Appendix B: Report 2

REPORT

Tonkin+Taylor

Momentum Land Living

Geotechnical Report for Plan Change Application

Prepared for
Momentum Land Ltd

Prepared by
Tonkin & Taylor Ltd

Date
May 2023

Job Number
1019317.1000R v3

Together we create and sustain a better world
www.tonkintaylor.co.nz

Document control

Title: Momentum Land Living					
Date	**Version**	**Description**	**Prepared by:**	**Reviewed by:**	**Authorised by:**
13/09/2022	1	Report for resource consent application	S Burgess	A Sleight	G Lovell
22/02/2023	2	Report for District Plan Variation submission	S Burgess	A Sleight	G Lovell
18/05/2023	3	Rename to South Block	S Burgess	R Brunton	A Sleight

Distribution:

Momentum Land Ltd 1 PDF copy

Tonkin & Taylor Ltd (FILE) 1 electronic copy

Table of contents

Client summary

This summary contains an overview of the key findings and conclusions presented in this report. However, no reliance should be placed on any part of this summary without referring to the relevant sections in the report. Sections within the main body of the report may contain information which puts into context the findings that are encapsulated within this summary.

This assessment summarises the investigations and analyses that have been completed to provide Master Planning conceptual design-level recommendations for the site and is intended to support the submission process to Variation 1 of the Proposed Waimakariri District Plan.

Table 0.1: Design summary

<table>
<tr><th>Considerations</th><th>Single to multi- storey structures</th></tr>
<tr><td>Summary of ground conditions</td><td>
<table>
<tr><th>Soil layer no.</th><th>Typical layer thickness (m)</th><th>Soil description</th></tr>
<tr><td>1a</td><td>0.2 - 0.4</td><td>Firm sandy SILT (topsoil)</td></tr>
<tr><td>1b</td><td>1.0 - 3.0</td><td>Soft to stiff SILT to sandy SILT</td></tr>
<tr><td>2z</td><td>0.0 – 3.0</td><td>Very soft SILT</td></tr>
<tr><td>2a</td><td>0.3 – 3.5</td><td>Loose to medium dense SAND with occasional silt bedding</td></tr>
<tr><td>2b</td><td>5.0 – 9.0</td><td>Dense to very dense SAND to Gravelly SAND</td></tr>
<tr><td>2c</td><td>0 – 1.5</td><td>Firm SILT</td></tr>
<tr><td>3a</td><td>unknown</td><td>Very dense GRAVEL</td></tr>
</table>
</td></tr>
<tr><td>Groundwater</td><td>For the purposes of geotechnical analysis, we have assumed a depth to shallow groundwater of 0.1 mbgl occurring below layer 1b.</td></tr>
<tr><td>Seismic site subsoil class</td><td>Class D – deep or soft soil.</td></tr>
<tr><td>Liquefaction</td><td>Minor to moderate liquefaction-related land damage may be expected to occur above an SLS level event (1/25 years) and above. Liquefaction response worsens in the south eastern portion of the site (zone 2 and 3). This response was reflected in aerial photography taken after the September 2010 earthquake which shows larger quantities of ejecta in this region.</td></tr>
<tr><td>Lateral spread</td><td>Lateral spread risk is created due to the increased ground surface level. Without mitigation, lateral spread may be expected to occur along the perimeter of the buildings toward the swale. We recommend mitigating this with deep ground improvement such as stone columns around the entire building perimeter. If future geotechnical investigations are favourable it is possible that the north and western boundary areas could be reduced to shallow ground improvement such as geogrid.</td></tr>
<tr><td>Static settlement</td><td>Preloading is expected on buildings identified to be in zones 2 and 3.</td></tr>
<tr><td>Development recommendations</td><td>With our prior understanding of the local ground conditions, we consider the density of subsurface investigations at the site to be sufficient and that the ground conditions discussed in this report can be mitigated through appropriate geotechnical engineering design. On this basis we consider the information available supports the rezoning of the site from Rural to Medium Density Residential.</td></tr>
</table>

Considerations	Single to multi- storey structures
	More geotechnical investigations are required at the detailed design phase to support a subdivision application.
Selected foundation/ ground improvement system (for zones, refer Figure 3.1)	
1 storey buildings, adjacent to the swale	Zone 1: TC2 type concrete slabs on the fill platform. Deep ground improvement (e.g. stone columns) to mitigate lateral spread effects, however this may be reduced to shallow ground improvement if ground conditions are favourable Preload is not likely to be required. Zone 2: Deep foundations (piles) or deep ground improvement (e.g. stone columns). Preload is likely to be required. Zone 3: Deep foundations (piles) or deep ground improvement (e.g. stone columns), however this may be reduced to shallow ground improvement. Preload is likely to be required.
1 storey buildings, not adjacent to the swale	Zone 1: TC2 type concrete slabs on the fill platform. Preload is not likely to be required. Zone 2: Deep foundations (piles) or deep ground improvement (e.g. stone columns), however this may be reduced to shallow ground improvement. Preload is likely to be required. Zone 3: Deep foundations (piles) or deep ground improvement (e.g. stone columns). Preload is likely to be required.
1 storey apartment buildings	Zone 1: Deep foundations (piles) - ground improvement and shallow foundations may be possible. Preload is not likely to be required.
2 storey apartment buildings	Zone 3: Deep foundations (piles), or a raft with a robust slab foundation may be possible. Preload is likely to be required to mitigate settlement of surface connections.
3 to 6 storey apartment buildings	Zone 1: Deep foundations (piles). Preload is not likely to be required. Zone 2: Deep foundations (piles). Preload is likely to be required to mitigate settlement of surface connections.

1 Introduction

This report presents the results of the initial geotechnical investigation and assessment completed by Tonkin & Taylor Ltd (T+T) for the proposed development of a block of land referred to as South Block, Moore Land, located to the north-east of Kaiapoi town centre. The work has been completed to support the submission to Variation 1 to the Proposed Waimakariri District Plan, which seeks to rezone the site to Medium Density Residential land use via a resource consent application process.

The work described in this document was commissioned by Momentum Land Ltd (MLL) and was completed in accordance with the letter of engagement dated 25 March 2022, job number 1019317.1000 and variation order (VO) 5, dated 27 March 2023.

1.1 Scope of work

The following scope of work has been completed by T+T for the purposes of this report:

- Geotechnical investigation comprising 1 borehole, 12 Cone Penetration Tests, geophysics, and laboratory testing.
- Preparation of geological profiles.
- Liquefaction analysis and lateral spreading assessment.
- Identification of foundation options for the proposed development.
- Geotechnical assessments including settlement, bearing capacity and CBR recommendations; and conceptual foundation options.
- Assessment of the site against Sections 106 1a) and 1b) of the Resource Management Act (RMA).
- Preparation of this geotechnical report outlining the findings of the above work.

1.2 Site description

The site is located at 310 Beach Road and comprises one block of land covering a total area of approximately 6 hectares. The site is accessed from Beach Road, approximately 0.8 km north-east of Kaiapoi town centre.

The site is bounded by Kaiapoi North school to the north, Beach Grove subdivision to the east, Beach Road to the south and residential homes to the west. The site is currently used as farmland and has two residential homes situated near the southern boundary.

The site is predominantly flat and is on average 0.8 m lower in elevation than the surrounding area; the exception to this is the undeveloped section of the Beach Grove subdivision to the east which is at a similar elevation.

1.3 Proposed development

The proposed development may include construction of stand-alone, attached, or semi-detached single storey dwellings and apartments of 2-6 storey buildings.

2 Assessment and interpretation of site conditions

2.1 Ground and groundwater conditions

2.1.1 Geology and faulting

Published geology of the Kaiapoi region[1,2] describes the site geology as alluvial estuarine and coastal Holocene Age silt deposits of the Christchurch and Springston Formations. Figure 2.1 shows an extract from the geomorphological map with the site boundary superimposed.

These formations comprise layers of interbedded river deposited alluvial gravel, over bank alluvial silt and freshwater swamp peat, coastal sand deposits, and estuarine sand and silt deposits.

Figure 2.1: Extract from geomorphological map.

2.1.2 Previous geotechnical investigations

Geoscience Consulting NZ Ltd undertook geotechnical investigations at the site in 2012 which comprised:

- Two Cone Penetrometer Tests (CPT) to a maximum depth of 9.25 m.

[1] Brown, L.J., 1973: Sheet S76 Kaiapoi (1st Edition) "Geological Map of New Zealand" 1:63,360 Department of Scientific and Industrial Research, Wellington, New Zealand.

[2] Barrell, D.J.A., 2015. Geomorphological map of eastern Canterbury. In: Begg, J.G.; Jones, K.E.; Barrell, D.J.A. (compilers) 2015. Geology and geomorphology of urban Christchurch and eastern Canterbury. GNS Science geological map 3. 1 DVD-ROM. Lower Hutt, New Zealand: GNS Science.

2.1.3 Current geotechnical investigations

Site-specific geotechnical investigations were carried out by T+T in March and April 2022 and comprised:

- 12 CPTs extending to a maximum depth of 10.0 mbgl (metres below ground level).
- One machine drilled borehole extending to a depth of 15.2 mbgl with Standard Penetration Tests (SPT) at 1.5 m centres. A double nested piezometer was installed with response zones between 2.8 - 3.3 m and between 5.5 - 6.5 mbgl.
- Laboratory testing of soils consisting of:
  - 3 No. Particle Size distribution tests.
  - 2 No. Atterberg limit tests.
- Geophysical testing, including:
  - 5 Multi-channel Analysis of Surface Waves (MASW) transects with a total survey length of 811 m.
  - 26 Ground Penetrating Radar (GPR) transects with a total survey length of 1456 m.

The locations of these tests are shown in Appendix A and the logs are provided in Appendix C. Geotechnical laboratory results are in Appendix D.

2.1.4 Geotechnical model

A preliminary ground model has been developed for the site based on the geotechnical investigations described above. Three cross-sections were developed from these site investigations, these are presented in Appendix B and a generalised site soil profile is summarised in Table 2.1 below.

Table 2.1: Generalised subsurface profile

Layer No.	Description	Inferred geological unit	Approx. depth to top of layer (m bgl)	Approx. layer thickness (m)	Approximate qc (MPa)
1a	Firm sandy SILT (topsoil)	Springston Formation	0.0	0.2 - 0.4	-
1b	Soft to stiff SILT to sandy SILT		0.2 - 0.4	1.0 - 3.0	0.5 - 4
2z	Very soft SILT	Christchurch Formation	0.0 - 6.0 (non-continuous layer)	0.0 – 3.0	0 - 1
2a	Loose to medium dense SAND with occasional silt bedding		1.0 - 3.0	0.3 – 3.5	3 - 15
2b	Dense to very dense SAND to Gravelly SAND		2.5 – 5.0	5.0 – 9.0	15 - 30
2c	Firm SILT		11.5 – 13.0 (likely non-continuous layer)	0 – 1.5	1- 2
3a	Very dense GRAVEL	Burnham Formation	13.0 – 15.0	Unconfirmed	20 -30+

Large, buried objects were identified under GPR between 1.0-3.0 mbgl. Based on previous experience on stage 4 of the nearby Beach Grove subdivision, these may be large buried trees.

2.1.5 Groundwater

2.1.5.1 Site observations

Observations made at the site during the CPT and BH investigations noted a variable depth to ground water from approximately 0.4 – 1.0 mbgl. The soil profile and our previous experience at the site indicates that ground water has an artesian component which results in ground water pressure readings recorded on the CPT traces which suggest shallow ground water levels. The near surface soil (Layer 1b) has low vertical permeability and, if left undisturbed, is expected to prevent the groundwater from rising above 1.0 mbgl over most of the site (the highest base of Layer 1b encountered in the investigations).

Groundwater monitoring undertaken between May 2021 and January 2022 on the nearby Beach Grove site have shown the semi-confined ground water level ranges between 0.8 mbgl and 0.2 m above ground level (agl).

In summary, artesian groundwater may be expected to be encountered between 1.0 m to 3.0 mbgl.

2.1.5.2 Groundwater levels summary

As ground water levels can vary seasonally and in response to seismic shaking, a groundwater level of 0.1 mbgl occurring below layer 1b has been adopted for design purposes.

Due to the relatively shallow groundwater and the possibility of artesian pressures at the site we recommend that careful consideration be paid to the effect of any earthworks activities undertaken at the site, particularly in relation to services installation, basements, tree pits, lighting or power poles and the like. Where practical it would be preferable to avoid deep penetrations through the low-permeability near-surface soils which help to seal the occasionally artesian groundwater pressures below.

2.2 Seismicity

2.2.1 Seismic site subsoil class

In terms of NZS 1170.5[3] the site subsoil class is assessed to be Class D (deep or soft soil). This recommendation is based on published geological information[4] that indicates the depth to bedrock is greater than 100 m beneath the site.

The site is not considered to be Class E (Very soft soil) because the soft soil deposits are less than 10 metres thick. The site is not considered to be Class C (Shallow soil site) because the maximum depth of soil is exceeded.

2.2.2 Ground shaking hazard

Evaluation of the expected seismic performance of the site (including liquefaction effects) is guided by the seismic shaking hazard assessed for the site and the requirements of the New Zealand Building Code, which considers the design earthquake scenarios derived from “NZS 1170 – Structural Design Actions” representing the following design performance requirements:

- Serviceability limit state 1 (SLS1) – the building should suffer little or no structural damage and remain accessible and safe to occupy. There may be minor damage to building fabric that is readily repairable.

[3] Standards New Zealand (2004) – *NZS 1170.5:2004 – Structural Design Actions Part 5: Earthquake Actions – New Zealand.*
[4] Brown, L. J. and Weeber, J. H. (1992), *Geology of the Christchurch Urban Area.* Institute of Geological & Nuclear Sciences Limited Geological Map 1. Scale 1:25,000.

- Ultimate limit state (ULS) – the building is expected to suffer moderate to significant structural damage, but not to collapse.

The design earthquake scenarios are described in terms of an event moment magnitude (M_w) and peak horizontal ground acceleration (PGA_H) and were derived assuming a building design life of 50 years and an Importance Level (IL) of IL2 and IL3 as set out in NZS 1170. Two SLS1 scenarios (SLS1a and SLS1b) were assessed. The SLS1b scenario represents an alternative SLS1 scenario that is also considered when using the Boulanger and Idriss (2014)[5] liquefaction triggering analysis in the Christchurch area, in accordance with guidance updates released by MBIE[6]. ULS scenarios were assessed for both IL2 and IL3 developments. In addition, a 100-year return period event was also considered to evaluate the consequences of liquefaction in an intermediate earthquake level between the SLS and ULS cases.

The earthquake scenarios adopted for analysis are presented in Table 2.2 below.

Table 2.2: Liquefaction Design earthquake scenarios

	SLS1a	SLS1b	100 yr	ULS IL2	ULS IL3
Return period (years)	25	25	100	500	1000
Moment magnitude (M_w)	7.5	6.0	6.0	7.5	7.5
Peak horizontal ground acceleration (PGA_H)	0.13 g	0.19 g	0.30 g	0.35 g	0.44g

2.3 Liquefaction assessment

2.3.1 Liquefaction observations during the Canterbury earthquakes

2.3.1.1 Aerial photographs

A review of satellite and aerial photographs[7] taken following the 4 September 2010 earthquake event indicates evidence of moderate surface ejecta across the site largely concentrated in the south eastern third of the site.

2.3.1.2 Shaking intensity

The estimated conditional PGA_H levels that Christchurch experienced during each major event within the Canterbury earthquake sequence (CES) have been modelled by Bradley and Hughes (2012)[8] based on records from ground motion recording stations all over the city.

The conditional mean PGA_H levels modelled show that during the 4 September 2010 event the shaking intensity may have been around 0.25 g at the site. For the 22 February 2011 event the site may have experienced a peak shaking intensity of 0.19 g. The 13 June and 23 December 2011 events may have generated around 0.1 – 0.15 g shaking intensity at the site.

We note that the shaking intensities that the site likely experienced during the 4 September 2010 event (170% of SLS level) means that it has been “sufficiently tested at SLS” according to Section 13.5.1 of the MBIE Guidance. This means that liquefaction-related land damage at the site in a future

[5] Boulanger, R. W. and Idriss, I. M. (2014). *CPT and SPT Based Liquefaction Triggering Procedures.* Center for Geotechnical Modeling, Dept. of Civil and Environmental Engineering, University of California at Davis.
[6] Ministry of Business, Innovation & Employment (2014). Clarifications and updates to the guidance. *Repairing and rebuilding houses affected by the Canterbury earthquakes.*
[7] New Zealand Geotechnical Database (2012). *Aerial Photography.* Map Layer CGD0100 – 1 June 2012. Retrieved 25 July 2022 from https://www.nzgd.org.nz/.
[8] Bradley and Hughes (2012). *Conditional Peak Ground Accelerations in the Canterbury Earthquakes for Conventional Liquefaction Assessment* – Technical Report for the Ministry of Business, Innovation and Employment.

SLS earthquake is expected to be no worse than what the site has already experienced. However, loss of surface crust due to construction activities in the areas where the ground surface is not built up with hardfill may affect the amount of ejecta observed on the surface in a similar sized event. We understand that this situation will not apply to any of the buildings or roads to be constructed on the site, however it may apply to basements (if these are included in future buildings) or drainage areas.

2.3.2 Liquefaction susceptibility

Seismic liquefaction occurs when excess pore pressures are generated in loose, saturated, generally cohesionless soil during strong earthquake shaking, causing the soil to undergo a loss of shear strength and stiffness. This loss of shear strength and stiffness can result in settlement and/or horizontal movement (lateral spreading) of soil. The occurrence of liquefaction is dependent on several factors, including the intensity and duration of ground shaking, soil density, particle size distribution and depth to groundwater.

Based on experience gained during the CES and the various tools available for characterising and analysing the geotechnical nature of the soils underlying the site, the liquefaction susceptibility of each of the soil layers outlined in the geological model (Table 2.1) is assessed to be:

- Layer 1a (topsoil) is not expected to liquefy.
- Layer 1b (sandy silt to silt) is considered to generally be susceptible to liquefaction. We note that the interbedded nature of this layer means that there are liquefiable sand and silt mixtures interlayered between non-liquefiable (more plastic) silts.
- Layer 2a (sand) is generally considered to be liquefiable given its composition and density.
- Layer 2b, (sand to gravelly sand) comprising sand and gravelly sand is less likely to be susceptible to liquefaction due to the density and material composition, however some lenses within this deposit are likely to liquefy. Ground surface consequences are likely to be reduced by the depth and density of the layer.
- Layers 2z, 2c and 3a (soft silt, firm silt, and gravel) are also not expected to be susceptible to liquefaction due to the composition and density of these deposits. The silts encountered in these layers generally have plasticity.

2.3.3 Liquefaction triggering

The liquefaction triggering analyses have been carried out using the methodologies presented in Boulanger and Idriss (2014)[9], with corresponding one-dimensional, post-liquefaction reconsolidation "index" settlement (S_{V1D}) calculated using Zhang et. al. (2002)[10]. The liquefaction analyses adopted a fine fitting parameter (C_{FC}) value of 0 and used a probability of liquefaction triggering (P_L) of 15% in accordance with typical design practice. Liquefaction severity number (LSN) and consideration of crust thickness have been used as a guide to assess the expected liquefaction-induced land damage. A 257-page pdf output of these assessments in natural ground without any fill is held on file and can be made available on request.

LSN is a depth-weighted index that has been developed based on a comparison of liquefaction analyses completed for thousands of CPTs undertaken in Christchurch following the Canterbury Earthquakes with detailed observations of the land and building performance (in terms of liquefaction-damage) during each of the major earthquakes. This index provides a more useful indicator of the potential consequences on the land due to liquefaction rather than simply predicting whether liquefaction is likely to occur and the magnitude of S_{V1D}.

[9] Boulanger, R. W. & Idriss, I. M. (2014) *CPT and SPT Based Liquefaction Triggering Procedures*. Centre for Geotechnical Modelling, Dept. of Civil and Environmental Engineering, University of California at Davis.
[10] Zhang, G. Robertson, P. K. & Brachman, R. W. I. (2002) *Estimated liquefaction-induced ground settlements from CPT for level ground*. Canadian Geotechnical Journal, 39, 1168-80.

In general, excluding lateral spreading effects, the results of the liquefaction triggering analysis indicate that:

- There is a variable response across the site which has informed initial indications for geotechnical zones discussed in Section 3.3.
- In general, liquefaction is expected to be triggered in a proportion of Layers 1b sandy silts and 2a sands under SLS level shaking. Additionally, lenses of Layer 2b sand are expected to liquefy under ULS IL2 and IL3 shaking.
- The cumulative thickness of the materials expected to liquefy increases as the shaking intensity level increases from SLS to ULS IL3, with most of the development of liquefiable layers occurring between SLS (25 year) and 100 year return periods.
- The placement of imported fill improves the sites liquefaction response.

2.3.4 Liquefaction consequences

Once liquefaction has triggered, the consequences of liquefaction (without any fill added to the surface) can include:

- Ground surface damage including total and differential settlement.
- A sudden reduction in bearing capacity of the liquefied soils.
- Lateral spreading of soils toward free faces.

We have assessed the potential for these consequences at this site with fill placed to 2.4 m RL (LVD) and have summarised the results in Table 2.3. Without fill placed on the site the liquefaction performance would correspond to a TC3 site.

The effect of the proposed earthworks and for the site are discussed in Section 3.4 below.

2.3.4.1 Lateral spreading

Lateral spreading is generally defined as the horizontal displacement of blocks of surficial soil towards an open slope face because of liquefaction of the underlying soils. The occurrence of lateral spreading generally requires the presence of a relatively continuous liquefiable layer extending to an open slope face such as a riverbank or open channel. Displacements can range from a few centimetres to a metre or more. The MBIE guidelines define lateral stretch as “The degree of lateral stretching of the ground which may occur across a building footprint in an earthquake” as opposed to global lateral movement which is defined as “where the entire superstructure and foundation is able to move as one along with the global movement of the block”. The MBIE guidelines state that to be categorised TC2, the lateral stretch over the building footprint must be less than 50 mm in a SLS earthquake event and 100 mm in an ULS earthquake event.

Typically, the site and surrounding areas are flat, although the site is to be raised by approximately 1.5 m. While the site contour plans are not yet finalised, it should be assumed that lateral spread risk exists at all boundary edges and on any open slope faces and needs to be considered as part of future site development.

The conceptual site configuration has a stormwater swale around the perimeter of the site. This creates the potential for lateral spreading to occur along the edges of the site, with buildings moving towards the swale under liquefied conditions. Lateral spread mitigation methods such as Stone Column ground improvement are anticipated beneath any buildings close to the exterior of the site, as shown in Figure 2.2. Deep ground improvement to mitigate lateral spread should be expected in these areas as part of this site development. It is possible after further geotechnical investigations that mitigation measures may be reduced to shallow ground improvement in favourable areas with less liquefaction hazard. Any changes to the extent of the swale may result in changes to the scope and extent of any ground improvement.

The swale should be constructed in a configuration so that a lateral spread risk is not worsened for the surrounding properties. Where the swale depth is deeper than the existing ground surface, ground improvement measures may be required to protect existing properties.

The potential distribution of lateral ground displacement may impact differently on different structures and underground services, and this will need to be considered during future design phases.

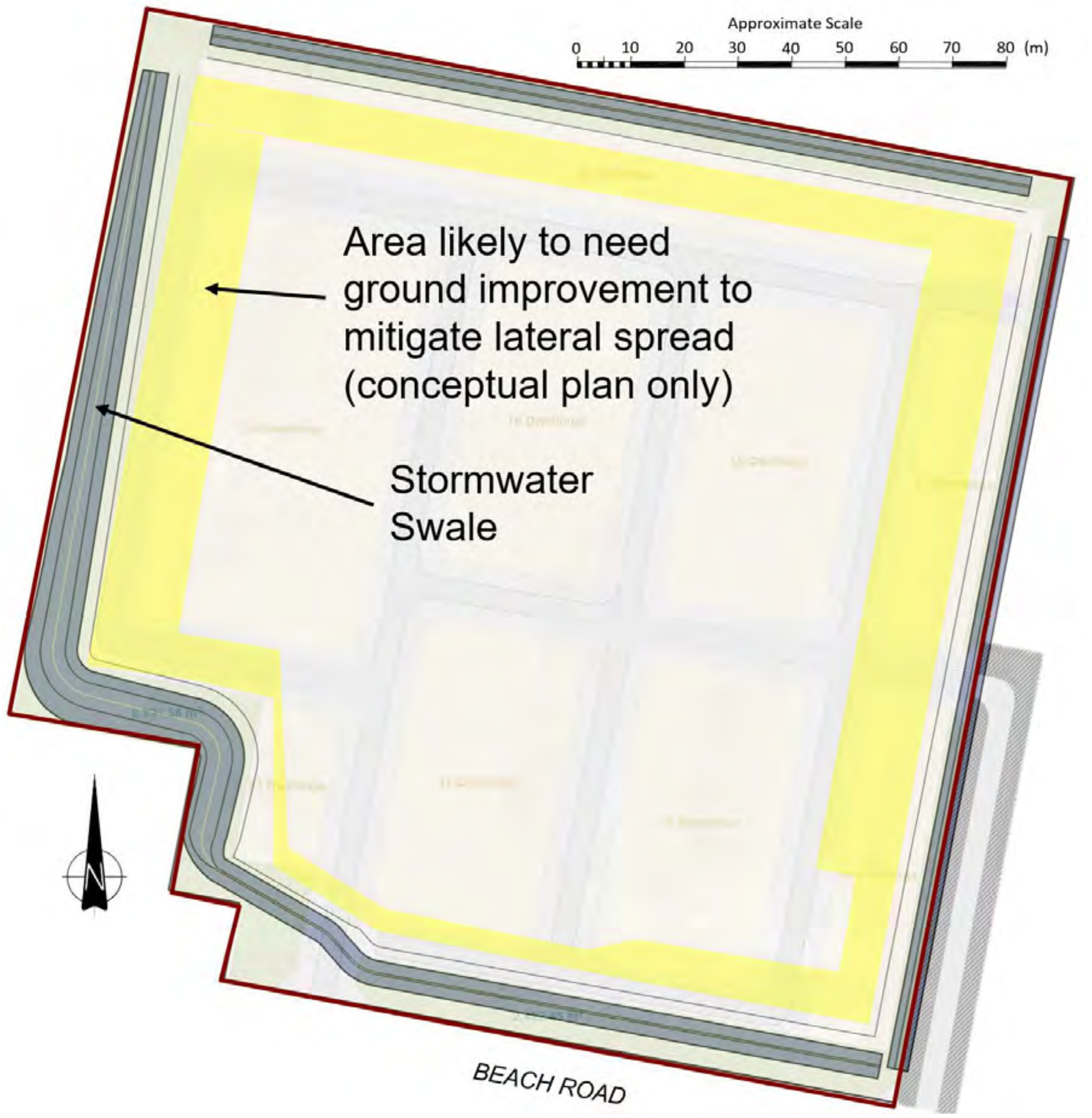


Figure 2.2: Potential lateral spread zone ground improvement areas – concept only.

Table 2.3: Liquefaction consequences summary (using groundwater level of 0.5 mbgl (current ground level) and a fill level of 2.4 m RL (LVD))

Liquefaction consequence	Method	Results				Commentary	Implications for this site
		SLS (25 yr) M_w=6.0, PGA=0.19g	**ILS (100 yr)** M_w=6.0, PGA=0.30g	**ULS, IL2 (500 yr)** M_w=7.5, PGA=0.35g	**ULS, IL3 (1000 yr)** M_w=7.5, PGA=0.44g		
Ground surface damage including total and differential settlement.	Crust Thickness, CT [11]	Range: 1.5 to 3.4 m Average: 2.2 m	Range: 1.4 to 2.5 m Average: 1.9 m	Range: 1.4 to 2.5 m Average: 1.9 m	Range: 1.4 to 2.5 m Average: 1.9 m	Observations from Christchurch and other earthquakes are that the greater the depth to liquefied soils (crust thickness) the less damage is likely to be reflected at the ground surface. Examples of sand boils and damaging differential settlement are few for sites with a crust thickness >3.5 m.	Ground surface damage (such as sand boils) may be expected in SLS and ULS events.
	Calculated one-dimensional post liquefaction reconsolidation settlement (S_{V1D})[12]	Range: 3 to 65 mm Average: 27 mm	Range: 14 to 93 mm Average: 40 mm	Range: 16 to 102 mm Average: 48 mm	Range: 18 to 103 mm Average: 103 mm	Ishihara (1996) produced guidelines correlating the magnitude of calculated settlement with observed ground damage. 0 – 100 mm settlement was associated with light to no damage with minor cracks on the ground surface.	Light ground surface damage expected due to liquefaction.
	Liquefaction Severity Number (LSN) [13] [14]	Range: 1 to 15 Average: 7	Range: 3 to 21 Average: 11	Range: 6 to 23 Average: 13	Range: 7 to 23 Average: 14	LSN is a parameter calculated on the basis of investigation data considering liquefaction potential and its depth. This parameter has been correlated with evidence of surface ground damage in Christchurch. LSN < 10 indicates 'Insignificant' ground damage, with no significant excess pore water pressures. LSN values of 5 – 15 indicates 'Mild' ground damage, with negligible deformation of the ground and small settlements. LSN values of 10 – 25 indicates 'Moderate' ground damage associated with relatively small differential settlements.	Minor to Moderate ground surface damage expected due to liquefaction in SLS event. 'High' ground surface damage expected due to liquefaction in ULS event.

Note:

All liquefaction triggering analyses were undertaken using the liquefaction triggering method of Boulanger and Idriss 2014, adopting a probability of liquefaction value of PL=15% (in accordance with normal design practice) and the default fines correction fitting parameter of CFC=0. For the calculated SV1D and LSN, the calculation was limited to the top 10 m of the soil profile.)

Results shown for SLS1b, not SLS1a. SLS1b dominates response.

[11] Bowen, H.J. and Jacka, M.E (2013) Liquefaction induced ground damage in the Canterbury Earthquake: Predictions versus reality. Proceedings of the 19th NZGS Geotechnical Symposium. Editor CY Chin. Queenstown, New Zealand.
[12] The values in this table were calculated using the methods described in Ministry of Business, Innovation & Employment (MBIE) Canterbury Guidance - Repairing and rebuilding houses affected by the Canterbury earthquakes, Version 3, December 2012.
[13] van Ballegooy, S., Lacrosse, V., Jacka, M. and Malan, P. (2013) LSN – a new methodology for characterising the effects of liquefaction in terms of relative land damage severity. Proceedings of the 19th NZGS Geotechnical Symposium. Editor CY Chin. Queenstown, New Zealand.
[14] MBIE/NZGS (2021) Earthquake Geotechnical Engineering Practice series: Module 3 – Identification, assessment and mitigation of liquefaction hazards, November 2021, in particular Table 5.1.

2.1 Settlement

Previously, deposits of compressible silts have been identified on the nearby Beach Grove subdivision. Similar smaller deposits of up to 3 metres thickness have been identified in localised areas on this site. Additional investigations are required to rule out the presence of further deposits and to understand the extent and behaviour of the deposits identified by these investigations.

Whilst more investigations are required, initial investigations findings are shown in Figure 2.3 and these areas have informed initial indications for geotechnical zones discussed in Section 3.3.

Figure 2.3: Preliminary areas soft compressible silt deposits.

3 Geotechnical implications for site development

3.1 General

The recommendations and opinions which are contained in this report are based upon data from geotechnical investigations on the site and surrounding areas. The nature and continuity of sub-surface conditions away from the investigation locations is inferred, and it must be appreciated that the actual conditions may vary from the assumed geotechnical model.

3.2 General development considerations

In general, observations made throughout Christchurch during the Canterbury Earthquakes indicated that buildings that were clad with lightweight wall and roof materials performed better under earthquake loading than those which were clad with heavyweight materials. Therefore, we recommend that new structures proposed for the site are also constructed from lightweight materials. If heavyweight materials are to be used, then it should be restricted to single storey structures and the bottom level of multi-storey buildings. Alternatively, if heavyweight cladding materials are used on all levels and/or buildings higher than 2 to 3 storeys, then more robust foundation works are likely to be required to achieve satisfactory seismic performance.

Buildings that have a regular or symmetrical footprint (e.g. rectangular, L or T-shaped) and a smaller plan area have also been observed to perform better during the Canterbury Earthquakes i.e. less damage and generally easier to repair. Therefore, we recommend that regular building shapes be adopted for the proposed development, and consideration be given to any opportunities that might arise to divide large buildings into a number of smaller separate structures.

Mixed foundation systems within the same structure are not recommended, e.g. suspended timber floor with slab on grade, unless appropriate allowance is made for differential movement under strong earthquake shaking.

3.3 Geotechnical “zones” and associated building foundation recommendations

Based on the results of the initial geotechnical investigations, liquefaction assessment, and soft soil assessments discussed above, we have categorised the site into three preliminary geotechnical “zones”. Our delineation of these zones is based on the expected future seismic performance of the ground. The three zones are shown in Figure 3.1.

Conceptual foundation options considered suitable for each zone are presented in Table 3.1 below. These options are based on a possible range of buildings which may be constructed on the site. Further investigations and design will be required as planning progresses. Depending on how the site is developed, there are opportunities to optimise building layouts on the site.

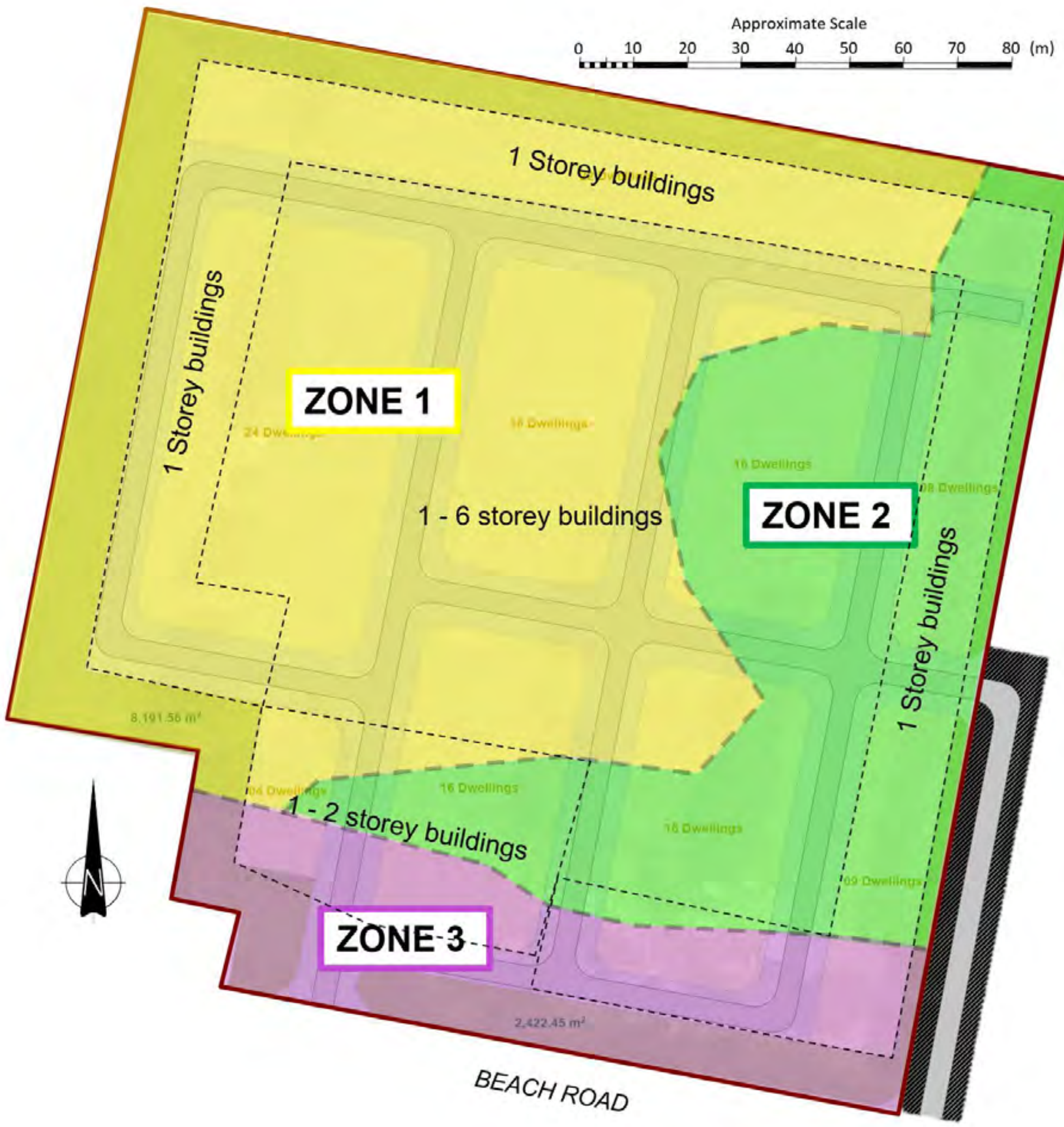


Figure 3.1: Geotechnical Zones and building types.

Table 3.1: Summary of geotechnical "zones" and foundation concepts

Zone number	Potential ground damage from design seismic events[1]	Conceptual foundation description
Zone 1	• Lateral ground stretching is expected at free face slopes. • At ILS and ULS, liquefaction-induced ground damage likely to be 'minor to moderate'. Some targeted geogrid may be required in some localised areas. Additionally, further investigations may undercover hotspots of higher liquefaction	**For 1 storey buildings, adjacent to a swale:** • TC2 type concrete slabs in accordance with MBIE guidance founded on the fill platform are recommended. • Deep ground improvement expected to mitigate lateral spread effects, however this may be reduced to shallow ground improvement where more favourable ground conditions are present. • Preload is not likely to be required.

Zone number	Potential ground damage from design seismic events[1]	Conceptual foundation description
	potential that may require ground improvement. • Initial geotechnical investigations do not indicate significant deposits of compressible silts, however additional areas are possible.	**For 1 storey buildings, not adjacent to a swale:** • TC2 type concrete slabs in accordance with MBIE guidance founded on the fill platform are recommended. • Preload is not likely to be required. **For 2 storey buildings:** • Ground improvement and shallow foundations likely, subject to future investigations and provided there is separation between buildings on different foundation types. • Otherwise, deep foundations to the dense sand or gravel layer. • Preload is not likely to be required. **For 3 to 6 storey buildings:** • Deep foundations (piles) to the dense sand or gravel layer. • Preload is not likely to be required.
Zone 2	• Lateral ground stretching is expected at free face slopes. • At ILS and ULS, liquefaction-induced ground damage likely to be 'minor to moderate'. Ground improvement is likely to be required and will be confirmed with additional investigations. Ground improvement may vary from additional geogrid to full depth ground improvement such as stone columns. • Consolidation settlement is possible in compressible silts. Investigations indicate the deposit up to 3 m thick and top of layer at or near to current ground surface.	**For 1 storey buildings, adjacent to a swale:** • Buildings on deep foundations (piles) to the dense sand or gravel layer or deep ground improvement. • Preload is likely to be required. **For 1- 2 storey buildings, not adjacent to a swale:** • Buildings on deep foundations to the dense sand or gravel layer or deep ground improvement, however this may be reduced to shallow ground improvement with further investigation and analysis. • Preload is likely to be required. **For 3 to 6 storey buildings:** • Deep foundations to the dense sand or gravel layer. • Preload is likely to be required to mitigate settlement of surface connections.
Zone 3	• Lateral ground stretching is expected at free face slopes. • At ILS and ULS, liquefaction-induced ground damage likely to be 'minor to moderate'. Ground improvement is likely to be required and will be confirmed with additional investigations. Ground improvement may vary from additional geogrid to full depth ground improvement such as stone columns. • Consolidation settlement expected from compressible silts. Investigations indicate that the deposit is present between 4 to7	**For 1 storey buildings, adjacent to a swale:** • Buildings on deep foundations to the dense sand or gravel layer or deep ground improvement. • Preload is likely to be required. **For 1 storey buildings, not adjacent to a swale:** • Buildings on deep foundations to the dense sand or gravel layer or deep ground improvement, however this may be reduced to shallow ground improvement with further investigation and analysis. • Preload is likely to be required. **For 2 storey buildings:**

Zone number	Potential ground damage from design seismic events[1]	Conceptual foundation description
	mbgl in the west from 6 to 9 mbgl in the east.	• Deep foundations to the dense sand or gravel layer, or a raft with a robust slab foundation if ground conditions allow. • Preload is likely to be required to mitigate settlement of surface connections.

Note:

1. As discussed in Section 2.3 Liquefaction assessment, there is little difference between the predicted settlement-related liquefaction consequences at ILS (250 yr ARI event), ULS-IL2 and ULS-IL3 levels of shaking (however lateral spreading displacements in Zones 2 and 3 could be expected to increase with higher levels of shaking).

The geotechnical “zones” should be updated once more geotechnical investigations are available. We recommend that, where possible, each building footprint is located entirely within one geotechnical “zone” where possible after these updates are completed. Where this is not achievable and new buildings are proposed to straddle geotechnical “zones”, the more conservative foundation concept for both zones is recommended to be adopted for that entire building footprint, and the effects of differential foundation performance considered as part of structural design and detailing. The purpose is to reduce the differential foundation performance within a building footprint by providing more uniform founding conditions.

3.4 Site fill requirements

The ground improvement strategy proposed for the site is similar to the strategy adopted for the already completed stages of the Beach Grove subdivision. This comprises placement of a layer of Bidim followed by placement and compaction of engineered fill.

The engineered fill raft provides increased bearing capacity for shallow foundations and reduces the likelihood of differential settlements occurring under the houses. It will also reduce the likelihood of surface manifestation of liquefied sand and silt.

3.5 Earthworks and services

3.5.1 Elevated groundwater

The proximity of the groundwater surface to the ground surface may have implications for earthworks and services installation, depending on the nature of works and time of year. Specifically, we recommend avoiding excavation into the subsurface as it is likely to be affected by groundwater, which could cause issues during construction. Groundwater level at the site is expected to fluctuate over time but was noted to be generally at depths of 0.5-1.0 mbgl during the recent 2022 investigations.

3.5.2 Preliminary pavement design parameters

The proposed fill platform is expected to achieve a design %CBR of 7.

3.5.3 Preliminary bearing capacity estimate

Allowable bearing capacity on the compacted hardfill is expected to be at least 100 kPa however, this will need to be confirmed by shallow geotechnical investigations at each proposed building location during construction. For buildings founding in natural ground or within 1.0 m of natural ground, a bearing capacity assessment will be conducted once building details are known.

3.5.4 Services

The majority of the services are likely to be located within the engineered fill; however, some service trenches may need to be extended below the depth of the engineered fill. Where this occurs, it is recommended that the trench base and sides be wrapped with a geotextile to help reduce the formation of flow paths to the ground surface for liquefied sand.

As far as is practicable, penetrations through the engineered fill and non-liquefiable crust into the underlying liquefiable soils should be avoided or minimised. When the pipes and/or bedding extends into the sand area there is a risk that water from the sand layer will move into the fill around the pipe and may flow along the pipeline. Those services, where penetrations are unavoidable, a water stop should be installed on either side of the area where the pipe and/or bedding extends into the sand and the silt layer sealed using either silt or site concrete. The sealing layer should extend either around the pipe, with a minimum 50 mm cover, or to the base of the engineered fill. A precast concrete collar can be used as the water stop. Alternatively, the waterstop can be formed from insitu won silt or site concrete. If the water stop is formed from silt, the minimum thickness should be 600 mm.

3.5.5 Deep foundations and deep ground improvement

Any deep ground improvement or pile works should consider the potential flow path created by piercing the upper silt layer as discussed above in Section 3.5.4.

4 Recommendations for development

4.1 Risk assessment for subdivision application

Module 2[15] of the NZGS and MBIE geotechnical earthquake guidance documents, gives advice for geotechnical investigations for subdivision developments.

With our prior understanding of the local ground conditions, we consider the density of subsurface investigations at the site to be sufficient for the purposes of technically supporting a rezoning of the site to residential use. Further geotechnical investigations will be required to support the subdivision application at the detailed design phase to inform building design recommendations.

Initial indications show that the fill platform is likely to achieve a TC2 equivalence in the north-western section of the site (Zone 1). In the south eastern portion of the site (Zones 2 and 3) ground improvement methods are likely to be required. These improvement measures could range from additional grid placement to stone columns as previously discussed. For both zones, more geotechnical investigations must be undertaken to confirm this during the detailed design phase. Additional improvement is required to mitigate lateral spread, as previously discussed.

4.2 RMA Section 106

Section 106 of the RMA (1991) includes subdivision consent provisions relating to risk from natural hazards. This includes a combined assessment of likelihood, material damage and subsequent use, and the option of specifying consent conditions for the purpose of avoiding, remedying, or mitigating the effects of natural hazards.

This geotechnical report is intended to help inform a Section 106 assessment by providing information about geotechnical-related natural hazards:

- The proposed development at the site is considered feasible from a geotechnical perspective.
- The two key geotechnical-related natural hazards for the site are considered to be earthquake-induced liquefaction and static settlement. Other geotechnical hazards are considered to either have a low likelihood of occurring or are unlikely to result in significant material damage to land or structures.
- We consider that:
  - liquefaction-induced ground surface damage is expected to be within the criteria for TC2-type foundations (MBIE foundation guidelines[16]) and for Medium Liquefaction Vulnerability (MBIE/MfE liquefaction planning guidance[17]).
  - the likely subsequent use of the land is unlikely to accelerate, worsen or result in geotechnical-related hazards.
  - Settlement due to compressible silts is expected to be controlled to within design tolerances using mitigation measures such as preloading.
- On this basis, we consider that liquefaction-related natural hazard risk can be appropriately mitigated via subdivision consent conditions similar to those previously specified on the Beach Grove subdivision.

[15] New Zealand Geotechnical Society (NZGS) and Ministry of Business, Innovation and Employment (MBIE) (2021). *Earthquake Geotechnical Engineering Practice in New Zealand. Module 2 – Geotechnical investigations for earthquake engineering - Earthquake geotechnical engineering practice. Rev. 1, November 2021.*

[16] Revised issue of Repairing and rebuilding houses affected by the Canterbury earthquakes, Revision 3, Dec 2012, MBIE

[17] MBIE/MfE (2017) Planning and engineering guidance for potentially liquefaction-prone land, Ministry of Business, Innovation & Employment and Ministry for the Environment. https://www.building.govt.nz/building-code-compliance/b-stability/b1-structure/planning-engineering-liquefaction-land/.

- The potential for any future erosion is expected to be managed by the Erosion and Sediment Control Plan for the site (to be prepared by others).
- Inundation from stormwater has not been considered in this report. This is expected to be addressed as part of the detailed civil engineering design for the subdivision (to be prepared by others).

5 Further work

Additional deep geotechnical investigations will be required to:

- Support a subdivision development application[18].
- Better define the liquefaction response on the site.
- Better define the presence of compressible silts on the site.
- Inform foundation selection and develop foundation parameters for buildings.

T+T can scope and organise this additional testing once development details are confirmed. This is likely to consist of 2-3 additional boreholes and 9 additional CPTs to refusal.

Design and selection of the building foundation systems should be made in collaboration with the Structural Engineer, the Geotechnical Engineer, Civil Engineer, and the Client, once more detail of actual building configurations is available. This should allow more complete consideration of seismic performance expectations, financial constraints, and constructability.

A lateral spread assessment should be undertaken in conjunction with the foundation design.

[18] New Zealand Geotechnical Society (NZGS) and Ministry of Business, Innovation and Employment (MBIE) (2021). Earthquake geotechnical engineering practice. Module 2: Geotechnical investigations for earthquake engineering - Earthquake geotechnical engineering practice, November 2021.

6 Applicability

This report has been prepared for the exclusive use of our client Momentum Land Ltd, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

All of the recommendations and interpretations presented in this report are preliminary in nature and must be reviewed as part of the future design process for any development works.

We understand and agree that our client will submit this report to support their submission to Waimakariri District Council and that the Council will use this report for the purpose of assessing that submission.

Recommendations and opinions in this report are based on data from discrete investigation locations. The nature and continuity of subsoil away from these locations are inferred but it must be appreciated that actual conditions could vary from the assumed model.

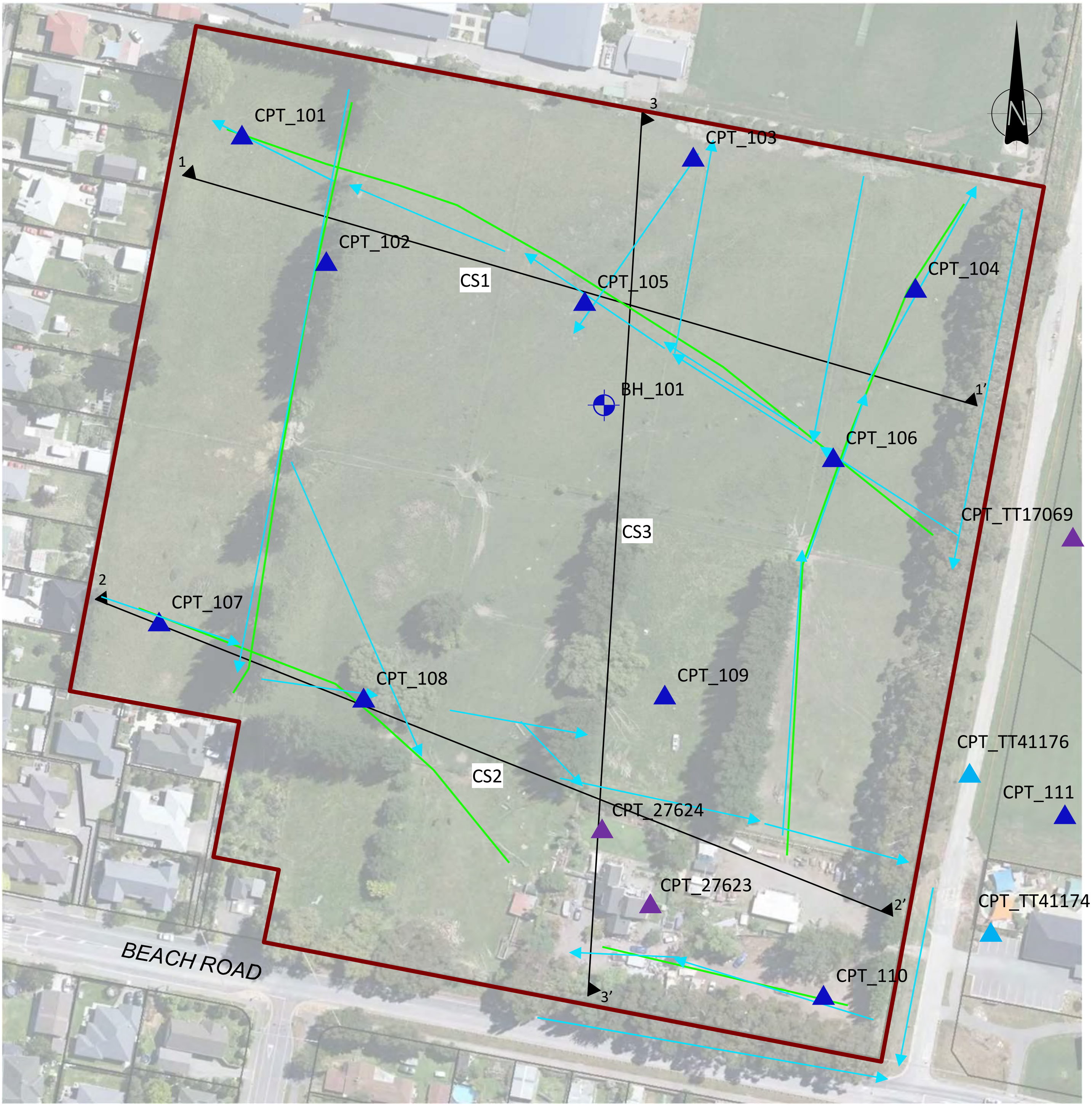
Tonkin & Taylor Ltd
Environmental and Engineering Consultants

Report prepared by:	Authorised for Tonkin & Taylor Ltd by:
Sam Burgess Geotechnical Engineer	Anna Sleight Project Director

Review by: Richard Brunton

18-May-23
\\ttgroup.local\corporate\christchurch\tt projects\1019317\issueddocuments\2023-05-10.snb.south block_georeport_final.docx

# Appendix A	Site investigation location plan



Approximate Scale

0 10 20 30 40 50 60 70 80 (m)

LEGEND

	Site Outline		T+T Borehole (2022)
	Cross Section		T+T CPT (2022)
	MASW line		T+T CPT (2014)
	GPR line		CPT by others

Notes:

1. Aerial image sourced from Canterbury Maps, Date Accessed: 08/03/2021
2. Geotechnical Investigations by others sourced from the NZGD

Tonkin+Taylor

www.tonkintaylor.co.nz

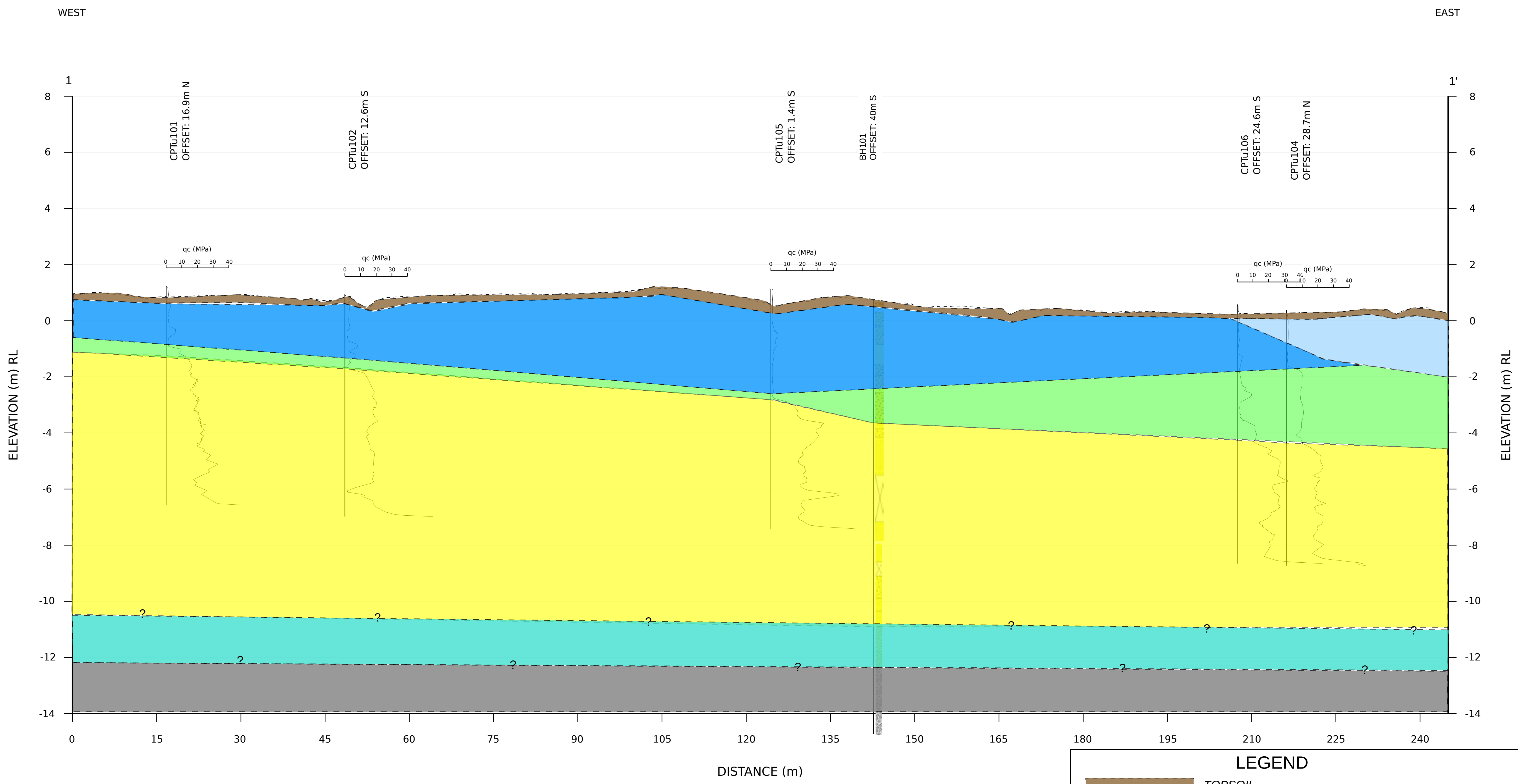
DRAWN	SNB	05.23
DRAFTING CHECKED		
APPROVED		
FILE : Christchurch\TTProjects\1019317\WorkingMaterial\01_Geotechnical\05_CrossSectionAndPlan		
APPROX. SCALE (AT A4 SIZE) NTS		
PROJECT No. 1019317.1000		

MOORE LAND SOUTH BLOCK
MOMENTUM LAND LIMITED
SITE PLAN WITH GEOTECHNICAL INVESTIGATIONS

FIG. No. Figure 1

REV. 1

Appendix B Geological cross sections



SECTION 1
SCALE: 1:750H (1:150V)

T+T Cross-Section Generator (CSG)
Start Point (NZTM): 1572745.1 mE, 5197274.82 mN
End Point (NZTM): 1572979.8 mE, 5197207.37 mN
Date Generated: 2022-05-30, 20:45:06
CSG Version: 1.2 (First Release)
Vertical Datum: Investigation RLs are shown in NZVD2016 datum.
Elevation data: sourced from Waimakariri District Council/Environment Canterbury LiDAR set '2014 Rangiora'. Data captured March 14, 2014 to June 23, 2014

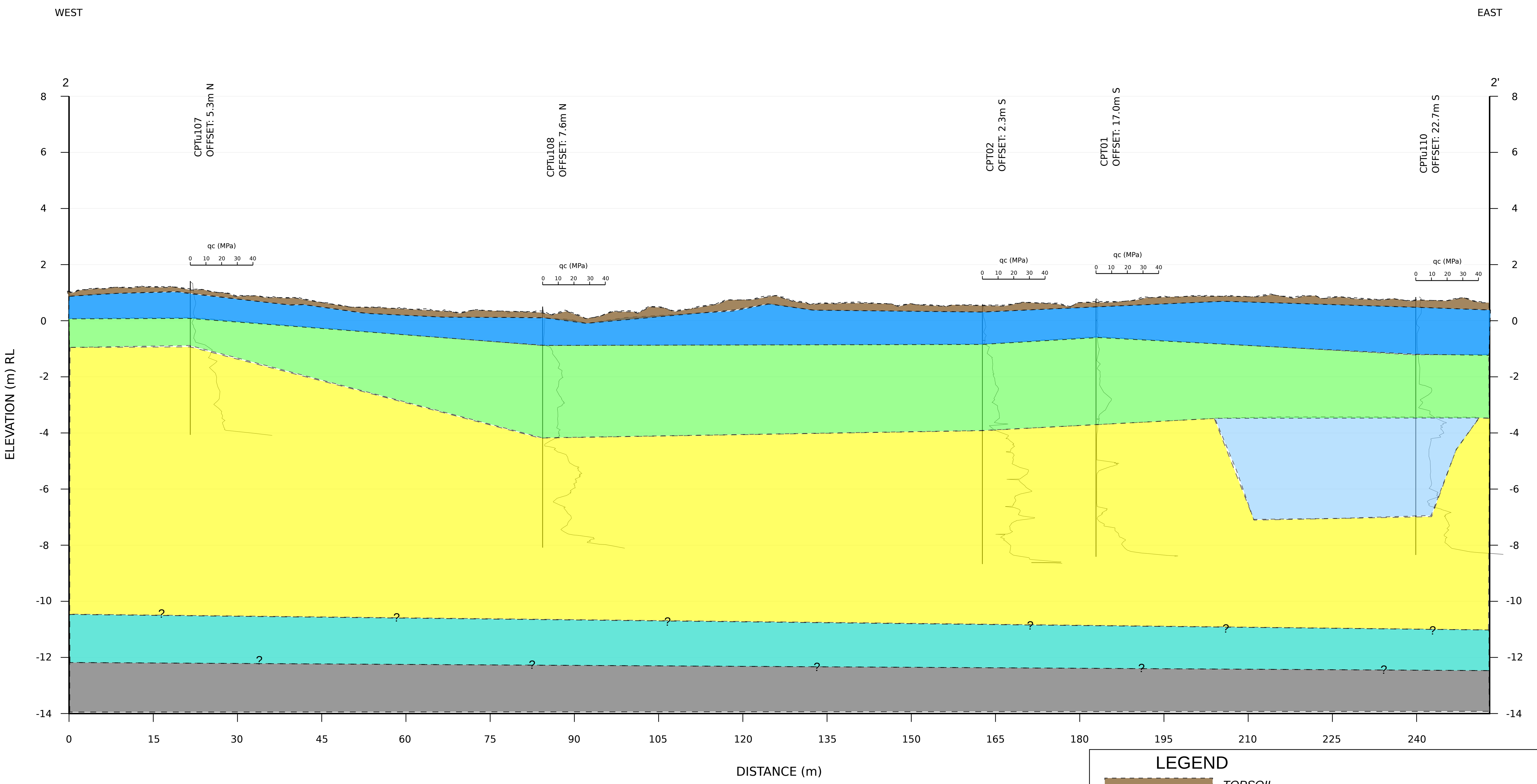
LEGEND

- *TOPSOIL*
- *SOFT TO STIFF SANDY SILT TO SILT*
- *VERY SOFT SILT*
- *LOOSE TO MEDIUM DENSE SAND WITH OCCASIONAL SILT BEDS*
- *DENSE TO VERY DENSE SAND TO GRAVELLY SAND*
- *FIRM SILT*
- *GRAVEL*

Tonkin+Taylor
Exceptional thinking together www.tonkintaylor.co.nz

PROJECT No.	1019317.1000	
DESIGNED	SNB	May.23
DRAWN	SNB	May.23
CHECKED	AFS	May.23
APPROVED		DATE

CLIENT	**MOMENTUM LAND LTD**
PROJECT	**MOORE LAND SOUTH BLOCK**
TITLE	GEOTECHNICAL CROSS SECTION 1
SCALE (A3)	AS SHOWN
FIG No.	2
REV	2



SECTION 2
SCALE: 1:750H (1:150V)

T+T Cross-Section Generator (CSG)
Start Point (NZTM): 1572720.77 mE, 5197148.59 mN
End Point (NZTM): 1572954.99 mE, 5197055.61 mN
Date Generated: 2022-05-30, 20:41:13
CSG Version: 1.2 (First Release)
Vertical Datum: Investigation RLs are shown in NZVD2016 datum.
Elevation data: sourced from Waimakariri District Council/Environment Canterbury LiDAR set '2014 Rangiora'. Data captured March 14, 2014 to June 23, 2014

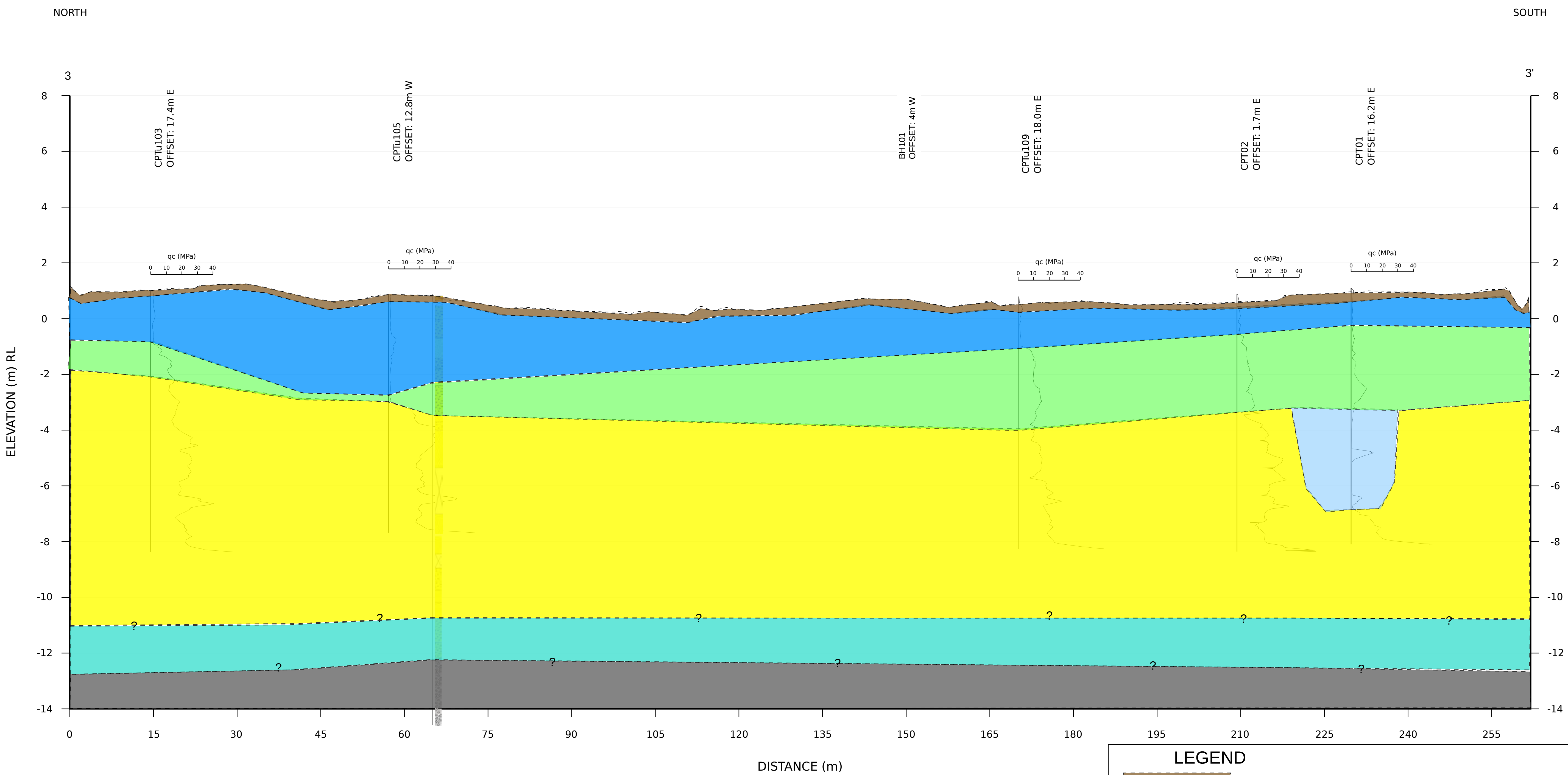
LEGEND

- *TOPSOIL*
- *SOFT TO STIFF SANDY SILT TO SILT*
- *VERY SOFT SILT*
- *LOOSE TO MEDIUM DENSE SAND WITH OCCASIONAL SILT BEDS*
- *DENSE TO VERY DENSE SAND TO GRAVELLY SAND*
- *FIRM SILT*
- *GRAVEL*

Tonkin+Taylor
Exceptional thinking together www.tonkintaylor.co.nz

PROJECT No.	1019317.1000	
DESIGNED	SNB	May.23
DRAWN	SNB	May.23
CHECKED	AFS	May.23
APPROVED	DATE	

CLIENT	**MOMENTUM LAND LTD**
PROJECT	**MOORE LAND SOUTH BLOCK**
TITLE	GEOTECHNICAL CROSS SECTION 2
SCALE (A3)	AS SHOWN
FIG No.	3
REV	2



SECTION 3
SCALE: 1:750H (1:150V)

T+T Cross-Section Generator (CSG)
Start Point (NZTM): 1572879.9 mE, 5197295.62 mN
End Point (NZTM): 1572866.75 mE, 5197034.14 mN
Date Generated: 2022-05-30, 21:42:32
CSG Version: 1.2 (First Release)
Vertical Datum: Investigation RLs are shown in NZVD2016 datum.
Elevation data: sourced from Waimakariri District Council/Environment Canterbury LiDAR set '2014 Rangiora'. Data captured March 14, 2014 to June 23, 2014

LEGEND

- *TOPSOIL*
- *SOFT TO STIFF SANDY SILT TO SILT*
- *VERY SOFT SILT*
- *LOOSE TO MEDIUM DENSE SAND WITH OCCASIONAL SILT BEDS*
- *DENSE TO VERY DENSE SAND TO GRAVELLY SAND*
- *FIRM SILT*
- *GRAVEL*

Tonkin+Taylor
Exceptional thinking together www.tonkintaylor.co.nz

PROJECT No.	1019317.1000	
DESIGNED	SNB	May.23
DRAWN	SNB	May.23
CHECKED	AFS	May.23
APPROVED		DATE

CLIENT	**MOMENTUM LAND LTD**
PROJECT	**MOORE LAND SOUTH BLOCK**
TITLE	GEOTECHNICAL CROSS SECTION 3

SCALE (A3) AS SHOWN FIG No. 4 REV 2

Appendix C Site specific investigation results

- **Borehole.**
- **CPTs.**
- **MASW and GPR.**

BOREHOLE LOG

BOREHOLE No.: **BH01**

SHEET: 1 OF 2

PROJECT: Moore Land - Momentum	LOCATION: Christchurch
JOB No.: 1019317.1000	
CO-ORDINATES: (NZTM2000)	5197239 mN, 1572864 mE
R.L.:	0.64m
DATUM:	NZVD2016
DRILL TYPE:	Geoprobe 8140LC
METHOD:	Sonic core drilling
DRILL FLUID:	WATER
HOLE STARTED:	08/04/2022
HOLE FINISHED:	08/04/2022
DRILLED BY:	McMillan Drilling
LOGGED BY:	SAWH
CHECKED:	PELE

Depth (m)	Geological unit	Core recovery (%)	Method	Tests	Moisture	Consistency / Density	Description
0.00	Fill				M	S	*0.00m:* Organic SILT, trace clay; dark brown. Soft, moist, low plasticity. Organics, rootlets.
0.15	Springston Formation	87	SNC				*0.15m:* CORE LOSS - Suspect loose material washed out.
0.35				0.9 @ 1.00m PSD tested and Atteberg on 27/05/2022	M	F-St	*0.35m:* SILT, minor clay; dark greyish brown with mottled orange. Firm to stiff, moist, medium plasticity.
0.85							*0.85m:* Clayey SILT, trace sand; greyish brown with mottled orange. Firm to stiff, moist, high plasticity. Sand, fine.
1.50		100	SPT	1/1// 1/2/2/2 **N=7**		L	*1.50m:* Fine to medium SAND, minor silt; grey. Loose, moist, dilatant slow.
2.20		100	SNC	2.0 @ 2.10m PSD tested and Atteberg on 27/05/2022		St	*2.20m:* Sandy SILT; grey. Stiff, moist, low plasticity. Sand, fine to medium.
2.60				2.9 @ 3.00m PSD tested on 27/05/2022			*2.60m:* SILT, trace clay and trace sand; grey. Stiff, moist, low plasticity. Sand, fine. *2.80 - 3.05m:* some sand.
3.05		100	SPT	0/0// 0/0/1/1 **N=2**		VL	*3.05m:* Silty fine to medium SAND; grey. Very loose, moist, dilatant.
3.20		100	SNC			S-F	*3.20m:* SILT, minor clay; grey. Soft to firm, moist, medium plasticity. *3.50 - 4.30m:* Trace sand. Sand, fine.
4.30	Christchurch Formation					L-MD	*4.30m:* Fine SAND, minor silt; grey. Loose to medium dense, moist, dilatant.
4.50		100	SPT	1/1// 0/1/4/5 **N=10**			*4.50m:* Gravelly fine to coarse SAND; grey. Loose to medium dense, moist. Gravel, fine to coarse, rounded to sub-rounded.
4.80		100	SNC				*4.80m:* Fine SAND, trace silt; brownish grey. Loose to medium dense, moist, dilatant. *5.90 - 6.10m:* Trace gravel. Gravel, fine, rounded.
6.15		78	SPT	3/2// 3/3/4/5 **N=15**			*6.15m:* CORE LOSS.
		0	SNC				
7.80		100	SPT	7/6// 8/9/8/9 **N=34**	M	D	*7.80m:* Fine SAND, minor silt; brownish grey. Dense, moist, dilatant. *8.20 - 8.90m:* Trace gravel. Gravel, fine to medium.
		81	SNC				

RL (m): 0 at 0.64 m depth; -1 ... -7 at 1 m intervals.

COMMENTS: Groundwater not accurately measured. Double nested piezometer installed at 2.8 - 3.3 m and 5.5 - 6.5 m below existing ground level.

Hole Depth 15.2m

Scale 1:43

Rev.: A

BOREHOLE LOG

Tonkin+Taylor

PROJECT: Moore Land - Momentum	LOCATION: Christchurch	JOB No.: 1019317.1000	
CO-ORDINATES: (NZTM2000) 5197239 mN 1572864 mE	DRILL TYPE: Geoprobe 8140LC	HOLE STARTED: 08/04/2022	
R.L.: 0.64m	METHOD: Sonic core drilling	HOLE FINISHED: 08/04/2022	
DATUM: NZVD2016	DRILL FLUID: WATER	DRILLED BY: McMillan Drilling	
		LOGGED BY: SAWH	CHECKED: PELE

GEOLOGICAL UNIT/ ADDITIONAL OBSERVATIONS	CORE RECOVERY (%)	METHOD	TESTS	RL (m)	DEPTH (m)	MOISTURE	CONSISTENCY / DENSITY	DESCRIPTION
Christchurch Formation	81	SNC		-8	9	M	D	[CONT] *7.80m:* Fine SAND, minor silt; brownish grey. Dense, moist, dilatant.
	62	SPT	8/8// 11/12/13/12 **N=48**	-9				*9.12m:* CORE LOSS.
	53	SNC			10	M	D	*9.62m:* Gravelly fine to coarse SAND; brownish grey. Dense, moist. Gravel, fine to coarse, rounded to sub-rounded. *10.00 - 10.40m:* Gravel; fine to medium.
	100	SPT	2/1// 2/2/2/1 **N=7**	-10			L	*10.40m:* Fine SAND, minor silt; grey. Loose, moist, dilatant rapid.
					11		D	*10.85m:* Fine to coarse SAND, some gravel, minor shell fragments; grey. Dense, moist. Gravel, fine to medium, rounded to sub-rounded. *11.25 - 11.35m:* Some organics; dark brown. Organics, twigs (decomposed).
	100	SNC		-11	12		St	*11.35m:* SILT, trace clay; grey. Stiff, moist, medium plasticity.
	100	SPT	4/4// 4/3/4/4 **N=15**	-12			St-VSt	*12.25m:* SILT, minor sand, trace organics; grey. Stiff to very stiff, moist, low plasticity. Organics, twigs (decomposed). *12.50 - 12.80m:* Some sand.
Burnham Formation	100	SNC		-13	13		D-VD	*12.80m:* Sandy fine to coarse GRAVEL; grey. Dense to very dense, moist. Gravel, sub-rounded; sand, fine to coarse.
	100	SPT	3/7// 10/11/11/11 **N=43**		14			
	100	SNC		-14	15			*14.90 - 15.10m:* SAND, some gravel.
	98	SPT	17/31// 28/26/6 for 15mm **N=60**	-15	16			15.2m: END OF BOREHOLE
				-16				

COMMENTS: Groundwater not accurately measured. Double nested piezometer installed at 2.8 - 3.3 m and 5.5 - 6.5 m below existing ground level.

Hole Depth 15.2m

Scale 1:43

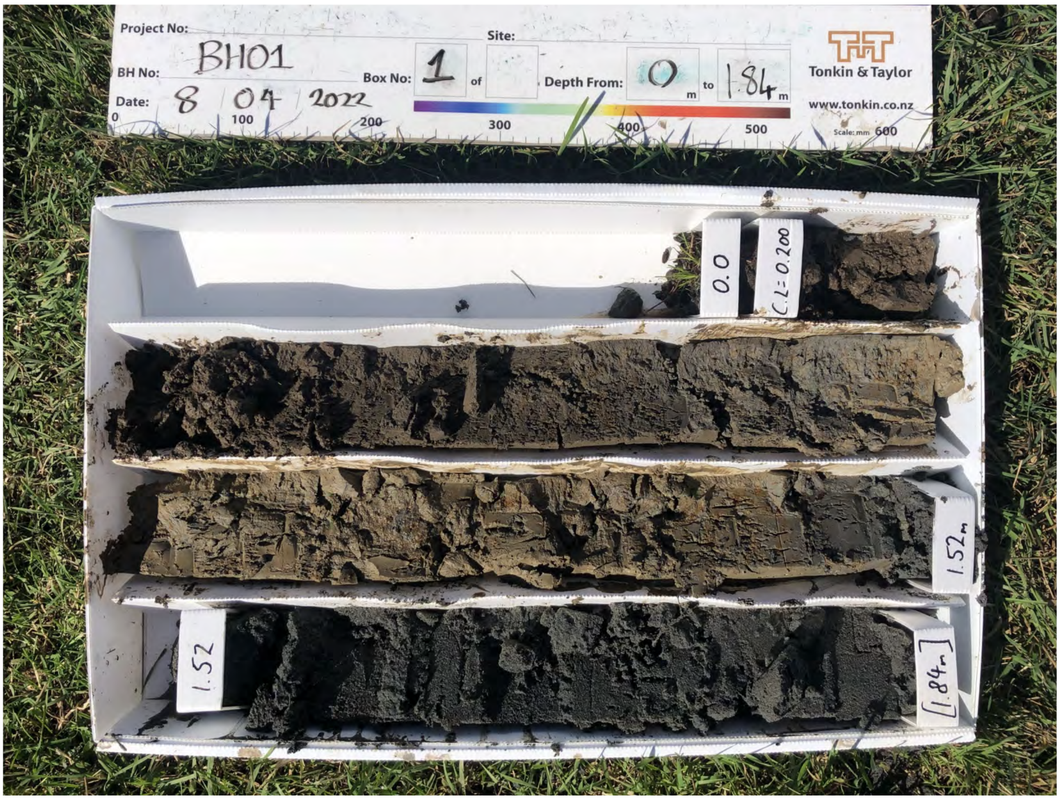
Rev.: A

CORE PHOTOS

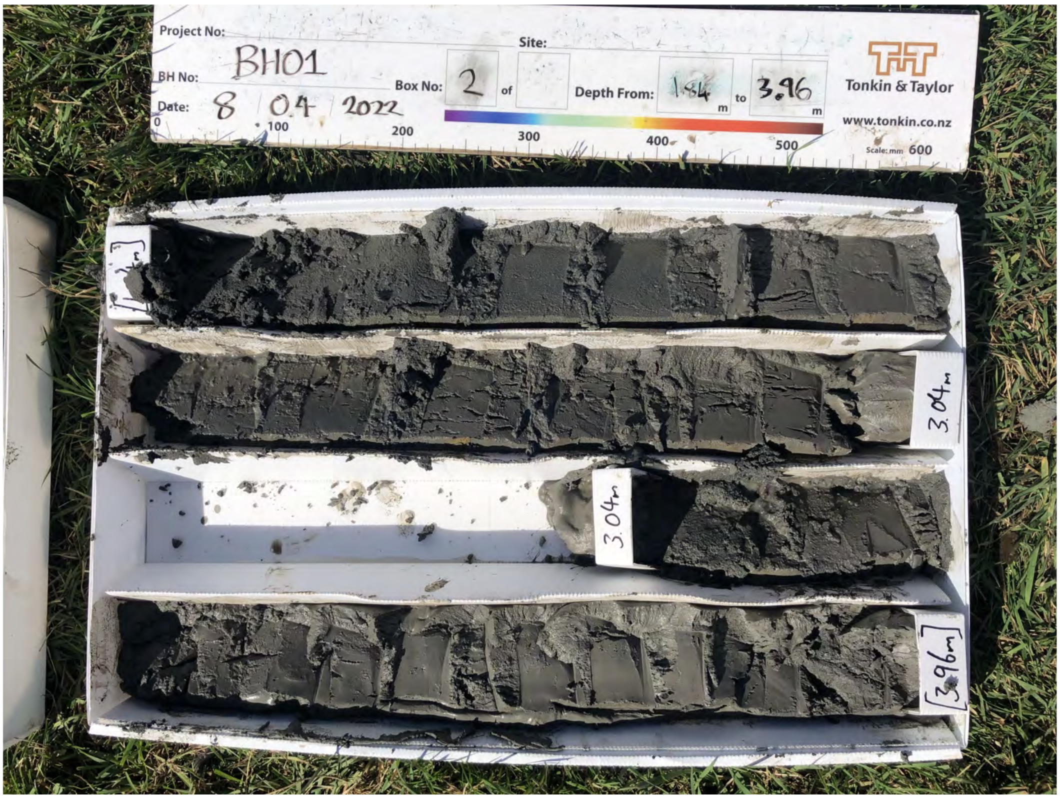
BOREHOLE No.: **BH01**

SHEET: 1 OF 4

PROJECT: Moore Land - Momentum	LOCATION: Christchurch	JOB No.: 1019317.1000
CO-ORDINATES: (NZTM2000) 5197239.14 mN 1572864.27 mE	DRILL TYPE: Geoprobe 8140LC	HOLE STARTED: 08/04/2022
	METHOD: Sonic core drilling	HOLE FINISHED: 08/04/2022
R.L.: 0.64m		DRILLED BY: McMillan Drilling
DATUM: NZVD2016	DRILL FLUID: WATER	LOGGED BY: SAWH CHECKED: PELE



0.00-1.84m



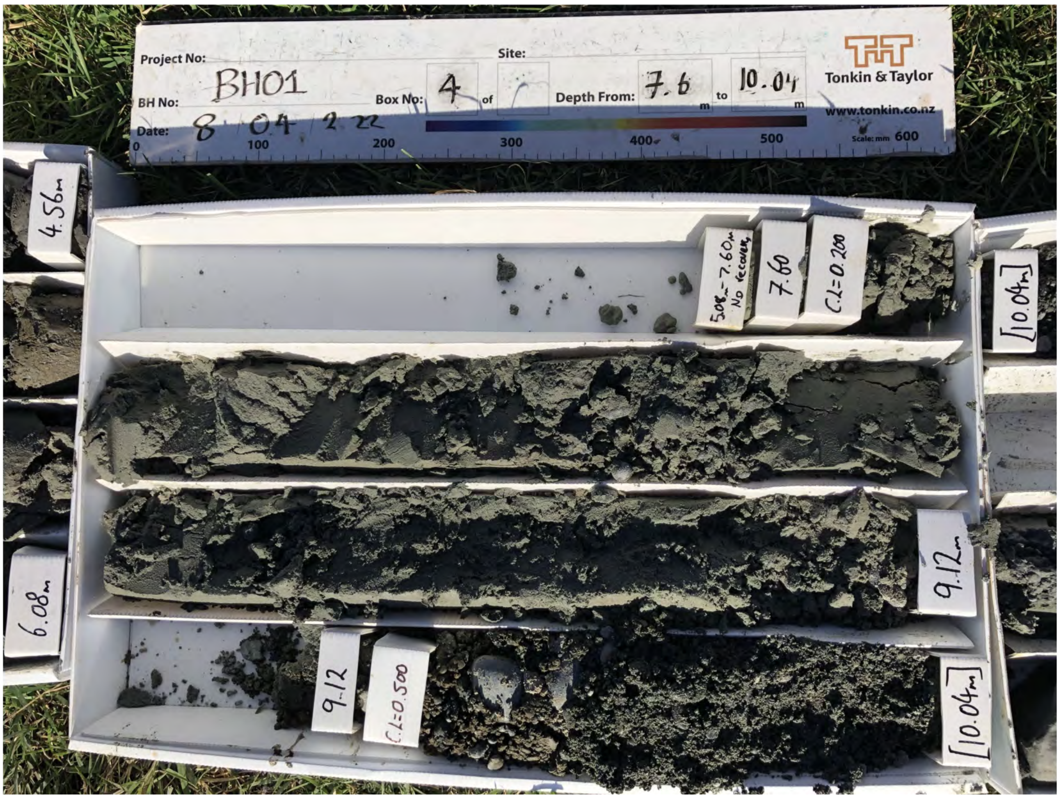
1.84-3.96m

CORE PHOTOS

BOREHOLE No.: **BH01**

PROJECT: Moore Land - Momentum	LOCATION: Christchurch	JOB No.: 1019317.1000
CO-ORDINATES: (NZTM2000) 5197239.14 mN 1572864.27 mE	DRILL TYPE: Geoprobe 8140LC	HOLE STARTED: 08/04/2022
	METHOD: Sonic core drilling	HOLE FINISHED: 08/04/2022
R.L.: 0.64m		DRILLED BY: McMillan Drilling
DATUM: NZVD2016	DRILL FLUID: WATER	LOGGED BY: SAWH CHECKED: PELE

3.96-6.08m



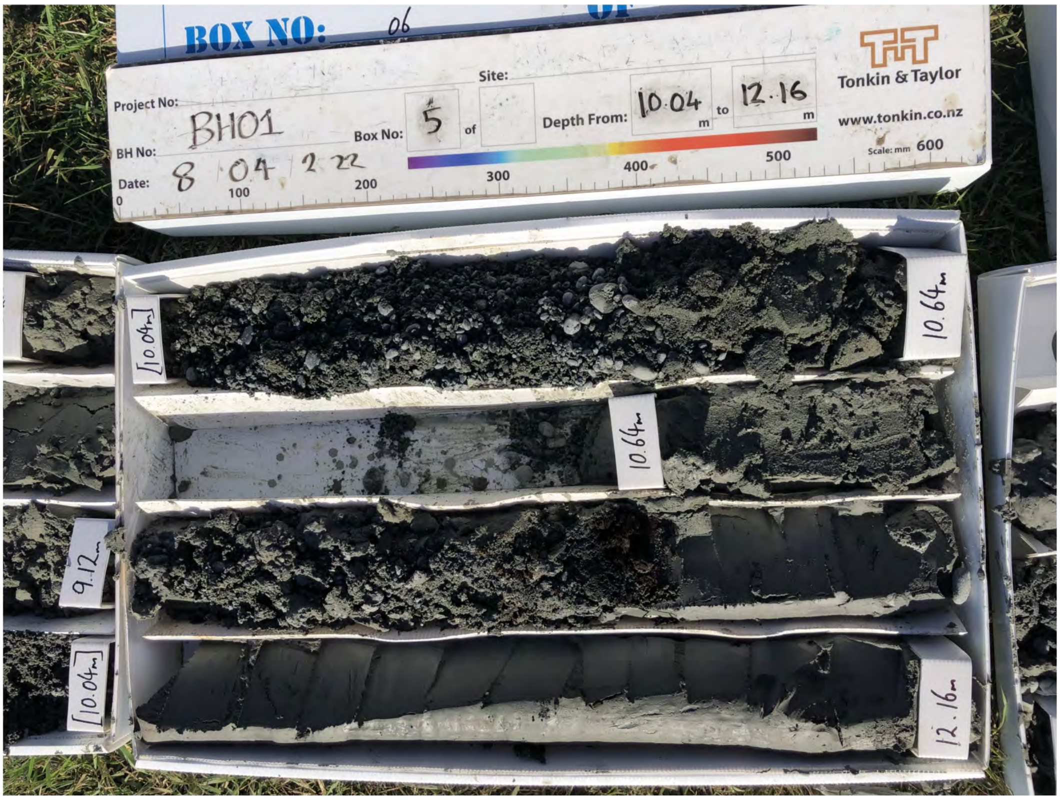
6.08-10.04m

CORE PHOTOS

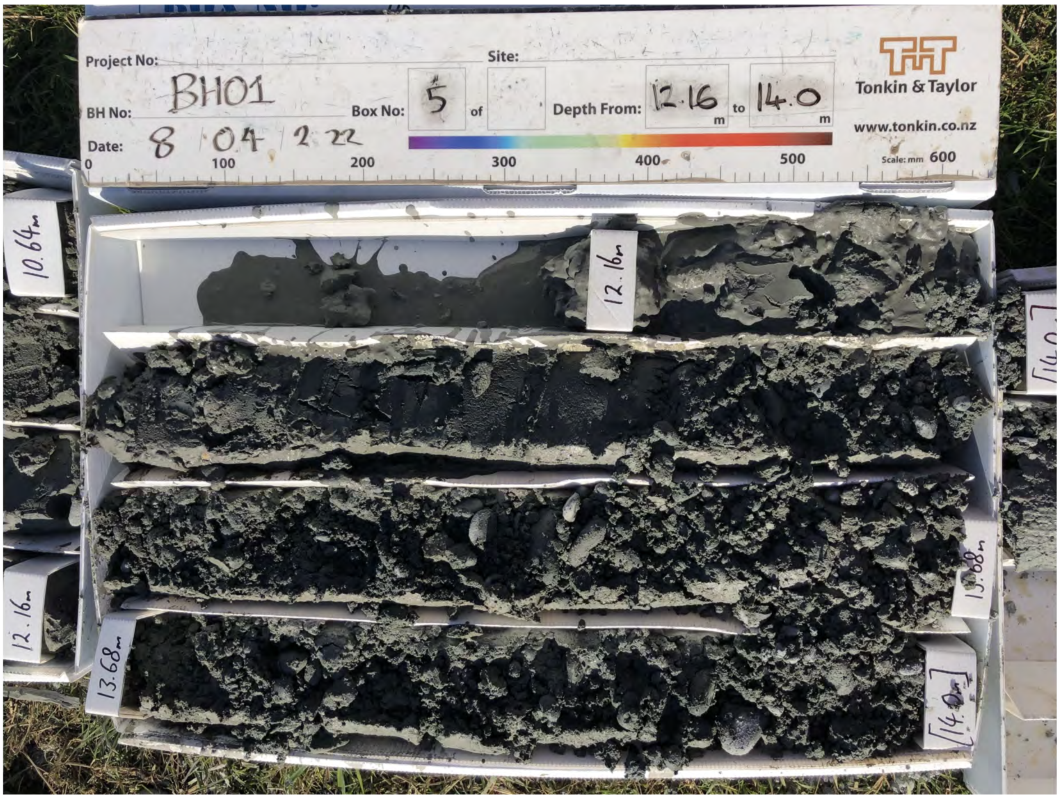
BOREHOLE No.: **BH01**

SHEET: 3 OF 4

PROJECT: Moore Land - Momentum		LOCATION: Christchurch	JOB No.: 1019317.1000
CO-ORDINATES: (NZTM2000)	5197239.14 mN 1572864.27 mE	DRILL TYPE: Geoprobe 8140LC	HOLE STARTED: 08/04/2022 HOLE FINISHED: 08/04/2022
R.L.:	0.64m	METHOD: Sonic core drilling	DRILLED BY: McMillan Drilling
DATUM:	NZVD2016	DRILL FLUID: WATER	LOGGED BY: SAWH CHECKED: PELE



10.04-12.16m



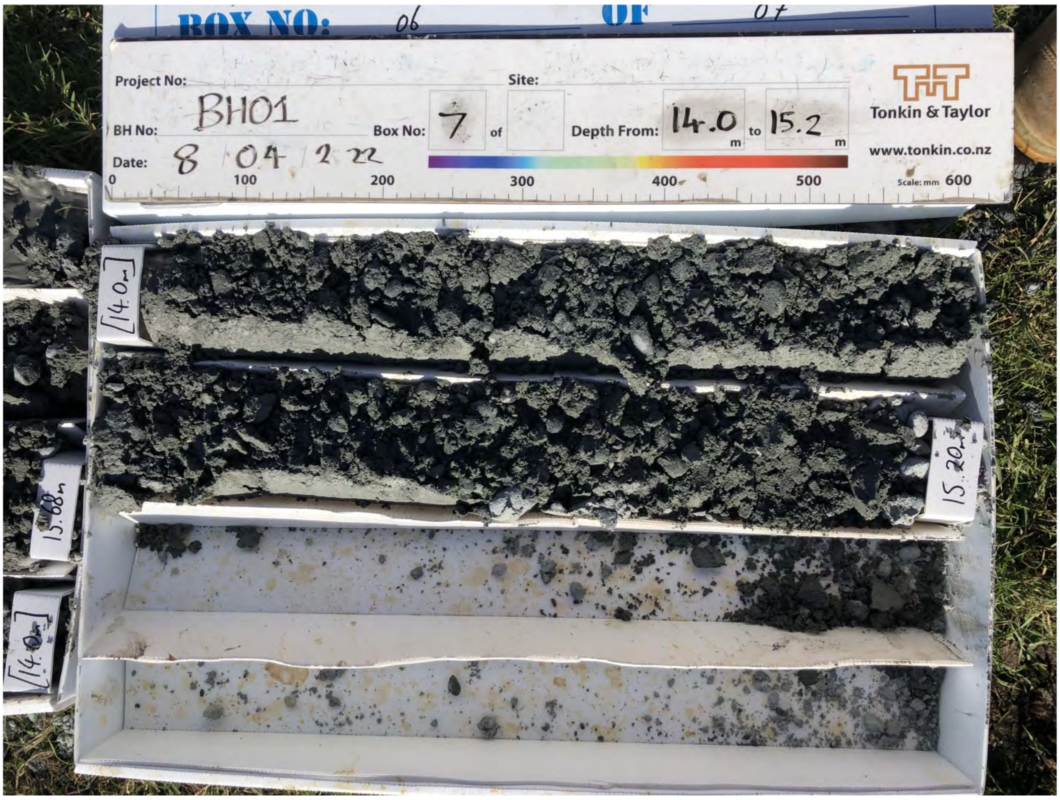
12.16-14.00m

CORE PHOTOS

BOREHOLE No.: **BH01**

SHEET: 4 OF 4

PROJECT: Moore Land - Momentum	LOCATION: Christchurch	JOB No.: 1019317.1000
CO-ORDINATES: (NZTM2000) 5197239.14 mN 1572864.27 mE R.L.: 0.64m DATUM: NZVD2016	DRILL TYPE: Geoprobe 8140LC METHOD: Sonic core drilling DRILL FLUID: WATER	HOLE STARTED: 08/04/2022 HOLE FINISHED: 08/04/2022 DRILLED BY: McMillan Drilling LOGGED BY: SAWH CHECKED: PELE



14.00-15.20m

CONE PENETRATION TEST (CPT) REPORT

McMILLAN Drilling

Client: Tonkin and Taylor Ltd

Location: Beach Grove Subdivision
Beach Road, Kaiapoi

Printed: 23/03/2022

Client: Tonkin and Taylor Ltd	Bore No.: CPTu101
Project: Beach Grove Subdivision	Job No.: 20724

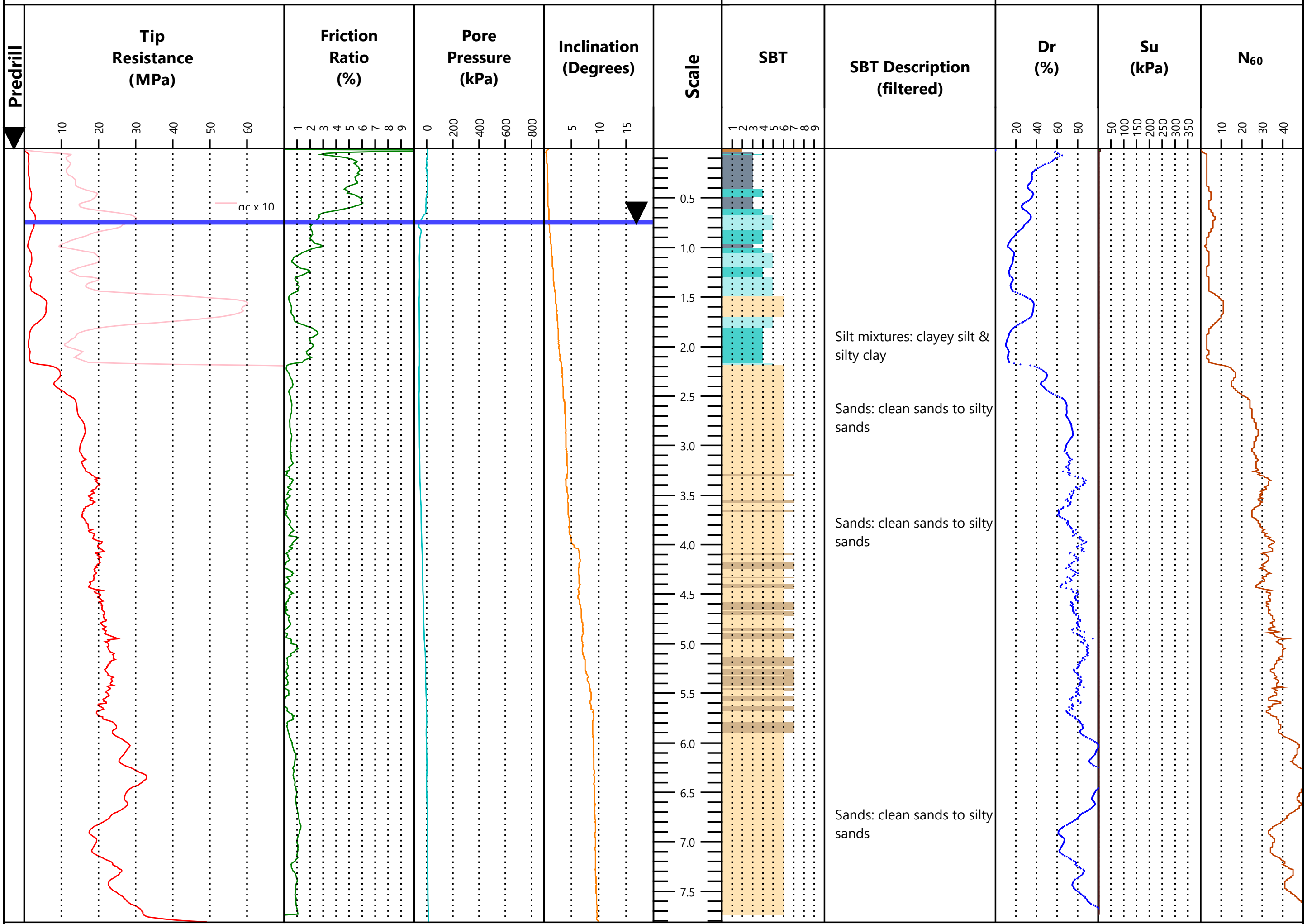
McMILLAN Drilling

Site Location: Beach Road, Kaiapoi **Date:** 18/3/2022

Grid Reference: 1572765.82m E, 5197286.5m N (NZTM) - Handheld GPS **Rig Operator:** E. Diaz

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

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



EOH: 7.81m

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

Predrill: -
Water Level: 0.75m
Collapse: 0.900m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.0835	0.0702
Local Friction	0.0072	0.0039
Pore Pressure	-0.0016	-0.0062

Termination

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

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

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

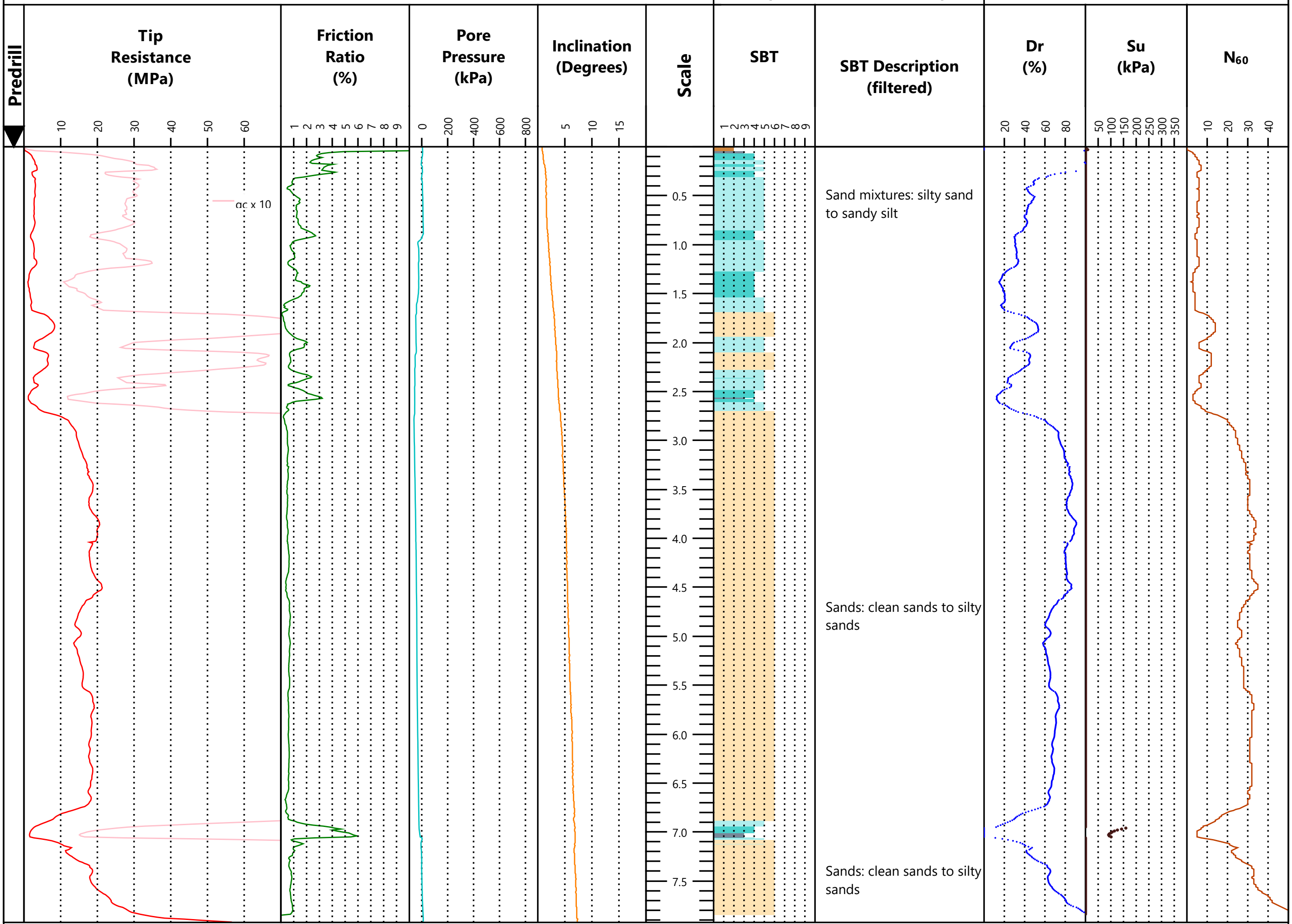
Notes & Limitations

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

Remarks

McMILLAN Drilling	**Client:** Tonkin and Taylor Ltd	**Bore No.:** **CPTu102**
	Project: Beach Grove Subdivision	**Job No.:** **20724**

Site Location: Beach Road, Kaiapoi
Date: 18/3/2022
Grid Reference: 1572788.24m E, 5197249.28m N (NZTM) - Handheld GPS
Rig Operator: E. Diaz
Elevation: 0.00m **Datum:** Ground
Equipment: Geomil Panther 100



EOH: 7.92m

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

Predrill: -
Water Level: -
Collapse: 0.380m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.1190	0.1287
Local Friction	0.0098	0.0095
Pore Pressure	0.0056	0.0045

Termination

Target Depth ☐

Effective Refusal

Tip ☑
Gauge ☐
Inclinometer ☐
Other ☐

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

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

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

Remarks

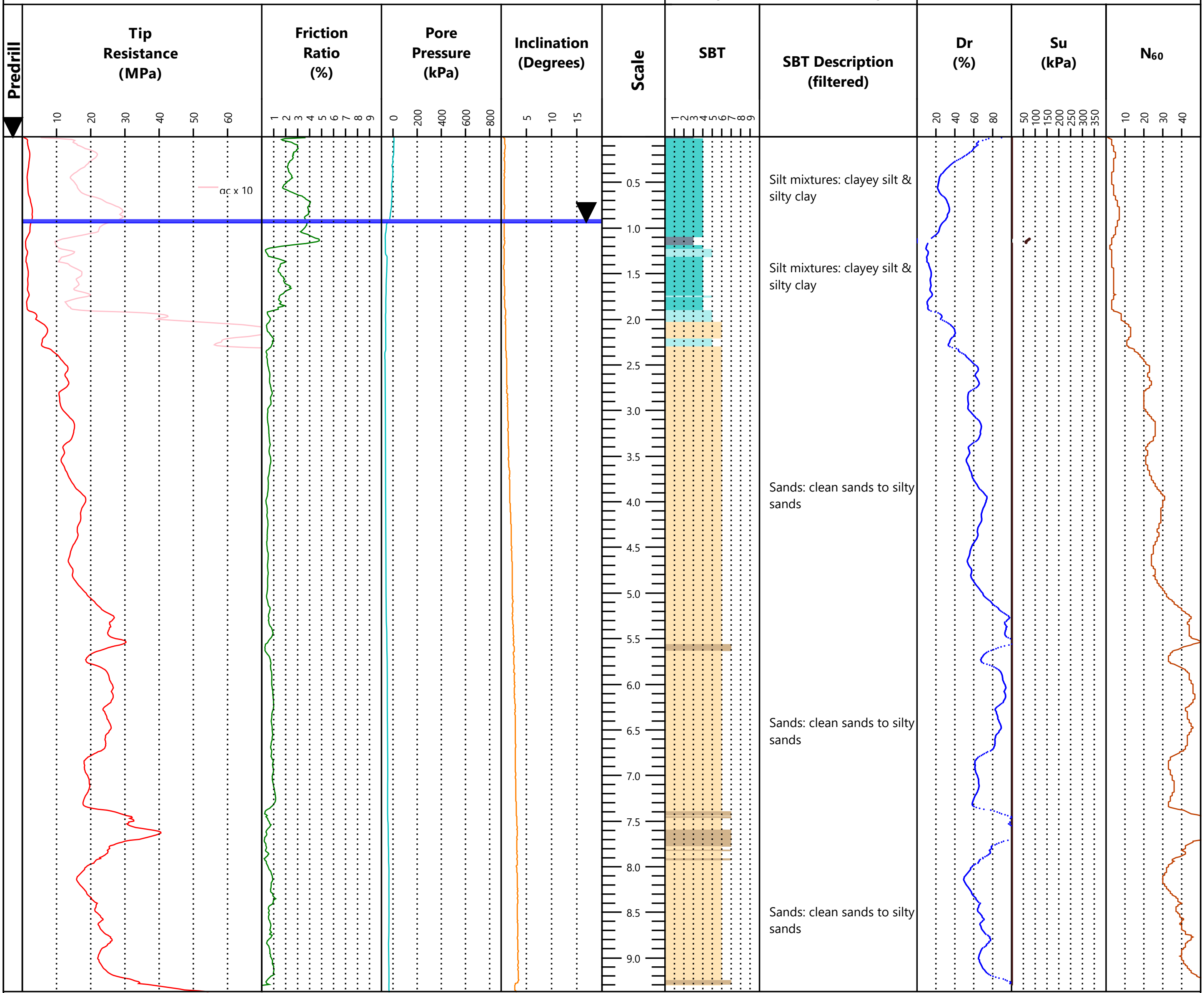
McMILLAN Drilling	**Client:** Tonkin and Taylor Ltd	**Bore No.:** **CPTu103**
	Project: Beach Grove Subdivision	**Job No.:** **20724**

Site Location: Beach Road, Kaiapoi **Date:** 18/3/2022

Grid Reference: 1572896.59m E, 5197280.25m N (NZTM) - Handheld GPS **Rig Operator:** E. Diaz

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

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



EOH: 9.37m

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

Predrill: -
Water Level: 0.93m
Collapse: 1.310m

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	0.1126	0.1044
Local Friction	0.0053	0.0043
Pore Pressure	0.0016	-0.0037

Termination

Target Depth ☐

Effective Refusal
- Tip ☑
- Gauge ☐
- Inclinometer ☐
- Other ☐

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

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

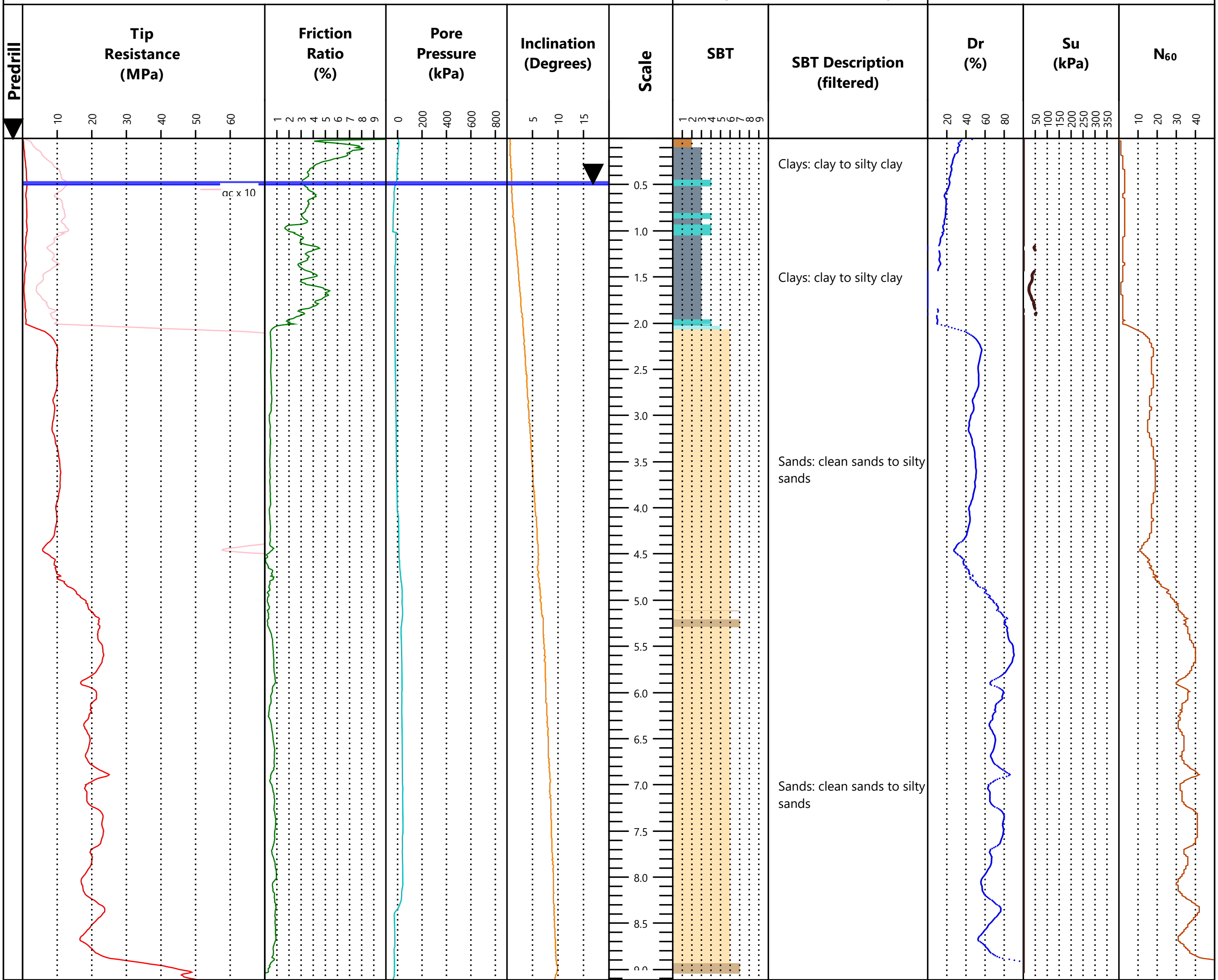
Notes & Limitations
Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P. K. Robertson and K.L. Cabal, Guide to Cone Penetration Testing for Geotechnical Engineering.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

Remarks

McMILLAN Drilling	Client: Tonkin and Taylor Ltd	Bore No.: CPTu104
	Project: Beach Grove Subdivision	Job No.: 20724

Site Location: Beach Road, Kaiapoi
Date: 18/3/2022
Grid Reference: 1572960.82m E, 5197242.64m N (NZTM) - Handheld GPS
Rig Operator: E. Diaz
Elevation: 0.00m **Datum:** Ground
Equipment: Geomil Panther 100

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



EOH: 9.11m

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

Predrill: -
Water Level: 0.49m
Collapse: 0.830m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.0932	0.1083
Local Friction	0.0111	0.0091
Pore Pressure	0.0104	0.0048

Termination

Target Depth ☐

Effective Refusal

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

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

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

Notes & Limitations

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

Remarks

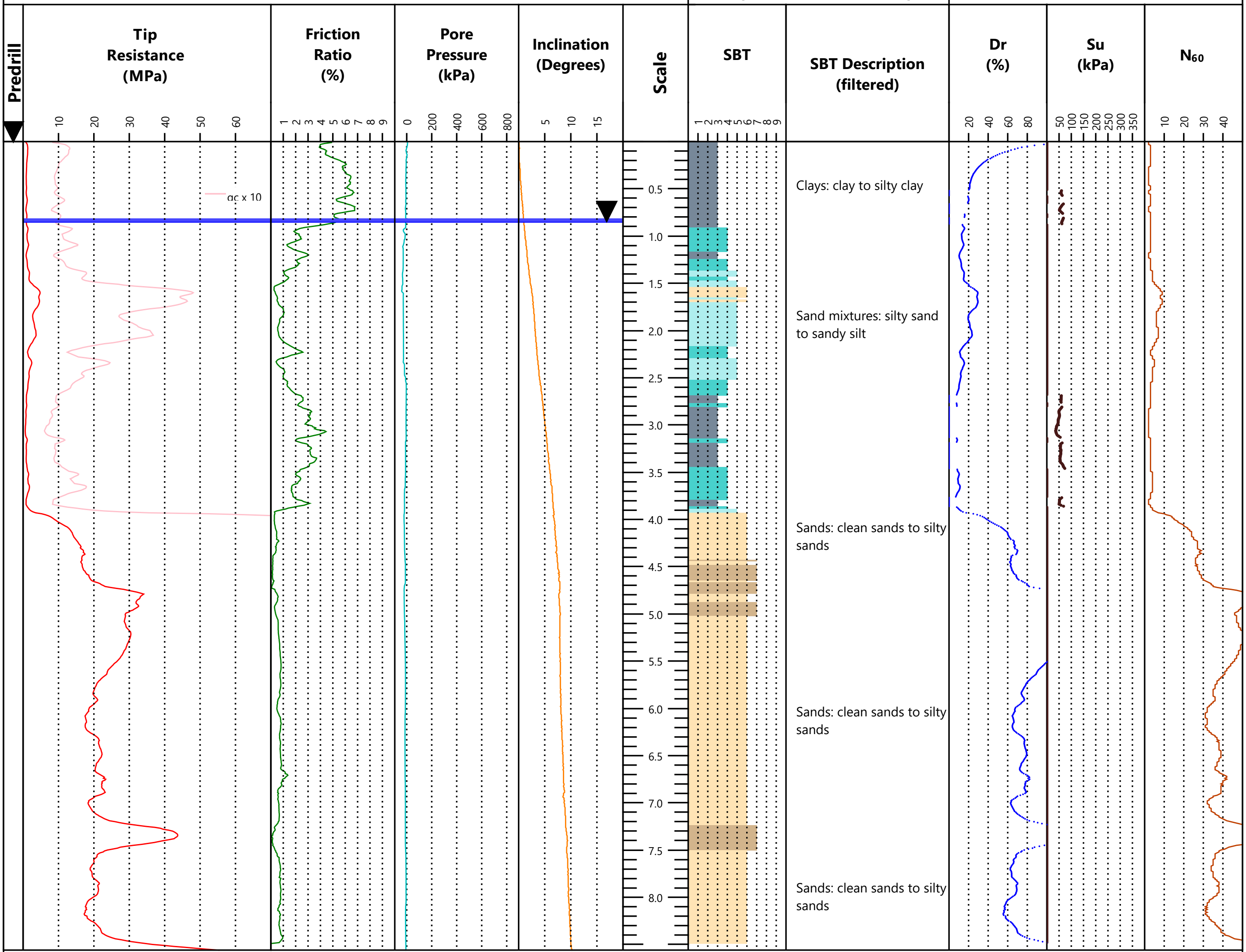
McMILLAN Drilling	Client: Tonkin and Taylor Ltd	Bore No.: CPTu105
	Project: Beach Grove Subdivision	Job No.: 20724

Site Location: Beach Road, Kaiapoi **Date:** 18/3/2022

Grid Reference: 1572864.27m E, 5197239.14m N (NZTM) - Handheld GPS **Rig Operator:** E. Diaz

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

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



EOH: 8.56m

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

Predrill: -
Water Level: 0.84m
Collapse: 1.450m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.0879	0.1142
Local Friction	0.0144	0.0093
Pore Pressure	0.0097	0.0050

Termination

Target Depth ☐

Effective Refusal
- Tip ☑
- Gauge ☐
- Inclinometer ☐
- Other ☐

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

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

Notes & Limitations

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

Remarks

McMILLAN Drilling	**Client:** Tonkin and Taylor Ltd	**Bore No.:** **CPTu106**
	Project: Beach Grove Subdivision	**Job No.:** **20724**

Site Location: Beach Road, Kaiapoi

Date: 17/3/2022

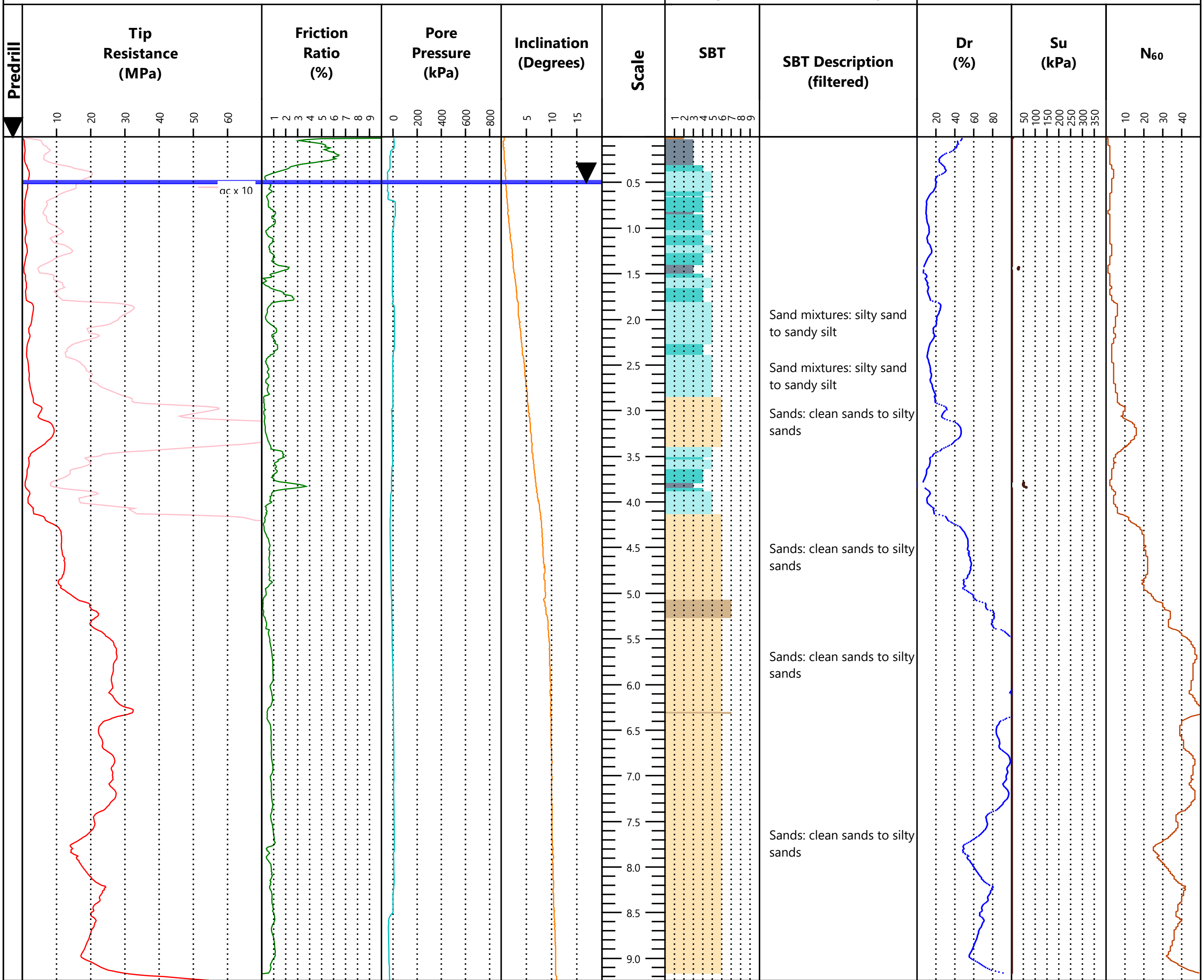
Grid Reference: 1572937.68m E, 5197193.9m N (NZTM) - Handheld GPS

Rig Operator: S. Cardona

Elevation: 0.00m **Datum:** Ground

Equipment: Geomil Panther 100

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



EOH: 9.24m

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

Predrill: -
Water Level: 0.50m
Collapse: 1.1m

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	0.0545	0.0731
Local Friction	0.0046	0.0030
Pore Pressure	-0.0035	-0.0113

Termination

Target Depth ☐

Effective Refusal

Tip	☑
Gauge	☐
Inclinometer	☐
Other	☐

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

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

Notes & Limitations

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

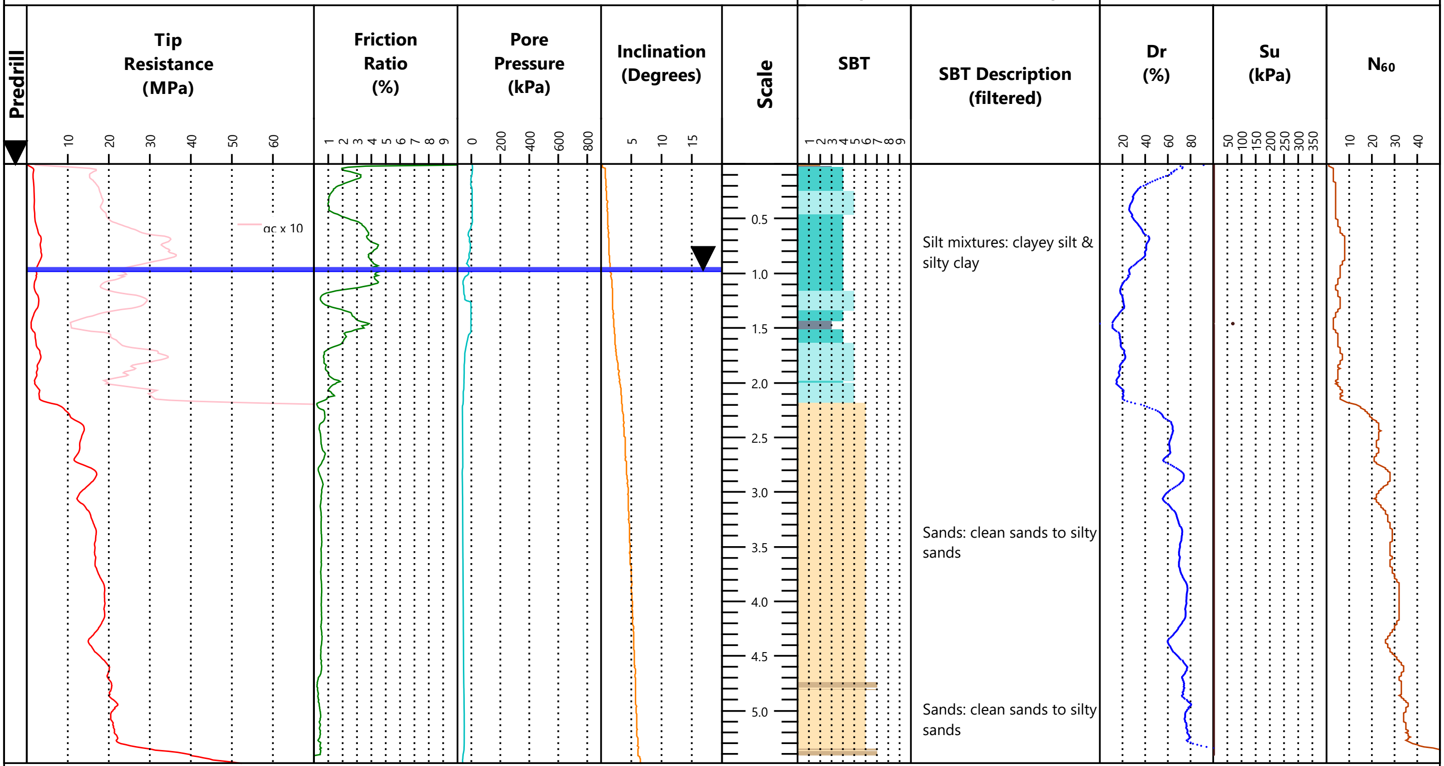
Remarks

McMILLAN Drilling	Client: Tonkin and Taylor Ltd	Bore No.: CPTu107
	Project: Beach Grove Subdivision	Job No.: 20724

Site Location: Beach Road, Kaiapoi **Date:** 18/3/2022

Grid Reference: 1572742.81m E, 5197145.59m N (NZTM) - Handheld GPS **Rig Operator:** E. Diaz

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



EOH: 5.48m

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

Predrill: -
Water Level: 0.97m
Collapse: 1.420m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.0652	0.0383
Local Friction	0.0081	0.0046
Pore Pressure	-0.0013	-0.0177

Termination

Target Depth ☐

Effective Refusal

Tip ☑
Gauge ☐
Inclinometer ☐
Other ☐

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

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

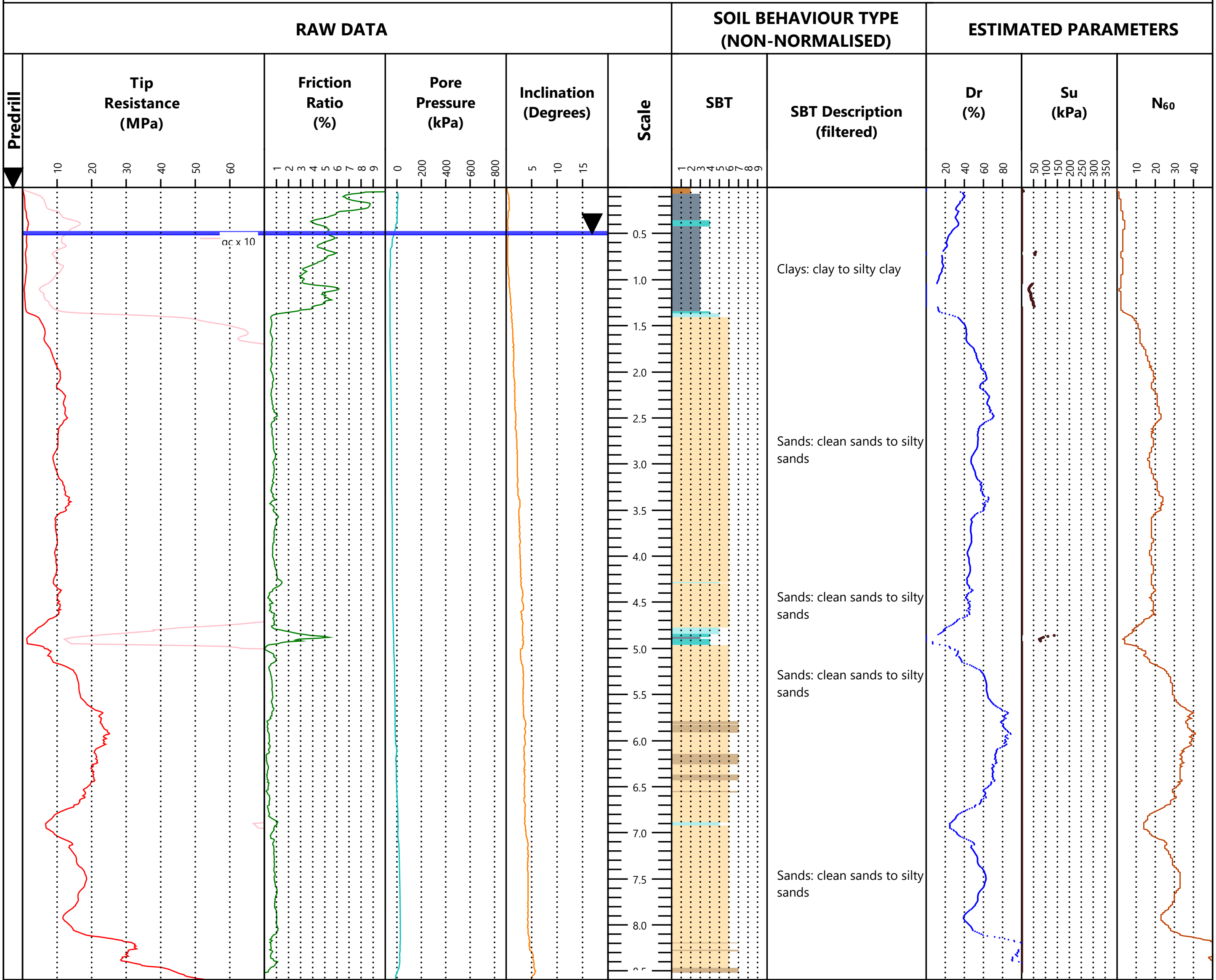
Notes & Limitations

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

Remarks

Client: Tonkin and Taylor Ltd	Bore No.: CPTu108
Project: Beach Grove Subdivision	Job No.: 20724

Site Location: Beach Road, Kaiapoi
Date: 18/3/2022
Grid Reference: 1572801.95m E, 5197124.5m N (NZTM) - Handheld GPS
Rig Operator: E. Diaz
Elevation: 0.00m **Datum:** Ground
Equipment: Geomil Panther 100



EOH: 8.59m

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

Predrill: -
Water Level: 0.50m
Collapse: 0.880m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.0923	0.2678
Local Friction	0.0233	0.0158
Pore Pressure	0.0054	0.0052

Termination

Target Depth ☐

Effective Refusal

Tip ☑
Gauge ☐
Inclinometer ☐
Other ☐

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

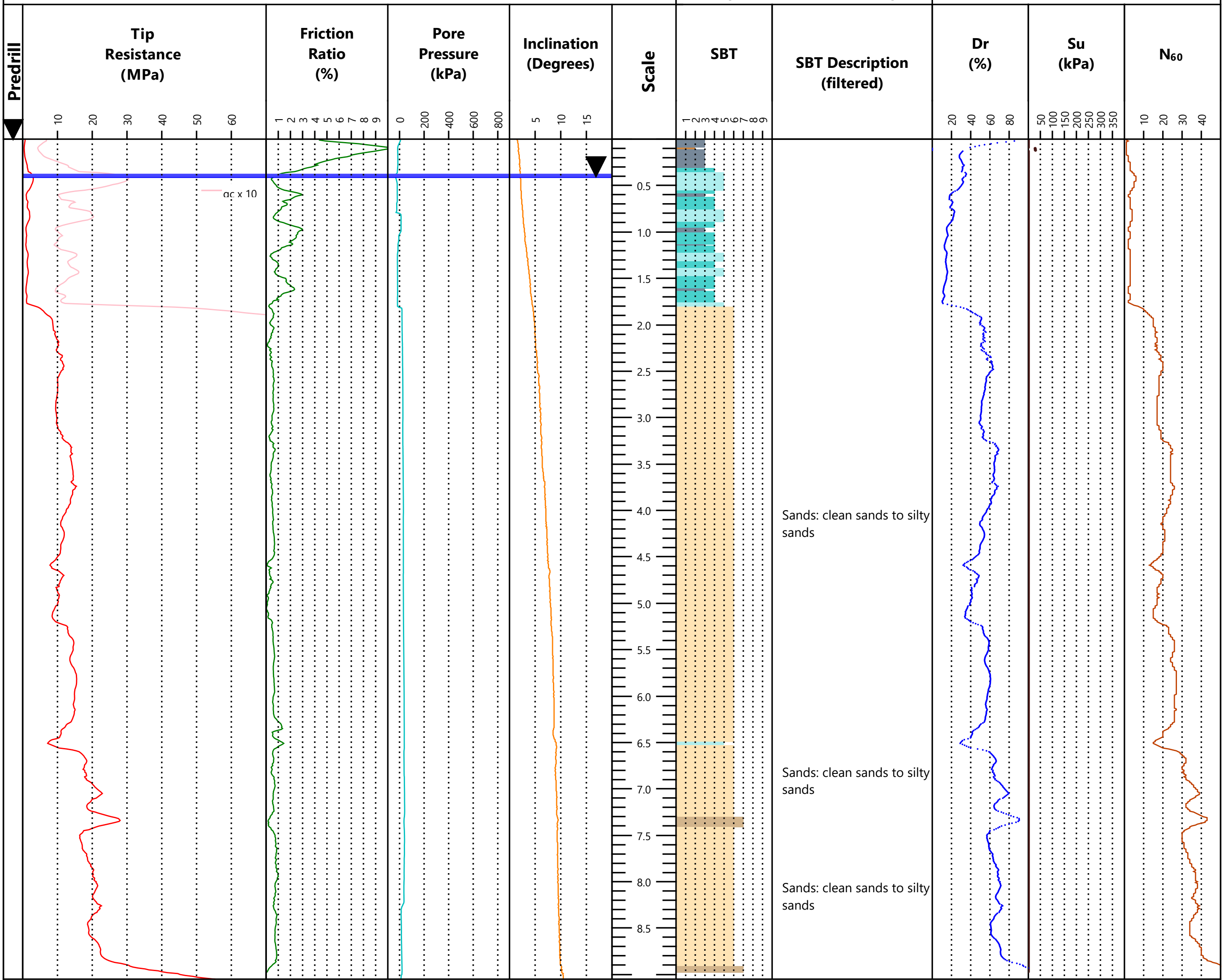
0 Undefined	5 Sand mixtures: silty sand to sandy silt
1 Sensitive fine-grained	6 Sands: clean sands to silty sands
2 Clay - organic soil	7 Dense sand to gravelly sand
3 Clays: clay to silty clay	8 Stiff sand to clayey sand
4 Silt mixtures: clayey silt & silty clay	9 Stiff fine-grained

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

Remarks

Client: Tonkin and Taylor Ltd	Bore No.: CPTu109
Project: Beach Grove Subdivision	Job No.: 20724

Site Location: Beach Road, Kaiapoi
Date: 17/3/2022
Grid Reference: 1572889.36m E, 5197124.86m N (NZTM) - Handheld GPS
Rig Operator: S. Cardona
Elevation: 0.00m **Datum:** Ground
Equipment: Geomil Panther 100



EOH: 9.05m

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

Predrill: -
Water Level: 0.40m
Collapse: 0.9m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.0201	0.0493
Local Friction	0.0138	0.0088
Pore Pressure	0.0095	0.0065

Termination

Target Depth ☐

Effective Refusal

Tip ☑
Gauge ☐
Inclinometer ☐
Other ☐

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

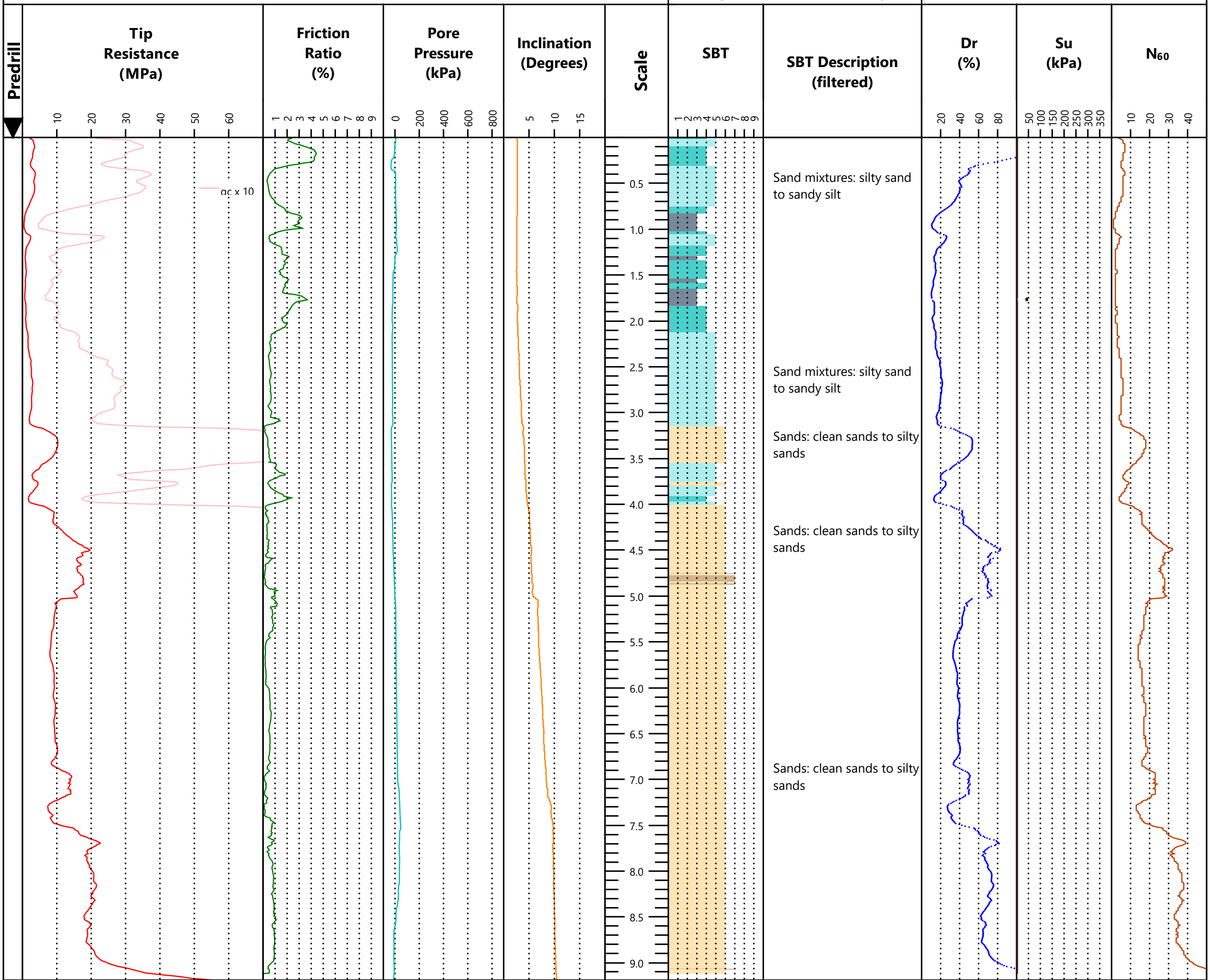
0	Undefined	5	Sand mixtures: silty sand to sandy silt
1	Sensitive fine-grained	6	Sands: clean sands to silty sands
2	Clay - organic soil	7	Dense sand to gravelly sand
3	Clays: clay to silty clay	8	Stiff sand to clayey sand
4	Silt mixtures: clayey silt & silty clay	9	Stiff fine-grained

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

Remarks

Client: Tonkin and Taylor Ltd	Bore No.: CPTu110
Project: Beach Grove Subdivision	Job No.: 20724

Site Location: Beach Road, Kaiapoi
Date: 17/3/2022
Grid Reference: 1572935.31m E, 5197038.97m N (NZTM) - Handheld GPS
Rig Operator: S. Cardona
Elevation: 0.00m **Datum:** Ground
Equipment: Geomil Panther 100



EOH: 9.19m

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

Predrill: -
Water Level: -
Collapse: 0.6m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.1101	0.0554
Local Friction	0.0032	0.0039
Pore Pressure	-0.0017	-0.0019

Termination

Target Depth ☐

Effective Refusal
Tip ☑
Gauge ☐
Inclinometer ☐
Other ☐

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

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

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

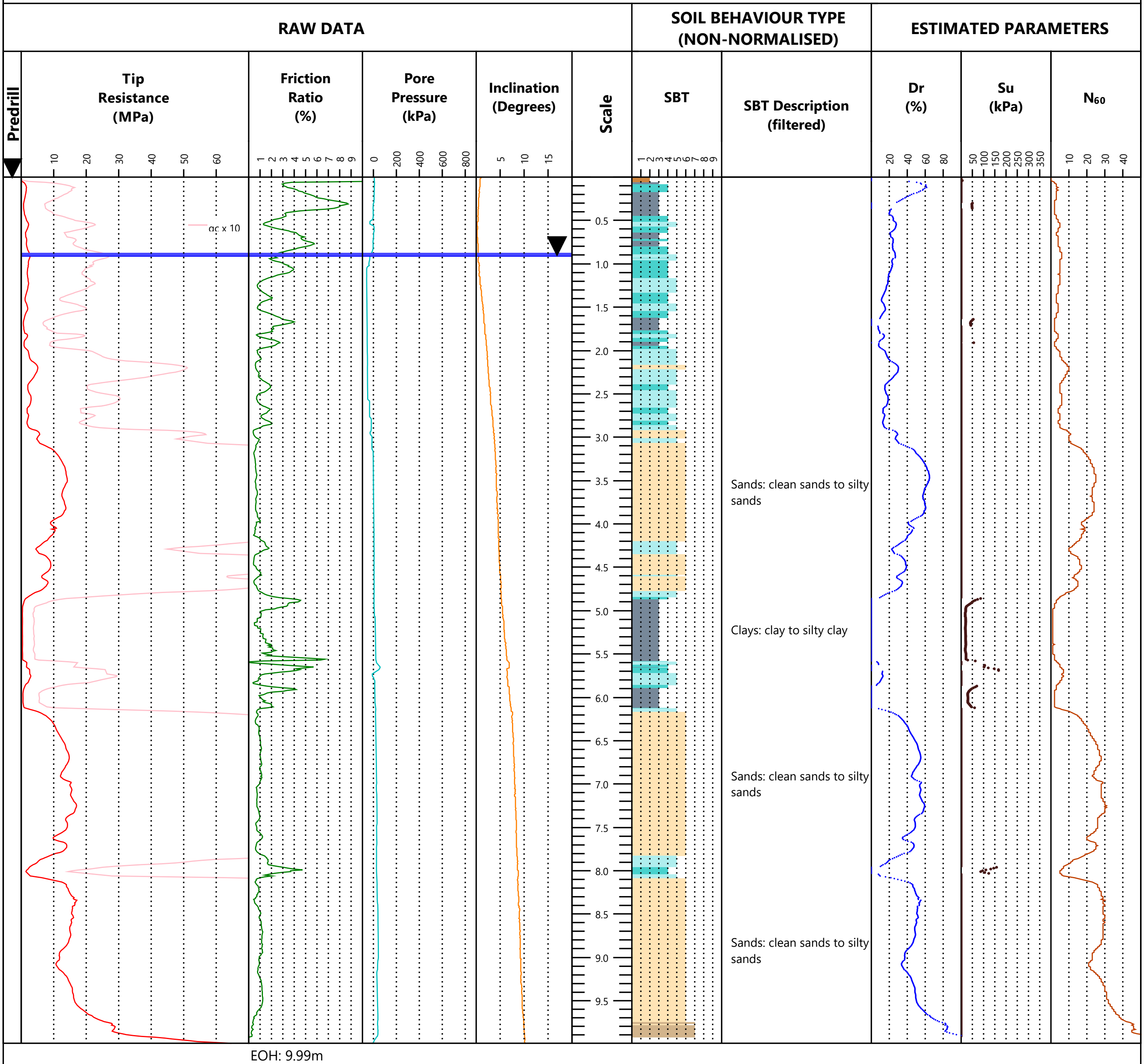
Remarks

McMILLAN Drilling	Client: Tonkin and Taylor Ltd	Bore No.: CPTu111
	Project: Beach Grove Subdivision	Job No.: 20724

Site Location: Beach Road, Kaiapoi **Date:** 17/3/2022

Grid Reference: 1573006.73m E, 5197084.9m N (NZTM) - Handheld GPS **Rig Operator:** S. Cardona

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



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

Predrill: -
Water Level: 0.90m
Collapse: 1.7m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.0584	0.0856
Local Friction	0.0052	0.0033
Pore Pressure	-0.0006	-0.0049

Termination

Target Depth ☐

Effective Refusal

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

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

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

Notes & Limitations

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

Remarks

Client: Tonkin and Taylor Ltd	**Bore No.:** CPTu111a
Project: Beach Grove Subdivision	**Job No.:** 20724

Site Location: Beach Road, Kaiapoi

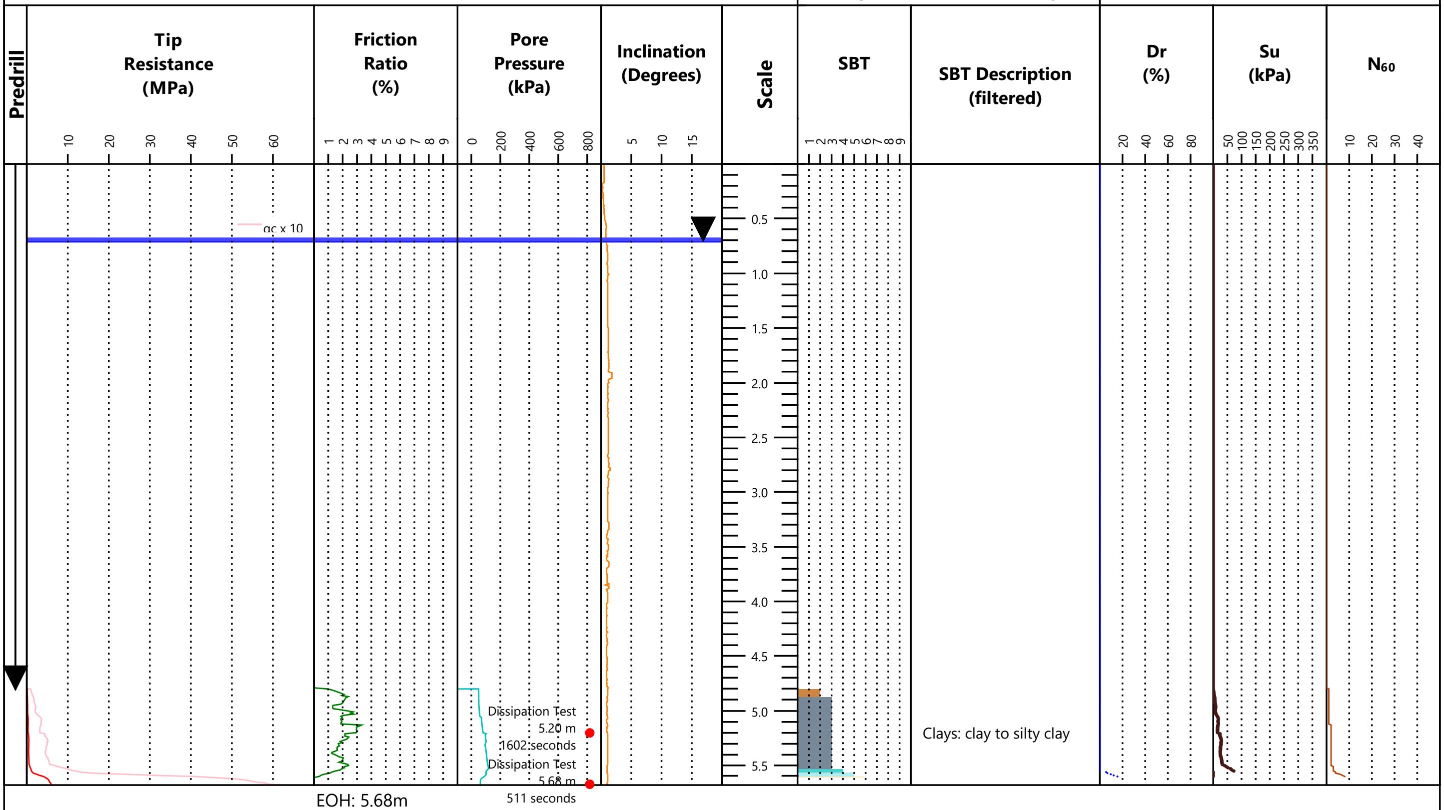
Date: 22/3/2022

Grid Reference: 1573007.22m E, 5197084.91m N (NZTM) - Map or aerial photograph

Rig Operator: S. Cardona

Elevation: 0.00m **Datum:** Ground

Equipment: Geomil Panther 100



Cone Type: I-C5F0p15XYP20-10 - Compression

Cone Reference: 150904

Cone Area Ratio: 0.75

Standards: ISO 22476-1:2012

Predrill: 4.80m

Water Level: 0.70m

Collapse: -

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.5138	0.5348
Local Friction	0.0048	0.0014
Pore Pressure	0.0167	0.0198

Termination

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

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

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

Notes & Limitations

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

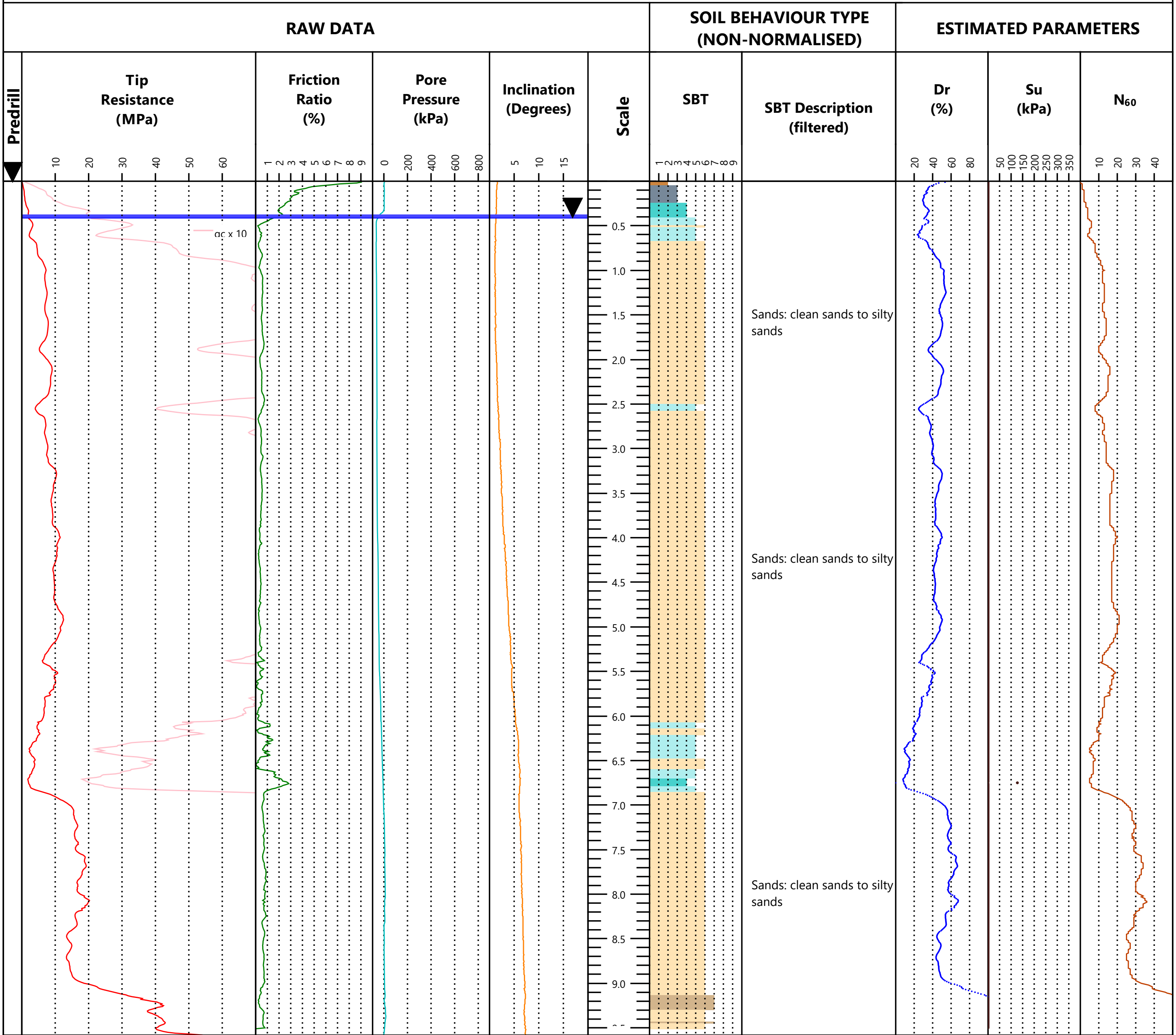
Remarks

Client: Tonkin and Taylor Ltd	Bore No.: CPTu112
Project: Beach Grove Subdivision	Job No.: 20724

Site Location: Beach Road, Kaiapoi **Date:** 17/3/2022

Grid Reference: 1573029.55m E, 5197091.21m N (NZTM) - Handheld GPS **Rig Operator:** S. Cardona

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



EOH: 9.58m

Cone Type: I-CFXYP20-10 - Compression

Cone Reference: 151125

Cone Area Ratio: 0.75

Standards: ISO 22476-1:2012

Predrill: -

Water Level: 0.40m

Collapse: 1.1m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.0058	0.0321
Local Friction	0.0089	0.0089
Pore Pressure	0.0122	0.0078

Termination

Target Depth ☐

Effective Refusal

Tip ☑
Gauge ☐
Inclinometer ☐
Other ☐

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

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

Notes & Limitations

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

Remarks

TEST DETAIL

PointID: CPTu101
Sounding: 1

Operator:	E. Diaz	**Date:**	18/3/2022
Cone Type:	I-CFXYP20-10 - Compression	**Predrill:**	0.00m
Cone Reference:	100992	**Water Level:**	0.75m
Cone Area Ratio:	0.75	**Collapse:**	0.900m

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	0.0835	0.0702
Local Friction	0.0072	0.0039
Pore Pressure	-0.0016	-0.0062

Termination

Target Depth ☐

Effective Refusal
- Tip ☑
- Gauge ☐
- Inclinometer ☐
- Other ☐

PointID: CPTu102
Sounding: 2

Operator:	E. Diaz	**Date:**	18/3/2022
Cone Type:	I-CFXYP20-10 - Compression	**Predrill:**	0.00m
Cone Reference:	151125	**Water Level:**	-
Cone Area Ratio:	0.75	**Collapse:**	0.380m

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	0.1190	0.1287
Local Friction	0.0098	0.0095
Pore Pressure	0.0056	0.0045

Termination

Target Depth ☐

Effective Refusal
- Tip ☑
- Gauge ☐
- Inclinometer ☐
- Other ☐

PointID: CPTu103
Sounding: 3

Operator:	E. Diaz	**Date:**	18/3/2022
Cone Type:	I-CFXYP20-10 - Compression	**Predrill:**	0.00m
Cone Reference:	100992	**Water Level:**	0.93m
Cone Area Ratio:	0.75	**Collapse:**	1.310m

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	0.1126	0.1044
Local Friction	0.0053	0.0043
Pore Pressure	0.0016	-0.0037

Termination

Target Depth ☐

Effective Refusal
- Tip ☑
- Gauge ☐
- Inclinometer ☐
- Other ☐

PointID: CPTu104
Sounding: 4

Operator:	E. Diaz	**Date:**	18/3/2022
Cone Type:	I-CFXYP20-10 - Compression	**Predrill:**	0.00m
Cone Reference:	151125	**Water Level:**	0.49m
Cone Area Ratio:	0.75	**Collapse:**	0.830m

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	0.0932	0.1083
Local Friction	0.0111	0.0091
Pore Pressure	0.0104	0.0048

Termination

Target Depth ☐

Effective Refusal
- Tip ☑
- Gauge ☐
- Inclinometer ☐
- Other ☐

PointID: CPTu105
Sounding: 5

Operator:	E. Diaz	**Date:**	18/3/2022
Cone Type:	I-CFXYP20-10 - Compression	**Predrill:**	0.00m
Cone Reference:	151125	**Water Level:**	0.84m
Cone Area Ratio:	0.75	**Collapse:**	1.450m

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	0.0879	0.1142
Local Friction	0.0144	0.0093
Pore Pressure	0.0097	0.0050

Termination

Target Depth ☐

Effective Refusal
- Tip ☑
- Gauge ☐
- Inclinometer ☐
- Other ☐

TEST DETAIL

PointID: CPTu106
Sounding: 6

Operator:	S. Cardona	**Date:**	17/3/2022
Cone Type:	I-CFXYP20-10 - Compression	**Predrill:**	0.00m
Cone Reference:	100992	**Water Level:**	0.50m
Cone Area Ratio:	0.75	**Collapse:**	1.1m

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	0.0545	0.0731
Local Friction	0.0046	0.0030
Pore Pressure	-0.0035	-0.0113

Termination

Target Depth ☐

Effective Refusal
- Tip ☑
- Gauge ☐
- Inclinometer ☐
- Other ☐

PointID: CPTu107
Sounding: 7

Operator:	E. Diaz	**Date:**	18/3/2022
Cone Type:	I-CFXYP20-10 - Compression	**Predrill:**	0.00m
Cone Reference:	100992	**Water Level:**	0.97m
Cone Area Ratio:	0.75	**Collapse:**	1.420m

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	0.0652	0.0383
Local Friction	0.0081	0.0046
Pore Pressure	-0.0013	-0.0177

Termination

Target Depth ☐

Effective Refusal
- Tip ☑
- Gauge ☐
- Inclinometer ☐
- Other ☐

PointID: CPTu108
Sounding: 8

Operator:	E. Diaz	**Date:**	18/3/2022
Cone Type:	I-CFXYP20-10 - Compression	**Predrill:**	0.00m
Cone Reference:	151125	**Water Level:**	0.50m
Cone Area Ratio:	0.75	**Collapse:**	0.880m

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	0.0923	0.2678
Local Friction	0.0233	0.0158
Pore Pressure	0.0054	0.0052

Termination

Target Depth ☐

Effective Refusal
- Tip ☑
- Gauge ☐
- Inclinometer ☐
- Other ☐

PointID: CPTu109
Sounding: 9

Operator:	S. Cardona	**Date:**	17/3/2022
Cone Type:	I-CFXYP20-10 - Compression	**Predrill:**	0.00m
Cone Reference:	151125	**Water Level:**	0.40m
Cone Area Ratio:	0.75	**Collapse:**	0.9m

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	0.0201	0.0493
Local Friction	0.0138	0.0088
Pore Pressure	0.0095	0.0065

Termination

Target Depth ☐

Effective Refusal
- Tip ☑
- Gauge ☐
- Inclinometer ☐
- Other ☐

PointID: CPTu110
Sounding: 10

Operator:	S. Cardona	**Date:**	17/3/2022
Cone Type:	I-CFXYP20-10 - Compression	**Predrill:**	0.00m
Cone Reference:	100992	**Water Level:**	-
Cone Area Ratio:	0.75	**Collapse:**	0.6m

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	0.1101	0.0554
Local Friction	0.0032	0.0039
Pore Pressure	-0.0017	-0.0019

Termination

Target Depth ☐

Effective Refusal
- Tip ☑
- Gauge ☐
- Inclinometer ☐
- Other ☐

TEST DETAIL

PointID: CPTu111
Sounding: 11

Operator: S. Cardona
Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 100992
Cone Area Ratio: 0.75

Date: 17/3/2022
Predrill: 0.00m
Water Level: 0.90m
Collapse: 1.7m

Termination

Target Depth ☐

Effective Refusal
Tip ☑
Gauge ☐
Inclinometer ☐
Other ☐

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.0584	0.0856
Local Friction	0.0052	0.0033
Pore Pressure	-0.0006	-0.0049

PointID: CPTu111a
Sounding: 111

Operator: S. Cardona
Cone Type: I-C5F0p15XYP20-10 - Compression
Cone Reference: 150904
Cone Area Ratio: 0.75

Date: 22/3/2022
Predrill: 4.80m
Water Level: 0.70m
Collapse: -

Termination

Target Depth ☐

Effective Refusal
Tip ☑
Gauge ☐
Inclinometer ☐
Other ☐

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.5138	0.5348
Local Friction	0.0048	0.0014
Pore Pressure	0.0167	0.0198

PointID: CPTu112
Sounding: 12

Operator: S. Cardona
Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 151125
Cone Area Ratio: 0.75

Date: 17/3/2022
Predrill: 0.00m
Water Level: 0.40m
Collapse: 1.1m

Termination

Target Depth ☐

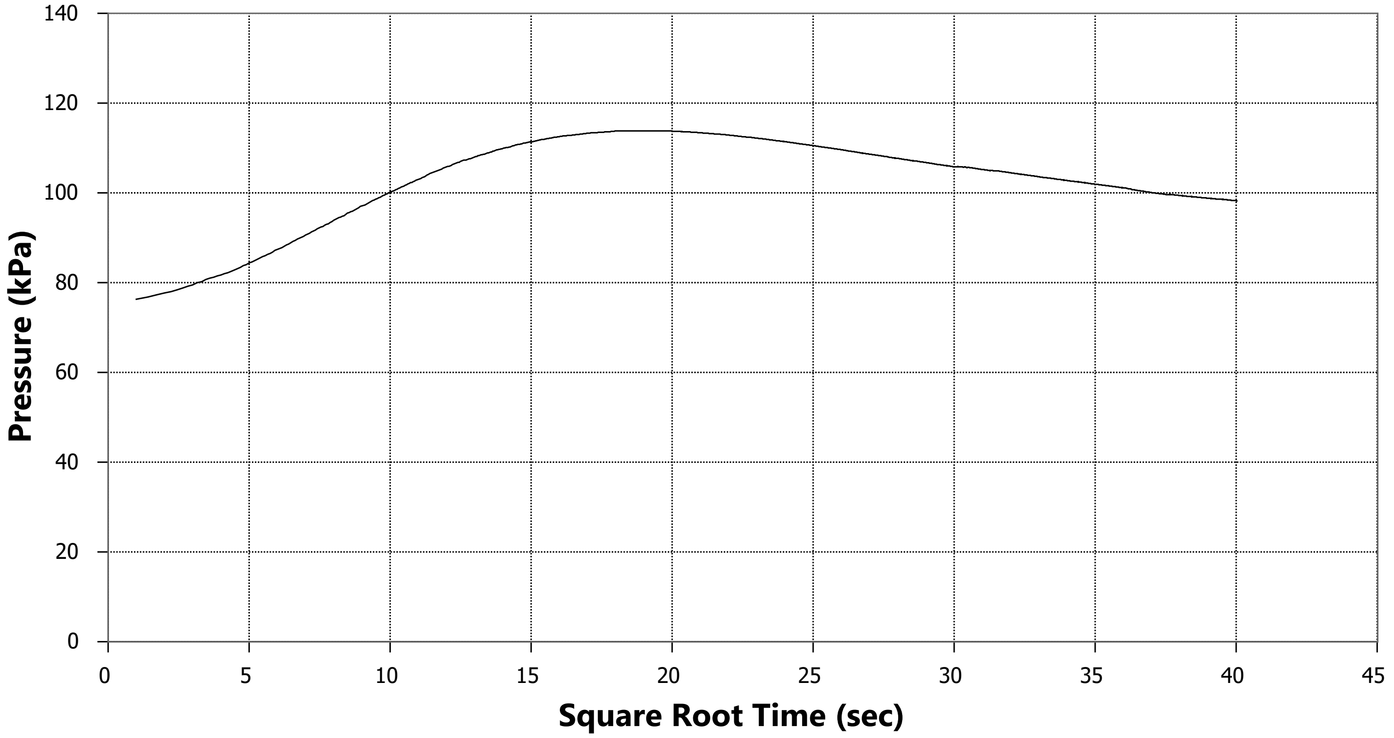
Effective Refusal
Tip ☑
Gauge ☐
Inclinometer ☐
Other ☐

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.0058	0.0321
Local Friction	0.0089	0.0089
Pore Pressure	0.0122	0.0078

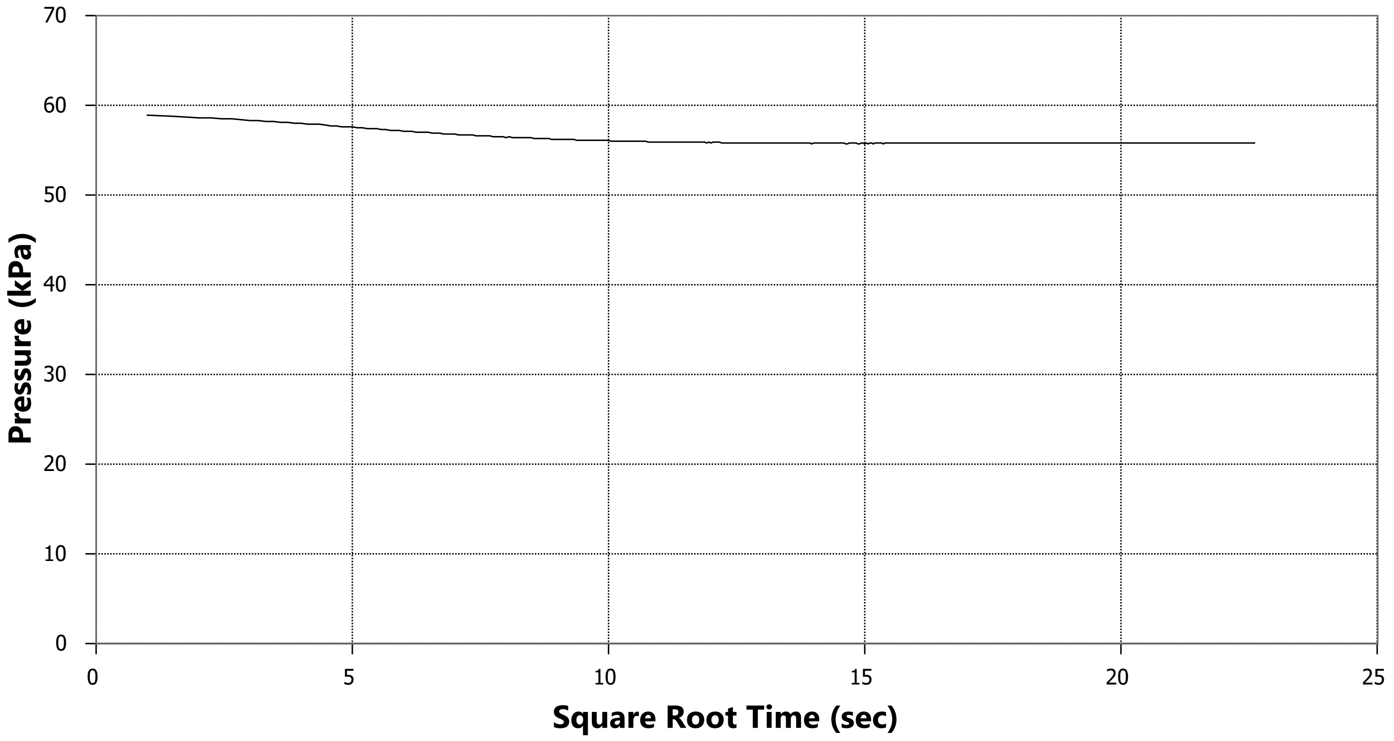
DISSIPATION TESTS

PointID: CPTu111a

Test Depth: 5.20 **Duration:** 1602 seconds



Test Depth: 5.68 **Duration:** 511 seconds



CPT CALIBRATION AND TECHNICAL NOTES

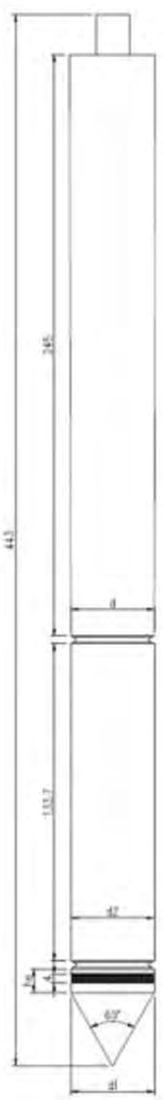
These notes describe the technical specifications and associated calibration references pertaining to the following cone types:

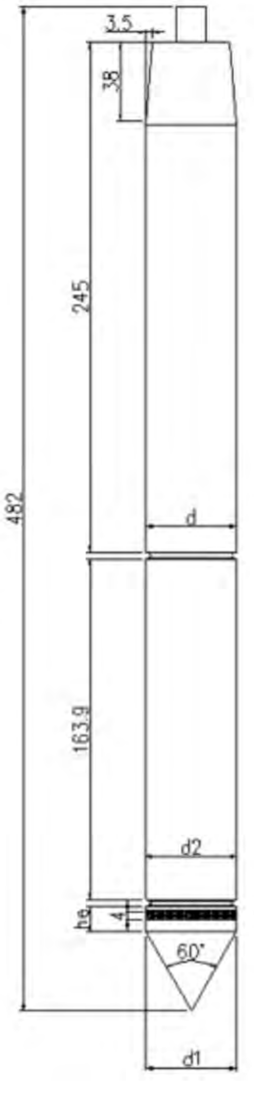
- I-CFXY-10 measuring cone resistance, sleeve friction and inclination (standard cone, $10cm^2$);
- I-CFXY-15 measuring cone resistance, sleeve friction and inclination (standard cone, $15cm^2$);
- I-CFXYP20-10 measuring cone resistance, sleeve friction, inclination and pore pressure (piezocone, $10cm^2$);
- I-CFXYP100-10 measuring cone resistance, sleeve friction, inclination and high range pore pressure (piezocone, $10cm^2$);
- I-C2xFXYP100-10 measuring cone resistance, high range sleeve friction, inclination and high range pore pressure (piezocone, $10cm^2$);
- I-C5F0p15XYP20-10 measuring sensitive cone resistance, sleeve friction, inclination and pore pressure (piezocone, $10cm^2$).
- I-CFXYP20-15 measuring cone resistance, sleeve friction, inclination and pore pressure (piezocone, $15cm^2$);

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

A.P. van den Berg Machinefabriek tel.: +31 (0)513-631355 info@apvandenberg.com	**DEVIATION of Straightness + MINIMUM Dimensions tip, friction jacket, cone adapter**	Standards: EN ISO 22476-1 APB-standard	

Type of cone:	Icone 10 cm^2	Icone 15 cm^2
ALLOWABLE SIZE VARIATION		
Diameter of tip:	$35{,}3 \le d1 \le 36{,}0$	$43{,}2 \le d_1 \le 44{,}1$
Diameter of centering ring CFP	$35{,}3 \le d1 \le 36{,}0$	$43{,}2 \le d_1 \le 44{,}1$
Diameter of friction jacket:	$d_1 \le d_2 < d_1 + 0{,}35$	$d_1 \le d_2 < d_1 + 0{,}43$
Height dimension of tip edge:	$7 \le h_e \le 10$	$9 \le h_e \le 12$
PRODUCTION DIMENSIONS		
Tip:	$d_1 = 35{,}7^{+0{,}2}_{0}$	$d_1 = 43{,}8^{+0{,}2}_{0}$
Jacket (C-cone):	$d_2 = 35{,}7^{+0{,}2}_{0}$	$d_2 = 43{,}7^{+0{,}2}_{0}$
Friction jacket (CF-cone):	$d_2 = 35{,}9^{+0{,}1}_{0}$	$d_2 = 44{,}0^{+0{,}1}_{0}$
Tip for used cone:	$d_1 = 35{,}5^{+0{,}1}_{0}$	$d_1 = 43{,}5^{+0{,}1}_{0}$
MINIMUM DIMENSIONS		
Minimum diameter jacket (C-cone):	$d_2 = 35{,}2$ (APB standard)	$d_2 = 43{,}0$ (APB standard)
Minimum diameter friction jacket (CF-cone):	$d_2 = 35{,}3$	$d_2 = 43{,}2$
Use "used cone"-tip when friction jacket diameter:	$d_2 \le 35{,}65$	$d_2 \le 43{,}7$
Minimum diameter of cone adaptor:	$d = 35{,}3$	$d = 43{,}8$
Maximum deviation of straightness:	1 mm on a length of 1000 mm (max. oscillation 1,0 mm.)	1 mm on a length of 1000 mm (max. oscillation: 2.0 mm)

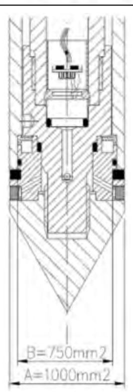




Tip and Local Friction sensor displacement

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

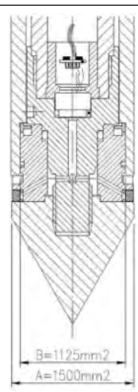
- $10cm^2$ cones: 80mm
- $15cm^2$ cones: 100mm



Cone area ratio

$\alpha = B / A = 0.75$

$\beta = 1 - B / A = 0.25$



McMILLAN Drilling

CPT CALIBRATION AND TECHNICAL NOTES

Calibration

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

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

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

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

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

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

McMILLAN Drilling

Calibration Certificate

CALIBRATION RvA K 178

a.p. van den berg

1.1 General

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

1.2 Calibration equipment

Reference measuring equipment:

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

1.3 Laboratory conditions:

Ambient temperature: 23.8 ± 2 °C

1.4 Measurement uncertainty

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

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

1.5 Standard and method of calibration

EN ISO 22476-1 2012 Class 2

1.6 Results

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

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

1.7 Remarks

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

Calibration Certificate

a.p. van den berg

1.1 General

Cone number:	150904
Cone type:	I-C5F0p15XYP20-10
Description:	Tip 7.5 MPa Sleeve 0.15 MPa Inclinometer 20° Pore 2MPa
Part number:	0100269A
Certificate number:	150904-3
Client:	McMillan

1.2 Calibration equipment

	calibrated
Autolog 3000	
Autolog 3000	August 2017 (Peekel: SN# 2628009)
Autolog 3000	August 2017 (Peekel: SN# 2628009)
Autolog 3000	August 2017 (Peekel: SN# 2628009)
Reference Loadcell 20kN H22789	Sept 2017 (HBM: 64667 2017-09)
Reference Loadcell 20kN H22789	Sept 2017 (HBM: 64667 2017-09)
Reference Sensor 40 Bar 5447380	Aug 2018 (GE Druck: 0079091)
Reference ACS-080-SC00-HP2-PM 08/11 470481	March 2015 (Trescal: 1503-02689)
Reference ACS-080-SC00-HP2-PM 08/11 470481	March 2015 (Trescal: 1503-02689)

1.3 Standard
EN ISO 22476-1 2012 Class 2

1.4 Result
The sensor complies to the above standard

Calibrated by: C.J. Ouwejan
Date: 09/04/2019
Signature:

QA Manager: N.R.E. de Jong
Date: 09/04/2019
Signature:

Calibration Certificate

a.p. van den berg

Zero Value			Max. Deviation from Zero Value		
Cone	0.006	[MPa]	Cone	0.375	[MPa]
Sleeve	0.001	[MPa]	Sleeve	0.0075	[MPa]
Pore(u2)	1.2	[kPa]	Pore(u2)	100.0	[kPa]

Ref [MPa]	Cone [MPa]	Cone-Ref [kPa]
0.000	0.006	6
0.108	0.109	1
0.197	0.193	-4
0.423	0.422	-1
0.821	0.832	11
1.222	1.240	18
2.094	2.119	25
3.014	3.039	25
4.065	4.088	23
5.035	5.051	16
6.090	6.103	13
7.579	7.582	3

Ref [MPa]	Sleeve [MPa]	Sleeve-Ref [kPa]
0.000	0.000	0
0.007	0.008	1
0.013	0.014	1
0.020	0.020	0
0.025	0.025	0
0.042	0.041	-1
0.056	0.056	0
0.070	0.070	0
0.081	0.081	0
0.094	0.094	0
0.110	0.111	1
0.136	0.136	0

Ref [MPa]	Pore(u2) [MPa]	Pore(u2)-Ref [kPa]
0.000	0.001	1
0.107	0.110	3
0.199	0.201	2
0.318	0.321	3
0.401	0.405	4
0.596	0.600	4
0.849	0.853	4
1.040	1.044	4
1.223	1.227	4
1.386	1.390	4
1.615	1.619	4
2.061	2.063	2

McMILLAN Drilling

Calibration Certificate

CALIBRATION RvA K 176

a.p. van den berg

1.1 General

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

1.2 Calibration equipment

Reference measuring equipment:

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

1.3 Laboratory conditions:

Ambient temperature: 23.0 ± 2 °C

1.4 Measurement uncertainty

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

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

1.5 Standard and method of calibration

EN ISO 22476-1 2012 Class 2

1.6 Results

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

Calibrated by:	D.Bisschops
Calibration Date:	24 November 2021
Signature:	
QA Manager:	N.R.E. de Jong
Date:	24 November 2021
Signature:	

Expiration date according to EN ISO 22476-1: 25 May 2022

1.7 Remarks

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

May 2022

Geophysical Investigation:
MASW & GPR Survey
Stage 3 – Block 1, Beach Grove Subdivision, Kaiapoi

Report prepared for Tonkin & Taylor

Geophysical Report

Southern Geophysical
3/28 Tanya St, Bromley
Christchurch 8062
Tel. 03 384 4302
www.southerngeophysical.com

Table of Contents

Data collected and report prepared for Southern Geophysical Ltd by:

- M. Martin (BSc), Geophysicist | Survey Manager
- R. McConachie (PgDip), Geophysicist
- A. Aspinwall (MASt), Geophysics Technician

Capability Statement:

Southern Geophysical Ltd's experienced team provides geophysical contracting and consulting services to clients in the energy, geotechnical, civil engineering, mineral, archaeological, agricultural, and environmental sectors. We have one of the largest equipment resources for shallow geophysical surveys in the independent private sector in New Zealand.

We are proudly Canterbury owned and operated and have been since our beginnings in 2004. We operate in New Zealand, Australia, the Pacific Islands, and Antarctica. Some of the major projects that we have worked on include:

- Recovery and rebuild projects in Christchurch, post Canterbury Earthquake Sequence
- Deep ground water reconnaissance surveying in Wellington and Invercargill
- Wind farm site investigations
- Basalt bedrock profiling and lava cave detection throughout Auckland
- Port infrastructure investigations
- Large scale UXO surveys
- Seismic network maintenance
- Cemetery surveys

Southern Geophysical Ltd has extensive experience with geophysical investigations. We have worked on over 2000 projects throughout New Zealand, working with geotechnical and engineering companies, allowing us to be involved with many of the larger infrastructure projects throughout New Zealand.

Our team is confident and capable of utilising the widest range of geophysical systems. We have clocked up over 3000 hours of GPR applications over the last 17 years, run numerous large scale MASW surveys, and have fielded hundreds of kilometres of EM31 and EM61 investigations throughout New Zealand. The team is highly skilled in the processing of all data acquired, reporting in the way the client requests and being at the end of the phone or an email to respond to queries which may arise.

SGL Job Reference: 2369
Version 2 (Issued May 4 2022)

Internally reviewed by:
C. Ruegg (MSc), Senior Geophysicist

SITESAFE MEMBER

Summary:

Southern Geophysical Ltd (SGL) was contracted to undertake a geophysical survey using Multi-channel Analysis of Surface Waves (MASW) and Ground Penetrating Radar (GPR) at Block 1 of Stage 3, Beach Grove Subdivision, Kaiapoi on the 25th of March, 2022. The aims of the survey were to model the shear-wave velocity structure of the subsurface and identify and map the extents of peat in the shallow subsurface, if present.

Only areas directly below the transects have been investigated. If more detailed information on any part of the site is required, additional geophysical investigations could be conducted with closer transect spacing.

Methodology:

Site Description:

The terrain was undeveloped grass paddocks with livestock fencing and paved driveways and no significant topography (Figure 1 and 4). Weather conditions were fine with little to no wind.

MASW:

MASW is a geophysical technique that uses the dispersive nature of surface waves to model shear-wave velocity versus depth.

A MASW survey is undertaken as a series of transects or points across the surface of the site. The MASW points in this survey were collected using a 24-channel towed seismic array, with 4.5 Hz geophones. The geophone spacing was 1 m and the source offset was 10 m. The seismic source was an 8 lb sledgehammer impacting an aluminium plate. Recording parameters for the MASW survey were set with a 0.25 ms sample interval, 1.5 s record length, 24 dB gains, and an electrical trigger system.

The field records were processed using the Kansas Geological Survey software package SurfSeis6++ ©. The geometry for each point was set according to the survey parameters and the dispersion curves were generated and edited. The inversions were run using a 10-layer variable depth model. The velocity data were interpolated into 2D profiles showing V_s variations with depth (Figures 2 and 3). The output shear-wave velocity data are included as data files (CSV format), supplementary to this report.

GPR:

GPR is a non-invasive geophysical technique for imaging subsurface conditions. A few of the more common applications are identification of concrete thicknesses, soil strata, bedrock boundaries, underground pipelines, voids, boulders and buried trees. It has the highest resolution of any geophysical method for imaging near surface features. GPR operation in the field is conducted by moving an antenna across the surface of the ground along pre-determined grid lines. The antenna transmits pulses of electromagnetic signal at frequencies ranging from 25 MHz to 2700 MHz into the ground and detects the reflected signal from subsurface features. The strength of the reflected signal is largely dependent on the contrast in dielectric between the subsurface materials encountered. The antenna is connected to a central control computer that collects, displays, and stores the data received from the antenna. The resolution possible with GPR is determined by the frequency of the electromagnetic signal. Higher frequency GPR systems produce higher resolutions. The depth of penetration, however, decreases with increasing frequency. In order to maximise depth penetration at Beach Grove Block 1, a shielded GSSI 200 MHz HyperStacking® GPR system was used.

The GPR acquisition parameters used at Beach Grove Block 1, Kaiapoi were:

- Antenna frequency – 200 MHz
- Trace increment – 2.5 cm
- Sample per trace - 4096
- Time increment - 0.0933 ns
- GPR system - Panasonic Toughpad G1 and GSSI 200 MHz HyperStacking® antenna
- Radar data format – RADAN

Processing:

Post-processing was applied to the GPR radargrams using the Reflexw© software package. The processing steps were:

1. Remove header gains
2. Time zero selected (positive first peak of direct wave)
3. Apply dewow (10 ns time window)
4. Apply energy decay
5. Apply background removal for whole line
6. To display the depths correctly in the radargrams a replacement velocity of 0.09 m/ns was used (found from reflection hyperbolas)
7. The data were exported to the Golden Software Surfer © program and output as 2D radargram profiles

Example radargrams are annotated in Figures 5 and 6. All data have been digitally archived and are available on request.

GNSS:

Survey positions were recorded using a Geo 7X Trimble GNSS system with a Tornado antenna. The GNSS positions were differentially corrected using a local GeoNet base station. The GNSS points were output in NZTM 2000, with heights in Mean Sea Level (MSL). The accuracy of the survey positions is +/- 0.1 m. The site had no significant topographic changes, and the transects have not been corrected for elevation.

Results:

A total of five MASW transects and 26 GPR transects were acquired at the site, with a total MASW survey length of approximately 811 m (Figure 1) and a total GPR survey length of 1456 m (Figure 4).

The MASW results were correlated against CPT logs provided by Tonkin & Taylor, CPTs in line with the MASW survey lines were plotted on the site map and MASW profiles. The depth at which most CPT's refuse correlates with approximately 200 m/s shear-wave velocity (Figures 2 and 3).

The GPR data were of good quality and clearly imaged from the subsurface to 3 m depth (Figures 5 and 6). Some larger buried objects were seen in the GPR radargrams up to 3 m depth. Channel features were seen in several radargrams; however, these features could not be interpolated due to the large distance between transects. The presence of numerous buried features (possible buried trees) as well as an increase in the GPR depth penetration within the channel feature's extents, may indicate that the channel features are infilled with "peat".

Limitations:

The MASW survey was considered to be of good quality, with modelled shear-wave velocities to 15 - 20 m depth. The velocities in the top 5 m are likely to be more accurate than the deeper velocities, due to the presence of velocity inversions.

In homogenous soils, with gradually increasing shear-wave velocities and no sharp lateral discontinuities, the accuracy of the shear-wave velocities derived from the MASW processing is considered to be +/- 10%.[1] The quality of the seismic data and the dispersion curves used in this report are good, with a good signal-to-noise ratio. If there is a velocity inversion present in the shear-wave profile (decreasing velocity with depth), the shear-wave velocity of the

[1] Stephenson, W.J., Louie, J.N., Pullammanappallil, S., Williams, R.A., and Odum, J.K. 2005. Blind Shear-wave Velocity Comparison of ReMi and MASW Results with Boreholes to 200 m in Santa Clara Valley: Implications for Earthquake Ground-Motion Assessment. *Bulletin of the Seismological Society of America*, Vol. 95, pp. 2506-2516.

reduced velocity zone and the thickness of that zone can often be underestimated by the inversion process.

The capabilities of a MASW system to confidently model shear-wave velocities with depth is dependent on the frequency of the geophones used, the spacing between the geophones, the distance of the shot offset, and the frequency and velocity of the surface waves at any given point. Designing the MASW system for a survey is a balancing act between achieving good resolution in the near surface, while still achieving the required target depths.

The highest confidence results from the MASW system used at Beach Grove are shear-wave velocities from 100 m/s to 500 m/s, and a depth range of 2 m to 15 m. Less than 2 m and up to 25 m depth the confidence is still good, and the system is theoretically able to accurately model shear-wave velocities up to 700 m/s. Any results deeper than 25 m or shear-wave velocities greater than 700 m/s should be treated with caution. A more detailed discussion on the accuracy of the MASW derived shear-wave velocities can be provided on request.

While the limitations of the MASW method should be considered when evaluating these results, the quality of the data collected at the site and the confidence in the shear-wave velocities derived from the MASW data are good.

Disclaimer:

This document has been provided by Southern Geophysical Ltd subject to the following: Non-invasive geophysical testing has limitations and is not a complete source of testing. Often there is a need to couple non-invasive methods with invasive testing methods, such as drilling, especially in cases where the non-invasive testing indicates anomalies.

This document has been prepared for the particular purpose outlined in the project proposal and no responsibility is accepted for the use of this document, in whole or in part, in other contexts or for any other purpose. Southern Geophysical Ltd did not perform a complete assessment of all possible conditions or circumstances that may exist at the site. Conditions may exist which were undetectable given the limited nature of the enquiry Southern Geophysical Ltd was retained to undertake with respect to the site. Variations in conditions often occur between investigatory locations, and there may be special conditions pertaining to the site which have not been revealed by the investigation and which have not therefore been taken into account. Accordingly, additional studies and actions may be required by the client.

We collected our data and based our report on information which was collected at a specific point in time. The passage of time affects the information and assessment provided by Southern Geophysical Ltd. It is understood that the services provided allowed Southern Geophysical Ltd to form no more than an opinion of the actual conditions of the site at the time the site was visited and cannot be used to assess the effect of any subsequent changes for whatever reason. Where data are supplied by the client or other sources, including where previous site investigation data have been used, it has been assumed that the information is correct. No responsibility is accepted by Southern Geophysical Ltd for incomplete or inaccurate data supplied by others. This document is provided for sole use by the client and is confidential to that client and its professional advisers. No responsibility whatsoever for the contents of this document will be accepted to any person other than the client. Any use which a third party makes of this document, or any reliance on or decisions to be made based on it, is the responsibility of such third parties. Southern Geophysical Ltd accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this document.

DRAWING -	**Figure 1: Block 1 MASW Survey Site Map**
LOCATION -	**Beach Grove, Kaiapoi**

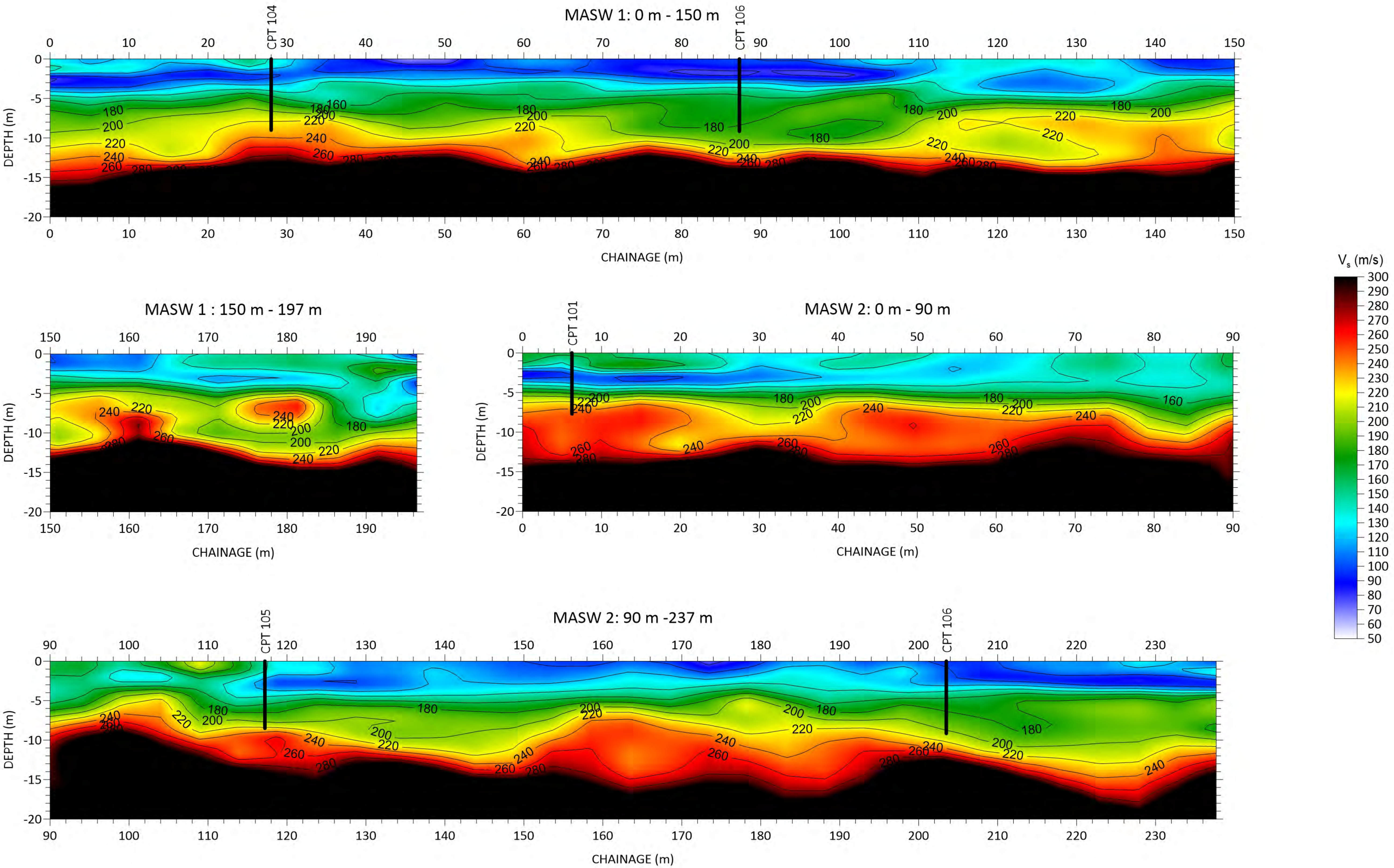
NOTES -

Coordinates projected in NZTM2000.
Aerial photograph sourced from LINZ, Crown Copyright ©
<u>CPT & Borehole</u>
CPT locations labelled by cyan triangles
Borehole labelled by black borehole symbol
<u>MASW</u>
MASW lines marked with red circles, chainage marked at every 2nd shot point

N

A3

Southern Geophysical
www.southerngeophysical.com



DRAWING - **Figure 2: Block 1, MASW 2D V_s Profiles 1 & 2**

LOCATION - **Beach Grove, Kaiapoi**

NOTES - MASW V_s profile has contour intervals of 20 m/s (V_s).

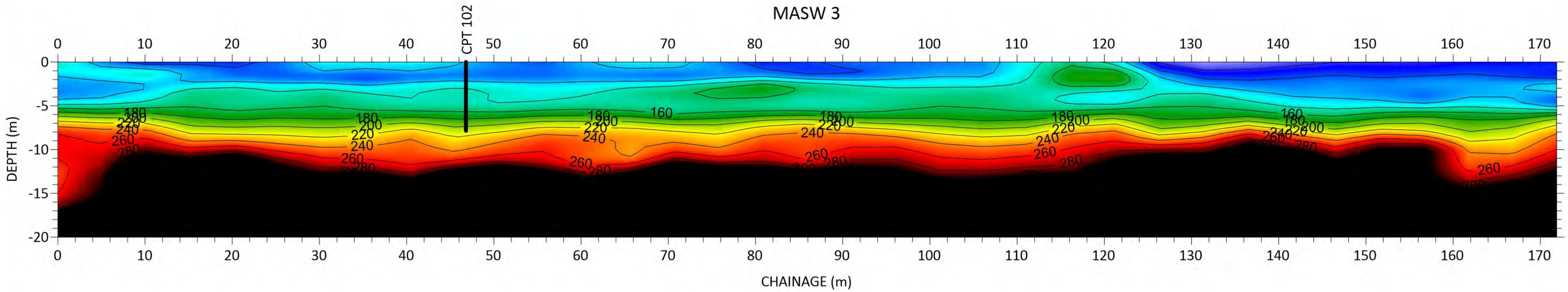
CPTs are drawn to refusal depths provided by Tonkin & Taylor.

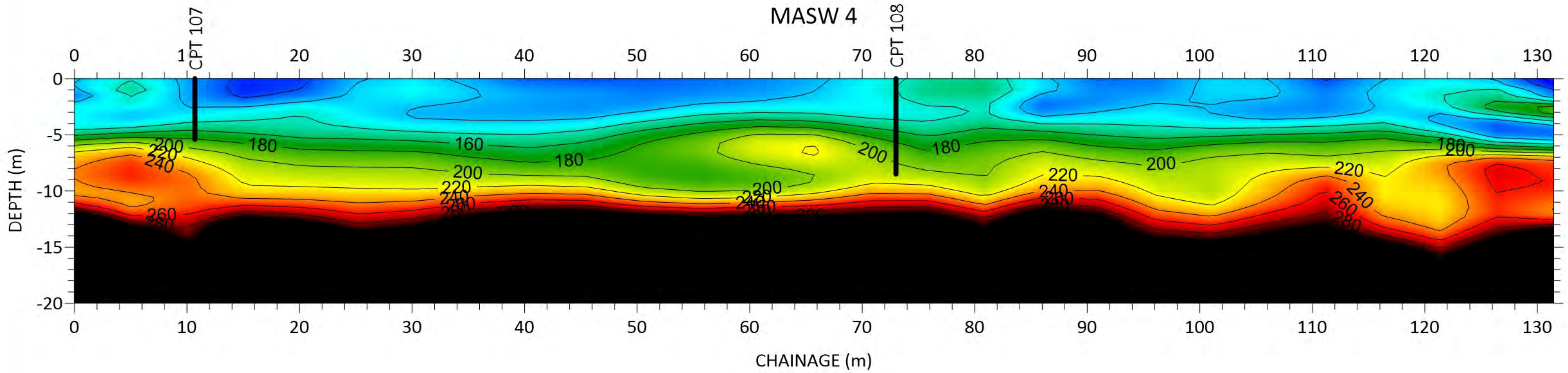
See site map for location of points.

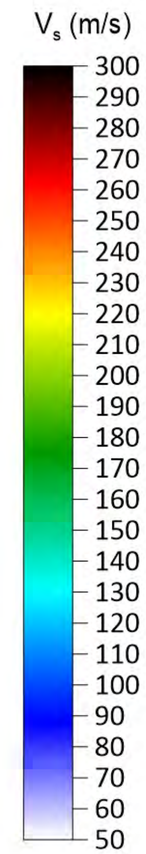
A3

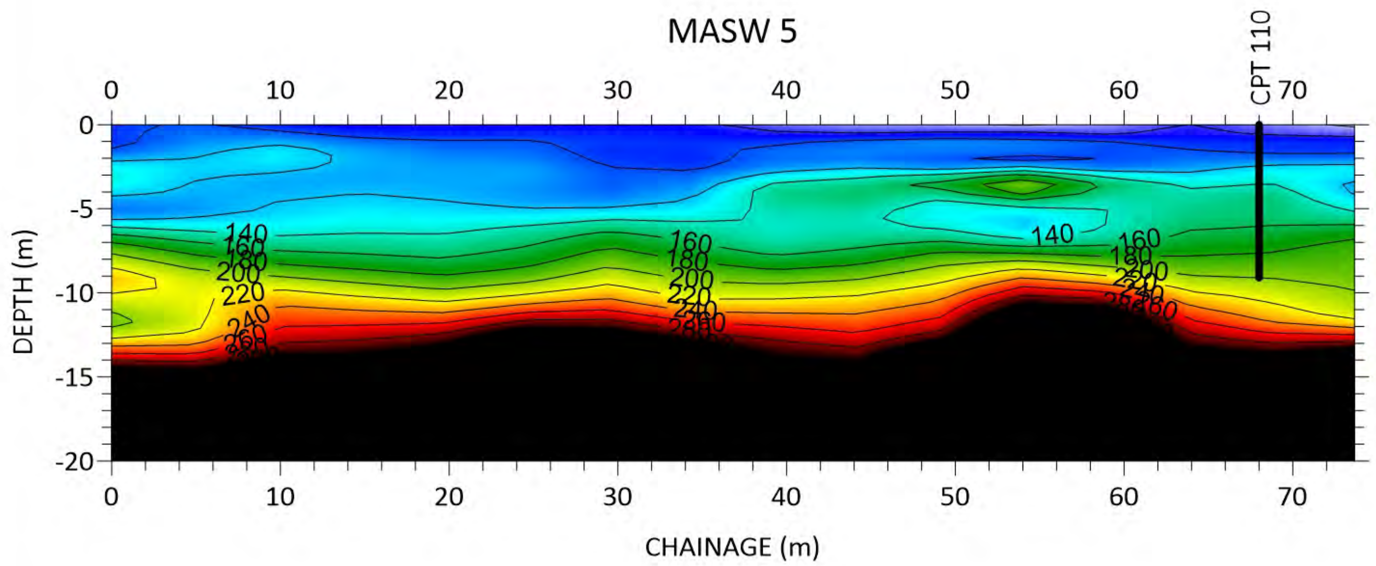
Southern Geophysical

www.southerngeophysical.com









DRAWING - **Figure 3: Block 1, MASW 2D V_s Profiles 3, 4, & 5**	NOTES - MASW V_s profile has contour intervals of 20 m/s (V_s). CPTs are drawn to refusal depths provided by Tonkin & Taylor. See site map for location of points.
LOCATION - **Beach Grove, Kaiapoi**	A3

Southern Geophysical
www.southerngeophysical.com

DRAWING - **Figure 4: Block 1 GPR Survey Site Map**

LOCATION - **Beach Grove, Kaiapoi**

NOTES -
Coordinates projected in NZTM2000.
Aerial photograph sourced from LINZ, Crown Copyright ©

CPT & Borehole
CPT locations labelled by cyan triangles
Borehole labelled by black borehole symbol

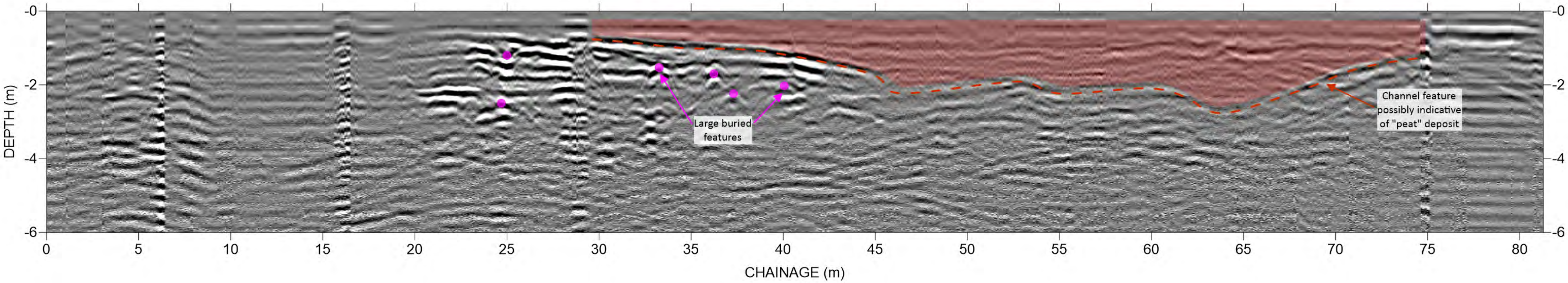
GPR
GPR lines marked by blue arrows, and are labelled at the start.

N

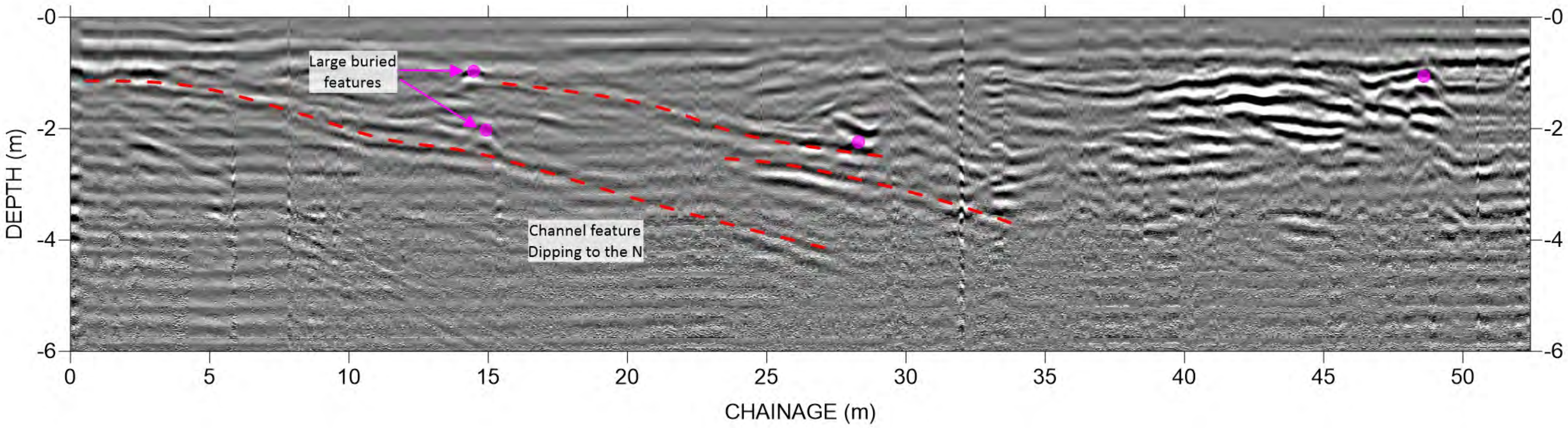
A3

Southern Geophysical
www.southerngeophysical.com

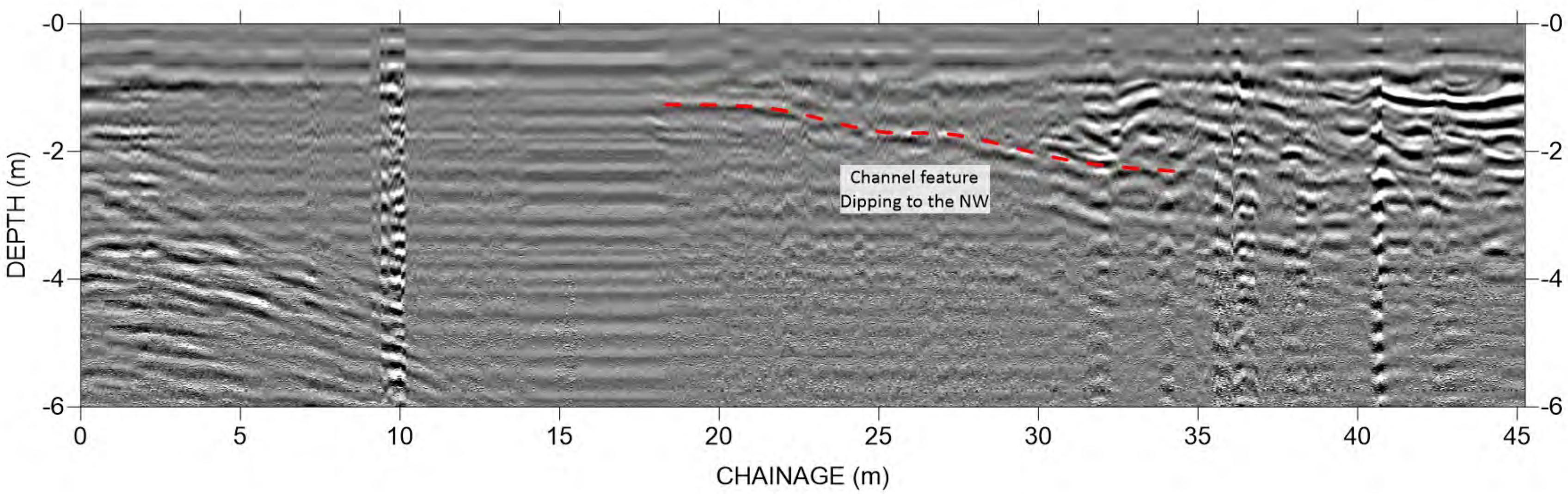
GPR-2



GPR-3



GPR-6



DRAWING - **Figure 5: Block 1 Example GPR Radargrams Lines 2, 3, and 6**

LOCATION - **Beach Grove, Kaiapoi**

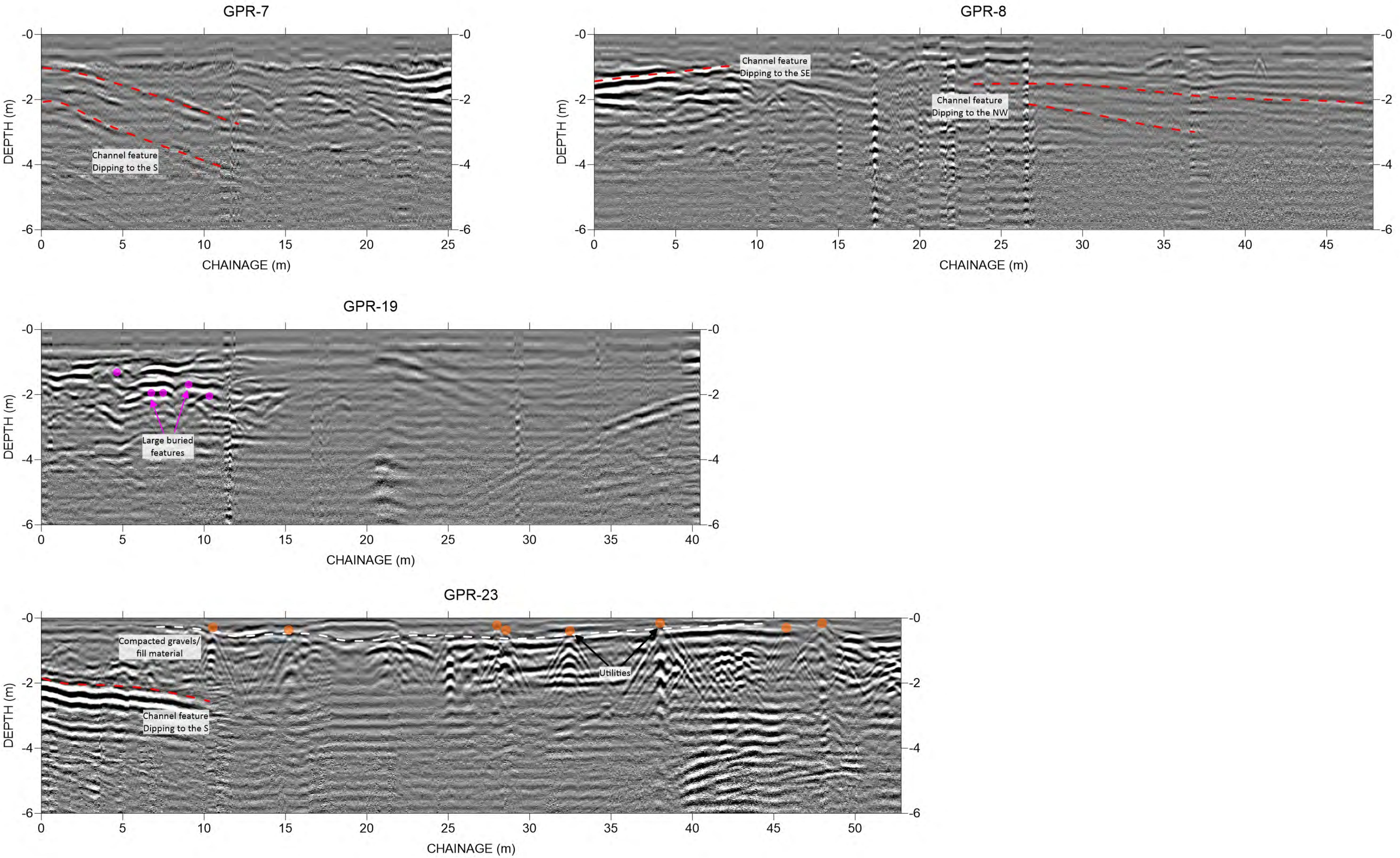
NOTES -

The image scales are:
GPR: 2:1

See site map for location of lines.

A3

Southern Geophysical
www.southerngeophysical.com



DRAWING - **Figure 6: Block 1 Example GPR Radargrams Lines 7, 8, 19, and 23**

LOCATION - **Beach Grove, Kaiapoi**

NOTES -

The image scales are:
GPR: 2:1

See site map for location of lines.

A3

Southern Geophysical
www.southerngeophysical.com

Appendix A: Field Photographs

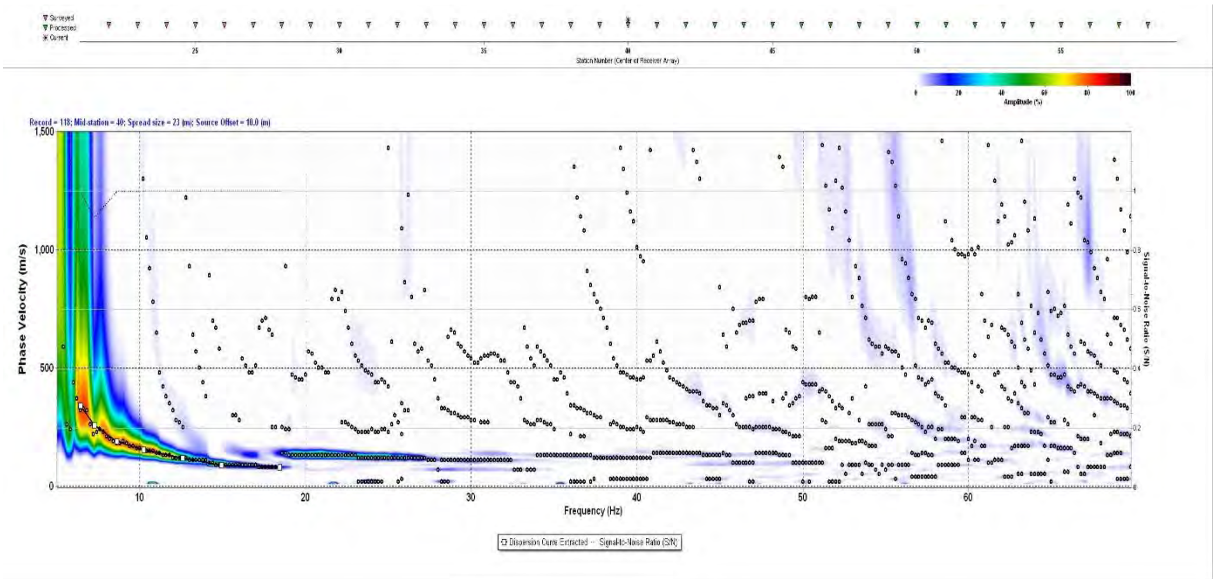
Geophone array and aluminium plate set up on Line 1.

MASW data acquisition on Line 2.

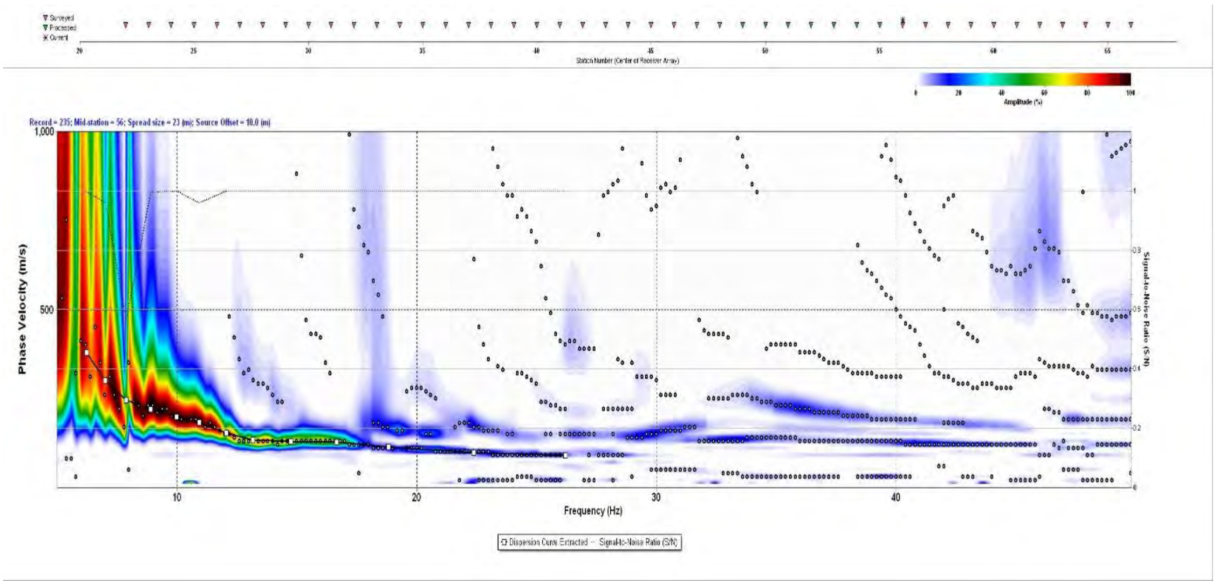
MASW survey along fence line for Line 3.

Geophone array and aluminium plate set up on Line 5.

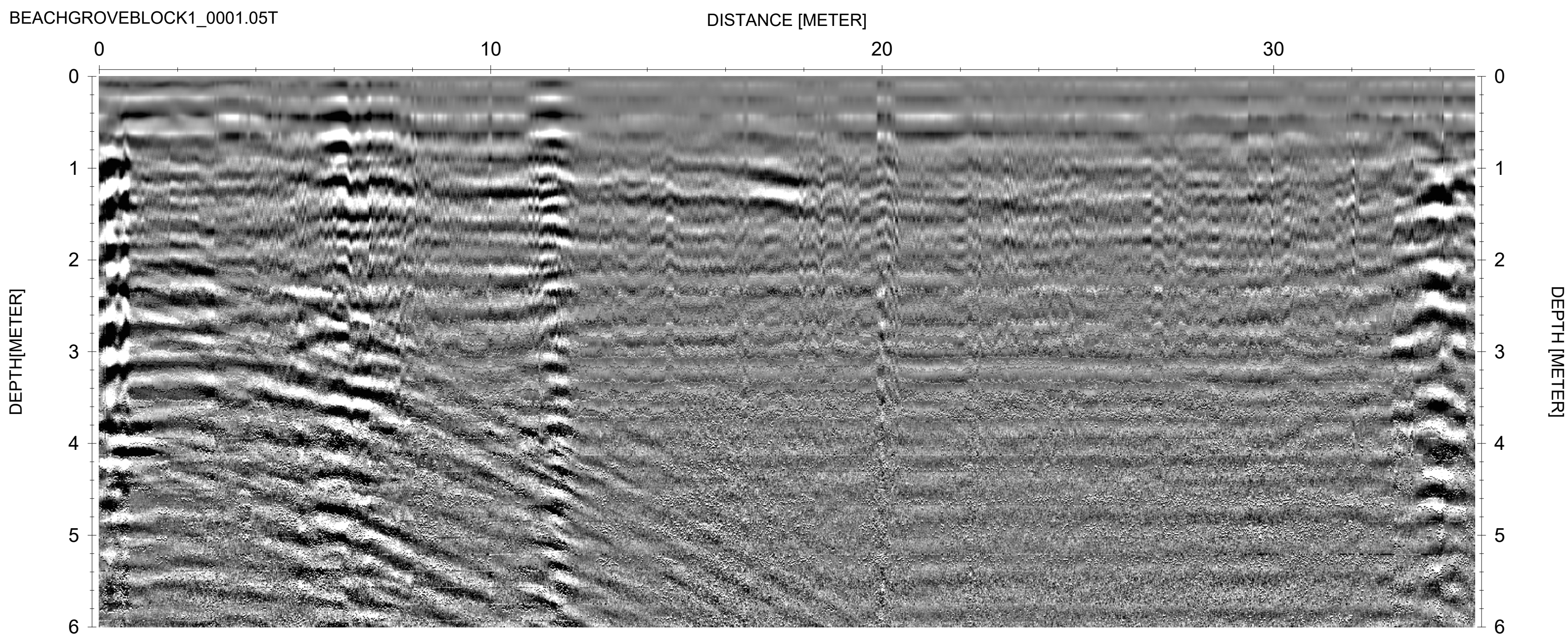
Appendix B: MASW Dispersion Curve Examples

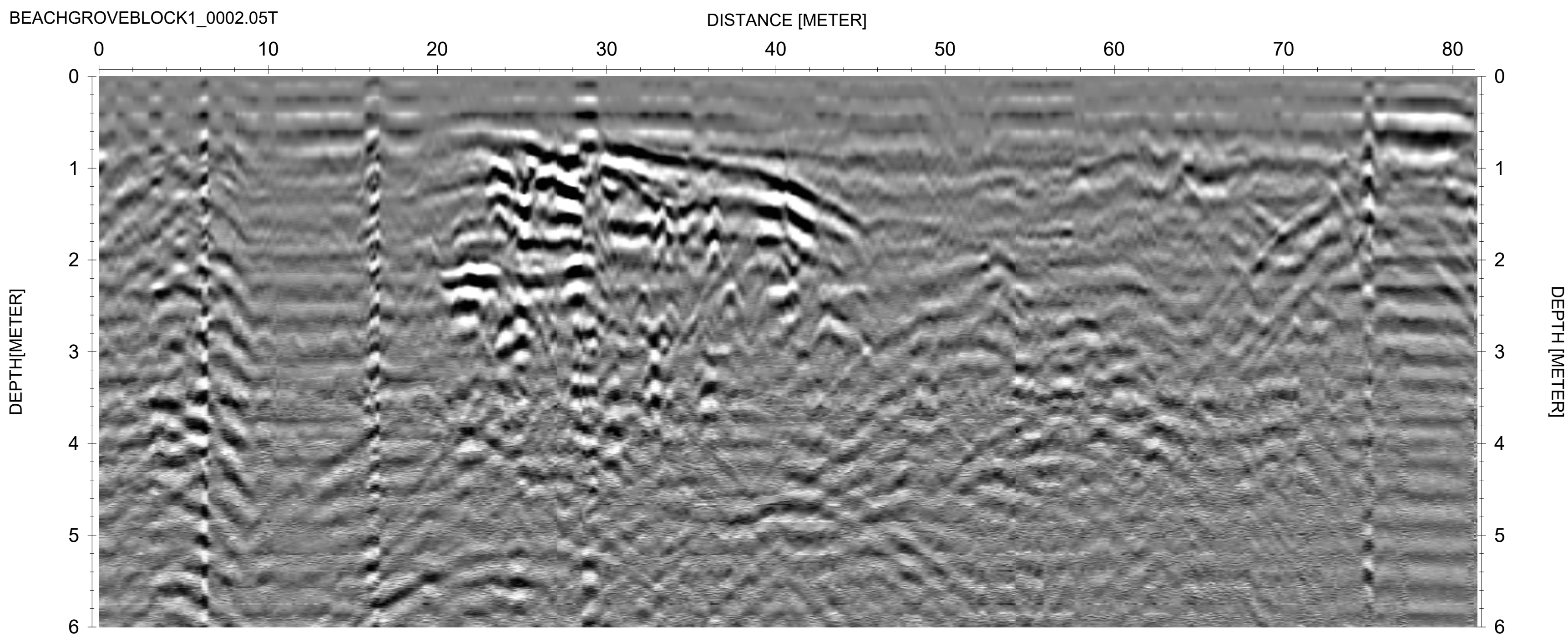


MASW dispersion curve pick from Line 1, chainage 106 m.

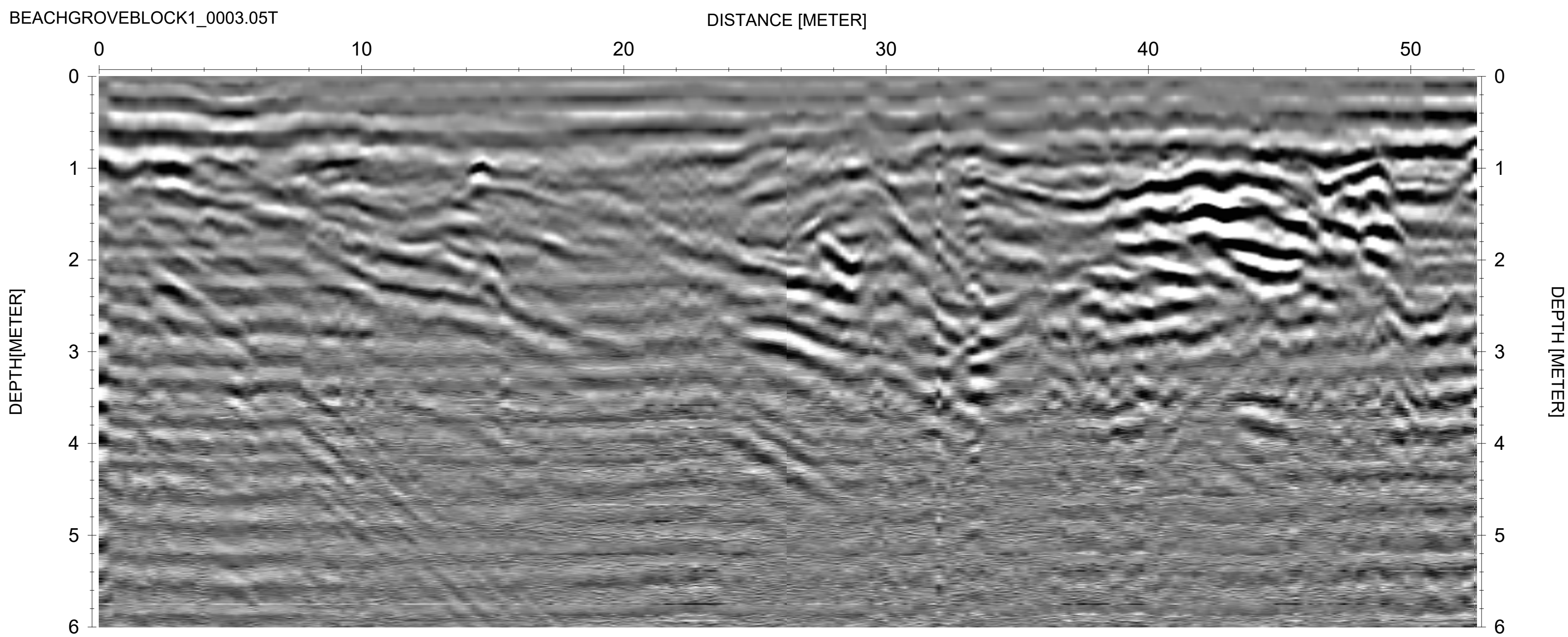


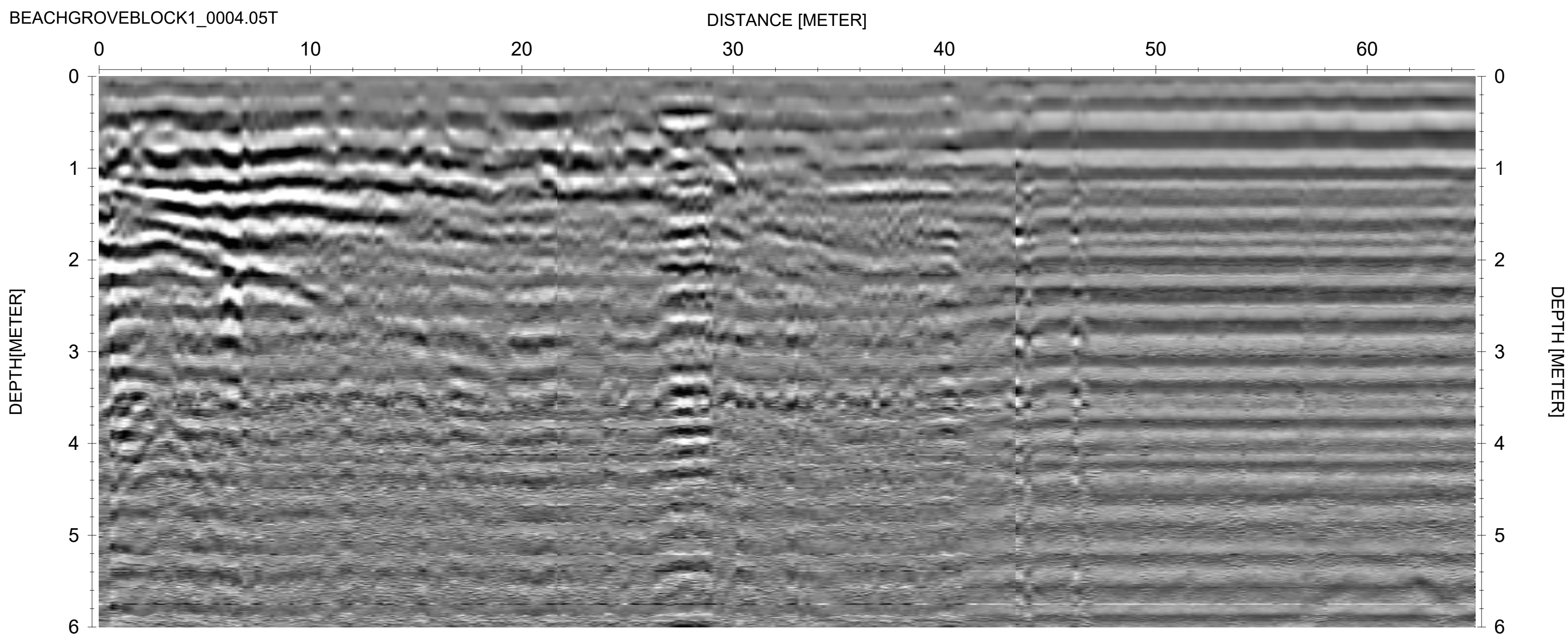
MASW dispersion curve pick from Line 2, chainage 61 m.

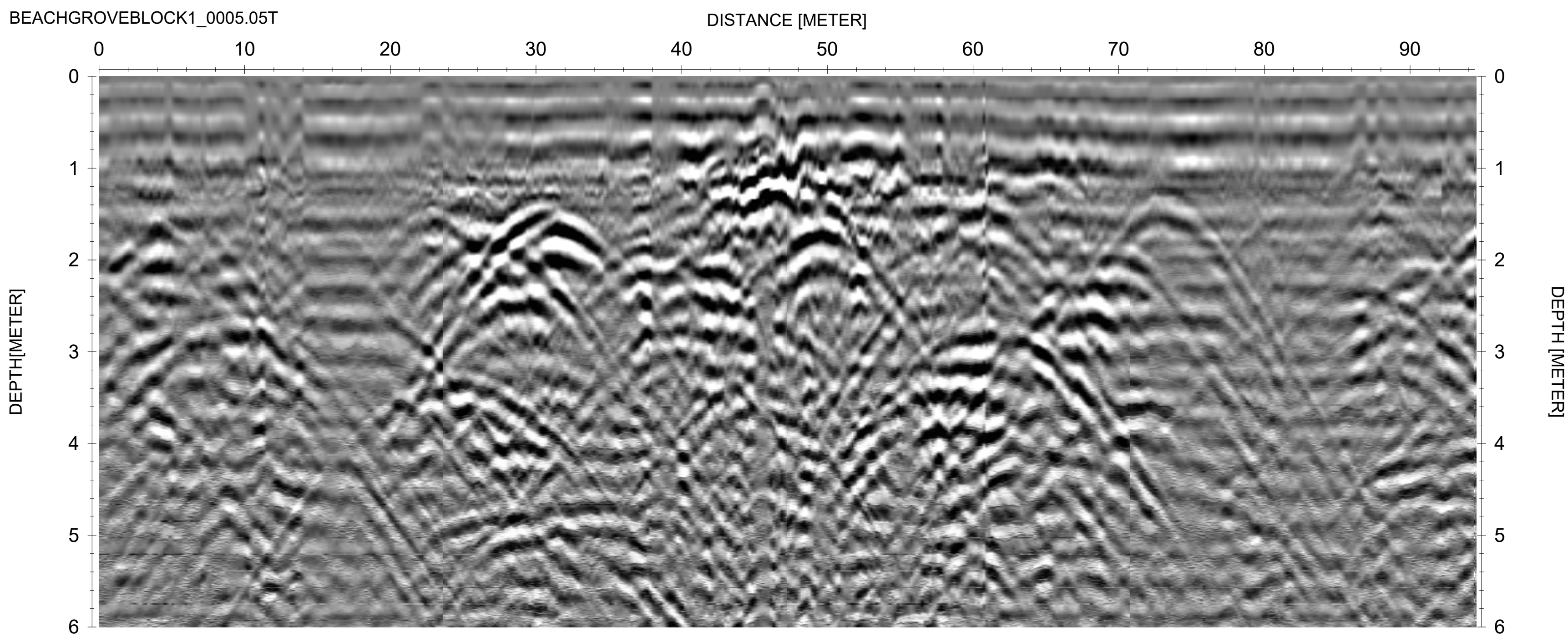

BEACHGROVEBLOCK1_0001.05T
DISTANCE [METER]
0
10
20
30
DEPTH[METER]
0
1
2
3
4
5
6
DEPTH [METER]

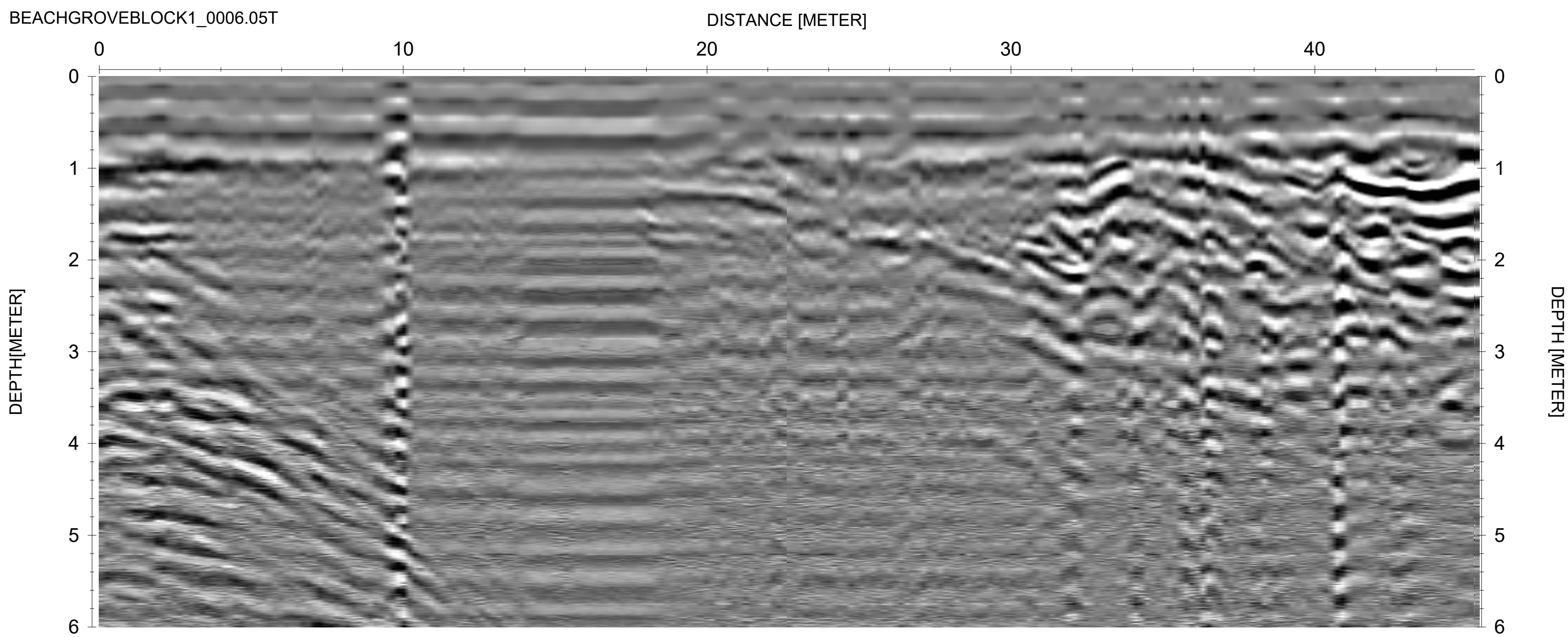


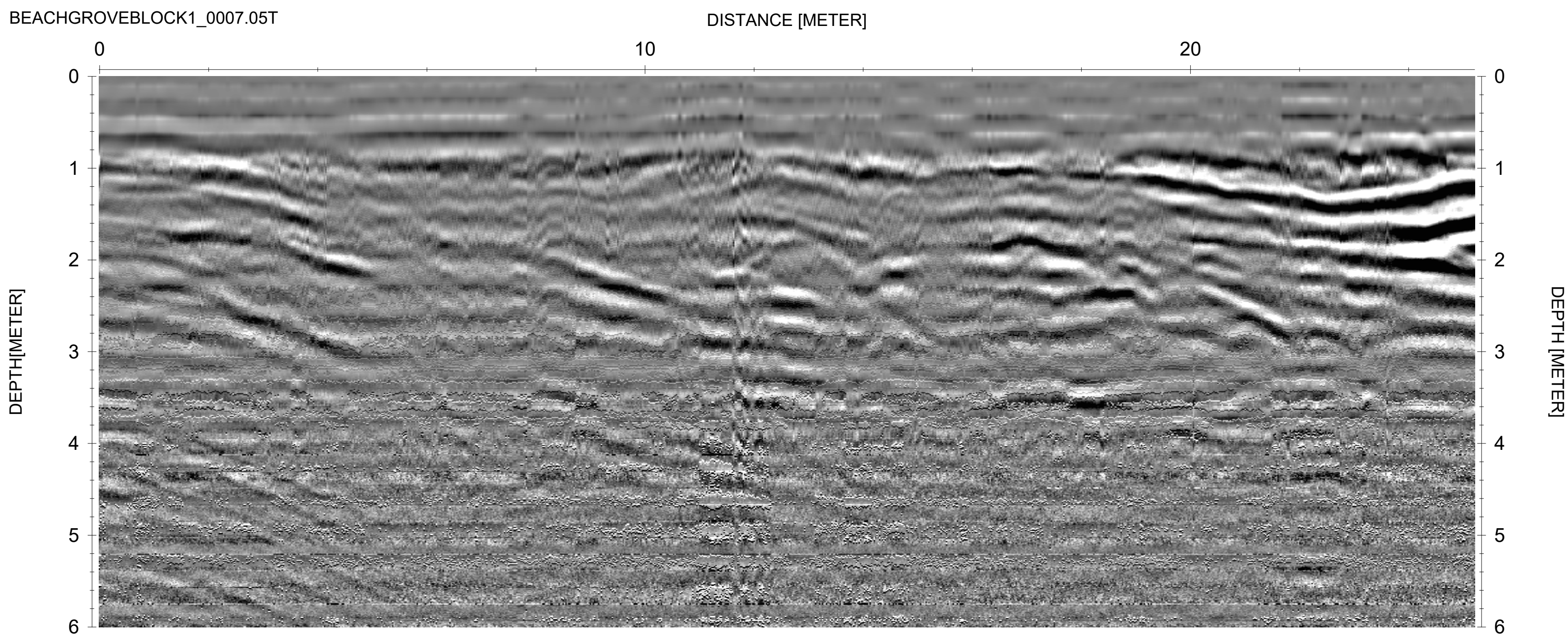
BEACHGROVEBLOCK1_0002.05T
DISTANCE [METER]
0
10
20
30
40
50
60
70
80
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6

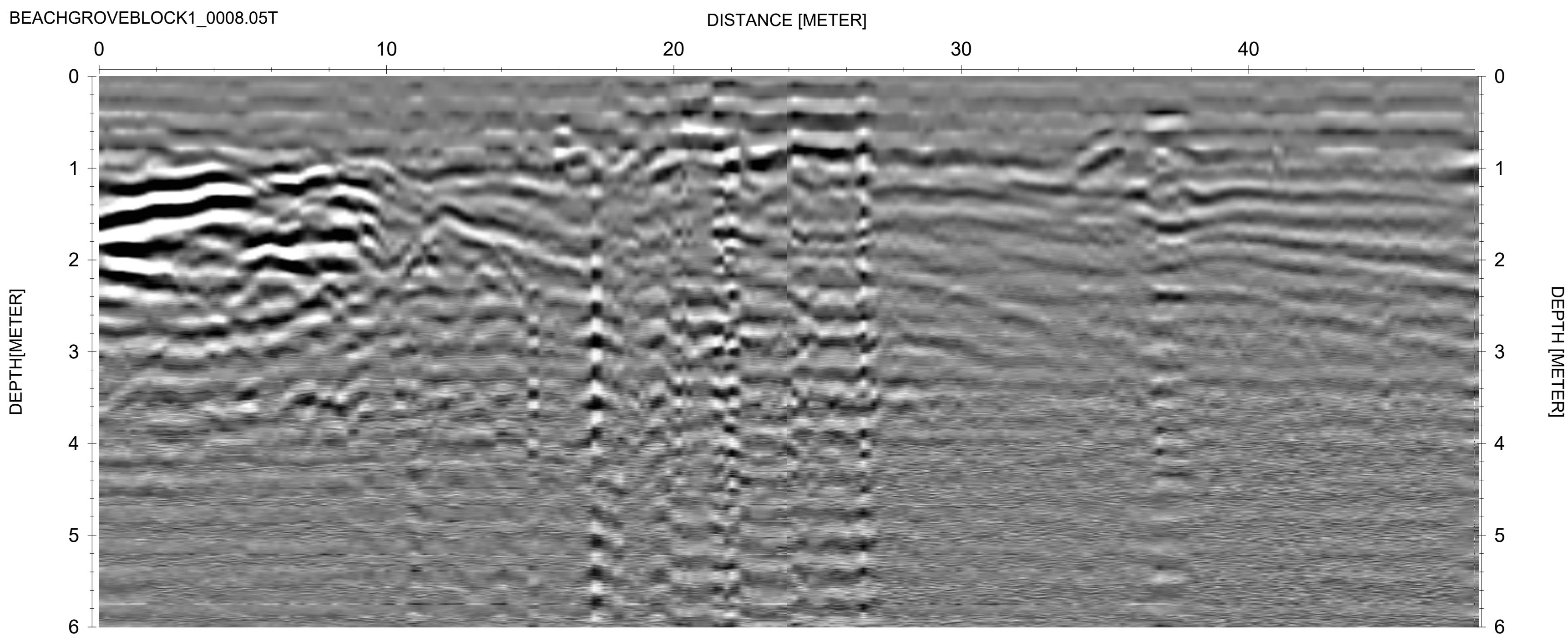

BEACHGROVEBLOCK1_0003.05T
DISTANCE [METER]
0
10
20
30
40
50
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6

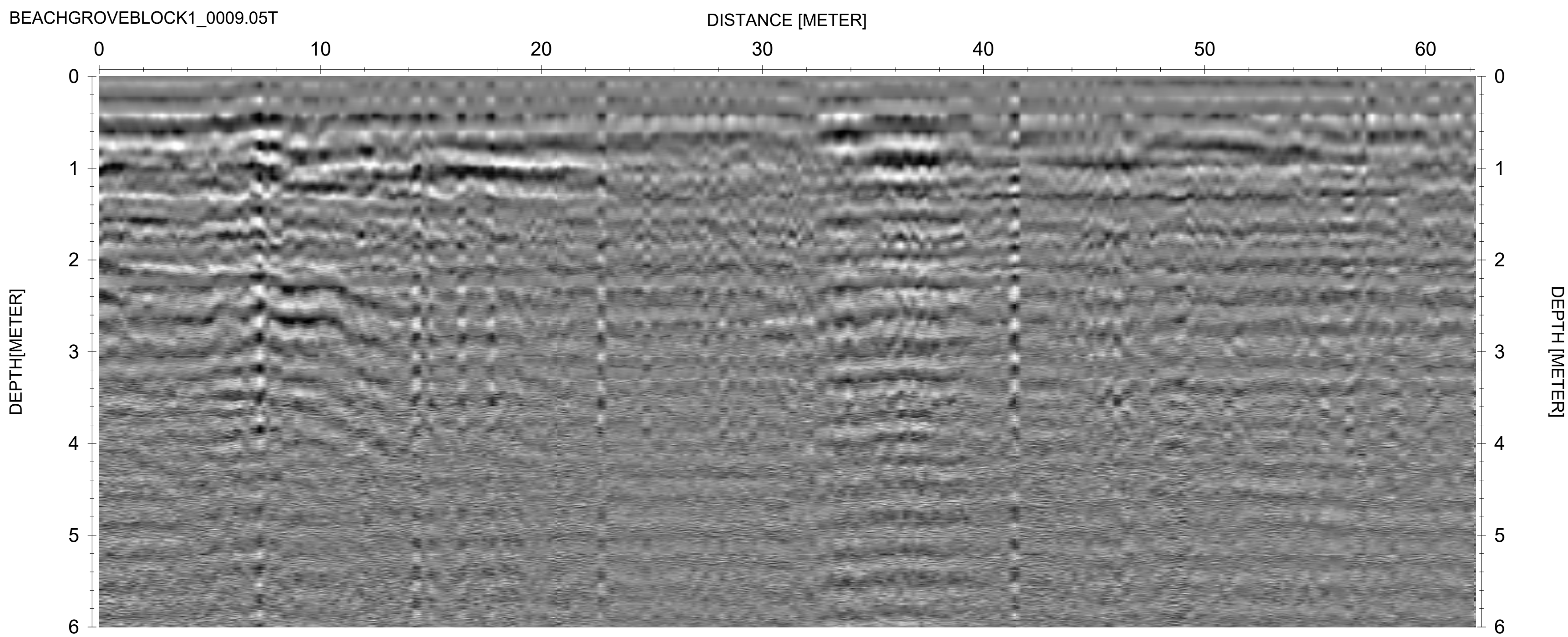

BEACHGROVEBLOCK1_0004.05T
DISTANCE [METER]
0
10
20
30
40
50
60
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6

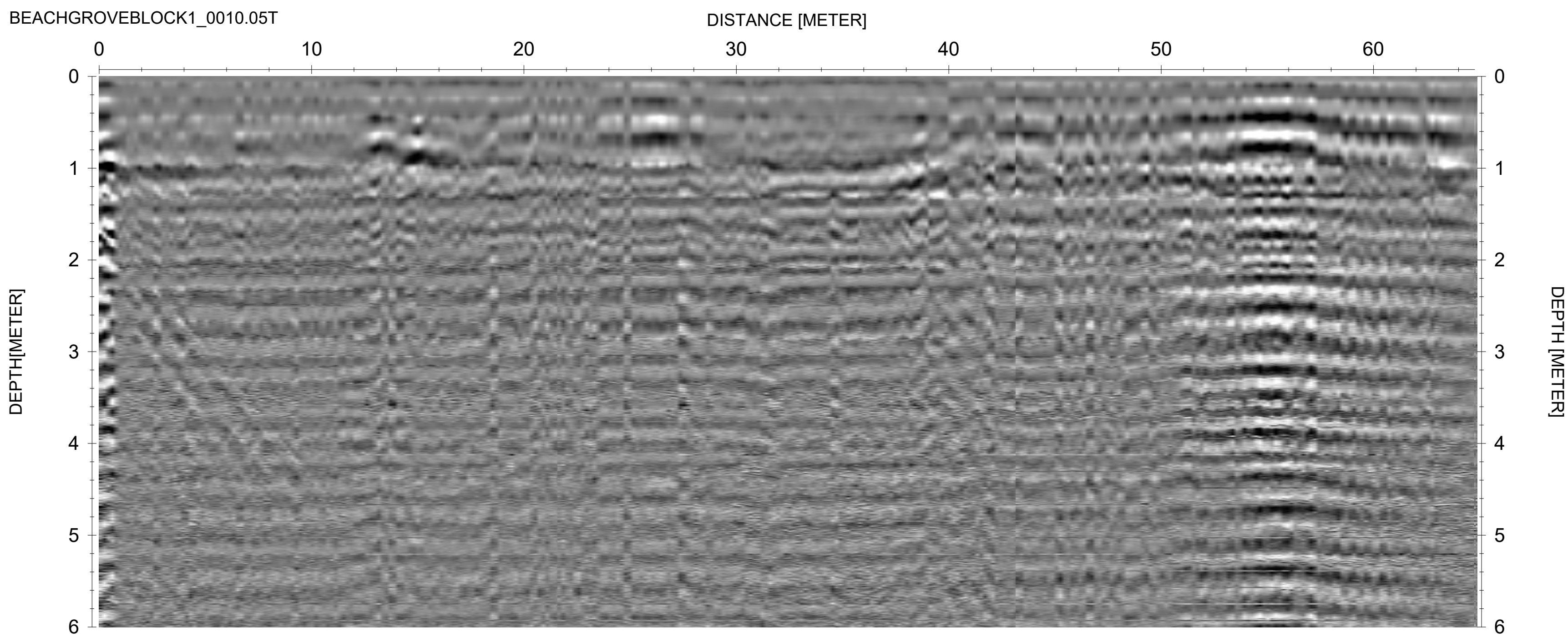

BEACHGROVEBLOCK1_0005.05T
DISTANCE [METER]
0 10 20 30 40 50 60 70 80 90
DEPTH[METER]
0 1 2 3 4 5 6
DEPTH [METER]
0 1 2 3 4 5 6


BEACHGROVEBLOCK1_0006.05T
DISTANCE [METER]
0
10
20
30
40
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6

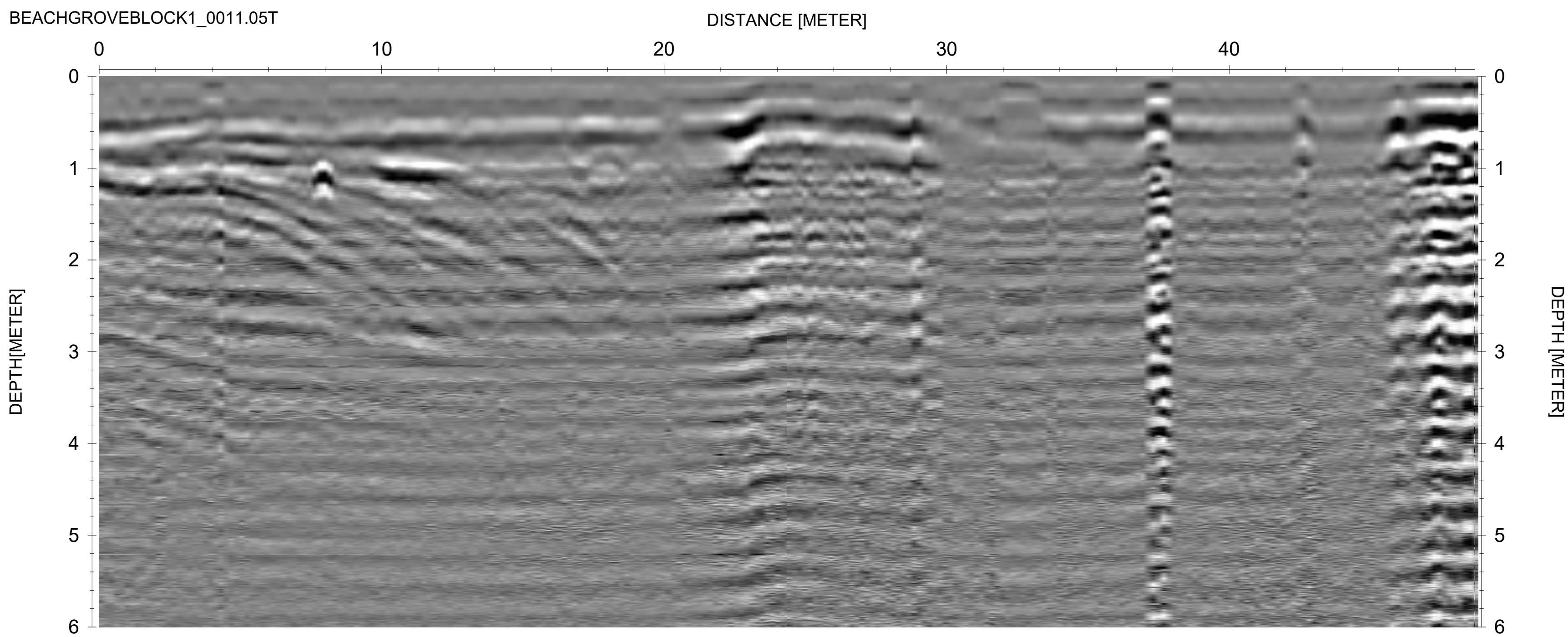

BEACHGROVEBLOCK1_0007.05T
DISTANCE [METER]
0
10
20
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6

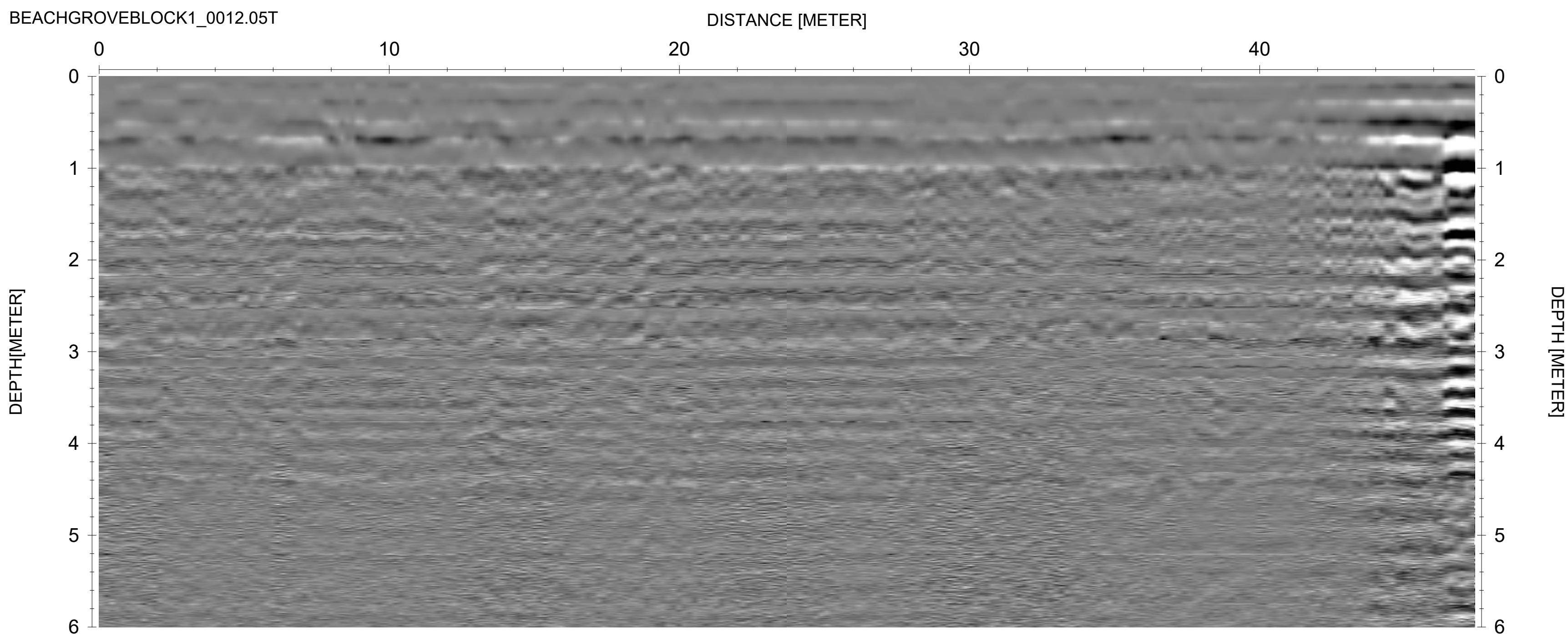

BEACHGROVEBLOCK1_0008.05T
DISTANCE [METER]
0
10
20
30
40
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6

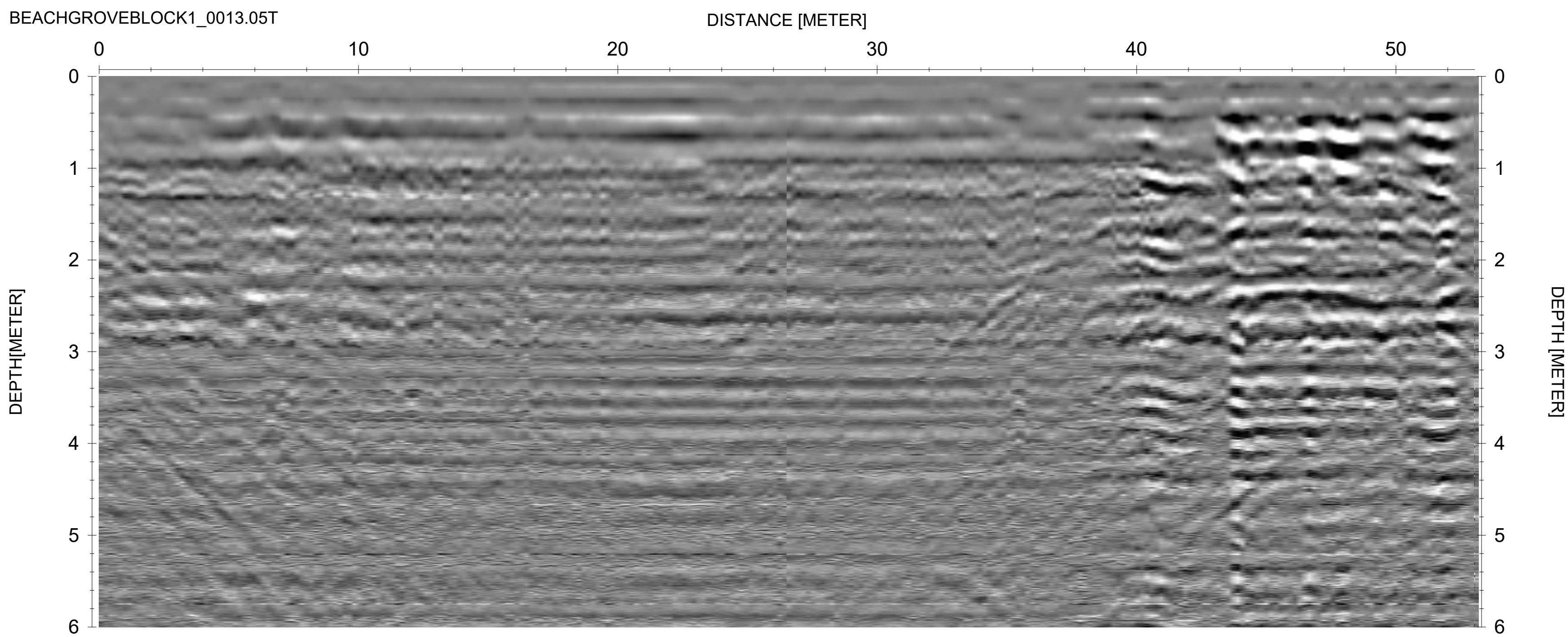

BEACHGROVEBLOCK1_0009.05T
DISTANCE [METER]
0
10
20
30
40
50
60
DEPTH[METER]
0
1
2
3
4
5
6
DEPTH [METER]



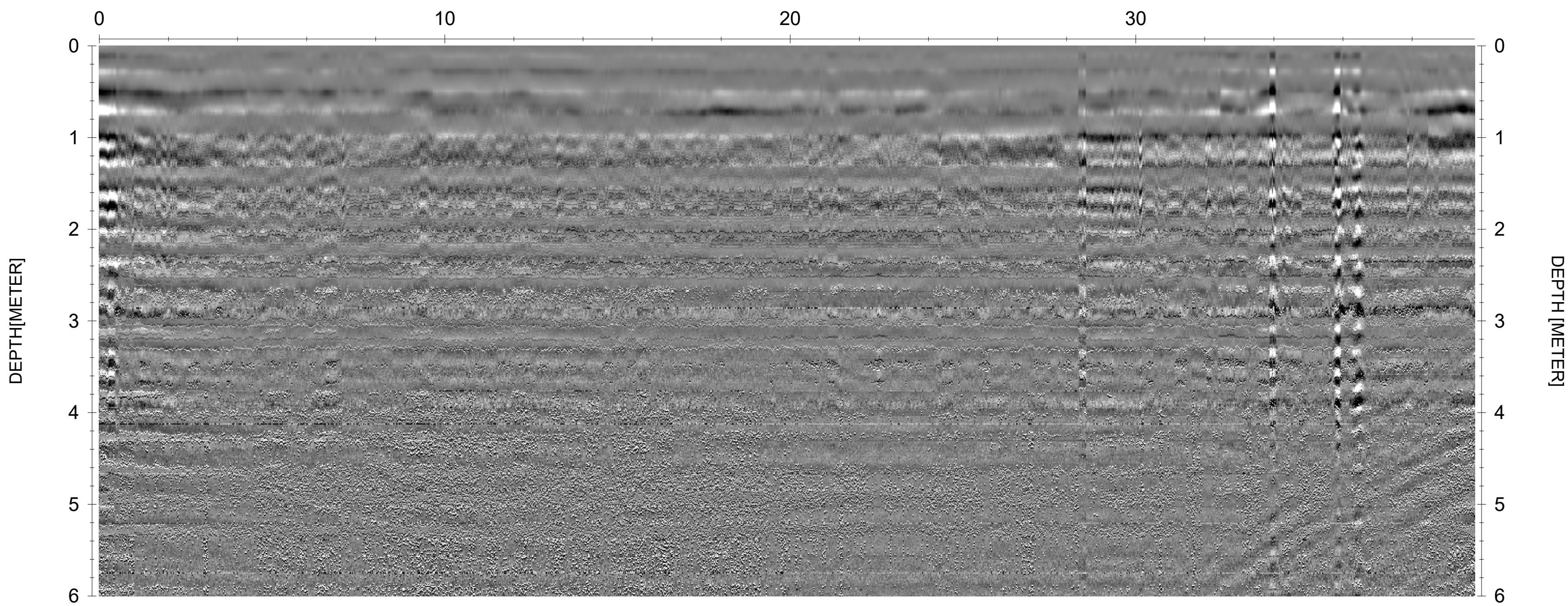
BEACHGROVEBLOCK1_0010.05T
DISTANCE [METER]
0
10
20
30
40
50
60
0
1
2
3
4
5
6
DEPTH[METER]
DEPTH [METER]

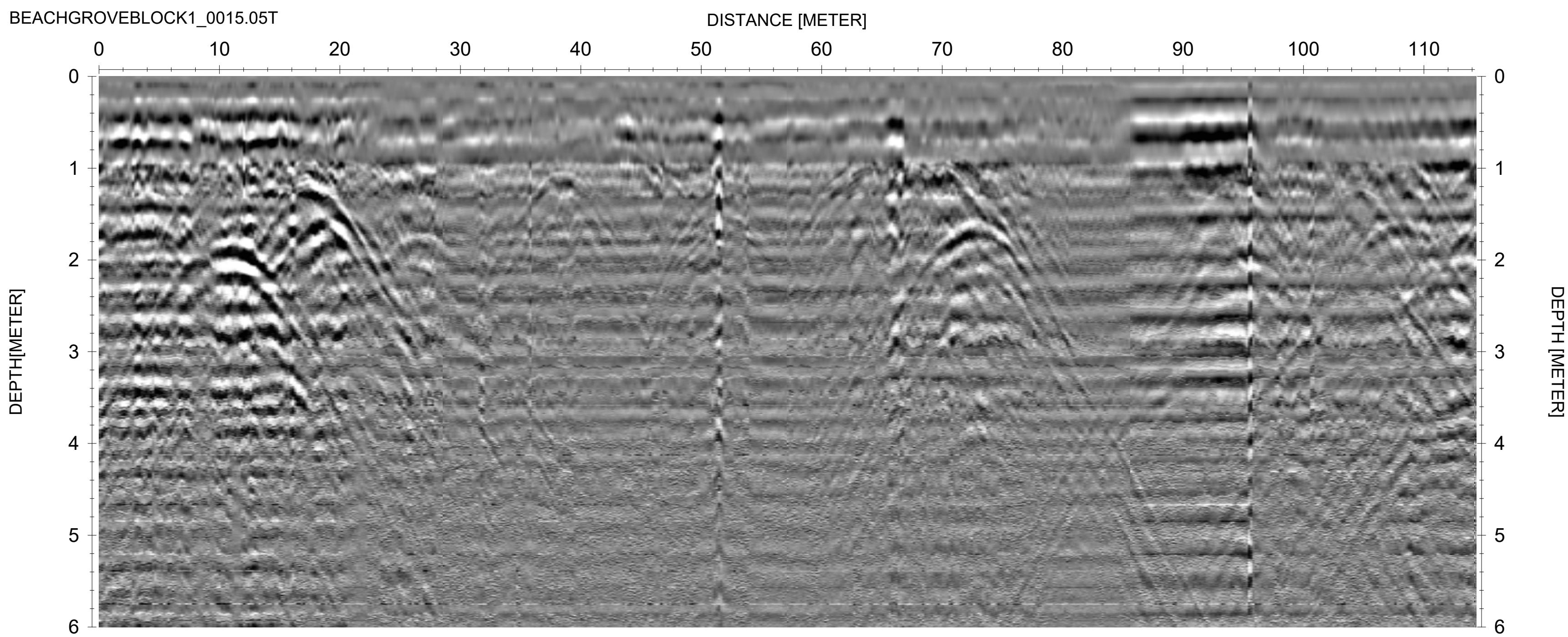

BEACHGROVEBLOCK1_0011.05T
DISTANCE [METER]
0
10
20
30
40
0
1
2
3
4
5
6
DEPTH[METER]
DEPTH [METER]

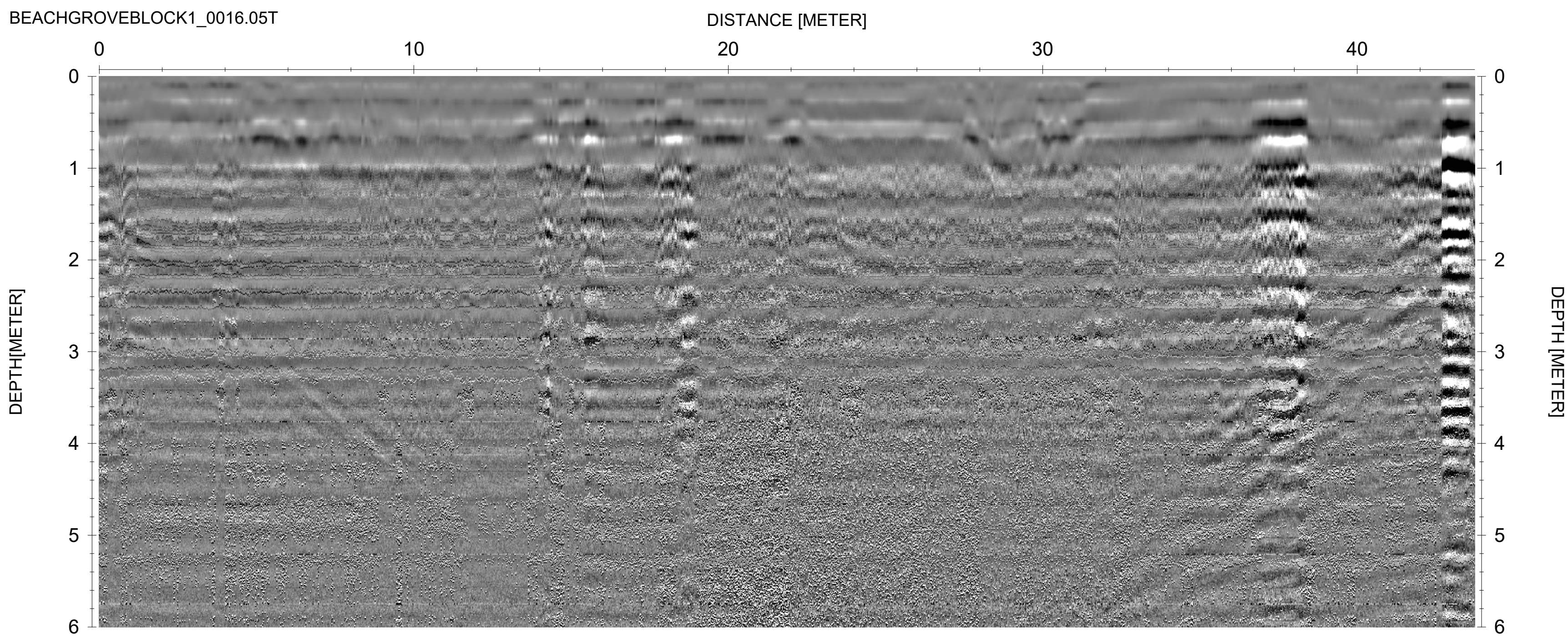

BEACHGROVEBLOCK1_0012.05T
DISTANCE [METER]
0
10
20
30
40
DEPTH[METER]
0
1
2
3
4
5
6
DEPTH [METER]

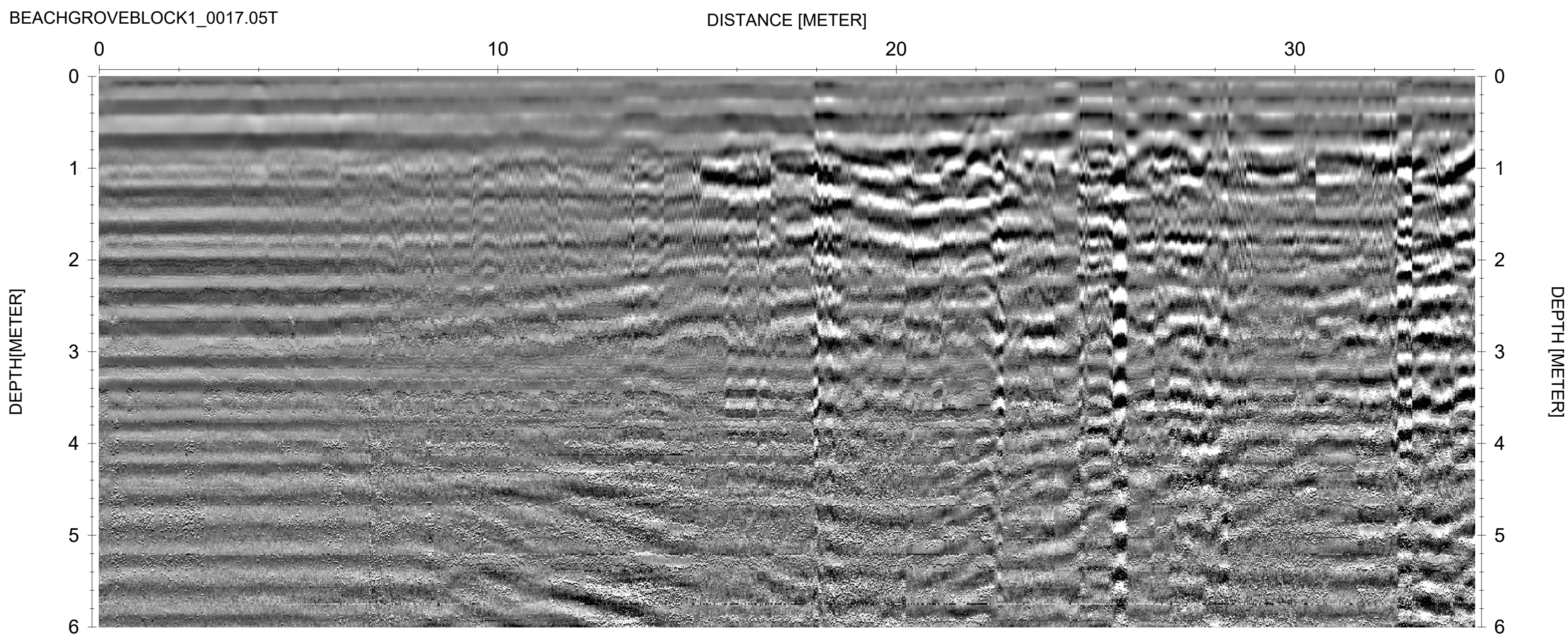

BEACHGROVEBLOCK1_0013.05T
DISTANCE [METER]
0
10
20
30
40
50
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6

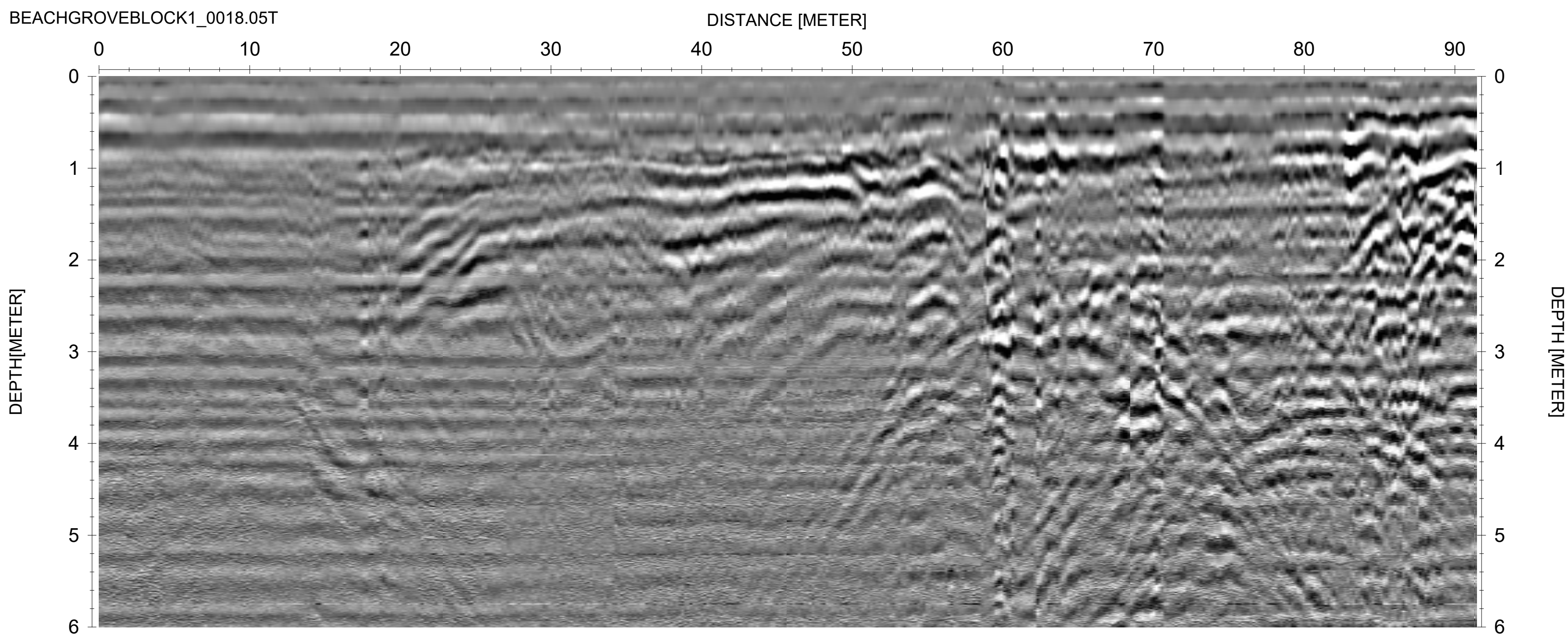
BEACHGROVEBLOCK1_0014.05T



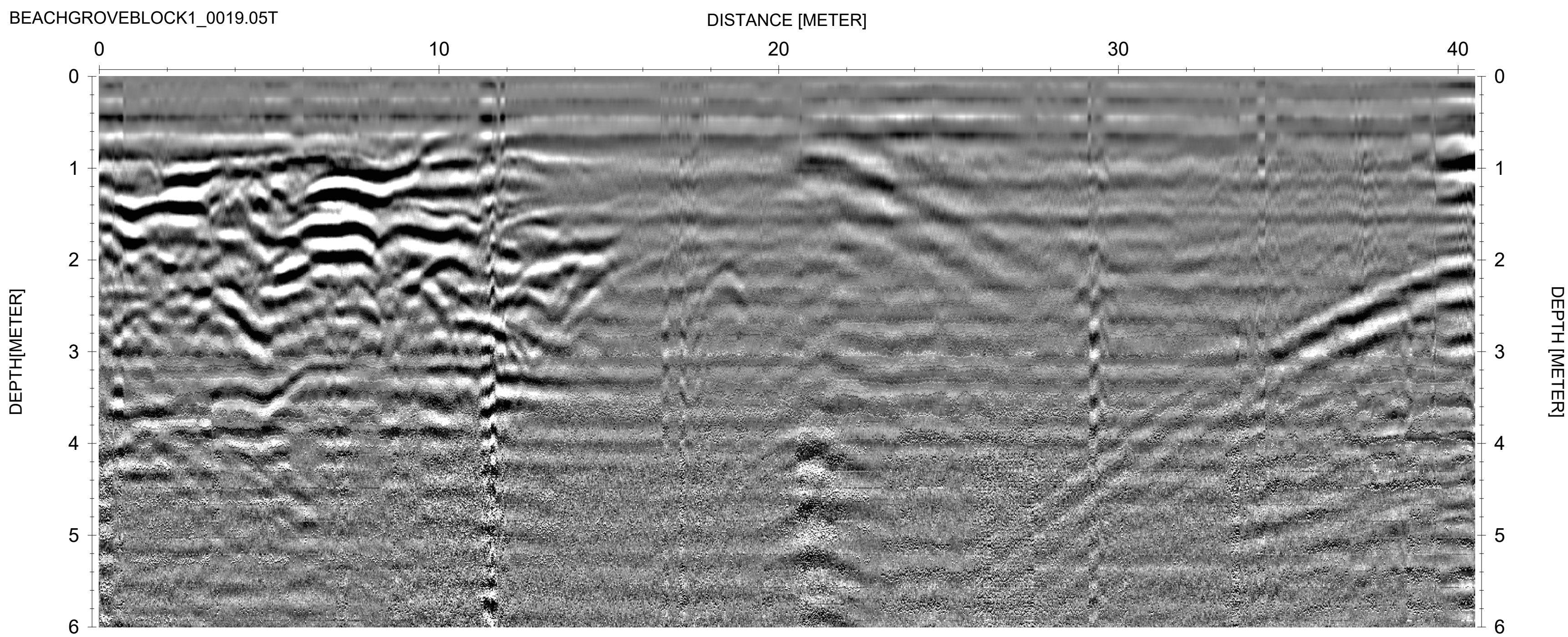

BEACHGROVEBLOCK1_0015.05T
DISTANCE [METER]
0
10
20
30
40
50
60
70
80
90
100
110
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6

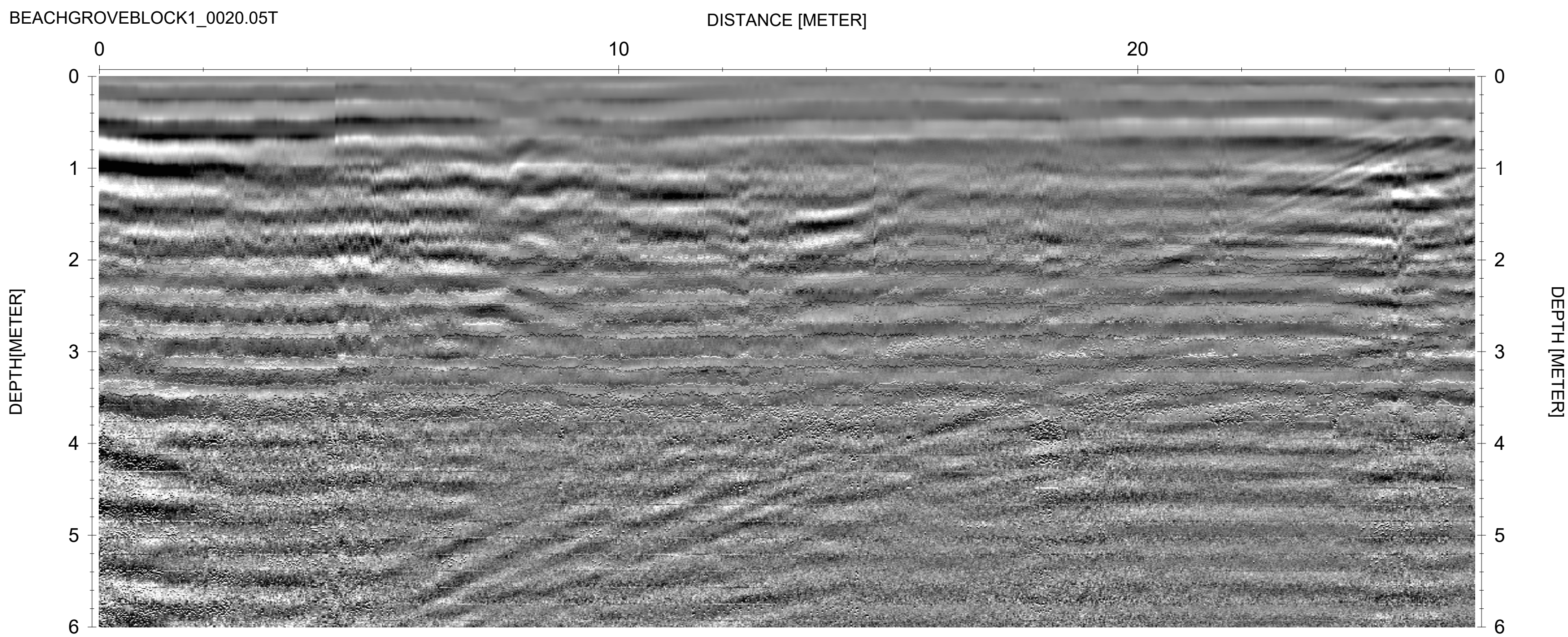

BEACHGROVEBLOCK1_0016.05T
DISTANCE [METER]
0
10
20
30
40
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6


BEACHGROVEBLOCK1_0017.05T
DISTANCE [METER]
0
10
20
30
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6

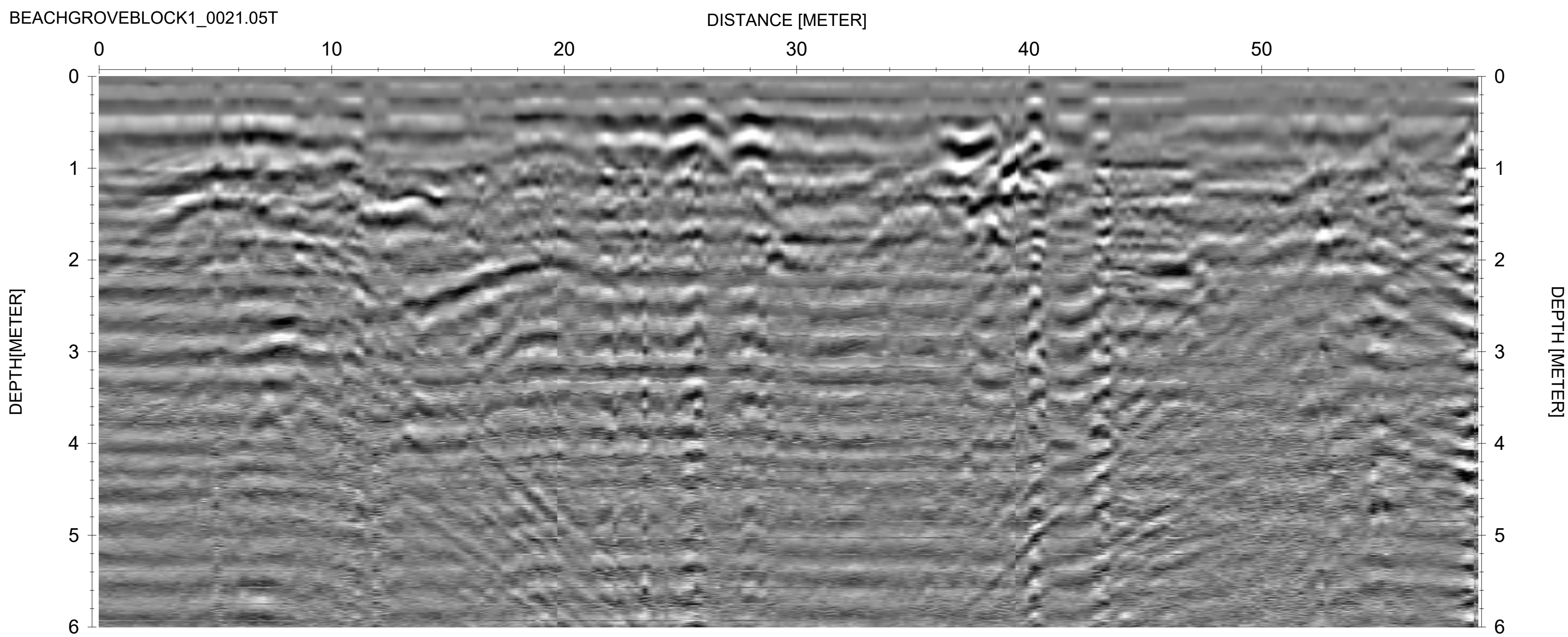


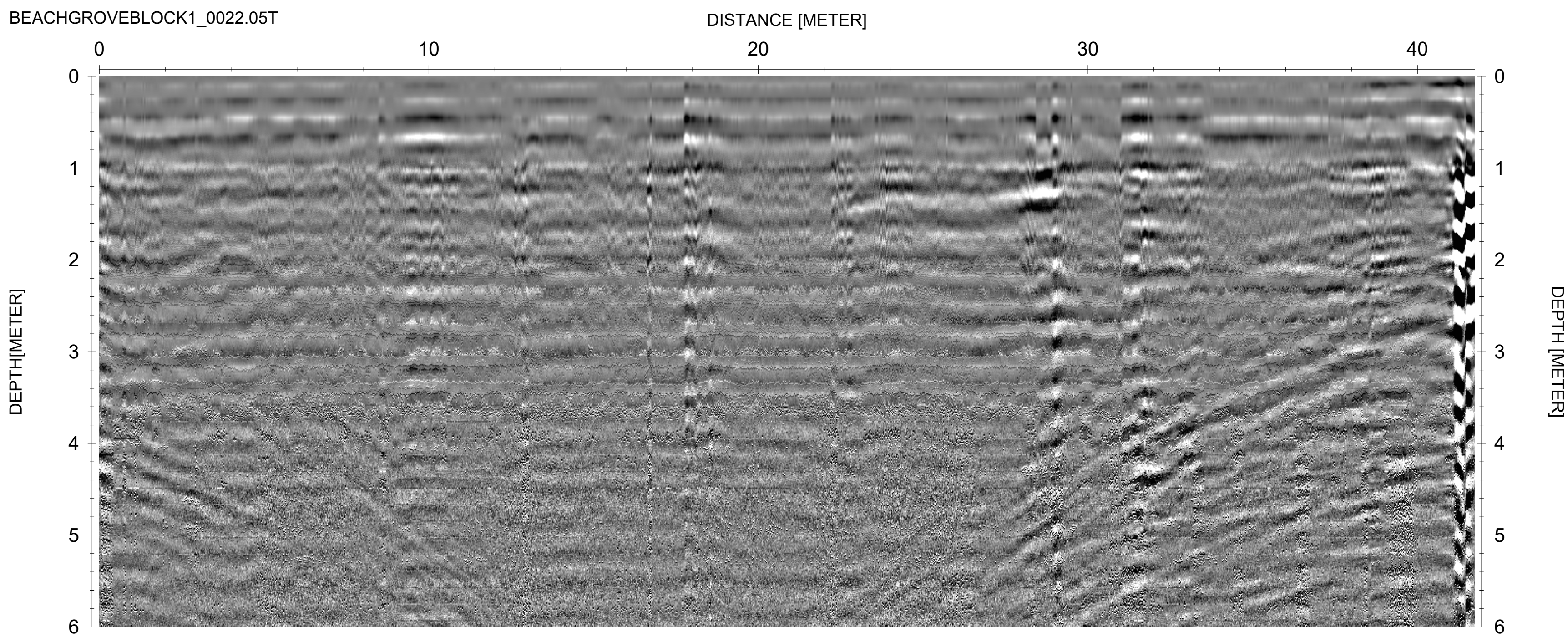
BEACHGROVEBLOCK1_0018.05T
DISTANCE [METER]
0
10
20
30
40
50
60
70
80
90
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6

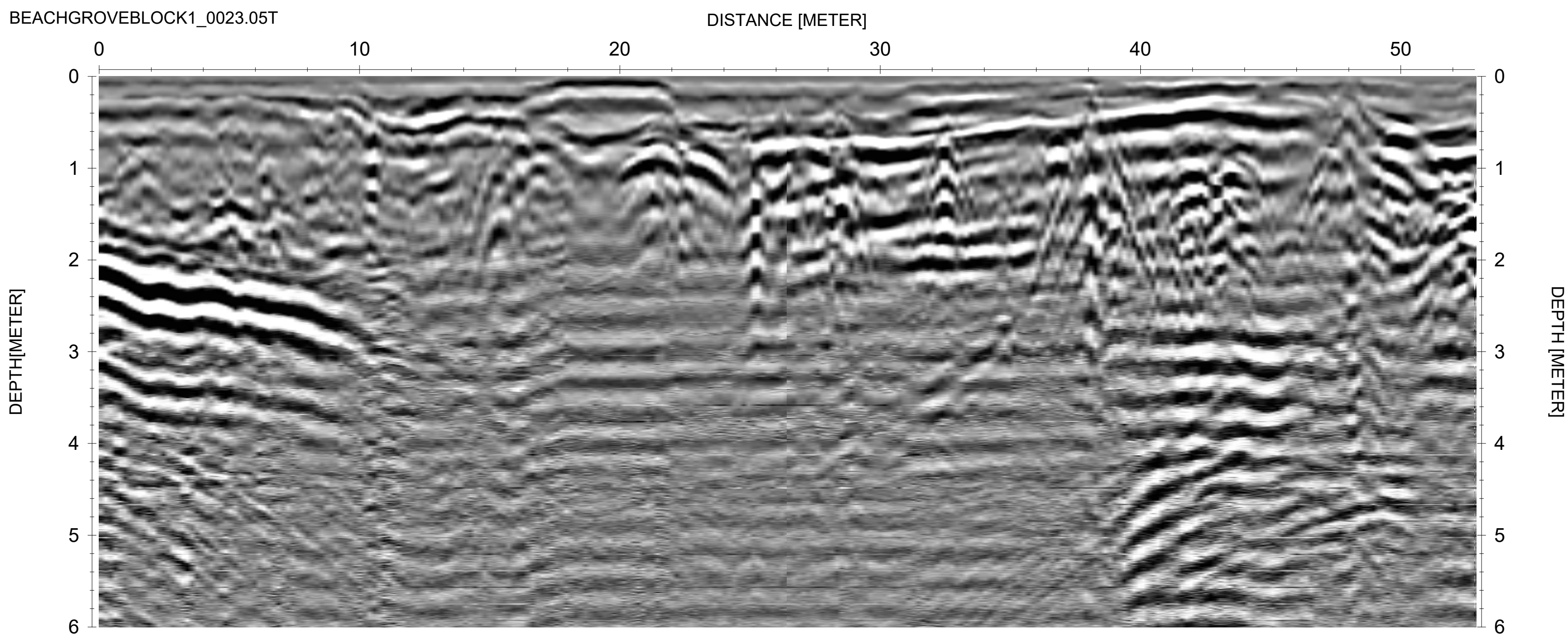

BEACHGROVEBLOCK1_0019.05T
DISTANCE [METER]
0
10
20
30
40
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6

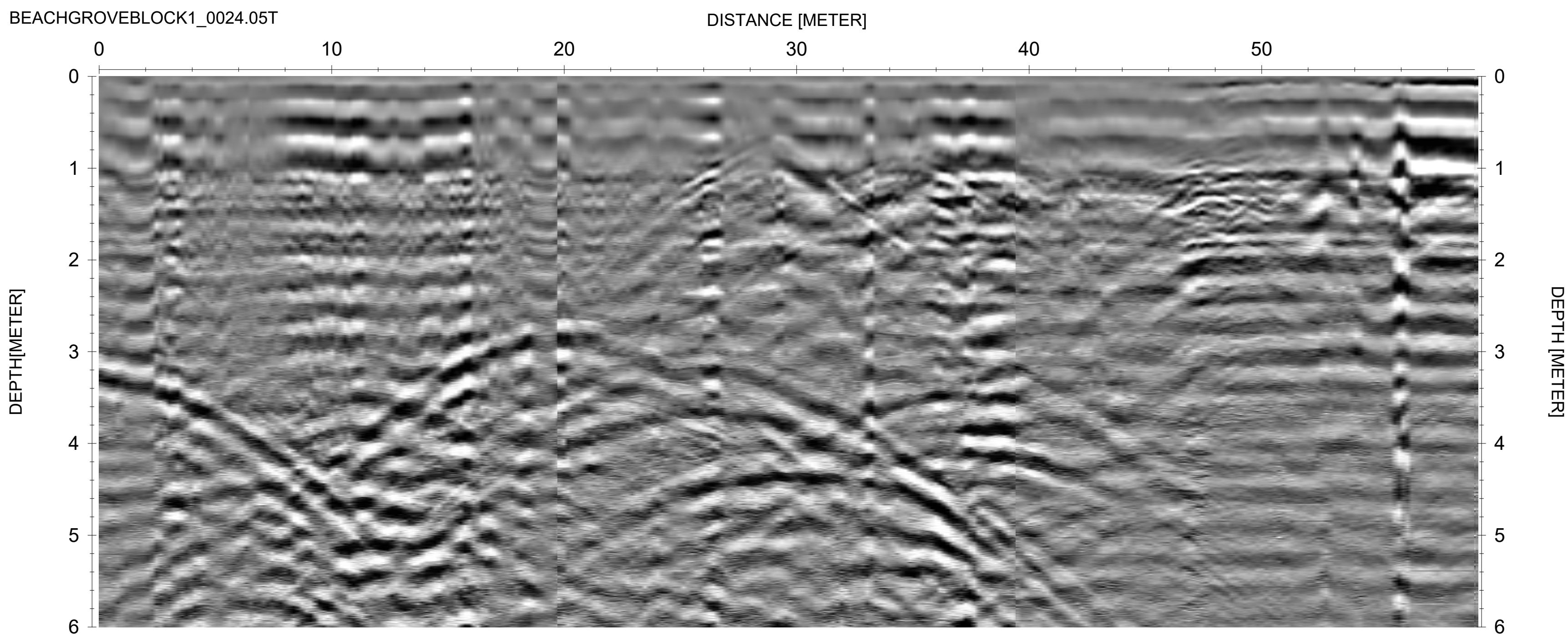


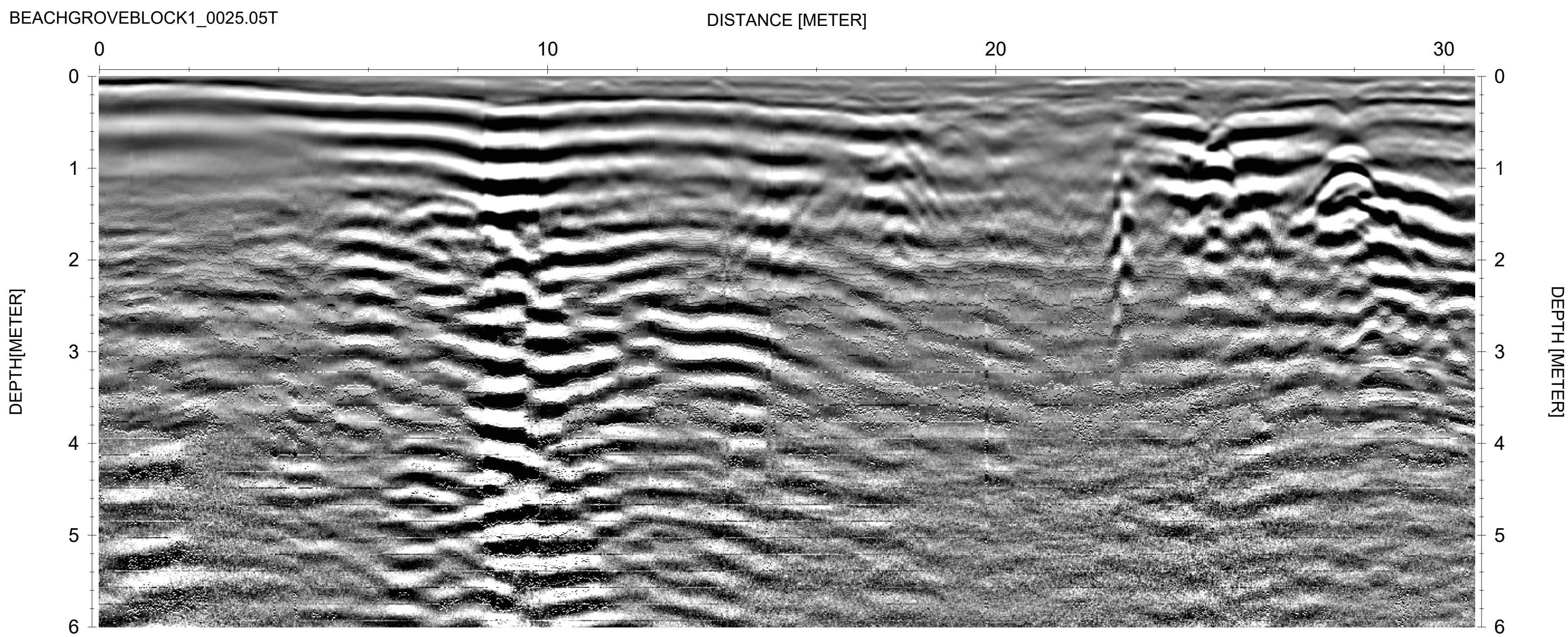
BEACHGROVEBLOCK1_0020.05T
DISTANCE [METER]
0
10
20
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6


BEACHGROVEBLOCK1_0021.05T
DISTANCE [METER]
0
10
20
30
40
50
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6


BEACHGROVEBLOCK1_0022.05T
DISTANCE [METER]
0
10
20
30
40
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6

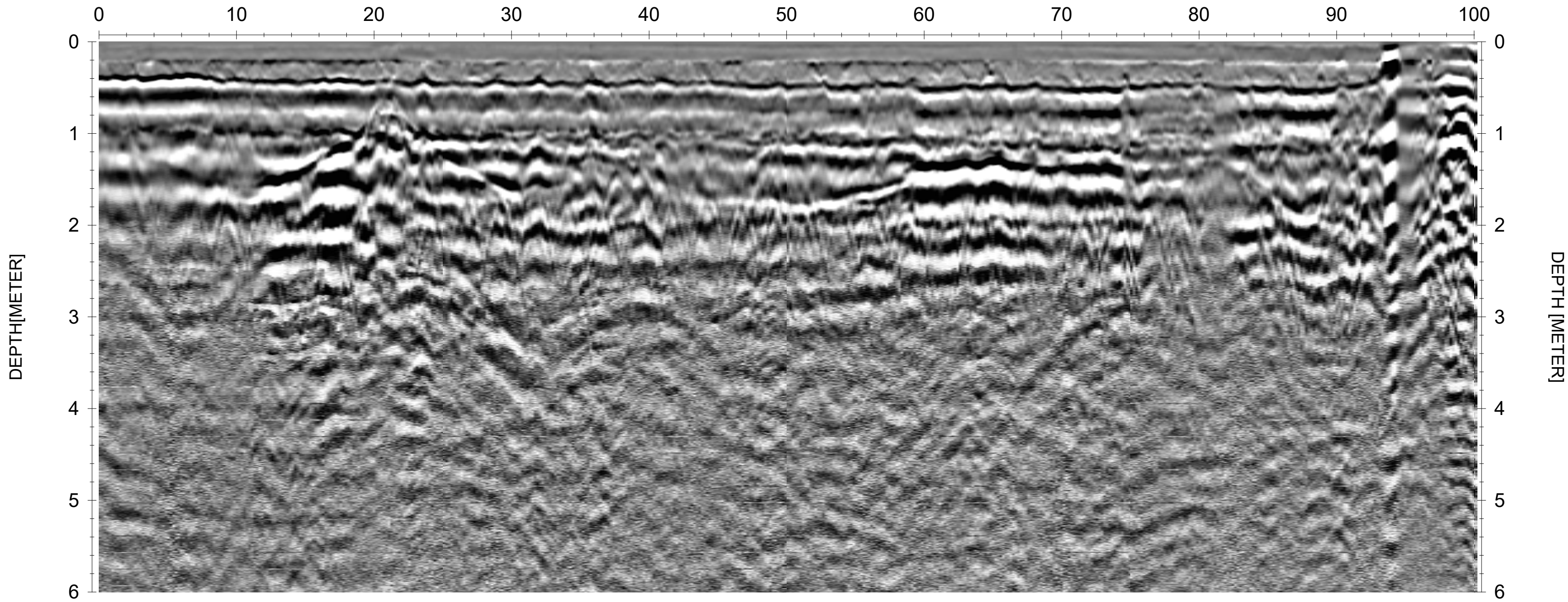

BEACHGROVEBLOCK1_0023.05T
DISTANCE [METER]
0
10
20
30
40
50
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6


BEACHGROVEBLOCK1_0024.05T
DISTANCE [METER]
0
10
20
30
40
50
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6


BEACHGROVEBLOCK1_0025.05T
DISTANCE [METER]
0
10
20
30
0
1
2
3
4
5
6
DEPTH[METER]
DEPTH [METER]

BEACHGROVEBLOCK1_0026.05T

DISTANCE [METER]



Appendix D Laboratory results

GEOTECHNICS

Our Ref: 1100960.0016.0.0/Rep1
Customer Ref: 1019317.0
30 May 2022

Tonkin + Taylor Ltd
60 Cashel Street,
Christchurch

Attention: Peter Lee

Dear Peter

Momentum Land, Beach Road

Laboratory Test Report

Samples from the above mentioned site have been tested as received according to your instructions and the results are included in this report. Results apply only to the sample(s) tested.

Descriptions are enclosed for your information, but are not covered under the IANZ endorsement of this report.

This report has been prepared for the benefit of Tonkin + Taylor Ltd, with respect to the particular brief given to us and it cannot be relied upon in other contexts or for any other purpose without our prior review and agreement.

This report may be reproduced only in full.

Samples not destroyed during testing will be retained for one month from the date of this report before being discarded. If we can be of any further assistance, feel free to get in touch. Contact details are provided at the bottom of this page.

GEOTECHNICS LTD

Report prepared by:

....
Jack Singh
Laboratory Technician
Approved Signatory

Authorised for Geotechnics by:

....
Vic 'O'Connor
Project Director

ACCREDITED IANZ TESTING LABORATORY

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

Report checked by:

....

Corey Papu Gread
Christchurch Manager

30-May-22
\\ttgroup.local\corporate\geotechnicsgroup\projects\1100960\1100960.0016 - momentun land\workingmaterial\20220530.jasi.1100960.0016.0.0.rep1.docx

45A Parkhouse Road
Wigram
Christchurch 8042
New Zealand
p +64 3 361 0300

Geotechnics Project Number	1100960.0016.0.0
QESTLab Work Order ID	W22CH-0109
Customer Project ID	1019317.0

Determination of Liquid & Plastic Limit, Plasticity Index - NZS 4402: 1986 Tests 2.2 (4 Point), 2.3 & 2.4

TEST DETAILS

LOCATION	**Description**	Momentum Land - Block 1 - Beach Road		
	Data	N/A		
SAMPLE	**Geotechnics ID**	S22CH000484		
	Reference	Jar 1	**Top Depth**	0.9m
	Sampled By	Others, Tested As Received	**Bottom Depth**	1.0m
	Description	Clayey SILT with trace sand, grey mix with brown. Moist, high plasticity.		
SPECIMEN	**Reference**	N/A	**Depth**	N/A
	Description	N/A		

TEST RESULTS

Liquid Limit	**58**
Plastic Limit	**30**
Plasticity Index	**28**

TEST REMARKS

• The material used for testing was natural, fraction passing a 425um sieve. • This test result is IANZ accredited.•Date tested 26/05/2022

Approved Signatory	Jack Singh
Date	27/05/2022

GEOTECHNICS

45a Parkhouse Road
Wigram
Christchurch 8042
www.geotechnics.co.nz

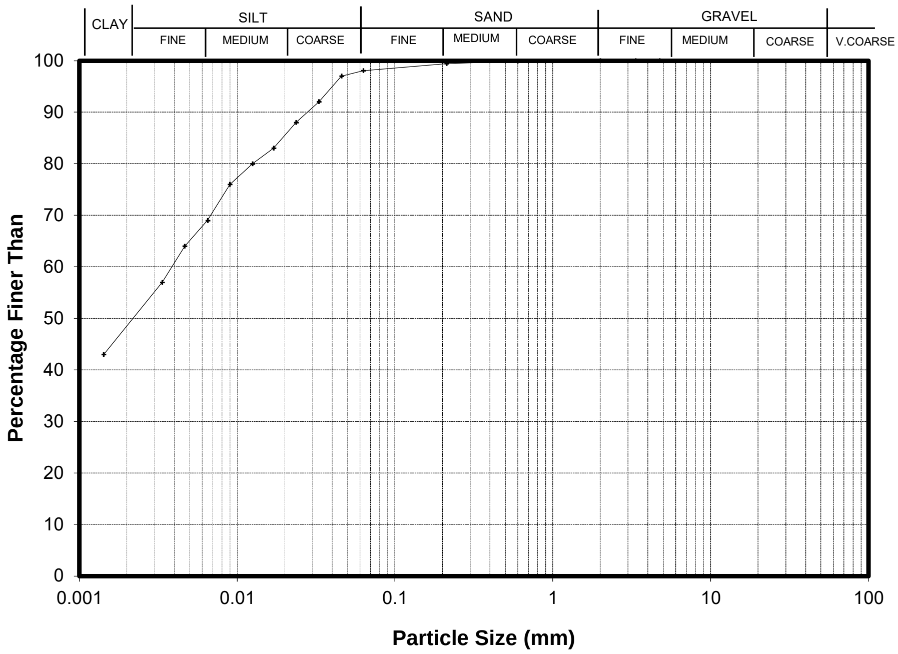
W22CH-0109

Page 1 of 1

Your Job No.: 1019317.0
Our Job No.: 1100960.0016.0.0

Site: Momentum Land Living - Beach Road
BH No.: Jar 1 Sample ID.: S22CH000484 Depth: 0.9m - 1.0m
Test Method Used : NZS 4402:1986 Test 2.8.4 Hydrometer

PARTICLE SIZE ANALYSIS



Sieve (mm)	Total % Passing	Sieve (mm)	Total % Passing
4.75	100		
3.35	100		
2.00	100		
0.600	100		
0.212	99		
0.063	98		

Equivalent Particle Diameter D (mm)	% of Particles Finer than D
0.0458	97
0.0329	92
0.0237	88
0.0170	83
0.0125	80
0.0090	76
0.0065	69
0.0047	64
0.0034	57
0.0014	43

Sample history : Natural, whole soil
Description : Clayey SILT with trace sand, grey mix with brown. Moist, high plasticity.

Solid Density (assumed) : 2.65t/m^3

Remarks : A sub sample was split from the original sample for hydrometer analysis. This sample was soaked with a dispersing agent (~2 hrs), then the mechanical shaker was used, until the material was brought into suspension, before proceeding with the test.
Suspension pH 8.0
The classification of sand-silt-clay components were described on the basis of particle size analysis. Use of assumed values in calculations is at the customers discretion and risk.
Sample description is not IANZ accredited.

Entered by : JASI Date : 27/5/2022 Checked by :CXPG Date : 27/5/2022

45A Parkhouse Road
Wigram
Christchurch 8042
New Zealand
p +64 3 361 0300

GEOTECHNICS

Geotechnics Project Number	1100960.0016.0.0
QESTLab Work Order ID	W22CH-0109
Customer Project ID	1019317.0

Determination of Liquid & Plastic Limit, Plasticity Index - NZS 4402: 1986 Tests 2.2 (4 Point), 2.3 & 2.4

TEST DETAILS

LOCATION	**Description**	Momentum Land - Block 1 - Beach Road		
	Data	N/A		
SAMPLE	**Geotechnics ID**	S22CH000486		
	Reference	Jar 3	**Top Depth**	2.9m
	Sampled By	Others, Tested As Received	**Bottom Depth**	3.0m
	Description	SILT with some sand and minor clay, grey. Moist.		
SPECIMEN	**Reference**	N/A	**Depth**	N/A
	Description	N/A		

TEST RESULTS

Liquid Limit	**31**
Plastic Limit	**Not Suitable**
Plasticity Index	**Not Obtainable**

TEST REMARKS

• The material was unsuitable for testing the Plastic Limit. • This test result is IANZ accredited.•Date tested 26/05/2022

Approved Signatory Jack Singh

Date 27/05/2022

GEOTECHNICS

45a Parkhouse Road
Wigram
Christchurch 8042
www.geotechnics.co.nz

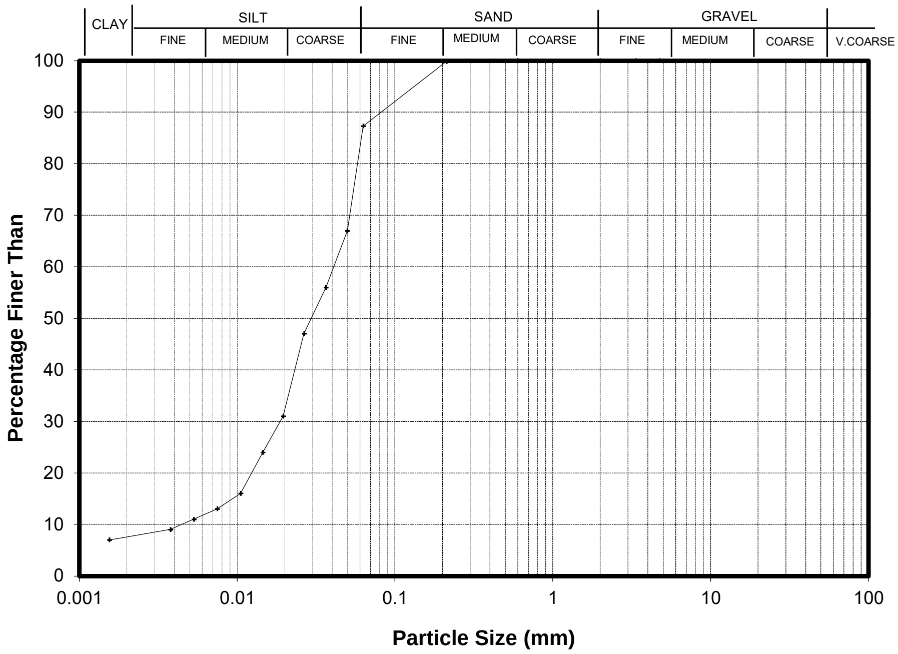
W22CH-0109

Page 1 of 1

Your Job No.: 1019317.0
Our Job No.: 1100960.0016.0.0

Site: Momentum Land Living - Beach Road
BH No.: Jar 3 Sample ID.: S22CH000486 Depth: 2.90m - 3.0m
Test Method Used : NZS 4402:1986 Test 2.8.4 Hydrometer

PARTICLE SIZE ANALYSIS



Sieve (mm)	Total % Passing	Sieve (mm)	Total % Passing
4.75	100		
3.35	100		
2.00	100		
0.600	100		
0.212	100		
0.063	87		

Equivalent Particle Diameter D (mm)	% of Particles Finer than D
0.0499	67
0.0365	56
0.0265	47
0.0196	31
0.0145	24
0.0105	16
0.0075	13
0.0053	11
0.0038	9
0.0015	7

Sample history : Natural, whole soil
Description : SILT with some sand and minor clay, grey. Moist.

Solid Density (assumed) : 2.65t/m^3

Remarks : A sub sample was split from the original sample for hydrometer analysis. This sample was soaked with a dispersing agent (~2 hrs), then the mechanical shaker was used, until the material was brought into suspension, before proceeding with the test.
Suspension pH 8.0
The classification of sand-silt-clay components were described on the basis of particle size analysis. Use of assumed values in calculations is at the customers discretion and risk.
Sample description is not IANZ accredited.

Entered by : JASI Date : 27/5/2022 Checked by :CXPG Date : 27/5/2022

45A Parkhouse Road
Wigram
Christchurch 8042
New Zealand
p +64 3 361 0300

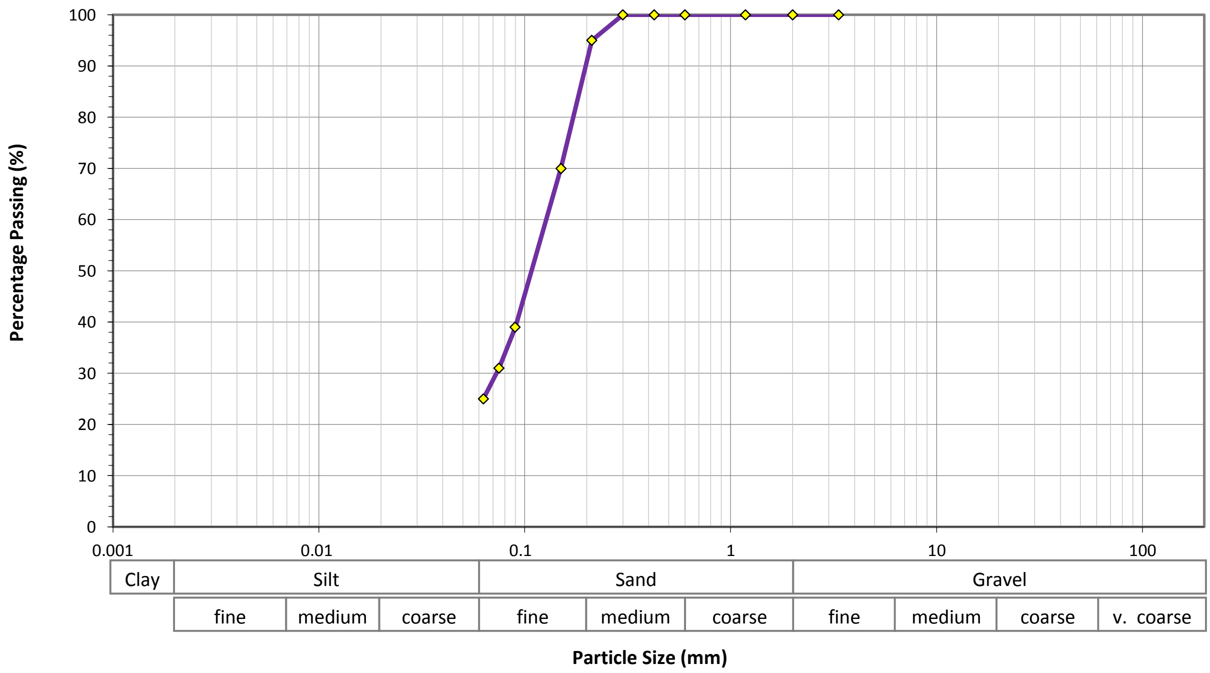
Geotechnics Project Number 1100960.0016.0.0
QESTLab Work Order ID W22CH-0109
Customer Project ID 1019317.0

Determination of the Particle Size Distribution - NZS 4402:1986 Test 2.8.1 (Wet Sieve)

TEST DETAILS

LOCATION	**Description**	Momentum Land - Block 1 - Beach Road		
	Data	N/A		
SAMPLE	**Geotechnics ID**	S22CH000485		
	Reference	Jar 2	**Top Depth**	2.0m
	Sampled By	Others, Tested As Received	**Bottom Depth**	2.1m
	Description	Silty fine to medium SAND, grey. Wet.		
SPECIMEN	**Reference**	N/A	**Depth**	N/A
	Description	N/A		

TEST RESULTS



Sieve Size (mm)	Percentage Passing (%)	Sieve Size (mm)	Percentage Passing (%)	Sieve Size (mm)	Percentage Passing (%)	Sieve Size (mm)	Percentage Passing (%)
150	-	26.5	-	4.75	-	0.300	100
100	-	19.0	-	3.35	100	0.212	95
75.0	-	16.0	-	2.00	100	0.150	70
63.0	-	13.2	-	1.18	100	0.090	39
53.0	-	9.50	-	0.600	100	0.075	31
37.5	-	6.70	-	0.425	100	0.063	25

TEST REMARKS

• The material used for testing was natural, whole soil. • The percentage passing the <0.063mm was obtained by difference. • This test result is IANZ accredited.•Date tested 26/05/2022

Approved Signatory Jack Singh

Date 27/05/2022

www.tonkintaylor.co.nz