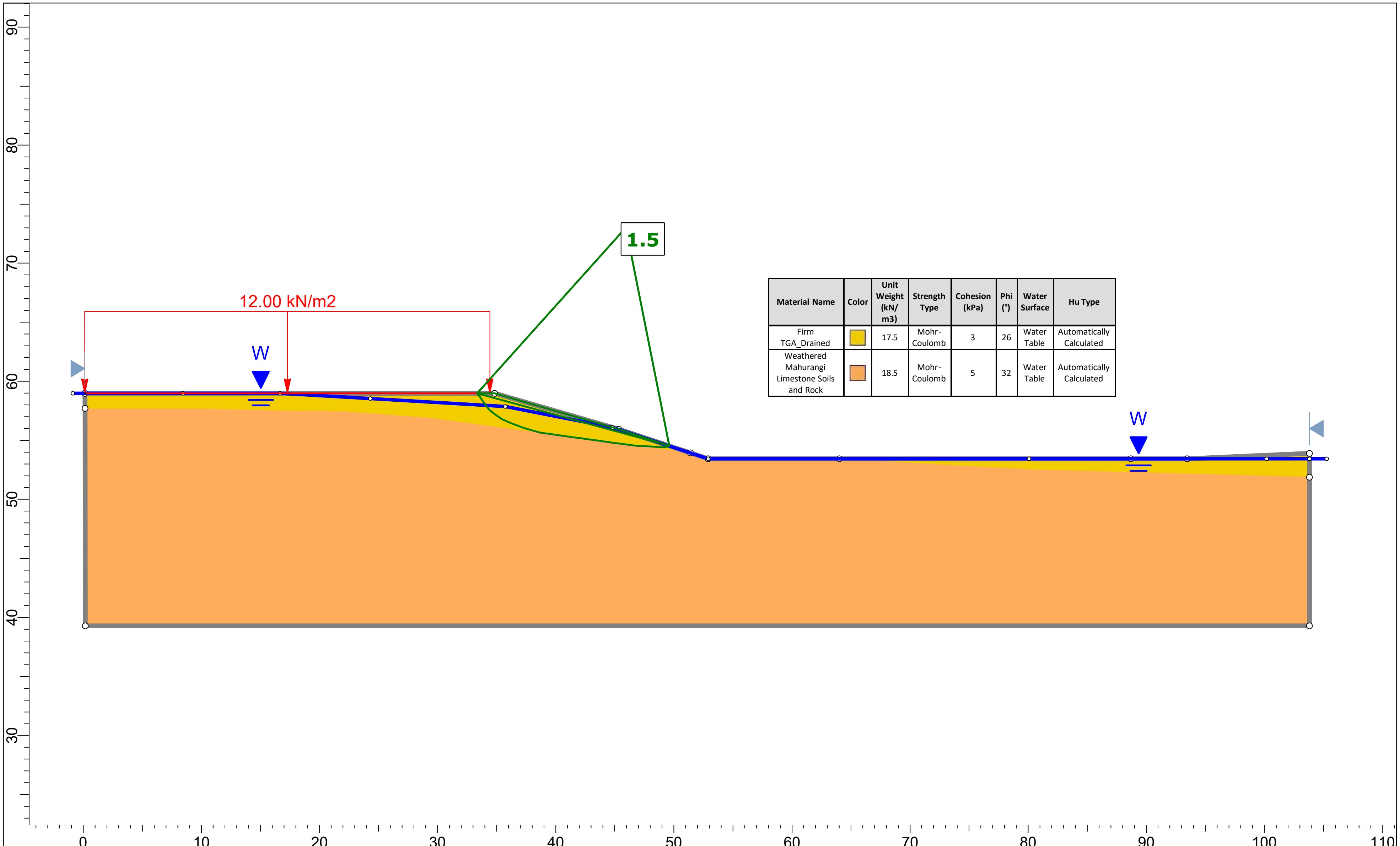
Project	ASP - Stage 2		
Analysis Description	Zone 1B - Normal Groundwater		
Drawn By	KMB	*Company*	Initia Ltd
Date	22/10/2025	*File Name*	Zone 1B.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and Rock		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated

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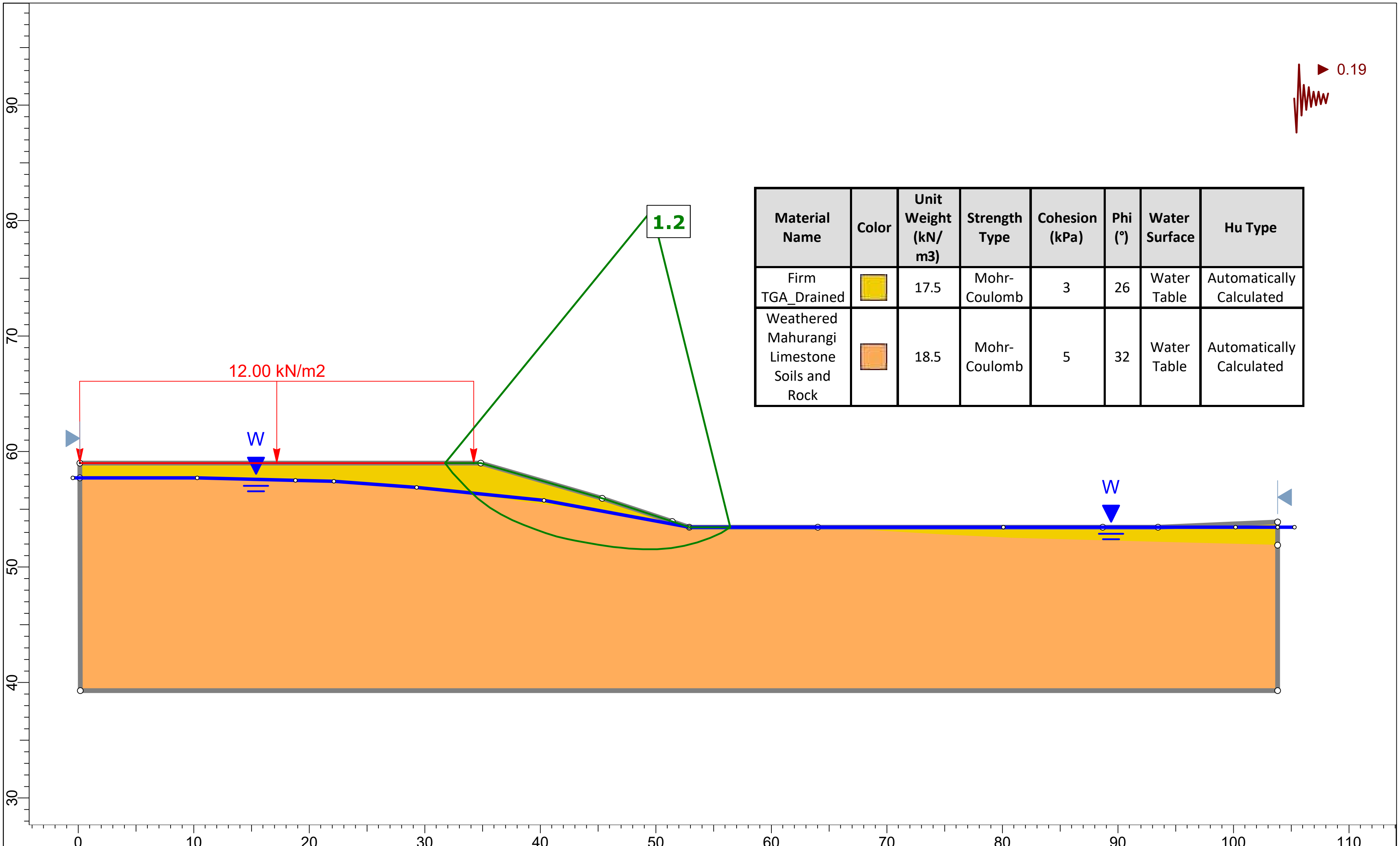
Project	ASP - Stage 2		
Analysis Description	Zone 1B - Elevated GW		
Drawn By	KMB	*Company*	Initia Ltd
Date	22/10/2025	*File Name*	Zone 1B.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and Rock		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated

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Project	ASP - Stage 2		
Analysis Description	Zone 1B - Seismic		
Drawn By	KMB	*Company*	Initia Ltd
Date	22/10/2025	*File Name*	Zone 1B.slmd

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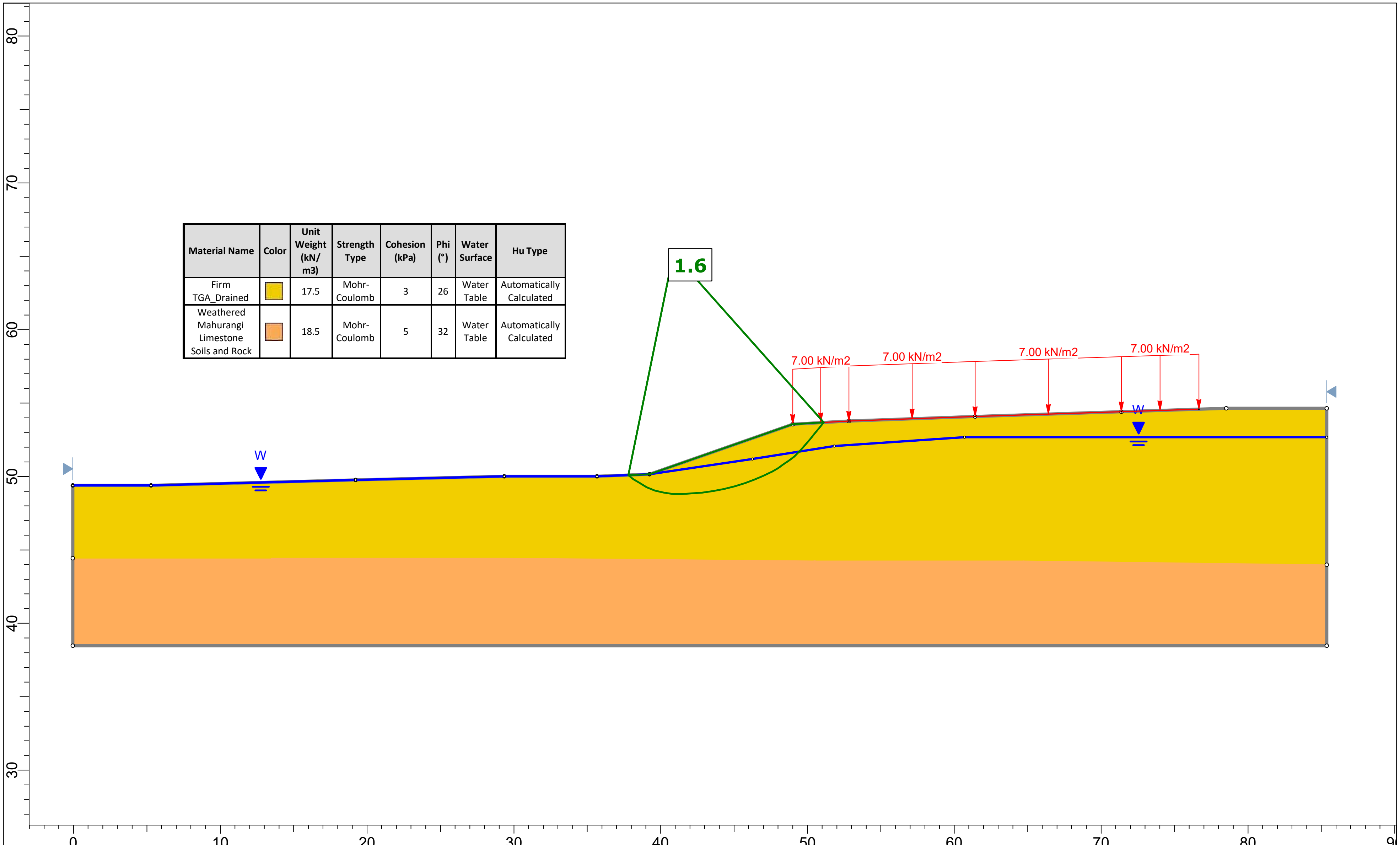

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and Rock		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated

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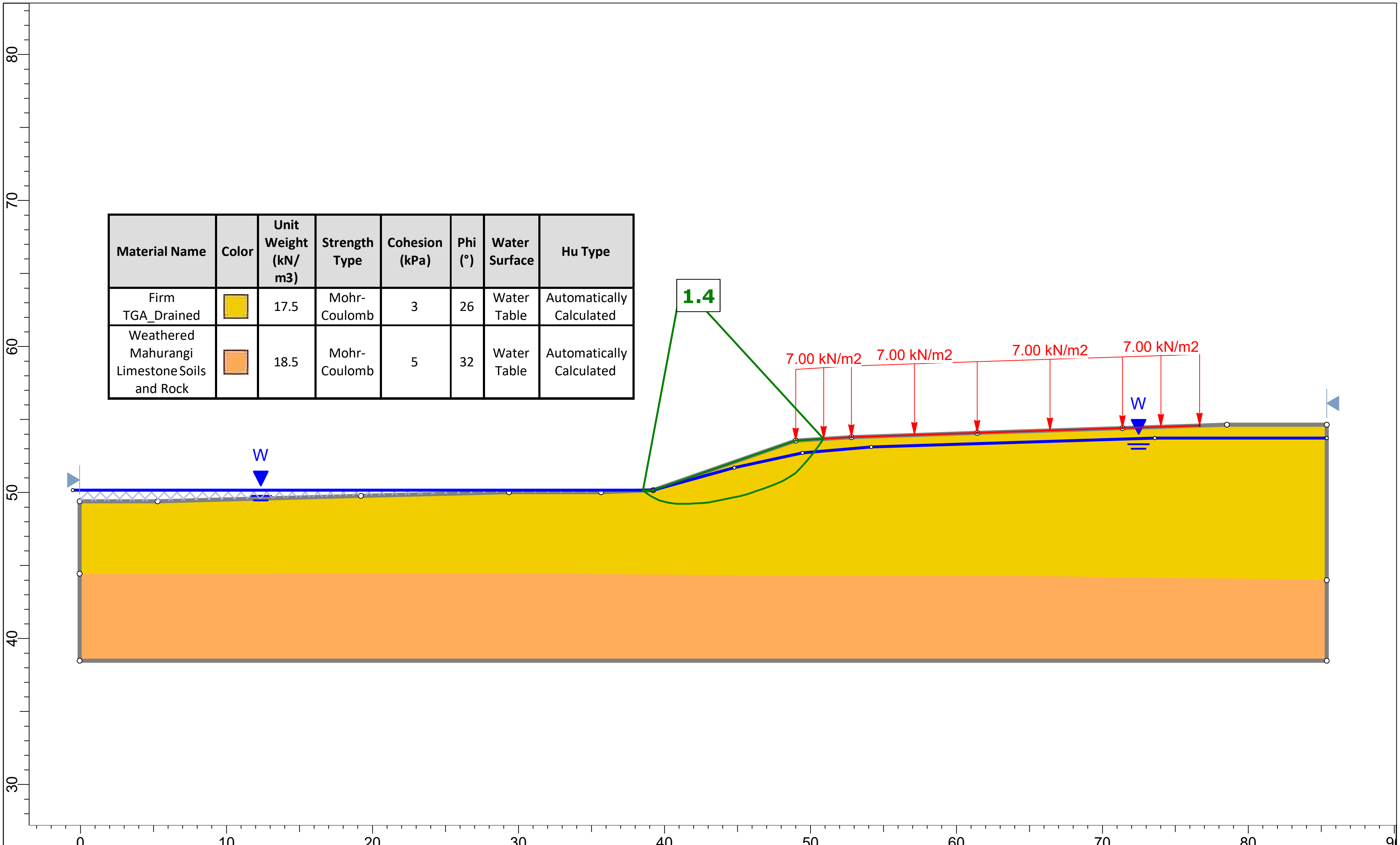
Project	ASP - Stage 2		
Group	Static Normal GW	*Scenario*	Zone 2
Drawn By	KMB	*Company*	Initia
2/11/2025	2/11/2025	*File Name*	Zone 2.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and Rock		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated

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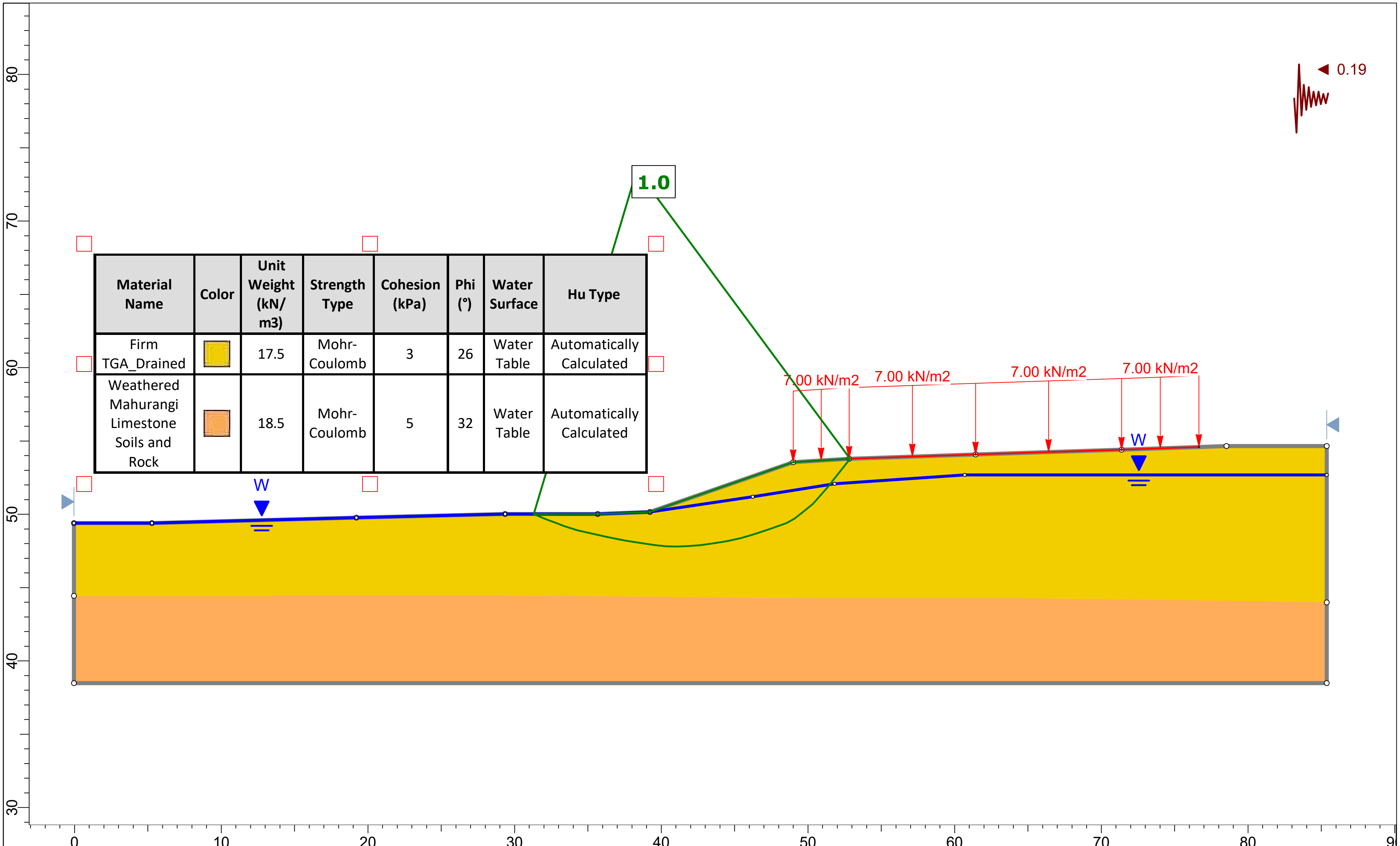
Project	ASP - Stage 2		
Group	Static Extreme GW	*Scenario*	Zone 2
Drawn By	KMB	*Company*	Initia
2/11/2025	2/11/2025	*File Name*	Zone 2.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and Rock		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated

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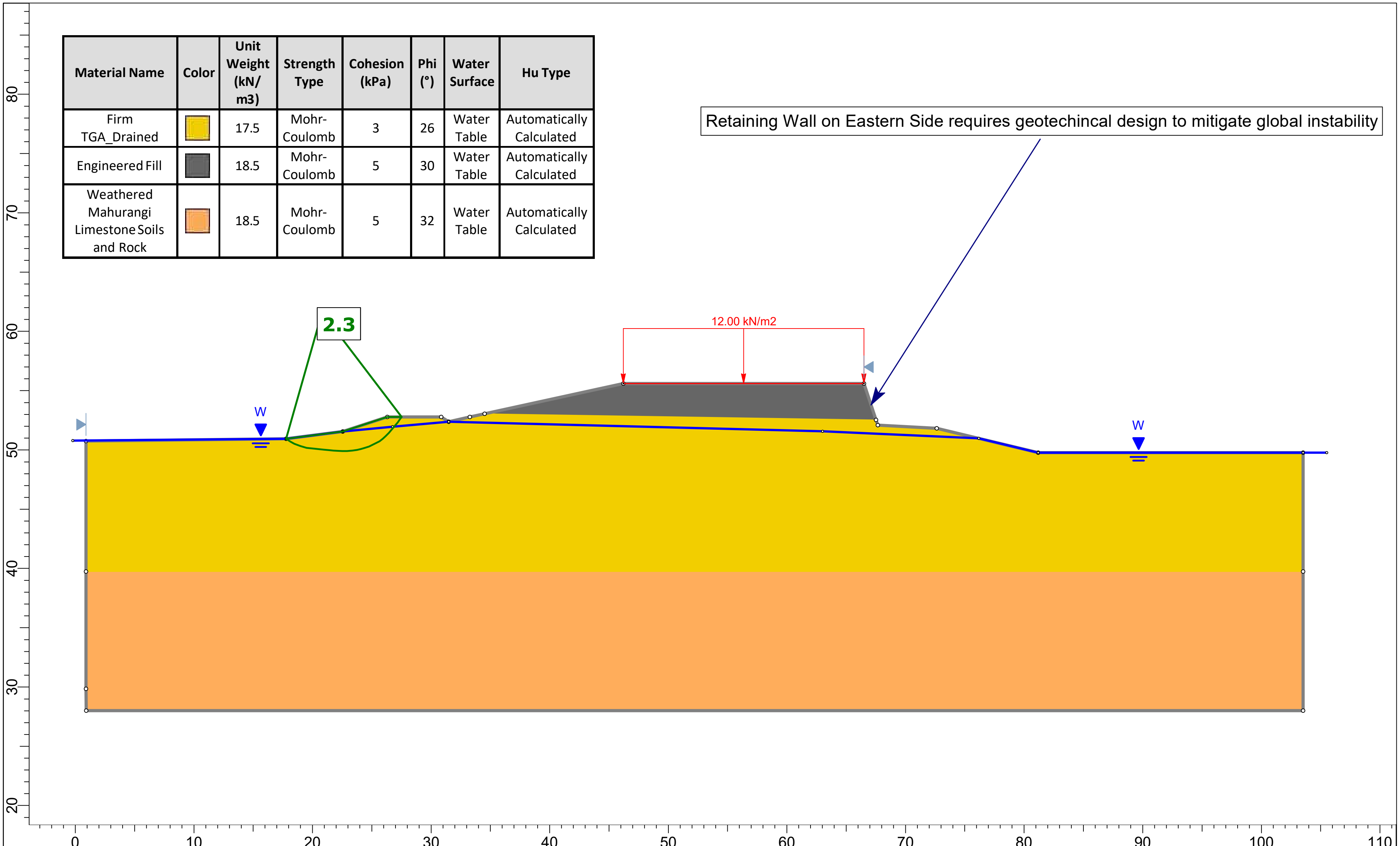
Project	ASP - Stage 2		
Group	ULS Seismic	*Scenario*	Zone 2
Drawn By	KMB	*Company*	Initia
2/11/2025	2/11/2025	*File Name*	Zone 2.slmd

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Engineered Fill		18.5	Mohr-Coulomb	5	30	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and Rock		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated



Project	ASP - Stage 2		
Group	Static Normal GW	*Scenario*	Zone 3 Critical Batter
Drawn By	KMB	*Company*	Initia
2/11/2025	2/11/2025	*File Name*	Zone 3.slmd

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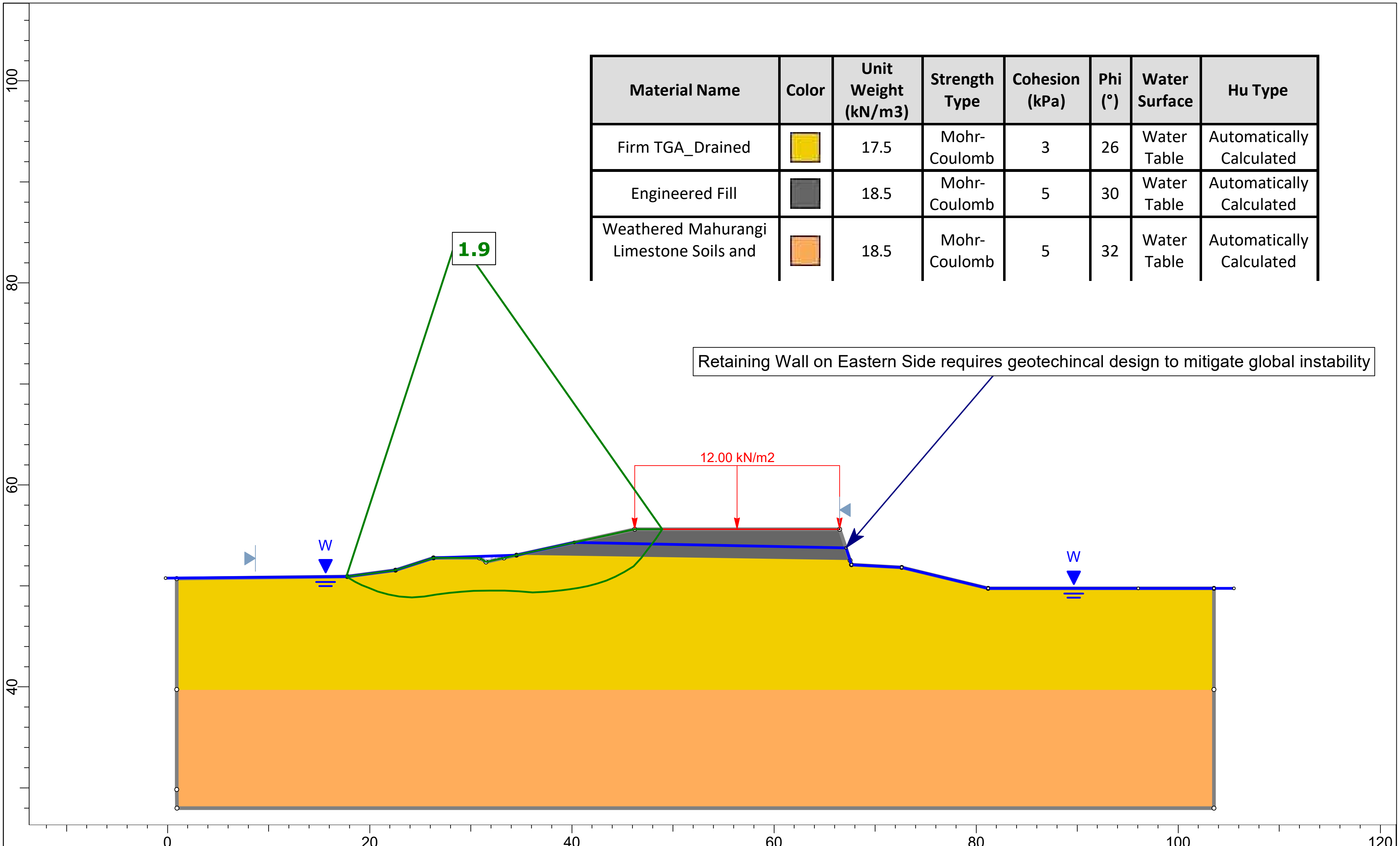

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Engineered Fill		18.5	Mohr-Coulomb	5	30	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated

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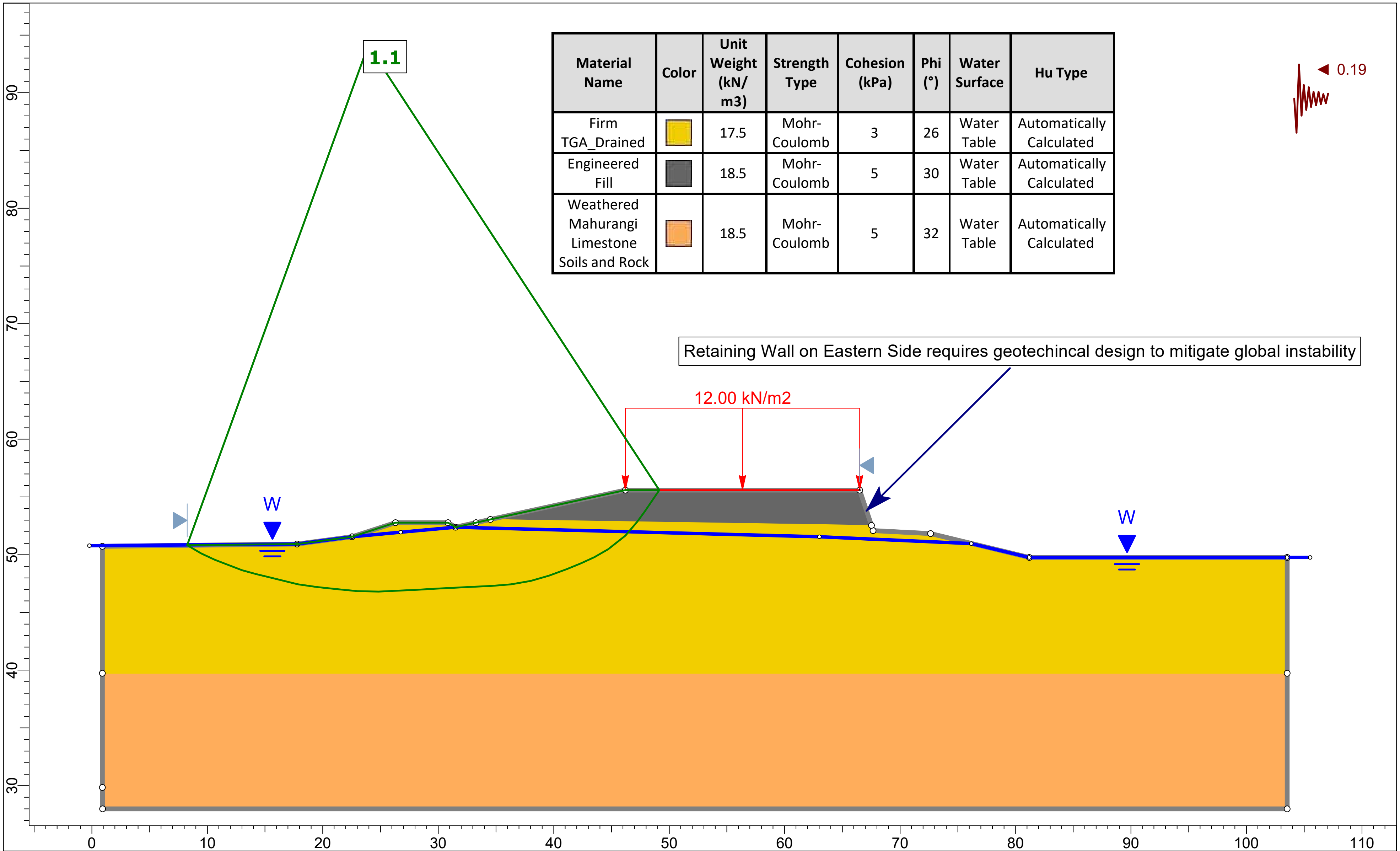
Project	ASP - Stage 2		
Group	Static Elevated GW	*Scenario*	Zone 3 Critical Batter
Drawn By	KMB	*Company*	Initia
2/11/2025	2/11/2025	*File Name*	Zone 3.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Engineered Fill		18.5	Mohr-Coulomb	5	30	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and Rock		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated

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Project	ASP - Stage 2		
Group	ULS Seismic	*Scenario*	Zone 3 Critical Batter
Drawn By	KMB	*Company*	Initia
2/11/2025	2/11/2025	*File Name*	Zone 3.slmd

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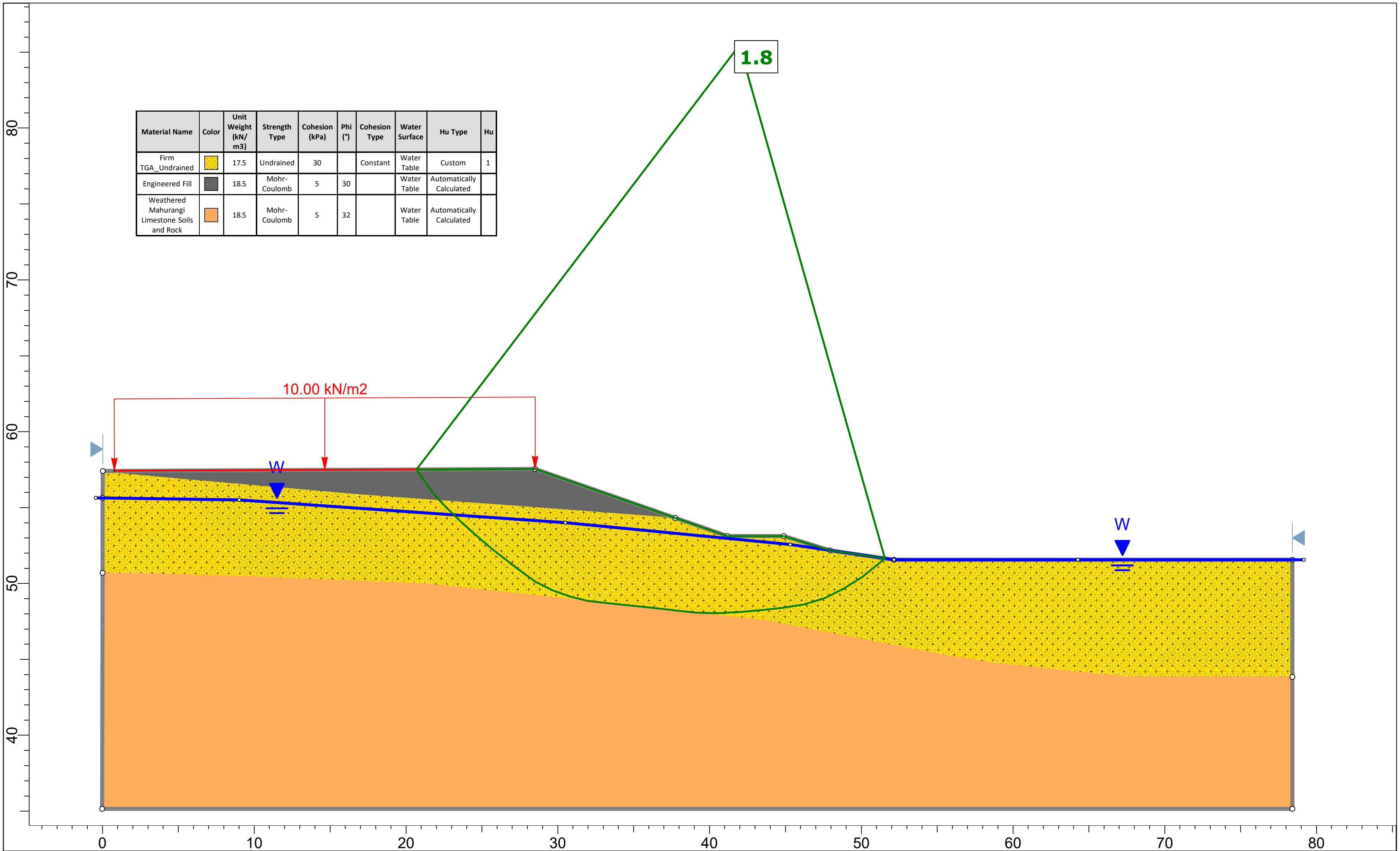

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Cohesion Type	Water Surface	Hu Type	Hu
Firm TGA_Undrained		17.5	Undrained	30		Constant	Water Table	Custom	1
Engineered Fill		18.5	Mohr-Coulomb	5	30		Water Table	Automatically Calculated	
Weathered Mahurangi Limestone Soils and Rock		18.5	Mohr-Coulomb	5	32		Water Table	Automatically Calculated	

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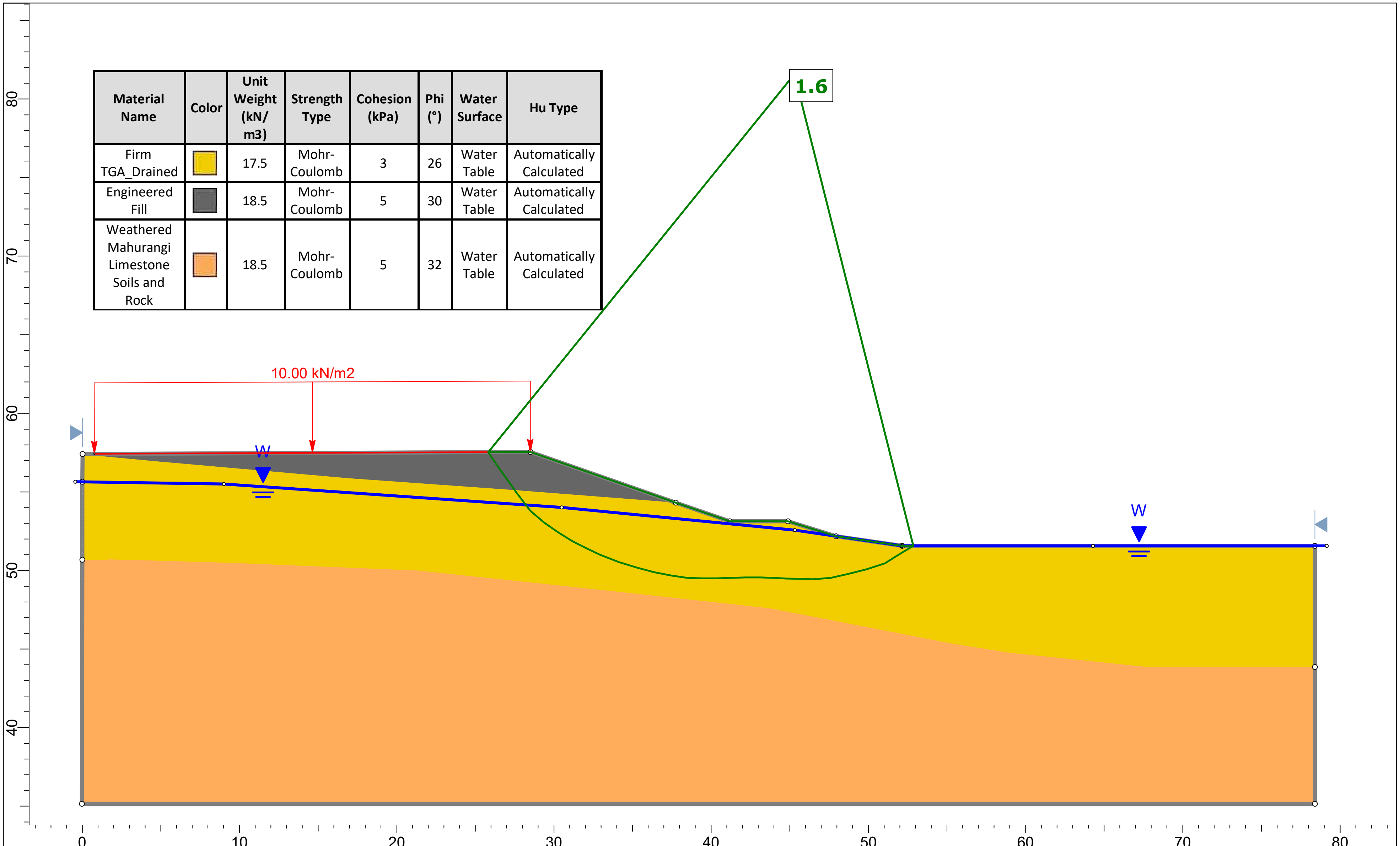
Project	ASP - Stage 2		
Analysis Description	Zone 4A - Construction (Undrained alluvium)		
Drawn By	KMB	*Company*	Initia Ltd
Date	22/10/2025	*File Name*	Zone 4A.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Engineered Fill		18.5	Mohr-Coulomb	5	30	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and Rock		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated

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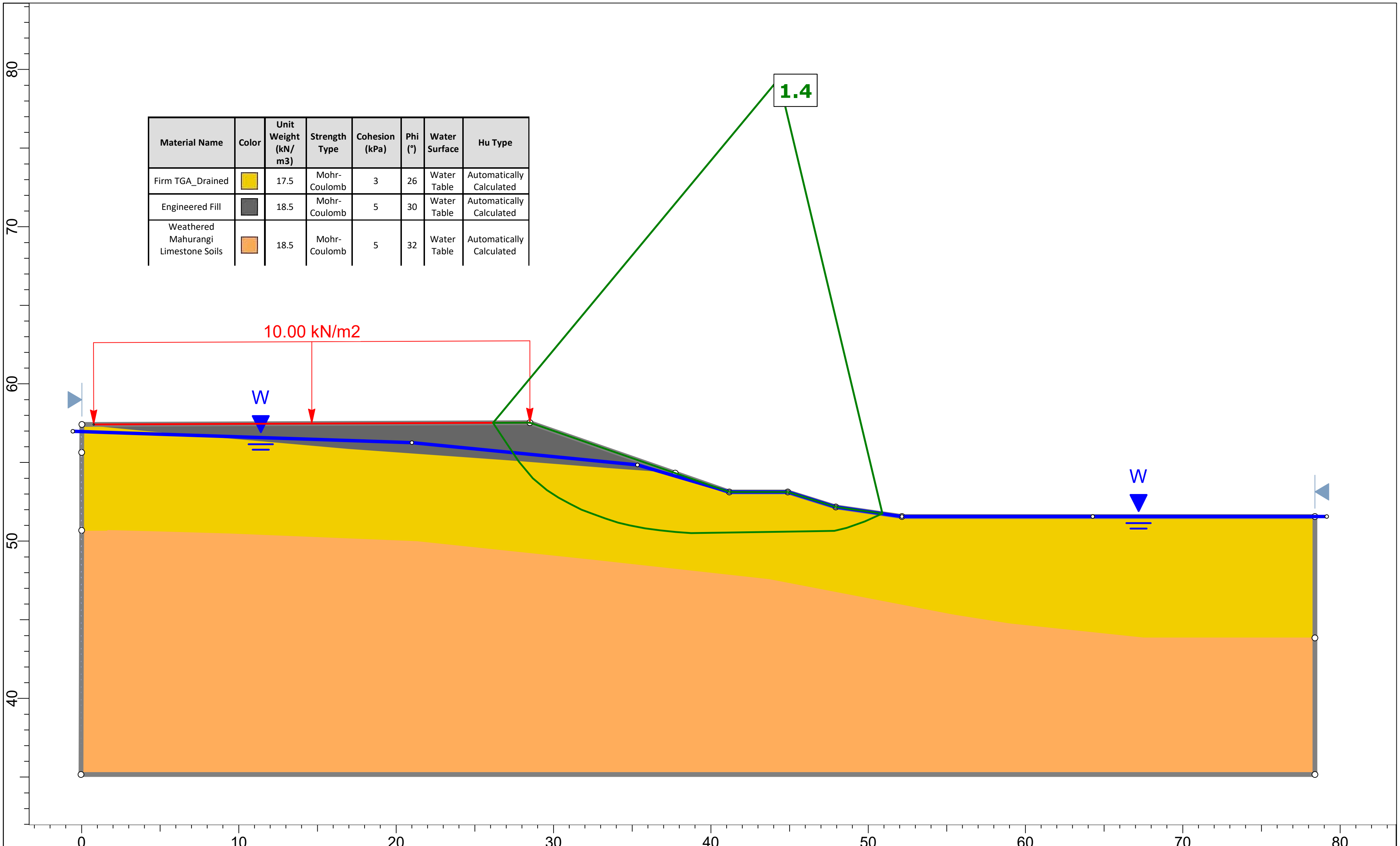
Project	ASP - Stage 2		
Analysis Description	Zone 4A - Normal GW		
Drawn By	KMB	*Company*	Initia Ltd
Date	22/10/2025	*File Name*	Zone 4A.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Engineered Fill		18.5	Mohr-Coulomb	5	30	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated

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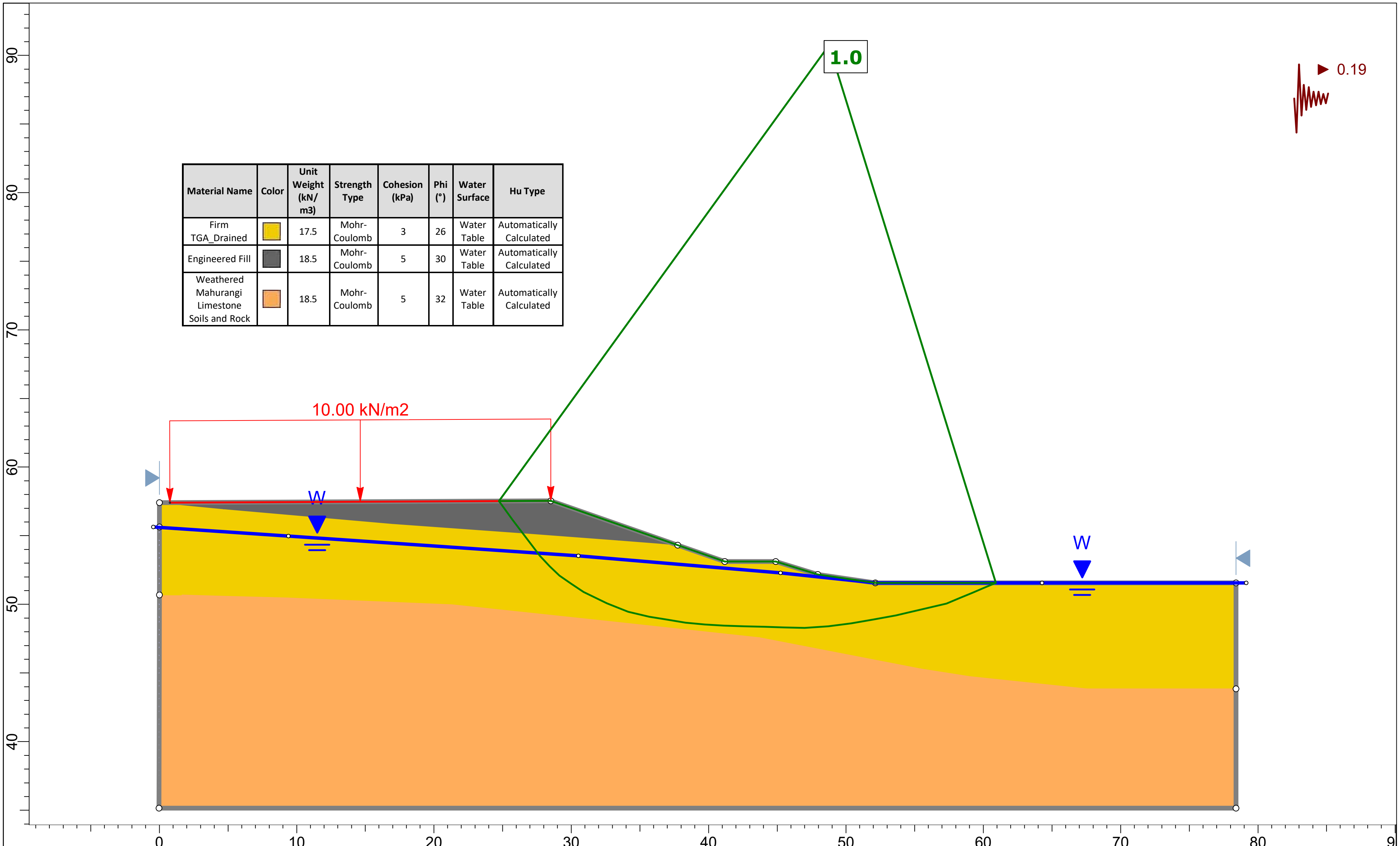
Project	ASP - Stage 2		
Analysis Description	Zone 4A - Elevated GW		
Drawn By	KMB	*Company*	Initia Ltd
Date	22/10/2025	*File Name*	Zone 4A.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Engineered Fill		18.5	Mohr-Coulomb	5	30	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and Rock		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated

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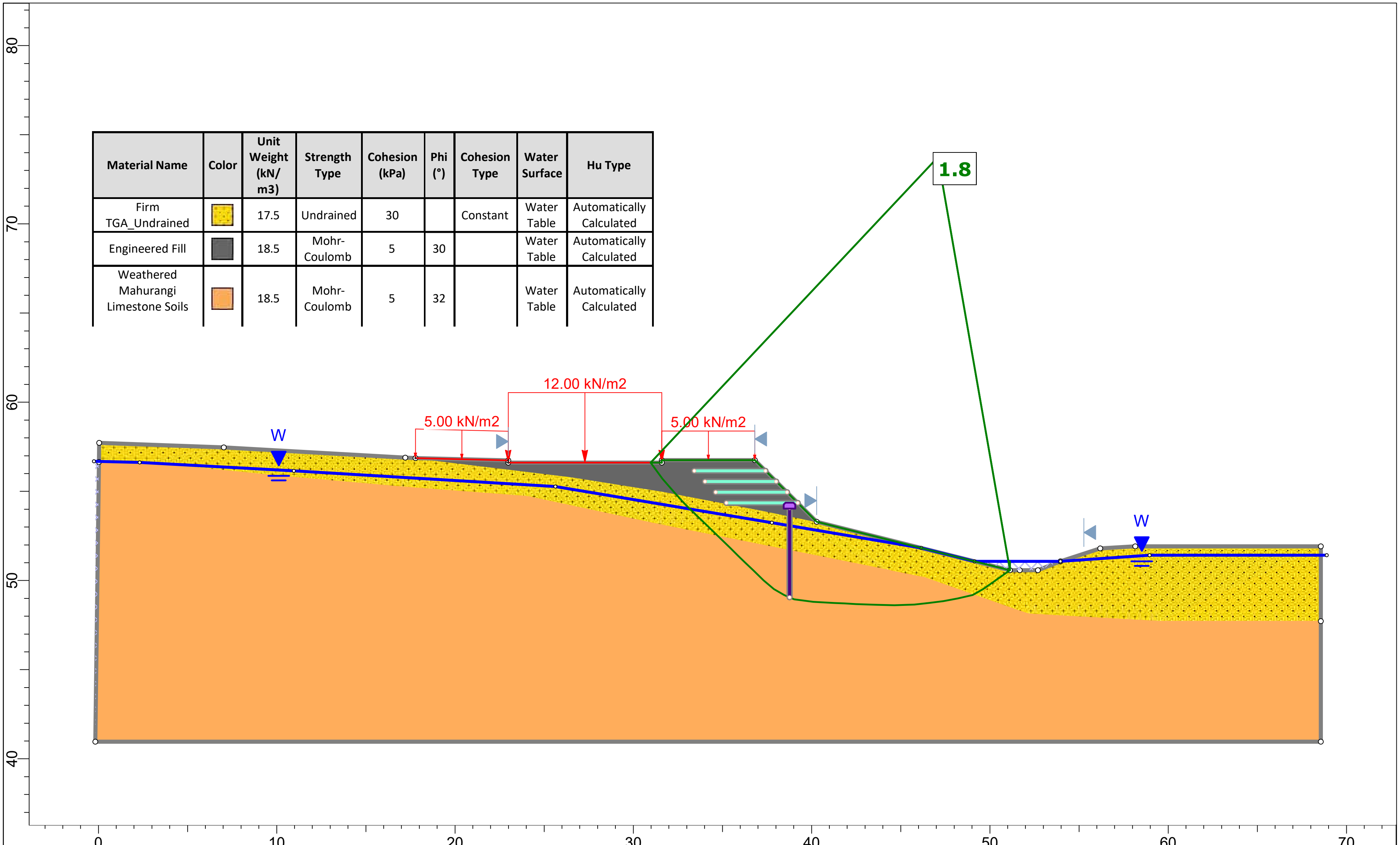
Project	ASP - Stage 2		
Analysis Description	Zone 4A - Seismic		
Drawn By	KMB	*Company*	Initia Ltd
Date	22/10/2025	*File Name*	Zone 4A.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Cohesion Type	Water Surface	Hu Type
Firm TGA_Undrained		17.5	Undrained	30		Constant	Water Table	Automatically Calculated
Engineered Fill		18.5	Mohr-Coulomb	5	30		Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils		18.5	Mohr-Coulomb	5	32		Water Table	Automatically Calculated

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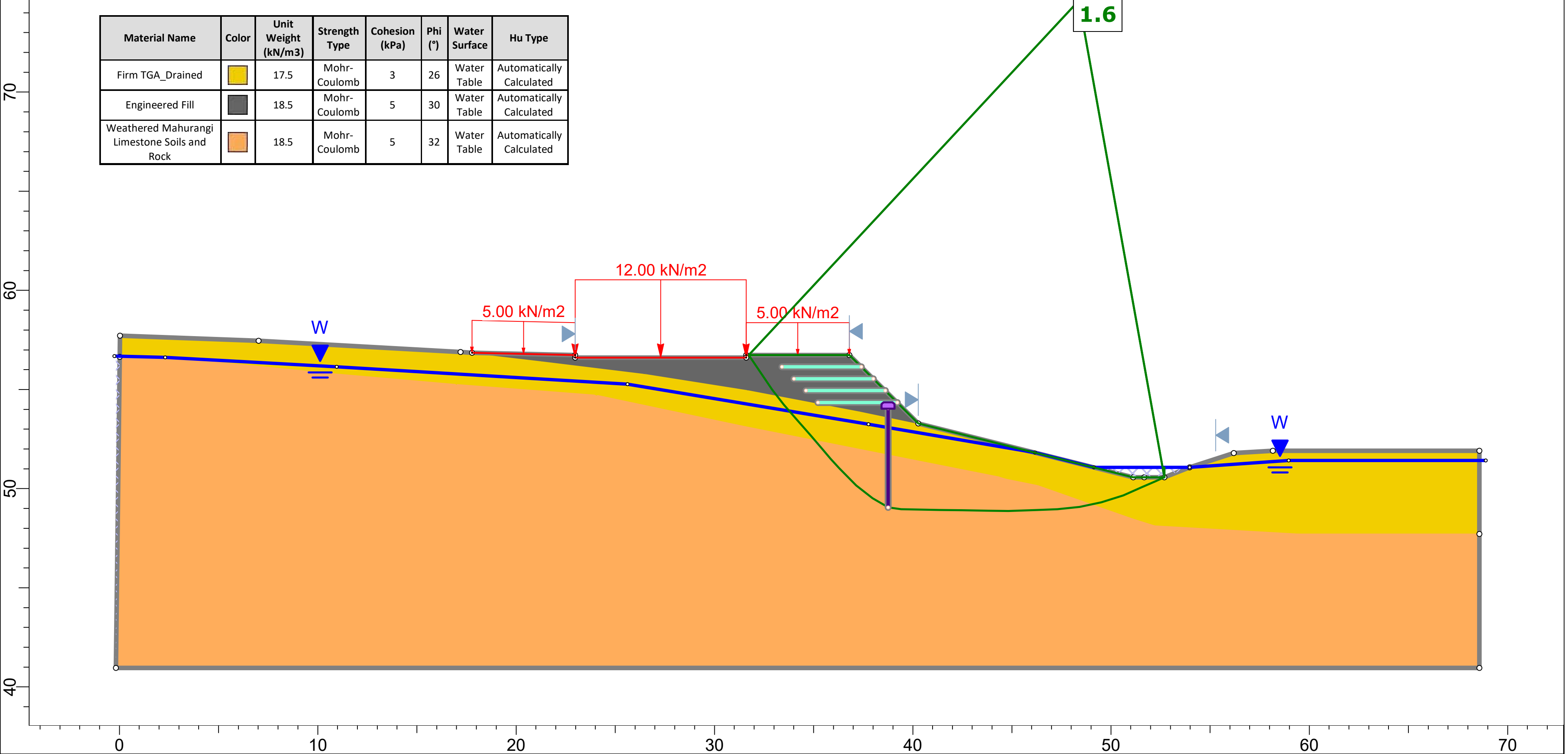
Project	ASP - Stage 2		
Analysis Description	Zone 4B - Construction Undrained		
Drawn By	KMB	*Company*	Initia Ltd
Date	22/10/2025	*File Name*	Zone 4B.slmd

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Failure Mode	Pile Shear Strength (kN)	Force Orientation	Force Orientation	Anchorage	Strip Coverage (%)	Allowable Tensile Strength (kN/m)
MSE Grid		Geosynthetic	Active (Method A)		No	Friction Angle & Adhesion	5	30	Linear				Parallel to Reinforcement	None	100	20
Palisade Pile		Pile/Micro Pile	Active (Method A)	1						Shear	50	Perpendicular to pile				

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Engineered Fill		18.5	Mohr-Coulomb	5	30	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and Rock		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated



Project	ASP - Stage 2		
Analysis Description	Zone 4B - Normal GW		
Drawn By	KMB	*Company*	Initia Ltd
Date	22/10/2025	*File Name*	Zone 4B.slmd

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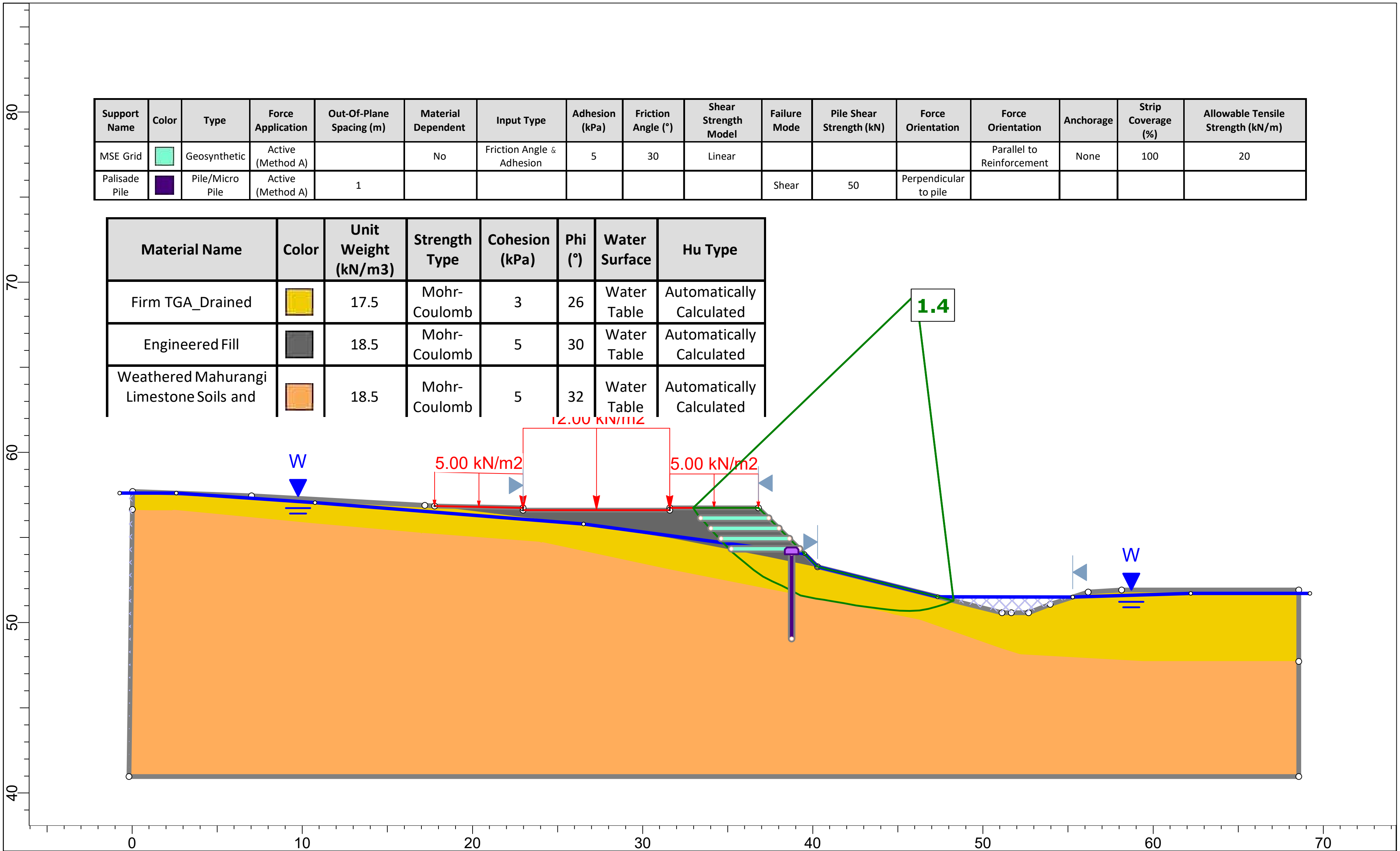
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Failure Mode	Pile Shear Strength (kN)	Force Orientation	Force Orientation	Anchorage	Strip Coverage (%)	Allowable Tensile Strength (kN/m)
MSE Grid		Geosynthetic	Active (Method A)		No	Friction Angle & Adhesion	5	30	Linear				Parallel to Reinforcement	None	100	20
Palisade Pile		Pile/Micro Pile	Active (Method A)	1						Shear	50	Perpendicular to pile				

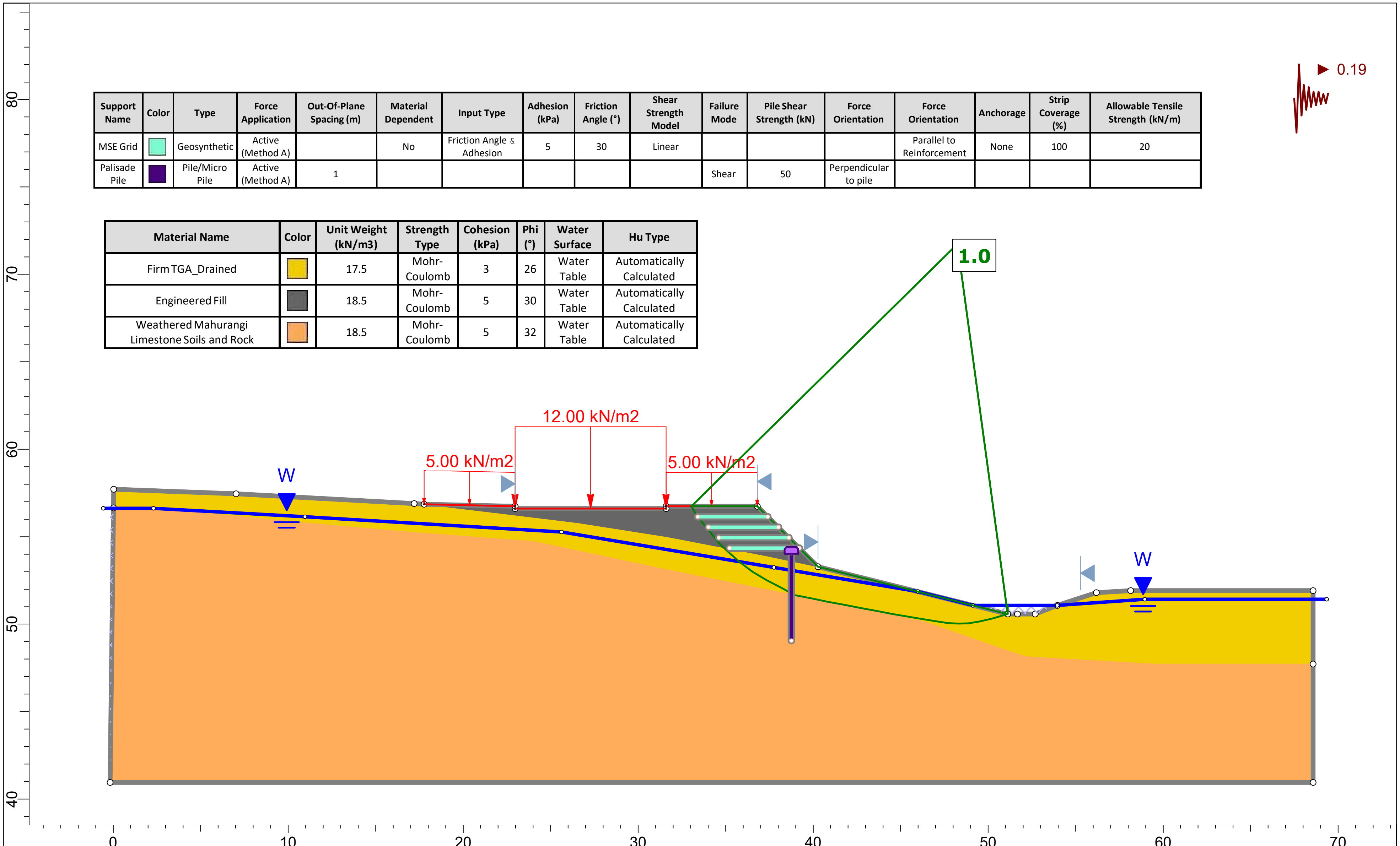
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Engineered Fill		18.5	Mohr-Coulomb	5	30	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated



INITIA
GEOTECHNICAL SPECIALISTS

SLIDEINTERPRET 9.028

Project	ASP - Stage 2		
Analysis Description	Zone 4B - Elevated GW		
Drawn By	KMB	*Company*	Initia Ltd
Date	22/10/2025	*File Name*	Zone 4B.slmd



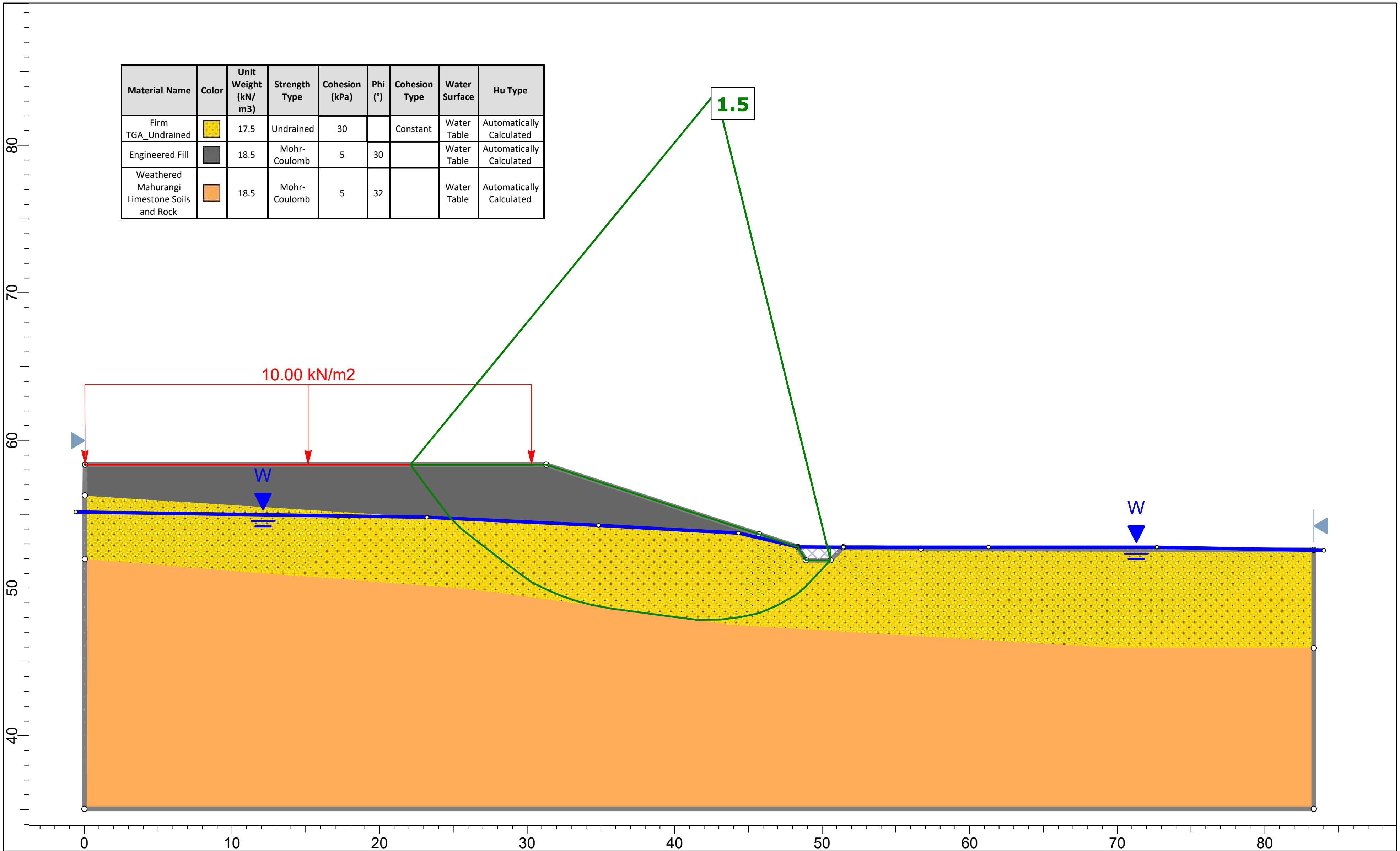
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Failure Mode	Pile Shear Strength (kN)	Force Orientation	Force Orientation	Anchorage	Strip Coverage (%)	Allowable Tensile Strength (kN/m)
MSE Grid		Geosynthetic	Active (Method A)		No	Friction Angle & Adhesion	5	30	Linear				Parallel to Reinforcement	None	100	20
Palisade Pile		Pile/Micro Pile	Active (Method A)	1						Shear	50	Perpendicular to pile				

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Engineered Fill		18.5	Mohr-Coulomb	5	30	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and Rock		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated

INITIA
GEOTECHNICAL SPECIALISTS

SLIDEINTERPRET 9.028

Project	ASP - Stage 2		
Analysis Description	Zone 4B - Seismic		
Drawn By	KMB	*Company*	Initia Ltd
Date	22/10/2025	*File Name*	Zone 4B.slmd

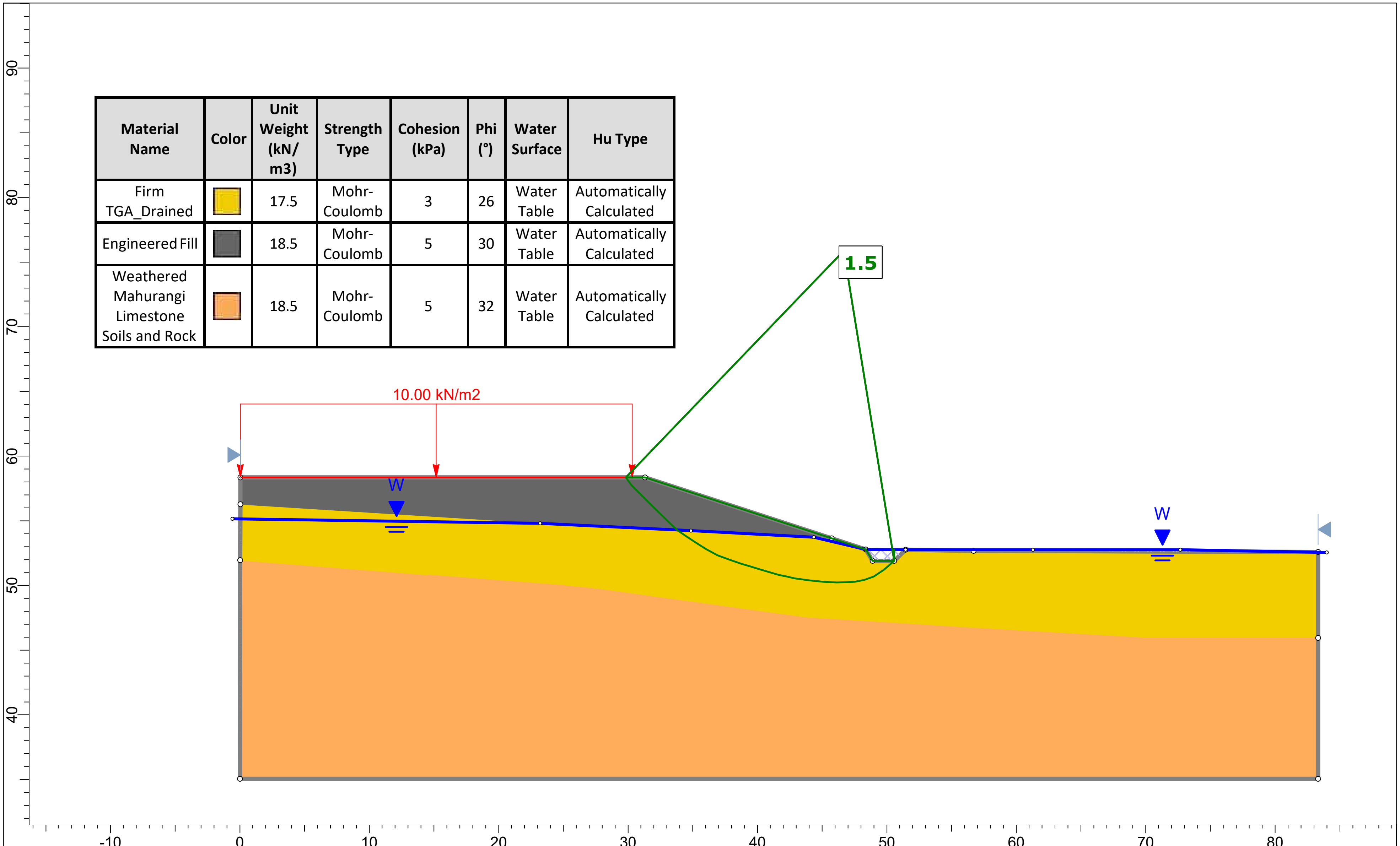


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Cohesion Type	Water Surface	Hu Type
Firm TGA_Undrained		17.5	Undrained	30		Constant	Water Table	Automatically Calculated
Engineered Fill		18.5	Mohr-Coulomb	5	30		Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and Rock		18.5	Mohr-Coulomb	5	32		Water Table	Automatically Calculated

INITIA
GEOTECHNICAL SPECIALISTS

SLIDEINTERPRET 9.028

Project	ASP - Stage 2		
Analysis Description	Zone 4C - Construction Undrained		
Drawn By	KMB	*Company*	Initia Ltd
Date	22/10/2025	*File Name*	Zone 4C.slmd



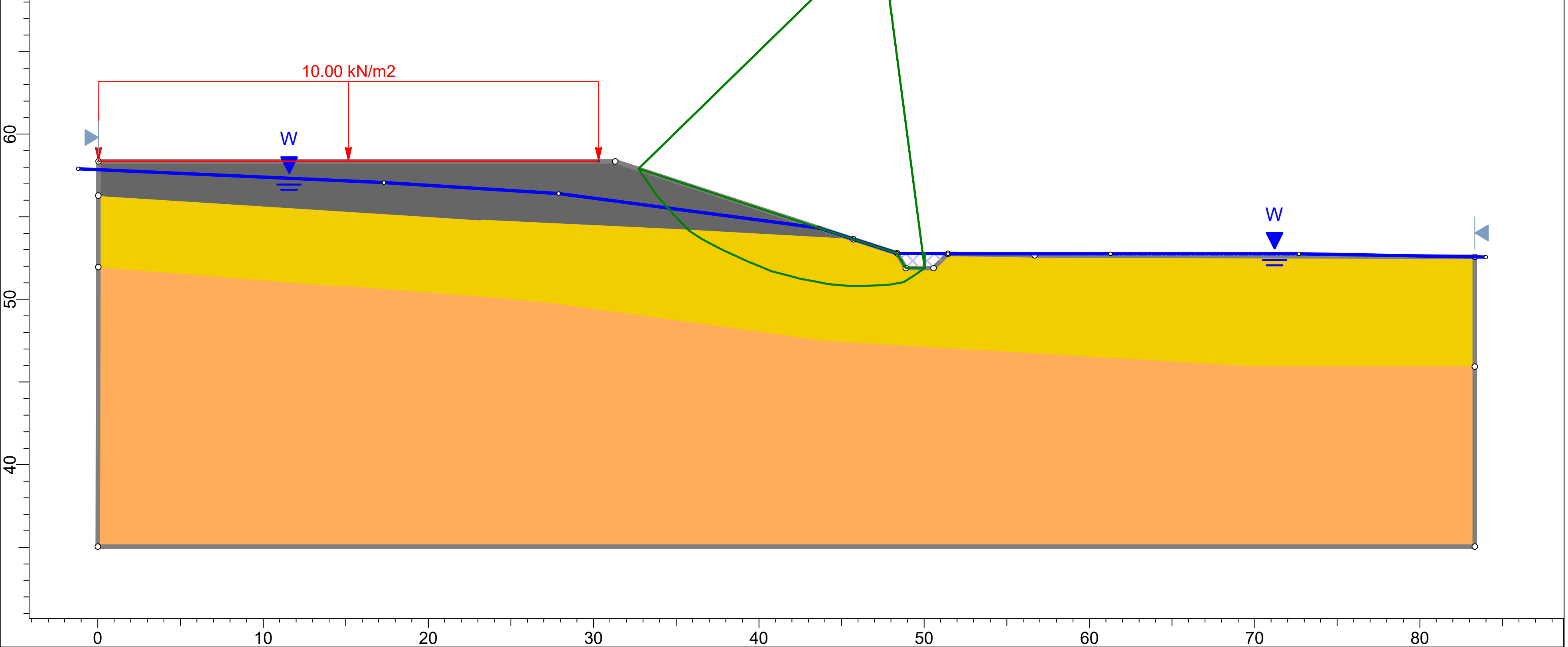
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Engineered Fill		18.5	Mohr-Coulomb	5	30	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and Rock		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated

INITIA
GEOTECHNICAL SPECIALISTS

SLIDEINTERPRET 9.028

Project	ASP - Stage 2		
Analysis Description	Zone 4C - Static Normal GW		
Drawn By	KMB	*Company*	Initia Ltd
Date	22/10/2025	*File Name*	Zone 4C.slmd

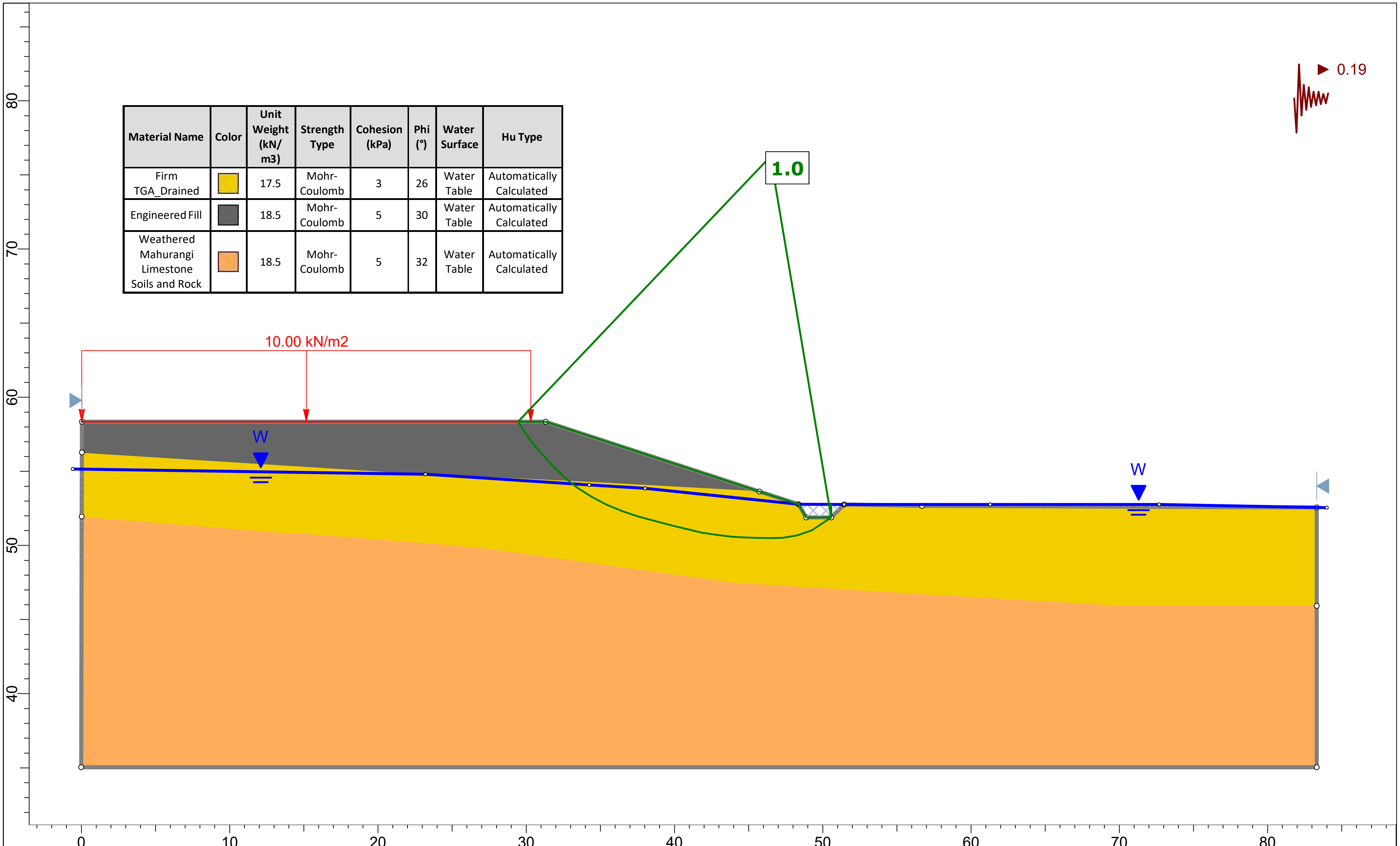
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Engineered Fill		18.5	Mohr-Coulomb	5	30	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and Rock		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated



Project	ASP - Stage 2		
Analysis Description	Zone 4C - Static Elevated GW		
Drawn By	KMB	Company	Initia Ltd
Date	22/10/2025	File Name	Zone 4C.slmd

INITIA GEOTECHNICAL SPECIALISTS

SLIDEINTERPRET 9.028



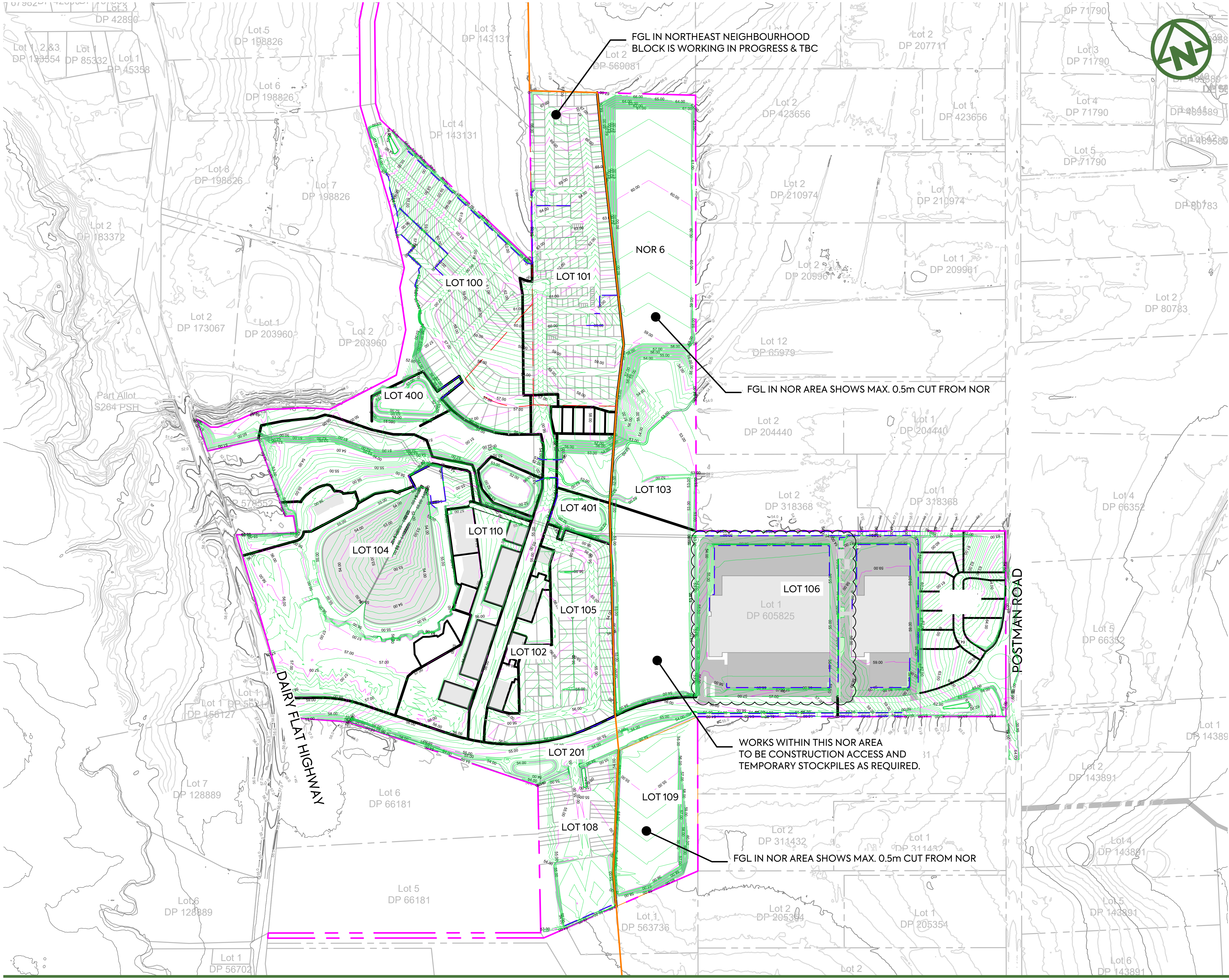
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type
Firm TGA_Drained		17.5	Mohr-Coulomb	3	26	Water Table	Automatically Calculated
Engineered Fill		18.5	Mohr-Coulomb	5	30	Water Table	Automatically Calculated
Weathered Mahurangi Limestone Soils and Rock		18.5	Mohr-Coulomb	5	32	Water Table	Automatically Calculated

INITIA
GEOTECHNICAL SPECIALISTS

SLIDEINTERPRET 9.028

Project	ASP - Stage 2		
Analysis Description	Zone 4C - Seismic		
Drawn By	KMB	*Company*	Initia Ltd
Date	22/10/2025	*File Name*	Zone 4C.slmd

Appendix D Reference Civil Earthworks Drawings



NOTES:

1. TO BE READ IN CONJUNCTION WITH ARCHITECT'S DRAWINGS. ALL LEVELS ARE IN TERMS OF AUCKLAND VERTICAL DATUM 1946 (MSL).
2. ALL EARTHWORKS SHALL COMPLY WITH THE EROSION AND SEDIMENT CONTROL GUIDELINES FOR LAND DISTURBING ACTIVITIES, AC GD2016/005.
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9. THE CONTRACTOR SHALL CONFIRM THE LOCATION, LEVELS AND DETAILING OF ALL PROPOSED GEOTECHNICAL WORKS WITH THE GEOTECHNICAL ENGINEERS ON SITE PRIOR TO PROCEEDING WITH THOSE WORKS.

LEGEND:

- SITE BOUNDARY
- EXTENT OF EARTHWORKS
- PROPOSED (1.0m) MAJOR CONTOUR
- PROPOSED (0.2m) MINOR CONTOUR
- EXISTING CONTOURS
- CADASTRAL BOUNDARY
- NOR
- FUTURE
- PROPOSED RETAINING WALL

MCKENZIE & CO.

REV	DESCRIPTION	DRN BY	CHK BY	APP BY	DATE
B	DATA CENTRE RE-DESIGN	DK	SL	-	08/12/25
A	FIRST ISSUE	AL	SL	SL	07/11/25

CLIENT: AW HOLDINGS 2021 LIMITED

PROJECT: AUCKLAND SURF PARK
1350 DAIRY FLAT HIGHWAY
DAIRY FLAT, AUCKLAND

TITLE: EARTHWORKS
FINAL CONTOUR
OVERALL PLAN

PURPOSE OF ISSUE: FOR RESOURCE CONSENT

SCALE: 1:5000
DO NOT SCALE

DRAWING NO: 3325-2-2000

REV: B

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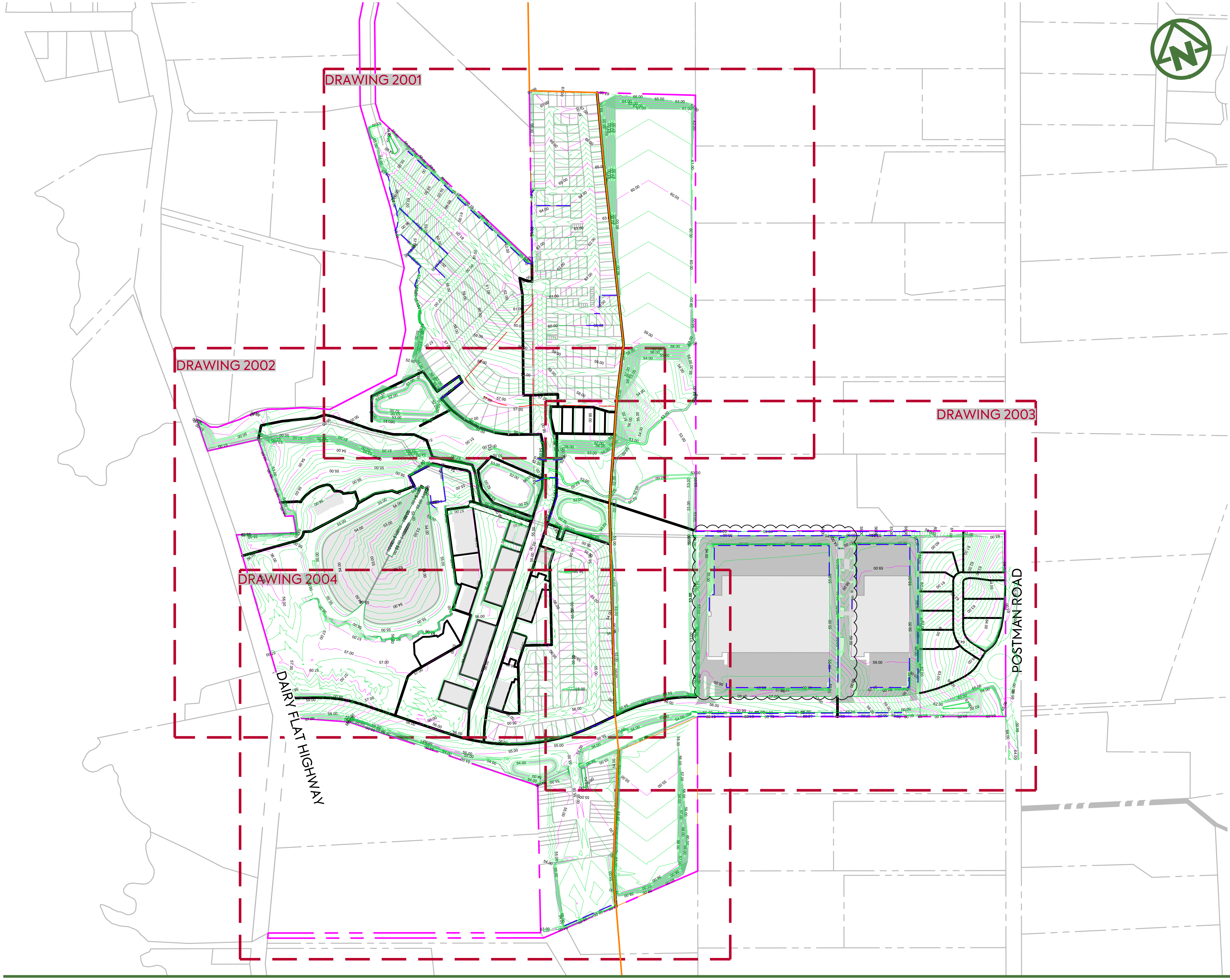

NOTES:

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LEGEND:

- SITE BOUNDARY
- EXTENT OF EARTHWORKS
- PROPOSED (1.0m) MAJOR CONTOUR
- PROPOSED (0.2m) MINOR CONTOUR
- EXISTING CONTOURS
- CADASTRAL BOUNDARY
- NOR
- FUTURE
- PROPOSED RETAINING WALL

REV	DESCRIPTION	DRN BY	CHK BY	APP BY	DATE
B	DATA CENTRE RE-DESIGN	DK	SL	-	08/12/25
A	FIRST ISSUE	AL	SL	SL	07/11/25

MCKENZIE & CO.

CLIENT: AW HOLDINGS 2021 LIMITED

PROJECT: AUCKLAND SURF PARK 1350 DAIRY FLAT HIGHWAY DAIRY FLAT, AUCKLAND

TITLE: EARTHWORKS FINAL CONTOUR PLAN KEY SHEET

PURPOSE OF ISSUE: FOR RESOURCE CONSENT

SCALE: 1:5000

DO NOT SCALE

DRAWING NO: 3325-2-2001

REV: B

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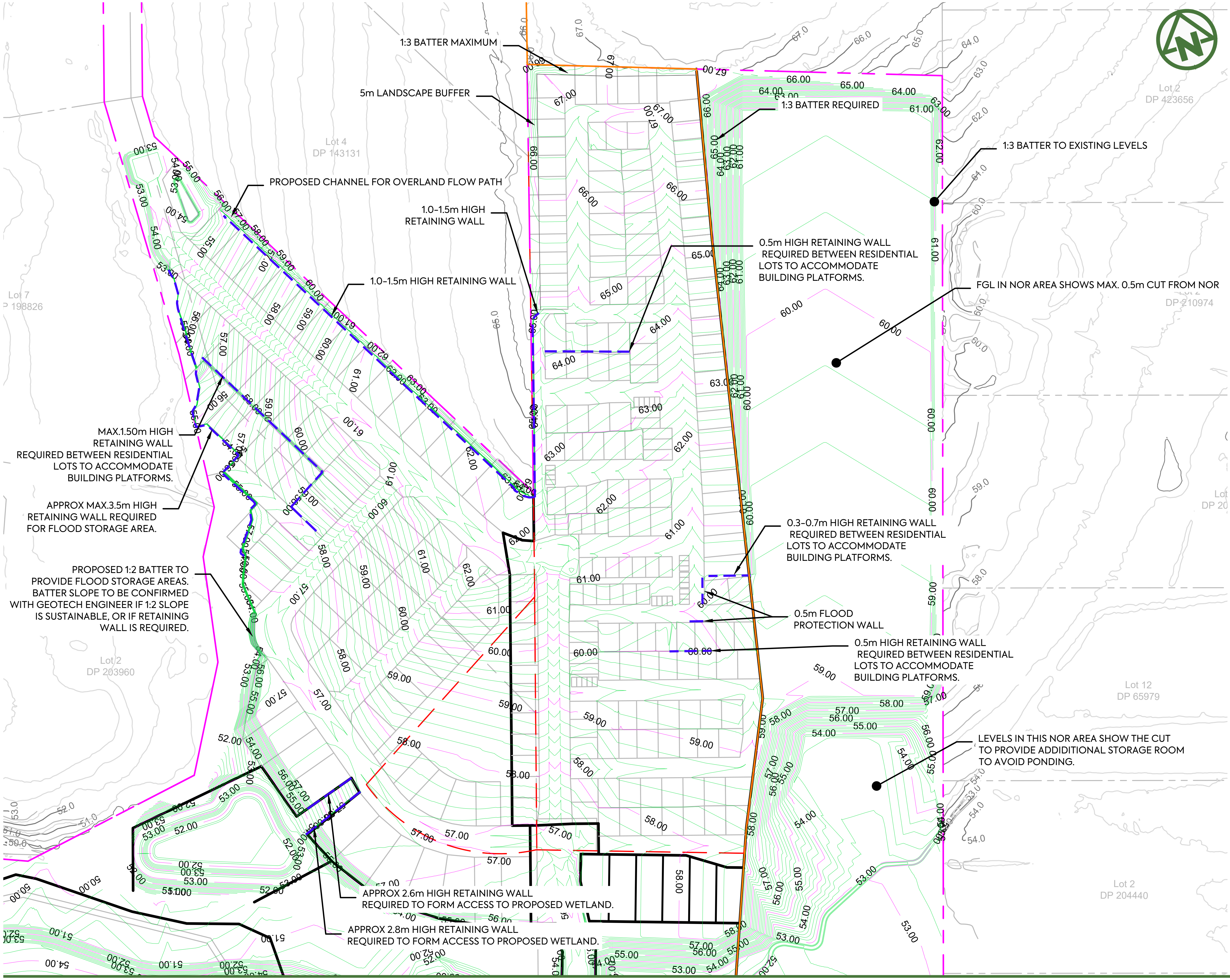

NOTES:

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LEGEND:

- SITE BOUNDARY
- EXTENT OF EARTHWORKS
- PROPOSED (1.0m) MAJOR CONTOUR
- PROPOSED (0.2m) MINOR CONTOUR
- EXISTING CONTOURS
- CADASTRAL BOUNDARY
- NOR
- FUTURE
- PROPOSED RETAINING WALL

REV	DESCRIPTION	DRN BY	CHK BY	APP BY	DATE
A	FIRST ISSUE	AL	SL	SL	07/11/25

MCKENZIE & CO.

CLIENT: AW HOLDINGS 2021 LIMITED

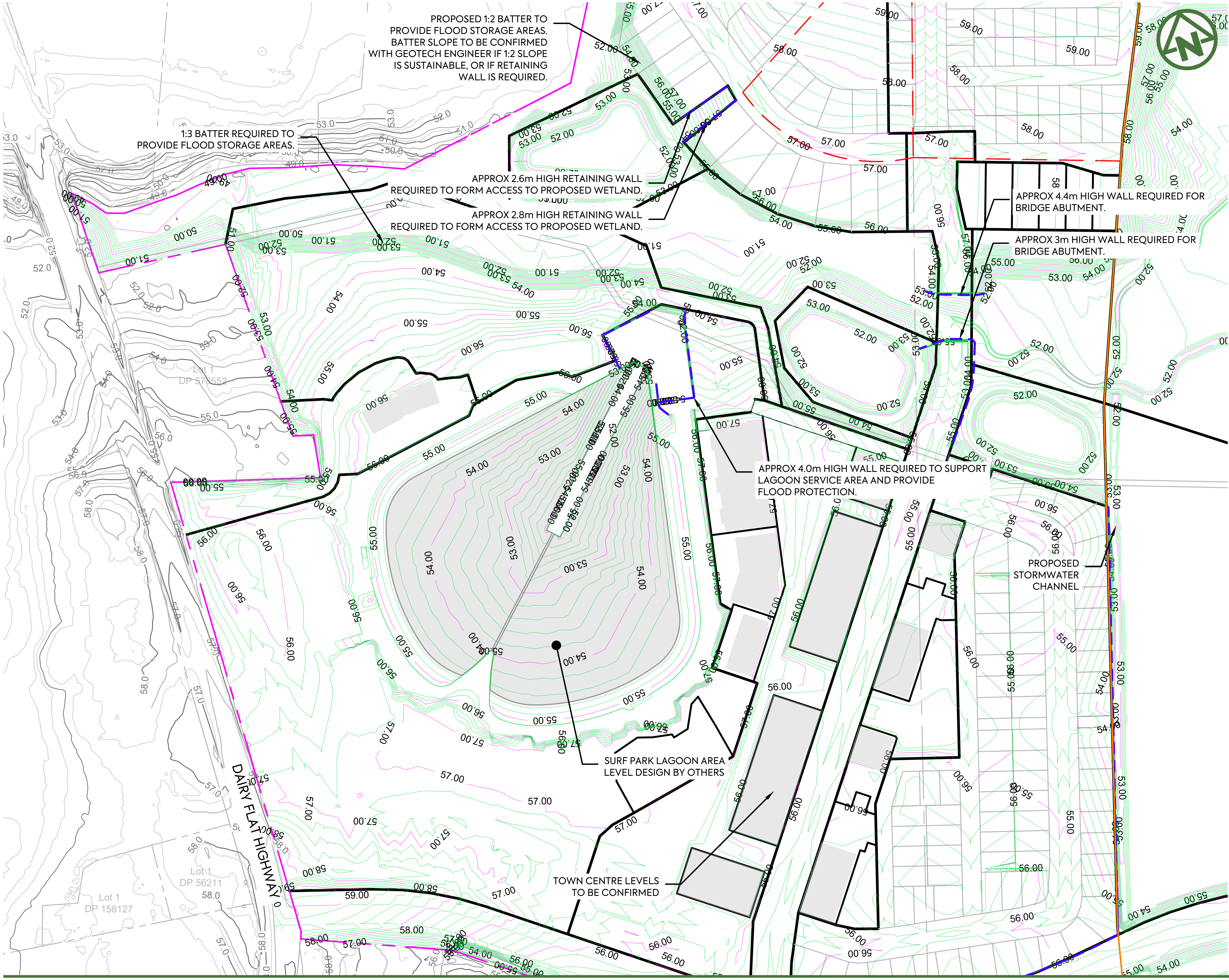
PROJECT: AUCKLAND SURF PARK
1350 DAIRY FLAT HIGHWAY
DAIRY FLAT, AUCKLAND

TITLE: EARTHWORKS
FINAL CONTOUR PLAN
SHEET 1

PURPOSE OF ISSUE: FOR RESOURCE CONSENT

SCALE: 1:2000
DO NOT SCALE

DRAWING NO: 3325-2-2002
REV: A



NOTES:

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LEGEND:

- SITE BOUNDARY
- EXTENT OF EARTHWORKS
- PROPOSED (1.0m) MAJOR CONTOUR
- PROPOSED (0.2m) MINOR CONTOUR
- EXISTING CONTOURS
- CADASTRAL BOUNDARY
- NOR
- FUTURE
- PROPOSED RETAINING WALL

REV	DESCRIPTION	DRN BY	CHK BY	APP BY	DATE
A	FIRST ISSUE	AL	SL	SL	07/11/25

MCKENZIE & CO.

CLIENT: AW HOLDINGS 2021 LIMITED

PROJECT: AUCKLAND SURF PARK, 1350 DAIRY FLAT HIGHWAY, DAIRY FLAT, AUCKLAND

TITLE: EARTHWORKS FINAL CONTOUR PLAN SHEET 2

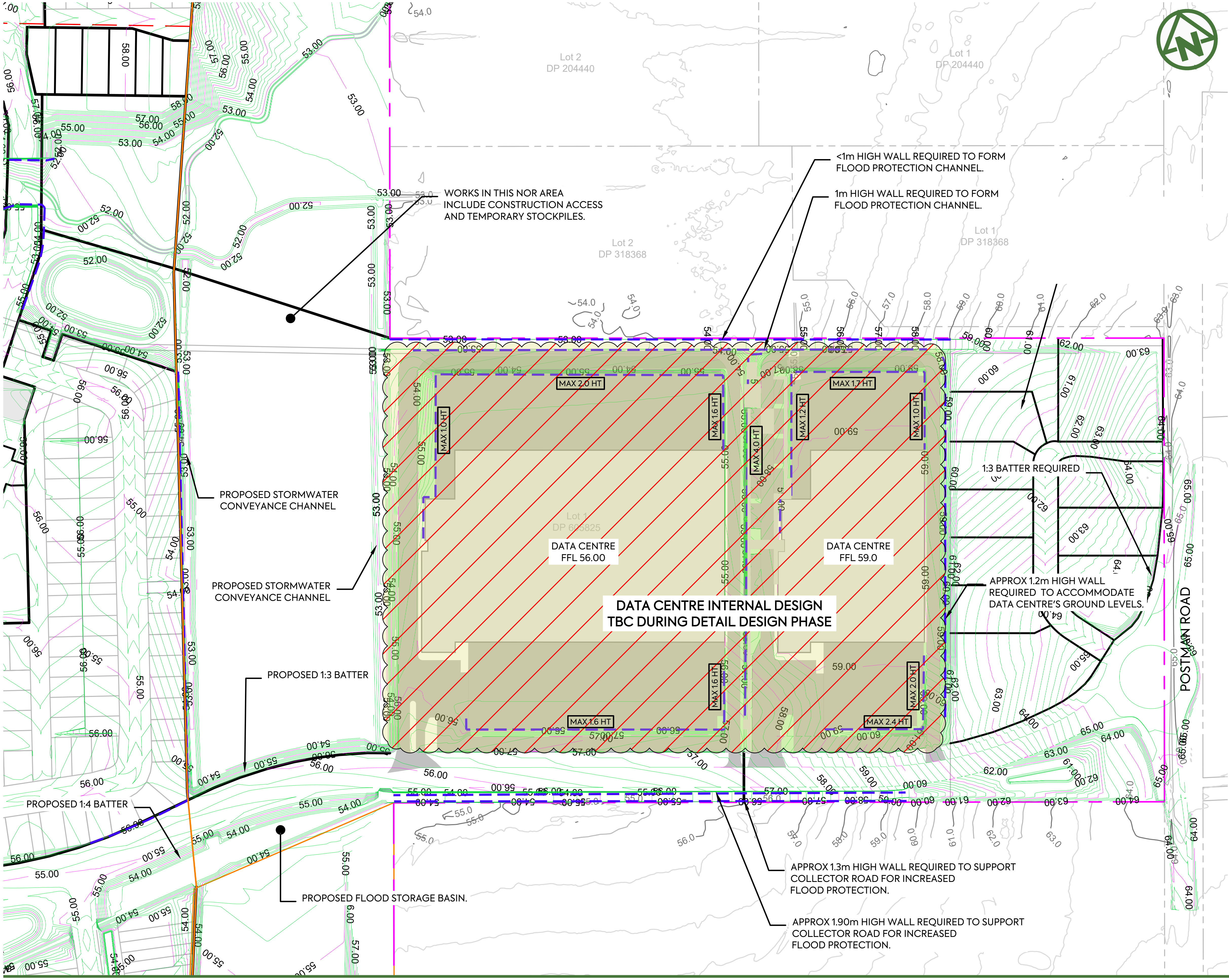
PURPOSE OF ISSUE: FOR RESOURCE CONSENT

SCALE: 1:2000

DO NOT SCALE

DRAWING NO: 3325-2-2003

REV: A



NOTES:

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- EXTENT OF EARTHWORKS
- PROPOSED (1.0m) MAJOR CONTOUR
- PROPOSED (0.2m) MINOR CONTOUR
- EXISTING CONTOURS
- CADASTRAL BOUNDARY
- NOR
- FUTURE
- PROPOSED RETAINING WALL

REV	DESCRIPTION	DRN BY	CHK BY	APP BY	DATE
B	DATA CENTRE RE-DESIGN	DK	SL	-	08/12/25
A	FIRST ISSUE	AL	SL	SL	07/11/25

MCKENZIE & CO.

CLIENT: AW HOLDINGS 2021 LIMITED

PROJECT: AUCKLAND SURF PARK, 1350 DAIRY FLAT HIGHWAY, DAIRY FLAT, AUCKLAND

TITLE: EARTHWORKS FINAL CONTOUR PLAN SHEET 3

PURPOSE OF ISSUE: FOR RESOURCE CONSENT

SCALE: 1:2000

DO NOT SCALE

DRAWING NO: 3325-2-2004

REV: B

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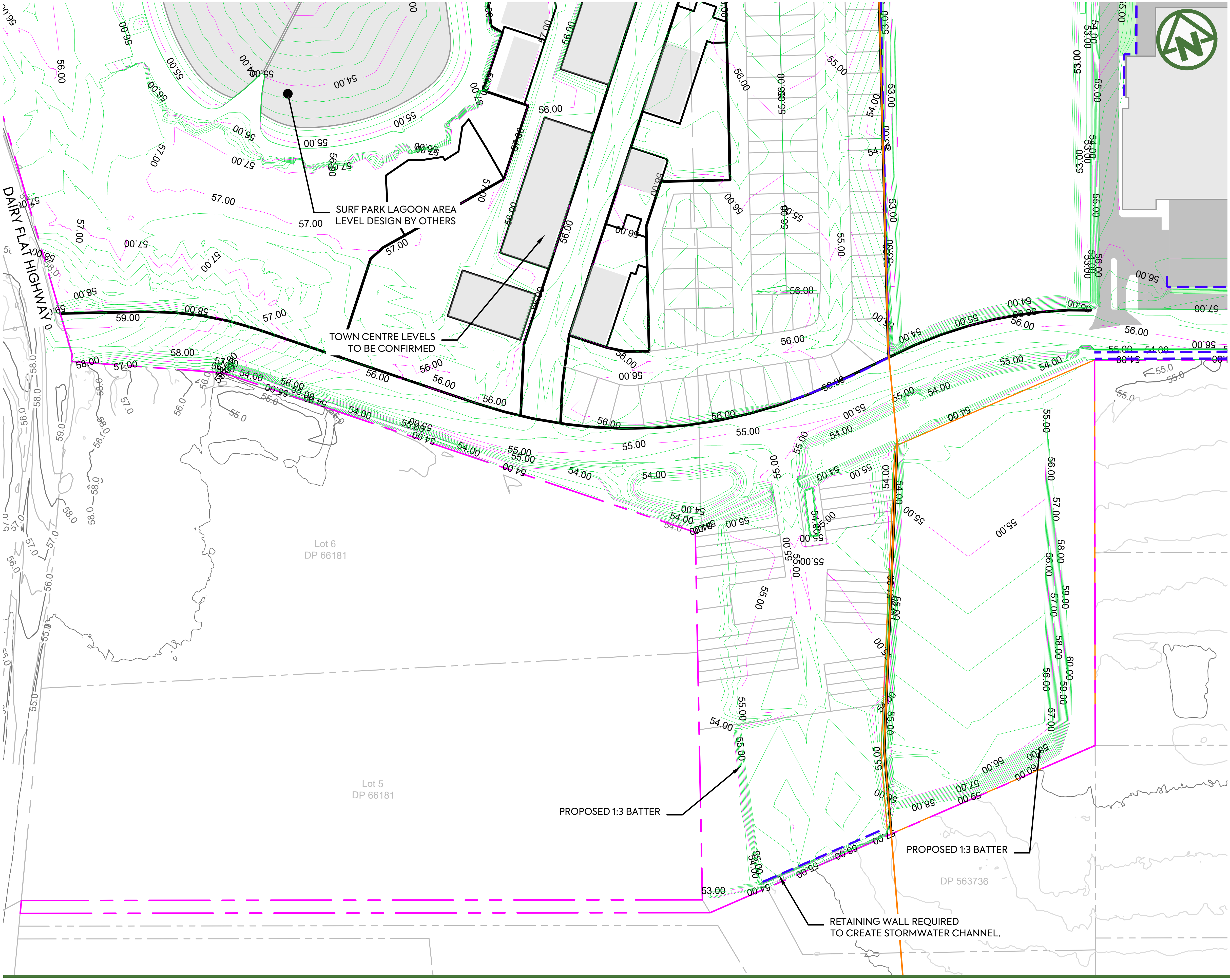

NOTES:

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LEGEND:

- SITE BOUNDARY
- EXTENT OF EARTHWORKS
- PROPOSED (1.0m) MAJOR CONTOUR
- PROPOSED (0.2m) MINOR CONTOUR
- EXISTING CONTOURS
- CADASTRAL BOUNDARY
- NOR
- FUTURE
- PROPOSED RETAINING WALL

REV	DESCRIPTION	DRN BY	CHK BY	APP BY	DATE
A	FIRST ISSUE	AL	SL	SL	07/11/25

MCKENZIE & CO.

CLIENT: AW HOLDINGS 2021 LIMITED

PROJECT: AUCKLAND SURF PARK
1350 DAIRY FLAT HIGHWAY
DAIRY FLAT, AUCKLAND

TITLE: EARTHWORKS
FINAL CONTOUR PLAN
SHEET 4

PURPOSE OF ISSUE: FOR RESOURCE CONSENT

SCALE: 1:2000
DO NOT SCALE

DRAWING NO: 3325-2-2005
REV: A

NOTES:

1. ALL LEVELS ARE IN TERMS OF AUCKLAND VERTICAL DATUM 1946 (MSL). THE CONTOUR INTERVAL IS: 0.50m
2. CUT / FILL BETWEEN AS-BUILT SURFACE (SURVEYED BY MCCL IN JUNE 2025) AND ASSUMED TOPSOIL STRIPPING FOR AREAS NOT PREVIOUSLY WORKED, STAGE 2 SUBGRADE SURFACE.
   - SUBGRADE LEVEL ASSUMED **250 mm BELOW DESIGN SURFACE** FOR LOTS, ROAD BERMS, AND FOOTPATHS.
   - SUBGRADE LEVEL ASSUMED **500 mm BELOW DESIGN SURFACE** FOR PUBLIC CARRIAGEWAYS.
3. TOTAL EARTHWORK VOLUMES TBC
   VOLUME OF CUT: -275,922 m^3
   VOLUME OF FILL: +282,546 m^3
   BALANCE: +6,624 m^3
   AREA: 50,4800m^2
4. NO COMPACTION, BULKING FACTORS HAVE BEEN APPLIED.
5. TOPSOIL STRIPPING DEPTHS TBCAND VOLUMES SUBJECT TO CHANGE.

LEGEND:

- SITE BOUNDARY
- PROPOSED FILL
- PROPOSED CUT
- PROPOSED FILL (MAJOR CONTOUR)
- PROPOSED CUT (MAJOR CONTOUR)
- CUT/FILL (ZERO CONTOUR)
- CADASTRAL BOUNDARY
- 100yr ARI FLOOD PLAIN

REV	DESCRIPTION	DRN BY	CHK BY	APP BY	DATE
B	DATA CENTRE RE-DESIGN	DK	SL	-	08/12/25
A	FIRST ISSUE	AL	SL	SL	07/11/25

MCKENZIE & CO.

CLIENT:
AW HOLDINGS 2021 LIMITED

PROJECT:
AUCKLAND SURF PARK
1350 DAIRY FLAT HIGHWAY
DAIRY FLAT, AUCKLAND

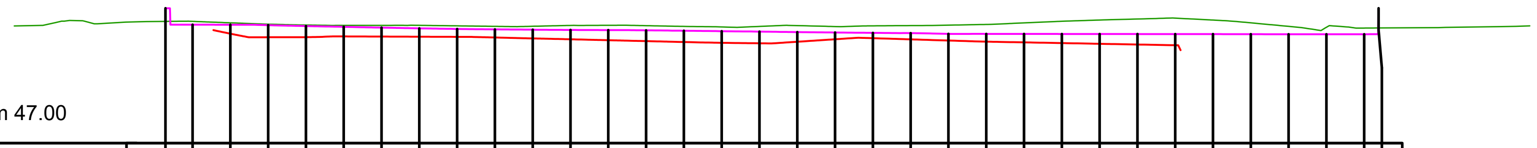
TITLE:
EARTHWORKS
CUT FILL
OVERALL PLAN

PURPOSE OF ISSUE:
FOR RESOURCE CONSENT

SCALE:
1:5000
DO NOT SCALE

DRAWING NO: 3325-2-2100
REV: B

C:\12DS\DATA\MCKFS01\3325-2 SURF PARK STAGE 2_5052\DESIGN\6.5. DRAWINGS\3325-2-2100.DWG

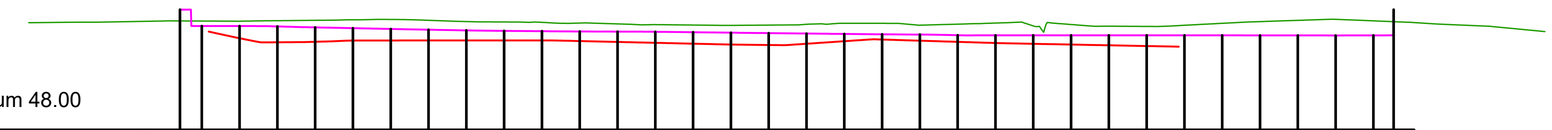


Datum 47.00

DESIGN LEVEL	TOPO GROUND	Cut/Fill	EX. DESIGN LEVEL	Cut/Fill
55.95	55.08	-0.87		
54.86	55.08	0.22		
54.86	54.99	0.13	54.26	-0.59
54.83	54.89	0.07	54.03	-0.80
54.77	54.84	0.07	54.03	-0.74
54.72	54.81	0.09	54.08	-0.64
54.67	54.82	0.15	54.07	-0.59
54.62	54.82	0.20	54.06	-0.56
54.58	54.78	0.21	54.05	-0.52
54.55	54.75	0.20	54.01	-0.54
54.53	54.76	0.23	53.95	-0.58
54.51	54.81	0.30	53.89	-0.62
54.50	54.82	0.32	53.83	-0.67
54.49	54.80	0.31	53.77	-0.72
54.46	54.75	0.29	53.71	-0.75
54.43	54.71	0.28	53.66	-0.77
54.40	54.75	0.35	53.63	-0.77
54.37	54.80	0.43	53.73	-0.64
54.34	54.74	0.39	53.89	-0.45
54.32	54.78	0.46	53.96	-0.35
54.30	54.80	0.50	53.89	-0.41
54.25	54.83	0.58	53.81	-0.44
54.25	54.88	0.63	53.74	-0.52
54.25	54.98	0.73	53.69	-0.56
54.25	55.09	0.84	53.64	-0.60
54.24	55.17	0.93	53.59	-0.65
54.24	55.24	1.00	53.55	-0.69
54.24	55.30	1.06	53.50	-0.74
54.24	55.17	0.93		
54.23	54.98	0.75		
54.23	54.75	0.52		
54.22	54.69	0.46		
54.23	54.64	0.41		
55.95	54.64	-1.31		

LONGITUDINAL SECTION C

SCALE: HORIZ=1:1000
VERT=1:500



Datum 48.00

DESIGN LEVEL	TOPO GROUND	Cut/Fill	EX. DESIGN LEVEL	Cut/Fill
55.95	55.20	-0.75		
54.87	55.20	0.33		
54.87	55.19	0.32	54.06	-0.81
54.84	55.22	0.38	53.79	-1.05
54.78	55.24	0.47	53.82	-0.95
54.73	55.28	0.55	53.91	-0.82
54.67	55.30	0.63	53.91	-0.76
54.62	55.24	0.62	53.91	-0.71
54.58	55.16	0.58	53.91	-0.67
54.55	55.13	0.59	53.91	-0.63
54.53	55.11	0.58	53.91	-0.61
54.51	55.06	0.56	53.87	-0.63
54.50	55.01	0.51	53.81	-0.68
54.49	54.96	0.47	53.76	-0.73
54.46	54.93	0.47	53.70	-0.76
54.43	54.91	0.48	53.64	-0.78
54.40	54.93	0.53	53.61	-0.79
54.37	54.98	0.61	53.69	-0.67
54.34	55.05	0.71	53.86	-0.48
54.32	55.04	0.72	53.97	-0.35
54.30	54.92	0.62	53.90	-0.41
54.26	54.99	0.73	53.82	-0.44
54.25	55.06	0.81	53.74	-0.51
54.25	54.87	0.62	53.69	-0.56
54.25	54.97	0.71	53.64	-0.61
54.25	54.85	0.60	53.59	-0.66
54.25	54.84	0.59	53.54	-0.71
54.25	54.92	0.67		
54.25	55.05	0.80		
54.25	55.16	0.91		
54.25	55.24	1.00		
54.24	55.31	1.07		
54.25	55.21	0.96		
55.95	55.15	-0.80		

LONGITUDINAL SECTION D

SCALE: HORIZ=1:1000
VERT=1:500

REV	DESCRIPTION	DRN BY	CHK BY	APP BY	DATE
A	FIRST ISSUE	AL	SL	SL	07/11/25

MCKENZIE & CO.

CLIENT: AW HOLDINGS 2021 LIMITED

PROJECT: AUCKLAND SURF PARK
1350 DAIRY FLAT HIGHWAY
DAIRY FLAT, AUCKLAND

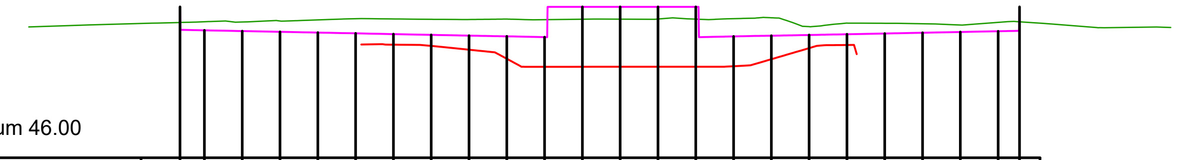
TITLE: EARTHWORKS
LAGOON OPTIONAL DESIGN
EARTHWORK SECTION
SHEET 1

PURPOSE OF ISSUE: FOR RESOURCE CONSENT

SCALE: AS SHOWN
DO NOT SCALE

DRAWING NO: 3325-2-2115 REV: A

C:\12DS\DATA\MCKFS01\3325-2 SURF PARK STAGE 2_5052\DESIGN\6.5. DRAWINGS\3325-2-2100.DWG



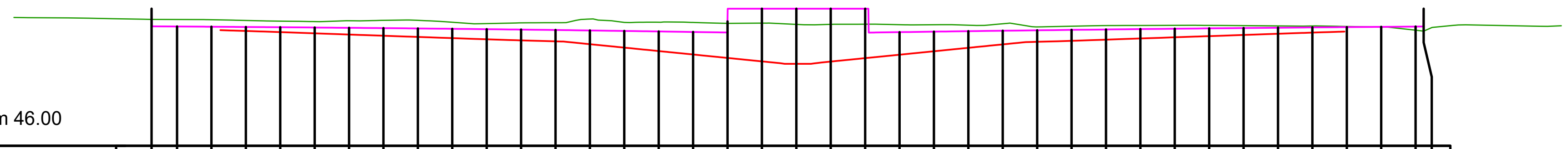
Datum 46.00

DESIGN LEVEL	55.95	54.40	54.35	54.30	54.25	54.20	54.15	54.10	54.05	54.00	53.95	55.95	55.95	55.95	55.95	54.00	54.05	54.09	54.14	54.19	54.24	54.29	54.34	55.95
TOPO GROUND	54.92	54.97	54.95	55.01	55.09	55.17	55.15	55.13	55.11	55.14	55.10	55.13	55.14	55.15	55.14	55.17	55.23	54.64	54.88	54.86	54.83	54.75	54.93	54.96
Cut/Fill	-1.03	0.56	0.60	0.71	0.84	0.96	1.00	1.02	1.06	1.14	1.15	-0.82	-0.81	-0.80	-0.81	1.17	1.18	0.55	0.73	0.67	0.59	0.46	0.59	-0.99
EX. DESIGN LEVEL							53.46	53.37	53.12	52.53	52.00	52.00	52.00	52.00	52.00	52.03	52.50	53.23	53.43					
Cut/Fill							-0.70	-0.73	-0.93	-1.48	-1.96	-3.95	-3.95	-3.95	-3.95	-1.97	-1.55	-0.87	-0.71					

LONGITUDINAL SECTION A

SCALE: HORIZ=1:1000

VERT=1:500



Datum 46.00

DESIGN LEVEL	55.95	54.66	54.64	54.62	54.60	54.58	54.56	54.54	54.52	54.49	54.46	54.43	54.40	54.37	54.34	54.30	54.26	55.02	55.95	55.95
TOPO GROUND	55.17	55.16	55.15	55.10	55.05	55.01	55.07	55.10	55.10	54.96	54.87	54.92	54.93	55.20	54.95	54.97	54.95	54.88	54.90	54.81
Cut/Fill	-0.78	0.51	0.51	0.47	0.44	0.43	0.51	0.57	0.59	0.47	0.41	0.49	0.53	0.83	0.62	0.67	0.69	-0.14	-1.05	-1.14
EX. DESIGN LEVEL				54.33	54.25	54.16	54.07	53.99	53.90	53.81	53.73	53.65	53.58	53.37	53.12	52.88	52.63	52.38	52.13	51.95
Cut/Fill				-0.29	-0.36	-0.42	-0.49	-0.55	-0.62	-0.68	-0.73	-0.78	-0.82	-1.00	-1.21	-1.42	-1.64	-2.65	-3.82	-4.00

DESIGN LEVEL	55.95	55.95	54.25	54.28	54.32	54.35	54.38	54.41	54.44	54.47	54.50	54.52	54.55	54.57	54.59	54.61	54.63	54.65	55.95
TOPO GROUND	54.81	54.82	54.79	54.78	54.75	54.86	54.63	54.68	54.72	54.73	54.74	54.74	54.72	54.70	54.70	54.66	54.62	54.39	54.32
Cut/Fill	-1.14	-1.13	0.54	0.50	0.44	0.51	0.24	0.26	0.28	0.26	0.24	0.21	0.17	0.13	0.10	0.05	-0.01	-0.26	-1.63
EX. DESIGN LEVEL	52.11	52.36	52.61	52.86	53.11	53.36	53.55	53.62	53.70	53.79	53.87	53.96	54.04	54.13	54.21				
Cut/Fill	-3.84	-3.59	-1.64	-1.42	-1.21	-0.99	-0.84	-0.80	-0.74	-0.68	-0.62	-0.56	-0.50	-0.44	-0.38				

LONGITUDINAL SECTION B

SCALE: HORIZ=1:1000

VERT=1:500

REV	DESCRIPTION	DRN BY	CHK BY	APP BY	DATE
A	FIRST ISSUE	AL	SL	SL	07/11/25

MCKENZIE & CO.

CLIENT:
AW HOLDINGS 2021
LIMITED

PROJECT:
AUCKLAND SURF PARK
1350 DAIRY FLAT HIGHWAY
DAIRY FLAT, AUCKLAND

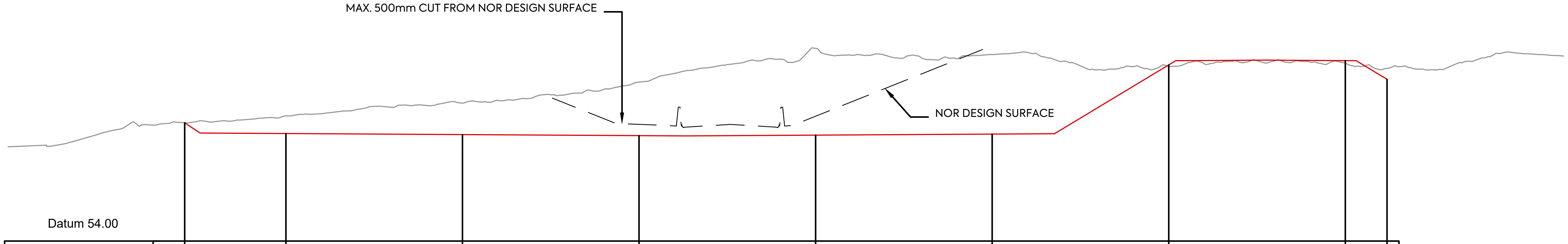
TITLE:
EARTHWORKS
LAGOON OPTIONAL DESIGN
EARTHWORK SECTION
SHEET 2

PURPOSE OF ISSUE:
FOR RESOURCE CONSENT

SCALE:
AS SHOWN
DO NOT SCALE

DRAWING NO: 3325-2-2116

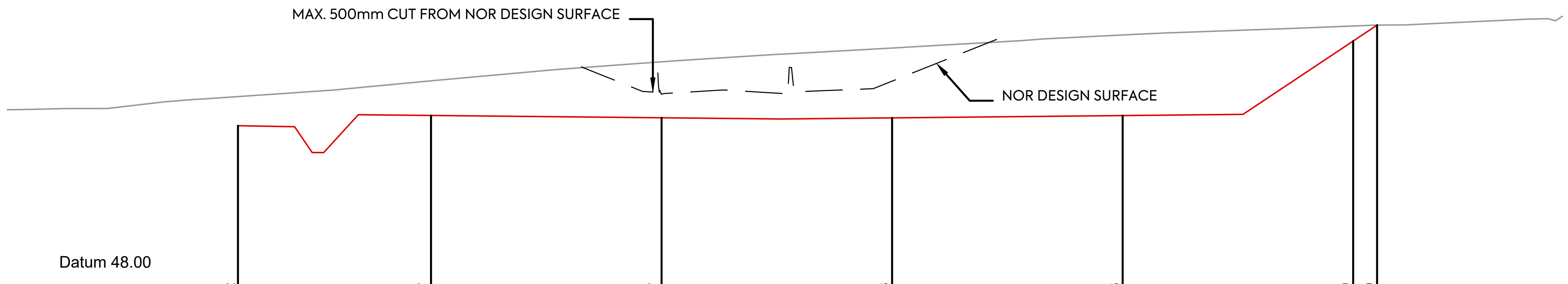
REV: A



Design Level	60.71	60.10	60.04	59.98	60.01	60.07	63.99	64.23	63.17
Ground Level	60.71	61.10	61.82	63.02	64.93	64.58	63.91	64.08	63.78
Cut/Fill	0.00	1.00	1.78	3.04	4.92	4.51	-0.08	-0.15	0.61
Chainages	-31.46	-20.00	0.00	20.00	40.00	60.00	80.00	100.00	104.72

NOR NOTRH EARTHWORK - TYPICAL SECTION

SCALE: HORIZ=1:500
VERT=1:250



Design Level	54.92	55.37	55.27	55.26	55.36	58.60	59.29
Ground Level	56.19	56.87	57.70	58.30	58.87	59.25	59.29
Cut/Fill	1.26	1.50	2.43	3.03	3.50	0.65	0.00
Chainages	-36.75	-20.00	0.00	20.00	40.00	60.00	62.08

NOR SOUTH EARTHWORK - TYPICAL SECTION

SCALE: HORIZ=1:500
VERT=1:250

REV	DESCRIPTION	DRN BY	CHK BY	APP BY	DATE
A	FIRST ISSUE	AL	SL	SL	07/11/25

MCKENZIE & CO.

CLIENT: AW HOLDINGS 2021 LIMITED

PROJECT: AUCKLAND SURF PARK
1350 DAIRY FLAT HIGHWAY
DAIRY FLAT, AUCKLAND

TITLE: EARTHWORKS
LAGOON OPTIONAL DESIGN
EARTHWORK SECTION
SHEET 3

PURPOSE OF ISSUE: FOR RESOURCE CONSENT

SCALE: AS SHOWN
DO NOT SCALE

DRAWING NO: 3325-2-2117
REV: A

Appendix E Reference Architectural Drawings

DISCLAIMER:
This pdf file and all information contained within are solely to be used for coordination purposes. All boundaries, levels, contours, existing services, trees and vegetation, district plan zones, roads, NOR boundaries, cadastral boundaries, stream centrelines, landscaping, and other site features have been traced from information provided by the client, civil engineer, or surveyor, or other consultants.
Data may be subject to change and SPA does not accept liability for any loss, damage or harm resulting directly or indirectly from the recipient or any other person placing, relying on, or making any use of, the preliminary data provided.

True North:

Original Scale / Print check: 0 10 30

Print in Colour

Notes:

DO NOT SCALE OFF DRAWINGS

All dimensions to be checked on site.

Rev	Description	Date
01 - WIP	Not in Transmittal Set	

Work in Progress

LEGEND:

- Superlot boundary
- Footpath
- Footpath (Flush)
- Cycleway
- Lawn
- Planting
- Concrete surface
- Hoggin surface
- Sand play safety surface
- 10m stream offset riparian planting
- Timber boardwalk
- Site boundary
- NOR boundary (indicative)
- Stream centre line
- 10m stream offset line
- Stormwater ponds/ raingardens
- Stream Park timber buildouts
- Raised vege planters
- Buildings
- Lounge bench seats
- Rocks
- Timber nature trail/ play-along-the-way
- Picnic tables
- BBQ's
- Picnic shelter
- Bench seats
- Trees

Note: Where the masterplan extends beyond the scope of SPA's work, it is shown **indicatively**. For the relevant legend and detailed information, please refer to the respective consultants' drawings.

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Prepared For:
AW Holdings Ltd

Project Title:
Auckland Surf Park Community
at
Dairy Flat Highway, Auckland
#

Not in Transmittal Set

Layout Name:
MASTERPLAN Illustrative Masterplan

Scale: 1:4000 Original Paper Size: A3

Job No.: **4663** Layout I.D: **RC_1.01** Revision: **WIP**

Version: SPA_27_01_02 WIP / CAD Ref: 4663_Auckland Surfpark Masterplan-Substantive Application Stage / Printed: 02/06/2026,2:50 PM

Appendix F Auckland Unitary Plan Groundwater Take/Divert Assessment

The proposed bulk earthworks have been assessed against the Auckland Unitary Plan rules E7.6.1.10 and E7.6.1.6. This has confirmed a take/diversion groundwater consent is required. The outcome of this is summarised in the Table below.

Groundwater Diversion - E7.6.1.10: Diversion of groundwater caused by any excavation, (including trench) or tunnel	
Permitted Activity Standard	Assessment
(1) All of the following activities are exempt from the Standards E7.6.1.10(2) – (6): (a) pipes cables or tunnels including associated structures which are drilled or thrust and are less than 1.2m in external diameter; (b) pipes including associated structures up to 1.5m in external diameter where a closed faced or earth pressure balanced machine is used; (c) piles up to 1.5m in external diameter are exempt from these standards; (d) diversions for no longer than 10 days; or, (e) diversions for network utilities and road network linear trenching activities that are progressively opened, closed and stabilised where the part of the trench that is open at any given time is no longer than 10 days.	Not exempt under any of these activities.
(2) Any excavation that extends below natural groundwater level, must not exceed: (a) 1ha in total area; and, (b) 6m depth below the natural ground level.	× Does Not Comply – excavations in Areas 1.1 and 1.2 exceed 1ha in area. Excavations in Area 1.1 locally exceed 6 m below the natural ground level.
(3) The natural groundwater level must not be reduced by more than 2m on the boundary of any adjoining site.	✓ Complies – groundwater is not expected to be reduced by more than 2 m beyond the site boundary. Some drawdown of transient groundwater in the Mahurangi Limestone rock mass is expected, however, drawdown levels beyond the boundary should be less than 2 m.
(4) Any structure, excluding sheet piling that remains in place for no more than 30 days, that physically impedes the flow of groundwater through the site must not: (a) impede the flow of groundwater over a length of more than 20m; and, (b) extend more than 2m below the natural groundwater level.	✓ Complies – no underground/basement structures proposed

Groundwater Diversion - E7.6.1.10: Diversion of groundwater caused by any excavation, (including trench) or tunnel	
Permitted Activity Standard	Assessment
(5) The distance to any existing building or structure (excluding timber fences and small structures on the boundary) on an adjoining site from the edge of any: (a) trench or open excavation that extends below natural groundwater level must be at least equal to the depth of the excavation; (b) tunnel or pipe with an external diameter of 0.2 - 1.5m that extends below natural groundwater level must be 2m or greater; or, (c) a tunnel or pipe with an external diameter of up to 0.2m that extends below natural groundwater level has no separation requirement	✓ Complies – existing buildings/structures are offset from the boundary a greater distance than the depth of adjacent excavations in the boundary.
(6) The distance from the edge of any excavation that extends below natural groundwater level, must not be less than: (a) 50m from the Wetland Management Areas Overlay; (b) 10m from a scheduled Historic Heritage Overlay; or, (c) 10m from a lawful groundwater take.	✓ Complies
Is groundwater diversion permitted under Standard E7.6.1.10?	✗ Does Not Comply - Excavation extends below the natural groundwater level so activity is not Permitted under Standard E7.6.1.10

Groundwater Take - E7.6.1.6: Dewatering or groundwater level control associated with a groundwater diversion permitted under Standard E7.6.1.10, all of the following must be met:	
Permitted Activity Standard	Assessment
(1) The water take must not be geothermal water;	✓ Complies – Take is not geothermal.
(2) The water take must not be for a period of more than 10 days where it occurs in peat soils, or 30 days in other types of soil or rock;	✗ Does Not Comply – proposed earthworks cuts are permanent
(3) The water take must only occur during construction.	✗ Does Not Comply – proposed earthworks cuts are permanent
Is groundwater take permitted under Standard E7.6.1.6?	✗ Does Not Comply

Is Resource Consent required for Groundwater Diversion and/or Take?	Groundwater Consent is Required

Appendix G PC120 Landslide Risk Assessment

A landslide risk assessment for the proposed ASPC development is presented below, in accordance with Appendix 24 of the Auckland Unitary Plan, PC120.

Stage 1: Desk study

Sub-stage		Results / Comments
1.1	Does landslide inventory show a pre-existing landslide at the location where the activity will occur.	No landslides have been mapped at the site location or in the vicinity, based on a review of: - The Auckland Council Geomaps landslide inventory; - The Natural Hazards Commission claims portal; - The Earth Sciences New Zealand Landslide Database and Geological Maps
1.3	Review Landslide Susceptibility Maps for landslide susceptibility at the site.	Large Scale Landslide Susceptibility Map: The site is mapped as very low susceptibility and the area within 150 m of the site is mapped as either very low or low susceptibility. Shallow Landslide Susceptibility Map: The site is predominantly mapped as very low susceptibility. There are several isolated areas mapped as having moderate to very high susceptibility to landlsides within the site and within 150 m of the site. The areas, shown on Figure G below, have been reviewed and are discussed below: <u>Zone (a):</u> has identified a shallow farm pond within the site. This pond will be infilled as part of the proposed works so poses no credible landslide hazard to the development. <u>Zone (b):</u> has identified a neighbouring farm pond offset 90 m from the subject site and situated at a lower elevation. Accordingly, this poses no credible landslide hazard to the development. <u>Zones (c – g):</u> have identified stream banks up to approximately 3 m in height that are offset a minimum of 75 m from the site and situated at a lower elevation than the site. Accordingly, this poses no credible landslide hazard to the development. <u>Zone (h):</u> has identified a localised over-steepened area above the existing main (east to west) stream channel in the site. The proposed earthworks in this location include regrading of the slope to between 1V:5H to 1V:6H so it will no longer pose a credible landslide hazard to the development. <u>Zones (i – j):</u> haves identified the fill batter of the existing horse arena platform. Proposed earthworks will cut down this area to form a wetland at the grades recommended in the Initia GIR. Accordingly, it will no longer pose a credible landslide hazard to the development.

Sub-stage		Results / Comments
		Zones (k – l): have identified sloping ground approximately 3 – 4 m in height on neighbouring properties facing the site. Accordingly, this does not pose a risk of instability to the site, however, should shallow instability occur, debris may deposit on the subject site. Accordingly, this has been reviewed in Stage 2 of this assessment. Zones (m – n): have identified steeper areas locally above the western boundary creek, within the site. As outlined in Section 7.5.1, localised shallow small-scale slumping has been identified above the creek along the western boundary so this maps a credible shallow landslide hazard. Accordingly, this has been reviewed in Stage 2 of the assessment below.
1.4	Review Landslide Susceptibility Maps for susceptibility to landslide runout.	Zones (k – l): Shallow instability in these areas could result in minor runout reaching the site, as outlined above. Accordingly, a risk assessment has been undertaken in Stage 2 below.
1.5	Determination of landslide susceptibility and requirement for Stage 2 assessment	Landslide susceptibility is considered to be very low, apart from potential for shallow instability in Zones (m – n) and potential for shallow instability runout from squares (k – l) on Figure G. Accordingly, a risk assessment has been completed below in relation to these areas.

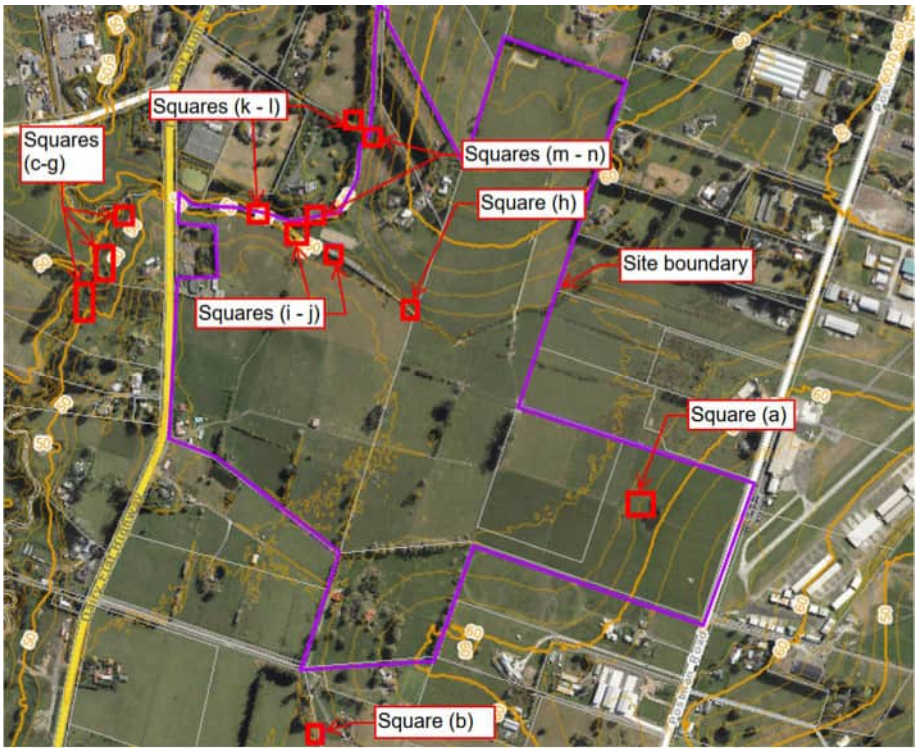

Figure G: Markup of “squares” mapped as Moderate, High or Very High Shallow Landslide Susceptibility (Source: AC Geomaps)

Stage 2 to 3: Risk Assessment

A risk assessment has been conducted in relation to:

- Landslip runout from shallow instability on neighbouring properties in zones k – l; and
- Shallow instability within the site in zones m – n above the existing western boundary creek.

In both areas, the geological conditions are generally expected to comprise shallow deposits of firm to stiff Tauranga Group alluvium (typically less than 2 m deep) underlain by a weathering profile of Mahurangi Limestone, as presented in the geological models for the site.

Likelihood Classification

For both zones k – l and m – n the only credible instability mechanisms is shallow slumping within low-strength alluvium, in locally over-steepened areas.

With reference to the table below, the likelihood of shallow slumping in both zones k – l and m – n is considered "likely" on the basis that a review of aerial imagery and a site walkover indicates shallow slumping has occurred in the past 100 years but has only locally occurred in two locations and the scarps are highly vegetated and do not show signs of recent movement occurred (demonstrating slumping does not occur frequently).

Likelihood Category	Description / Probability
Almost Certain	Occurs frequently or within 10 years (>10^{-1} per year)
<u>Likely</u>	<u>Expected within 100 years (10^{-2} per year)</u>
Possible	May occur within 1000 years (10^{-3} per year)
Unlikely	Unlikely but possible within 10,000 years (10^{-4} per year)
Rare	Extremely unlikely (<10^{-5} per year)
Barely Credible	Inconceivable or fanciful (<10^{-6} per year)

Consequence Classification

For zones k – l the consequence of land instability is runout/debris extending onto the project site. However, in both areas the runout would only extend into flood storage areas as the proposed development (buildings, roads, amenity areas) sit at a higher elevation than the slopes labelled as being subject to land instability. Accordingly, the consequence of instability is "insignificant" in accordance with the table below.

For zones m – n, the shallow slumping will have no effect on the stability of the proposed building platforms, access ways and amenity areas, provided these have been detailed in accordance with the recommendations in this Geotechnical Assessment Report. This is because the proposed fill embankments and retaining walls (located behind the mapped stability area and existing shallow slumping) can be appropriately designed to remain stable with allowance for slumping in the alluvium downslope (by either keying in fill embankments or gravity retention on the shallow rock, or by installing palisade piles/piled retaining walls socketed into the shallow rock). This is substantiated by the geological models and quantitative slope stability analyses for Zone 4B, presented in Section 7.5.3. Accordingly, the consequence of shallow slumping above the creek is "insignificant".

Consequence Category	Description
Catastrophic	Multiple fatalities, total building loss, major infrastructure failure
Major	Single fatality or serious injury, major structural damage
Medium	Injury requiring medical treatment, moderate structural damage
Minor	Minor injury or property damage
<u>Insignificant</u>	<u>Negligible damage or no injury</u>

Accordingly, based on the risk assessment and the specific review and analysis conducted for the site, the landslide hazard is assessed as low (acceptable) as shown on the risk matrix below. Accordingly, no further controls are required beyond the geotechnical recommendations already presented in this Geotechnical Assessment Report.

Consequence/ Likelihood	Insignificant	Minor	Medium	Major	Catastrophic
Almost Certain	Medium (tolerable)	High (significant)	High (significant)	High (significant)	High (significant)
Likely	Low (Acceptable)	Medium (tolerable)	High (significant)	High (significant)	High (significant)
Possible	Low (Acceptable)	Low (Acceptable)	Medium (tolerable)	High (significant)	High (significant)
Unlikely	Low (Acceptable)	Low (Acceptable)	Low (Acceptable)	Medium (tolerable)	High (significant)
Rare	Low (Acceptable)	Low (Acceptable)	Low (Acceptable)	Low (Acceptable)	Medium (tolerable)
Barely credible	Low (Acceptable)	Low (Acceptable)	Low (Acceptable)	Low (Acceptable)	Low (Acceptable)

Appendix H Statement of Qualifications and Experience

The geotechnical assessment to support the substantive application of Stage 2 of the Auckland Surf Park Community has been led and reviewed by Nick Speight, a Senior Geotechnical Engineer and Director of Initia. Nick has a Bachelor of Engineering (Hons) and is a Chartered Geotechnical Engineer with Engineering New Zealand. He has over 25 years of experience in geotechnical design and assessment in Auckland and has held geotechnical design lead roles on numerous large scale residential and industrial subdivision projects including the Ohinewai Sleepyhead Estate and The Landing industrial subdivision at Auckland International Airport. Nick also has extensive experience in geotechnical engineering for residential, commercial industrial buildings Nick was also the geotechnical lead for the Auckland Surf Park Community Stage 1 substantive application.

Nick has been supported by Kieran Bursell, a Geotechnical Engineer and Associate of Initia. Kieran has a Bachelor of Engineering (Hons) and has over 6 years of experience in geotechnical design and assessment in Auckland. Kieran's experience has been gained on subdivisions and buildings in areas with challenging ground conditions. He has recently held Project Geotechnical Engineer roles for the Auckland Radiology Group healthcare facility, Kainga Ora Concord Place residential subdivision, Te tau Waka residential subdivision and Cable Price industrial facility. Kieran was also the project Geotechnical Engineer for the Auckland Surf Park Community Stage 1 substantive application so is well accustomed to the project's geotechnical considerations.