



Geotechnical Assessment Report

Whare Manaaki Mauao Project

MAUAO TRUST
1 ADAMS AVENUE, MOUNT MAUNGANUI

9 DECEMBER 2025



Quality Information

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Report Approval Details				
Client	Mauao Trust			
Report Title	Geotechnical Assessment Report – Whare Manaaki Mauao Project			
Filename	678164-GEO-R001-GAR	Revision	0	
	Prepared By	Position	Signature	Date
Author	s 7(2)(a) ... Privacy	Engineering Geologist	s 7(2)(a) ... Privacy	04.12.2025
Geotechnical Review	s 7(2)(a) ... Privacy	Senior Geotechnical Engineer	s 7(2)(a) ... Privacy	08.12.2025

Document Control					
Revision	Description	Date	Author	Reviewer	Approver
0	Final	09.12.2025	s 7(2)(a) – Privacy		

Distribution				
Revision	No. of Copies	Format	Distributed to	Date
0	1	PDF	Mauao Trust	09.12.2025

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1 Introduction

Stratum Consultants Ltd (Stratum) has been engaged by Mauao Trust to carry out a geotechnical assessment for the proposed Whare Manaaki Mauao buildings at 1 Adams Avenue, Mount Maunganui (the site).

The purpose of this report is to assess the geotechnical suitability of the site and provide geotechnical recommendations for the proposed development.

This report is part of the documentation to be submitted to the Tauranga City Council (TCC) to support the resource and building consent applications for the proposed development.

2 Proposed Development

The Whare Manaaki Mauao development involves locating two prefabricated buildings to the site, as indicated in Figures 1 and 2 below. The proposed buildings are light-weight single-storey buildings supported on a piled foundation with a floor area of 40 m² and 63 m². A separate bathroom is to be located adjacent to one of the buildings and a deck joins the buildings and extends to the south-east of the buildings with stairs to the lower terrace. Both the buildings and deck will have a floor level of RL 7.50 m NZVD. Plans of the proposed buildings by Pod Life are included in Appendix A along with a proposed layout plan and sections by Stratum (678164-CIV-D001).

Minor earthworks are proposed as part of the development to achieve the required ground clearance below the buildings. The ground will be cut up to 0.5 m to achieve the finished ground level of RL 6.90 m NZVD.

TCC reticulated sewer is available at the site, and we understand the stormwater is to be connected into the existing system via an attenuation tank.

3 Site Description

3.1 General

The site is located at 1 Adams Avenue, Mount Maunganui and is partially within the parcel legally described as Lot 1 DP 429354 and partially within the Pilot Quay road reserve.

The site is located at the base of Mauao, on the south-eastern flank. Pilot Quay is located to the south of the site, and the Mount Maunganui Beachside Holiday Park is located to the north-east. The site was historically used as part of the Holiday Park and the landform in the area of the proposed buildings is currently defined by terraced camping platforms. Within the area of the proposed buildings, the terrace is at an elevation of approximately RL 7.0 m to RL 8.0 m New

Zealand Vertical Datum (NZVD). The site is generally grassed with some gravel and asphalt. Some large trees are located to the south-west of the site and a cabin is located on the terrace below the site to the south-east.

A steep bank is located to the south-east of the site, up to approximately 2.5 m high, and to the south a stone facing is located below the site, above Pilot Quay, up to 2.5 m high, tapering down to the west. Pilot Bay (Tauranga Harbour) is located approximately 30 m to the south of the site, with a bank up to approximately 5 m located below Pilot Quay.

To the north-west a steep bank is located above the site, up to approximately 2.5 m high, with the Mount Base Track located above the bank. Mauao is located to the north-west of the site, with steep slopes up to the summit at approximately RL 230 m NZVD.

A site overview is provided in Figure 1, and a location plan is provided in Figure 2 below. A site photo is displayed in Figure 3 below.

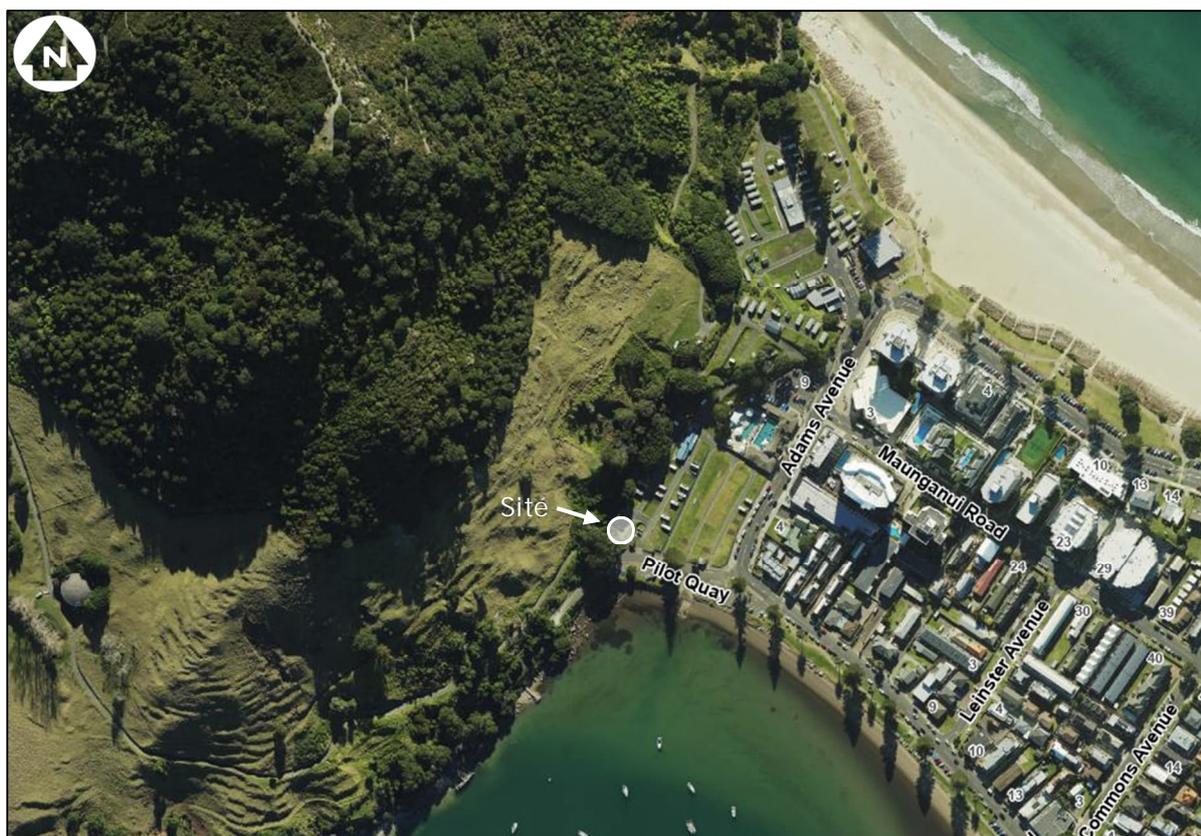


Figure 1 | Site overview showing the subject site location (TCC Mapi)¹.

¹ Tauranga City Council. Mapi. Retrieved from: <https://mapi.tauranga.govt.nz/Html5/index.html?viewer=Mapi>



Figure 2 | Site plan showing the approximate location of the proposed buildings. The topography of the site is shown by 1.0 m contours (TCC Mapi).

3.2 Site Observations

A walkover of the site and the slope to the north-east was completed by Stratum on the 29th of October 2025.

The following observations were noted:

- A number of near vertical (>70 degrees) slopes are located within the area which generally appear to be stable. Minor shallow erosion was evident on the slopes.
- The head scarps of relic slips were evident from the topography of the slopes above the site to the north and west.
- A spring is present to the north of the site, and seepage and boggy ground was evident near the middle of the historic landslide feature to the north, draining down to the base of the slope. Lush vegetation was evident in this area. The spring drains into a catchpit to the north of the site.
- There were no signs of seepage on the slopes directly above or below the terrace in the area of the proposed development.
- Boulders are evident on the slopes above the site. Boulders are also evident within the area of the proposed buildings, some of which have been placed (as seen in Figure 3 below).

Boulders are also evident in exposures on the terrace slopes above and below the site within a silty/sandy matrix, inferred to be colluvium.

- No unstable rocks were observed on or below the grassed slopes above the subject site during our site walkover.
- The slopes above the site are hummocky with some creep type movement exaggerated by stock movement. Some anthropogenic terraces are evident on the slopes above the site.
- On the slope below the terrace within the area of the proposed buildings, the slope is not uniform with some minor erosion.



Figure 3 | Site photo showing the platform area of the proposed buildings with terrace slopes above and below (note boulders have been placed). Photo taken facing north-east.

4 Desktop Information

4.1 Mapped Information

The Tauranga City Council Mapi identifies a number of constraints and hazards on or near the site;

- Archaeology sites are identified in the area;
- A number of public utilities are mapped within the area (shown on layout plan included in Appendix A);
- “Estuarine Deposits” are identified to the north-east of the site;
- Specific Design Foundations are required;
- Geomorphology is mapped as “Volcanic Hills and Ranges” on the western side and “Fixed Foredures” on the east;

- Coastal Erosion mapped on the southern margin and to the south of the site;
- Harbour inundation is mapped along the shore to the south and along Adams Avenue to the east of the site;
- An overland flow path is mapped through the area of the proposed buildings (shown on layout plan included in Appendix A) and flooding is identified on the low-lying areas to the south and east;
- Tsunami evacuation zone is mapped within the area;
- No mapped slope hazards are identified at the site; and,
- The liquefaction vulnerability at the site is mapped as 'low' on the western part of the site and 'possible' on the east.

The liquefaction risk, potential for static settlements and risk of slope instability is considered in the following assessment. The remaining items are outside the scope of this report.

The New Zealand Geotechnical Database (NZGD)² has been reviewed and some test pits, CPTs and hand augers are located in the campground area to the north-east of the site. The testing generally shows some beach sands and colluvium over deeper sand material, generally correlating to the ground conditions encountered in the investigation. The test results are generally associated with the reports outlined in Section 4.4 below and have been considered in a broader context, however have not specifically been used in the following assessment.

The Tauranga City Groundwater map³ has been reviewed which shows the groundwater in the area of the proposed buildings is expected to be around 5 m depth.

4.2 Historical Imagery

Historical aerial photographs dating back to the 1940s^{4,5} have been reviewed. A walkway was located around the area of the proposed development, now the Mount Base Track, with a number of trees located in the area of the proposed development. The area to the north-east was used as sports fields, which we understand was associated with the Holiday Park. The residential settlement and road network in the area was evident in the early 1940s. The landslide features to the north of the site are evident in the photograph from 1943 with trees in the area. Refer to Figure 4 below. The area developed over time with the rows of the campsites and access tracks evident in photographs from 1977 (refer to Figure 5) and the terrace in the area of the proposed development evident in the images from 1997 along with the construction of the hot pools to the north-east, and some cabins evident in the area in 2003 (refer to Figure 6 below). The area of the proposed buildings and

² New Zealand Geotechnical Database. Retrieved from <https://nzgd.org.nz/tenant/295/hierarchy/3563/level/1823/tag/Map>

³ Tauranga City Council. Groundwater (2022). Retrieved from <https://gis.tauranga.govt.nz/portal/apps/experiencebuilder/experience/?id=eaf07d993750457ea733b263ce5ecc3e>

⁴ Tauranga City Council. Mapi. Retrieved from: <https://mapi.tauranga.govt.nz/Html5/index.html?viewer=Mapi>

⁵ Retrolens. Historical Image Resource. Retrieved from <https://retrolens.co.nz>

Holiday Park to the north-east has largely remained unchanged, however the wider area (Mount Maunganui) has continued to develop.

Mauao in the 1940s was grassed on the lower portion with vegetation on the upper portion and western side and some tracks are evident. The vegetation varied slightly and the tracks became established over the years. A water tank was constructed to the south-west of the site, evident in the image from 1977. Landslide features are evident on the lower portion of the slope to the north of the site dating back to 1943. Various small to large scale, shallow and less commonly deeper instability is evident on the lower slopes over the years. Boulders are evident on the lower parts of the slopes in the aerial photographs, suggesting historic rockfall.



Figure 4 | Historical aerial photograph of the area from 1943 showing the approximate location of the subject site (TCC Mapi).



Figure 5 | Historical aerial photograph of the area from 1977 showing the approximate location of the subject site (TCC Mapi).



Figure 6 | Historical aerial photograph of the area from 2003 showing the approximate location of the subject site (TCC Mapi).

4.3 Published Geology

The site is located at the base of Mauao, on the interface between the steep slopes of the rhyolite dome to the north-west and the low-lying tombolo (sand spit) to the east/south-east.

The New Zealand Geology Web Map⁶ indicates that the site is underlain by Minden Rhyolite Subgroup (Whitianga Group) of Coromandel Volcanic Zone. The material is described as flow-banded rhyolite to rhyodacite lava; often as domes or dome complexes, some highly eroded.

The Minden Rhyolite Subgroup material is generally capped with a sequence of highly weathered airfall tephra (volcanic ash).

Due to the location of the development at the base of Mauao, some colluvium and/or debris may be encountered.

On the low-lying area to the east of the site, Holocene shoreline deposits are mapped. The material is described as beach deposits consisting of marine gravel, sand and mud on modern beaches.

No active faults are located in the vicinity of the site (more than 20 km away).

4.4 Existing Information

A number of existing stability assessments have been completed for the slopes at Mauao and the available reports have been summarised below. The existing stability assessments have been used in the following assessment.

Dr Laurie Richards completed the Mauao slope stability assessment in 1999⁷ which involved the assessment of the north-eastern and south-eastern slopes and the risk of instability to the hot pools, campground and walkways. The report identifies many historic landslides and discusses multiple modes of failures. The report concluded there were a number of potential landslide hazards above the campground, however the risk to the public is within acceptable levels and no engineering works are required to mitigate the hazard. It recommended routine topographical and geomorphological surveys and an annual site walkover should be completed.

Shrimpton & Lipinski Ltd undertook a site geotechnical assessment report in 2000 for the redevelopment of the campground⁸. The ground investigation showed the site conditions comprised foredune sands and colluvium. The report recommends a 5 m setback from the cut batter slope on the western margin of the camping ground, above the amenities block.

⁶ GNS Science (2013). New Zealand Geology Web Map. Retrieved from data.gns.cri.nz/geology/

⁷ Dr Laurie Richards, Rock Engineering Consultants. Mauao Stability Assessment. Report to TCD, 31 May 1999.

⁸ Shrimpton & Lipinski Ltd. Site geotechnical report. Harbour camping ground. Adams Avenue, Mount Maunganui. Reference: 14795, dated March 2000.

Avalon Industrial Services Ltd (Avalon) completed a report in 2003 on the rock slopes and rockfall hazards at Mauao⁹. The report identified unstable rock on the upper bluff, including an area above the campground. The report also noted that as a result the loss of vegetation following the 2003 fire, the rockfall hazard increased as there is minimal vegetation to slow or trap rockfall and the loss of root binding has left slopes prone to scour.

References have been found to several further reports prepared by Avalon in 2004 and 2005, however we were unable to locate a copy of these reports. Some of the information was summarised in the 2005 report discussed below.

Avalon completed a report on the rockfall hazard and risk assessment for the southern campground and hot pools (Zone 6) in 2005¹⁰, which includes the area of the proposed development. A 150 year storm event occurred in May 2005, causing flood damage resulting in rockfall, surface scour, and over 40 slips and dropouts on Mauao. Additional rainfall caused a 2 m to 3 m boulder to roll approximately 25 m down the slope above the hot pools, which was the subject of the specific assessment. The potential rockfall sources were identified and many boulders were considered marginally stable. The statistical risk of rockfall causing fatalities in the campground was recalculated and indicated a fatality return period of approximately 200 years. The report recommended either monitoring, rock scaling, planting or consideration of catch fence options for protection of the campground from rockfall.

Terrane Geotechnical Solutions completed a peer review in 2006¹¹ of four of the Avalon stability reports completed for Mauao. The review generally supports the conclusions in the Avalon rockfall assessments and highlights that scaling will reduce the risk, however would not eliminate the risk of rockfall and that mitigation structures may be required if the risk to the campground is deemed too high. The review also identifies the risk of 'mass slips' and that although limited information is available the risk is considered to be low to very low. It indicates that the level of rockfall risk has been reduced by the management programme over the last 8 years (including regular inspections, rodent control, access restrictions and rock scaling to remove the highest risk boulders identified in the Avalon reports), and recommends continuing the maintenance programme.

Tonkin & Taylor (T&T) undertook a geotechnical assessment for the proposed Mount Maunganui hot pools redevelopment in 2010¹². The assessment was located to the south of the hot pools, north-east of the subject site, and the key points are provided below:

⁹ Avalon Industrial Services Ltd. Mauao rock slopes & Rockfall hazards. Reference No: 0314, dated 7 August 2003.

¹⁰ Avalon Industrial Services Ltd. Mauao slopes; 2005; Zone 6 – Southern campground and hot pools. Rockfall hazard & risk assessment. Reference No: 0534 A (Issue 2), dated 10 August 2005.

¹¹ Terrane Geotechnical Solutions. Mauao rockfall hazard – peer review report. Reference: 3071, dated 22 February 2006.

¹² Tonkin & Taylor. Geotechnical Assessment Report Mount Maunganui Hot Pools Redevelopment. Tauranga City Aquatics Ltd. Reference: 851305001, dated August 2010.

- The ground conditions comprised topsoil underlain by fill, underlain by layers of sands (beach deposits), interbedded with airfall ash and weathered rhyolite below 20 m bgl.
- The risk of liquefaction is considered to be high in a ULS case and there is a moderate risk of lateral spread.
- The upper 1.0 m to 1.5 m was not suitable for foundations due to topsoil, fill and loose sands.
- A rockfall assessment using the Australian Geomechanics Society (AGS) guidelines based on the previous Avalon reports gives a likelihood of “Possible” and the consequence is “Minor”. The overall risk rating is classified as “Moderate”. Given the ongoing maintenance and annual review of the risk of rockfall and undertaking of scaling where required, the rockfall risk rating to the hot pools was reduced to “Low”.
- Based on visual appraisal using the AGS guidelines, the risk of landsliding causing damage to the hot pools is considered “Unlikely” and the consequence is considered to be “Medium” with an overall risk considered to be “Low” and “Very Low” for the southern area.
- A stiffened raft foundation is recommended to mitigate the risk of liquefaction and ground improvement is recommended to remediate the topsoil, uncontrolled fill and loose sands.

A paper by Zach Martin and Marc-André Brideau¹³ details the spatio-temporal distribution of mass movements on Mauao. Four main types of mass movements have been identified: rotational slides, debris flow, debris avalanche concentrated on the lower slopes and rock fall from the upper slopes. The landslides were correlated largely to rainfall events, soil types and anthropogenic changes. No instability was mapped in the area of the proposed development with only a small surficial failure noted on the lower slopes above the site.

Geoconsult completed a geotechnical investigation report for the proposed alterations to the existing amenities block in the northern part of the Mount Maunganui Beachside Holiday Park. The ground conditions are summarised as non-engineered fill, underlain by historical slip debris and beach deposits. The risk of liquefaction is considered to be low provided a pod raft floor is used. The risk of instability was considered and the report identifies that there is a risk of inundation, however the alteration is not considered to increase the risk to the existing structure. The report recommends that appropriate access on the downslope side of the building to ensure safe egress in the case of instability. The report recommends a piled raft to address the unsuitable surficial soils and risk of liquefaction.

BECA completed a geotechnical investigation for the Awaiti project¹⁴, a viewing platform along the base track on the north-eastern side of Mauao. The report identifies a number of landslides in the area, however concludes that the risk of instability to the proposed structure on the outer edge of

¹³ Zach Martin and Marc-André Brideau. Spatio-temporal distribution of mass movements on Mount Maunganui, New Zealand. 6th Canadian Geohazards Conference, 2014.

¹⁴ BECA. Awaiti Project Geotechnical Investigation and Desktop Summary Report. Reference: 4280807-318756720-120, dated 30 September 2022.

the base track is at low risk. A stability analysis was completed as part of an RFI response¹⁵ that confirmed the slopes in the area were considered stable.

We understand some controlled scaling has occurred over the years including over approximately 500m³ in 2003 and some slip remediation works have been completed on the southern slopes. However, we understand no permanent engineering structures for mitigation of rock fall or inundation hazards have been constructed on Mauao to date.

5 Site Investigation

The investigations undertaken as part of the assessment consisted of the following:

- Site walkover by senior engineering geologist;
- Two cone penetration tests (CPTs) (CPT01a and CPT02) to approximately 6.0 m and 8.3 m below ground level (bgl), respectively. An initial test at CPT01 was completed, however refused at approximately 3.3 m bgl. CPT01a and CPT02 also refused due to dense ground before reaching the target depth;
- Four hand auger boreholes (HA01 to HA04) to depths 0.6 m and 5.0 m bgl. Some of the hand augers terminated before the target depth due to refusal or the hole collapsing. Multiple attempts were made to advance the hand augers where the test locations terminated due to refusal, with the greatest depth achieved shown on the logs; and,
- Four Scala penetrometers adjacent to the hand augers (SP01 to SP04) to depths between 2.1 m and 4.9 m bgl.

Testing depths specified are below ground level (bgl) at the time of testing. The site walkover and hand investigations were undertaken by Stratum and the CPT tests were undertaken by Geo Data Solutions, all on the 29th of October 2025. Due to the early refusal of CPT01 (and retest, CPT01a), CPT01 has not been used as part of the specific geotechnical assessment.

Testing was undertaken at the locations shown on the test location plan presented in Appendix B. Borehole logs and test results are included in Appendix B.

¹⁵ BECA. BC330131 Request for Further Information #3 - Category 1 Geo-Professional Review. Reference: 4280807-318756720-337, dated 13 April 2023.

6 Site Conditions

The shallow ground conditions were variable over the site, as expected due to the variable depositional environments and previous earthworks completed at the site.

Topsoil and fill, considered to be uncontrolled, was encountered to depths between 0.1 m and 1.1 m bgl. HA03 refused at 0.6 m within topsoil. The underlying material in HA01 and HA04 comprised sands up to 2.4 m and 0.2 m, respectively. Scala penetrometer testing indicates the surficial sands are generally dense. The sands are inferred to be Holocene shoreline deposits.

The underlying material across the site comprised predominately silts to the base of the hand augers at depths between 1.2 m and 5.0 m bgl, with some sands encountered below 3.2 m bgl in HA02. Strength testing shows the silt is very stiff to hard and Scala penetrometer testing indicates the sand is dense. As evident in cut banks within the area of the proposed development, boulders are present within the silt matrix that likely caused refusal in the testing. The material is inferred to be reworked undifferentiated volcanic ash and colluvium.

A summary of the subsurface conditions encountered are provided in Table 1 below and a ground model is included in Appendix B.

Table 1 | Summary of subsurface conditions

Inferred Geology (Soil Description)	Depth Range (m) to the base of the layer			
	HA01	HA02	HA03	HA04
Topsoil/Fill (sand and silt)	1.1	1.1	0.6*	0.1
Holocene shoreline deposits (Sand)	2.4	-	-	0.2
Reworked undifferentiated volcanic ash and colluvium (silt and sand)	5.0*	4.1*	-	1.2*

*Maximum depth investigated.

The deeper ground conditions are interpreted from the CPT traces. The CPT results are generally consistent with the shallow ground conditions encountered, however some of the inferred material in the CPT trace is finer than described in the hand augers.

CPT01/CPT01a shows interbedded sands, silts and clays in the upper approximately 1.2 m bgl, underlain by sands to approximately 2.7 m. The underlying material comprises predominately silts and clays up to approximately 5.6 m bgl. Sand was encountered to the base of CPT01a at 6.3 m bgl.

CPT02 shows predominately silts and clays with some sand mixtures up to approximately 6 m bgl, with an increase in sand content to approximately 7.3 m bgl. The underlying material comprised predominantly clays to approximately 8.25 m bgl, where a layer of dense material was encountered at the base of the CPT at 8.3 m bgl, inferred to be the top of the rhyolite.

Groundwater was encountered within HA01 at 4.8 m bgl, however not encountered in the remainder of the hand augers although saturated sand was encountered at the base of HA02 at 4.1 m bgl, assumed to be the perched groundwater table. Groundwater was not encountered following the CPT testing in CPT01 and CPT01a, the holes were recorded as collapsed dry at 1.2 m and 4.4 m bgl, respectively. Ground water was recorded in CPT02 at 2.4 m bgl, possibly due to elevated groundwater pressures at depth. The groundwater tables encountered are expected to be perched and influenced by the spring to the north of the site.

The ground conditions encountered provide a general overview of the likely founding conditions. Confirmation of the ground conditions below the proposed new foundations will be required during construction to ensure they are in accordance with the design assumptions given in this report.

7 Geotechnical Assessment

7.1 Seismic Soil Class

The site soils comprised silts and sands. The seismic subsoil category at this site has been assessed as Class C (shallow soil sites) in terms of NZS 1170.5 section 3.1.3, due to the termination on the rhyolite at approximately 8.3 m depth.

7.2 Foundation Bearing Capacity

The site investigation results indicate the surficial soils is variable and does not comply with the definition of “good ground” as per NZS 3604:2011 “Timber Framed Building” due to the uncontrolled fill and thick topsoil. Good ground with an ultimate bearing capacity of 300 kPa (allowable bearing capacity of 100 kPa) was available within the natural soils below the topsoil and uncontrolled fill at depths between 0.1 m and 1.1 m at the test locations.

7.3 Settlement

We have considered the potential for static settlements as a result of the potential building loads. No significant organic material or compressible soil was encountered, however the layers of weak fine grained material (clays and silts) encountered can settle if loaded. As noted above, the CPTs generally indicate a finer material than encountered in the hand augers.

The CPT data was analysed using CPeT-IT (v 2.3.1.9), developed by Geologismiki, to provide an estimate of load induced settlements for foundations. Note that the analysis was limited due to the depth of the CPTs, however the material at depth is not expected to be susceptible to settlement.

A load scenario of 100 kPa on a 0.45 m diameter foundation at 1.2 m depth, reflecting a piled foundation was used for the settlement analysis.

The analyses indicated that under a piled foundation the predicted primary settlements are less than 3 mm. The total predicted settlements expected to be less than 4 mm in CPT01a and approximately 29 mm in CPT02, predominately occurring below approximately 7.5 m bgl. We consider that the predicted long-term static settlements in the deep soil layers are over-predicted as the existing overburden pressure from the soil below a depth of approximately 5 m is more than 100 kPa. As the proposed building loads are relatively low compared to the overburden pressure, the natural soil at depth is therefore not expected to settle significantly under the proposed building loads. Additionally, the secondary settlements are not affected by the load applied and reflects the consolidation of the soils based on overburden pressures. Furthermore, the site will be cut, reducing the overall applied loads within the building platform.

The risk of static settlement from the inferred colluvial material has been considered. The strength of the material is generally stiff to hard and medium dense to dense. The colluvial material predates the 1940s and the proposed development is light-weight on piles. The risk of static settlement as a result of the colluvial material is therefore considered to be low. The risk of static settlement within the fill is also expected to be low as it is competent and has been in place for several decades, however due to the organic content it is recommended that the piles found below any topsoil and uncontrolled fill material. Static settlements as a result of the proposed development are therefore expected to be within acceptable limits of 25 mm given in NZS 3604:2011, provided the foundation recommendations below are followed and the bearing capacity is not exceeded.

Settlement analysis results are included in Appendix C.

7.4 Liquefaction

A quantitative assessment has been completed of the potential of the site soils to liquefy. Liquefaction is a term used to describe the strength loss experienced by a saturated cohesionless soil when subjected to cyclic loading (i.e. earthquakes). Soil that is susceptible to liquefaction tends to contract when subject to cyclic stresses, which induces excess pore water pressure that leads to a reduction in shear strength. Recently deposited and loose saturated natural soils and poorly compacted fills can be highly susceptible to liquefaction.

We have carried out an assessment of the liquefaction risk and consequent ground movements in general accordance with the NZGS/MBIE publication “Earthquake geotechnical engineering practice – Module 3: identification, assessment and mitigation of liquefaction hazards”, dated November 2021.

The CPT data was analysed using Cliq (v 2.3.1.15), developed by Geologismiki. This software was used to calculate the soil resistance against liquefaction using the Boulanger and Idriss (2014) method, including clay-like behaviour (cyclic softening) and dry sand settlements.

The liquefaction analysis undertaken provides estimates of free-field settlements, representing the potential vertical ground deformations in the absence of structural loads or ground improvement. These values are derived from simplified empirical procedures and are intended to characterise the general site response under seismic loading. It is noted that free-field settlement estimates do not directly account for soil–structure interaction effects, foundation stiffness, or load distribution, and therefore may not represent the actual differential settlements experienced by a structure. Such interactions are typically addressed in the context of detailed foundation design.

Design ground accelerations were calculated in accordance with the NZGS/MBIE publication “Earthquake geotechnical engineering practice – Module 1”, dated November 2021 for three design scenarios:

- Ultimate Limit State (ULS) considers the 1 in 500 year event, and under these conditions a building should not collapse, but may suffer significant damage to the point that it’s not economic to repair.
- Intermediate (ILS) considers the 1 in 100 year event.
- Serviceability Limit State (SLS) considers the 1 in 25 year event. A building should still remain functional under these conditions.

The peak ground accelerations (PGA) and earthquake magnitude determined appropriate at the subject site for a Building Importance Level 2 and site subsoil class C are presented in Table 2 below. Note that the CPTs refused in the upper 10 m, therefore the assessment is limited to the depth of the CPTs, however the material below 10 m bgl is not expected to be susceptible to liquefaction.

A groundwater table depth of 4.5 m bgl has been applied in the assessment. The groundwater depth is conservative to allow for water table fluctuations and considers the perched ground water table encountered in the investigation.

Table 2 | Seismic parameters

Condition	PGA	Magnitude (M_w)
Ultimate Limit State (ULS, 1:500 year event)	0.30	5.9
Intermediate Limit State (ILS, 1:100 year event)	0.15	5.9
Serviceability Limit State (SLS, 1:25 year event)	0.07	5.9

The analyses indicate that liquefaction under SLS conditions is negligible and minor settlements are predicted following an ILS event.

Some of the loose sands below the groundwater table are expected to liquefy during an ULS earthquake event. Vertical settlement during the ULS case is predicted to be up to approximately 50 mm. Refer to Table 3 below for the predicted vertical settlements.

The liquefaction severity number (LSN) predicts no expression of liquefaction during an SLS event and little expression of liquefaction during an ILS and ULS event.

Table 3 | Predicted liquefaction induced vertical settlements

Test	ULS	ILS	SLS
	Vertical Settlements (mm)		
CPT01a	13	2	Negligible
CPT02	49	6	Negligible

The risk of lateral spreading was considered due to the site being located near the bank below the building platform to the south and east and further to the slope above the harbour to the south. The critical analysis was the slope above the harbour due to the total height of the free face relative to the depth of the liquefiable soil layers. A total free face height of 6 m and distances between 28 m and 56 m was applied in the analysis.

Lateral displacement is negligible in an SLS event and predicted to be less than 80 mm in an ILS event. The ULS lateral displacements are calculated to be 381 mm in CPT01a and 1,267 mm in CPT02. The lateral stretch considers the difference between the lateral movement at the front and at the back of the building, which is predicted to be up to approximately 200 mm in a ULS event, see Table 4 below. The lateral displacement and stretch are considered to be overestimated, as liquefaction is predicted in isolated layers below approximately 5 m depth that are not laterally continuous, therefore significant lateral stretch is unlikely to be able to occur. We recommend ULS lateral stretch up to 100 mm is considered in the building design.

Table 4 | Predicted lateral displacements in a ULS case

	Lateral Displacements (mm) at distance to free face given in brackets		Lateral Stretch (mm)
CPT01a	381 (28 m)	266 (44 m)	115
CPT02	1,267 (45 m)	1,064 (56 m)	203

Liquefaction analysis plots are included in Appendix C.

In summary, no significant liquefaction is expected in a SLS event and minor liquefaction is predicted in an ILS event. Some of the sand and silt layers below the groundwater level are expected to liquefy during an ULS earthquake, with minor to moderate vertical settlements and lateral stretch predicted.

The predicted settlement in the SLS case is consistent with a TC1 site, however the vertical settlement and lateral stretch in the ULS case is consistent with a TC2 site (see technical guidance documentation¹⁶ published after the Canterbury earthquakes).

The ground conditions generally comprise dense sands and stiff silts with groundwater expected to be at least 4.5 m below ground level and non-liquefiable material to at least 5.1 m bgl, forming a natural crust/raft. Additionally, the proposed development involves light-weight structures supported by a piled foundation, which is suitable for TC2 conditions. Provided the foundation piles are limited to 3.0 m depth, the risk of liquefaction is considered to be low and does not need to be further considered in the design.

7.5 Slope Stability

7.5.1 Introduction

Mauao has a history of landslides, including rotational/translational slides, debris flow, debris avalanche and rock falls. A detailed stability assessment of the slopes above the site is outside the scope of this report, however the previous assessments referenced and outlined in Section 4.4 have been reviewed, with relevant points summarised below. Setbacks have been provided in Section 7.5.4 for the terrace slopes above and below the site.

7.5.2 Rockfall

The risk of rock fall affecting the Holiday Park adjacent to the site was specifically assessed by Avalon in 2005, which identified “the assumed total frequency of boulders entering the camp ground (...) is one every 25 years”. The 2006 peer review by Terrane reiterated that the rockfall risk has been managed by regular inspections, scaling etc., and recommends this maintenance programme is continued. T&T (2010) further classified the likelihood rating under the AGS guidelines to be “low”, provided the rockfall hazard monitoring/maintenance by TCC is continued and any high risk rocks are removed. No unstable rocks were observed on or below the grassed slopes above the subject site during our site walkover.

7.5.3 Landslides

A number of landslides are evident on the lower portion of Mauao. The landslides generally appear to be associated with springs above the base track, however the historic deep-seated failures do not appear to have regressed, with recent landslides limited to shallow circular failures. The risk of landslides generally increases following periods of heavy rain and around landform changes such as the walking tracks, with additional influences include seismic, wind, wildlife, wildfire and decay of supporting vegetation. T&T classified the risk of landslides causing damage to the hot pools,

¹⁶ Repairing and rebuilding houses affected by the Canterbury earthquakes, published by the Ministry of Business, Innovation and Employment, 2013.

based on the AGS guidelines, as “Low” to “Very Low”, partly due to the distance from the development to the slopes.

7.5.4 Terrace Slopes

The proposed development is located on a terrace with steep slopes (>70 degrees) above and below the site up to approximately 2.5 m high. Minor instability is evident on the cut banks in the area and uncontrolled fill is present below the site, therefore a setback is required from steep cut slopes to provide a margin of safety for the proposed development. Given the evidence of instability and uncontrolled fill, we recommend a setback from the slopes below based on 3H:1V projection from the toe of the slope below the site. Additionally, we recommend a nominal setback of 2 m from the toe of the slopes above the site, to mitigate against any erosion or shallow instability.

Setbacks have been specified to ensure the building is not affected by minor instability on the sloping ground above and below. Specific assessment and/or specific engineering design will be required for any buildings or excavations within the defined setbacks to ensure the building platform is not affected by slope instability and debris inundation. The slopes at the site may be affected by minor, shallow instability however instability is less than likely and, given the setbacks provided, the risk is considered to be within acceptable limits.

The proposed buildings are located outside a 3H:1V setback from the toe of the slope, however the deck is located within the setback and we therefore recommend the deck piles found below the 3H:1V slope, refer to Section 8.1.2 below for recommendations.

7.5.5 Conclusions

In summary, the existing reports generally indicated low or acceptable risk from rockfall and landslides. The risk of instability from rockfall and landslides from the slopes above the subject site is generally considered to be lower than the areas assessed in the existing reports, located to the north of the subject site. The subject site has a lower risk as the site is situated away from the area of seepage and relic landslides to the north, and the high risk rockfall source areas are generally located to the north and the slope aspect is to the north-east. Additionally, the anthropogenic terraces on the slopes above the subject site show that no significant instability has occurred in the last few centuries. While some shallow instability has occurred in localised areas, there is a low likelihood of runout affecting the subject site and the existing path for the Mount Base Track and vegetation above the site provides some additional protection. We recommend that TCC continues to monitor and manage the risk of rockfall from the slopes above the site.

As mentioned in Section 7.5.4, standard setbacks have been established from the terrace slopes above the below the building area.

8 Recommendations

8.1 Foundations

8.1.1 Proposed Buildings

As outlined in Section 7.2 above, the near surface ground conditions are not suitable for the support of standard shallow foundations due to the depth of topsoil and uncontrolled fill. A bearing capacity of 300 kPa (allowable bearing capacity of 100 kPa) was available below the topsoil and uncontrolled fill at depths between 0.1 m and 1.1 m bgl.

For the proposed buildings supported on a piled foundation we recommend deepened piles to found within the stiff silt and medium dense sand below the unsuitable surficial material. A minimum depth of 1.4 m bgl is required to ensure all piles extend below the unsuitable material, unless inspected by a geo-professional. The lateral restraint is to be calculated using a friction angle of 32 degrees and a unit weight of 16 kN/m³ can be assumed. Piles should be limited to 3.0 m below the ground level to maintain adequate offset above the liquefiable soils, unless further assessed.

Alternatively, shallow ground improvement could be completed to remove the topsoil and uncontrolled fill to utilise a foundation designed in accordance with NZS 3604:2011. Earthworks would need to be undertaken in accordance with Section 8.2 of this report.

In accordance with the recommendations in NZBC, Appendix B1/VM2, the following strength reduction factors shall be applied to the geotechnical ultimate bearing capacity for the structural design of foundations:

- Load combinations involving earthquake overstrength: 0.8.
- All other load combinations: 0.5.

Any foundation works should be carried out by, or under the supervision of, a Licensed Building Practitioner, once building consent has been obtained.

8.1.2 Deck Piles

The deck extends to the east of the proposed buildings and is considered to be an IL1 level structure. The proposed deck extends within the setback from the steep cut slope below the site. We recommend the piles are specifically designed to allow for the topsoil and uncontrolled fill as outlined in Section 7.2 above and for the potential risk of instability from the slope below. We recommend piles found below the topsoil and uncontrolled fill, which was encountered up to 1.1 m bgl within the area of the proposed buildings, however may increase in depth towards to crest of the slope. We recommend the piles are inspected by a geo-professional to ensure the piles found in competent material. Below the unsuitable material, a geotechnical ultimate bearing capacity of 300 kPa is available. Further, we recommend the piles found 0.3 m below the 3H:1V

projection from the toe of the slope, to mitigate the risk of instability. On the eastern side of the terrace, it is expected that at this depth the piles will also be founding below any uncontrolled fill.

8.2 Earthworks

No significant earthworks are proposed for the development, with less than 0.5 m of cut expected to be required and no filling being proposed, unless shallow ground improvements are undertaken. Any earthworks, including excavation, preparation of subgrade, and backfill should be performed in accordance with the geotechnical recommendations presented in this report, the Tauranga City Council Infrastructure Development Code and applicable portions of NZS 4431:2022.

Earthworks should not take place until the building consent application has been approved. Should earthworks take place without a building consent, the works may not be able to be certified.

If shallow ground improvements are undertaken, earthworks should be carried out in accordance with the following recommendations:

- The topsoil and any unsuitable soils shall be removed from the extent of the building platform areas. Where possible, the excavation shall extend outside the foundation by the depth of excavation, or at least 1 m.
- The subgrade of the building platforms shall be inspected and tested by a geotechnical engineer prior to any fill being placed.
- An approved fill material shall be placed and compacted in layers of 200 mm up to the proposed subgrade level.
- Granular fill shall be compacted to achieve an average of at least 15 blows per 300 mm penetration with the Scala penetrometer, with no single value to be less than 3 blows per 100 mm. Granular fill material should achieve an average of 95% MDD (maximum dry density), with no single result to be less than 92% MDD, unless otherwise determined appropriate by the geotechnical engineer.
- Cohesive fill shall be compacted to achieve a minimum average undrained shear strength of 150 kPa, with no single value to be less than 130 kPa. Air voids shall be less than 10%, unless otherwise determined appropriate by the geotechnical engineer.
- The contractor shall ensure works are carried out to maintain stability of temporary slopes, in particular those in excess of 0.5 m in height and 1:2 (V:H) in steepness.

The finished platform shall be shaped with appropriate grades to prevent surface water ponding during and after construction.

Any earthworks where fill material is greater than 0.6 m in depth should be performed under observation from a geotechnical engineer and will require testing and certification.

We recommend that permanent cut and fill batters are formed at a gradient of 1V:2H or less, unless fully supported by a retaining wall or further stability measures are undertaken to stabilise the

batters. Batters should be topsoiled and grassed to reduce erosion. Structures should be setback at least 1.5 m from the crest of any slopes or filled batters, unless deepened foundations are used.

9 Conclusion

Provided the recommendations of this report are complied with, it is our professional opinion that the ground conditions within the area of the proposed buildings at 1 Adams Avenue, Mount Maunganui are suitable for building development in accordance with the requirements of the New Zealand Building Code. Specific designed deepened piles are required.

The risk of rockfall and landslides from Mauao has not been specifically assessed, however based on the existing assessments, the risk is considered to be within acceptable levels, provided the ongoing monitoring and maintenance programme is continued. Standard offsets have been established from the terrace cut slopes above and below the building platform.

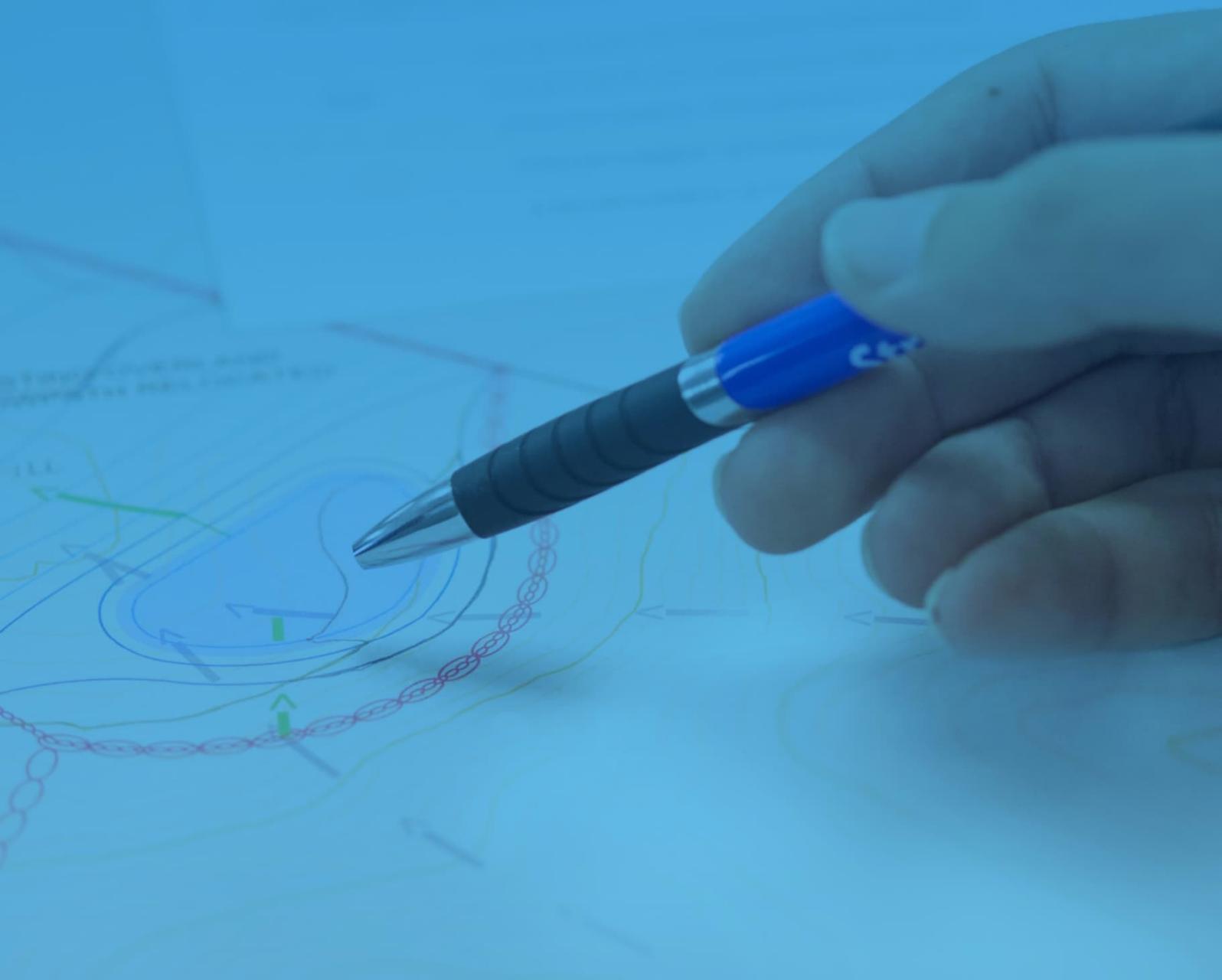
10 Limitations

The assessment given in this report is based on limited site data from discrete test locations. Variations in ground conditions could exist across the site. The nature and continuity of subsoil conditions away from the test sites are inferred and it must be appreciated that actual conditions could vary from the assumed model.

This report has been prepared for the sole benefit of Mauao Trust with respect to the assessment of the geotechnical suitability for the proposed Whare Manaaki Mauao project at 1 Adams Avenue, Mount Maunganui. It is not to be relied upon or used out of context by any other person without reference to Stratum Consultants Ltd. The reliance by other parties on the information or opinions contained in the report shall, without prior review and agreement in writing, be at such party's sole risk.

Appendix A

Proposed Plans





FOR COUNCIL APPROVAL
NOT FOR CONSTRUCTION

SERVICES NOTE
Where existing services are shown, they are indicative only and may not include all site services. Stratum Consultants Ltd does not warrant that all, or indeed any services are shown. It is the contractor's responsibility to locate and protect all existing services prior to and for the duration of the contract works.



SERVICES KEY:

- Stormwater - Existing (Public) — SW —
- Stormwater - New (Private) — SW —
- Stormwater Manhole - (Public) (Symbol: circle with dot)
- Stormwater Sump - (Public) (Symbol: square)
- Wastewater - Existing (Public) — WW —
- Wastewater - New (Private) — SS —
- Wastewater Manhole - (Public) (Symbol: circle with dot)
- Wastewater Inspection / Rodding Eye (Private) (Symbol: red dot)
- Water - Existing (Public) — W —
- Water - New (Private) — W —
- New Power (Private) — —

NOTES:
GENERAL

- Boundaries have been obtained from LINZ Data Service and have not been verified.
- Appurtenant or proposed / existing easements in relation to the subject land may not have been shown if not required for compliance of this proposal.

SURVEY
Survey date: 14.01.2025

- Datum
Horizontal datum: Bay of Plenty 2000
Vertical datum: NZVD 2016
Origin Mark:
Source: LINZ Data Service

SURFACES

- Existing contour interval
Major = 5.0m (Symbol: thick orange line)
- Minor = 1.0m (Symbol: thin orange line)

SERVICES

- Services have been obtained from TCC GIS and have not been ground verified

FLOOD RISK

- Floodplain areas have been obtained from TCC GIS and have not been ground verified
Minor Overland Flow Path (Symbol: pink shaded area)

No.	Date	Drawn	Approved	Issue/Revision
A	20.11.25	TH	-	Issue for Consent
B	-	-	-	-
C	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-

Mauao Trust
1 Adams Avenue
Mount Maunganui

Whare Manaaki Mauao Project
Proposed Layout

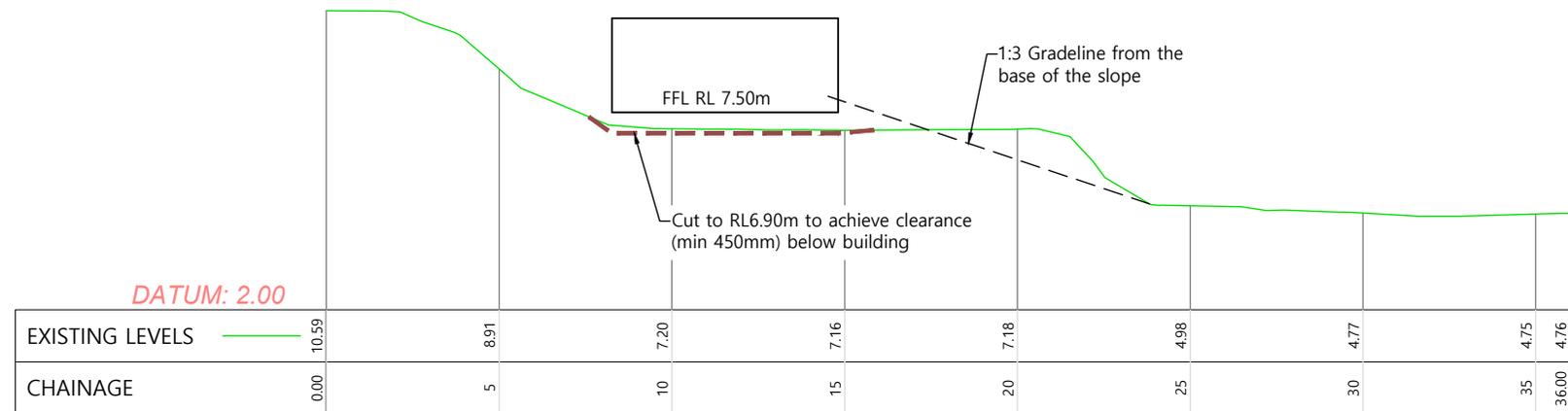
Drawing No. 678164 - CIV - D001	
Sheet No. 02	Issue A
A3 SCALE: 1:200	

Stratum
CONSULTANTS

OFFICE: TAURANGA CONTACT: 07 571 4500

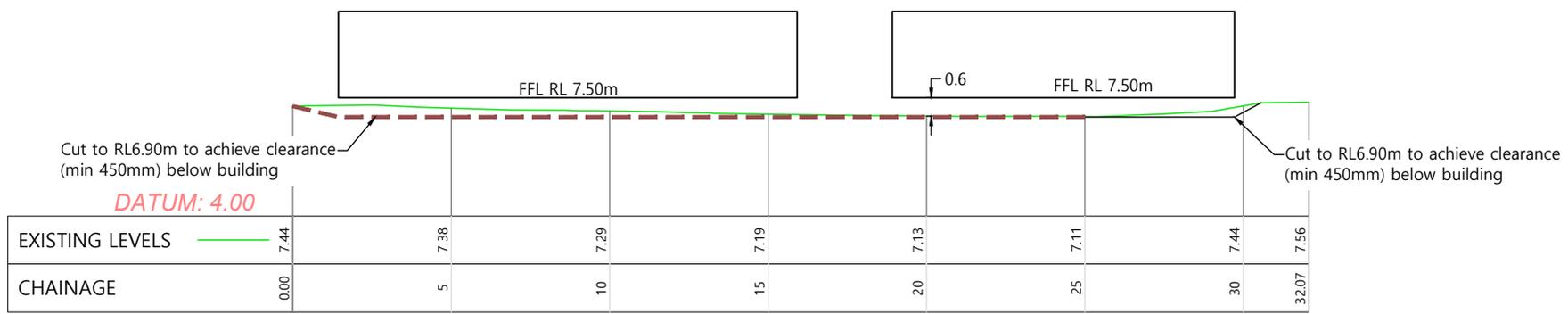
FOR COUNCIL APPROVAL
NOT FOR CONSTRUCTION

SERVICES NOTE
Where existing services are shown, they are indicative only and may not include all site services. Stratum consultants Ltd does not warrant that all, or indeed any services are shown. It is the contractors responsibility to locate and protect all existing services prior to and for the duration of the contract works.



A - LONG SECTION

SCALE: Horizontal 1:100 Vertical 1:100



B - LONG SECTION

SCALE: Horizontal 1:100 Vertical 1:100

No.	Date	Drawn	Approved	Issue/Revision
A	20.11.25	TH	-	Issue for Consent
B	-	-	-	-
C	-	-	-	-
-	-	-	-	-
-	-	-	-	-
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-	-	-	-	-
-	-	-	-	-

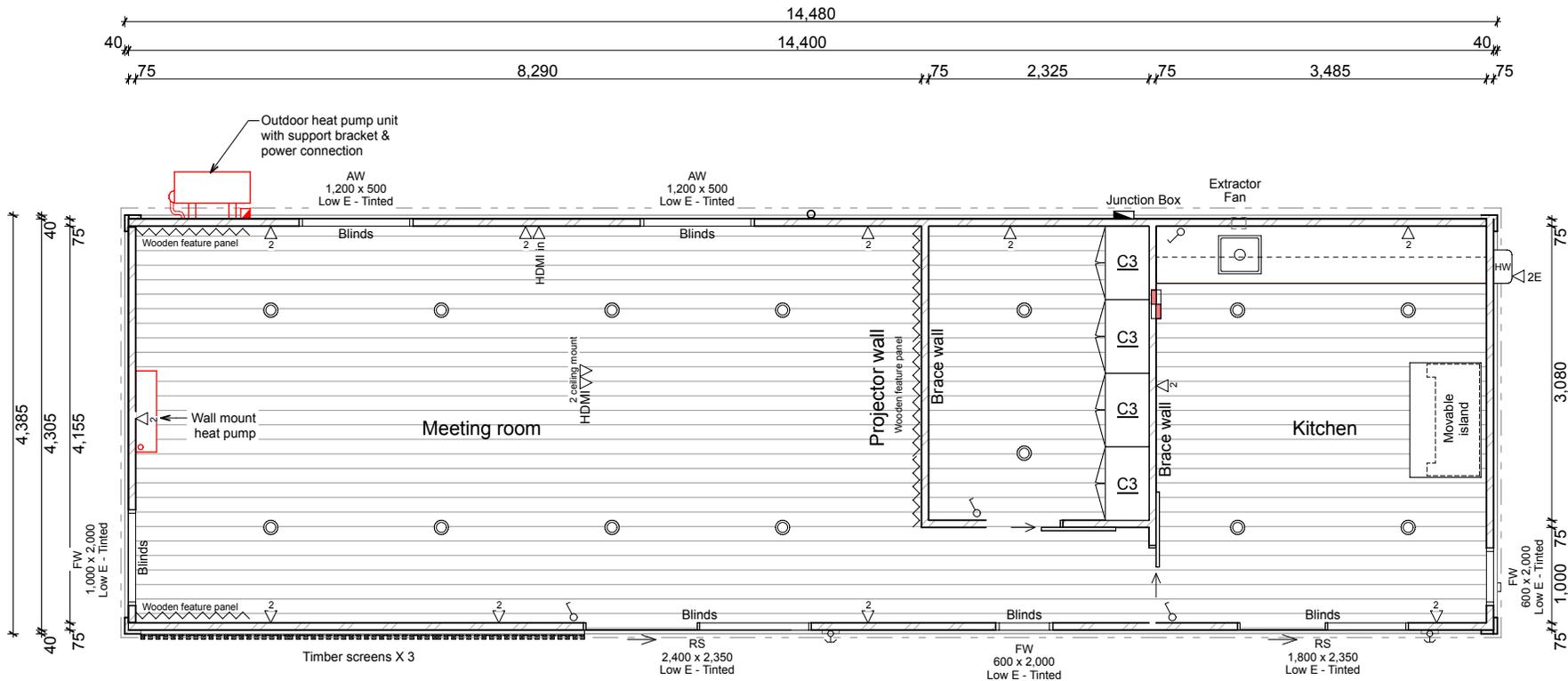
Mauao Trust
1 Adams Avenue
Mount Maunganui

Whare Manaaki Mauao Project
Proposed Sections

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Sheet No. 03	Issue A
A3 SCALE: 1:150	

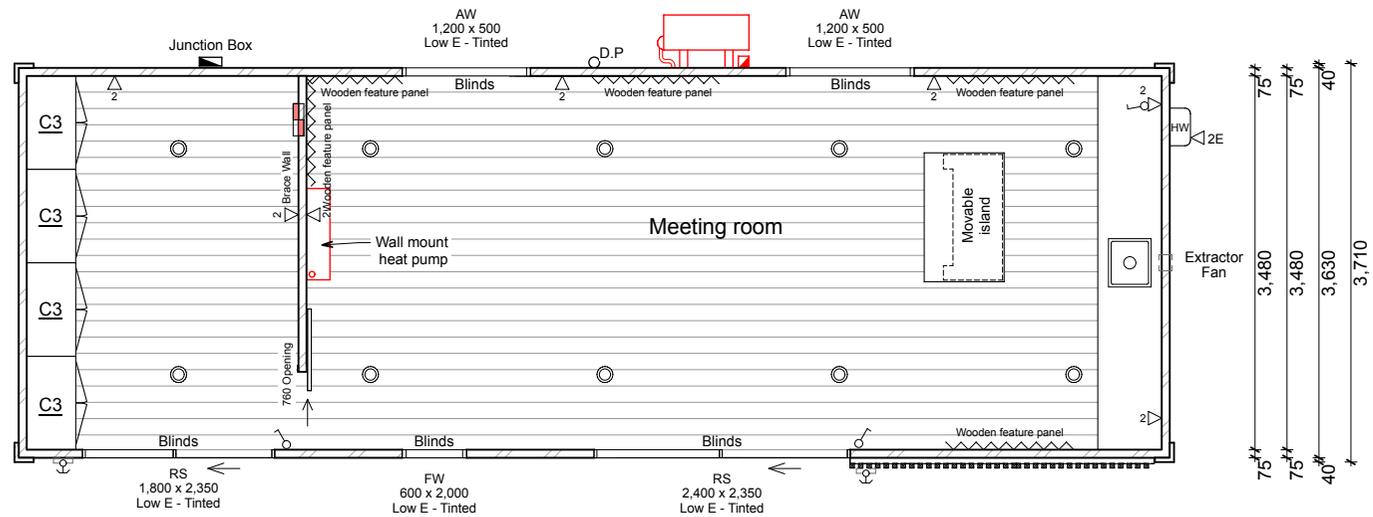
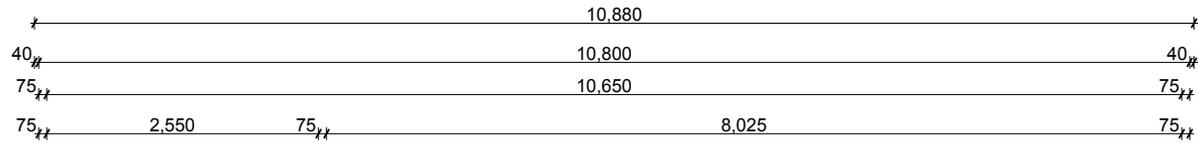
Stratum
CONSULTANTS

OFFICE: TAURANGA CONTACT: 07 571 4500



ELECTRICAL KEY:		WINDOW CODES:	
	Double plug		FROSTED / Obscure glass
	Externally rated double plug 1.5m high		Toughened glass.
	TV (Power only) 1.57m High with low isolation switch.		Fixed Window
	Ceiling mounted LED light		Awning Window
	Light switch		Ranch slider
	External Wall light	(All windows to be double glazed & designed to Very High Windzone)	
	Smoke alarm	PLOT: 10/11/2025	
	Distribution Board		
	Caravan plug - in 0.3m high		
	Junction box - in 0.3m high		
	Potable Water Feed		
	Washing machine taps (install double waste)		

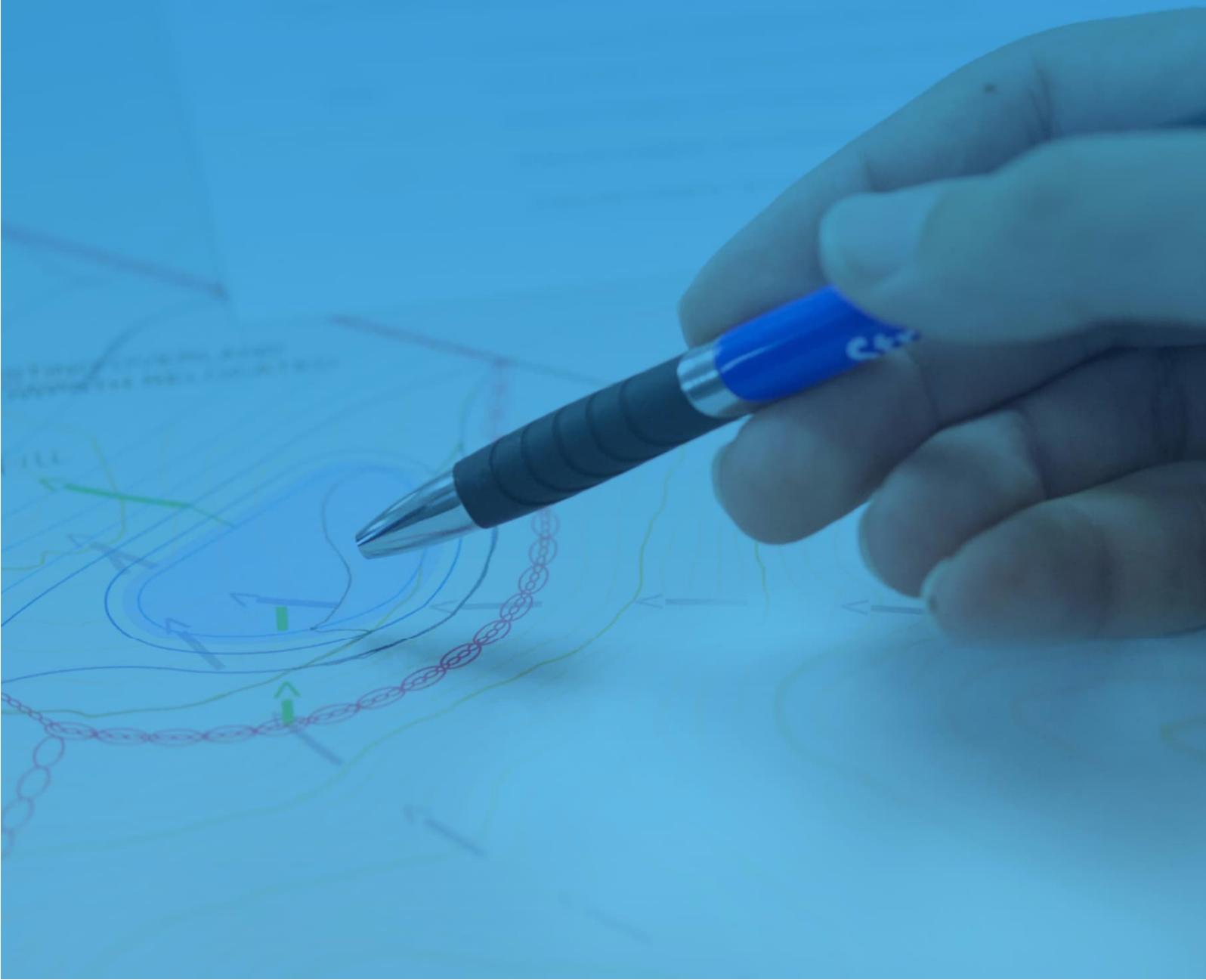




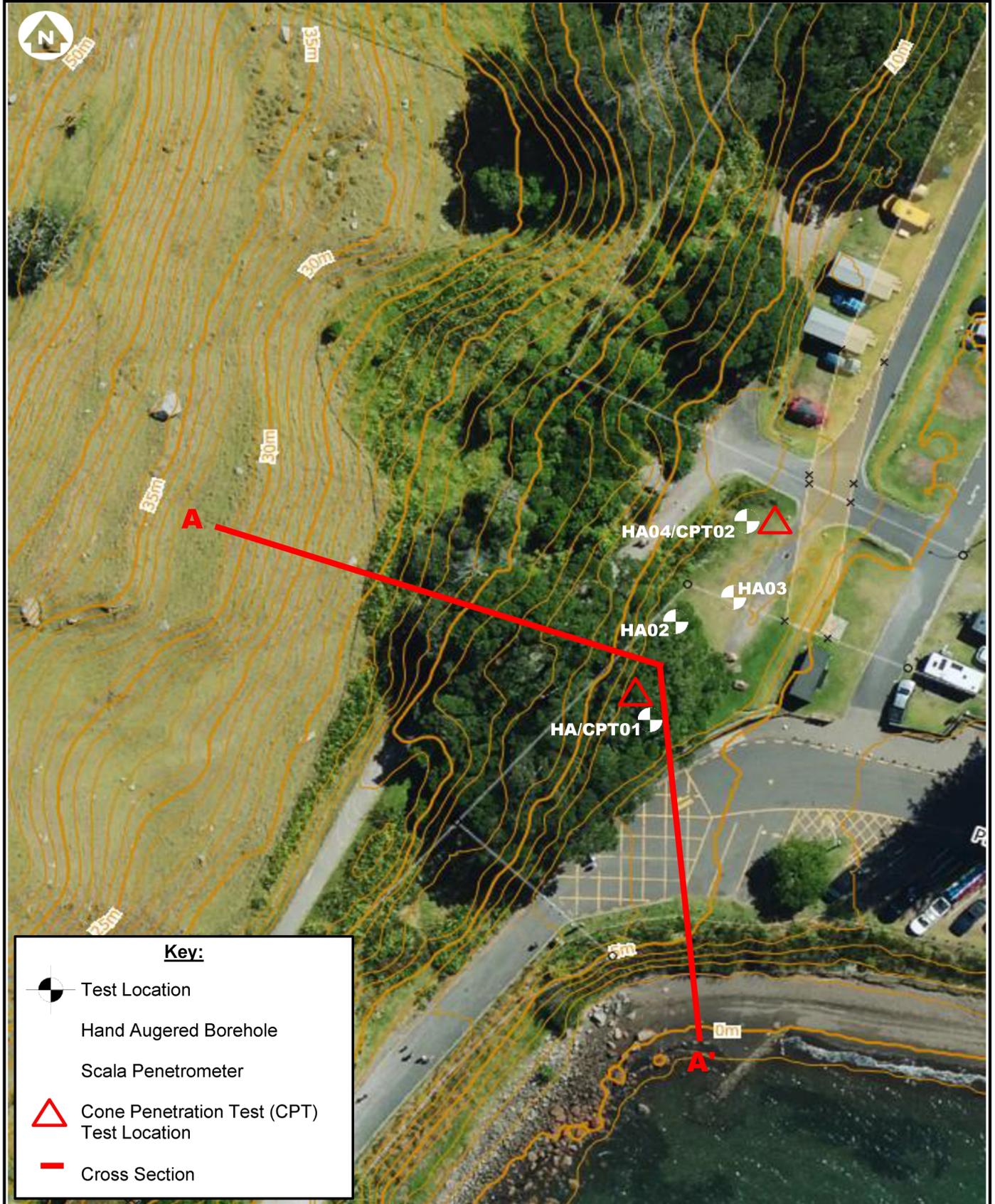
ELECTRICAL KEY:		WINDOW CODES:	
	Double plug		FR. Frosted / Obscure glass
	Externally rated dble plug 1.5m high		Tgh. Toughened glass.
	TV (Power only) 1.57m High with low isolation switch.		FW. Fixed Window
	Ceiling mounted LED light		AW. Awning Window
	Light switch		R/S. Ranch slider
	External Wall light	(All windows to be double glazed & designed to Very High Windzone) PLOT: 10/11/2025	
	Smoke alarm		
	Distribution Board		
	Caravan plug - in 0.3m high		
	Junction box - in 0.3m high		
	Potable Water Feed		
	Washing machine taps (install double waste)		



Appendix B
Site Investigation Data and Location Plans
and Cross-section



Test Location Sketch



Hand Augered Borehole

Borehole No : HA01

Associated Penetrometer No : SP01

Depth (m)	Groundwater	Graphic Log	DESCRIPTIONS	Strength	Soil Class(USCS)	Soil Strengths
0.0m		≡≡≡	(TOPSOIL) Organic SILT minor fine - medium sand, black, moist, low plasticity.		OL	<p>Soil Strengths</p> <p>Legend: □ Scala Penetrometer, ◆ Shear Vane - Undisturbed, × Shear Vane - Remoulded</p> <p>SCALA PENETROMETER RESULTS Blows per 100mm</p> <p>SHEAR VANE RESULTS In Situ Strength (kPa)</p> <p>Vane No. SN518 Calibrated April 2025</p>
0.5m		•••	(FILL) SAND fine - medium some organic silt, dark brown, medium dense - very dense, moist, poorly graded.	MD VD D	SM	
1.0m		×××	(FILL) Clayey SILT minor fine - medium sand some shells, dark brown, hard, moist, moderate plasticity.	H	ML	
1.5m		•••	SAND fine - medium, light grey, dense, moist, poorly graded.	D	SP	
2.0m		•••		No Test		
2.5m		×××	Clayey SILT some fine - medium sand, light brown, hard, moist, low plasticity.	H	ML	
3.0m		×××	Clayey SILT minor fine - medium sand, light brown, very stiff - hard, moist, low plasticity.	H		
3.5m		×××		Vst		
4.0m		×××		H		
4.5m		×××		Vst		
4.8m	▼	×××	Poor Retrieval.			
5.0m		×××	Target Depth			
<p>Notes: Groundwater encountered at 4.8m.</p>						
Cohesive Material			Non Cohesive Material		Classification Symbols and Soil Description	
Very Soft VS Soft S Firm F Stiff St			Very Loose VL Loose L Medium Dense MD		Based on Field Description of Soil and Rock,	
Very Stiff Vst Hard H			Dense D Very Dense VD		New Zealand Geotechnical Society Inc, 2005.	

Hand Augered Borehole

Borehole No : HA02

Associated Penetrometer No : SP02

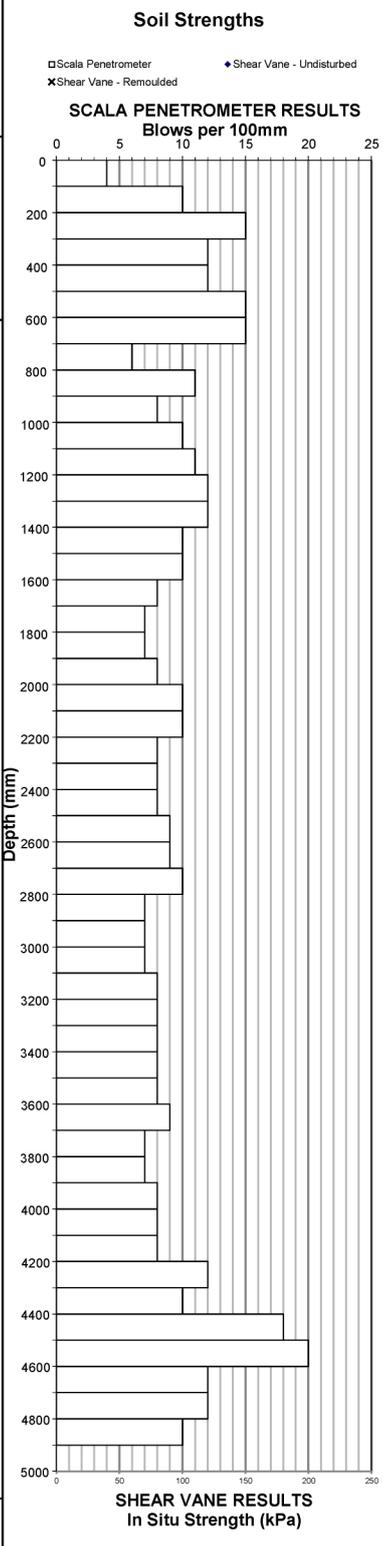
Depth (m)	Groundwater	Graphic Log	DESCRIPTIONS	Strength	Soil Class(USCS)	Soil Strengths
0.0m		☞☞☞	(TOPSOIL) Organic SILT minor fine - medium sand, black, moist, low plasticity.			<p>Soil Strengths</p> <p>Legend: □ Scala Penetrometer, ◆ Shear Vane - Undisturbed, ✕ Shear Vane - Remoulded</p> <p>SCALA PENETROMETER RESULTS Blows per 100mm</p> <p>SHEAR VANE RESULTS In Situ Strength (kPa)</p> <p>Vane No. SN518 Calibrated April 2025</p>
0.5m		☞☞☞			OL	
1.0m		✕✕✕	Clayey SILT minor fine - medium sand minor organic silt, dark brown, hard, moist, moderate plasticity.		H	
1.5m		✕✕✕	Clayey SILT minor fine - medium sand, dark brown, very stiff - hard, moist, moderate plasticity.			
2.0m		✕✕✕			ML	
2.5m		✕✕✕	Changes to light grey minor light brown streaking.	Vst		
3.0m		✕✕✕	Changes to light brown.		H	
3.5m		•••	SAND fine, light grey, dense, wet, poorly graded.		D	
4.0m		•••	No Retrieval.			
4.5m	▼	•••	Hint light grey sand.		SP	
5.0m			End of Borehole - Collapsing			
<p>Notes: Groundwater inferred to be at 4.1m bgl.</p>						
<p>Cohesive Material Very Soft VS Soft S Firm F Stiff St Very Stiff Vst Hard H</p>			<p>Non Cohesive Material Very Loose VL Loose L Medium Dense MD Dense D Very Dense VD</p>		<p>Classification Symbols and Soil Description Based on Field Description of Soil and Rock, New Zealand Geotechnical Society Inc, 2005.</p>	

Hand Augered Borehole

Borehole No : HA03

Associated Penetrometer No : SP03

Depth (m)	Groundwater	Graphic Log	DESCRIPTIONS	Strength	Soil Class(USCS)
0.0m		⋈ ⋈ ⋈	(TOPSOIL) Organic silty SAND fine - medium, black, moist, low plasticity.		OL
0.5m		⋈ ⋈ ⋈			
0.5m		⋈ ⋈ ⋈	End of Borehole - Refusal		
1.0m					
1.5m					
2.0m					
2.5m					
3.0m					
3.5m					
4.0m					
4.5m					
5.0m					



Notes:
 Groundwater not encountered.

Cohesive Material	Non Cohesive Material	Classification Symbols and Soil Description
Very Soft VS Soft S Firm F Stiff St	Very Loose VL Loose L Medium Dense MD	Based on Field Description of Soil and Rock, New Zealand Geotechnical Society Inc, 2005.
Very Stiff Vst Hard H	Dense D Very Dense VD	

Client: Mauao Trust

Project Title: Geotechnical Investigation

Page: 6

No of Pages: 7



Site Address: Pilot Quay, Mount Maunganui

Date Started: 29/10/2025

Date Finished: 29/10/2025

File Number: 678164-GEO-S002

Logged By: s 7(2)(a) ... Privacy

Shear Vane Test Results

Notes: UTP: Unable to penetrate.

Shear Vane readings are used to designate soil strength for cohesive soils only unless otherwise specified.

Shear Vane SN518
Calibrated Apr-25
Constant 1.603

Depth (mm)	HA01			S/V No.		SN518
	Reading	kPa	Strength	Reading	kPa	Sensitivity
800	UTP	-	H	-	-	-
1000	140	224	H	-	-	-
2500	UTP	-	H	-	-	-
2700	UTP	-	H	-	-	-
2900	140	224	H	-	-	-
3100	140	224	H	-	-	-
3300	UTP	-	H	-	-	-
3500	124	199	Vst	20	32	6.2
3700	150	240	H	20	32	7.5
3900	140	224	H	-	-	-
4100	110	176	Vst	24	38	4.6
4300	100	160	Vst	12	19	8.3
4500	120	192	Vst	20	32	6.0
4700	85	136	Vst	20	32	4.3

Depth (mm)	HA02			S/V No.		SN518
	Reading	kPa	Strength	Reading	kPa	Sensitivity
1000	UTP	-	H	-	-	-
1200	UTP	-	H	-	-	-
1400	140	224	H	-	-	-
1600	140	224	H	-	-	-
1800	130	208	H	20	32	6.5
2000	112	180	Vst	20	32	5.6
2200	110	176	Vst	20	32	5.5
2400	120	192	Vst	24	38	5.0
2600	105	168	Vst	15	24	7.0
2800	140	224	H	-	-	-
3000	140	224	H	-	-	-

Depth (mm)	HA04			S/V No.		SN518
	Reading	kPa	Strength	Reading	kPa	Sensitivity
300	140	224	H	-	-	-
500	140	224	H	-	-	-
700	140	224	H	-	-	-
900	120	192	Vst	24	38	5.0
1100	120	192	Vst	20	32	6.0

Client: Mauao Trust

Page: 7

Project Title: Geotechnical Investigation

No of Pages: 7



Site Address: Pilot Quay, Mount Maunganui

Date Started: 29/10/2025

Date Finished: 29/10/2025

File Number: 678164-GEO-S002

Logged By: s 7(2)(a) ... Privacy

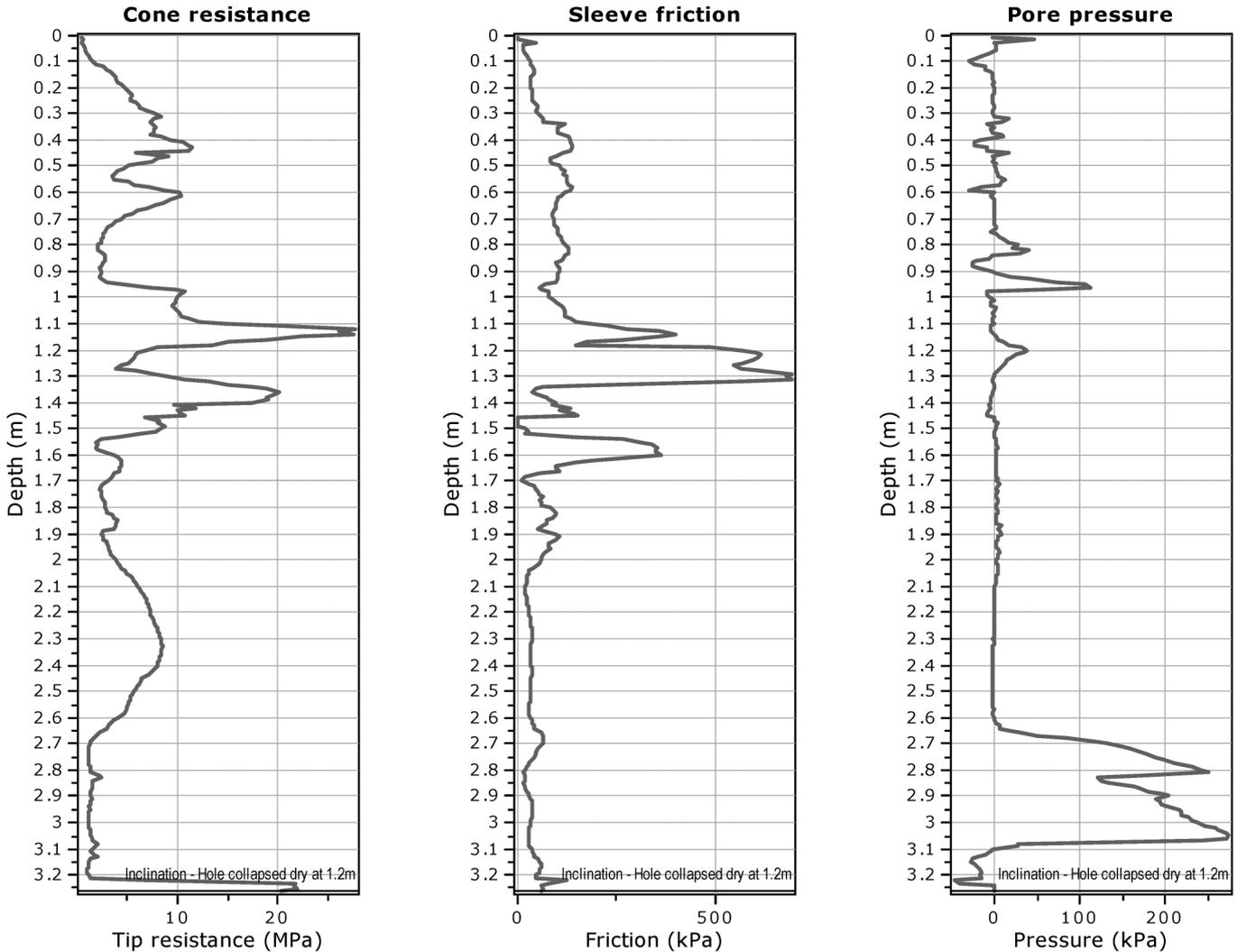
Scala Penetrometer Results

Probe description: 9kg hammer falling 500mm striking a steel anvil driving a 16mm diameter rod fitted with a 20mm diameter cone

Depth of penetration begins at the existing ground level. Scala Penetrometer readings are used to designate soil strength for non-cohesive soils only unless otherwise specified.

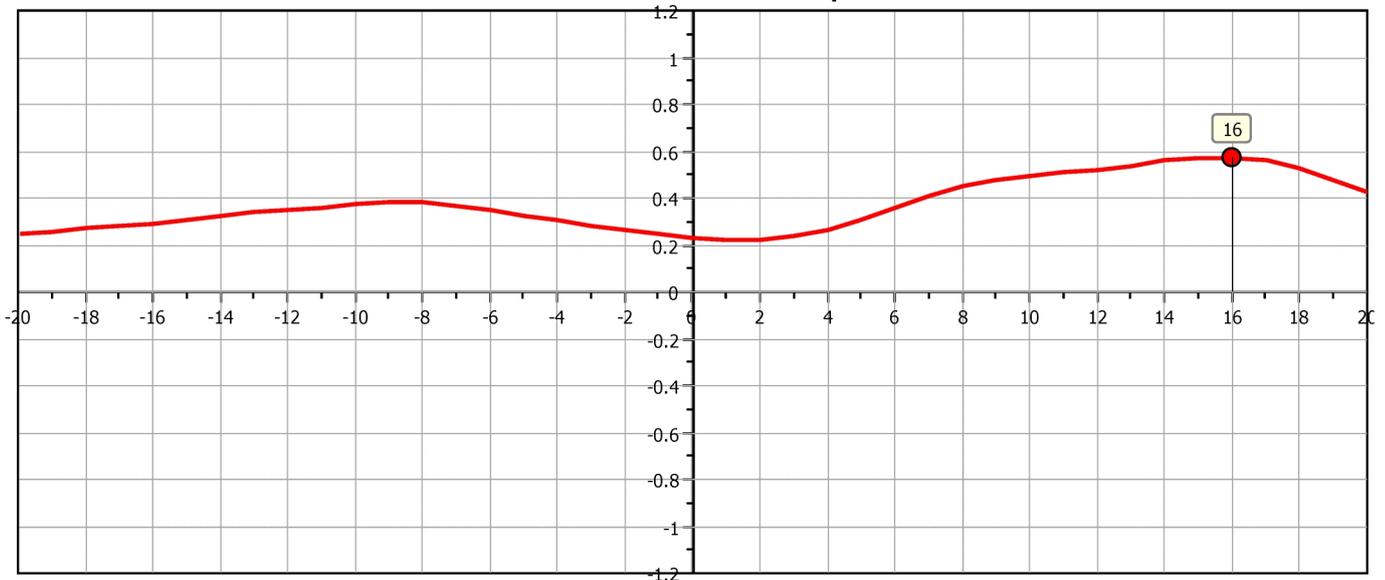
Depth of Penetration	SP01	SP02	SP03	SP04				
GL Start (mm)	Blows Per 100mm	Blows Per 100mm	Blows Per 100mm	Blows Per 100mm				
0								
100	2	2	4	2				
200	5	6	10	3				
300	6	8	15	2				
400	6	15	12	3				
500	20	18	12	3				
600	12	18	15	3				
700	10	14	15	4				
800	16	12	6	14				
900	15	14	11	8				
1000	12	10	8	8				
1100	14	12	10	7				
1200	14	12	11	8				
1300	16	12	12	8				
1400	14	15	12	9				
1500	12	16	10	15				
1600	12	11	10	11				
1700	15	12	8	11				
1800	10	12	7	10				
1900	10	10	7	12				
2000	9	10	8	12				
2100	17	10	10	13				
2200	Bouncing	11	10	10				
2300		13	8	13				
2400		20	8	13				
2500		12	8	14				
2600		13	9	10				
2700		10	9	10				
2800		10	10	12				
2900		10	7	14				
3000		10	7					
3100		12	7					
3200		12	8					
3300		9	8					
3400		11	8					
3500		13	8					
3600		11	8					
3700		8	9					
3800		7	7					
3900		7	7					
4000		8	8					
4100		10	8					
4200		11	8					
4300		14	12					
4400		20	10					
4500		21	18					
4600		18	20					
4700		12	12					
4800		14	12					
4900		12	10					
5000								

Notes:



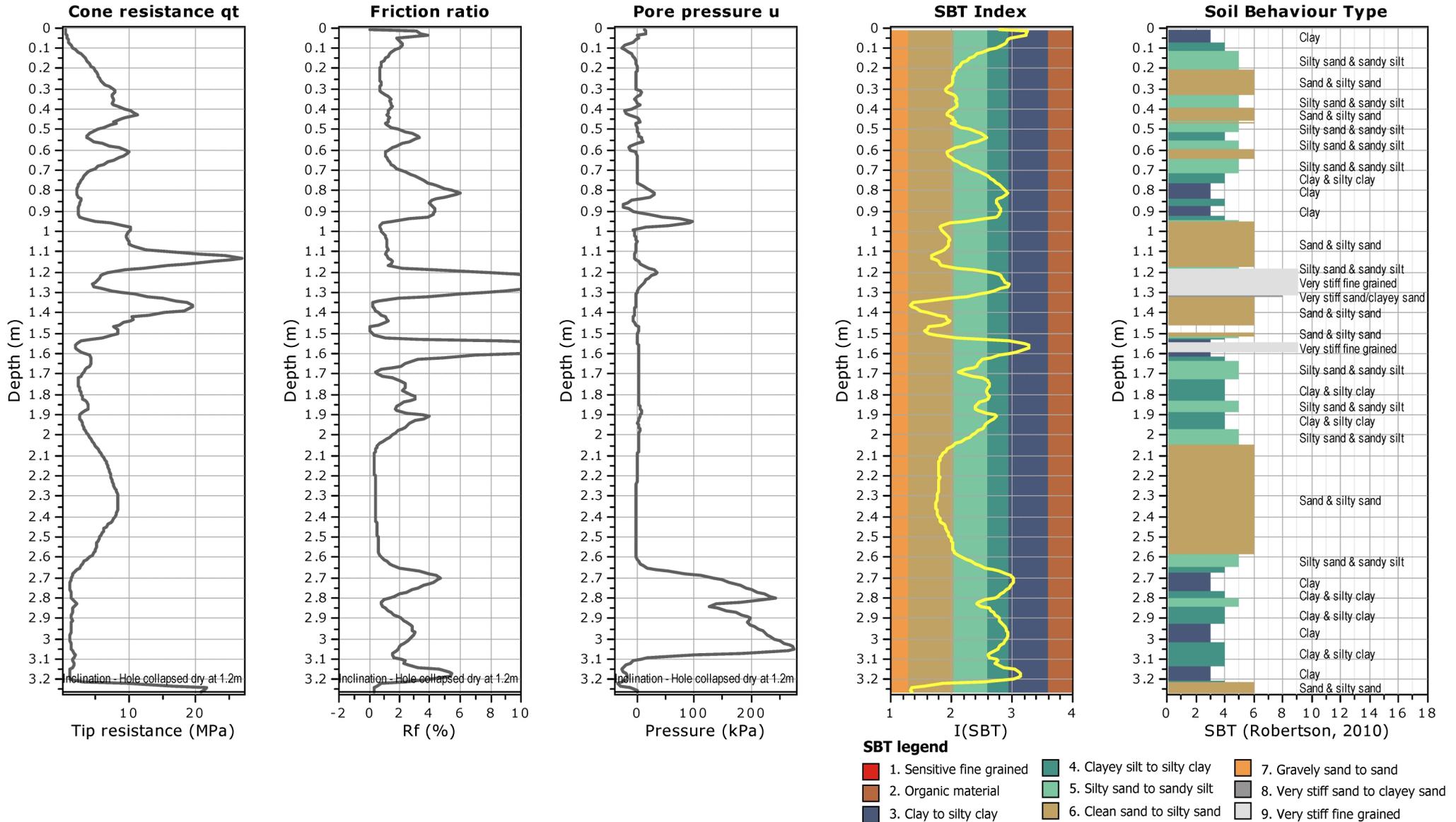
The plot below presents the cross correlation coefficient between the raw q_c and f_s values (as measured on the field). X axes presents the lag distance (one lag is the distance between two successive CPT measurements).

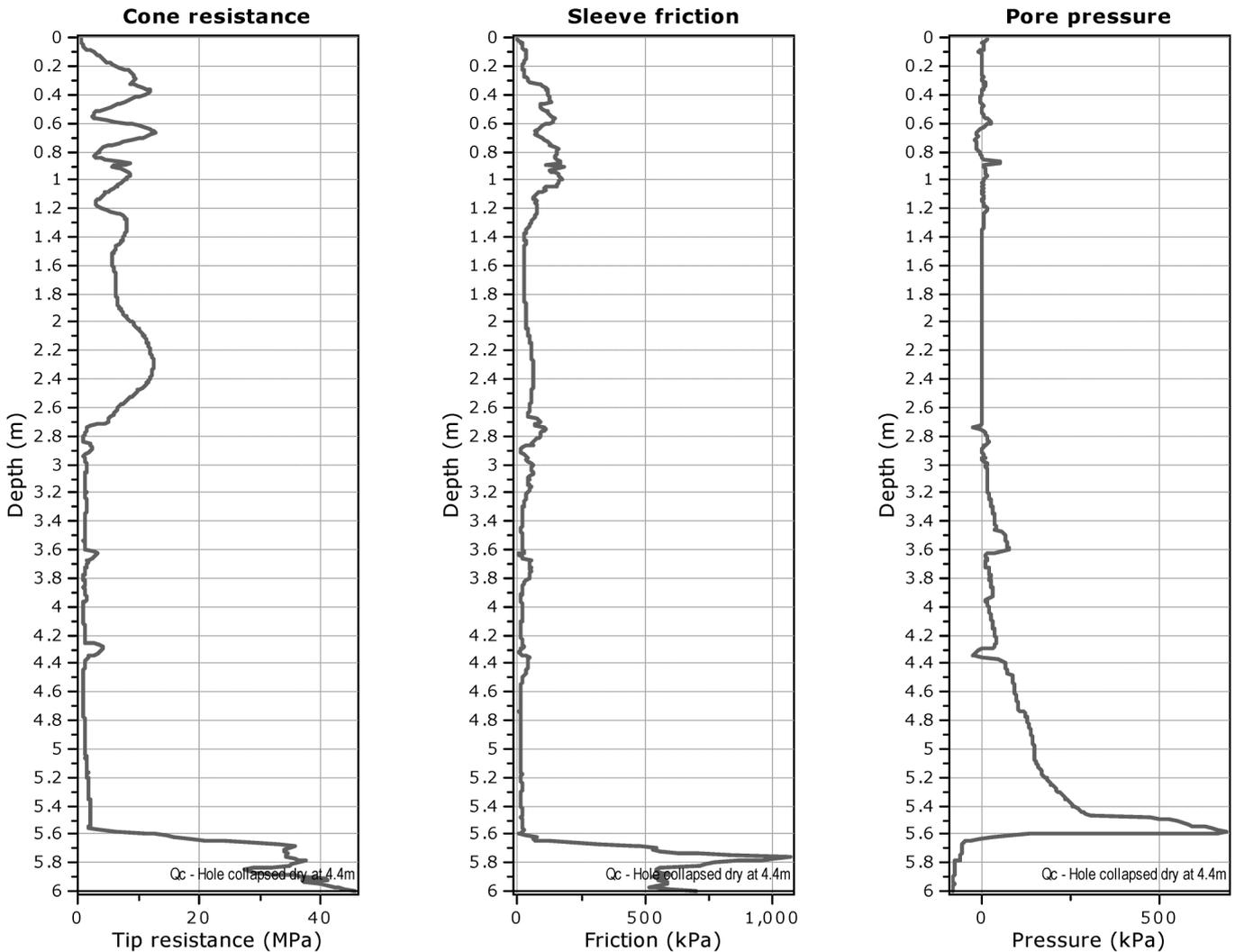
Cross correlation between q_c & f_s



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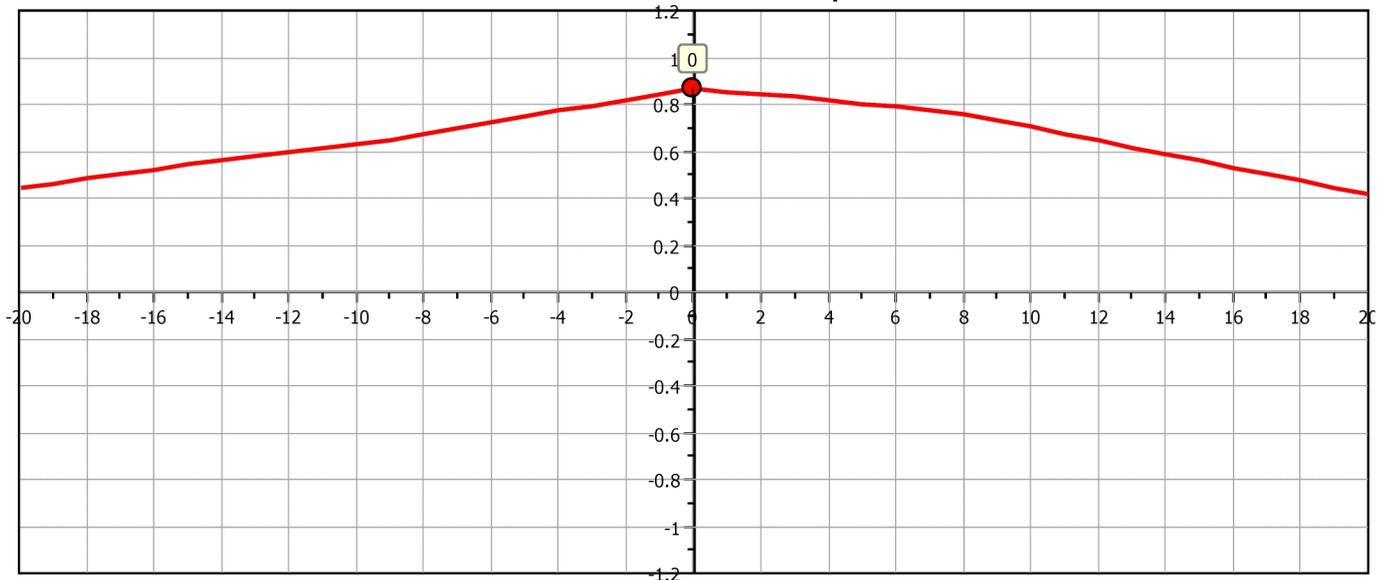
Location: Pilot Quay, Mt Maunganui | Holes dipped onsite using Dipmeter





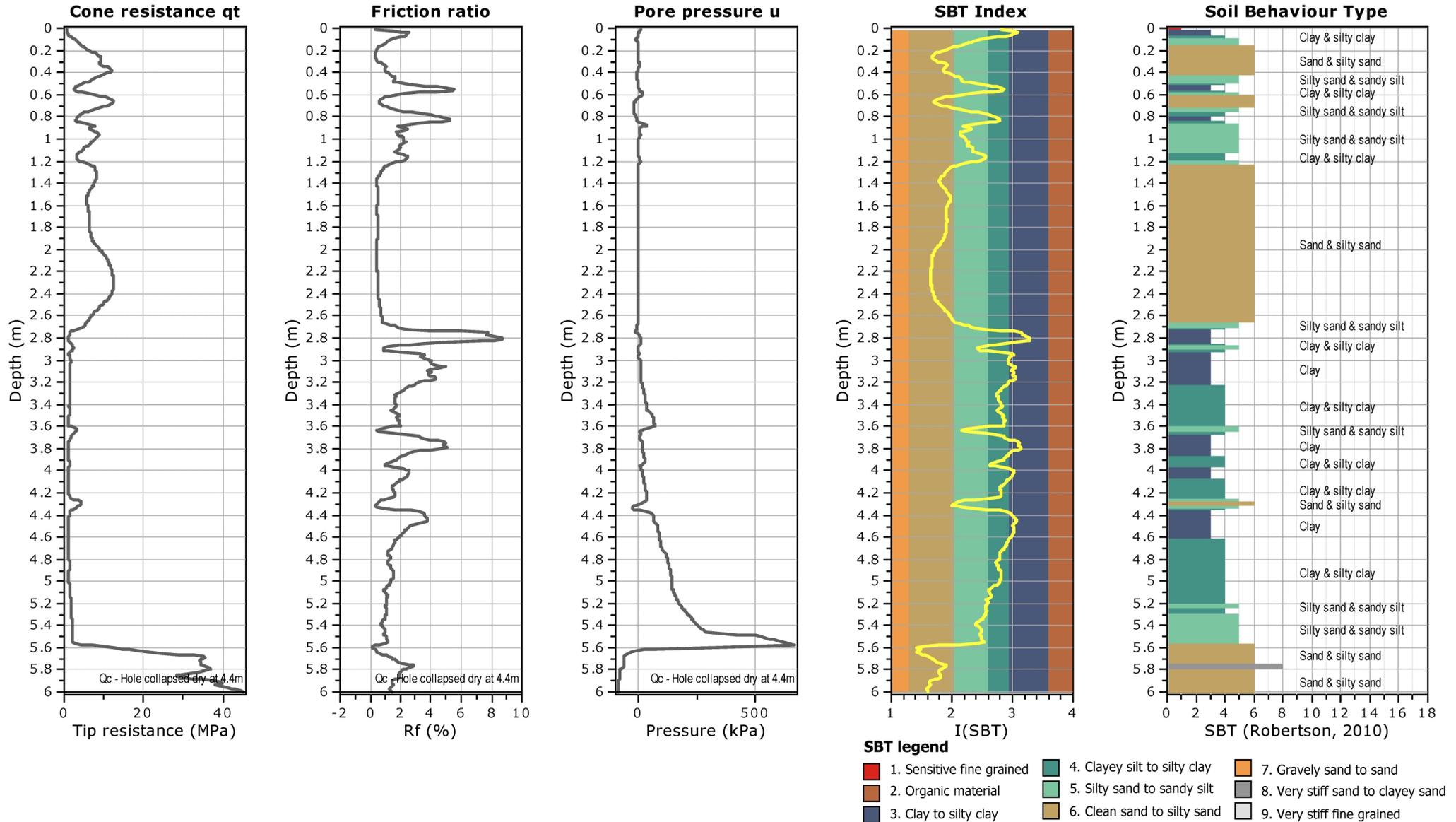
The plot below presents the cross correlation coefficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two successive CPT measurements).

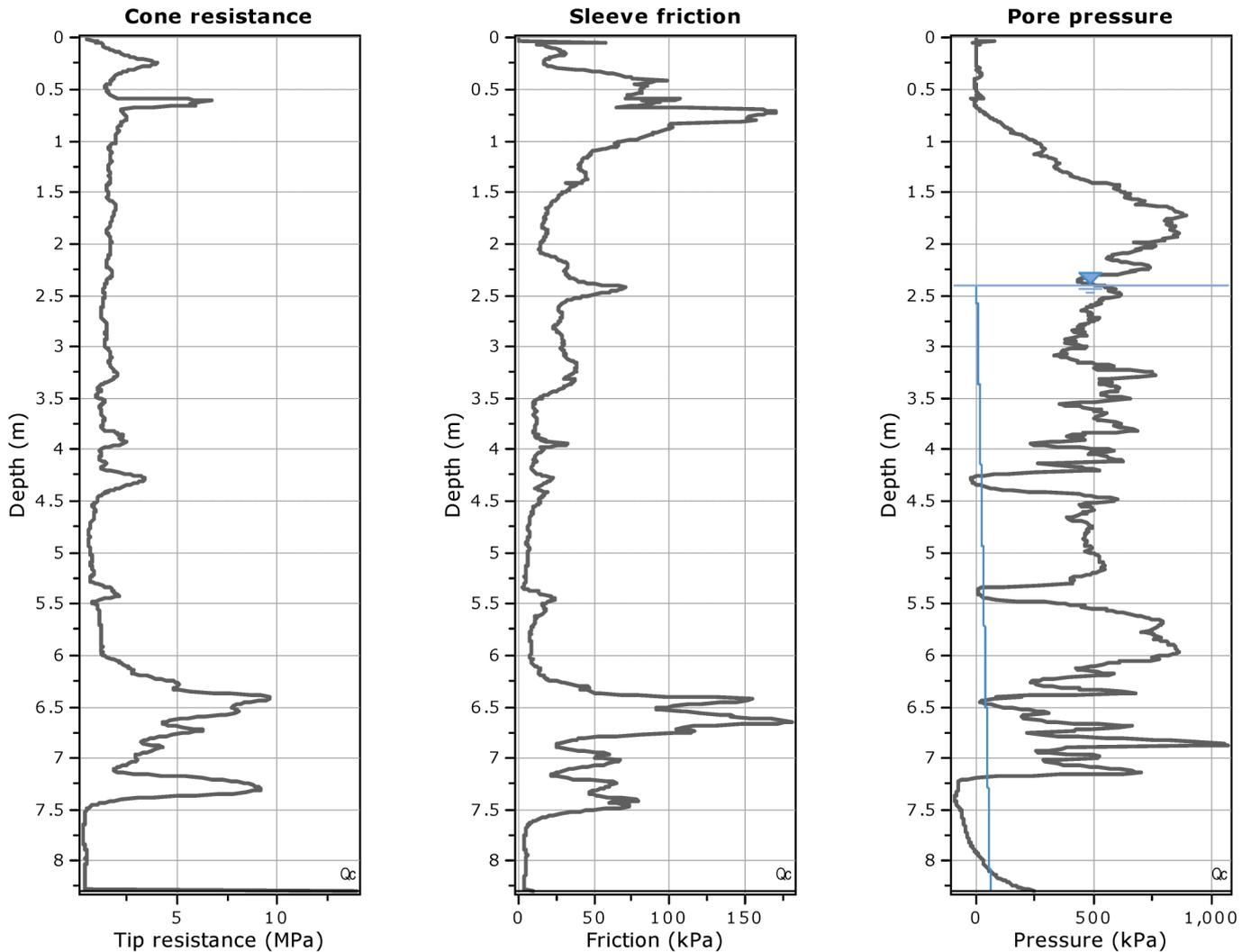
Cross correlation between qc & fs



Project: Stratum Consultants | 678164 | GDS NZ Ltd

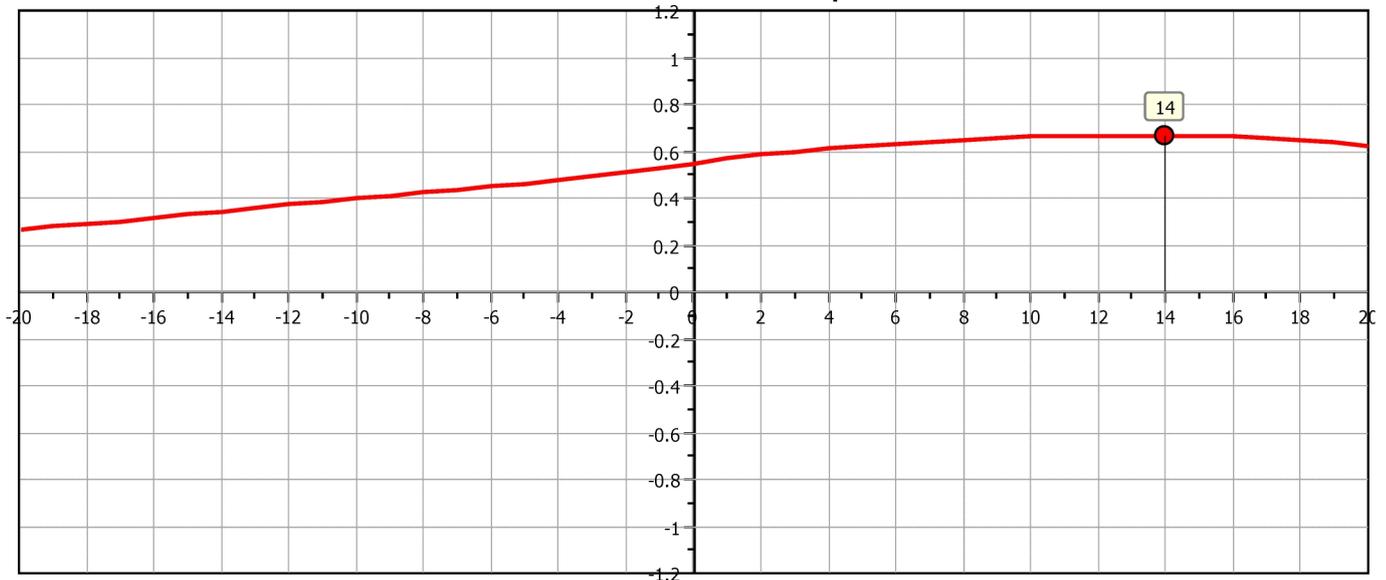
Location: Pilot Quay, Mt Maunganui | Holes dipped onsite using Dipmeter

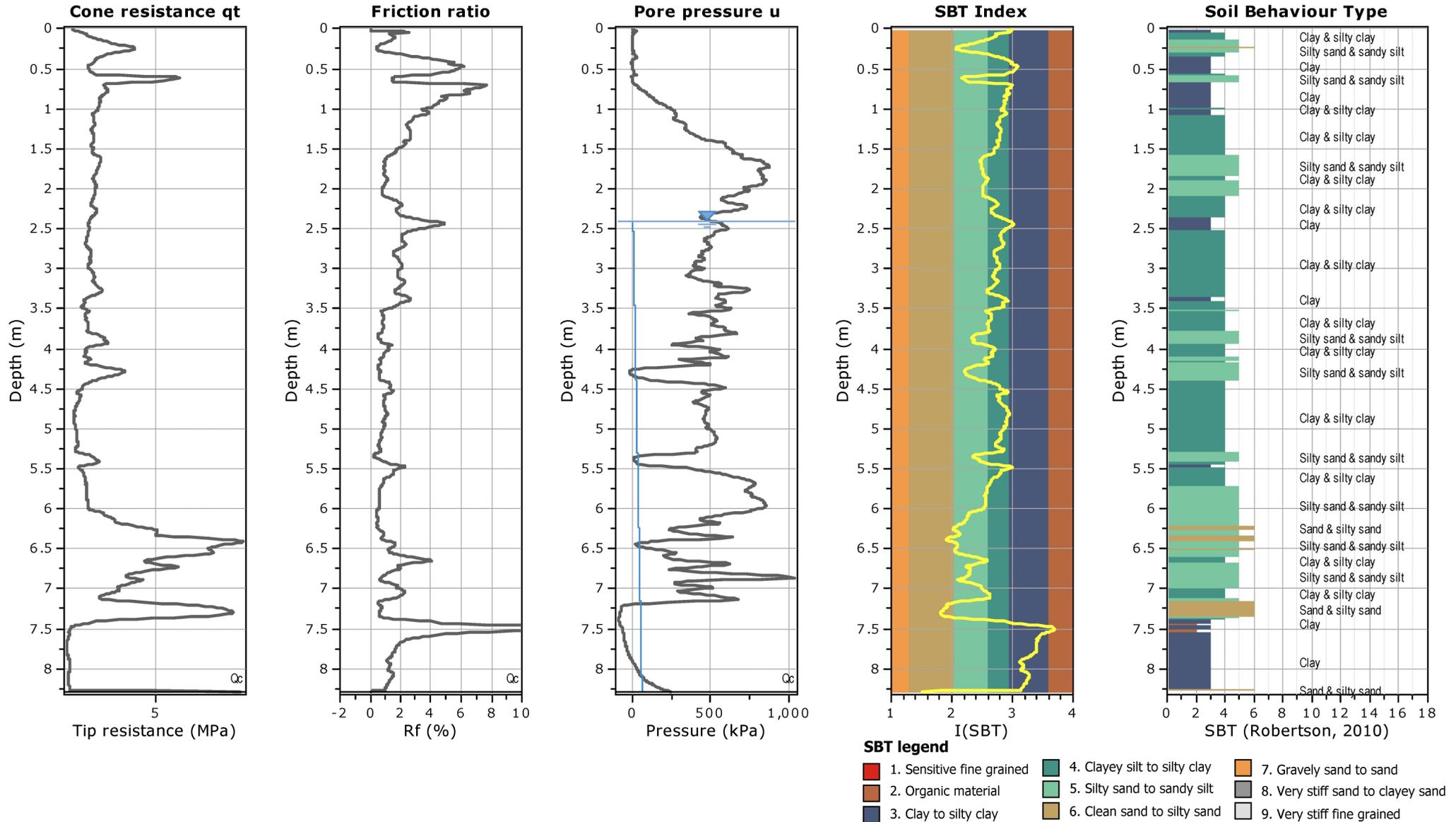




The plot below presents the cross correlation coefficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two successive CPT measurements).

Cross correlation between qc & fs





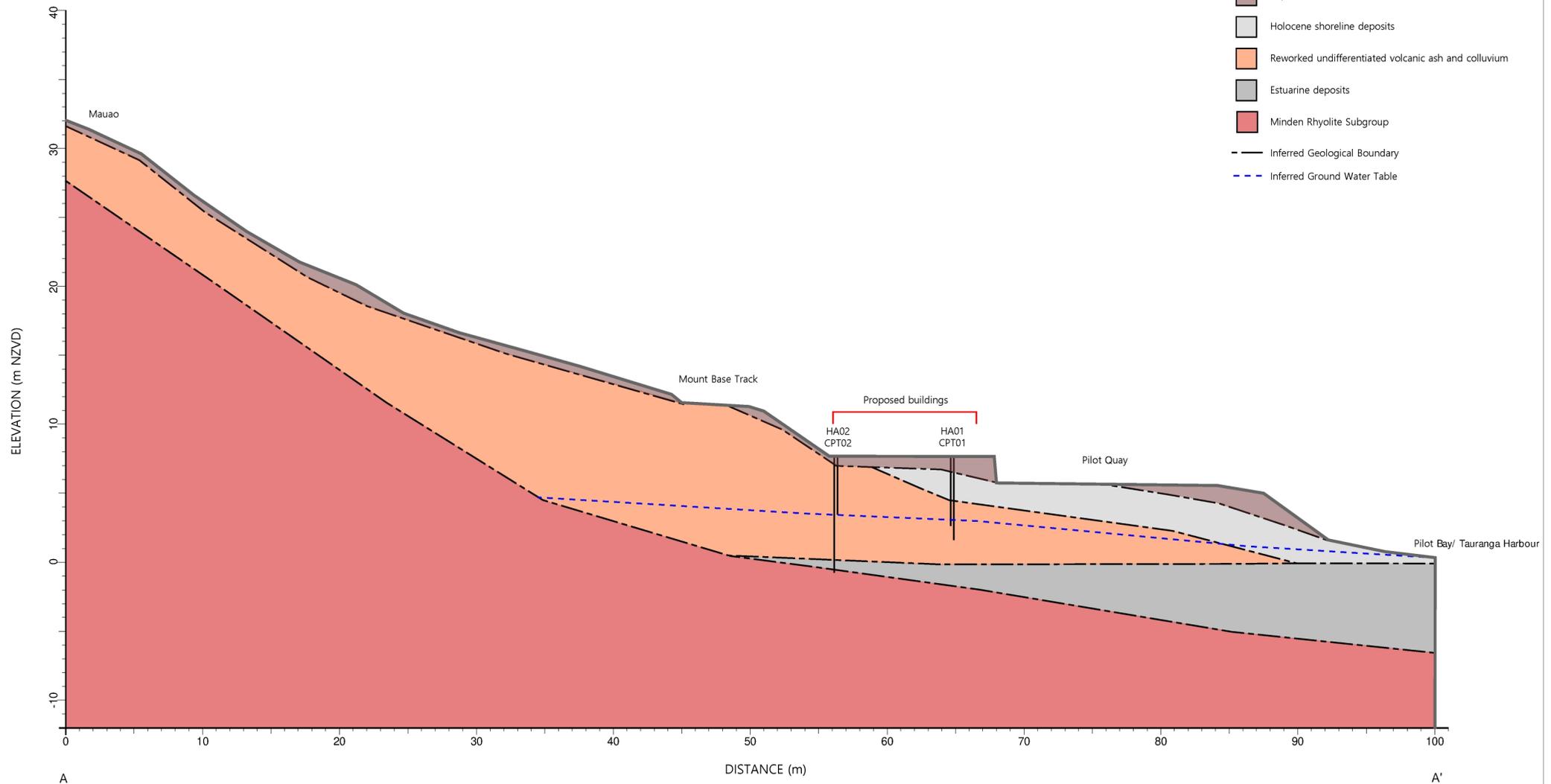
NOTES:

GENERAL

- Areas and measurements are approximate and subject to survey.
- Vertical datum: NZVD 2016
- Contours have been obtained from TCC Mapi and have not been verified on ground.

Geotechnical Units:

- Topsoil/Uncontrolled Fill
- Holocene shoreline deposits
- Reworked undifferentiated volcanic ash and colluvium
- Estuarine deposits
- Minden Rhyolite Subgroup
- Inferred Geological Boundary
- Inferred Ground Water Table



No	Date	Drawn	Approved	Issue/Revision
A	04.12.25	S	7(2)(a) - Priv	Geotechnical Assessment Report
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-

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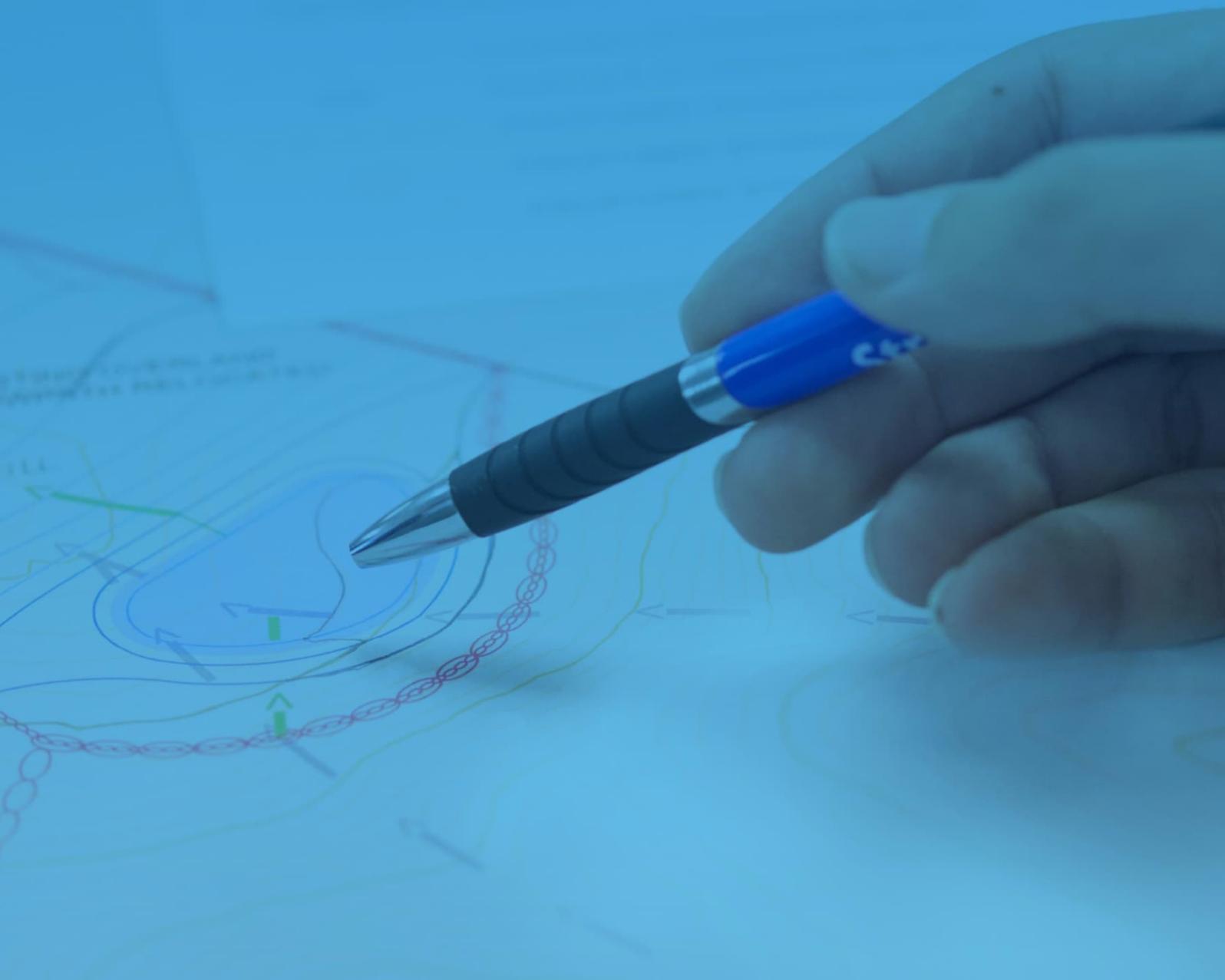
Geological Cross Section

Drawing No. 678164-GEO-D001	
Sheet No. 01	Issue A
A3 SCALE: NTS	

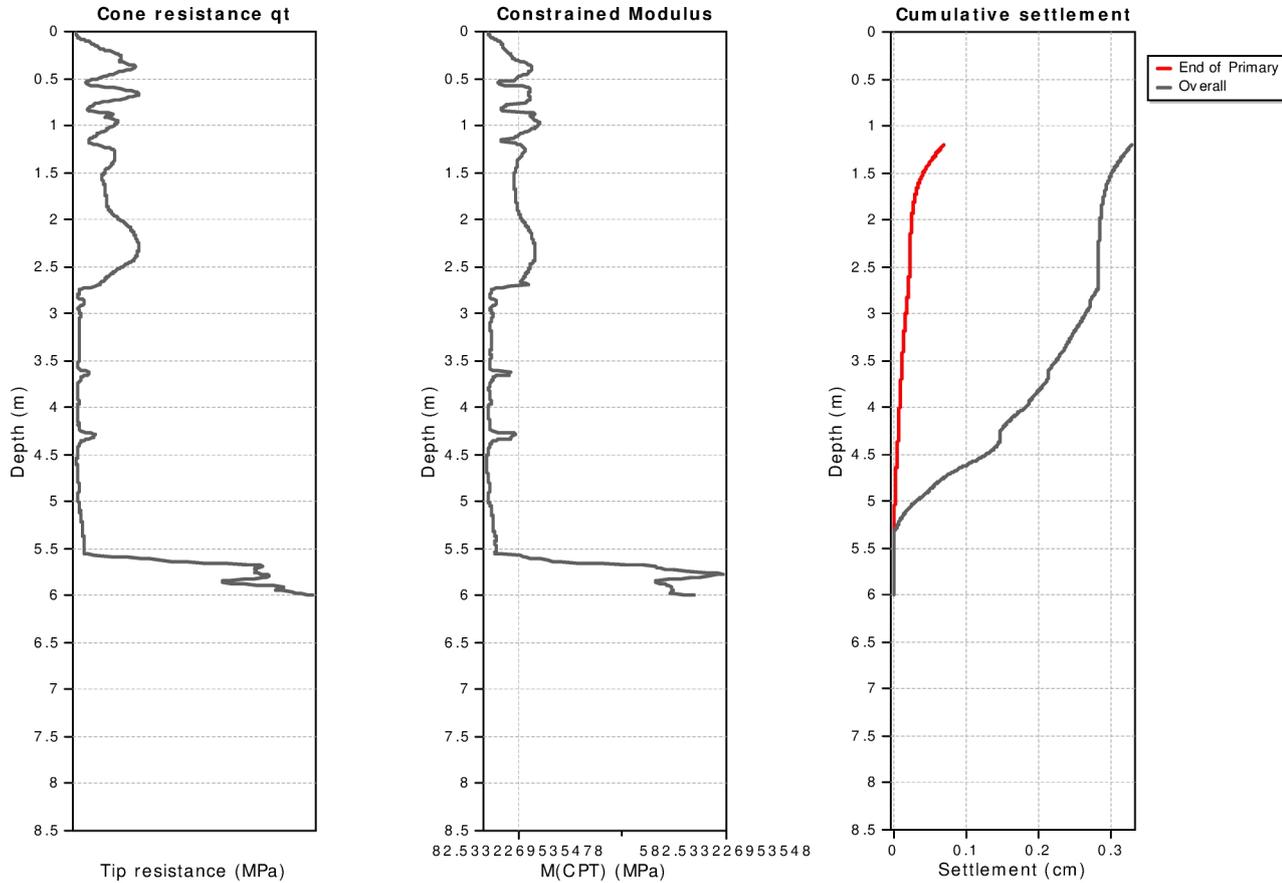


Appendix C

Liquefaction and Settlement Analysis



Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Circular
 Footing diameter: 0.45 (m)
 L/B: 1.0
 Footing pressure: 100.00 (kPa)
 Embedment depth: 1.20 (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

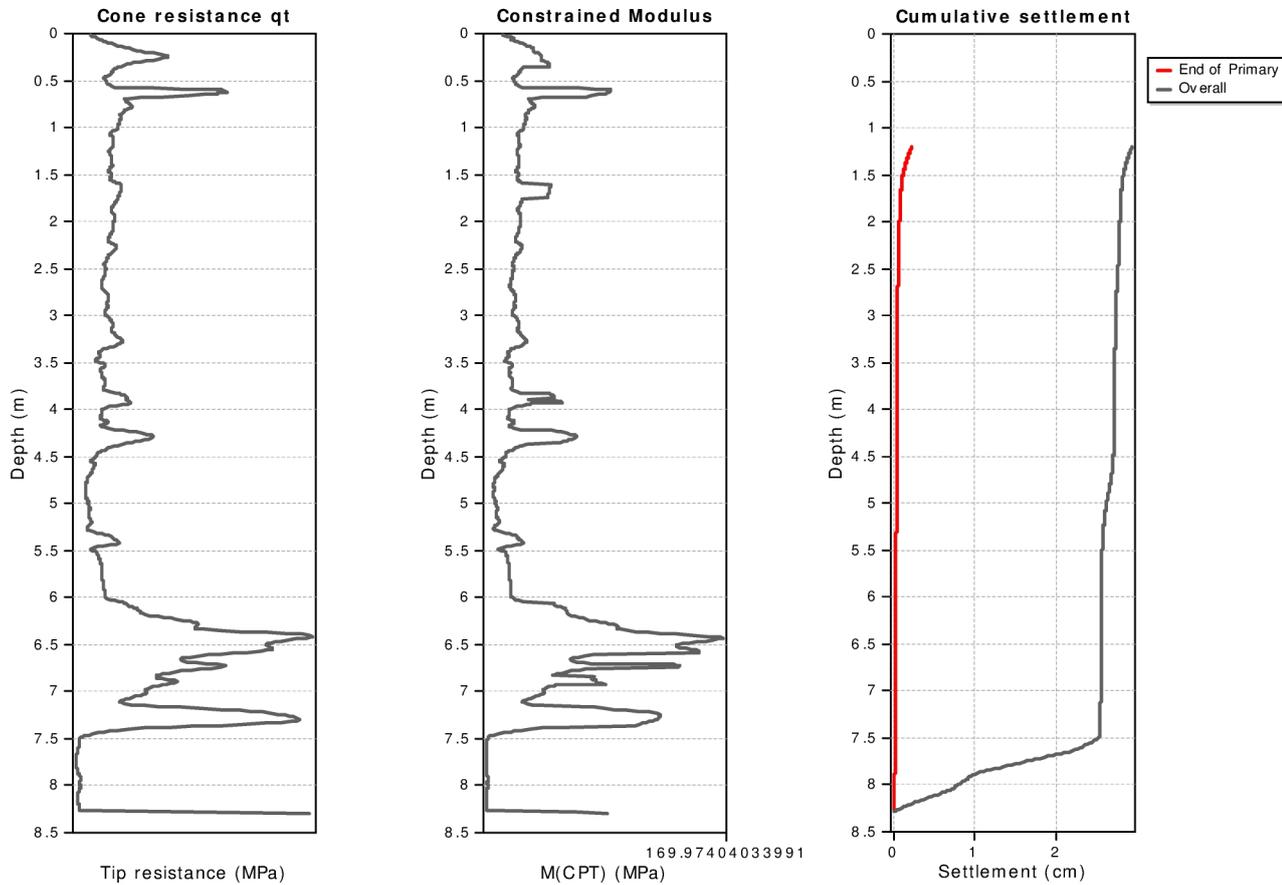
* Primary settlements calculation is performed according to the following formula:

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

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

$$S = C_a \cdot \Delta z \cdot \log(t)$$

Settlements calculation according to theory of elasticity*



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 L/B: 1.0
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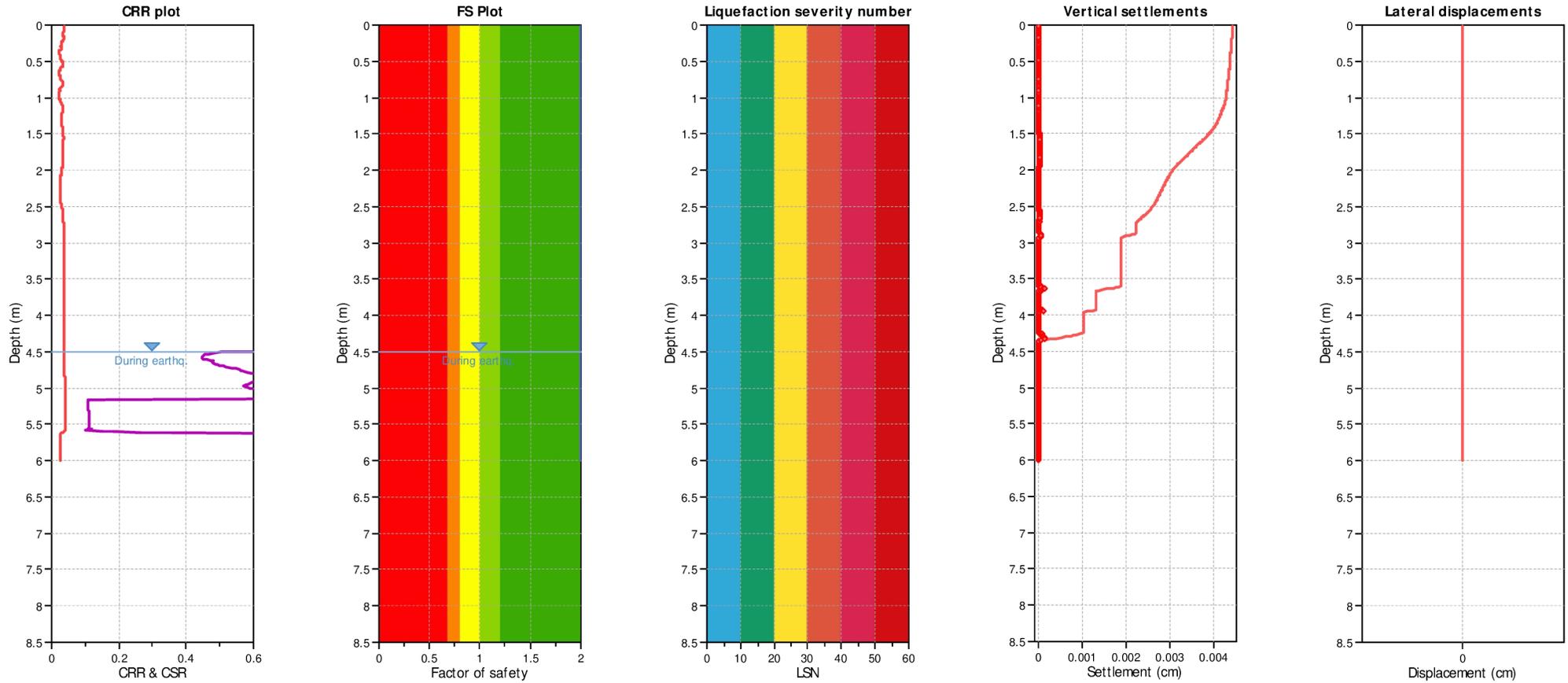
* Primary settlements calculation is performed according to the following formula:

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

$$S = C_a \cdot \Delta z \cdot \log(t)$$

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_v applied:	Yes
Earthquake magnitude M_w :	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.07	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	7.00 m	Fill height:	N/A	Limit depth:	N/A

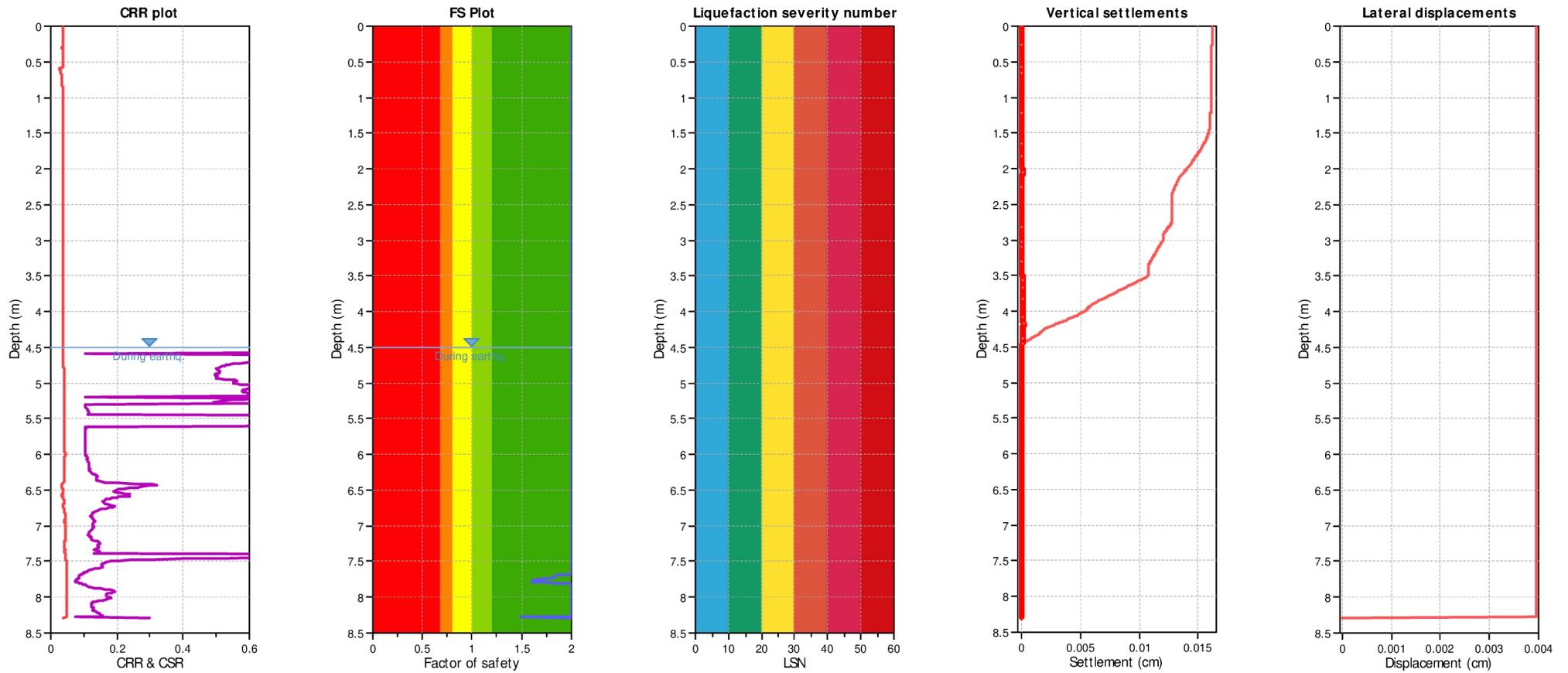
F.S. color scheme

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

LSN color scheme

- Severe damage
- Major expression of liquefaction
- Moderate to severe exp. of liquefaction
- Moderate expression of liquefaction
- Minor expression of liquefaction
- Little to no expression of liquefaction

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	4.50 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on I _c value	I _c cut-off value:	2.60	K _q applied:	Yes
Earthquake magnitude M _w :	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.07	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.40 m	Fill height:	N/A	Limit depth:	N/A

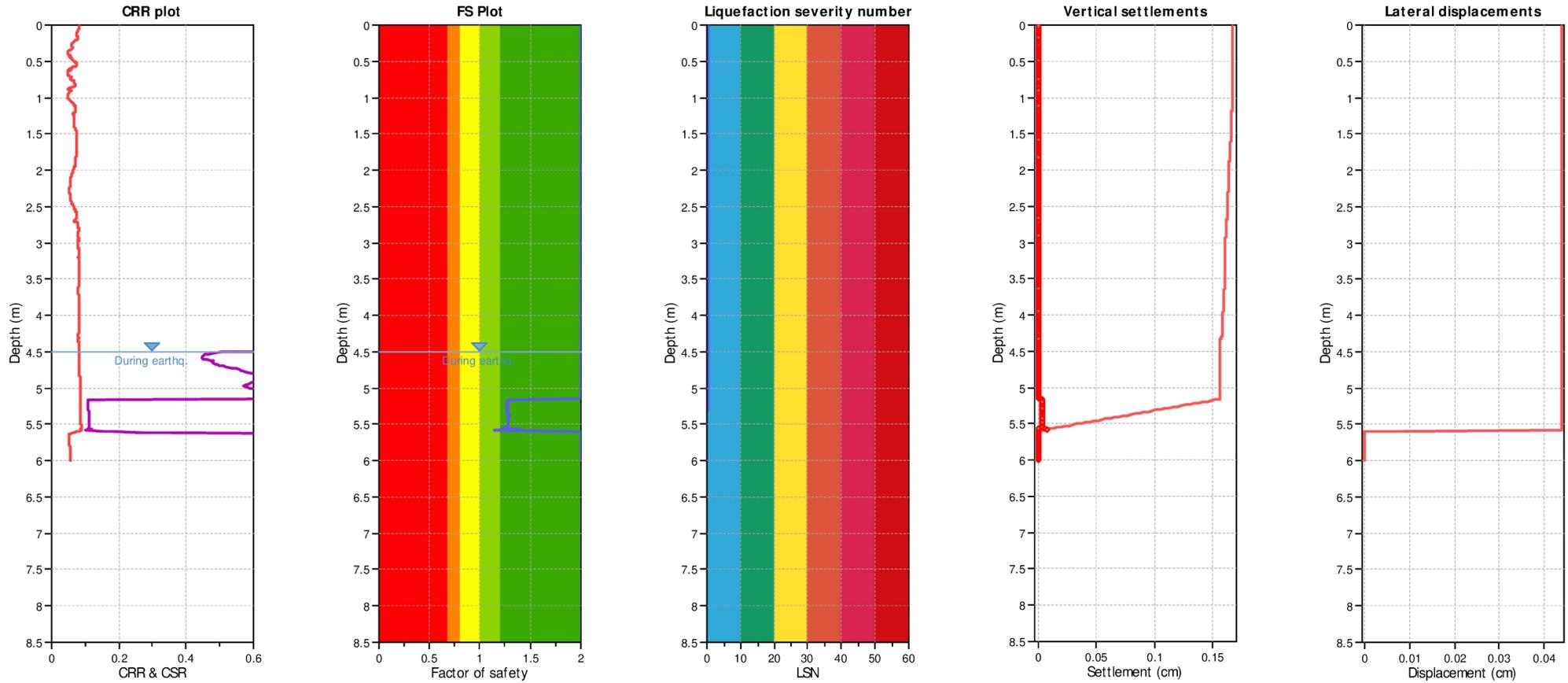
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Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_v applied:	Yes
Earthquake magnitude M_w :	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.15	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	7.00 m	Fill height:	N/A	Limit depth:	N/A

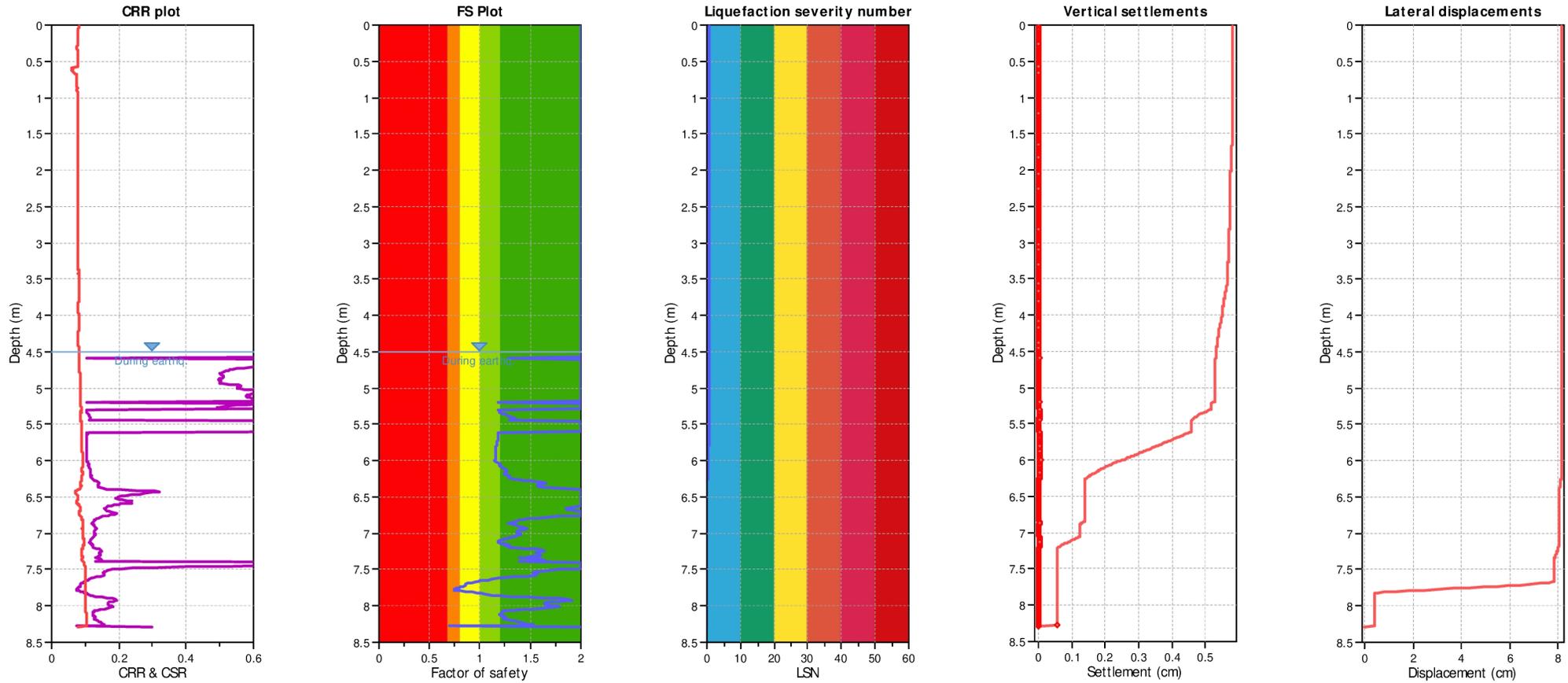
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Liquefaction analysis overall plots



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Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on I _c value	I _c cut-off value:	2.60	K _σ applied:	Yes
Earthquake magnitude M _w :	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.15	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.40 m	Fill height:	N/A	Limit depth:	N/A

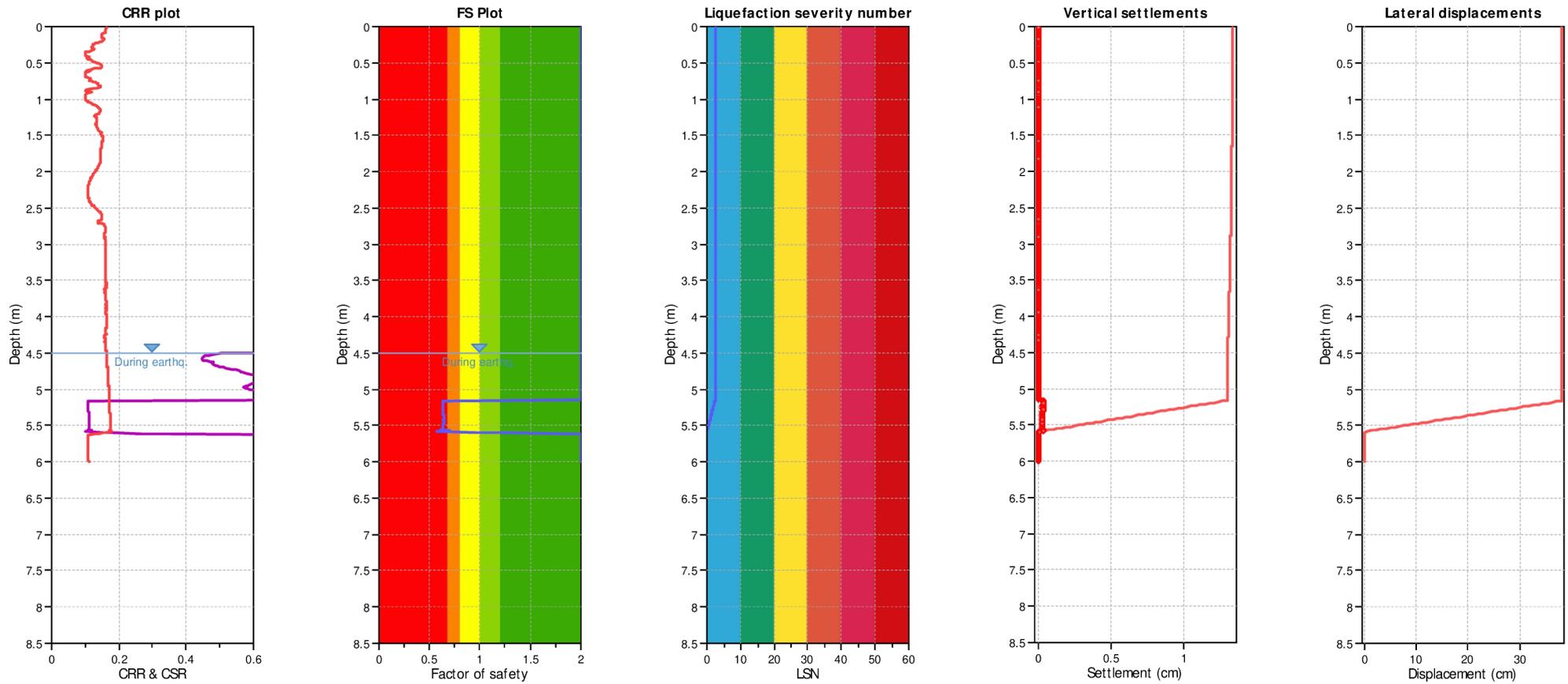
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Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _v applied:	Yes
Earthquake magnitude M _w :	5.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.30	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	7.00 m	Fill height:	N/A	Limit depth:	N/A

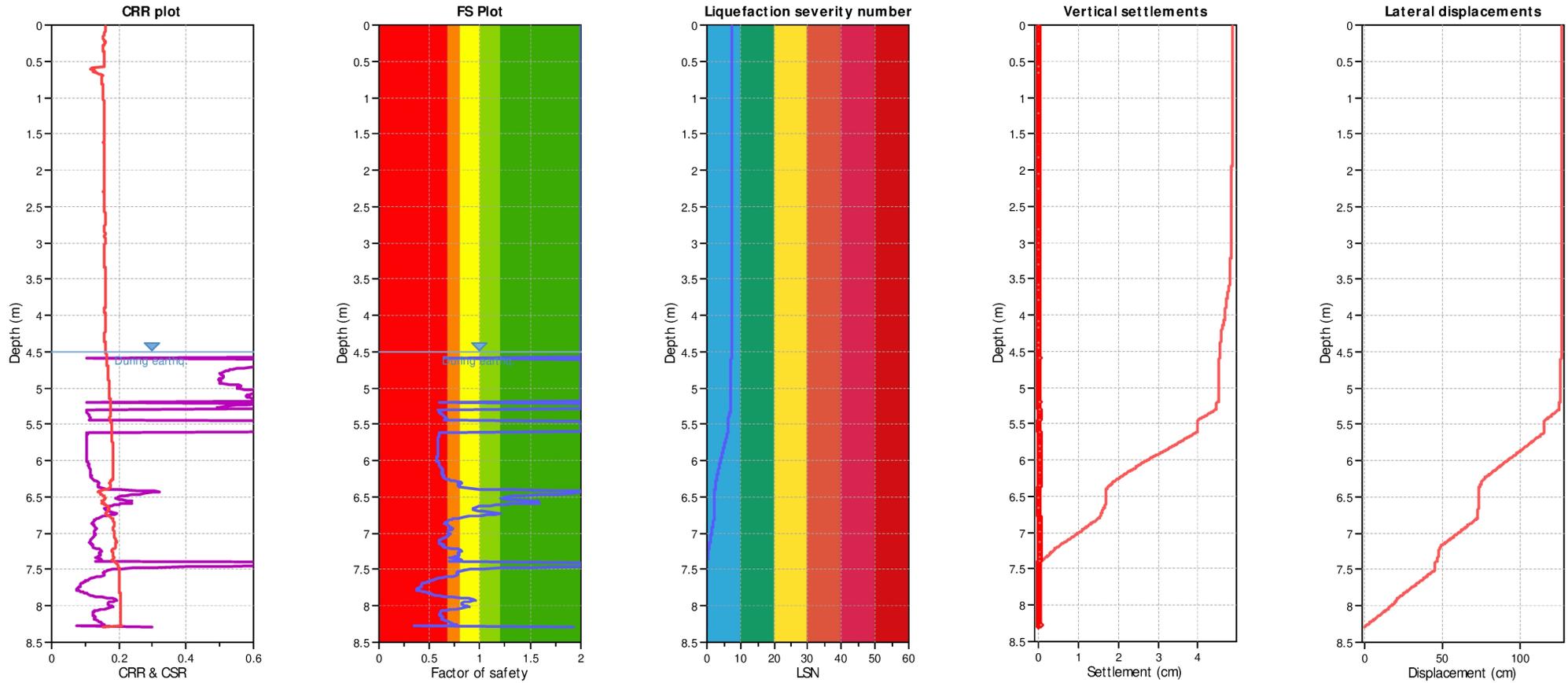
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Liquefaction analysis overall plots



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Peak ground acceleration:	0.30	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	2.40 m	Fill height:	N/A	Limit depth:	N/A

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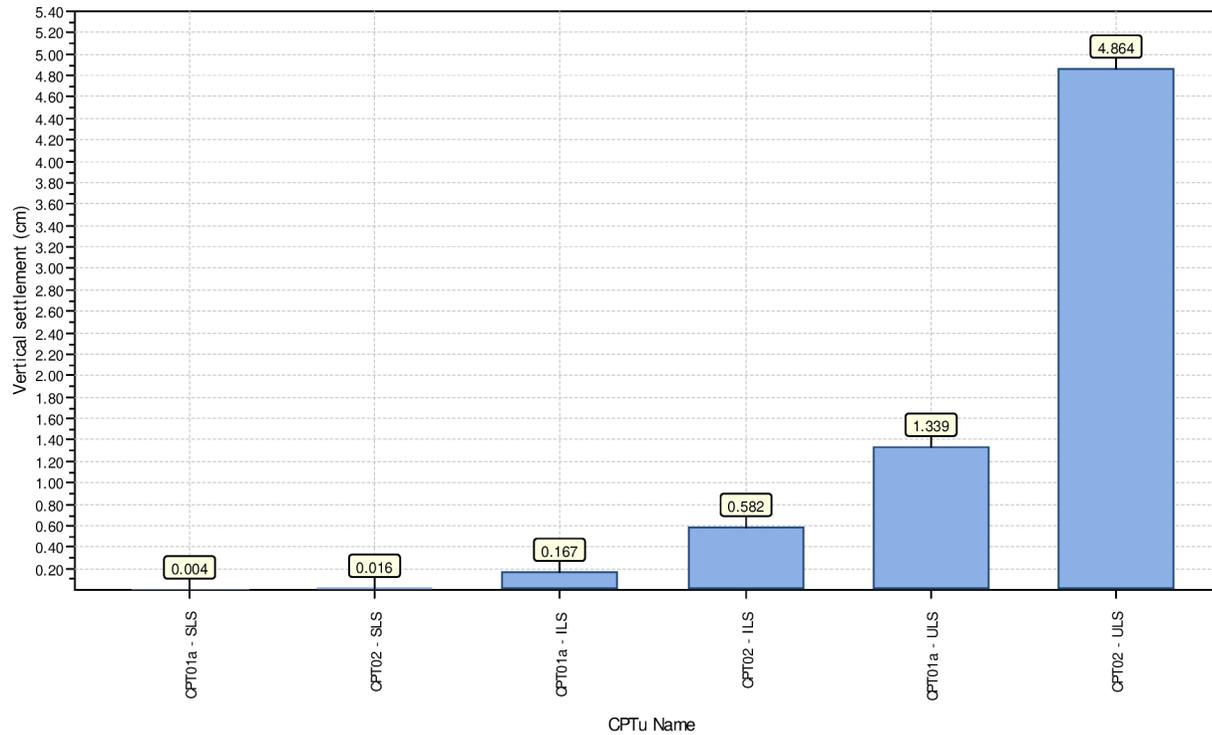
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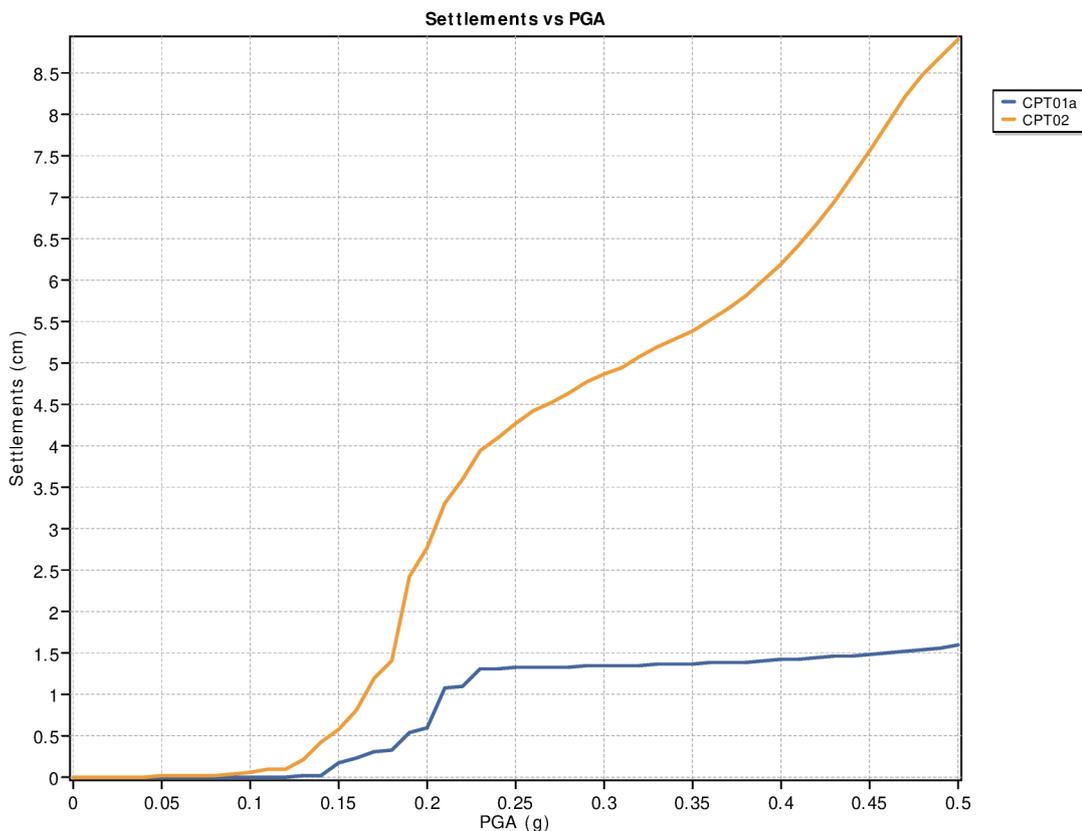
Project title : Liquefaction Assessment

Location : 1 Adams Avenue, Mount Maunganui

Overall vertical settlements report



PGA Based Parametric Analysis



:: CPT main liquefaction parameters details ::

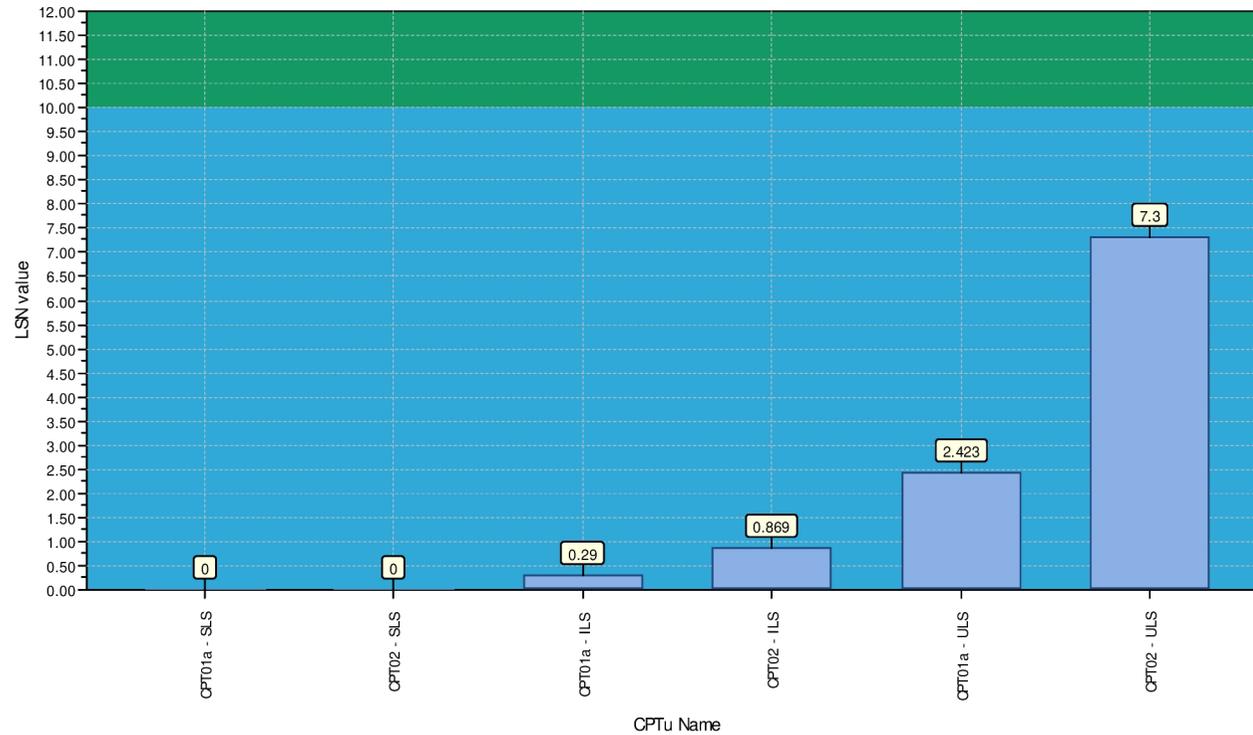
CPT Name	Assesment method	Earthquake Mag.	GWT in situ (m)	GWT earthq. (m)
CPT01a	Boulanger & Idriss (2014)	5.90	7.00	4.50
CPT02	Boulanger & Idriss (2014)	5.90	2.40	4.50



Project title : Liquefaction Assessment

Location : 1 Adams Avenue, Mount Maunganui

Overall Liquefaction Severity Number report



LSN color scheme

- Severe damage
- Major expression of liquefaction
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Basic statistics

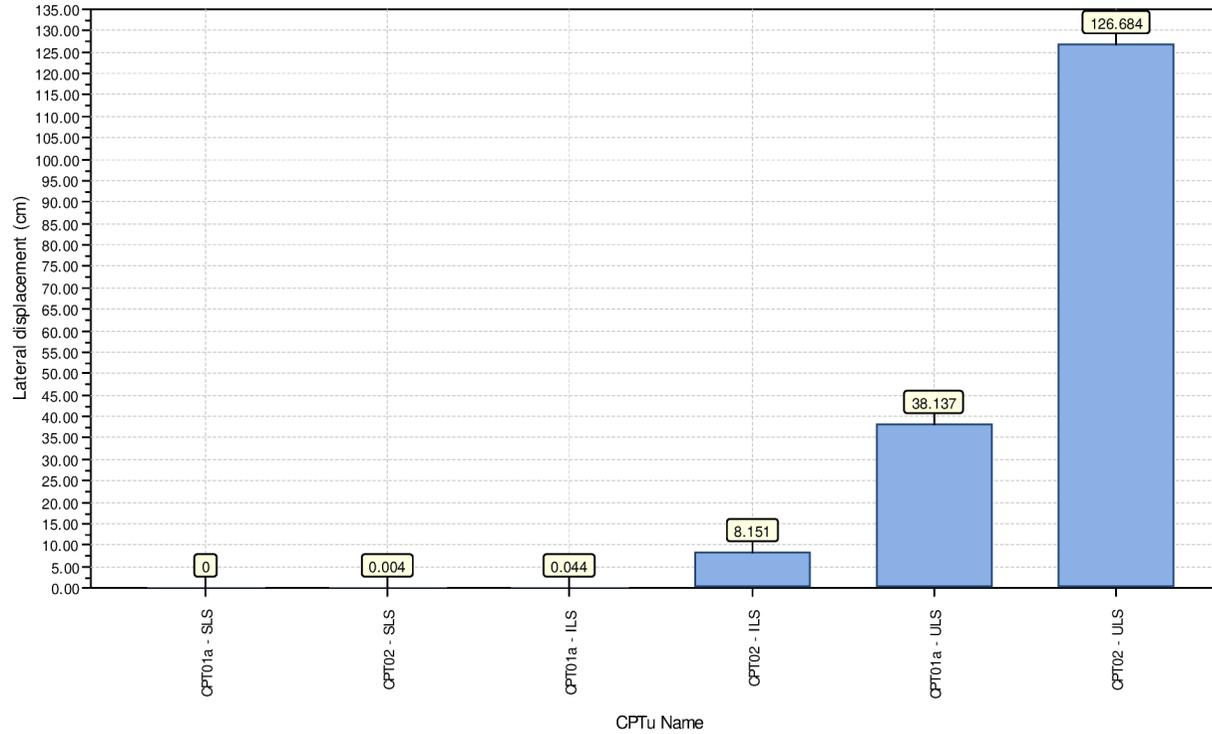
- Total CPT number: 6
- 100% little liquefaction
- 0% minor liquefaction
- 0% moderate liquefaction
- 0% moderate to major liquefaction
- 0% major liquefaction
- 0% severe liquefaction



Project title : Liquefaction Assessment

Location : 1 Adams Avenue, Mount Maunganui

Overall lateral displacements report - front of buildings

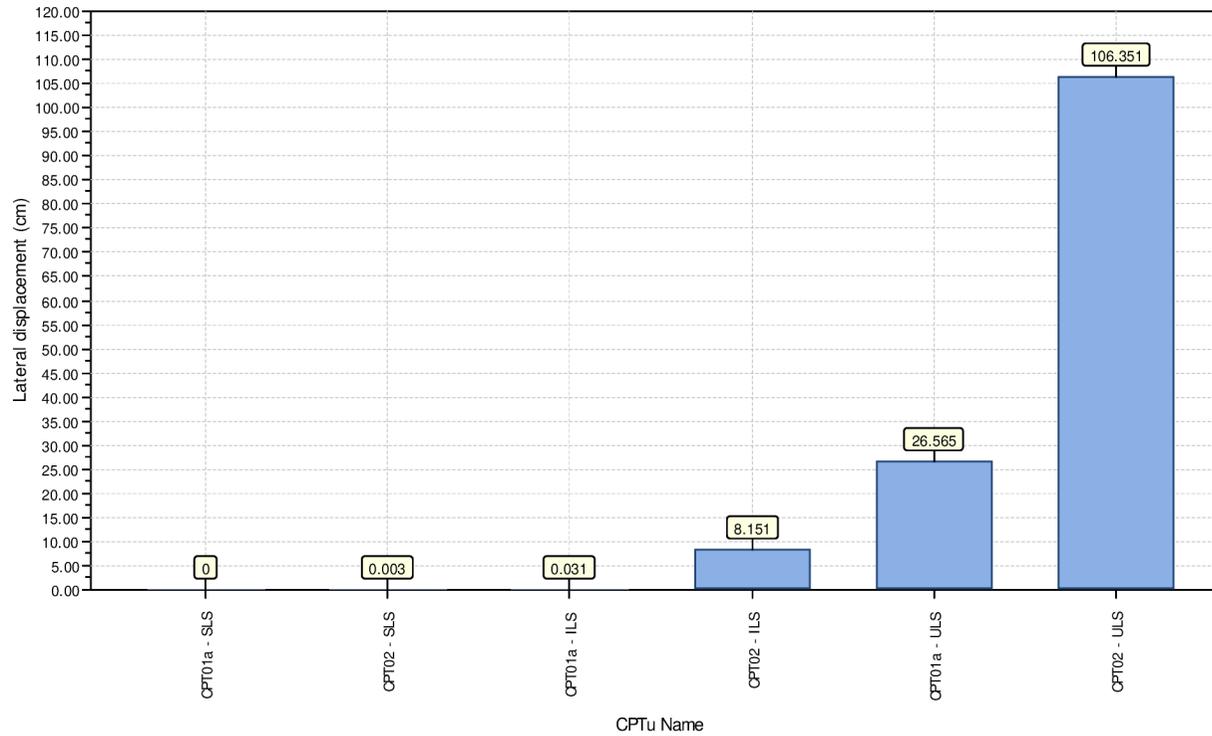




Project title : Liquefaction Assessment

Location : 1 Adams Avenue, Mount Maunganui

Overall lateral displacements report - back of buildings



Stratum

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