



APPENDIX I

Geotechnical Report

03 December 2024

15 Ryburn Road Plan Change

Ohaupo

GEOTECHNICAL INVESTIGATION REPORT

Honey Comb Developments LP

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1	03 December 2024	Final report issued



EXECUTIVE SUMMARY

This report presents the results of geotechnical investigations and a geohazards assessment to support a Private Plan Change (PPC) application to Waipa District Council and provides the basis for the Statement of Professional Opinion in Section 8.

The subject site is located on the corner (southwest) of Ryburn and Ohaupo Roads, where it is proposed to develop a large-lot residential subdivision, with associated roading infrastructure. The site is underlain by Late Pleistocene aged Walton Subgroup clays on the upper terraces and river deposits in the base of the valley comprising Recent Alluvium silts and clays with interbedded peat.

The high point of the site is at ~RL71.0m in the north-eastern corner of the property on an upper terrace with rolling hills running along the north, east and southern boundaries. In the base of the valley in the southwest corner of the property, the topography is nearly level/ very gently sloping, with a low point of ~RL39.0m. The hills of the upper terrace comprises subtle erosional gully features created by overland flow paths extending to the west.

We consider that the site is suitable for the proposed level of development subject to our geohazard assessment and geotechnical recommendations summarised as follows:

- Slope stability analyses for the hills is recommended at detailed design stage to demonstrate compliance with the project design criteria.
- Preliminary estimates of static settlements for widespread floor loads of 10 to 42kPa are calculated to range from 5mm to 50mm (primary, over 6 months) and an additional 0 to 10mm (secondary, over design life of 50 years) for a floor size of 50m². The majority of the settlement appears to be occurring within 1m of the ground surface.
- The ULS liquefaction risk for this site is minor (<45mm predicted vertical settlement) based on the liquefaction analysis results, without an ageing factor applied. Typically, shallow foundation types are considered feasible, subject to further assessment.
- A preliminary geotechnical ultimate bearing pressure of 300 kPa should be available for foundations in most areas. However reduced bearing pressures may be required in the low lying areas where slightly lower shear strength/ density soils are exposed near finished levels. Improvement of near surface soil bearing capacity could be achieved with conventional compaction equipment.

CONTENTS

1.0	INTRODUCTION	6
2.0	SITE DESCRIPTION	6
2.1	Site Location.....	6
2.2	Landform.....	6
3.0	PROPOSED DEVELOPMENT	7
4.0	INVESTIGATION SCOPE	7
4.1	Field Investigation	7
5.0	GROUND MODEL	8
5.1	Published Geology.....	8
5.2	Stratigraphic Units	9
5.3	Groundwater	9
6.0	GEOHAZARDS ASSESSMENT	10
6.1	Context.....	10
6.2	Seismicity	10
6.3	Liquefaction.....	10
6.3.1	Methodology	10
6.3.2	Results	11
6.4	Cyclic Softening	11
6.5	Lateral Spread	11
6.6	Slope Stability.....	12
6.7	Erosion	12
6.8	Load Induced Settlement.....	12
6.9	Expansive Soils	13
7.0	RECOMMENDATIONS	13
7.1	Seismic Site Subsoil Category.....	13
7.2	Liquefaction / Lateral Spread Mitigation	13
7.3	Static Settlement.....	13
7.4	Earthworks	14
8.0	STATEMENT OF PROFESSIONAL OPINION	14
9.0	FURTHER WORK	15
10.0	CLOSURE	15

Appendices

APPENDIX A	Drawings
APPENDIX B	Proposed Development Plans
APPENDIX C	Cone Penetrometer Test Results
APPENDIX D	Hand Auger Logs
APPENDIX E	Liquefaction Results
APPENDIX F	Settlement Results
APPENDIX G	Historical Photographs

Figures

Figure 1: Site Location Plan. Reference: OpenStreetMaps	6
Figure 2: Regional Geology (QMAP)	8
Figure 3: Relationship between earthquake magnitude and mean number of uniform stress cycles	11
Figure 4: Relationship between cyclic strength ratio and number of uniform stress cycles	11

Tables

Table 1: Summary of Strata Encountered	9
Table 2: Groundwater Measurements	9
Table 3: Design Peak Ground Acceleration (PGA) for Various Limit States.....	10
Table 4: Liquefaction Analyses Results.....	11
Table A.1: Drawing Numbers	

1.0 INTRODUCTION

CMW Geosciences (CMW) was engaged by Honey Comb Developments to carry out a geotechnical investigation of a site located at 15 Ryburn Road, Ohaupo, where it is proposed to develop a 40-lot residential subdivision.

The scope of work and associated terms and conditions of our engagement were detailed in our services proposal letter referenced HAM2024-0126AA Rev 0, dated 31 October 2024.

This report is to support a Private Plan Change (PPC) application to Waipa District Council.

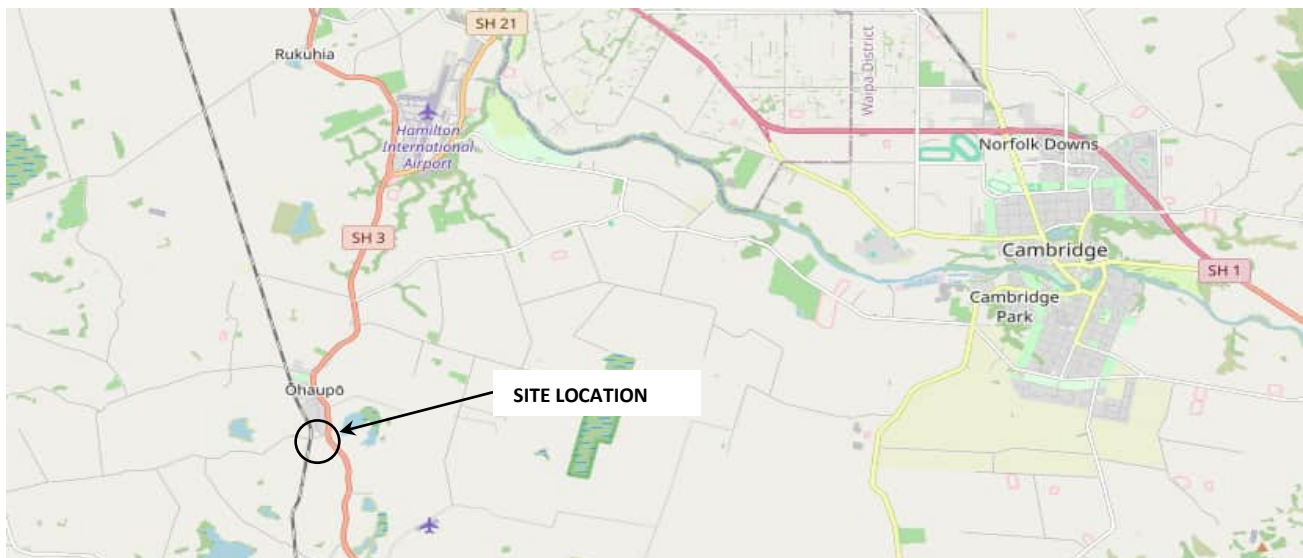
2.0 SITE DESCRIPTION

2.1 Site Location

The site comprises an area of approximately 20.7Ha and is located along the southern boundary of Ohaupo village on the south-west corner of Ryburn and Ohaupo Roads as shown on **Figure 1** below.

The property legal description is Lot 2 DP 606757.

Figure 1: Site Location Plan. Reference: OpenStreetMaps



2.2 Landform

The current general landform, together with associated features located within and adjacent to the site is presented on the attached Site Plan as **Drawing 01**.

The site is situated within the head of a valley grading moderately from approximately RL71m on the north-eastern boundary to RL39m at the south-western end of the site. The site can be divided into two distinct landforms, the upper terrace comprising a semi-circle of low rolling hills (5 to 45 degree slopes) forming the head of the valley along the north, east and southern boundaries, and a lower alluvial plain forming a near level surface at the valley floor grading which grades gently (<5 degrees) to the south-west. A previous slip face is present above the northern pond (45 degree slopes) as shown on **Drawing 02**.

Localised areas of shallow soil creep are evident on steeper slopes.

There are four man-made ponds in the approximate centre of the property. Uncontrolled Fill from the pond excavation(s) has been stockpiled up to 3m high near the eastern-most pond. A series of open drains convey surface water and pond discharges to an open drain running along the western boundary that drains towards the south.

The site is bound to the north by Ryburn Road, to the east by Ohaupo Road (State Highway 3) and to the west by the North Island Main Trunk (NIMT) Railway line. Three existing residential lots are located in the north-eastern corner of the site and four farm buildings are spread across the site.

Historical aerial photographs¹ show the northernmost ponds were formed in 2014 then extended in 2020, and the easternmost ponds were formed in 2017 and extended in 2018. Historical photographs are presented in **Appendix G**.

3.0 PROPOSED DEVELOPMENT

The current development proposal, as shown by the concept prepared by Cogswell Surveyors in **Appendix B**, is to create 40 residential lots varying in size from around 2526m² to 9300m², with an access road from Ryburn Road and several accessway lots. A reserve is proposed in the centre of the site (Lot 200), and Lot 201 is proposed as a stormwater overland flow path/swale.

At the time of undertaking this investigation and of writing this report, the project was in the early stages of planning and it was anticipated that the geotechnical investigation would provide details of preliminary feasibility options for the site.

No architectural or engineering design drawings have been supplied to date. We have prepared this report on the basis that a future development will comprise cuts of the hills on the northern and eastern sides of the site, and fills in the southwestern side of the site to support residential land development with shallow strip and pad foundations and widespread floor loads of up to 10kPa.

4.0 INVESTIGATION SCOPE

4.1 Field Investigation

Following a dial before you dig search, and onsite service location, the field investigation was carried out on 6 November 2024. All fieldwork was carried out under the direction of CMW Geosciences in general accordance with the NZGS specifications² and logged in accordance with NZGS guidance³. The scope of fieldwork completed was as follows:

- Undertook a walkover survey of the site to assess the general landform and site conditions;
- Four Cone Penetrometer Tests, denoted CPT01 to CPT04, were pushed to depths of between 9.1 and 14.4m (where they all met refusal). The CPTs are to develop the ground model through the likely earthworks excavation depths and through the underlying zone of influence of fill embankments and future building foundations and to provide parameters for retaining wall and foundation design. Results of the CPT's, presented as traces of tip resistance (qc), friction resistance (fs) and friction ratio are presented in *Error! Reference source not found.*;

¹ Retrolens website, Sourced from <http://retrolens.nz> and licensed by LINZ CC-BY 3.0 and <https://earth.google.com/>

² NZ Geotechnical Society (2017) NZ Ground Investigation Specification, Volume 1 – Master Specification

³ NZ Geotechnical Society (2005), Field Description of Soil and Rock, Guideline for the field classification and description of soil and rock for engineering purposes.

- Twelve hand auger boreholes, denoted HA01 to HA12, were drilled using a 50mm diameter auger to target depths of up to 4.0m below existing ground levels to visually observe the near surface soil profile and to facilitate Dynamic cone (Scala) penetrometer (DCP) / vane shear strength testing. Engineering logs of the hand auger boreholes, together with peak and remoulded vane shear strengths are presented in *Error! Reference source not found.*;
- The approximate locations of the respective investigation sites referred to above are shown on the Site Plan **Drawing 01**. Test locations were measured using hand-held GPS to an accuracy of $\pm 4\text{m}$.

5.0 GROUND MODEL

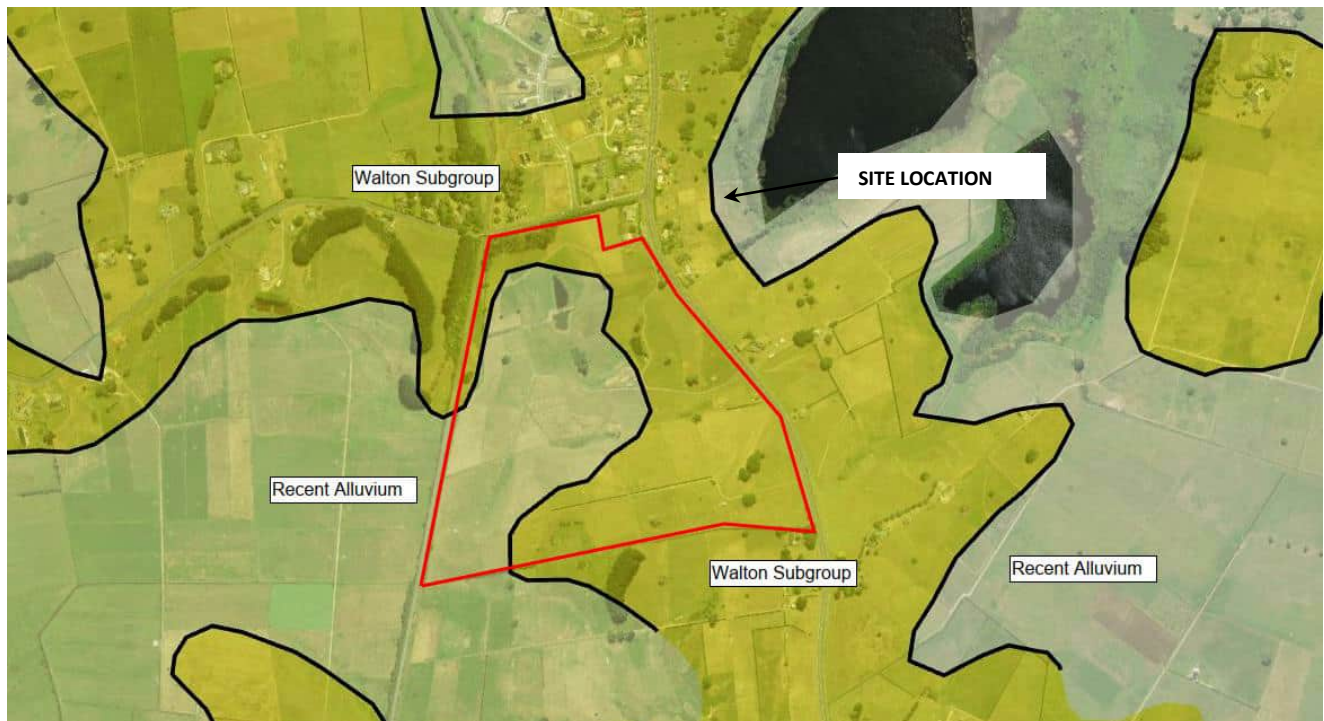
5.1 Published Geology

The published geological map⁴ for the area indicates the site is underlain by Late Pleistocene aged lake sediments of the Piako Subgroup/Recent Alluvium comprising locally derived lacustrine mud, silt, gravel and peat as illustrated in **Figure 2** below.

The low hills to the north, west and south of the site are shown to be underlain by older volcanic silts and clays of the Walton Subgroup derived from insitu and fluvially reworked and weathered non-welded distal ignimbrites that are mantled with weathered volcanic ash.

The geologically older Walton Subgroup represents an older (1.2 million year old) landform that is present below the younger Recent Alluvium deposits.

Figure 2: Regional Geology (QMAP)



Based on the known history of the site, some superficial depths of fill are anticipated as a result of farming activities.

⁴ Waikato 1:250,000 Geological Map, No 4, Institute of Geological and Nuclear Sciences Limited, 2005.

5.2 Stratigraphic Units

The ground conditions encountered and inferred from the investigation were considered to be generally consistent with the published geology for the area and can be generalised according to the following subsurface sequences.

The distribution of the various units encountered is presented in the appended Geological Plan and the Geological Sections on **Appendix A** and presented below in **Table 1**.

Table 1: Summary of Strata Encountered

Unit	Depth to Top of Unit (m)		Thickness (m)	
	Min	Max	Min	Max
Topsoil	0		0.05	0.4
Uncontrolled Fill/Organic Silt* - Firm to stiff silt	0.05		0.25	0.65
Recent Alluvium** - Firm to very stiff interbedded silt/sand	0.2	0.7	2.6	3.4
Walton Subgroup – Very stiff to hard sandy or silty clay	0.2	0.4	1.9	5.0
Walton Subgroup – Dense to Very Dense Sand	2.4	5.0	4.0	>12
Notes: * Strata encountered in HA02 and HA09 only ** Strata encountered in HA02, 05, 08 and HA09 only. Nd organic Silt mixed with fill in HA02. Where holes collapsed due to groundwater, thickness inferred from Scala results.				

5.3 Groundwater

During the investigation, which was completed in late spring conditions (November 2024), groundwater was encountered within the CPTs and boreholes at the depths provided in **Table 2**.

Table 2: Groundwater Measurements

Test Location	Measured Depth (mbgl)	Test Location	Measured Depth (mbgl)	Test Location	Measured Depth (mbgl)
CPT01	0.5 (2.5)	HA03	1.5	HA09	1.3
CPT02	0.6 (4.5)	HA04	Not encountered	HA10	Not encountered
CPT03	0.3 (1.5)	HA05	0.9	HA11	Not encountered
CPT04	Not encountered (2.0)	HA06	2.5	HA12	Not encountered
HA01	0.9	HA07	Not encountered		
HA02	2.5	HA08	1.3		
Notes: Corrected values shown in brackets indicate depth where pore pressure began to increase in CPT results. mbgl = metres below ground level					

With regards to groundwater encountered within the CPT test holes we consider the measured results to be unrealistically high, likely due to the water level within the CPT's being measured immediately following testing, not giving the groundwater enough time to stabilise.

A more realistic measure of groundwater levels is where the depth at which the CPT results indicate an increase in pore pressure. Additionally, the levels measured in the nearby hand augers were consistently deeper than those measured within the CPT's.

Therefore, for the purposes of our analysis we have utilised the corrected groundwater levels indicated in Table 2 above.

6.0 GEOHAZARDS ASSESSMENT

6.1 Seismicity

A seismic assessment has been carried out in general accordance with NZGS guidance⁶ to determine the peak horizontal ground acceleration or PGA (a_{max}) based on a 50-year design life in accordance with the NZ Building Code⁷ and Importance Level (IL) 2 structures for the proposed development. PGA values for the Serviceability Limit State (SLS), and Ultimate Limit State (ULS) are presented in **Table 3**.

Table 3: Design Peak Ground Acceleration (PGA) for Various Limit States

Limit State	AEP	R	PGA (g)	Magnitude _{eff}
SLS	1/25	0.25	0.06	5.9
ULS	1/500	1.0	0.26	5.9
Note: SLS = serviceability limit state; ULS = ultimate limit state; AEP = annual exceedance probability				

6.2 Liquefaction

6.2.1 Methodology

Liquefaction occurs in loose saturated cohesionless soils that are subject to cyclic shear loading during an earthquake. This process leads to pore pressure build-up, soil grains moving into suspension and temporary loss of strength causing vertical and lateral ground deformation.

In accordance with MBIE/NZGS guidance⁹ the liquefaction susceptibility of the soils at this site was assessed with respect to geological age and compositional (soil fabric and density) criteria, based on the following assumptions:

- Only saturated soils below the groundwater were modelled as being susceptible to liquefaction. Groundwater was encountered between 0.6 and 1.0m below existing ground level in the vicinity of the CPT test locations.
- In accordance with MBIE/NZGS guidance¹ and in the absence of site-specific shear wave velocity measurements, no aging / strength gain factor has been applied to the Pleistocene aged sands.
- Soils are also classified with respect to their grain size and plasticity to assess liquefaction susceptibility. For this project, a cut-off threshold soil behaviour type index value (I_c) of 2.6 was used to distinguish between liquefiable ($I_c < 2.6$) and non-liquefiable ($I_c > 2.6$) soils.

⁶ NZ Geotechnical Society publication "Earthquake geotechnical engineering practice, Module 1: Overview of the standards", (March 2016)

⁷ Ministry of Business, Innovation and Employment (1992) NZ Building Code Handbook, Third Edition, Amendment 13 (effective from 14 February 2014)

⁹ Earthquake Geotechnical Engineering Practice, Module 3: Identification, assessment and mitigation of liquefaction hazards", (November 2021)

- Specific liquefaction analyses were undertaken for an IL2 structure, using the software package CLiq using the Boulanger and Idriss (2014) method. The cyclic stress ratio (CSR), being a function of the earthquake magnitude for the design return period event, was compared to the cyclic resistance ratio (CRR), being a function of the CPT cone resistance (q_c) and friction ratio (Fr).
- Free-field liquefaction induced settlements were determined in accordance with Zhang et al. (2002). With respect to liquefaction response, consideration was given to a 10m cut-off depth to estimate index settlements as per MBIE¹⁰ guidance (foundation technical categories). These were compared to liquefaction settlement estimates over the full depth range of the CPT's with a depth weighting factor (e_v) applied ranging from 1 at the ground surface to 0 at 20m depth.

6.2.2 Results

Results are presented in **Appendix E** and can be summarised as in **Table 4**:

Table 4: Liquefaction Analyses Results

CPT No.	Total ULS Settlement (mm)	Index ULS Settlement (mm)	Depth to Liquefied Layer (m)
CPT01	40	45	2.5
CPT02	<5	<5	4.5
CPT03	25	30	1.5
CPT04	15	20	2.0

Note: All settlements and depths based on existing ground profile.
Index settlements are calculated based on the upper 10m of the soil profile using no depth weighting factor.
Total ULS settlements are based on the full depth of the CPT trace with a depth weighting factor applied.
Index settlements are for assessment of the site against the MBIE site Technical Category guidelines and are not comparable to the total ULS settlements.
liquefaction is not triggered in the SLS case.

Recommendations to mitigate effects of liquefaction settlements on the proposed development are provided below in Section 7.

6.3 Cyclic Softening

Our results indicated the fine-grained alluvium was not susceptible to strength loss due to cyclic softening, during a ULS seismic event, and need not be considered further.

6.4 Lateral Spread

Following the onset of liquefaction, the liquefied soils behave as a very weak undrained material, which can give rise to lateral spreading where a free face is present within the vicinity of the site.

Literature suggests that lateral spreading may occur if laterally persistent liquefied layers are present within a depth of 2 times the free face height. A free face may be a waterway/pond edge. This risk should be assessed once earthworks plans have been formalised and proposed building dimensions are known. Preliminary recommendations are provided in Section 7.2 below.

¹⁰Repairing and Rebuilding House affected by the Canterbury Earthquakes", (December 2012)

6.5 Slope Stability

The existing landform shows some signs of previous slope instability for slope above the northern pond, shown on **Drawing 03**. However there is no evidence of recent instability and our testing in the vicinity of the slope indicates the slope soils are comprised of very stiff clay. On this basis our preliminary assessment is that this slope as low to moderate risk of instability.

There is evidence of shallow soil creep on some of the steeper slopes around the site. However each lot either has sufficient building platform outside of the steep ground/soil creep zones, or will require earthworks to create a level building platform which will resolve the soil creep risk.

Detailed slope stability analyses are not warranted at this early stage of the proposed development. An assessment should be completed as a part of the detailed earthworks design. This would normally be completed at the resource consent stage of the development once proposed earthworks and landform.

If during later investigations it is found that this slope requires stability improvement, typical stability solutions would be appropriate such as, cutting the slope back to a more stable angle, constructing a series of subsoil drains to direct water away from the slope and/or the construction of retaining walls.

6.6 Erosion

The predominantly cohesive (clayey) nature of the natural soils, which could be used as engineered fill, means that there is a low risk of erosion if appropriate controls are not in place.

However, slopes should be designed to reduce the effects of erosion of susceptible materials, either by construction via appropriate gradients or forming of slopes with materials less susceptible to erosion.

6.7 Load Induced Settlement

Settlements due to building loads and possible fill have been estimated using the CPeT-IT software, which calculates the change in vertical stress based on a Boussinesq distribution, with the 1-D constrained modulus estimated from the CPT tip resistance (q_c). Although no earthworks plans were available at the time of writing, scenarios with 1m and 2m of fill have been assessed for the CPTs in the lower alluvial plain.

Preliminary results of the assessment for a range of building loads and indicative fill are presented in **Appendix F** and summarised in **Table 5** below.

Table 5: Load Induced Settlement Summary

CPT	Load Case	Load (kPa)	Settlement (mm)
CPT01	Residential Building	10	15
	Residential Building + 1m fill	26	30
	Residential Building + 2m fill	42	45
CPT02	Residential Building	10	10
CPT03	Residential Building	10	15
	Residential Building + 1m fill	26	35
	Residential Building + 2m fill	42	60
CPT04	Residential Building	10	5

Results show that settlements may exceed building code limits. Recommendations are given in Section 7.3.

6.8 Expansive Soils

There is some evidence of expansive soils in the Walton Subgroup soils in the upper terrace. From our experience with these soils this may be up to Class M. Some laboratory testing should be carried out at the resource consent stage to confirm.

7.0 RECOMMENDATIONS

7.1 Seismic Site Subsoil Category

Based on those ground conditions and specific geophysical data gathered by the University of Waikato and Auckland University, the typical site period for the site is expected to be about 3 seconds where the seismic site subsoil category is assessed as being Class D (deep soil site) in accordance with NZS1170.5

It is anticipated that future buildings will be considered Importance Level IL2 structures with respect to NZS1170.5.

7.2 Liquefaction / Lateral Spread Mitigation

Possible options for mitigating liquefaction induced settlements are as follows:

- Designing structures to accommodate the predicted settlements and in particular differential settlements. Estimated total settlements up 40mm excluding building induced shear settlement and settlement due to surface expression of liquefaction. It is recommended to consider differential settlement of 2/3 of the total settlement across the width of future buildings.
- Ground improvement to reduce total and differential settlements to acceptable levels. Possible ground improvement options include a fine-grained fill cap, reinforced gravel rafts.
- Piled foundations may also be considered for foundations. Design will need to take into account the depth of liquefiable layers and the effects of negative skin friction causing down-drag on piles.

The liquefaction assessments in this report may be further refined at the Resource Consent stage based on additional site specific data. In particular, confirmation of groundwater levels (both perched and regional water table), and laboratory testing to determine the fines content and plasticity of the alluvial sand/silt materials. This may reduce the predicted settlement.

7.3 Static Settlement

The estimated load induced settlements presented in Section 6.8 should be reviewed at Resource Consent stage and if necessary updated to reflect actual bulk earthfill and foundation loads and bearing pressures once that information is available.

Initial analysis indicates that load induced settlement within the low-strength recent alluvium may be excessive. Possible options for mitigating static settlements are as follows:

- Undercut and replace the recent alluvium with engineer controlled compacted fill;
- Construct a reinforced gravel raft foundations on top of the recent alluvium;
- Pile through the recent alluvium into the dense sands support to load bearing points within proposed buildings.

7.4 Earthworks and Land Drainage

Across the low-lying valley floor areas, some uncontrolled fill and organic silts are present. These soils are unsuitable for support building foundations will need to be cut to waste, or alternatively these materials could be left in place if appropriately preloaded or piled foundations are utilised. Appropriate controls are to be confirmed during detailed earthworks design.

In the upper terraces Walton Subgroup silts are sensitive to remoulding and moisture ingress. These soils can rapidly lose strength if overworked, over-trafficked, or allowed to get wet. Mitigation measures can include avoiding directly trafficking these soils, limiting the area exposed at any one time, shaping and compacting the cut surface to allow water to runoff rather than pond/infiltrate, and using appropriate earthworks methodologies – such as avoiding vibratory compaction until adequate cover has been placed. Disturbed soils may regain strength if left to settle for a period of days in fine weather, or they may need to be undercut.

Given the consistency of the soil units that will be encountered within the building platform excavations, it is expected that excavation of these materials will be readily achieved with normal earthworks plant, such as scrapers and bulldozers with scoops.

Subsoil drains are likely to be required across the site, due to anticipated shallow perched groundwater. Where the open swale drains are present within lots and roads/accessways these should be mucked out of any vegetation and unsuitable soils then have a subsoil drain installed comprising perforated draincoil laid at even grade surrounded by drainage aggregate and wrapped with geotextile filter fabric. Subsoil drain locations and details can be determined following further site investigations as a part of detailed earthworks design.

Further specific earthworks recommendations and specifications should be confirmed at the detailed design stage.

8.0 STATEMENT OF PROFESSIONAL OPINION

Based on the results of previous geotechnical investigations at the site and subject to the preliminary recommendations stated above, we consider that the site is suitable for the proposed level of development. The proposed Private Plan Change to enable large lot residential subdivision, with associated roading infrastructure and stormwater soakage/detention basin, is considered to be appropriate from a geotechnical perspective. This statement is dependent on further work as detailed in Section 9 below.

9.0 FURTHER WORK

Further geotechnical field investigation and design will be required at Resource Consent stage to suitably mitigate the geotechnical risks identified in **Section 7** above.

Our recommendations for further geotechnical work are as follows:

- Further slope stability analyses should be undertaken at the time of detailed design once design levels are unknown, including assessment of soil types, stormwater pond/swale stability, variation of water levels, potential for scour/erosion and any surcharge loading. A building restriction setback from the top of slope should be confirmed at this time.
- Section 106 of the Resource Management Act¹⁸ (RMA) requires an assessment of the risk from natural hazards to be carried out when considering the granting of a subdivision consent. S106 RMA specifically states that the assessment must consider the combined effect of the natural hazard likelihood and material damage to land or structures (consequence). This is a requirement at Resource Consent application stage.
- Subsoil drainage design including drain layout and construction detailing.
- Further settlement assessment once fill loads known.
- Earthworks material suitability assessment including sampling of proposed fill materials, laboratory testing and preparation of an project specific earthworks compaction control specification.
- Presentation of the above work in a Geotechnical Investigation Report suitable to support a Resource Consent application and / or detailed design as appropriate.

Proposed buildings should be subject to lot specific geotechnical site investigation, analyses and reporting at the time of Building Consent application.

10.0 CLOSURE

This report has been prepared for use by Honey Comb Developments LP in relation to the Ohaupo project in accordance with the scope, proposed uses and limitations described in the report. Should you have further questions relating to the use of your report please do not hesitate to contact us.

Where a party other than Honey Comb Developments LP seeks to rely upon or otherwise use this report, the consent of CMW should be sought prior to any such use. CMW can then advise whether the report and its contents are suitable for the intended use by the other party.

¹⁸ Resource Management Act (1991), as at 29 October 2019

USING YOUR CMW GEOTECHNICAL REPORT

Geotechnical reporting relies on interpretation of facts and collected information using experience, professional judgement, and opinion. As such it generally has a level of uncertainty attached to it, which is often far less exact than other engineering design disciplines. The notes below provide general advice on what can be reasonably expected from your report and the inherent limitations of a geotechnical report.

Preparation of your report

Your geotechnical report has been written for your use on your project. The contents of your report may not meet the needs of others who may have different objectives or requirements. The report has been prepared using generally accepted Geotechnical Engineering and Engineering Geology practices and procedures. The opinions and conclusions reached in your report are made in accordance with these accepted principles. Specific items of geotechnical or geological importance are highlighted in the report.

In producing your report, we have relied on the information which is referenced or summarised in the report. If further information becomes available or the nature of your project changes, then the findings in this report may no longer be appropriate. In such cases the report must be reviewed, and any necessary changes must be made by us.

Your geotechnical report is based on your project's requirements

Your geotechnical report has been developed based on your specific project requirements and only applies to the site in this report. Project requirements could include the type of works being undertaken; project locality, size and configuration; the location of any structures on or around the site; the presence of underground utilities; proposed design methodology; the duration or design life of the works; and construction method and/or sequencing.

The information or advice in your geotechnical report should not be applied to any other project given the intrinsic differences between different projects and site locations. Similarly geotechnical information, data and conclusions from other sites and projects may not be relevant or appropriate for your project.

Interpretation of geotechnical data

Site investigations identify subsurface conditions at discrete locations. Additional geotechnical information (e.g. literature and external data source review, laboratory testing etc) are interpreted by Geologists or Engineers to provide an opinion about a site specific ground models, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist due to the variability of geological environments. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. Interpretation of factual data can be influenced by design and/or construction methods. Where these methods change review of the interpretation in the report may be required.

Subsurface conditions can change

Subsurface conditions are created by natural processes and then can be altered anthropically or over time. For example, groundwater levels can vary with time or activities adjacent to your site, fill may be placed on a site, or the consistency of near surface conditions might be susceptible to seasonal changes. The report is based on conditions which existed at the time of investigation. It is important to confirm whether conditions may have changed, particularly when large periods of time have elapsed since the investigations were performed.

Interpretation and use by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a geotechnical report. To help avoid misinterpretations, it is important to retain the assistance of CMW to work with other project design professionals who are affected by the contents of your report. CMW staff can explain the report implications to design professionals and then review design plans and specifications to see that they have correctly incorporated the findings of this report.

Your report's recommendations require confirmation during construction

Your report is based on site conditions as revealed through selective point sampling. Engineering judgement is then applied to assess how indicative of actual conditions throughout an area the point sampling might be. Any assumptions made cannot be substantiated until construction is complete. For this reason, you should retain geotechnical services throughout the construction stage, to identify variances from previous assumption, conduct additional tests if required and recommend solutions to problems encountered on site. A Geotechnical Engineer, who is fully familiar with the site and the background information, can assess whether the report's recommendations remain valid and whether changes should be considered as the project develops. An unfamiliar party using this report increases the risk that the report will be misinterpreted.

Environmental matters are not covered

Unless specifically discussed in your report environmental matters are not covered by a CMW Geotechnical Report. Environmental matters might include the level of contaminants present of the site covered by this report, potential uses or treatment of contaminated materials or the disposal of contaminated materials. These matters can be complex and are often governed by specific legislation.

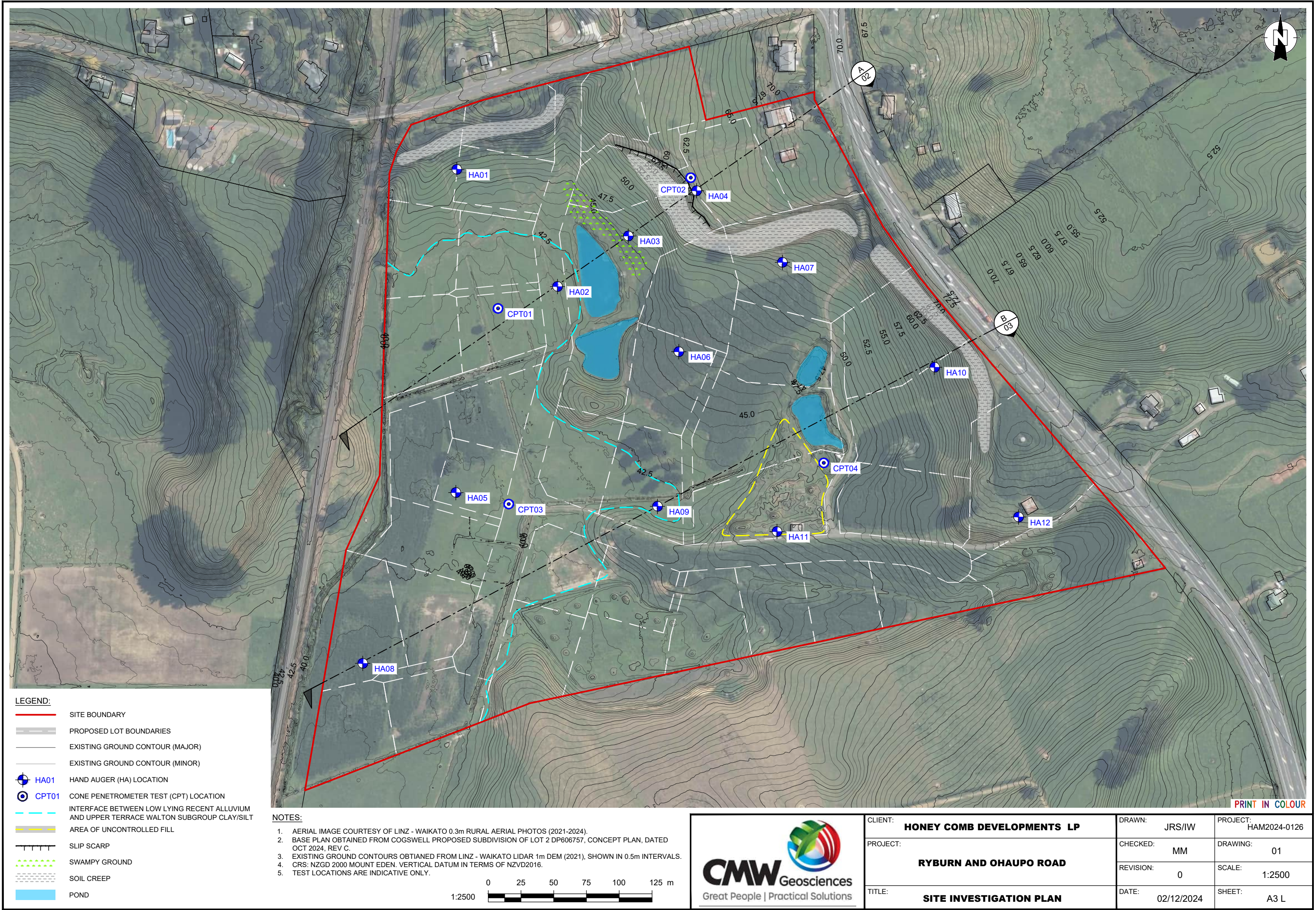
The personnel, equipment, and techniques used to perform an environmental study can differ significantly from those used in this report. For that reason, our report does not provide environmental recommendations. Unanticipated subsurface environmental problems can have large consequences for your site. If you have not obtained your own environmental information about the project site, ask your CMW contact about how to find environmental risk-management guidance.

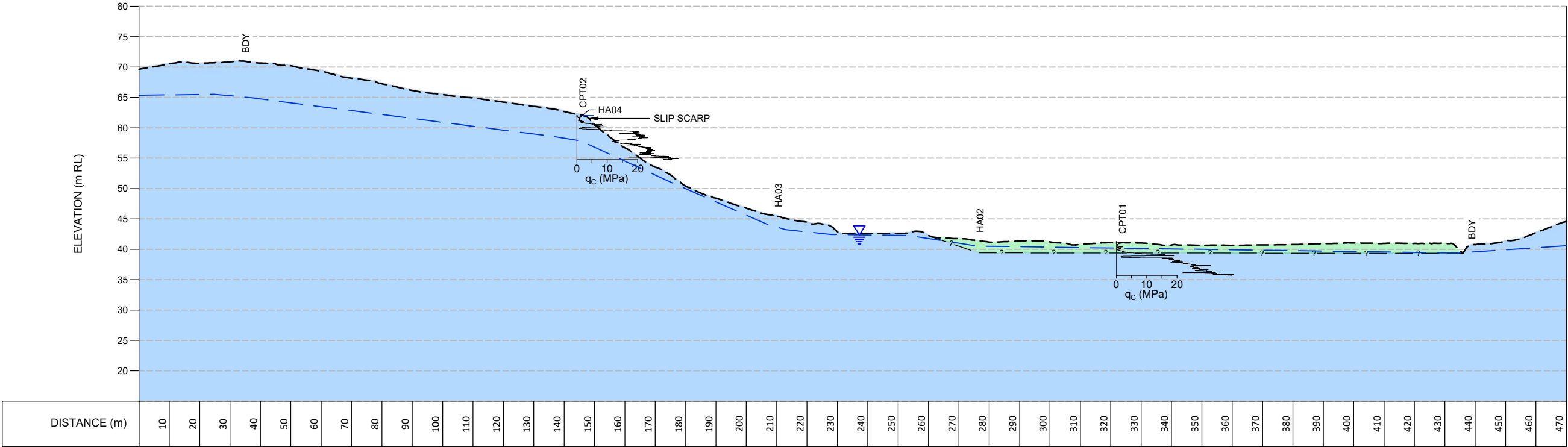
APPENDIX A

Drawings

Table A.1: Drawing Numbers

Title	Reference No.	Date	Revision
Site Investigation Plan	Drawing 01	02/12/2024	0
Cross Section A	Drawing 02	02/12/2024	0
Cross Section B	Drawing 03	02/12/2024	0





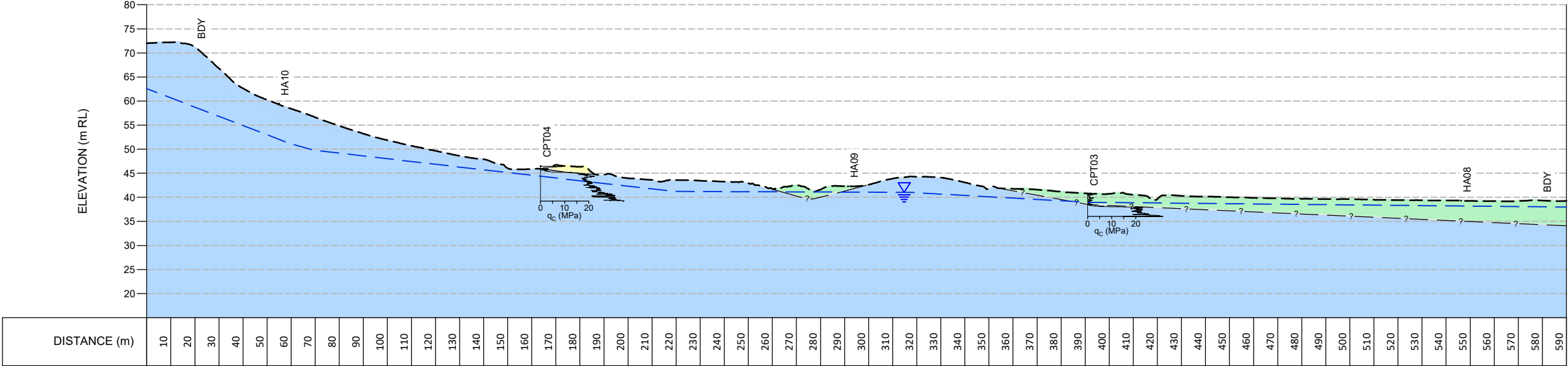
CROSS SECTION A

LEGEND:

---	EXISTING GROUND PROFILE
- ? -	INFERRED GEOLOGY BOUNDARY
— Δ —	GROUND WATER LEVEL
■	UNCONTROLLED FILL - FIRM TO STIFF SILT
■	RECENT ALLUVIUM - FIRM TO VERY STIFF INTERBEDDED SILT/SAND
■	WALTON SUBGROUP - VERY STIFF TO HARD SANDY OR SILTY CLAY

- NOTES:
- EXISTING GROUND PROFILE CREATED FROM WAIKATO LIDAR 1m DEM (2021).
 - VERTICAL DATUM IN TERMS OF NZVD2016.
 - TEST LOCATIONS ARE INDICATIVE ONLY.

	CLIENT:	HONEY COMB DEVELOPMENTS LP		DRAWN:	JRS/IW	PROJECT:	HAM2024-0126
	PROJECT:	RYBURN AND OHAUPO ROAD		CHECKED:	MM	DRAWING:	02
				REVISION:	0	SCALE:	1:1500
	TITLE:	CROSS SECTION A		DATE:	02/12/2024	SHEET:	A3 L



CROSS SECTION B

LEGEND:

- — — EXISTING GROUND PROFILE
- ? - - - INFERRED GEOLOGY BOUNDARY
- Δ - - - GROUND WATER LEVEL
- UNCONTROLLED FILL - FIRM TO STIFF SILT
- RECENT ALLUVIUM - FIRM TO VERY STIFF INTERBEDDED SILT/SAND
- WALTON SUBGROUP - VERY STIFF TO HARD SANDY OR SILTY CLAY

NOTES:

- EXISTING GROUND PROFILE CREATED FROM WAIKATO LIDAR 1m DEM (2021).
- VERTICAL DATUM IN TERMS OF NZVD2016.
- TEST LOCATIONS ARE INDICATIVE ONLY.



CLIENT:	HONEY COMB DEVELOPMENTS LP		DRAWN:	JRS/IW	PROJECT:	HAM2024-0126
PROJECT:	RYBURN AND OHAUPO ROAD		CHECKED:	MM	DRAWING:	03
			REVISION:	0	SCALE:	1:2000
TITLE:	CROSS SECTION B		DATE:	02/12/2024	SHEET:	A3 L

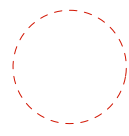
PRINT IN COLOUR

APPENDIX B

Proposed Development Plans



Total Area:	20.7170ha
Number of Lots:	40
Average Area:	4232m ²
Maximum Area:	9300m ²
Minimum Area:	2526m ²
Area of Road:	1.7584ha
Area of Reserve:	1.4054ha
Area of Access:	6230m ²

 = 30mØ Building Shape Factor, clear of:
- 5m setback from internal boundaries;
- 15m setback from strategic roads (S.H.3);
- 10m setback from other roads;
- 10m setback from rural zone.



COGSWELL
SURVEYING | ENGINEERING | PLANNING
www.cogswell.co.nz

CLIENT:
Honey Comb Developments LP
REF: 20904

PROJECT:
Large Lot Residential Development

TITLE:
Proposed Subdivision of Lot 2 DP 606757
CONCEPT PLAN
Ryburn & Ohaupo Road

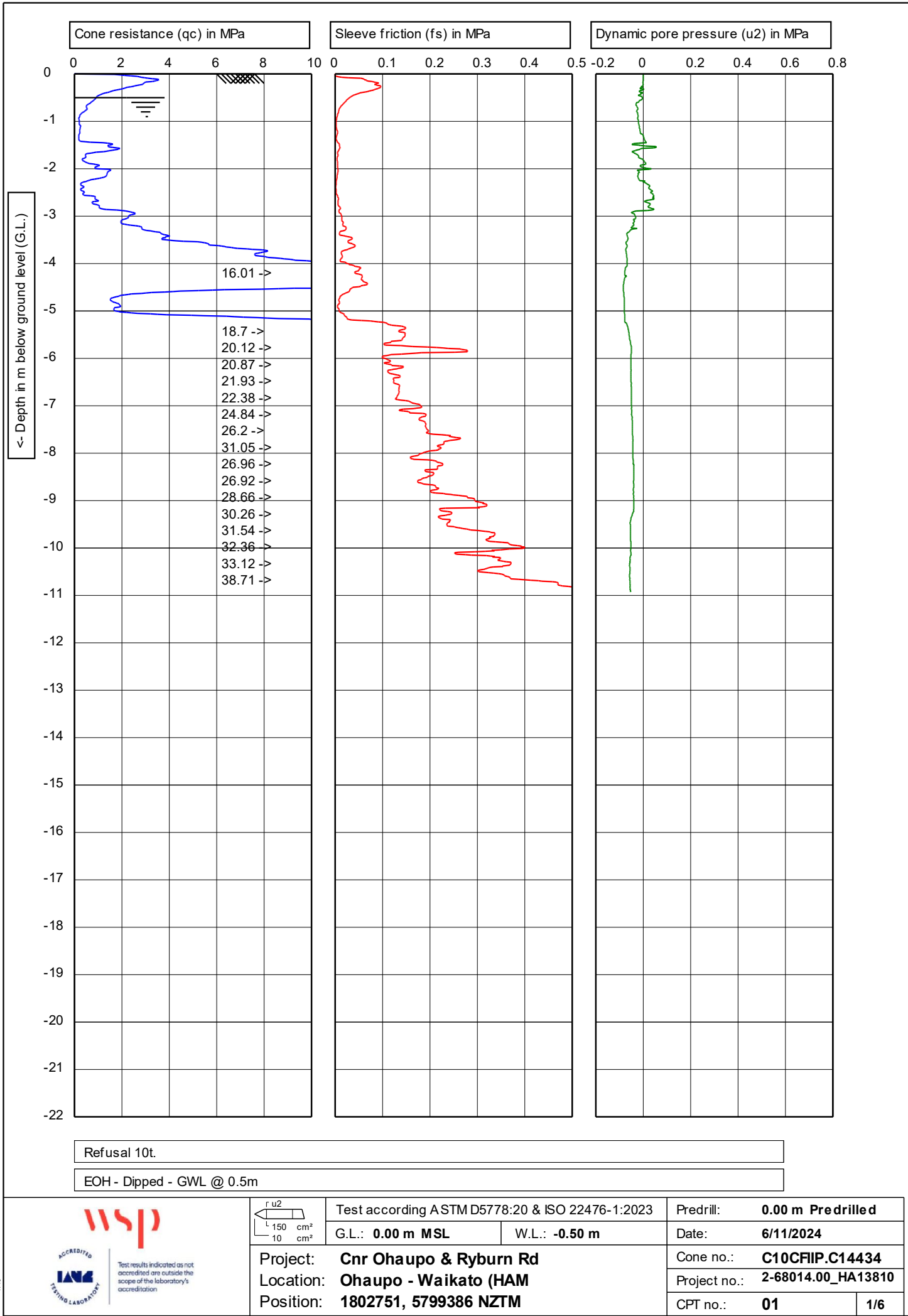
NOTES:
1. Boundaries, Areas and Dimensions are subject to approval and survey.
This drawing or design remains the property of, and may not be reproduced, without the written permission of Cogswell Surveys Ltd.

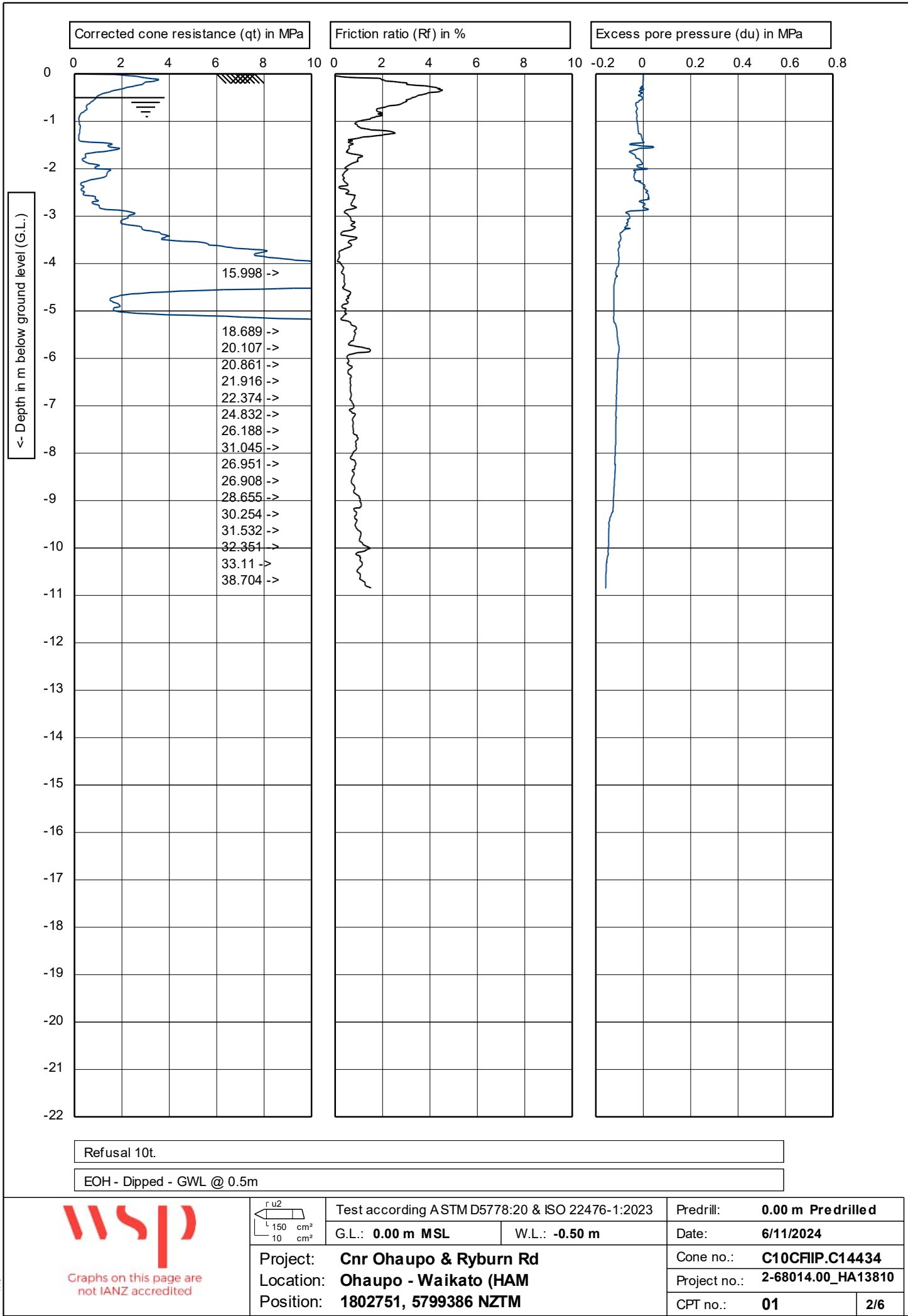
SCALE: 1:1000 @A1
1:2000 @A3
DATE: OCT 2024
DRAWING NUMBER: 5410-CP-2
REV: C

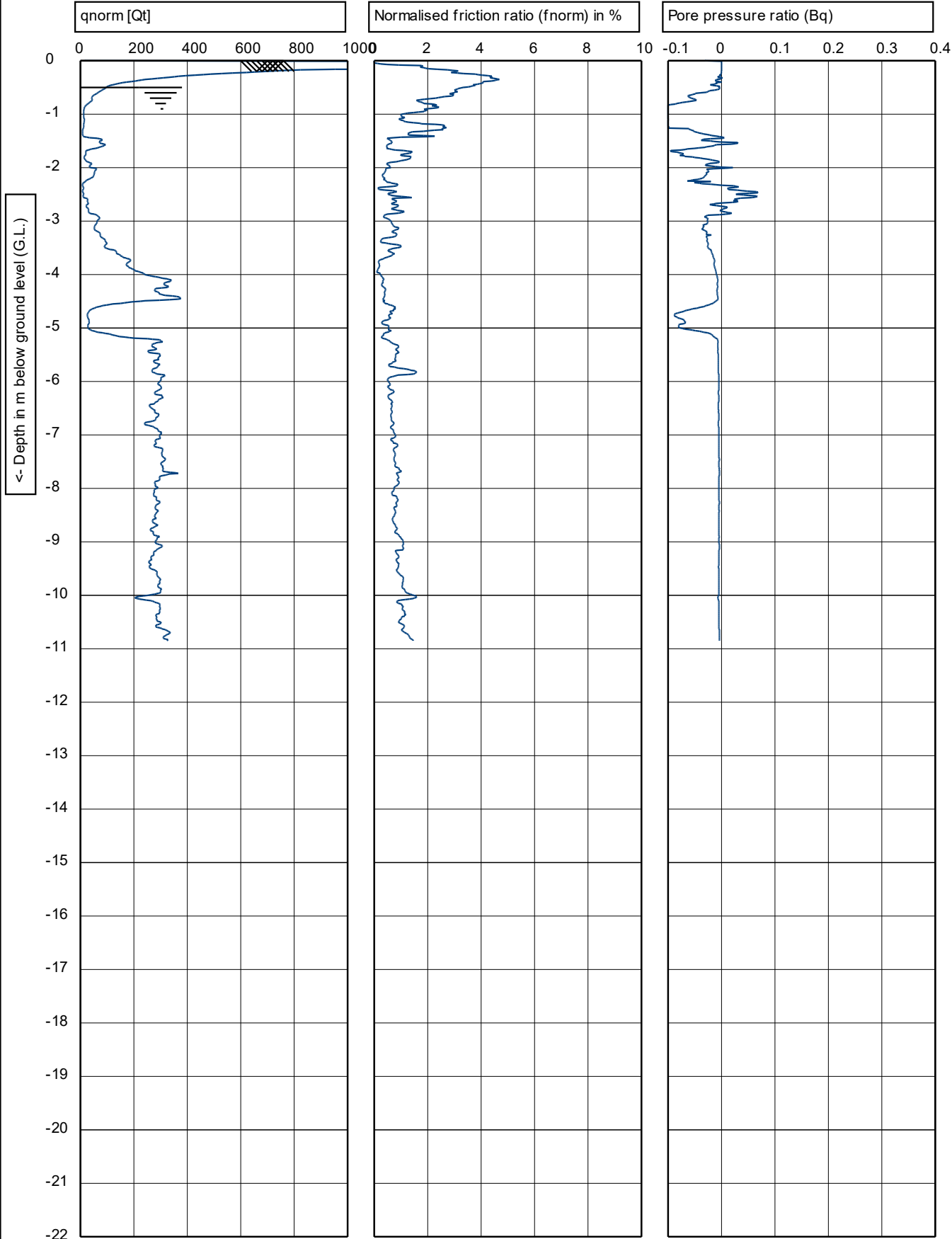
PLOTTED: Mon Oct 21 15:14:42 2024

APPENDIX C

Cone Penetrometer Test Results





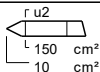


Refusal 10t.

EOH - Dipped - GWL @ 0.5m



Graphs on this page are
not IANZ accredited



Test according ASTM D5778:20 & ISO 22476-1:2023

G.L.: 0.00 m MSL

W.L.: -0.50 m

Predrill: 0.00 m Predrilled

Date: 6/11/2024

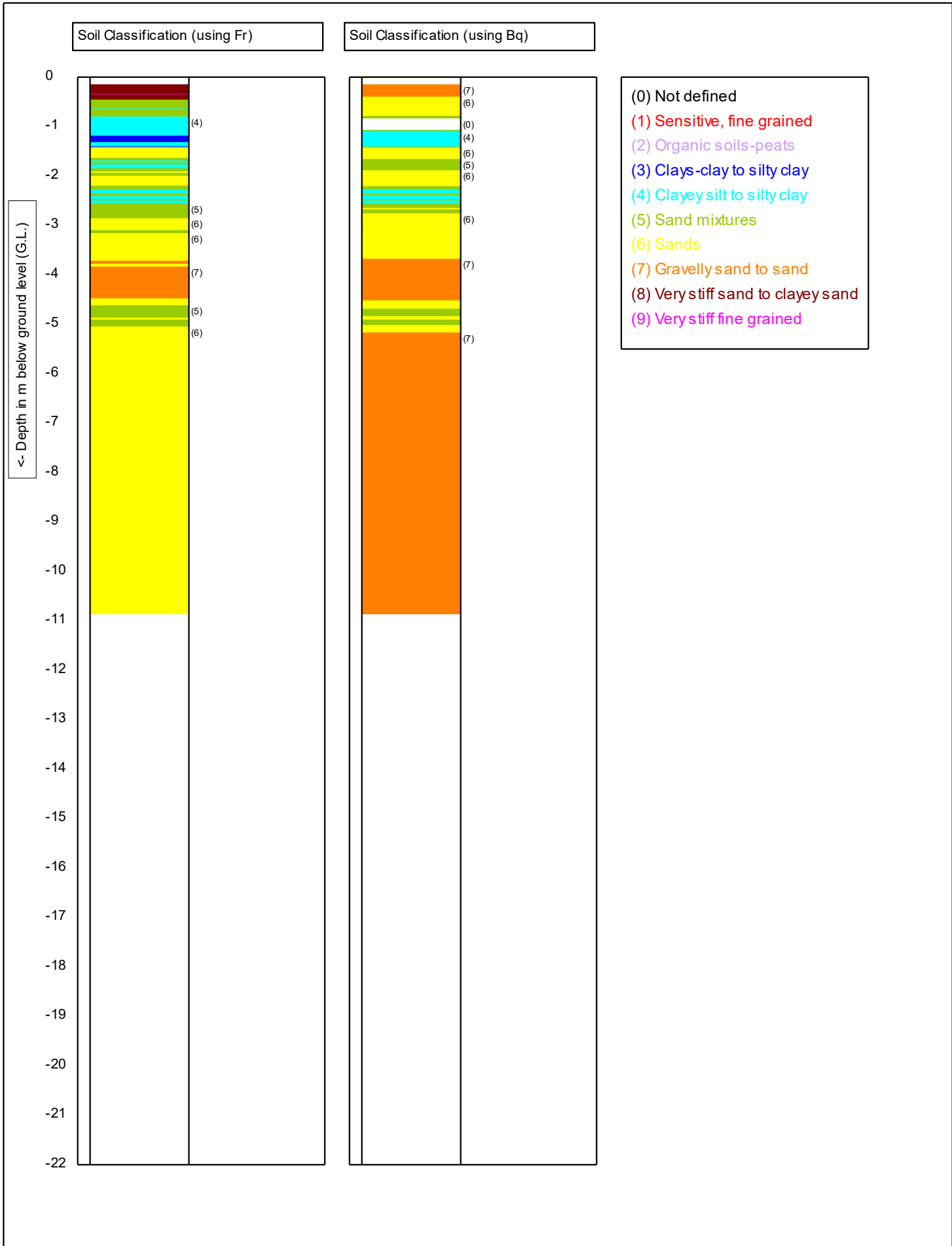
Cone no.: C10CFIIP.C14434

Project no.: 2-68014.00_HA13810

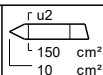
CPT no.: 01

3/6

Project: Cnr Ohaupo & Ryburn Rd
Location: Ohaupo - Waikato (HAM)
Position: 1802751, 5799386 NZTM



Graphs on this page are
not IANZ accredited



Test according ASTM D5778:20 & ISO 22476-1:2023

G.L.: 0.00 m MSL

W.L.: -0.50 m

Predrill: 0.00 m Predrilled

Date: 6/11/2024

Cone no.: C10CFIIP.C14434

Project no.: 2-68014.00_HA13810

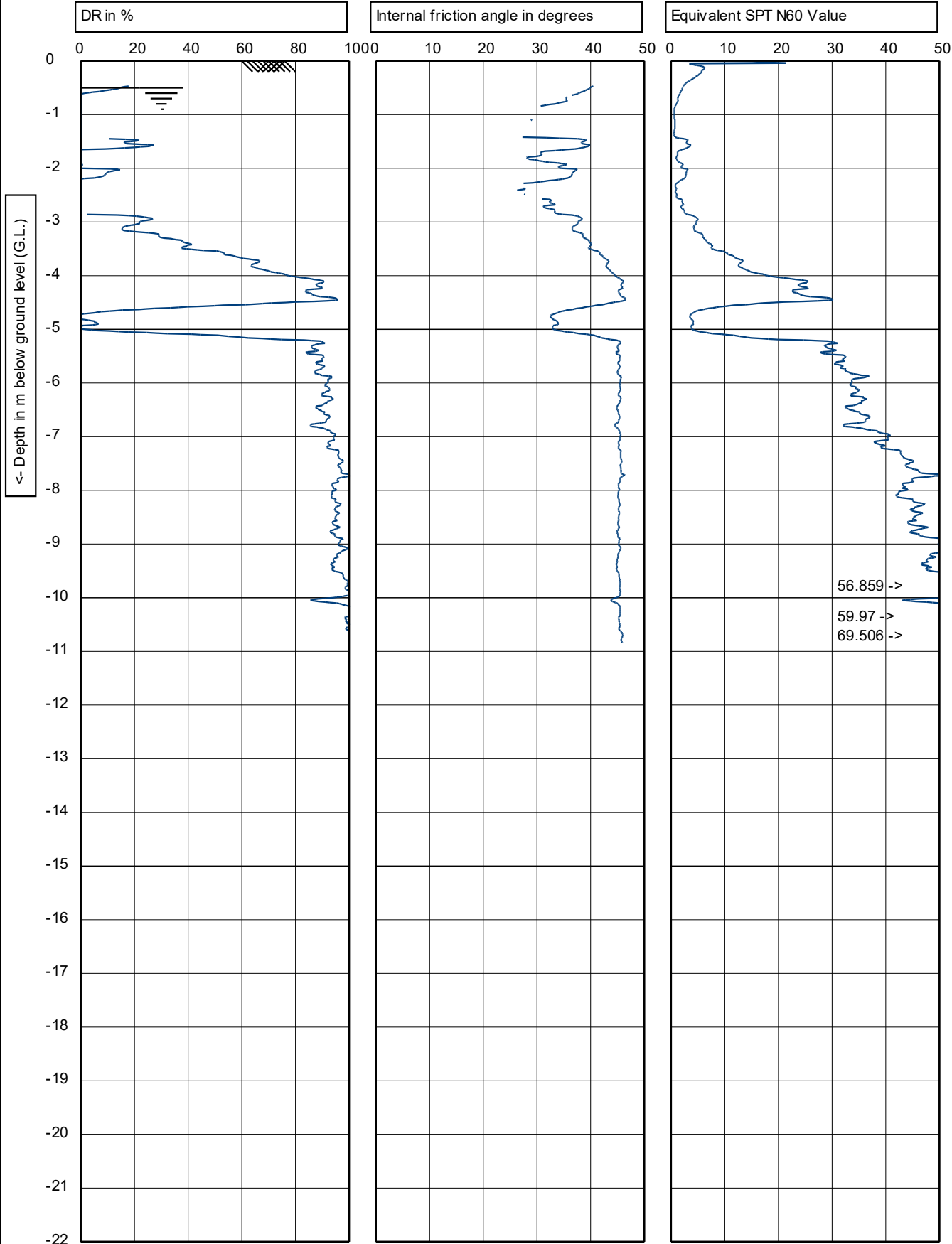
CPT no.: 01

4/6

Project: Cnr Ohaupo & Ryburn Rd

Location: Ohaupo - Waikato (HAM)

Position: 1802751, 5799386 NZTM



Refusal 10t.

EOH - Dipped - GWL @ 0.5m



Graphs on this page are
not IANZ accredited



Test according ASTM D5778:20 & ISO 22476-1:2023

G.L.: 0.00 m MSL

W.L.: -0.50 m

Predrill: 0.00 m Predrilled

Date: 6/11/2024

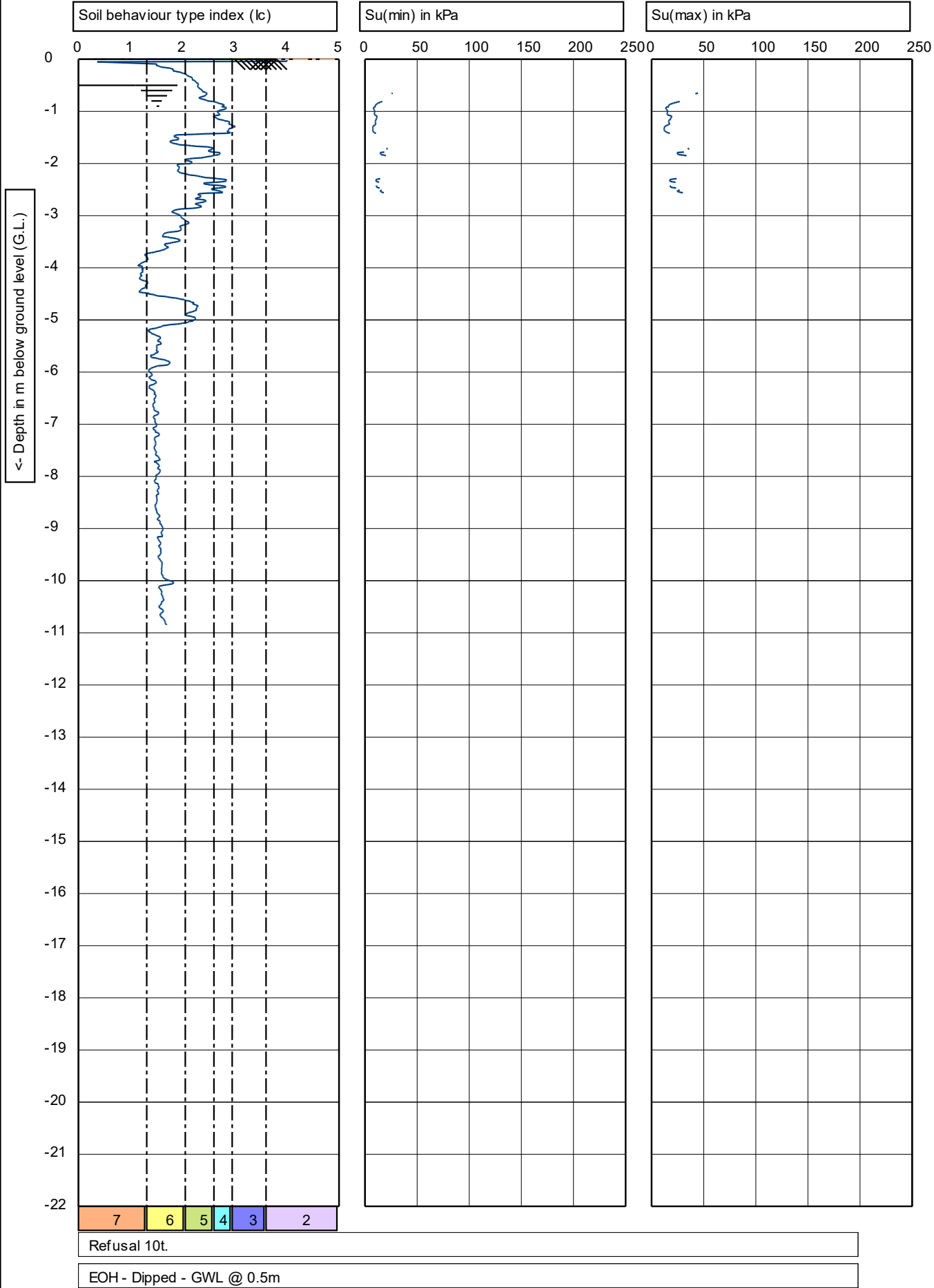
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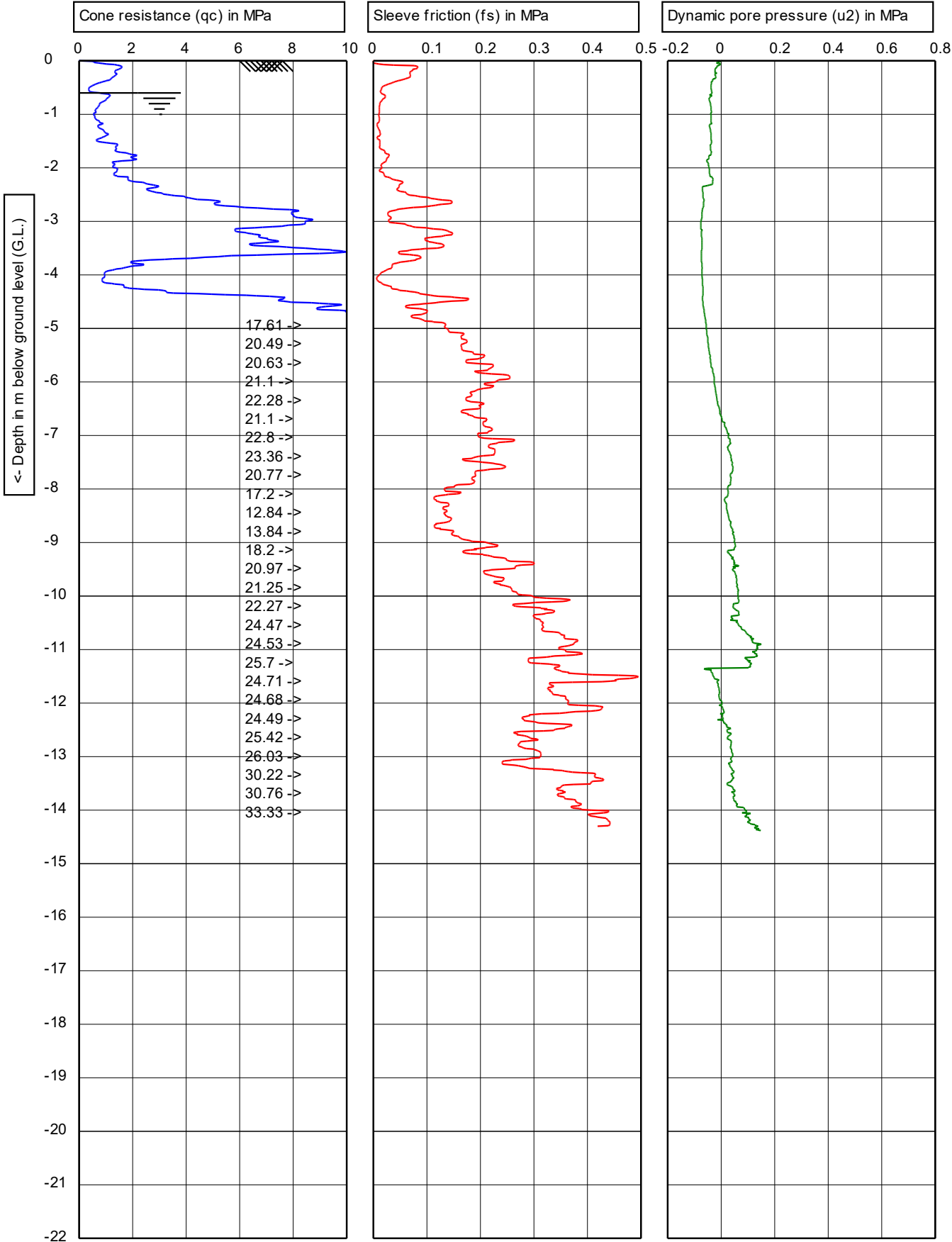
Project no.: 2-68014.00_HA13810

CPT no.: 01

5/6

Project: Cnr Ohaupo & Ryburn Rd
Location: Ohaupo - Waikato (HAM)
Position: 1802751, 5799386 NZTM



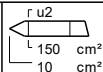


Refusal 10t.

EOH - Dipped - GWL @ 0.6m



Test results indicated as not accredited are outside the scope of the laboratory's accreditation



Test according ASTM D5778:20 & ISO 22476-1:2023

G.L.: 0.00 m MSL

W.L.: -0.60 m

Project: **Cnr Ohaupo & Ryburn Rd**
Location: **Ohaupo - Waikato (HAM)**
Position: **1802820, 5799507 NZTM**

Predrill: **0.00 m Predrilled**

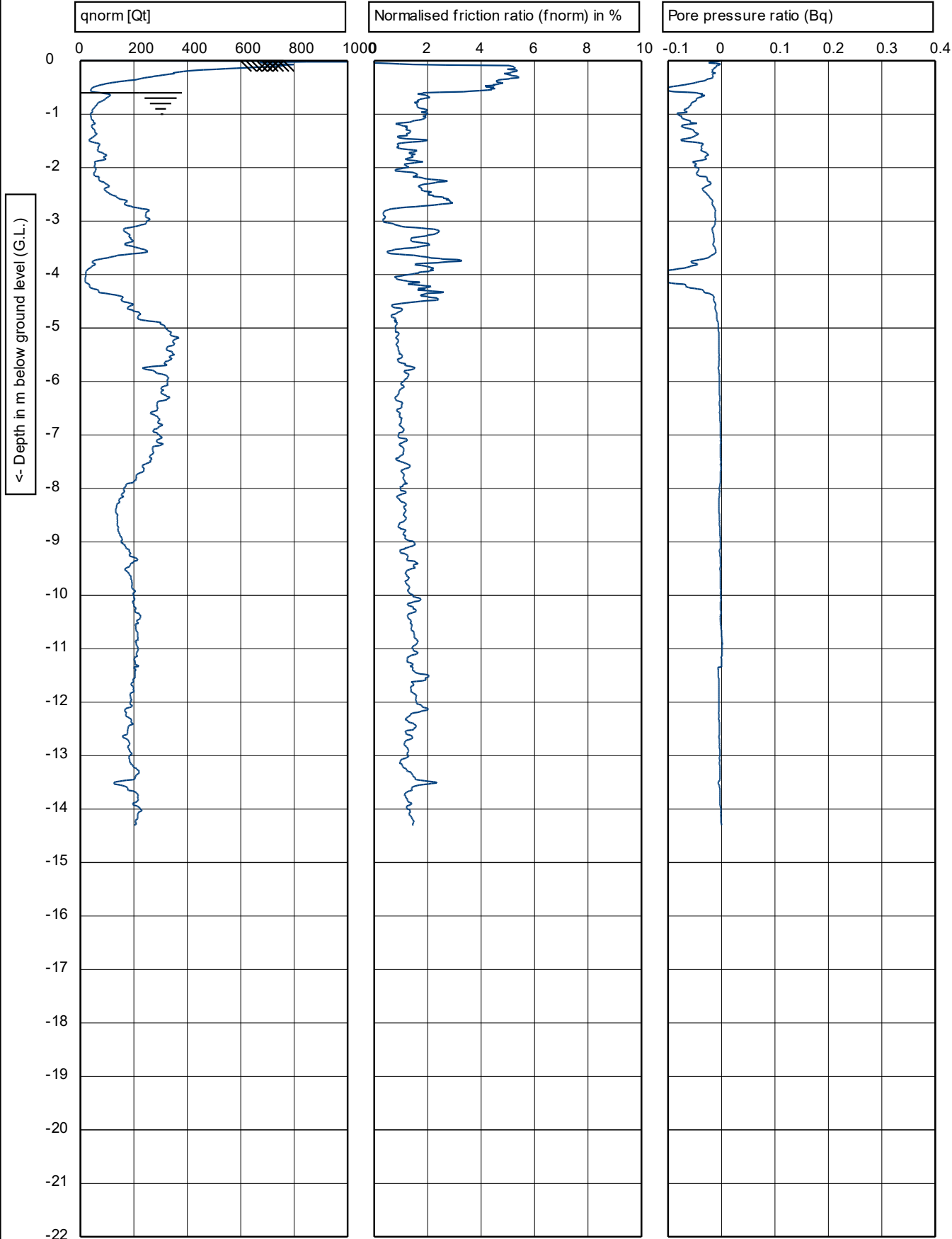
Date: **6/11/2024**

Cone no.: **C10CFIIP.C181009**

Project no.: **2-68014.00_HA13810**

CPT no.: **02**

1/6



Refusal 10t.

EOH - Dipped - GWL @ 0.6m



Graphs on this page are
not IANZ accredited



Test according ASTM D5778:20 & ISO 22476-1:2023

G.L.: 0.00 m MSL

W.L.: -0.60 m

Predrill: 0.00 m Predrilled

Date: 6/11/2024

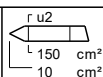
Cone no.: C10CFIIP.C181009

Project no.: 2-68014.00_HA13810

CPT no.: 02

3/6

Project: Cnr Ohaupo & Ryburn Rd
Location: Ohaupo - Waikato (HAM)
Position: 1802820, 5799507 NZTM



Test according ASTM D5778:20 & ISO 22476-1:2023

G.L.: 0.00 m MSL

W.L.: -0.60 m

Predrill:	0.00 m	Predrilled
-----------	--------	------------

Date:	6/11/2024
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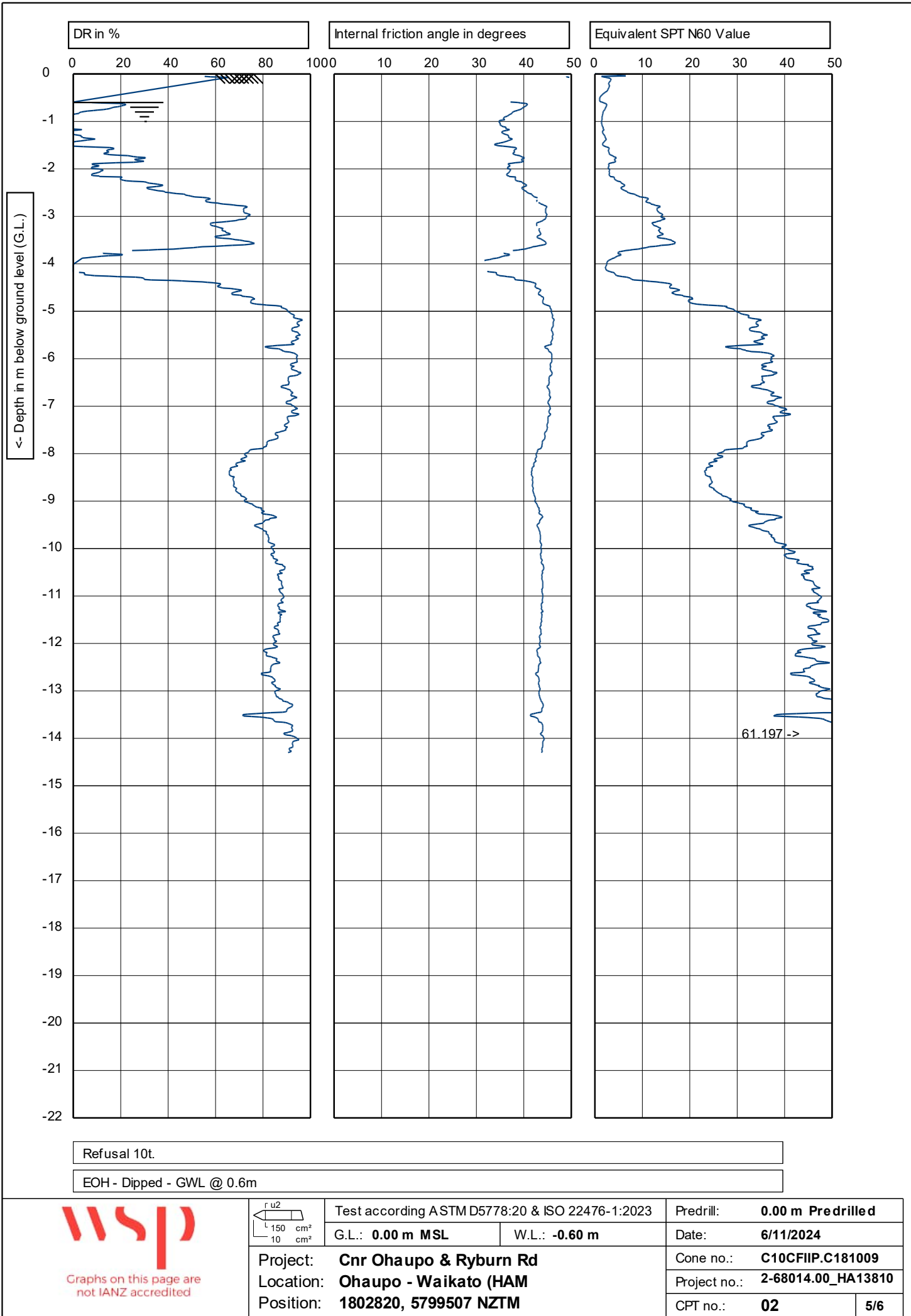
Cone no.:	C10CFIIP.C181009
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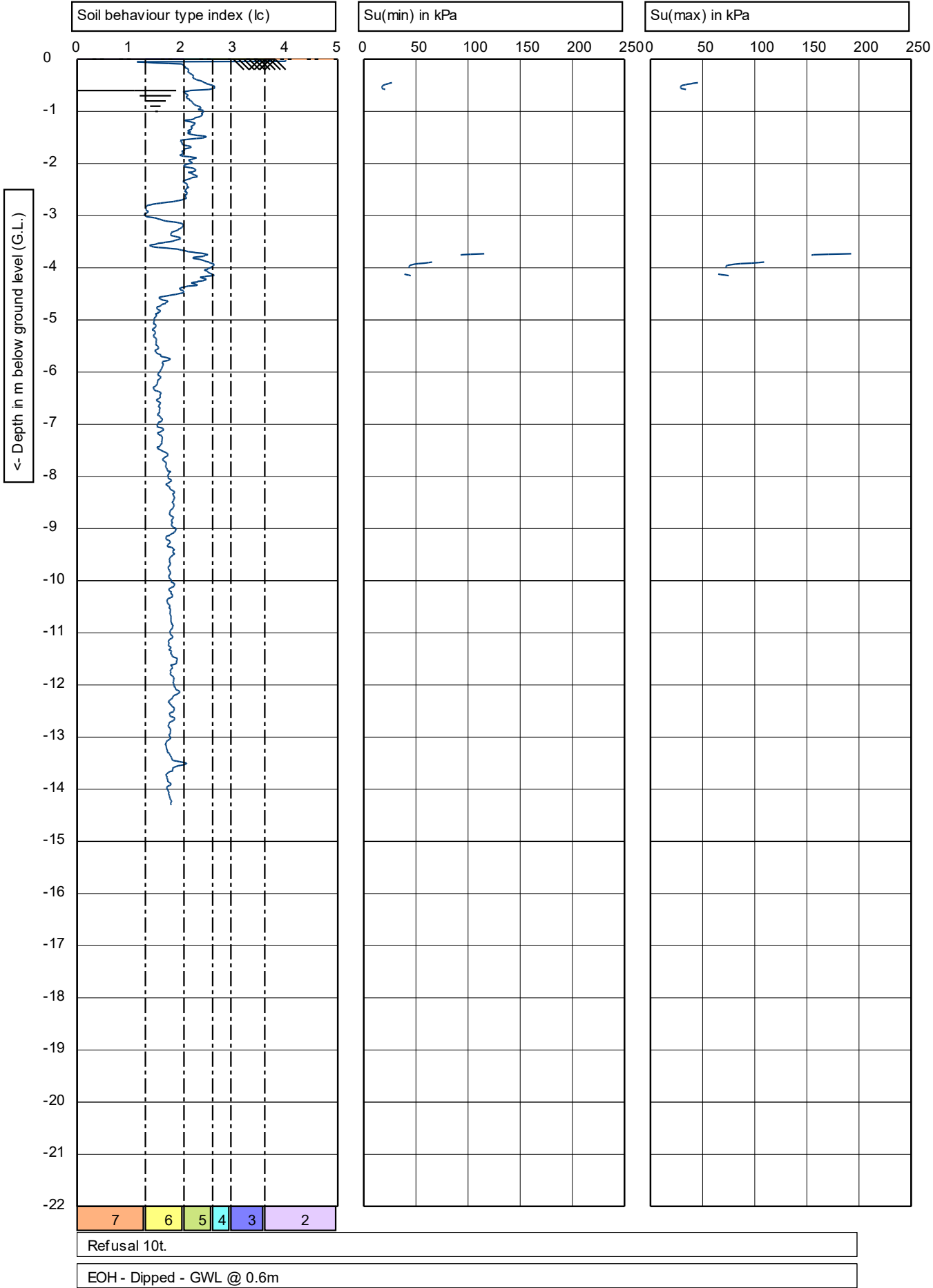
Project no.:	2-68014.00_HA13810
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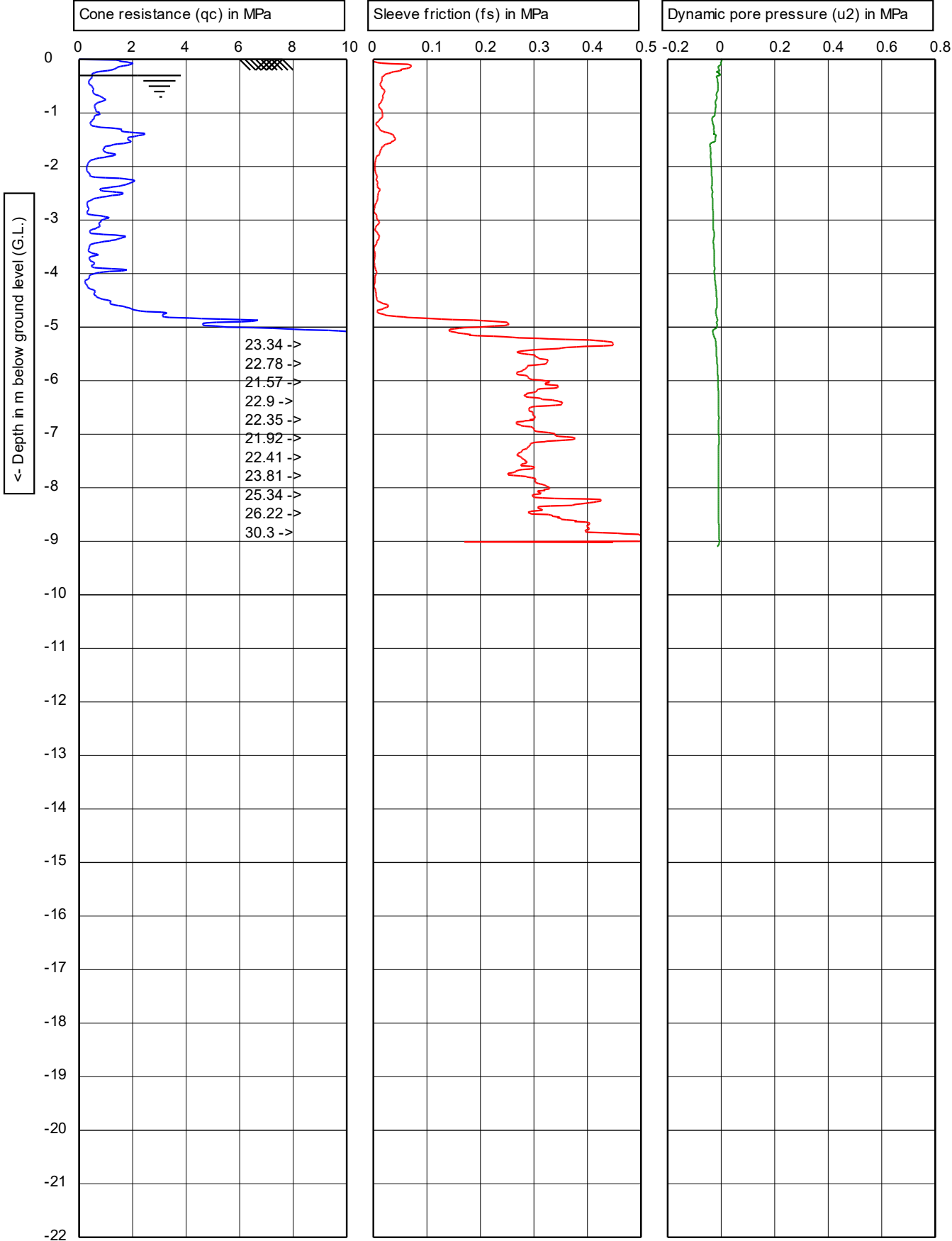
CPT no.:	02
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4/6

Project: Cnr Ohaupo & Ryburn Rd
Location: Ohaupo - Waikato (HAM
Position: 1802820, 5799507 NZTM





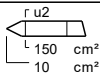


Refusal 10t.

EOH - Dipped - GWL @ 0.3m



Test results indicated as not accredited are outside the scope of the laboratory's accreditation



Test according ASTM D5778:20 & ISO 22476-1:2023

G.L.: 0.00 m MSL

W.L.: -0.30 m

Project: Cnr Ohaupo & Ryburn Rd
Location: Ohaupo - Waikato (HAM)
Position: 1802767, 5799271 NZTM

Predrill: 0.00 m Predrilled

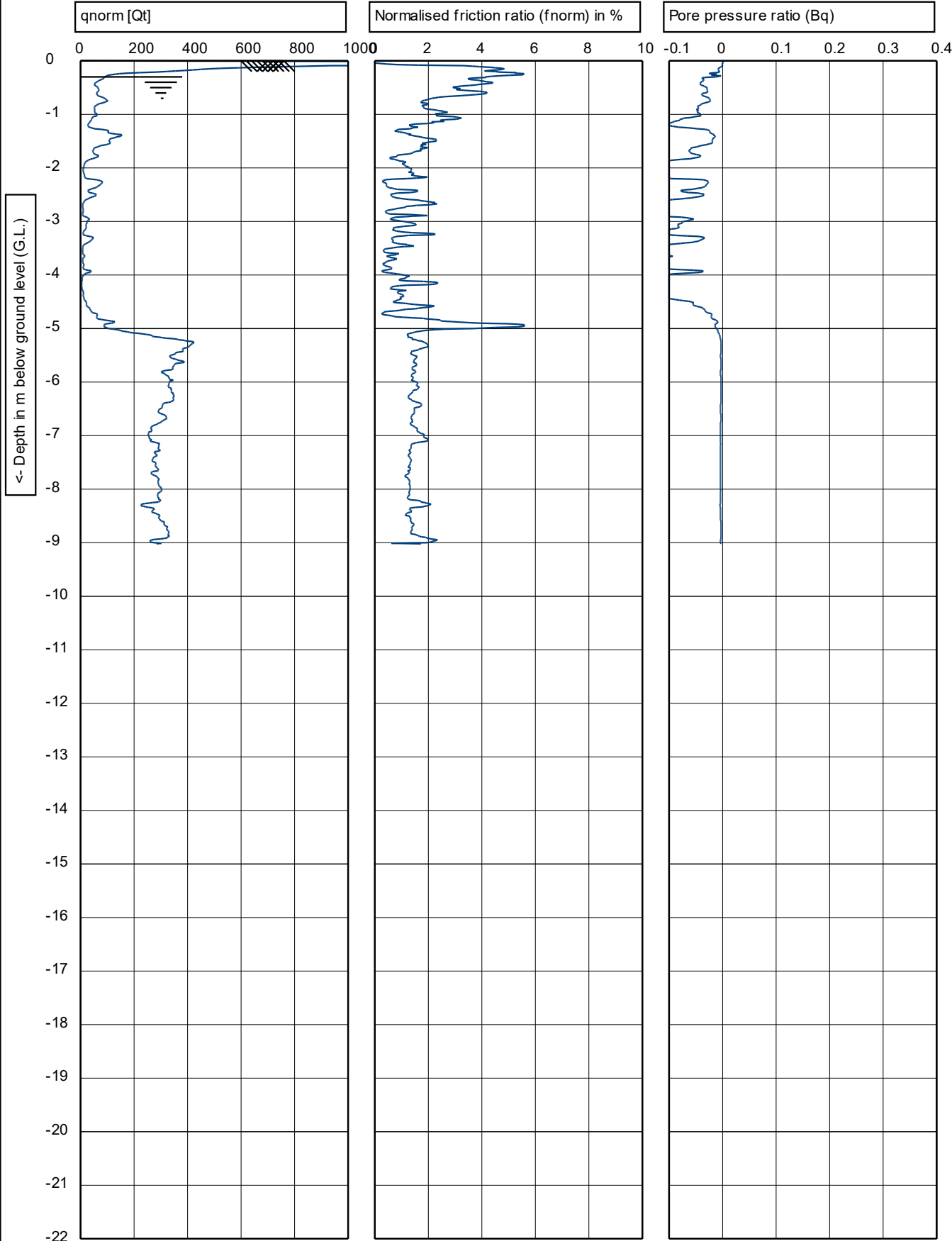
Date: 6/11/2024

Cone no.: C10CFIIP.C181009

Project no.: 2-68014.00_HA13810

CPT no.: 03

1/6



Refusal 10t.

EOH - Dipped - GWL @ 0.3m



Graphs on this page are
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Test according ASTM D5778:20 & ISO 22476-1:2023

G.L.: 0.00 m MSL

W.L.: -0.30 m

Predrill: 0.00 m Predrilled

Date: 6/11/2024

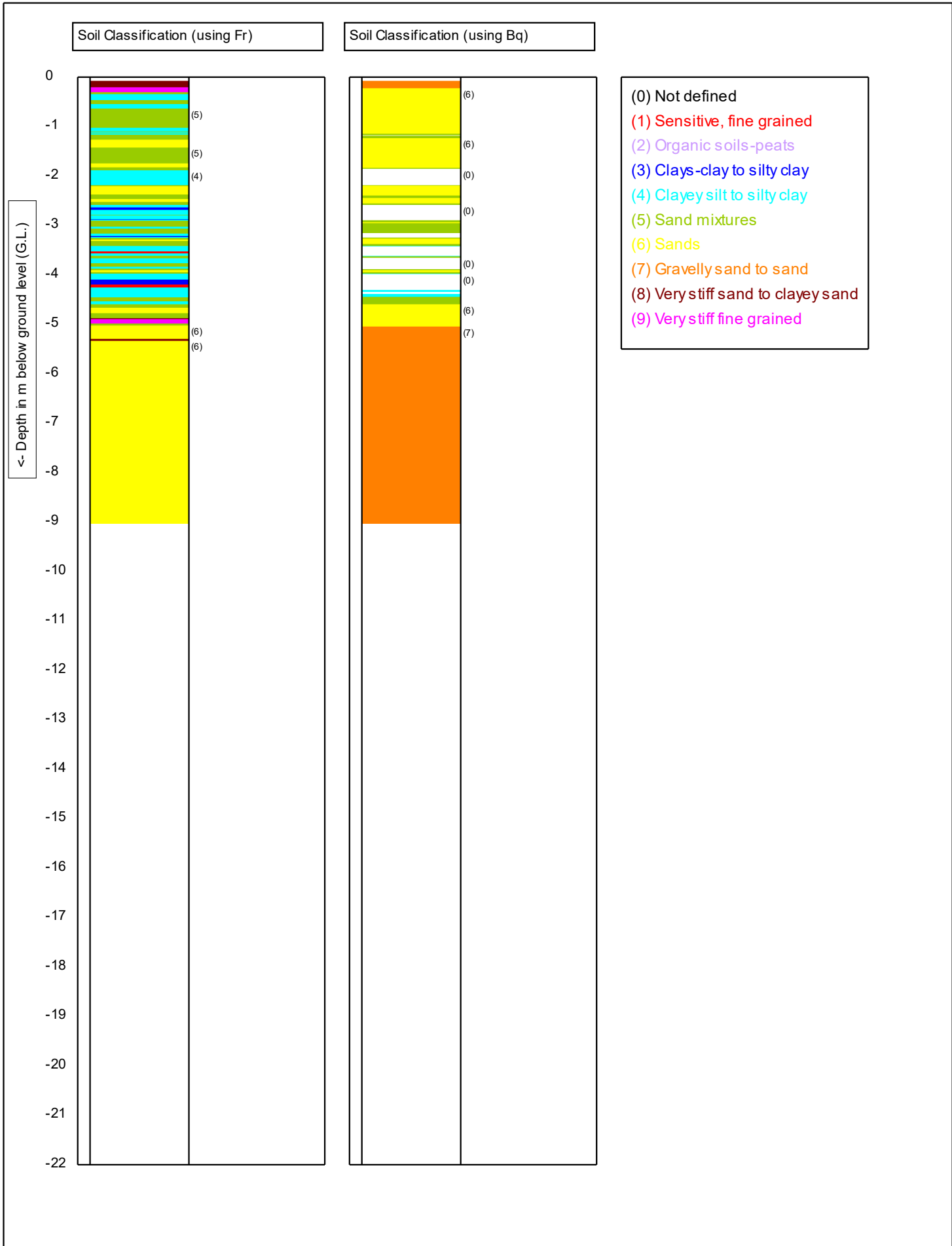
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Project no.: 2-68014.00_HA13810

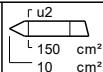
CPT no.: 03

3/6

Project: Cnr Ohaupo & Ryburn Rd
Location: Ohaupo - Waikato (HAM)
Position: 1802767, 5799271 NZTM



Graphs on this page are not IANZ accredited



Test according ASTM D5778:20 & ISO 22476-1:2023

G.L.: 0.00 m MSL

W.L.: -0.30 m

Predrill: 0.00 m Predrilled

Date: 6/11/2024

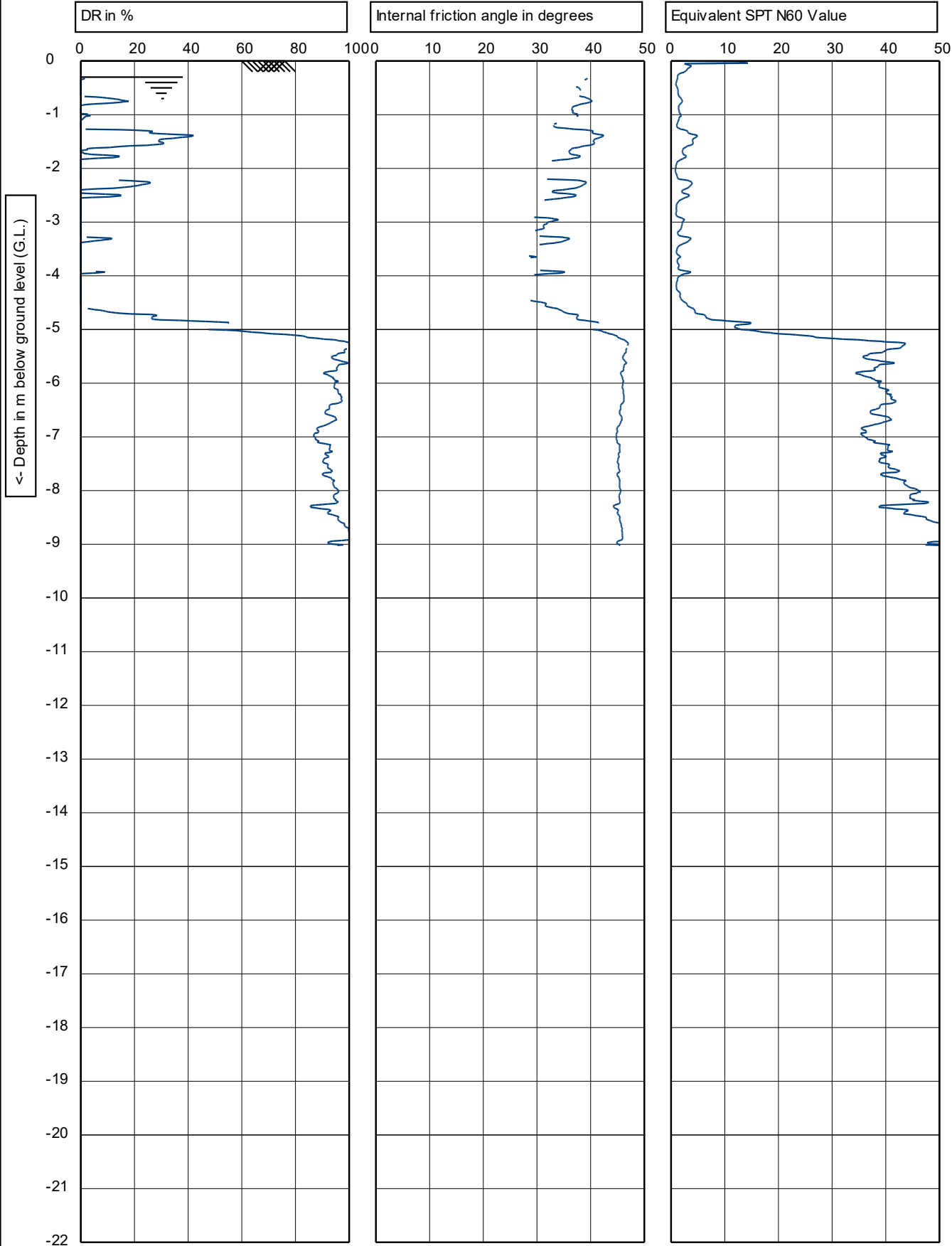
Cone no.: C10CFIIP.C181009

Project no.: 2-68014.00_HA13810

CPT no.: 03

4/6

Project: Cnr Ohaupo & Ryburn Rd
Location: Ohaupo - Waikato (HAM)
Position: 1802767, 5799271 NZTM

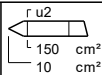


Refusal 10t.

EOH - Dipped - GWL @ 0.3m



Graphs on this page are
not IANZ accredited



Test according ASTM D5778:20 & ISO 22476-1:2023

G.L.: 0.00 m MSL

W.L.: -0.30 m

Predrill: 0.00 m Predrilled

Date: 6/11/2024

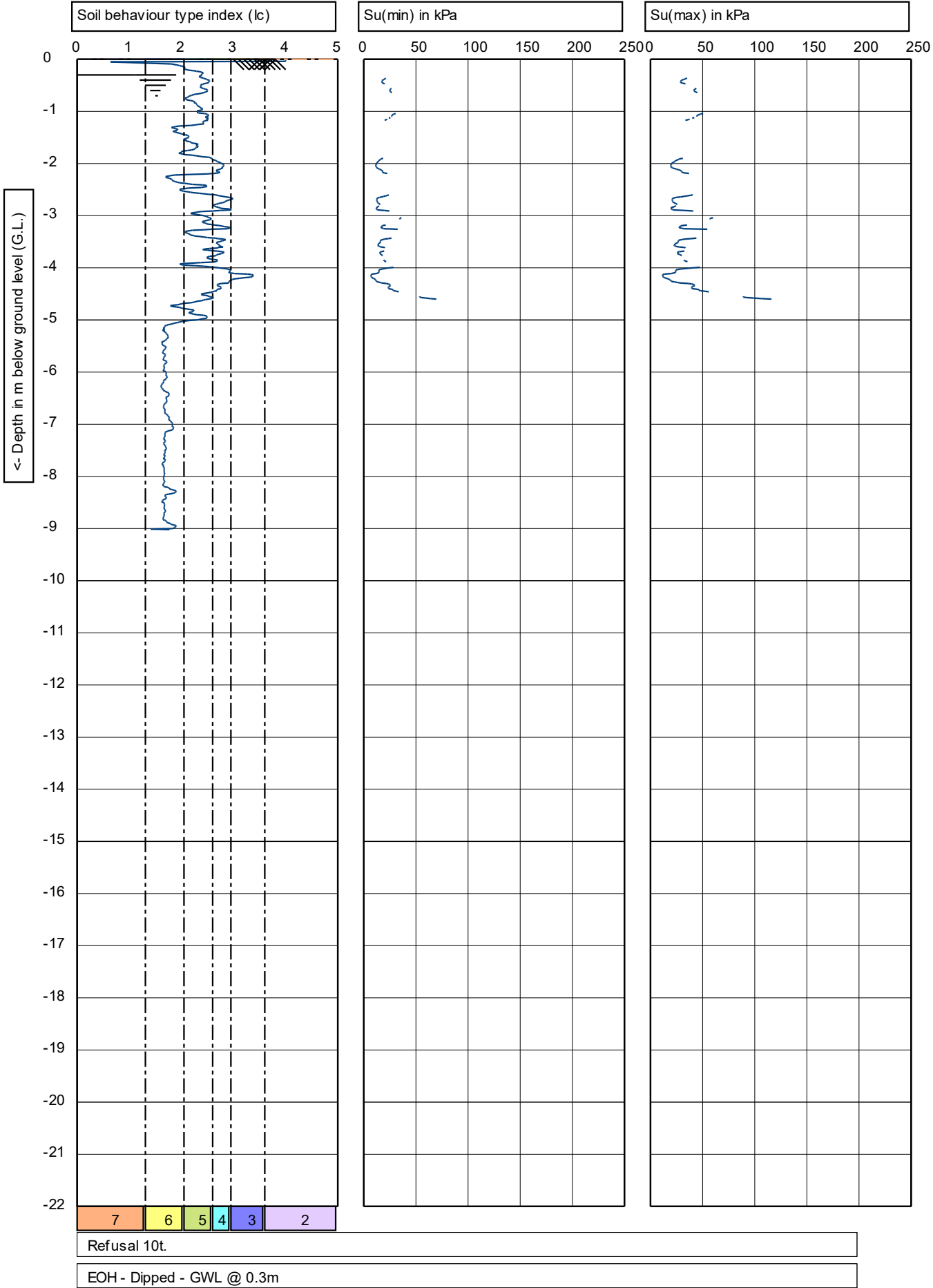
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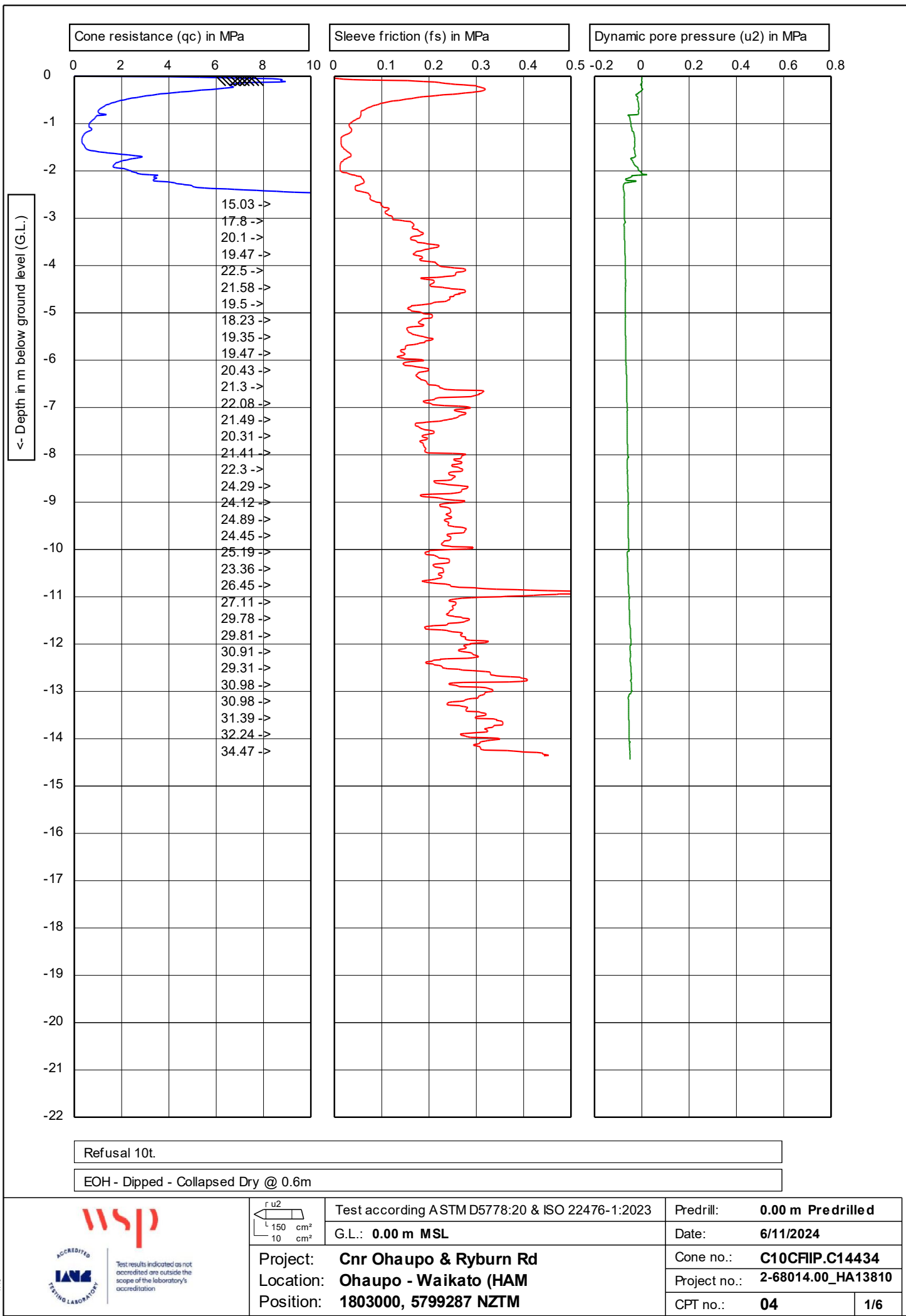
Project no.: 2-68014.00_HA13810

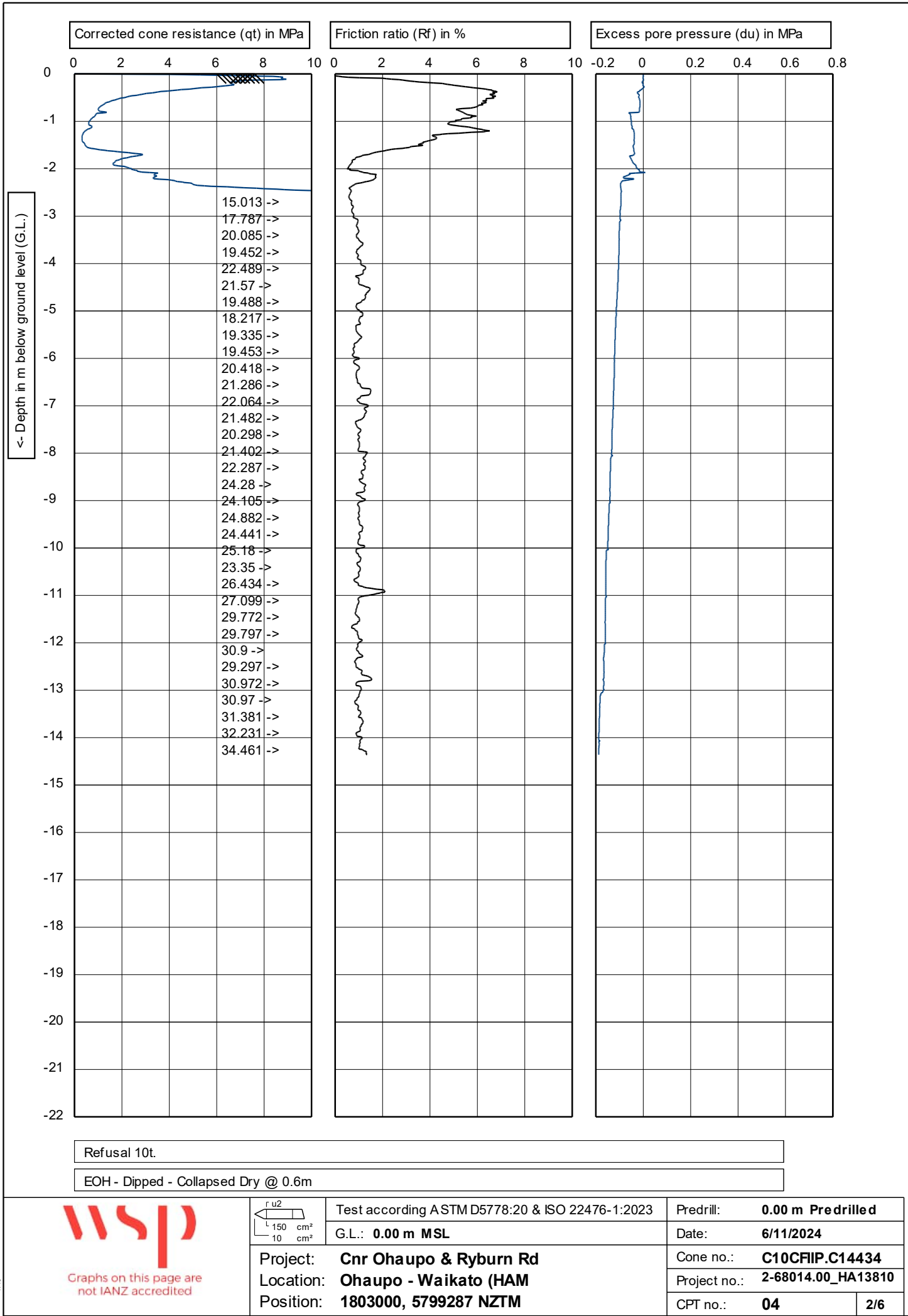
CPT no.: 03

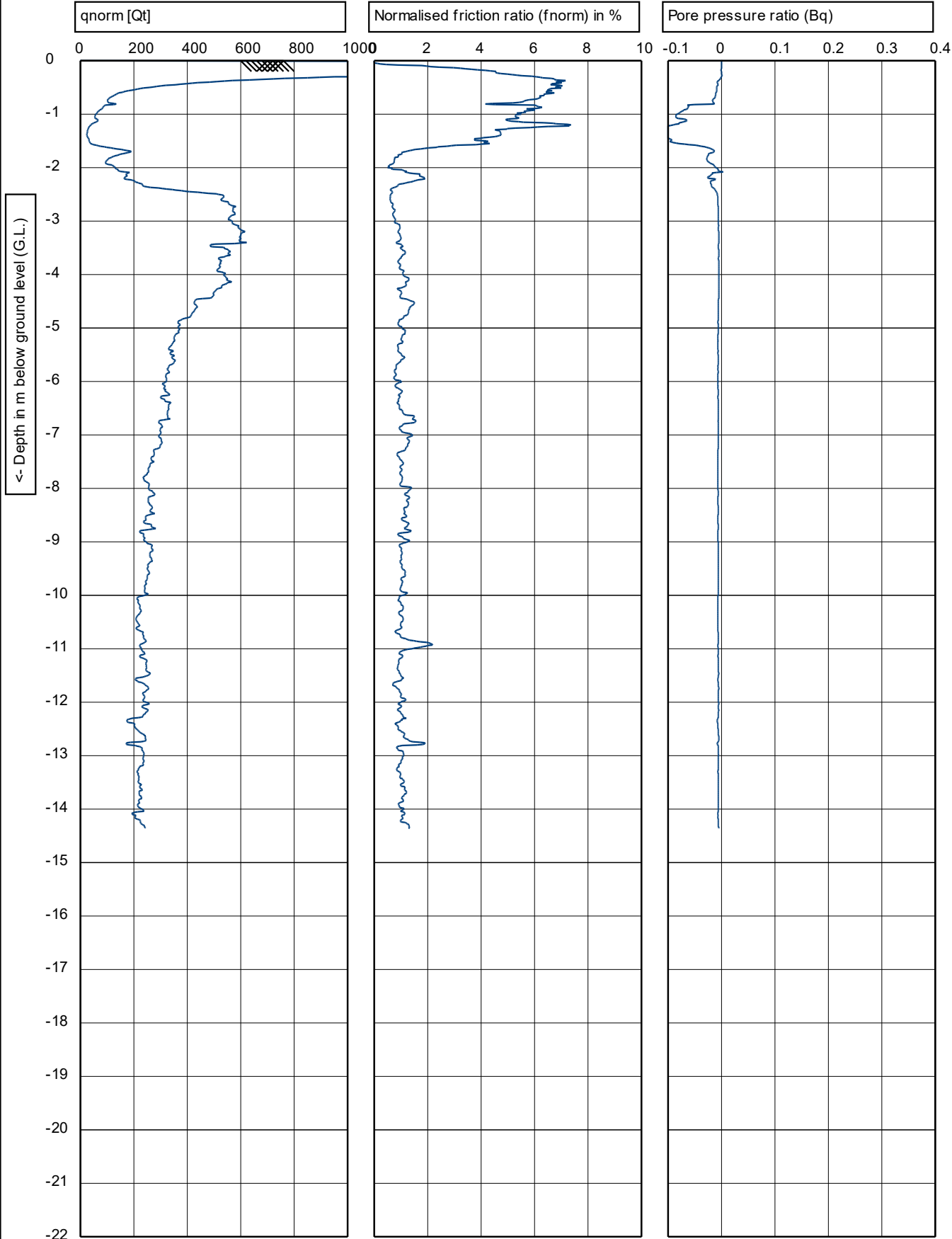
5/6

Project: Cnr Ohaupo & Ryburn Rd
Location: Ohaupo - Waikato (HAM)
Position: 1802767, 5799271 NZTM









Refusal 10t.

EOH - Dipped - Collapsed Dry @ 0.6m



Graphs on this page are not IANZ accredited



Test according ASTM D5778:20 & ISO 22476-1:2023

G.L.: 0.00 m MSL

Project: Cnr Ohaupo & Ryburn Rd
Location: Ohaupo - Waikato (HAM)
Position: 1803000, 5799287 NZTM

Predrill: 0.00 m Predrilled

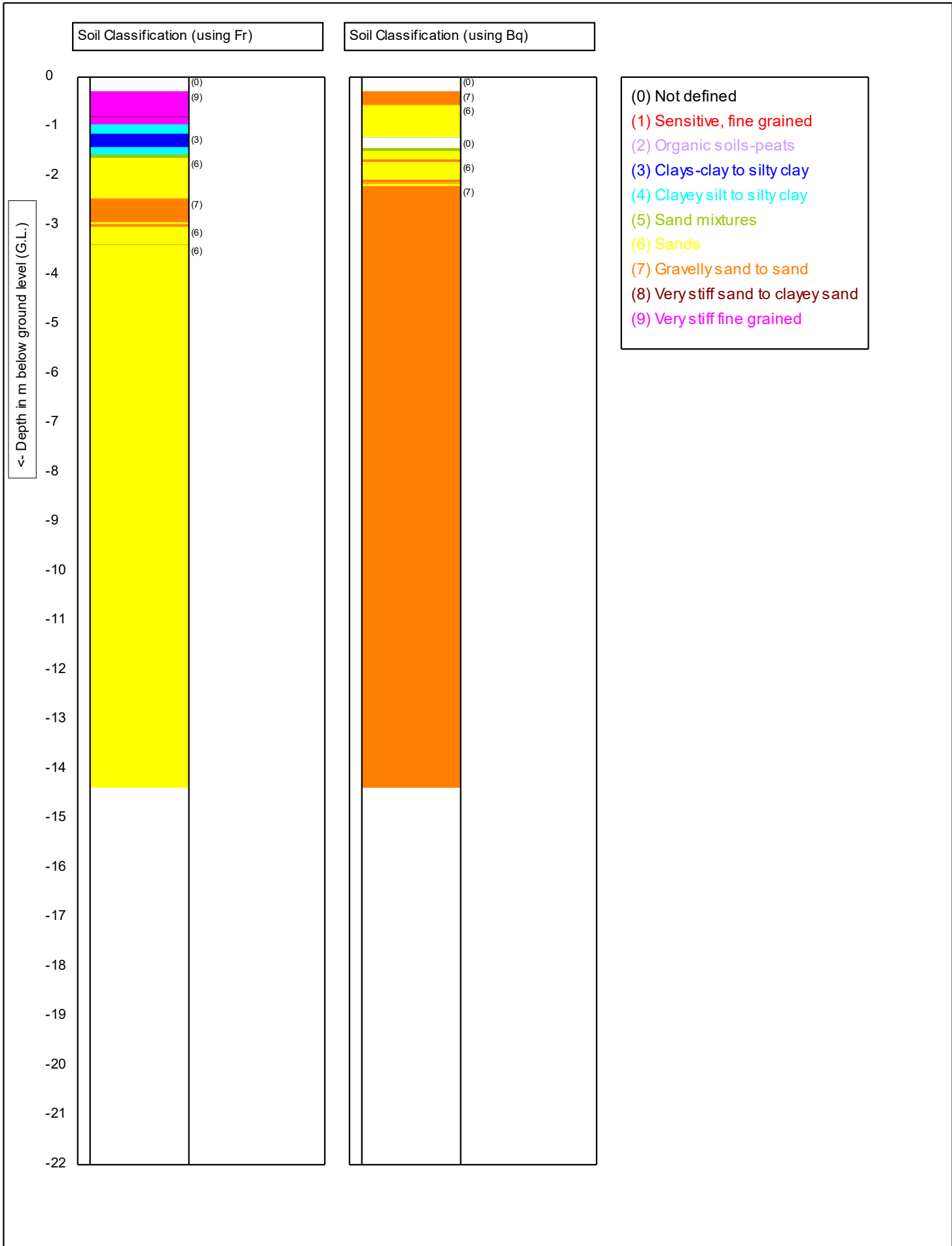
Date: 6/11/2024

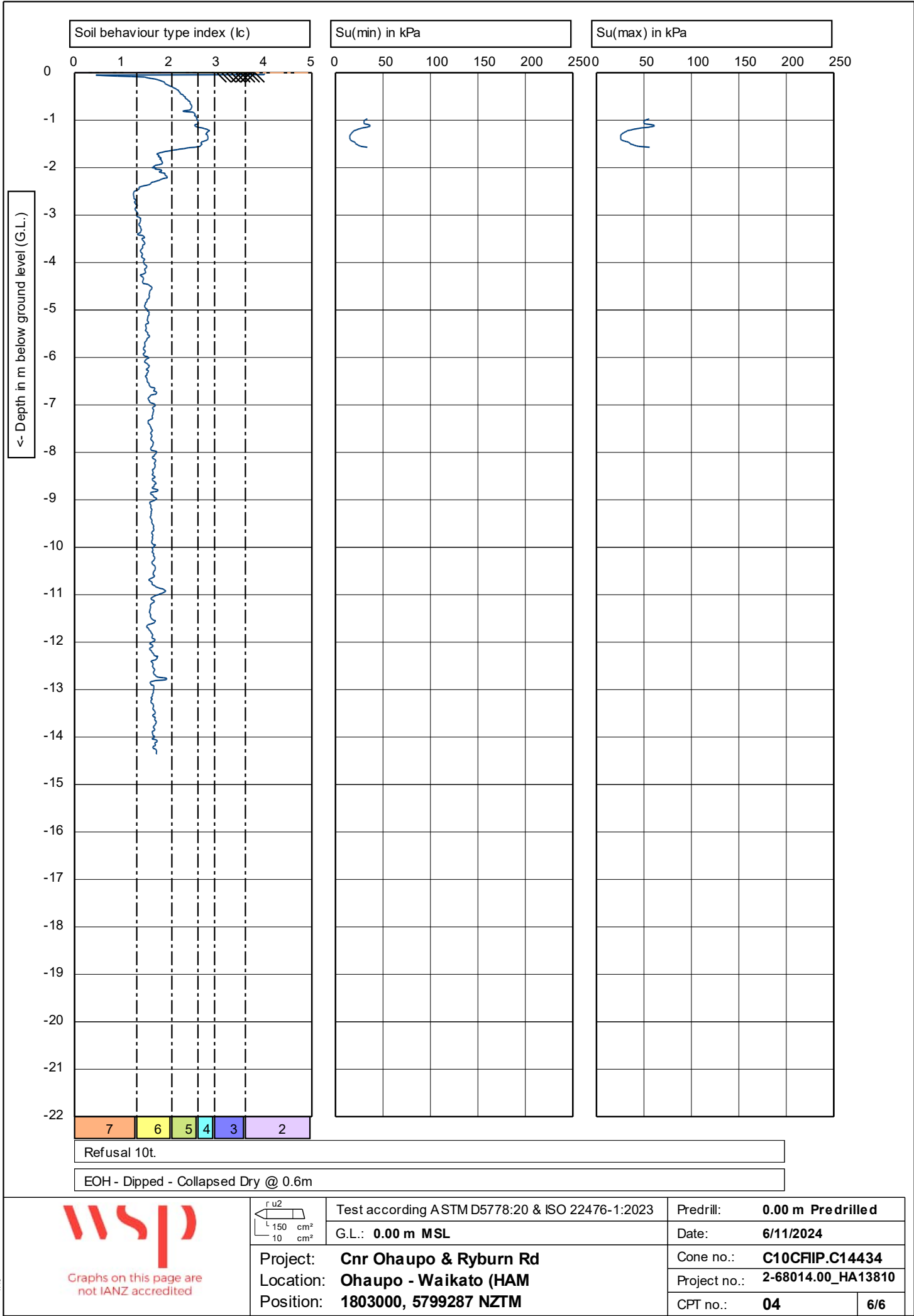
Cone no.: C10CFIIP.C14434

Project no.: 2-68014.00_HA13810

CPT no.: 04

3/6





APPENDIX D

Hand Auger Logs

Client: Ray George
Project: Honey Comb - Ohaupo Plan Change
Site Location: 15 Ryburn Road, Ohaupo
Project No.: HAM2024-0126
Date: 06/11/2024
Borehole Location: Refer to Site Plan



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Sheet 1 of 1

Position: 447672.1mE; 683381.5mN

Logged by: WD

Checked by: MM

Scale: 1:25

Survey Source: Hand-held GPS

Position: 447672.1mE: 683381.5mN Projection: Mt Eden 2000

Datum: Moturiki 1953

Termination Reason: Target depth reached.

Shear Vane No: 2532

DCP No:

17B

Remarks: Groundwater encountered at 2.5m

CMW Geosciences
Great People | Practical Solutions
1:25 Sheet 1 of 1

Sheet 1 of 1

Survey Source: Hand-held GPS

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - HA03

Client: Ray George
Project: Honey Comb - Ohaupo Plan Change
Site Location: 15 Ryburn Road, Ohaupo
Project No.: HAM2024-0126
Date: 06/11/2024
Borehole Location: Refer to Site Plan



CMW Geosciences
Great People | Practical Solutions

Logged by: AN Checked by: MM Scale: 1:25 Sheet 1 of 1

Position: 447815.8mE; 683331.4mN Projection: Mt Eden 2000 Datum: Moturiki 1953 Survey Source: Hand-held GPS

Groundwater	Samples & Insitu Tests		RL (m)	Depth (m)	Graphic Log	Material Description Soil: Soil symbol; soil type; colour; structure; bedding; plasticity; sensitivity; additional comments. (origin/geological unit) Rock: Colour; fabric; rock name; additional comments. (origin/geological unit)	Moisture Condition	Consistency/ Relative Density	Dynamic Cone Penetrometer (Blows/100mm)													
	Depth	Type & Results																				
									5	10	15											
	0.3 0.6 0.9 1.2 1.5 2.0 2.5 3.0 3.5 3.9	Peak = 193kPa Residual = 42kPa		OL: Organic SILT: dark brown. Low plasticity. (TS)	M	VSt																
		CH: CLAY: with some silt. Brown. High plasticity. Sensitive. (Walton Subgroup)		SC: Fine Sandy CLAY: with some silt. Light brown. High plasticity. Moderately sensitive to sensitive. (Walton Subgroup)							St to VSt											
		... at 0.80m, becoming orange brown														W						
		Peak = 146kPa Residual = 36kPa		SC: Fine to coarse Sandy CLAY: with minor fine gravel. Light grey mottled orange brown. High plasticity. Sensitive. (Walton Subgroup)	S	VSt to H																
		Peak = 140kPa Residual = 45kPa		... at 1.50m, becoming light grey mottled orange brown																		
		Peak = 164kPa Residual = 30kPa																				
		Peak = 92kPa Residual = 24kPa																				
		Peak = 104kPa Residual = 18kPa																				
		Peak = 146kPa Residual = 30kPa																				
		Peak = UTP																				
Peak = UTP																						
Peak = 193kPa Residual = 37kPa																						

Termination Reason: Target depth reached.
Shear Vane No: 2087 DCP No:
Remarks: Groundwater encountered at 1.5m

Client: Ray George
Project: Honey Comb - Ohaupo Plan Change
Site Location: 15 Ryburn Road, Ohaupo
Project No.: HAM2024-0126
Date: 06/11/2024
Borehole Location: Refer to Site Plan



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Sheet 1 of 1

Position: 447881.5mE; 683381.0mN Projection: Mt Eden 2000
Datum: Moturiki 1953

Logged by: AN Checked by: MM Scale: 1:25

Survey Source: Hand-held GPS

Termination Reason: Target depth reached.

Shear Vane No: 2087 DCP No:

Remarks: Groundwater not encountered.

Client: Ray George
Project: Honey Comb - Ohaupo Plan Change
Site Location: 15 Ryburn Road, Ohaupo
Project No.: HAM2024-0126
Date: 06/11/2024
Borehole Location: Refer to Site Plan



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Sheet 1 of 1

Position: 447704.8mE; 683163.0mN Projection: Mt Eden 2000
Datum: Moturiki 1953

Logged by: AN

Checked by: MM

Scale: 1:25

Survey Source: Hand-held GPS

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

CMW Geosciences
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1:25 Sheet 1 of 1

Sheet 1 of 1

Survey Source: Hand-held GPS

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

Client: Ray George
Project: Honey Comb - Ohaupo Plan Change
Site Location: 15 Ryburn Road, Ohaupo
Project No.: HAM2024-0126
Date: 06/11/2024
Borehole Location: Refer to Site Plan



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Sheet 1 of 1

Position: 448324.2mE; 683062.4mN

Logged by: AN

Checked by: MM

Scale: 1:25

Survey Source: Hand-held GPS

Position: 448324.2mE: 683062.4mN Projection: Mt Eden 2000

Datum: Moturiki 1953

Termination Reason: Target depth reached.

Shear Vane No: 2087

DCP No:

Remarks: Groundwater not encountered.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

Client: Ray George
Project: Honey Comb - Ohaupo Plan Change
Site Location: 15 Ryburn Road, Ohaupo
Project No.: HAM2024-0126
Date: 06/11/2024
Borehole Location: Refer to Site Plan



Position: 447868.6mE; 683123.7mN	Projection: Mt Eden 2000	
	Datum: Moturiki 1953	Survey Source: Hand-held GPS

Termination Reason: No retrieval.
Shear Vane No: 2532 DCP No: 18B
Remarks: Groundwater encountered at 1.3m

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

Client: Ray George
Project: Honey Comb - Ohaupo Plan Change
Site Location: 15 Ryburn Road, Ohaupo
Project No.: HAM2024-0126
Date: 06/11/2024
Borehole Location: Refer to Site Plan



Great People | Practical Solutions

Sheet 1 of 1

Position: 448052.7mE; 683234.1mN

Logged by: WD

Checked by: MM

Scale: 1:25

Projection: Mt Eden 2000

Datum: Moturiki 1953

Survey Source: Hand-held GPS

Termination Reason: Target depth reached.

Shear Vane No: 2532

DCP No:

Remarks: Groundwater not encountered.

Client: Ray George
Project: Honey Comb - Ohaupo Plan Change
Site Location: 15 Ryburn Road, Ohaupo
Project No.: HAM2024-0126
Date: 06/11/2024



Sheet 1 of 1

Survey Source: Hand-held GPS

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

Client: Ray George
Project: Honey Comb - Ohaupo Plan Change
Site Location: 15 Ryburn Road, Ohaupo
Project No.: HAM2024-0126
Date: 06/11/2024



Sheet 1 of 1

Survey Source: Hand-held GPS

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

APPENDIX E

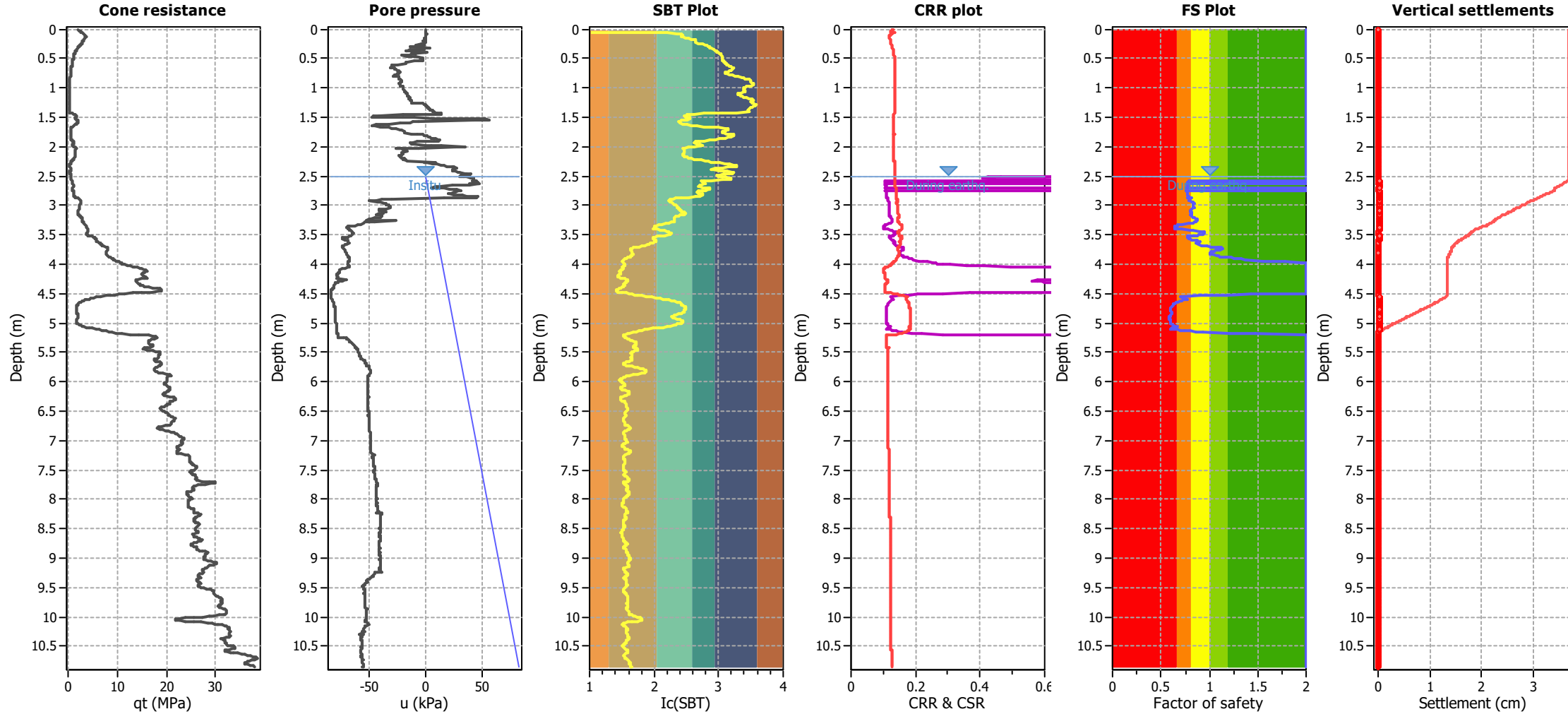
Liquefaction Results

Project:

Location:

CPT: CPT01

Total depth: 10.85 m



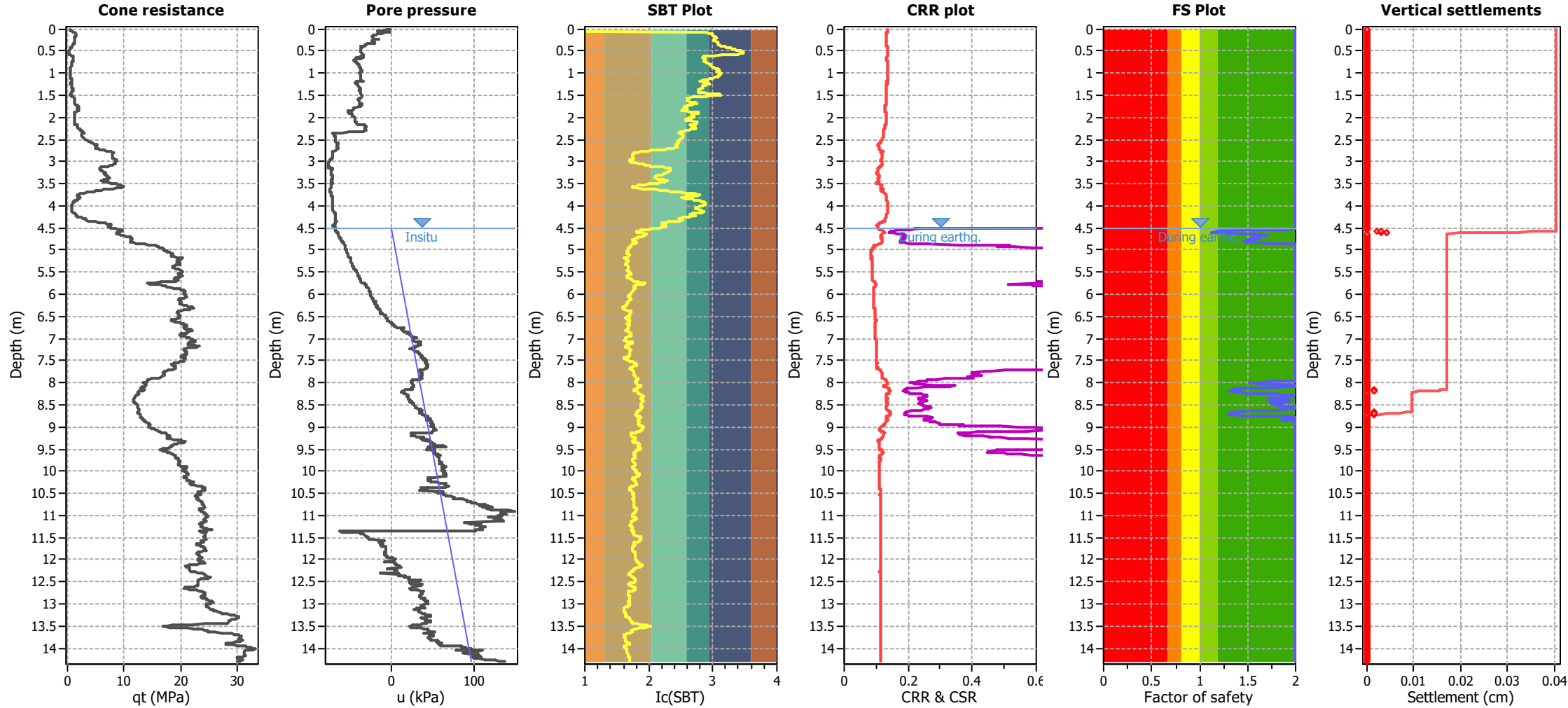
Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior	
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	applied:	.
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	0.25	Unit weight calculation:	Based on SBT	K_0 applied:	Yes	MSF method:	Method based

Project:

Location:

CPT: CPT02

Total depth: 14.30 m



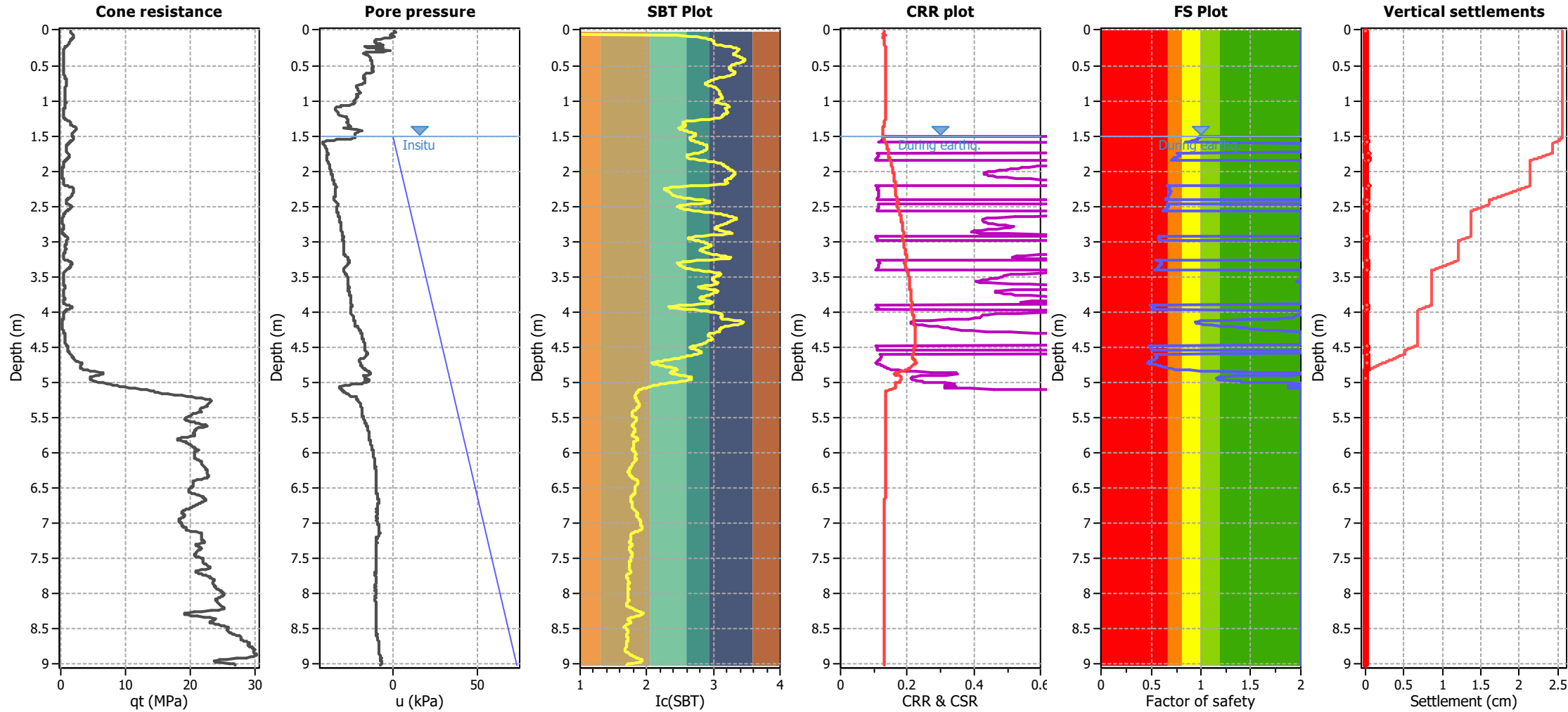
Analysis method:	B&I (2014)	G.W.T. (in-situ):	4.50 m	Use fill:	No	Clay like behavior	
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	4.50 m	Fill height:	N/A	applied:	.
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	0.25	Unit weight calculation:	Based on SBT	K_0 applied:	Yes	MSF method:	Method based

Project:

Location:

CPT: CPT03

Total depth: 9.02 m



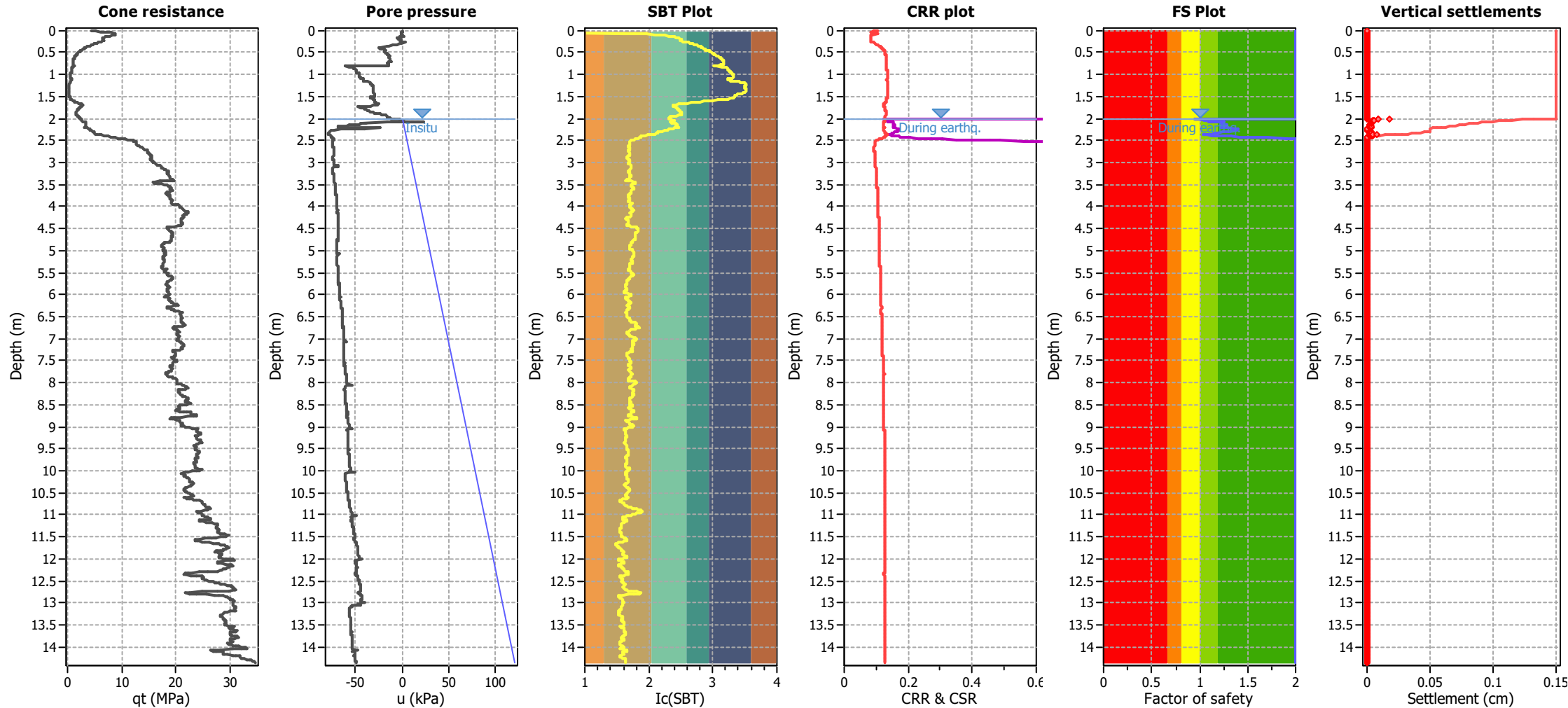
Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior	
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	applied:	.
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	0.25	Unit weight calculation:	Based on SBT	K_0 applied:	Yes	MSF method:	Method based

Project:

Location:

CPT: CPT04

Total depth: 14.36 m



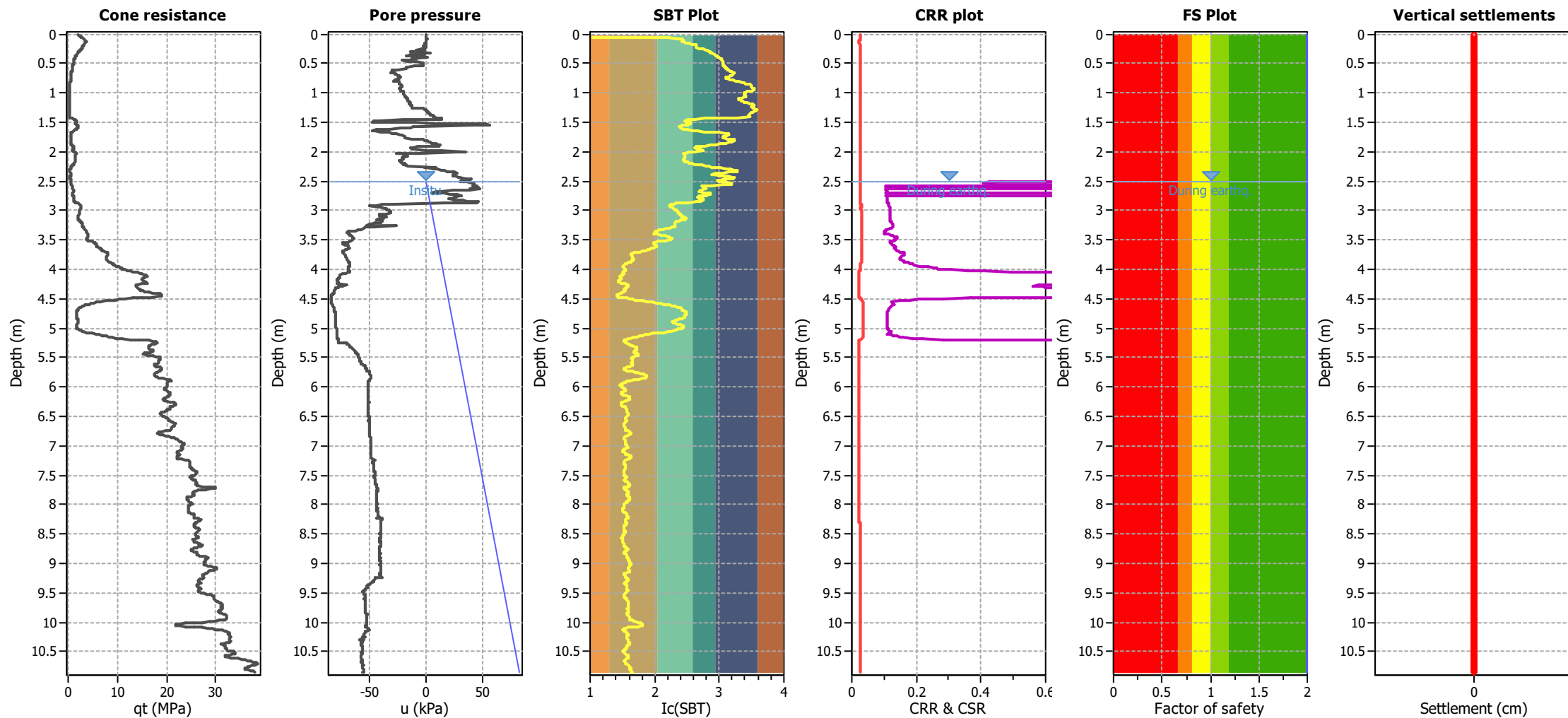
Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Use fill:	No	Clay like behavior	
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Fill height:	N/A	applied:	.
Points to test:	Based on I_c value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	5.90	I_c cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	0.25	Unit weight calculation:	Based on SBT	K_0 applied:	Yes	MSF method:	Method based

Project:

Location:

CPT: CPT01

Total depth: 10.85 m



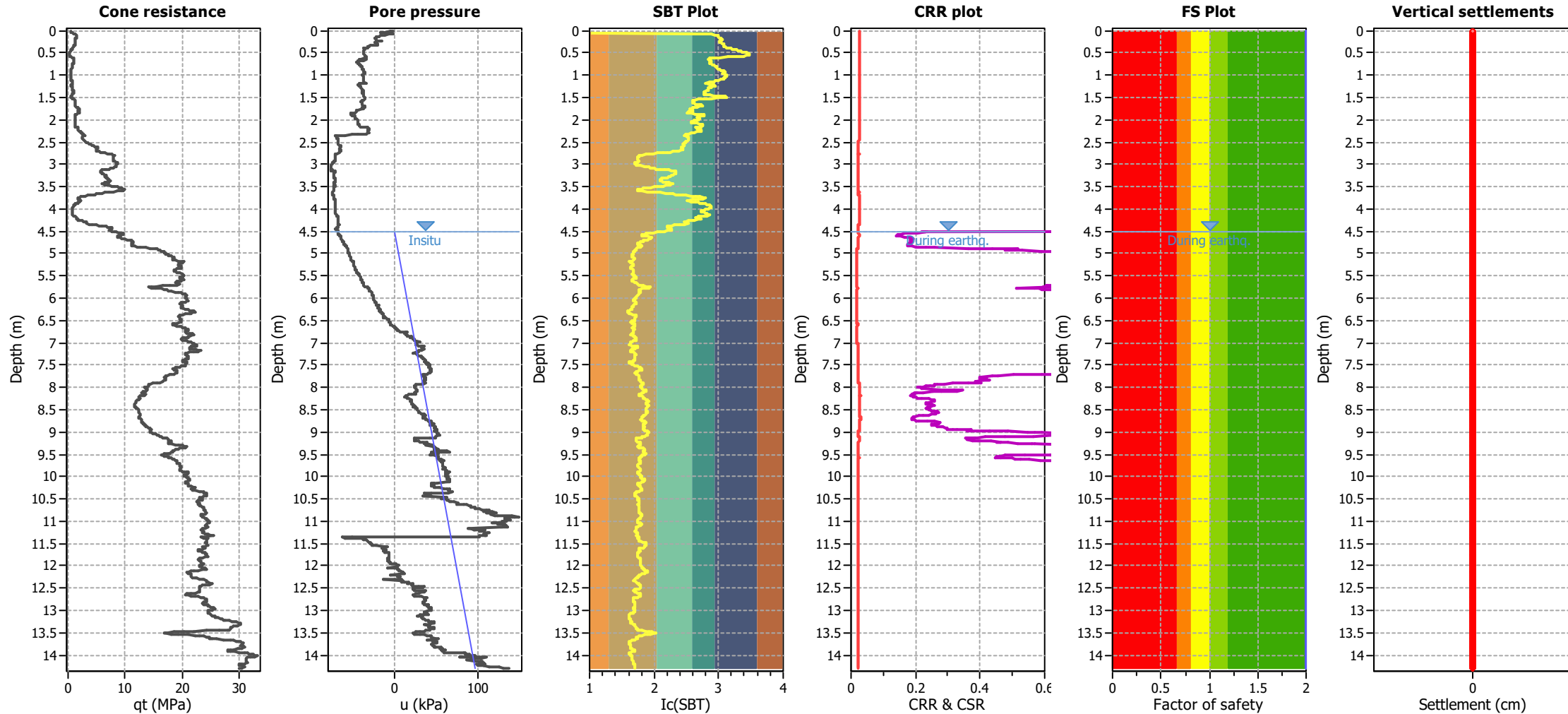
Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w :	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.05	Unit weight calculation:	Based on SBT	K_0 applied:	Yes		

Project:

Location:

CPT: CPT02

Total depth: 14.30 m



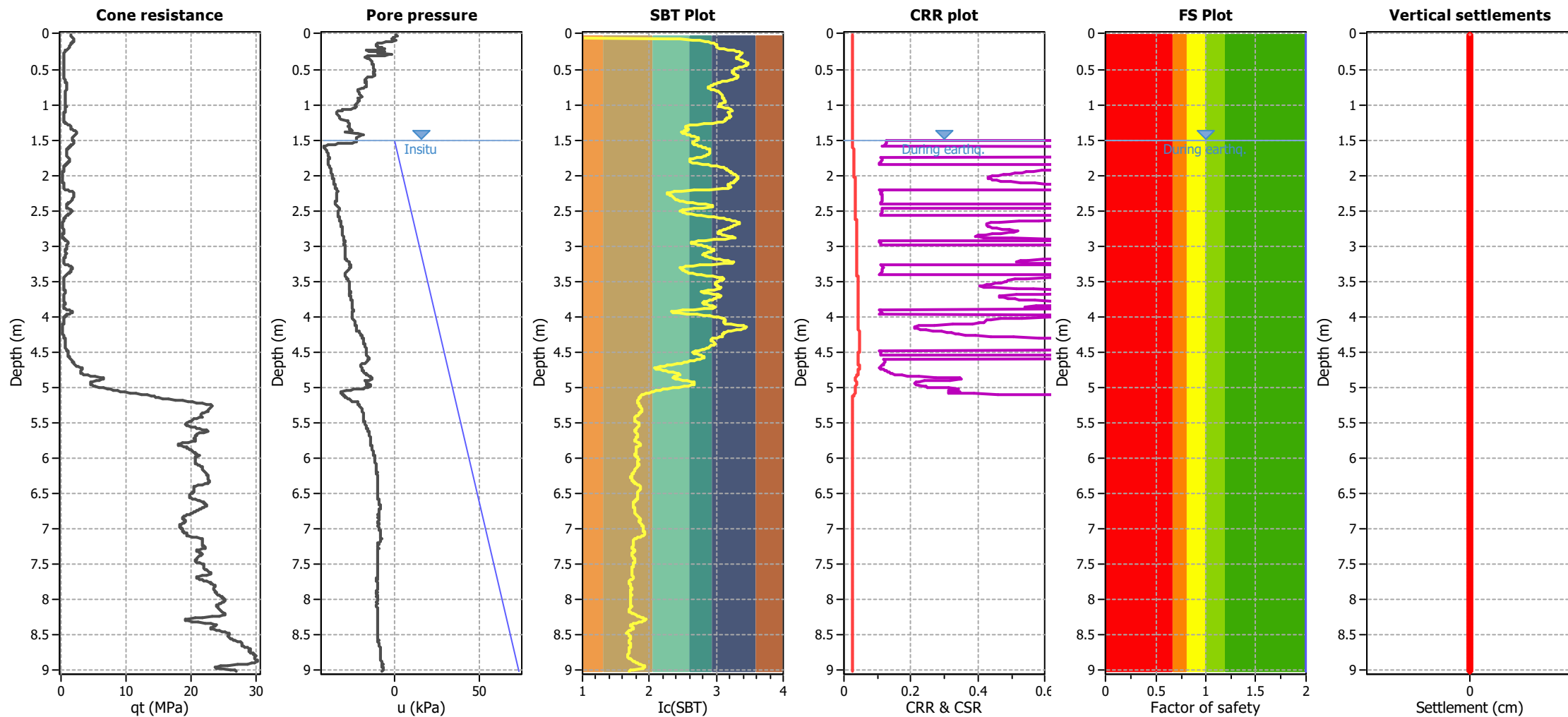
Analysis method:	B&I (2014)	G.W.T. (in-situ):	4.50 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	4.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w :	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.05	Unit weight calculation:	Based on SBT	K_0 applied:	Yes		

Project:

Location:

CPT: CPT03

Total depth: 9.02 m



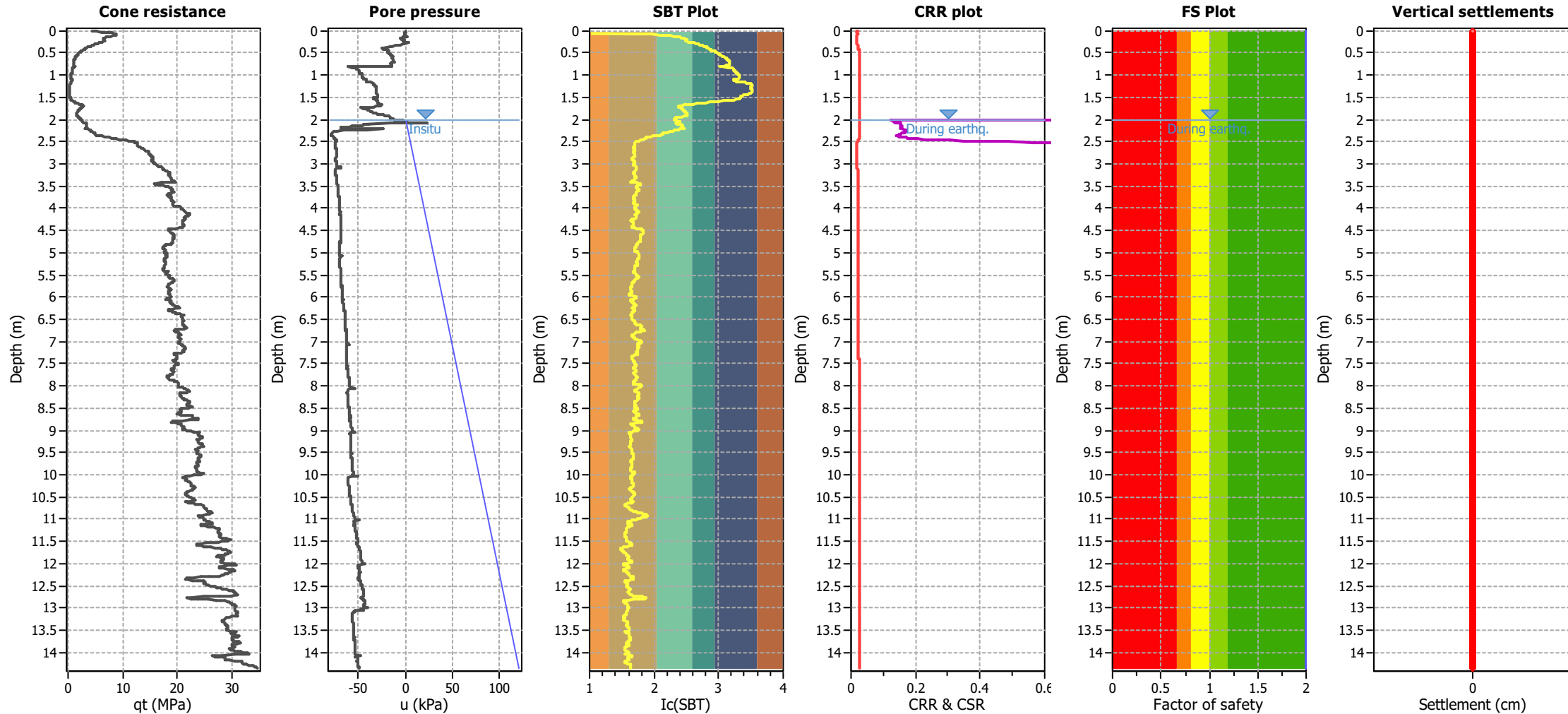
Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w :	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.05	Unit weight calculation:	Based on SBT	K_0 applied:	Yes		

Project:

Location:

CPT: CPT04

Total depth: 14.36 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m
Points to test:	Based on Ic value	Average results interval:	3
Earthquake magnitude M_w :	5.90	Ic cut-off value:	2.60
Peak ground acceleration:	0.05	Unit weight calculation:	Based on SBT

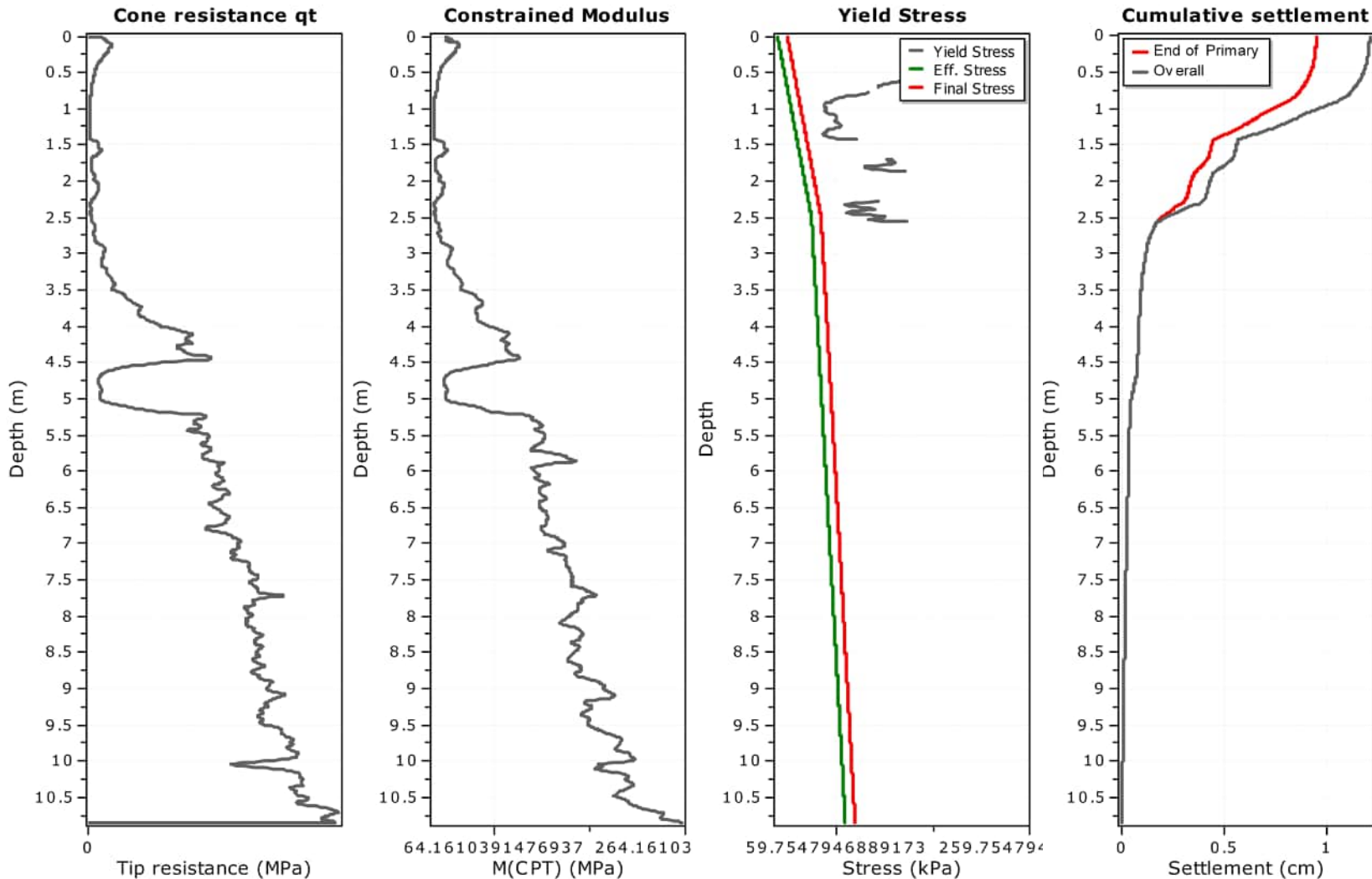
Use fill:	No
Fill height:	N/A
Fill weight:	N/A
Trans. detect. applied:	No
K_0 applied:	Yes

Clay like behavior applied:	.
Limit depth applied:	No
Limit depth:	N/A
MSF method:	Method based

APPENDIX F

Settlement Results

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 100.00 (m)
L/B: 1.0
Footing pressure: 10.00 (kPa)
Embedment depth: 0.00 (m)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: No
Calculate secondary settlements: Yes
Time period for primary consolidation: 6 months
Time period for second. settlements: 600 months

* Primary settlement calculation is performed according to the following formula:

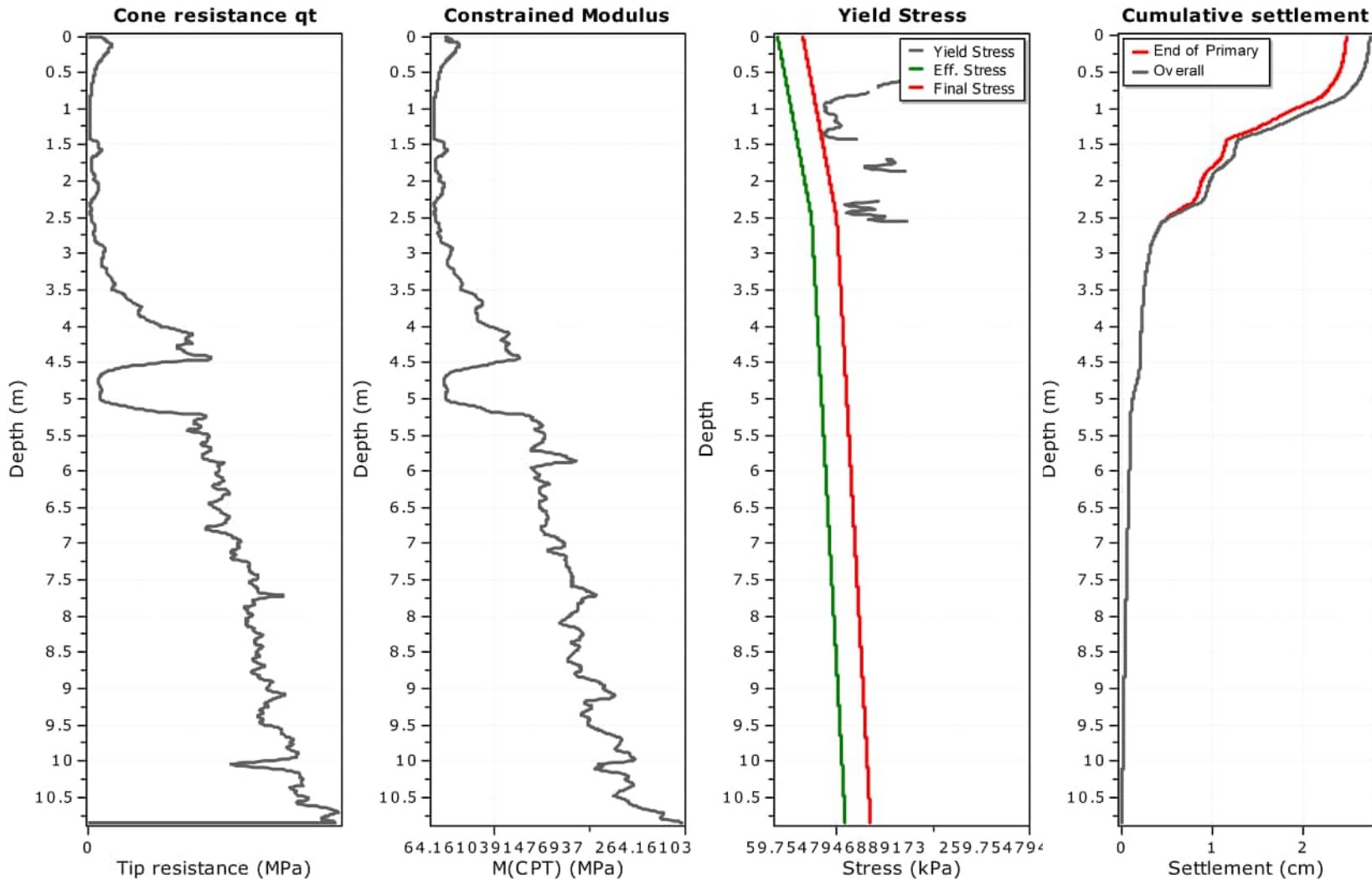
$$S = \sum \frac{\Delta \sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S_c = S_p \left(1 - e^{-\frac{t}{t_p}} \right)$$

where t_p is the duration of primary consolidation

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 100.00 (m)
L/B: 1.0
Footing pressure: 26.00 (kPa)
Embedment depth: 0.00 (m)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: No
Calculate secondary settlements: Yes
Time period for primary consolidation: 6 months
Time period for second. settlements: 600 months

* Primary settlement calculation is performed according to the following formula:

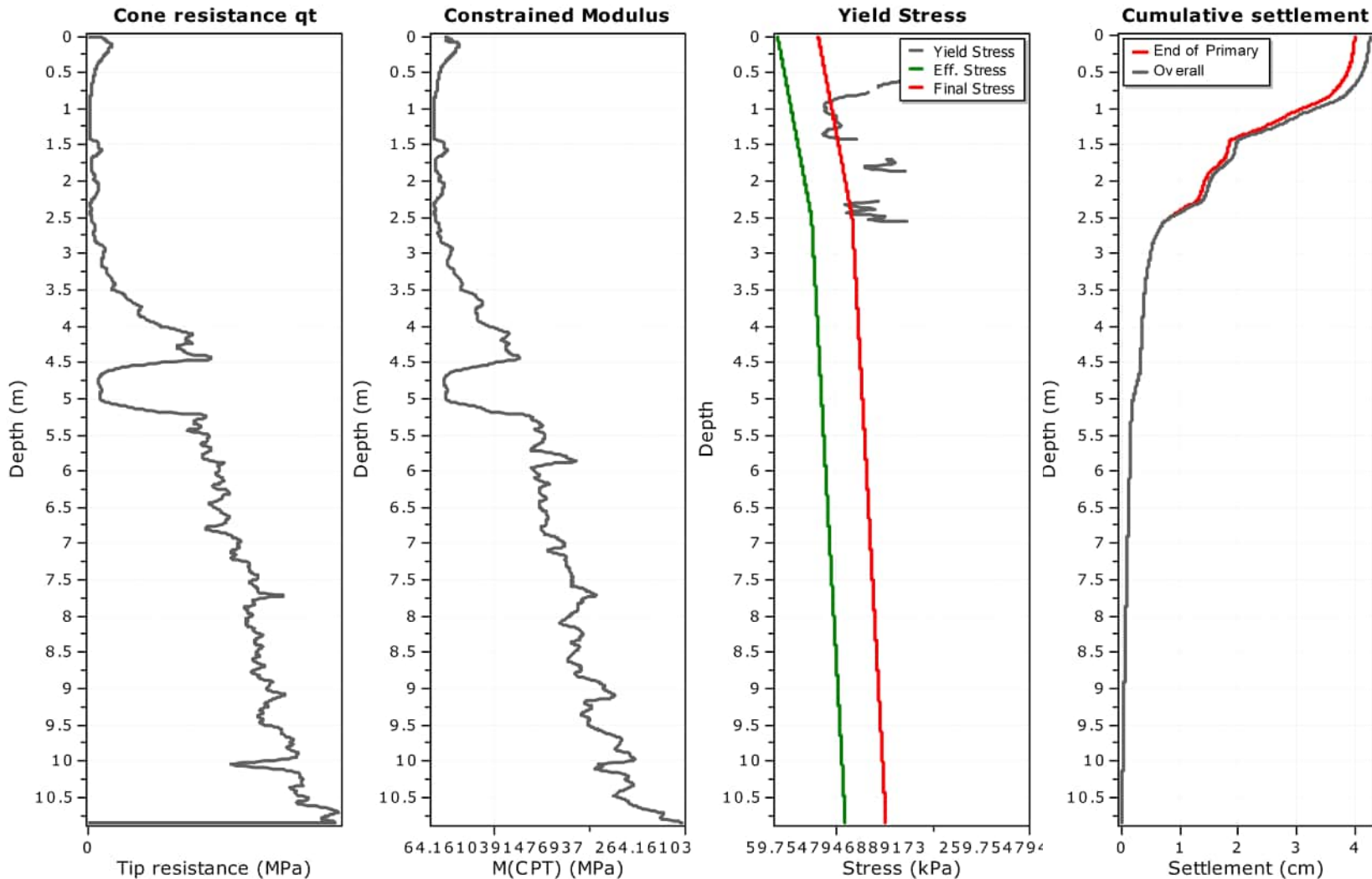
$$S = \sum \frac{\Delta \sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S_c = S_p \left(1 - e^{-\frac{t}{t_p}} \right)$$

where t_p is the duration of primary consolidation

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 100.00 (m)
L/B: 1.0
Footing pressure: 42.00 (kPa)
Embedment depth: 0.00 (m)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: No
Calculate secondary settlements: Yes
Time period for primary consolidation: 6 months
Time period for second. settlements: 600 months

* Primary settlement calculation is performed according to the following formula:

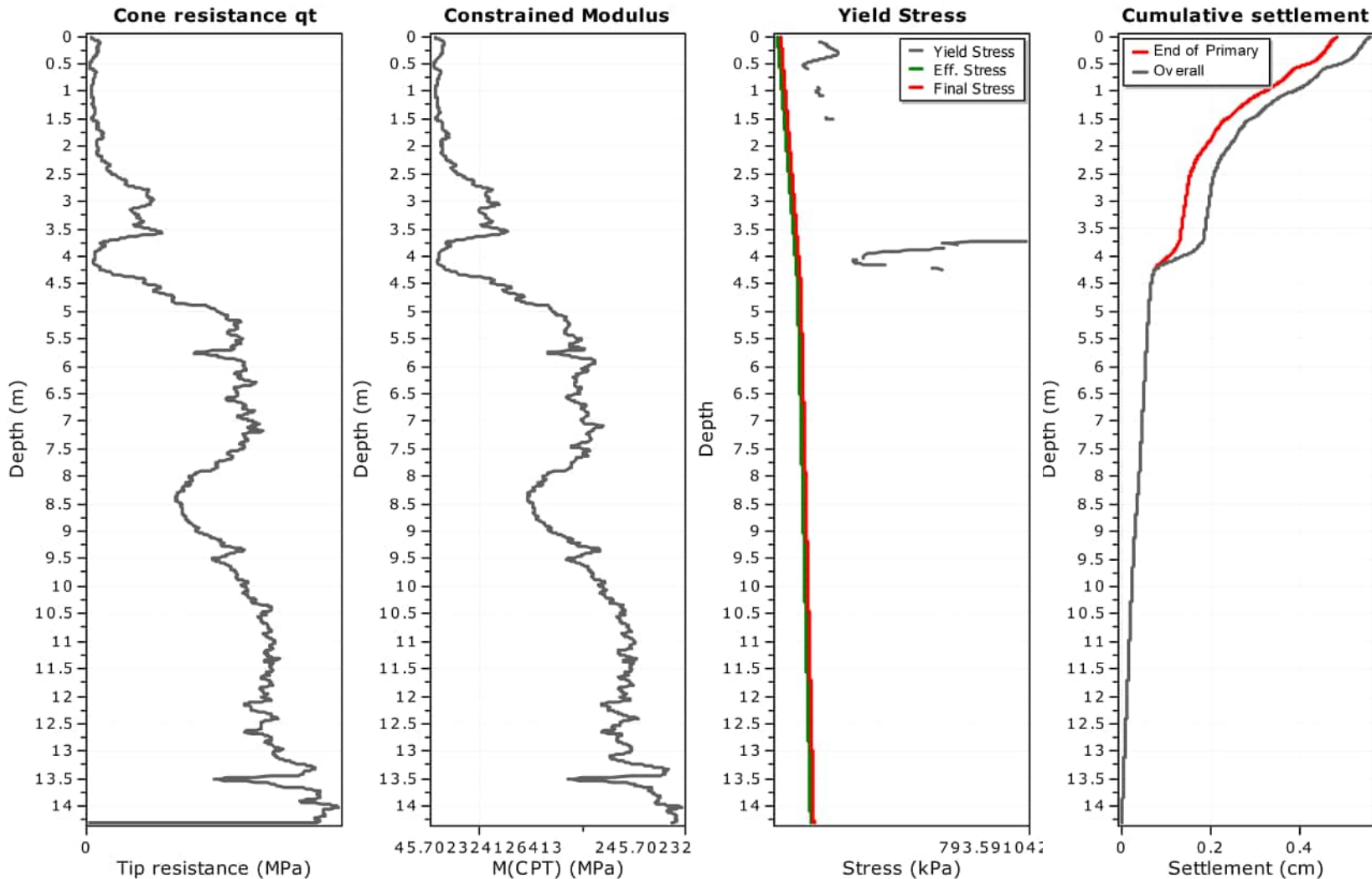
$$S = \sum \frac{\Delta \sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S_c = S_p \left(1 - e^{-\frac{t}{t_p}} \right)$$

where t_p is the duration of primary consolidation

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 100.00 (m)
L/B: 1.0
Footing pressure: 10.00 (kPa)
Embedment depth: 0.00 (m)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: No
Calculate secondary settlements: Yes
Time period for primary consolidation: 6 months
Time period for second. settlements: 600 months

* Primary settlement calculation is performed according to the following formula:

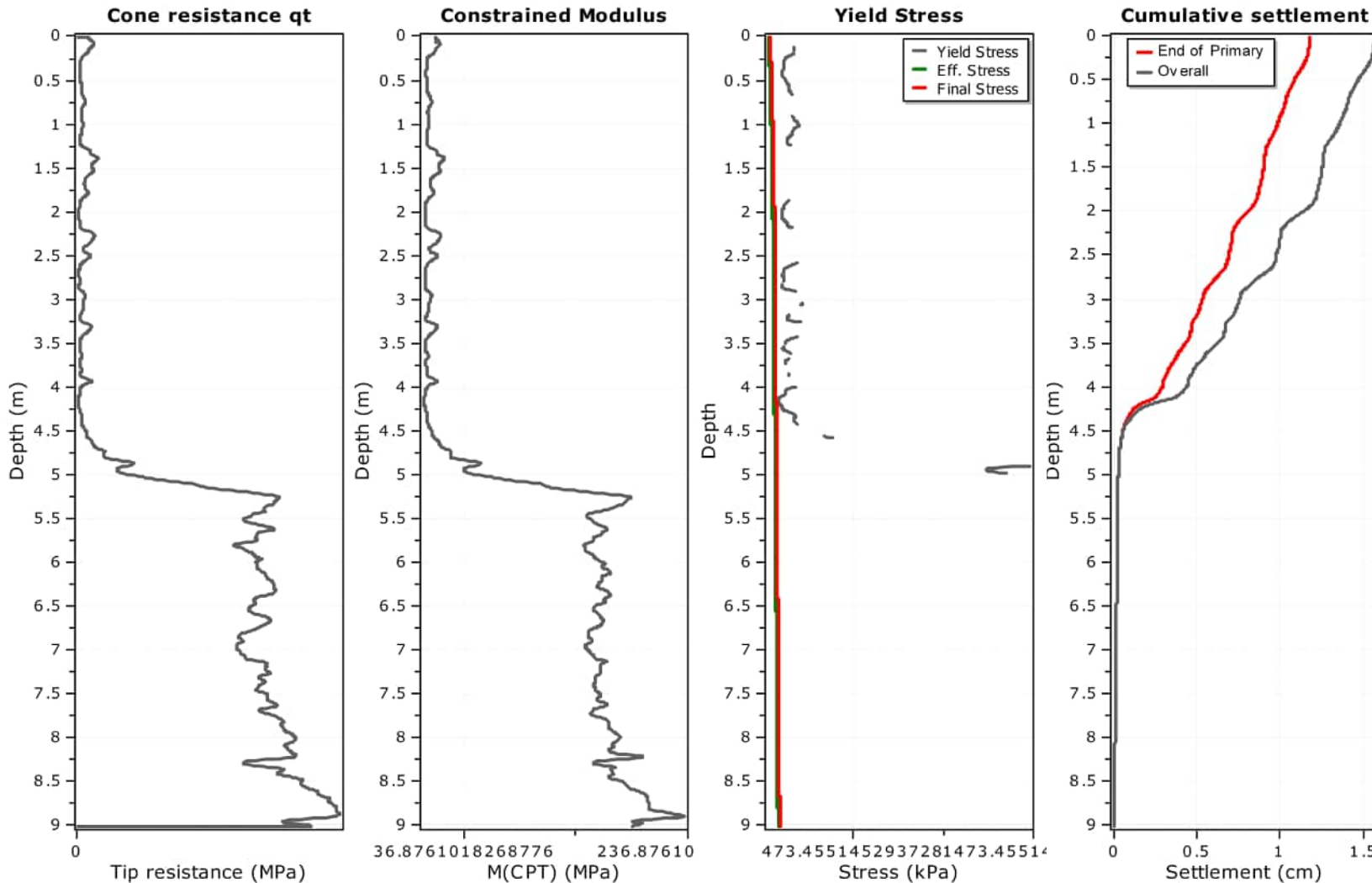
$$S = \sum \frac{\Delta \sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S_c = S_p \left(\frac{t}{t_p} \right)^n$$

where t_p is the duration of primary consolidation

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 100.00 (m)
L/B: 1.0
Footing pressure: 10.00 (kPa)
Embedment depth: 0.00 (m)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: No
Calculate secondary settlements: Yes
Time period for primary consolidation: 6 months
Time period for second. settlements: 600 months

* Primary settlement calculation is performed according to the following formula:

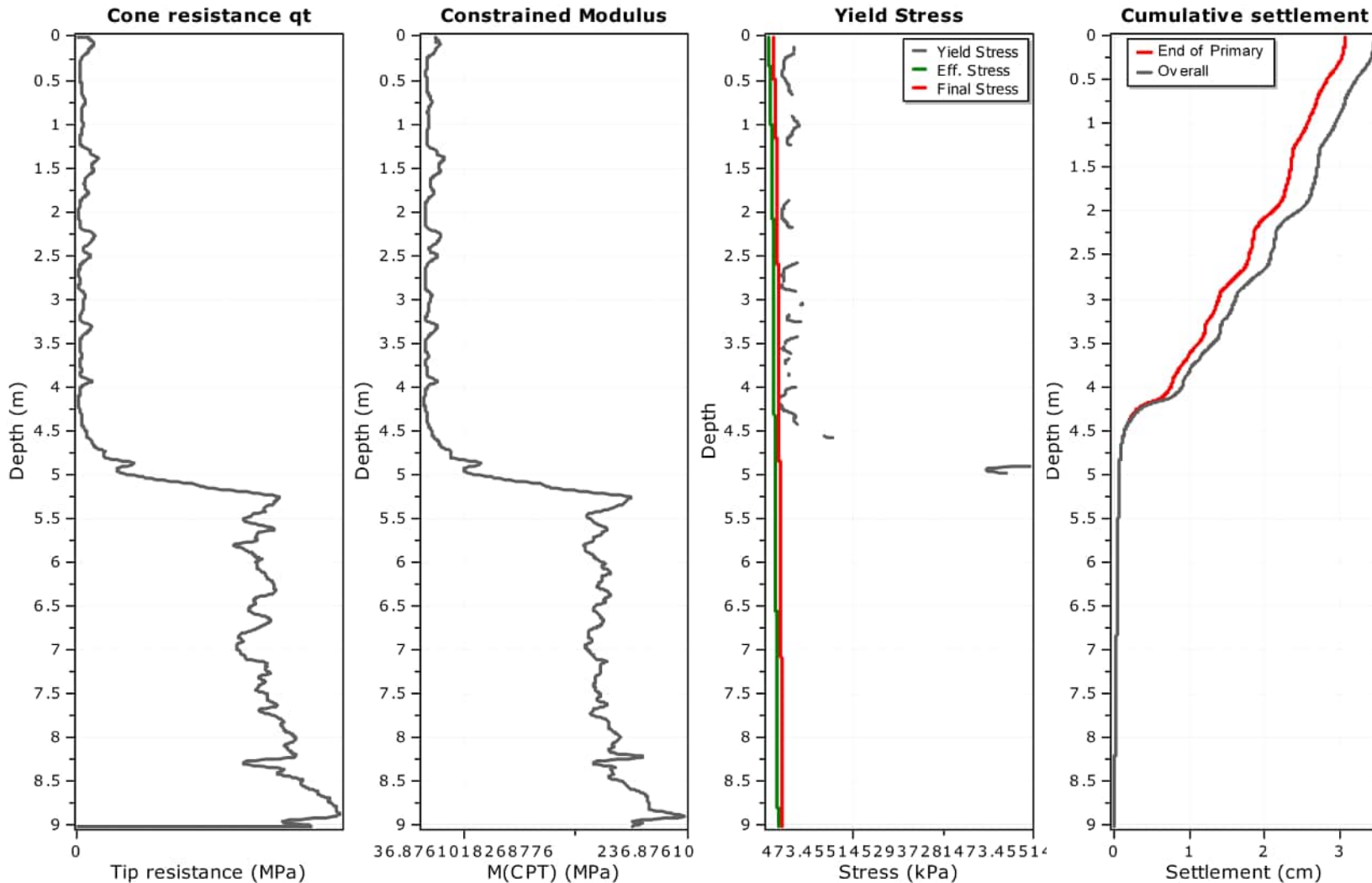
$$S = \sum \frac{\Delta \sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S_c = S_p \left(1 - e^{-\frac{t}{t_p}} \right)$$

where t_p is the duration of primary consolidation

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 100.00 (m)
L/B: 1.0
Footing pressure: 26.00 (kPa)
Embedment depth: 0.00 (m)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: No
Calculate secondary settlements: Yes
Time period for primary consolidation: 6 months
Time period for second. settlements: 600 months

* Primary settlement calculation is performed according to the following formula:

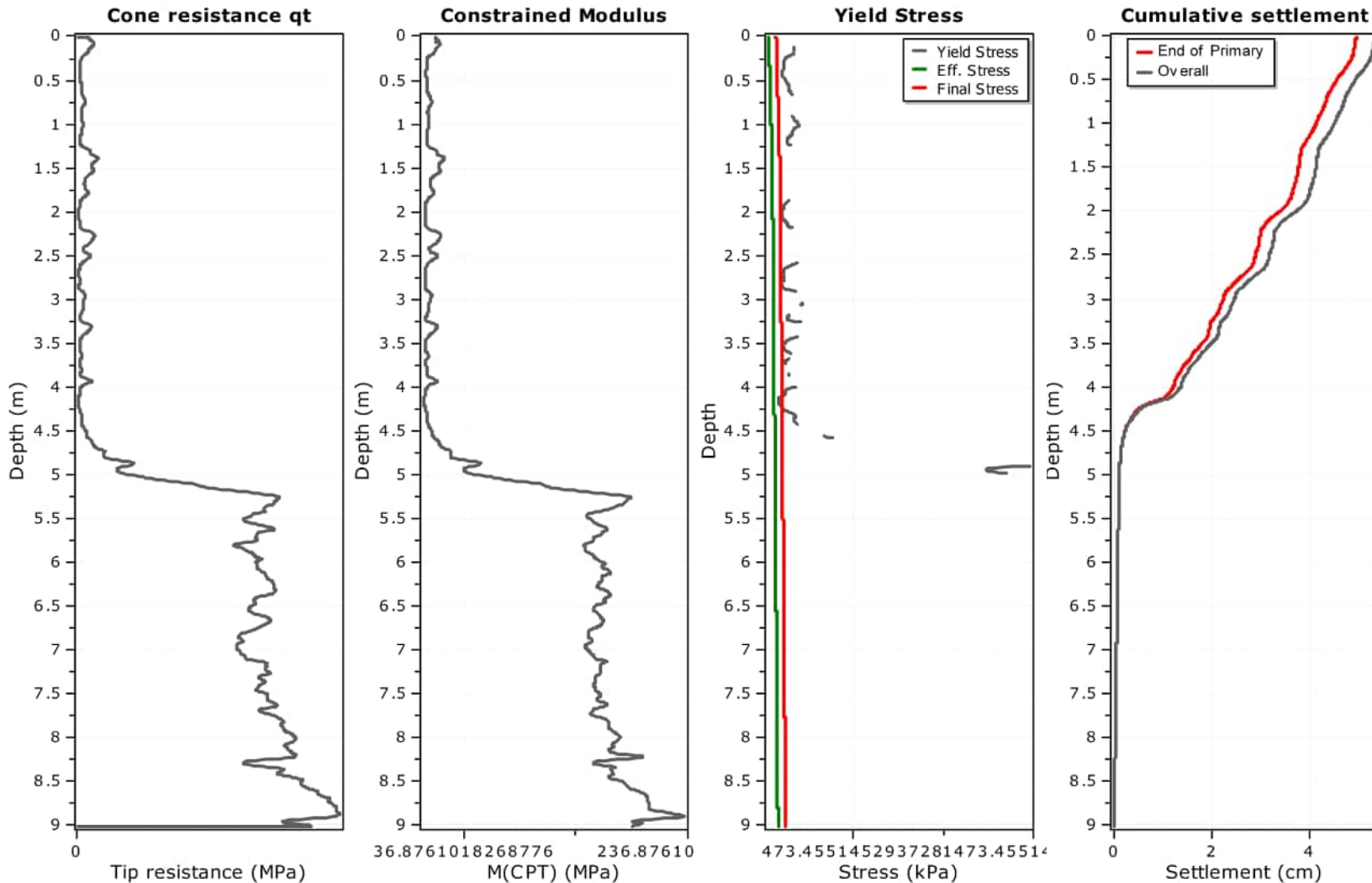
$$S = \sum \frac{\Delta \sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S_c = S_p \left(1 - e^{-\frac{t}{t_p}} \right)$$

where t_p is the duration of primary consolidation

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 100.00 (m)
L/B: 1.0
Footing pressure: 42.00 (kPa)
Embedment depth: 0.00 (m)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: No
Calculate secondary settlements: Yes
Time period for primary consolidation: 6 months
Time period for second. settlements: 600 months

* Primary settlement calculation is performed according to the following formula:

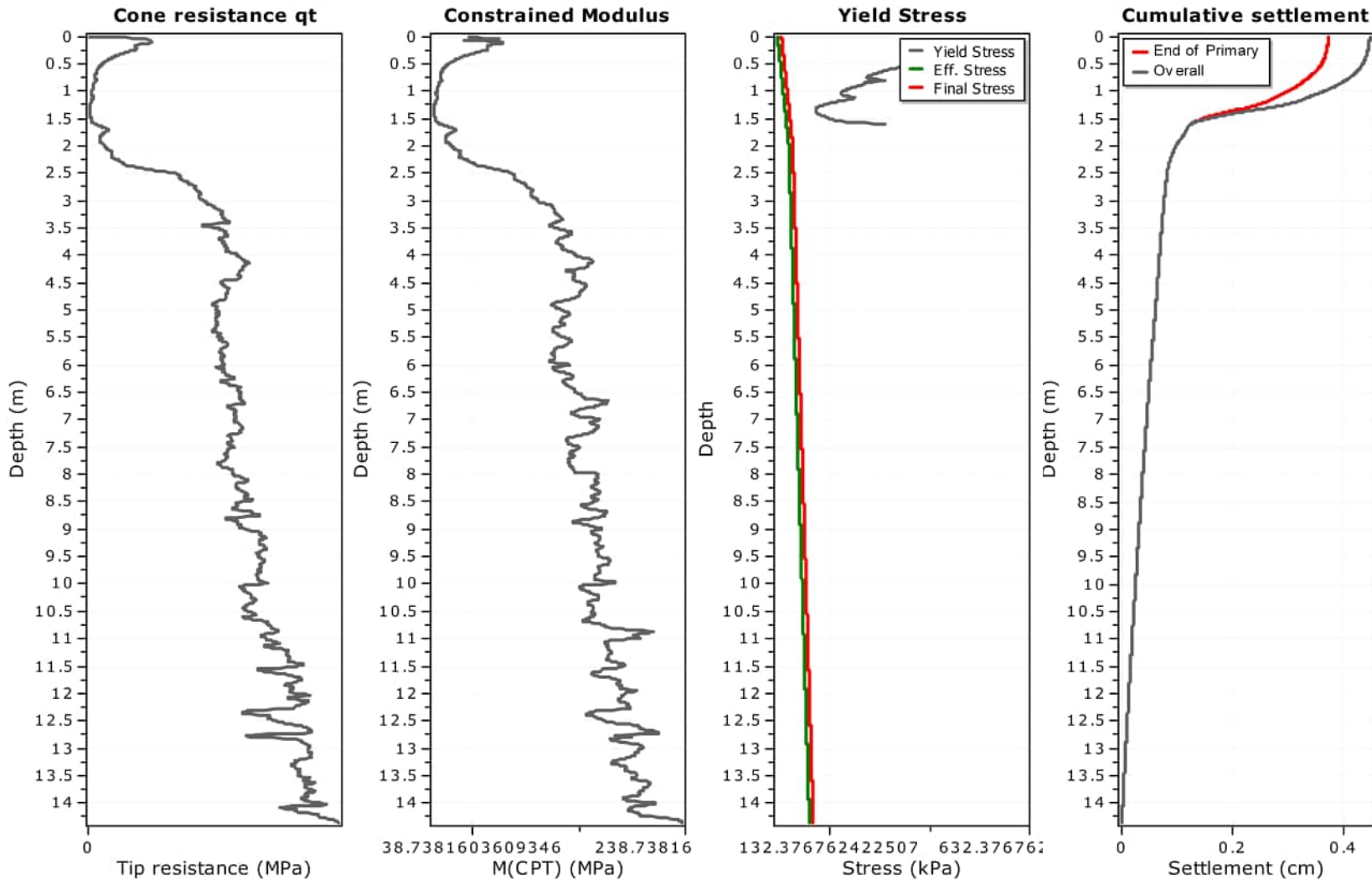
$$S = \sum \frac{\Delta \sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S_c = S_p \left(\frac{t}{t_p} \right)^n$$

where t_p is the duration of primary consolidation

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Rectangular
Footing width: 100.00 (m)
L/B: 1.0
Footing pressure: 10.00 (kPa)
Embedment depth: 0.00 (m)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: No
Calculate secondary settlements: Yes
Time period for primary consolidation: 6 months
Time period for second. settlements: 600 months

* Primary settlement calculation is performed according to the following formula:

$$S = \sum \frac{\Delta \sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S_{secondary} = S_{primary} \left(1 - e^{-\frac{t}{t_p}} \right)$$

where t_p is the duration of primary consolidation

APPENDIX G

Historical Photographs

1971



2014

Northern Pond Construction

Legend

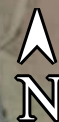
Boundary

Google Earth

Image © 2024 Maxar Technologies

Image © 2024 CNES / Airbus

300 m



Ryburn Rd

Ryburn Rd

Ryburn Rd

3

New Zealand State

2017

Eastern Pond Construction

Legend

Boundary

Ryburn Rd

Ryburn Rd

Ryburn Rd

3

N

Google Earth

Image © 2024 CNES / Airbus

300 m

New Zealand State



2018

Eastern Pond Extension

Legend

Boundary

Ryburn Rd

Ryburn Rd

Ryburn Rd

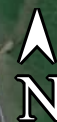
3

New Zealand State

Google Earth

Image © 2024 Maxar Technologies

300 m



2020

Northern Pond Extension

Legend

 Boundary

Ryburn Rd

Ryburn Rd

Ryburn Rd

3



Google Earth

Image © 2024 Maxar Technologies

300 m

New Zealand State



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