REPORT

Appendix A: Report 1

Tonkin+Taylor

Momentum Land

Geotechnical Report for Proposed Subdivision - North Block - for Plan Change Application

Prepared for
Momentum Land Ltd
Prepared by
Tonkin & Taylor Ltd
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30/09/22	1	Report for plan change application	S Burgess	A Sleight	G Lovell
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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 1.1: Design summary

<table>
<tr><td>Anticipated site development</td><td>Approximately 615 1-2 storey residential dwellings are proposed in total at the site. The site is proposed to be raised with hardfill by approximately 1.5 m to 2.4 m RL (Lyttleton Vertical Datum)).</td></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>0.5 - 2.0</td><td>Soft to stiff SILT to sandy SILT</td></tr>
<tr><td>2z</td><td>0.0 – 6.5</td><td>Very soft SILT with occasional sand laminations</td></tr>
<tr><td>2a</td><td>0.0 – 3.0</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>Unconfirmed</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.
Targeted geogrid may be required in some localised areas to address liquefaction-induced ground damage.</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 developed area in the direction of lower elevation. Based on the preliminary assessment, deep ground improvement may be required in some areas, but this could be refined with further investigation and analysis.</td></tr>
<tr><td>Static settlement</td><td>Preloading is expected to be required to mitigate static consolidation settlement of soft silts and is likely to be in the order 1.5 m high, left in place and monitored for a time period in the order of 12 months. Further investigations are likely to reduce the area that requires preload. Alternative settlement mitigation methods can be considered if time constraints dictate.</td></tr>
<tr><td>Foundation recommendation</td><td>We recommend TC2 type enhanced concrete slabs in accordance with MBIE guidance, founded on the fill platform.</td></tr>
</table>

Development recommendations	With our prior understanding of the local ground conditions, we consider the density of subsurface investigations at the site to be sufficient and we believe 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. More geotechnical investigations are required at the detailed design phase to support a full subdivision application.

1 Introduction

This report presents the results of the initial geotechnical investigation and assessment completed by Tonkin & Taylor Ltd (T+T) for the proposed Momentum Land Subdivision Development – North Block, in Kaiapoi. 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.

The work described in this document was commissioned by Momentum Land Ltd and was completed in accordance with the letter of engagement dated 28 April 2022, job number 1019317.2000.

1.1 Scope of work

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

- 12 Cone Penetration Tests (CPTs) and Geophysical testing.
- Preparation of geological profiles.
- High-level liquefaction analysis and lateral spreading assessment.
- Identification of foundation options for the proposed development.
- 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 143-151 Ferry Road and comprises three blocks of land covering a total area of approximately 28.5 hectares. The site is accessed from Beach Road, approximately 1.0 km north-east of Kaiapoi town centre.

The site is bounded by farmland to the north and east, Beach Grove subdivision to the south, and residential homes of the Moorcroft subdivision to the west. The site is currently used as farmland.

The site is predominantly flat and is similar in elevation to the surrounding area to the north and east. The Moorcroft subdivision to the west, is approximately 2.0 m higher and Beach Grove subdivision to the south which is approximately 1.5 m higher. The current alignment of McIntosh stream runs along the eastern boundary and flows north to south.

1.3 Proposed development

The proposed Momentum Land – North Block subdivision consists primarily of 1-2 storey residential use buildings. A mixed use hub is also proposed, comprising small commercial and residential buildings. Approximately 615 dwellings are proposed in total at the site, split amongst different building types including; standard residential lots, townhouses, and apartments. Figure 1 shows a concept plan of the site.

Figure 1: Plan showing the concept layout for the site. Site boundary shown in solid red (from Beach Grove Expansion Project – Discussion document, Momentum Land June 2022) (NTS).

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 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.

The shallow channel depressions typical of Estuarine plain deposits are evident on the site.

Figure 2: Extract from geomorphological map.

2.1.2 Current geotechnical investigations

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

- 12 CPTs extending to a maximum depth of 10.0 mbgl (metres below ground level).

[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

- Geophysical testing, including:
  - 8 Multi-channel Analysis of Surface Waves (MASW) transects with a total survey length of 3,000 m.
  - 63 Ground Penetrating Radar (GPR) transects with a total survey length of 3,175 m.

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

2.1.3 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	0.5 - 2.0	0.5 - 3
2z	Very soft SILT with occasional sand laminations	Christchurch Formation	0.8 – 8.0 (non-continuous layer)	0.0 – 6.5	0 - 1
2a	Loose to medium dense SAND with occasional silt bedding		0.8 - 2.0	0.0 – 3.0	3 - 15
2b	Dense to very dense SAND to Gravelly SAND		4.0 – 10.0	5.0 – 9.0	15 - 30
3a	Very dense GRAVEL	Burnham Formation	9.5 – 13.0	Unconfirmed	25 -30+

Large buried objects were identified at discrete locations in the GPR transects, at depths of between 1.0-2.0 mbgl. Similar objects were noted on GPR transects undertaken on Stage 4 of the nearby Beach Grove subdivision, which were confirmed as large buried trees during subsequent stripping of the site. Therefore, the objects identified beneath this site may also be buried trees.

2.1.4 Groundwater

2.1.4.1 Site observations

Observations made at the site during the CPT and BH investigations noted a variable depth to ground water from approximately 0.15 – 0.85 mbgl. The soil profile and our previous experience at the surrounding sites 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 0.8 mbgl over most of the site (the highest surface of Layer 2a 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).

2.1.4.2 Groundwater levels summary

As ground water levels can vary seasonally and in response to seismic shaking, a groundwater level of 0.1 mbgl (existing ground level) occurring below layer 1b has been adopted for design purposes. The base of Layer 1b is about 0.8 m bgl at its highest, however a conservative value of 0.5 m has been adopted for the liquefaction assessment.

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, 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.

In summary, artesian groundwater may be expected to be encountered between 0.8 m to 2.0 mbgl.

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 a Class E (Very soft soil) because the soft soil deposits are less than 10 metres thick. The site is not considered to be a 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.
- 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 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

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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.

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]. The ULS scenarios was assessed for IL2 (residential) developments. In addition, a 100-year return period event (ILS) 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**
Return period (years)	25	25	100	500
Moment magnitude (M_w)	7.5	6.0	6.0	7.5
Peak horizontal ground acceleration (PGA_H)	0.13 g	0.19 g	0.30 g	0.35 g

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.

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 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 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 drainage areas.

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.

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 (very 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. The occasional sand layers in the soft silt are likely to liquefy.

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 220 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}.

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

[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, 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 shaking.
- The cumulative thickness of the materials expected to liquefy increases as the shaking intensity level increases from SLS to ULS, 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. After fill has been placed, the site is generally expected to be a TC2 equivalent site. Targeted geogrid is likely be required in some localised areas to address liquefaction-induced ground damage. Additionally, further investigations may undercover hotspots of higher liquefaction potential that may require ground improvement that could range from shallow to deep ground improvement methods. 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 river bank 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 proposed to be raised by approximately 1.5 m. 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 green belt, wetland, farm, and stormwater management area around the eastern, northern, and western perimeter of the site which are expected to be at a lower elevation. This creates the potential for lateral spreading to occur along the edges of the site, with buildings moving towards the areas of lower elevation under liquefied conditions. Lateral spread mitigation methods such as stone column ground improvement should be anticipated beneath the perimeter roads and buildings, as shown in Figure 3. It is possible after further geotechnical investigations that mitigation measures may be reduced from deep to shallow ground improvement in favourable areas with less liquefaction hazard.

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 3: Lateral Spread 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		
Ground surface damage including total and differential settlement.	Crust Thickness, CT [11]	Range: 1.6 to 3.3 m Average: 2.1 m	Range: 1.6 to 2.9 m Average: 2.0 m	Range: 1.6 to 2.9 m Average: 2.0 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 to ULS events.
	Calculated one-dimensional post liquefaction reconsolidation settlement (S_{V1D})[12]	Range: 6 to 35 mm Average: 19 mm	Range: 15 to 71 mm Average: 36 mm	Range: 17 to 92 mm Average: 45 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: 2 to 9 Average: 5	Range: 4 to 16 Average: 10	Range: 5 to 17 Average: 12	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 deposits of up to 6.5 metres thickness have been identified in multiple areas on this site. Additional investigations are required to define the extent and behaviour of the deposits identified by these investigations.

The thickness and location of soft silt deposits on this site are highly variable and are present throughout the site. Until additional investigations are completed to understand the extent and behaviour of these deposits, it should be conservatively assumed that the entire site will require full settlement mitigation. This mitigation is likely to range from 0 to 1.5 metres of additional fill above the fill platform level, for a period of approximately 12 months. For planning purposes, 1.5 m of fill across the entire site should be assumed, however it is likely that after the additional investigations and analysis are complete, the area that requires 1.5 m additional preload will be greatly reduced.

From a settlement perspective the site can be developed into 1-2 storey residential lots with appropriate levels of preloading to mitigate post construction settlement risk.

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.

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.

3.3 Foundation recommendations

Based on the results of the initial geotechnical investigations, liquefaction assessment, and soft soil assessments discussed above, we recommend TC2 type enhanced concrete slabs in accordance with MBIE guidance founded on the fill platform.

Ground improvement is required as part of the foundation solution. Deep ground improvement for lateral spread mitigation is likely to be required on perimeter houses as discussed in Section 2.3.4.1 however this may be reduced to shallow ground improvement if further investigations indicate ground conditions are favourable. Targeted geogrid may also be required in some localised areas to address ILS and ULS liquefaction-induced ground damage or to mitigate consolidation settlement throughout the subdivision. Additionally, further investigations may undercover hotspots of higher liquefaction potential that may require ground improvement. We recommend building footprints are located entirely within a single mode of ground improvement and should not span multiple modes.

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 with layers of geogrid where required.

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 minimising 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 0.4-0.85 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 prior to construction.

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 layer there is a risk that water from the sand layer will move into the fill around the pipe and may flow along the pipeline. For 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 ground improvement

Any deep ground improvement 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 General

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 in the site and adjacent areas to be sufficient for the purposes of supporting a rezoning of the site to residential use. Further geotechnical investigations will be required to support the subdivision application and to inform detailed design.

This initial assessment indicates that the proposed fill platform is likely to generally achieve a TC2 equivalence. Some localised ground improvement is likely to be required which could range from additional grid placement to stone columns as previously discussed. More geotechnical investigations must be undertaken to confirm these conclusions. Additional improvement is required to mitigate lateral spread in specific areas, as previously discussed.

4.2 Provisional RMA Section 106 assessment

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.

As discussed in Section 4.1 above, the density of subsurface investigations provides sufficient information for a high-level assessment only. Based on the information available at the time of preparing this report, geotechnical-related natural hazards are assessed as follows:

- 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]) once fill is placed.
  – 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 expected to be able to 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, but is address in separate reports.

5 Further work

Additional deep geotechnical investigations will be required to:

- Support a subdivision development application and meet the geotechnical density requirements outlined in NZGS Module 2[18] .
- Better define the liquefaction response on the site.
- Better define the presence of compressible silts on the site.
- Develop foundation parameters for the buildings.

T+T can scope and organise this additional testing once development details are confirmed. This is likely to consist of 9-12 additional boreholes and approximately 50 additional CPTs to refusal.

A lateral spread assessment should be undertaken during future design stages 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

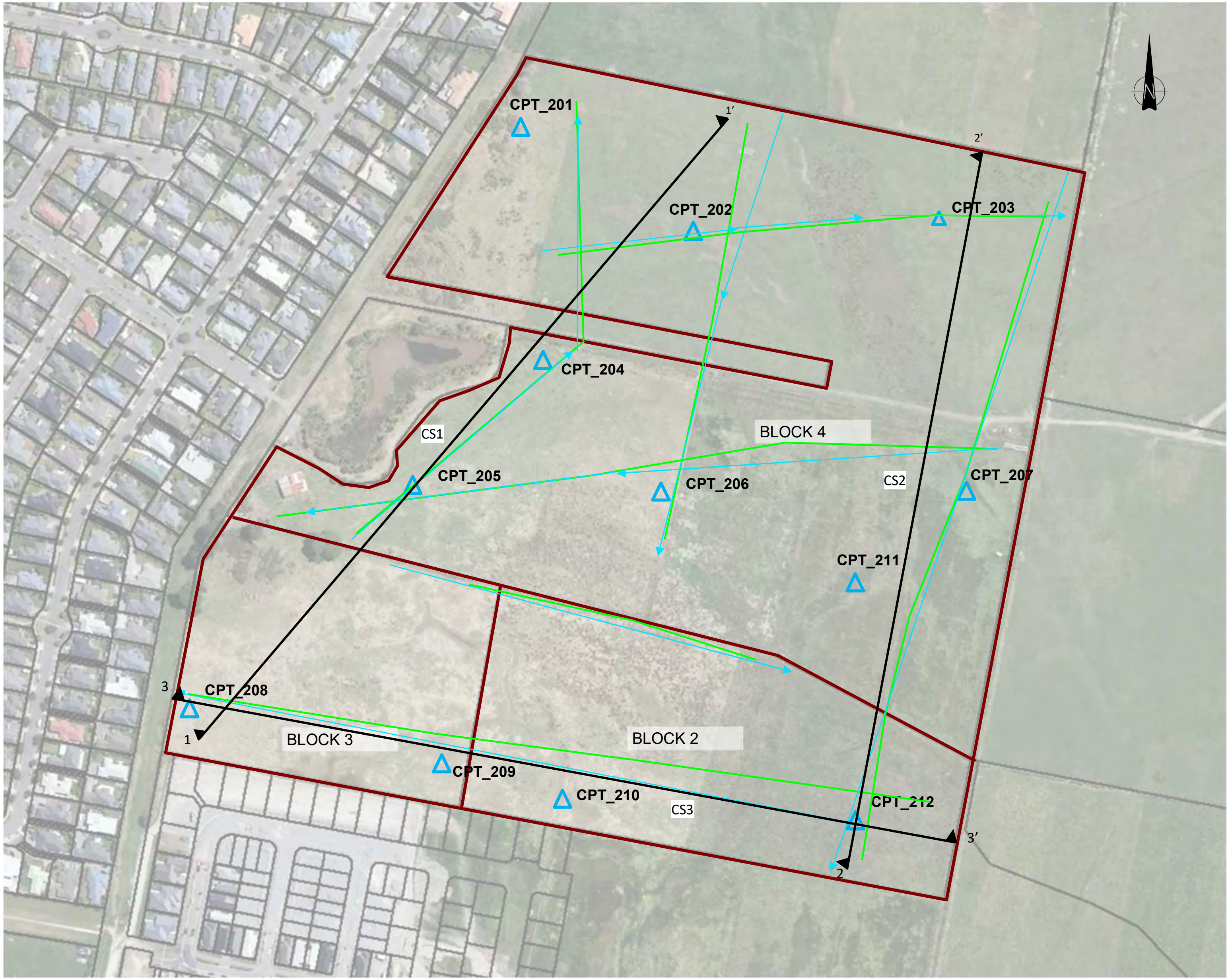
Reviewed by: Richard Brunton

18-May-23
\\ttgroup.local\corporate\christchurch\tt projects\1019317\issueddocuments\2023-05-15.snb.north block_georeport_final.docx

Appendix A Site investigation location plan

LEGEND

- Site boundary
- Cross Section
- MASW Line
- GPR Line
- T+T CPT (2022) — CPT_XX



Approx Scale

0 50 100 150 200 250 (m)

Notes:
1. Aerial image sourced from TTMapViewer 27/07/2021

Tonkin+Taylor
www.tonkintaylor.co.nz

DRAWN	SNB	MAY 23
DRAFTING CHECKED		
APPROVED		

FILE : FILE

APPROX. SCALE (AT A3 SIZE) NTS

PROJECT No. 1019317.2000

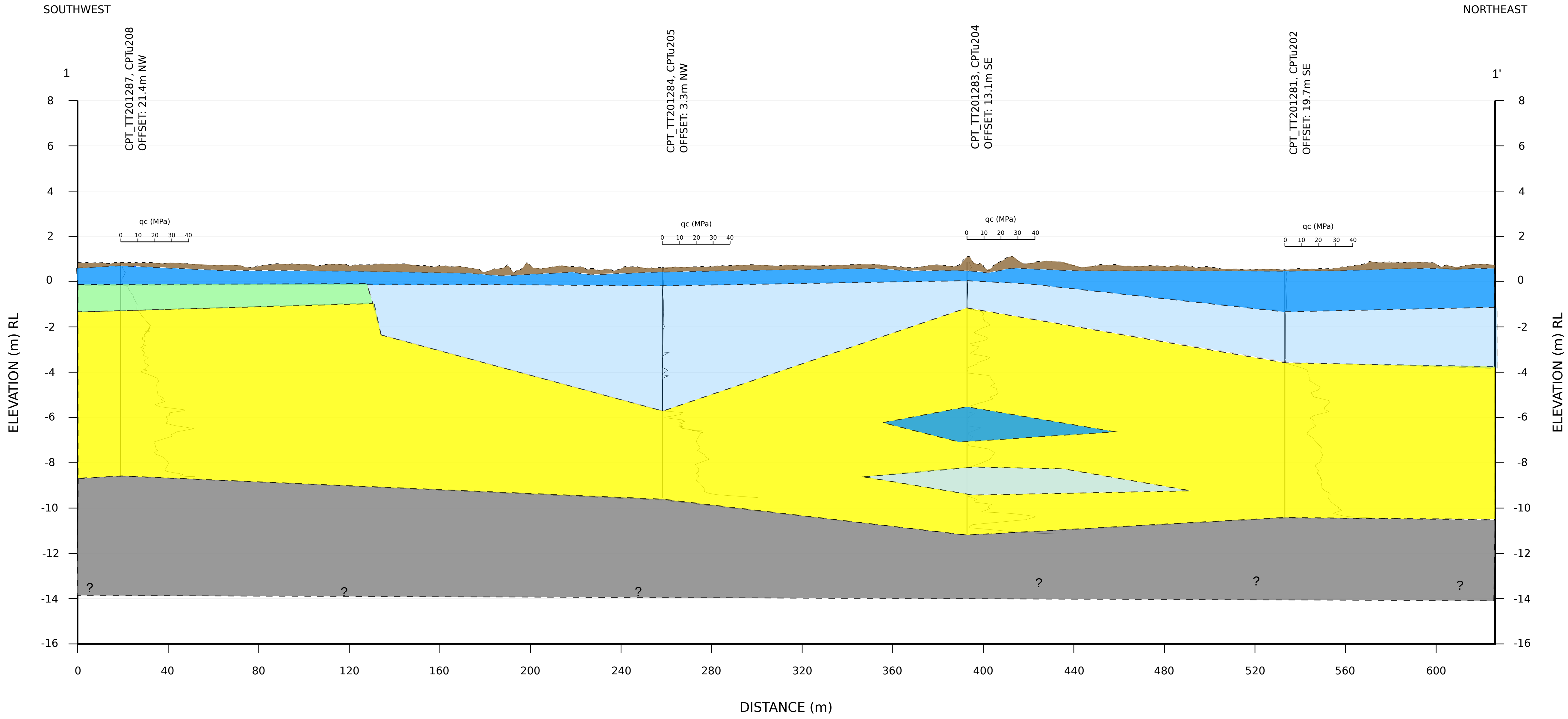
MOMENTUM LAND LIMITED
NORTH BLOCK

SITE PLAN WITH GEOTECHNICAL INVESTIGATIONS

FIG. No. Figure 1

REV. 0

Appendix B Geological cross sections



SECTION 1
SCALE: 1:2000H (1:200V)

T+T Cross-Section Generator (CSG)
Start Point (NZTM): 1573087.31 mE, 5197510.64 mN
End Point (NZTM): 1573492.29 mE, 5197987.83 mN
Date Generated: 2022-06-29, 13:36:27
CSG Version: 1.2 (First Release)
Vertical Datum: Investigation RLs are shown in NZVD2016 datum.
Groundline Source: LiDAR data licenced by: Waimakariri District Council. Dataset: Canterbury - Rangiora LiDAR (2014).

LEGEND

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

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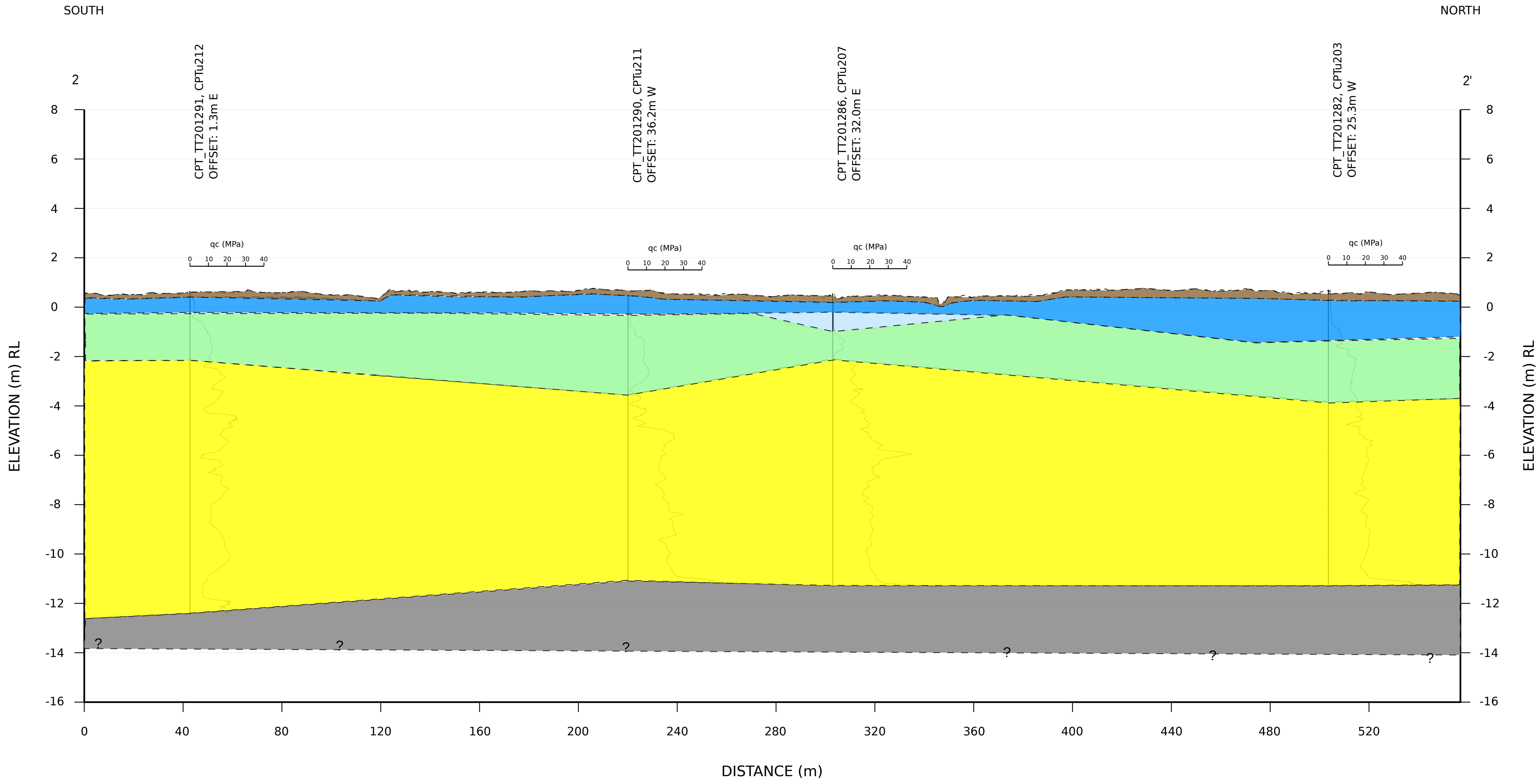
PROJECT No.	1019317.2000	
DESIGNED	SNB	May.23
DRAWN	SNB	May.23
CHECKED	AFS	May.23
APPROVED		DATE

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



SECTION 2
SCALE: 1:2000H
(1:200V)

LEGEND

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

T+T Cross-Section Generator (CSG)
Start Point (NZTM): 1573582.27 mE, 5197414.44 mN
End Point (NZTM): 1573684.46 mE, 5197961.98 mN
Date Generated: 2022-06-29, 16:29:24
CSG Version: 1.2 (First Release)
Vertical Datum: Investigation RLs are shown in NZVD2016 datum.
Groundline Source: LiDAR data licenced by: Waimakariri District Council. Dataset: Canterbury - Rangiora LiDAR (2014).

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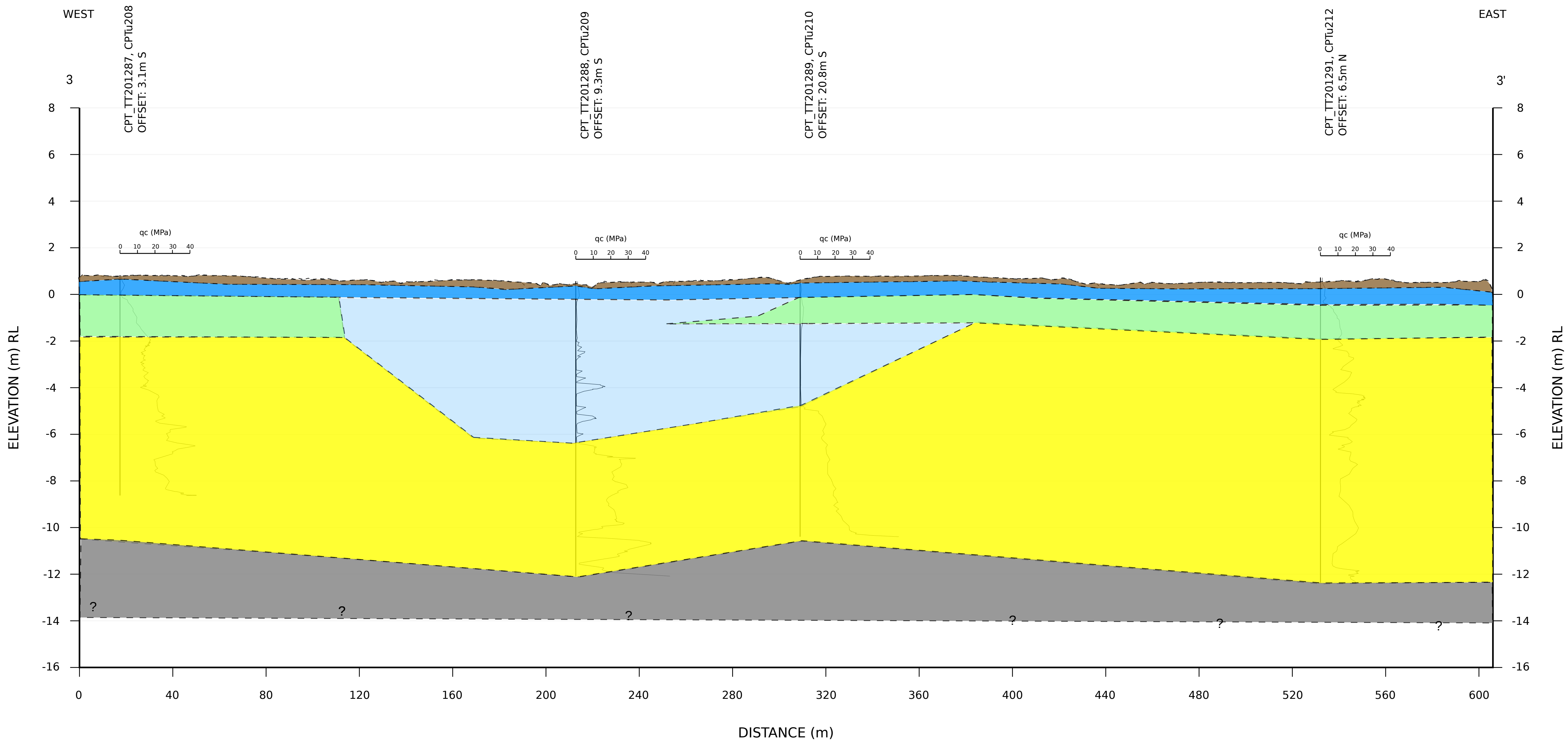
PROJECT No.	1019317.2000	
DESIGNED	SNB	May.23
DRAWN	SNB	May.23
CHECKED	AFS	May.23
APPROVED		DATE

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



SECTION 3
SCALE: 1:2000H
(1:200V)

LEGEND

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

T+T Cross-Section Generator (CSG)
Start Point (NZTM): 1573066.82 mE, 5197545.24 mN
End Point (NZTM): 1573662.63 mE, 5197436.62 mN
Date Generated: 2022-06-29, 16:31:54
CSG Version: 1.2 (First Release)
Vertical Datum: Investigation RLs are shown in NZVD2016 datum.
Groundline Source: LiDAR data licenced by: Waimakariri District Council. Dataset: Canterbury - Rangiora LiDAR (2014).

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PROJECT No.	1019317.2000	
DESIGNED	SNB	May.23
DRAWN	SNB	May.23
CHECKED	AFS	May.23
APPROVED	DATE	

CLIENT **MOMENTUM LAND LTD**
PROJECT **NORTH BLOCK**
TITLE GEOTECHNICAL CROSS SECTION 3
SCALE (A3) AS SHOWN FIG No. 4 REV 2

Appendix C Site specific investigation results

- **CPTs**
- **MASW and GPR**

CONE PENETRATION TEST (CPT) REPORT

Client: Tonkin and Taylor Ltd

Location: Beach Grove Subdivision
280 Beach Road, Kaiapoi

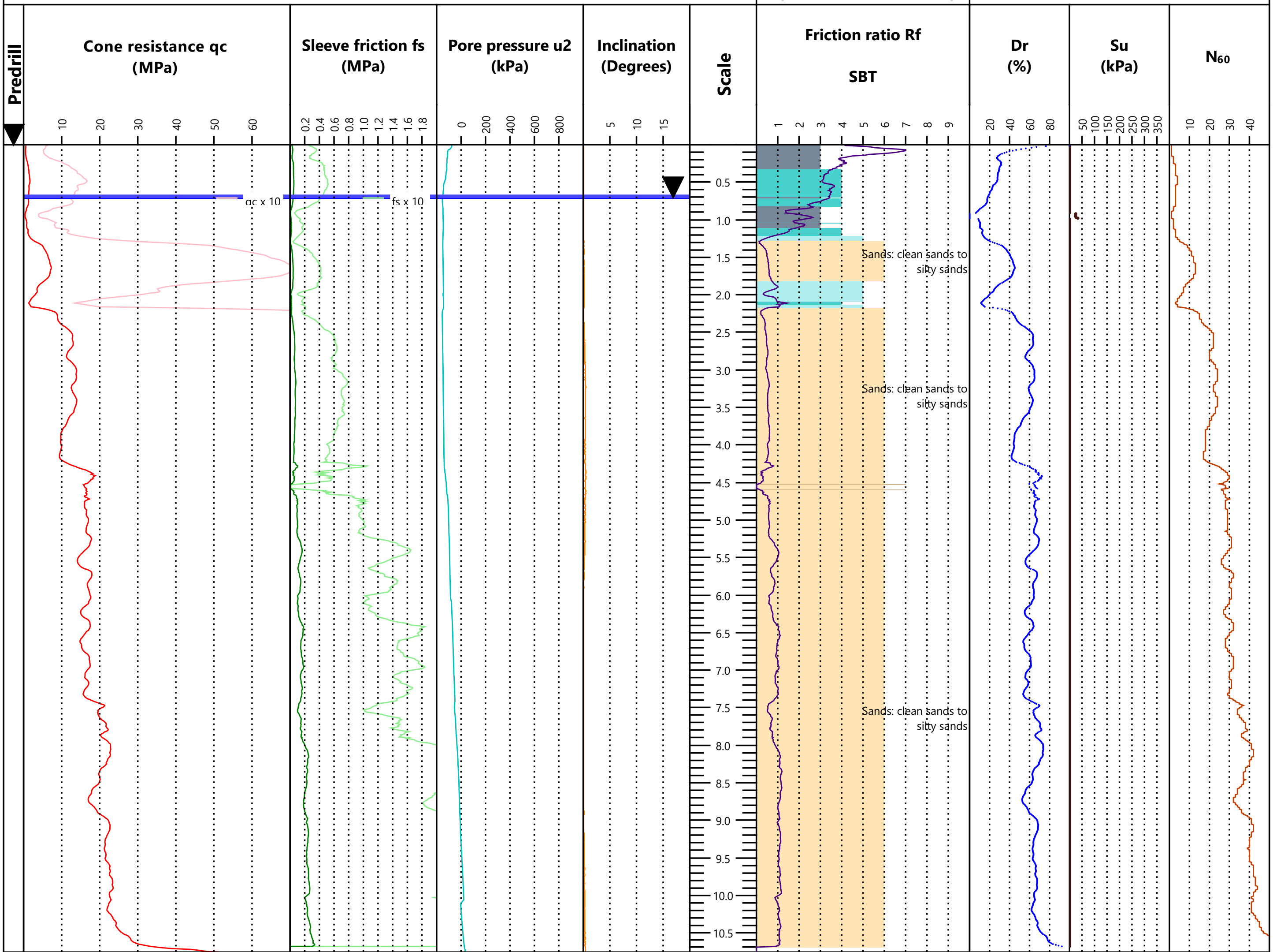
Printed: 05/05/2022

McMILLAN Drilling

Client: Tonkin and Taylor Ltd	**Bore No.:** CPTu201
Project: Beach Grove Subdivision	**Job No.:** 20872

Site Location: 280 Beach Road, Kaiapoi
Date: 3/5/2022
Grid Reference: 1573332.36m E, 5197980.65m N (NZTM) - Map or aerial photograph
Rig Operator: B. Wilson
Elevation: 0.00m **Datum:** Ground
Equipment: Geomil Panther 100

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



EOH: 10.76m

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

Predrill: -
Water Level: 0.70m
Collapse: 1.2m

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	0.8609	0.8792
Local Friction	0.0287	0.0259
Pore Pressure	0.0382	0.0409

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

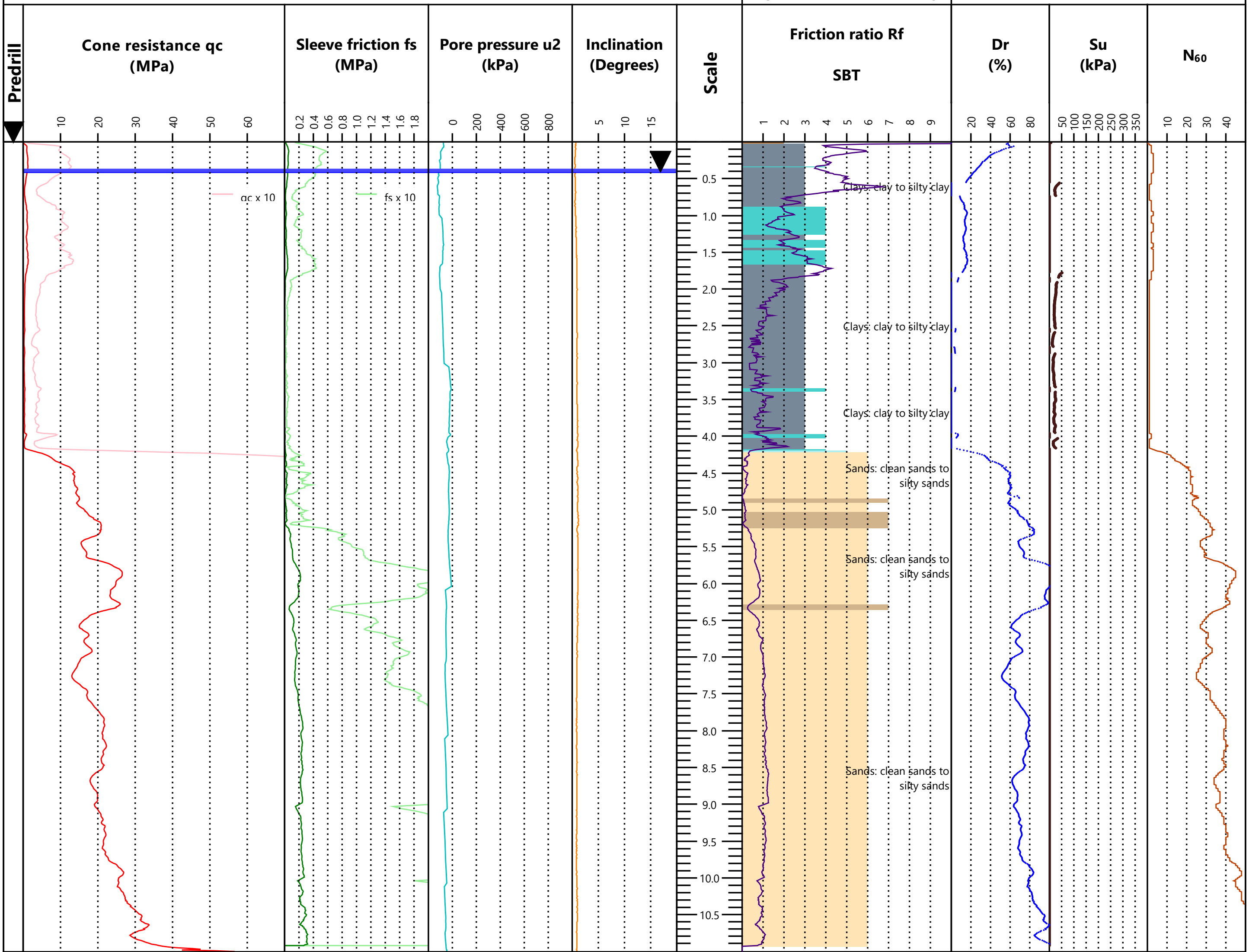
McMILLAN Drilling	Client: Tonkin and Taylor Ltd	Bore No.: CPTu202
	Project: Beach Grove Subdivision	Job No.: 20872

Site Location: 280 Beach Road, Kaiapoi **Date:** 3/5/2022

Grid Reference: 1573447.32m E, 5197904.45m N (NZTM) - Map or aerial photograph **Rig Operator:** B. Wilson

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

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



EOH: 11m

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

Predrill: -
Water Level: 0.40m
Collapse: 2.4m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.8834	0.8694
Local Friction	0.0258	0.0263
Pore Pressure	0.0453	0.0524

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

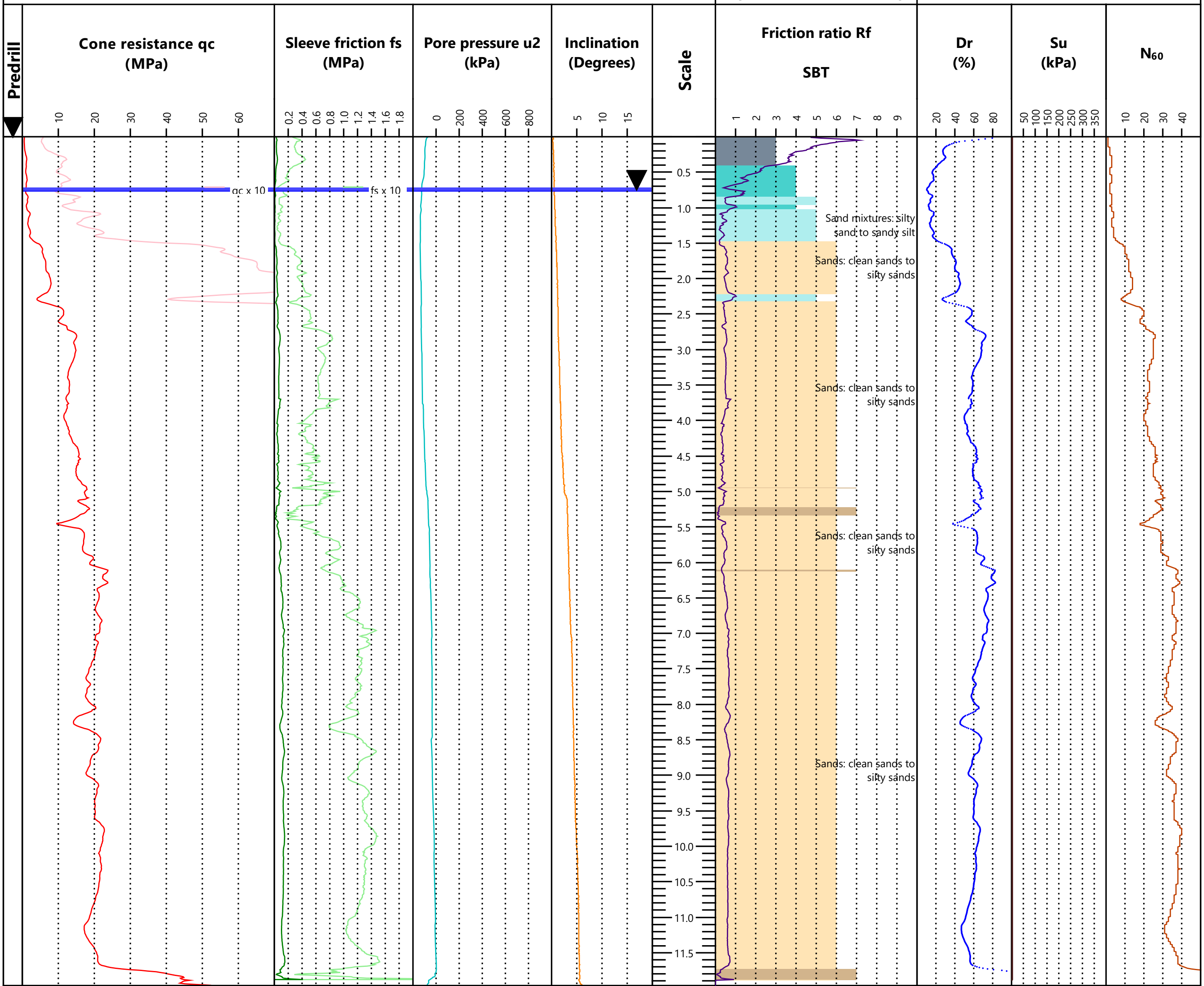
Client: Tonkin and Taylor Ltd	**Bore No.:** CPTu203
Project: Beach Grove Subdivision	**Job No.:** 20872

Site Location: 280 Beach Road, Kaiapoi **Date:** 3/5/2022

Grid Reference: 1573649.81m E, 5197914.09m N (NZTM) - Map or aerial photograph **Rig Operator:** B. Wilson

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

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



EOH: 11.96m

Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 111007
Cone Area Ratio: 0.75
Standards: ISO 22476-1:2012

Predrill: -
Water Level: 0.75m
Collapse: 1.0m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.1044	0.0632
Local Friction	0.0030	0.0038
Pore Pressure	-0.0075	-0.0101

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

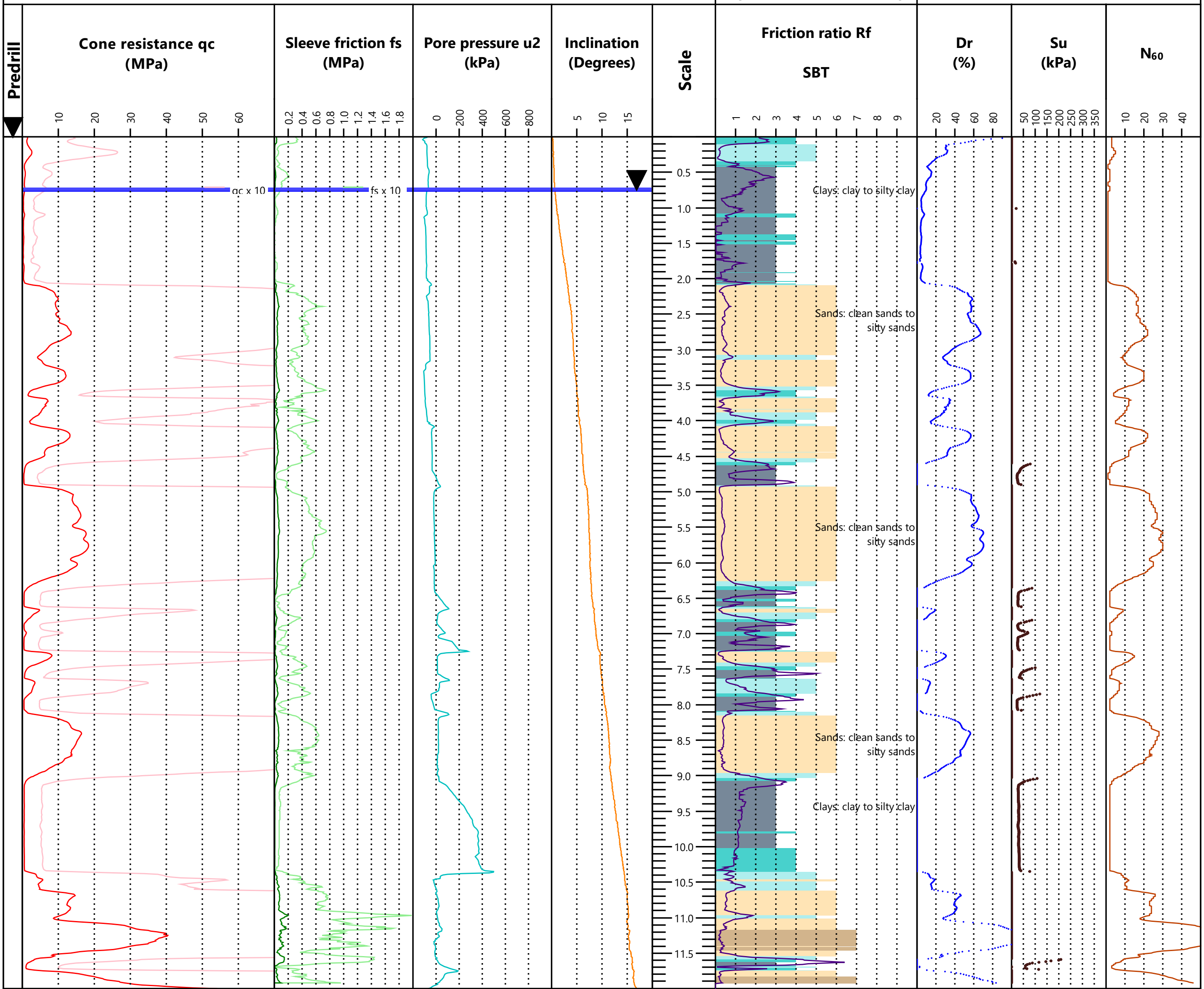
McMILLAN Drilling	Client: Tonkin and Taylor Ltd	Bore No.: CPTu204
	Project: Beach Grove Subdivision	Job No.: 20872

Site Location: 280 Beach Road, Kaiapoi **Date:** 2/5/2022

Grid Reference: 1573351.48m E, 5197801.66m N (NZTM) - Map or aerial photograph **Rig Operator:** B. Wilson

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

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



EOH: 12m

Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 111007
Cone Area Ratio: 0.75
Standards: ISO 22476-1:2012

Predrill: -
Water Level: 0.75m
Collapse: 1.1m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.1182	0.0917
Local Friction	0.0022	0.0024
Pore Pressure	-0.0092	-0.0088

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

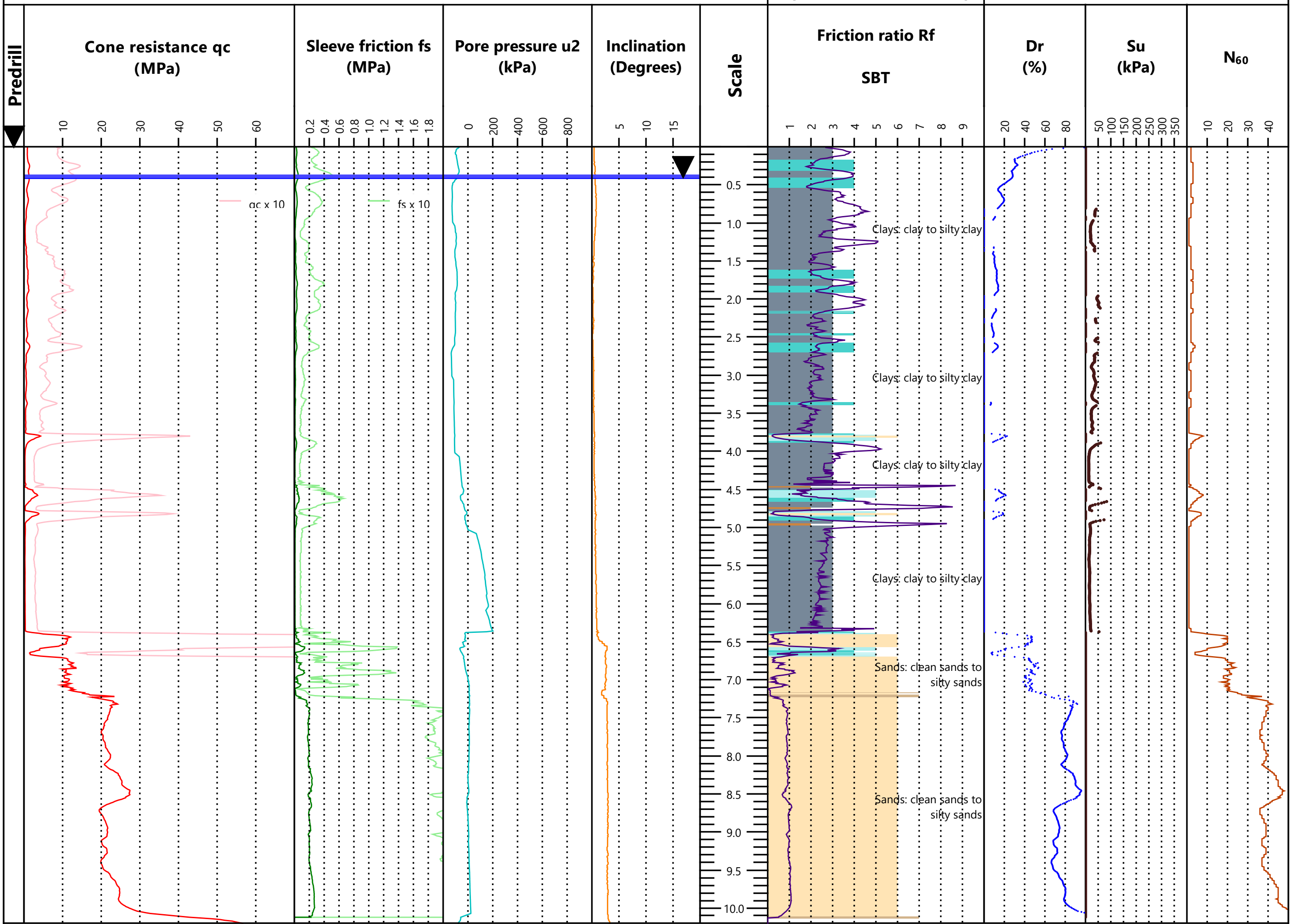
Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P. K. Robertson and K.L. Cabal, Guide to Cone Penetration Testing for Geotechnical Engineering. The interpretations 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

Inclination refusal at 10.63m.

McMILLAN Drilling	Client: Tonkin and Taylor Ltd	Bore No.: CPTu205
	Project: Beach Grove Subdivision	Job No.: 20872

Site Location: 280 Beach Road, Kaiapoi
Date: 2/5/2022
Grid Reference: 1573251.87m E, 5197709.62m N (NZTM) - Map or aerial photograph
Rig Operator: B. Wilson
Elevation: 0.00m **Datum:** Ground
Equipment: Geomil Panther 100



EOH: 10.2m

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

Predrill: -
Water Level: 0.40m
Collapse: 3.95m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.8467	0.8864
Local Friction	0.0269	0.0258
Pore Pressure	0.0512	0.0553

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

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Remarks

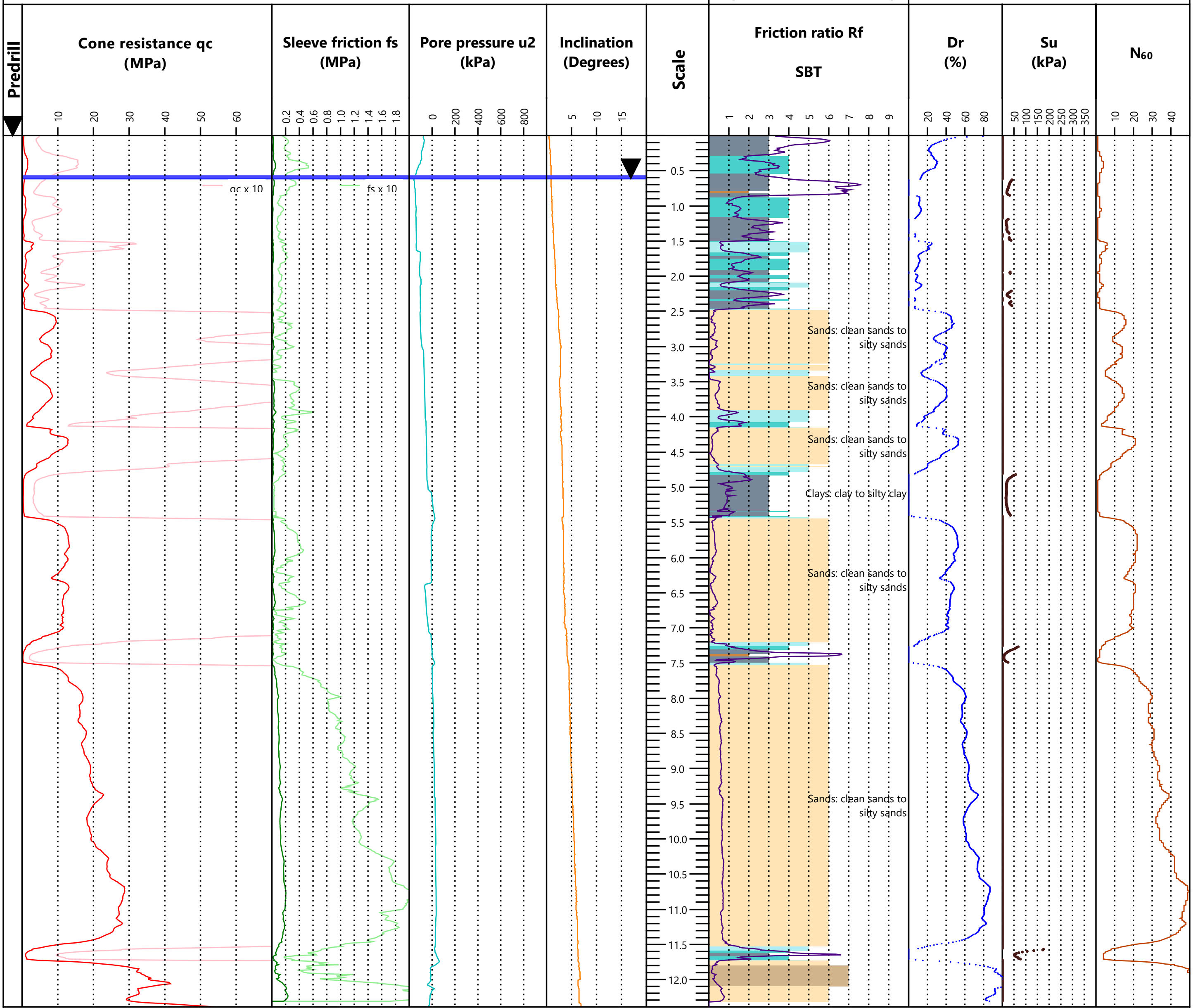
	Client: Tonkin and Taylor Ltd	**Bore No.:** CPTu206
McMILLAN Drilling	**Project:** Beach Grove Subdivision	**Job No.:** 20872

Site Location: 280 Beach Road, Kaiapoi **Date:** 2/5/2022

Grid Reference: 1573439.03m E, 5197703.38m N (NZTM) - Map or aerial photograph **Rig Operator:** B. Wilson

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

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



EOH: 12.39m

Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 111007
Cone Area Ratio: 0.75
Standards: ISO 22476-1:2012

Predrill: -
Water Level: 0.60m
Collapse: 2.0m

Termination

Target Depth ☐

Effective Refusal

Tip ☑
Gauge ☐
Inclinometer ☐
Other ☐

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	0.0843	0.0916
Local Friction	0.0040	0.0017
Pore Pressure	-0.0074	-0.0051

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

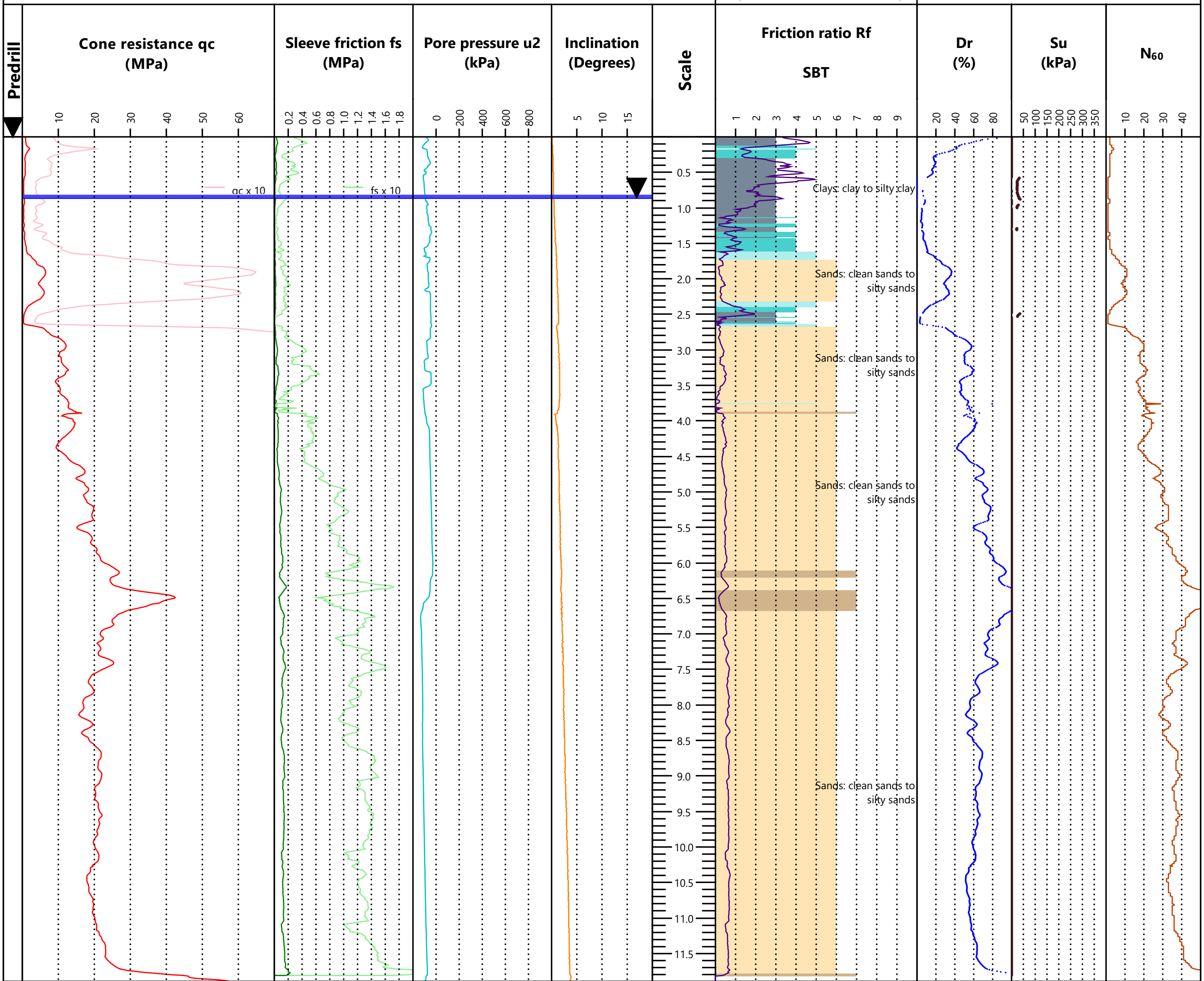
MCMILLAN Drilling	Client: Tonkin and Taylor Ltd	Bore No.: CPTu207
	Project: Beach Grove Subdivision	Job No.: 20872

Site Location: 280 Beach Road, Kaiapoi **Date:** 3/5/2022

Grid Reference: 1573669.27m E, 5197706.37m N (NZTM) - Map or aerial photograph **Rig Operator:** B. Wilson

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

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



EOH: 11.89m

Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 111007
Cone Area Ratio: 0.75
Standards: ISO 22476-1:2012

Predrill: -
Water Level: 0.85m
Collapse: 1.8m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.1333	0.1115
Local Friction	0.0018	0.0028
Pore Pressure	-0.0014	-0.0055

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

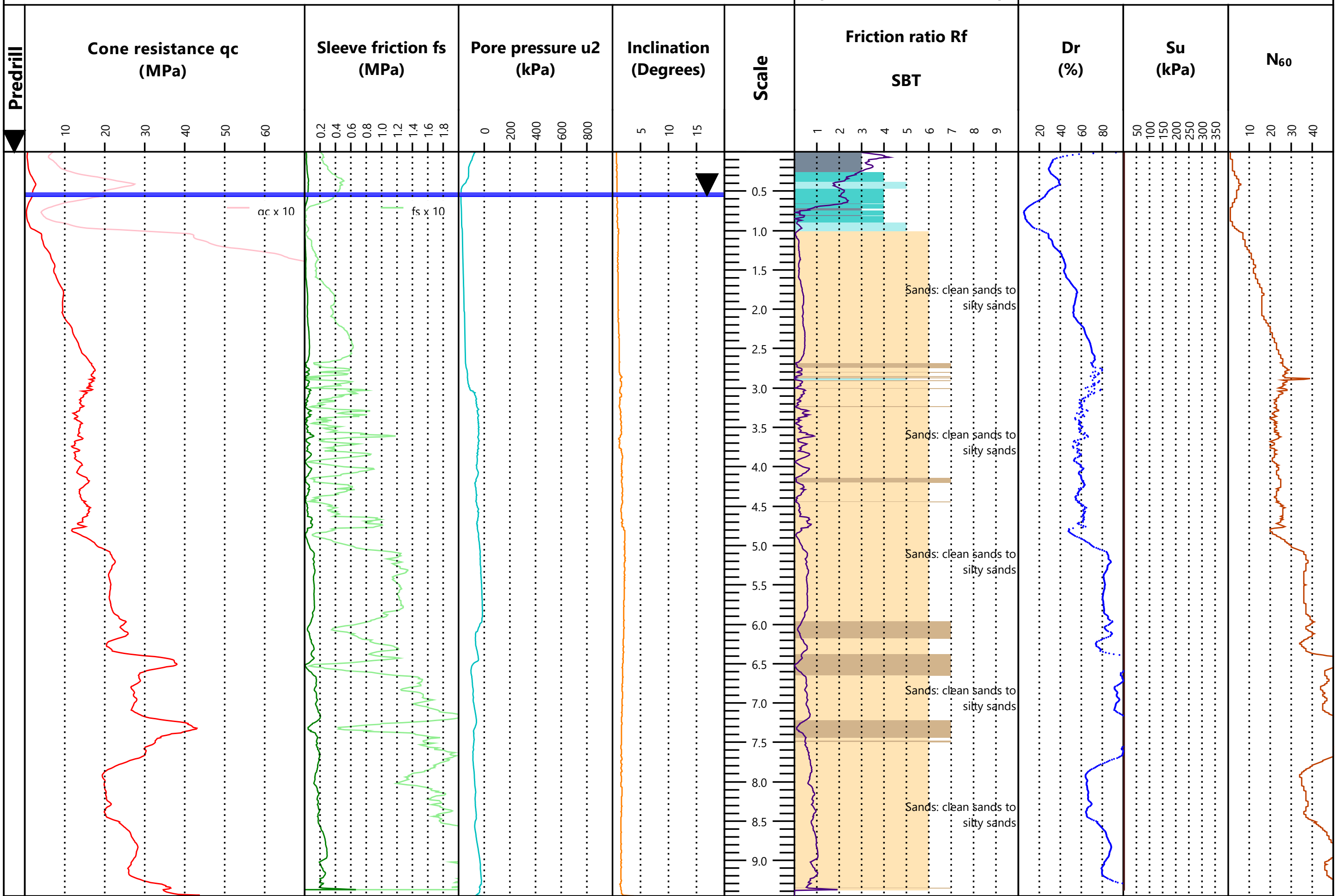
McMILLAN Drilling	Client: Tonkin and Taylor Ltd	Bore No.: CPTu208
	Project: Beach Grove Subdivision	Job No.: 20872

Site Location: 280 Beach Road, Kaiapoi **Date:** 4/5/2022

Grid Reference: 1573083.35m E, 5197539.08m N (NZTM) - Map or aerial photograph **Rig Operator:** B. Wilson

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

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



EOH: 9.45m

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

Predrill: -
Water Level: 0.55m
Collapse: 1.1m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.8976	0.8616
Local Friction	0.0264	0.0267
Pore Pressure	0.0429	0.0389

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

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Remarks

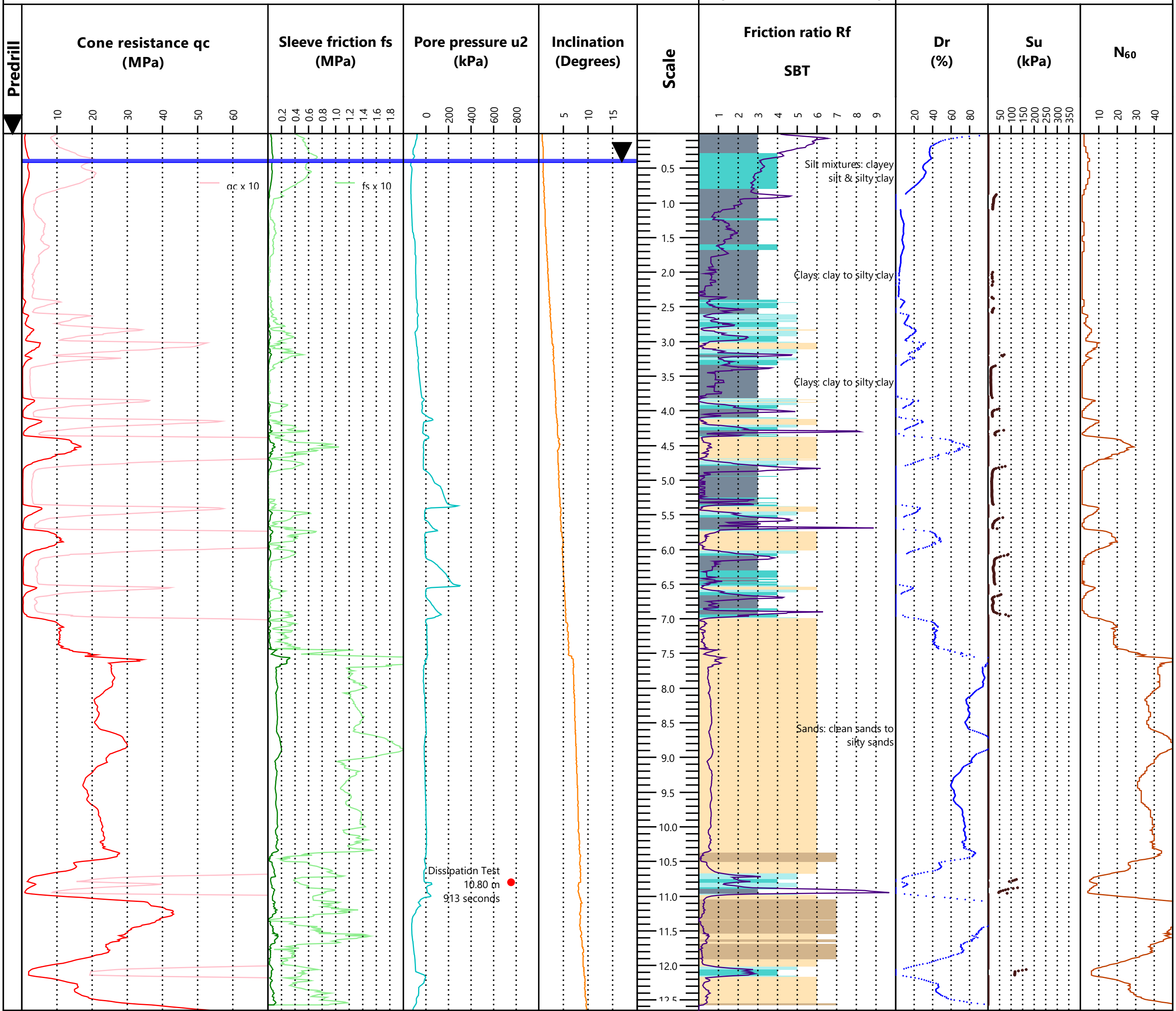
Client: Tonkin and Taylor Ltd	Bore No.: CPTu209
Project: Beach Grove Subdivision	Job No.: 20872

Site Location: 280 Beach Road, Kaiapoi | **Date:** 4/5/2022

Grid Reference: 1573274.48m E, 5197497.93m N (NZTM) - Map or aerial photograph | **Rig Operator:** B. Wilson

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

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



EOH: 12.65m

Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 111007
Cone Area Ratio: 0.75
Standards: ISO 22476-1:2012

Predrill: -
Water Level: 0.40m
Collapse: 3.0m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.1589	0.1093
Local Friction	0.0032	0.0029
Pore Pressure	-0.0050	-0.0053

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

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Remarks

McMILLAN Drilling

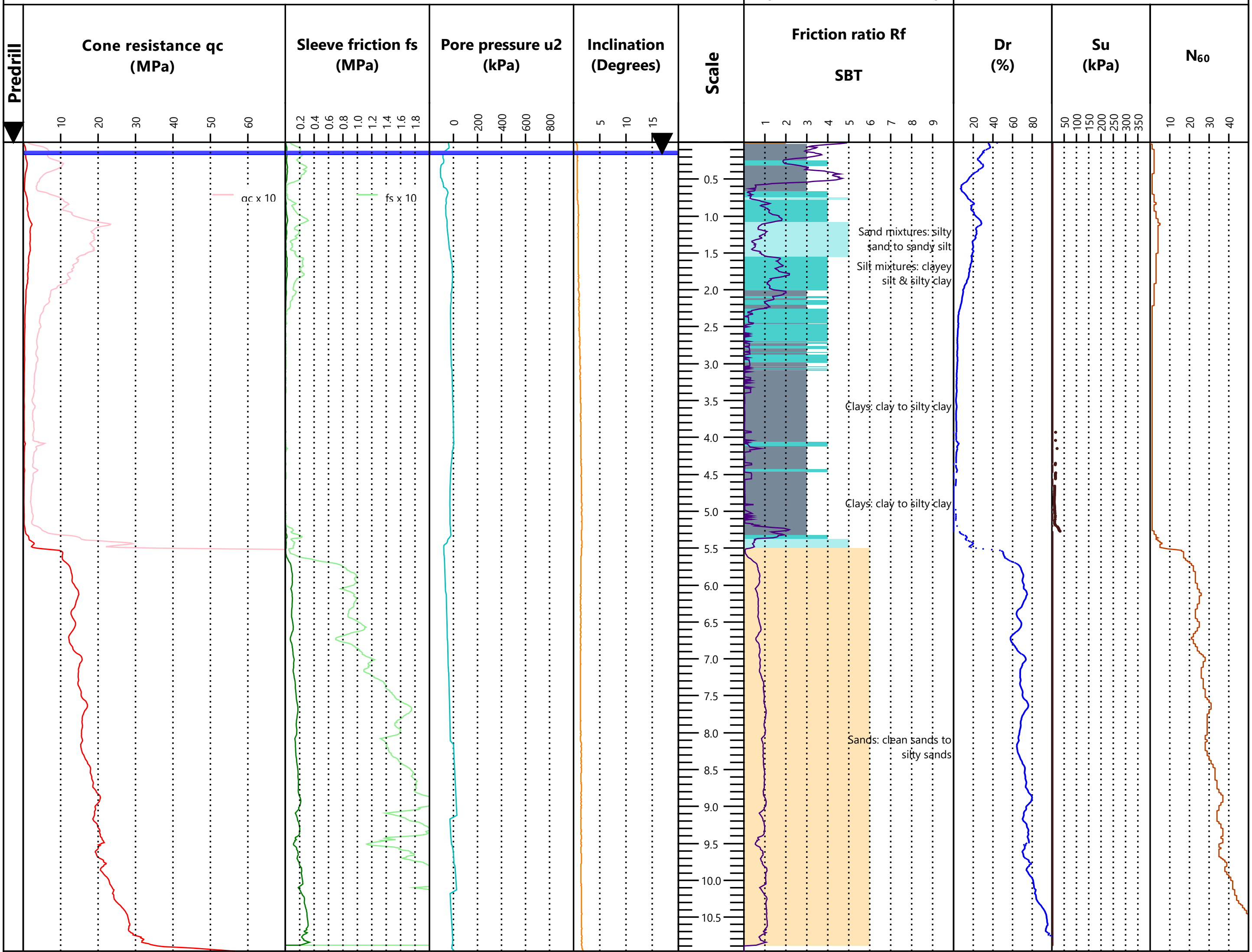
Client:	Tonkin and Taylor Ltd	**Bore No.:**	**CPTu210**
Project:	Beach Grove Subdivision	**Job No.:**	**20872**

Site Location: 280 Beach Road, Kaiapoi **Date:** 4/5/2022

Grid Reference: 1573367.04m E, 5197469.39m N (NZTM) - Map or aerial photograph **Rig Operator:** B. Wilson

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

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



EOH: 10.96m

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

Predrill: -
Water Level: 0.15m
Collapse: 2.3m

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	0.9305	0.9338
Local Friction	0.0263	0.0265
Pore Pressure	0.0150	0.0427

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

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Remarks

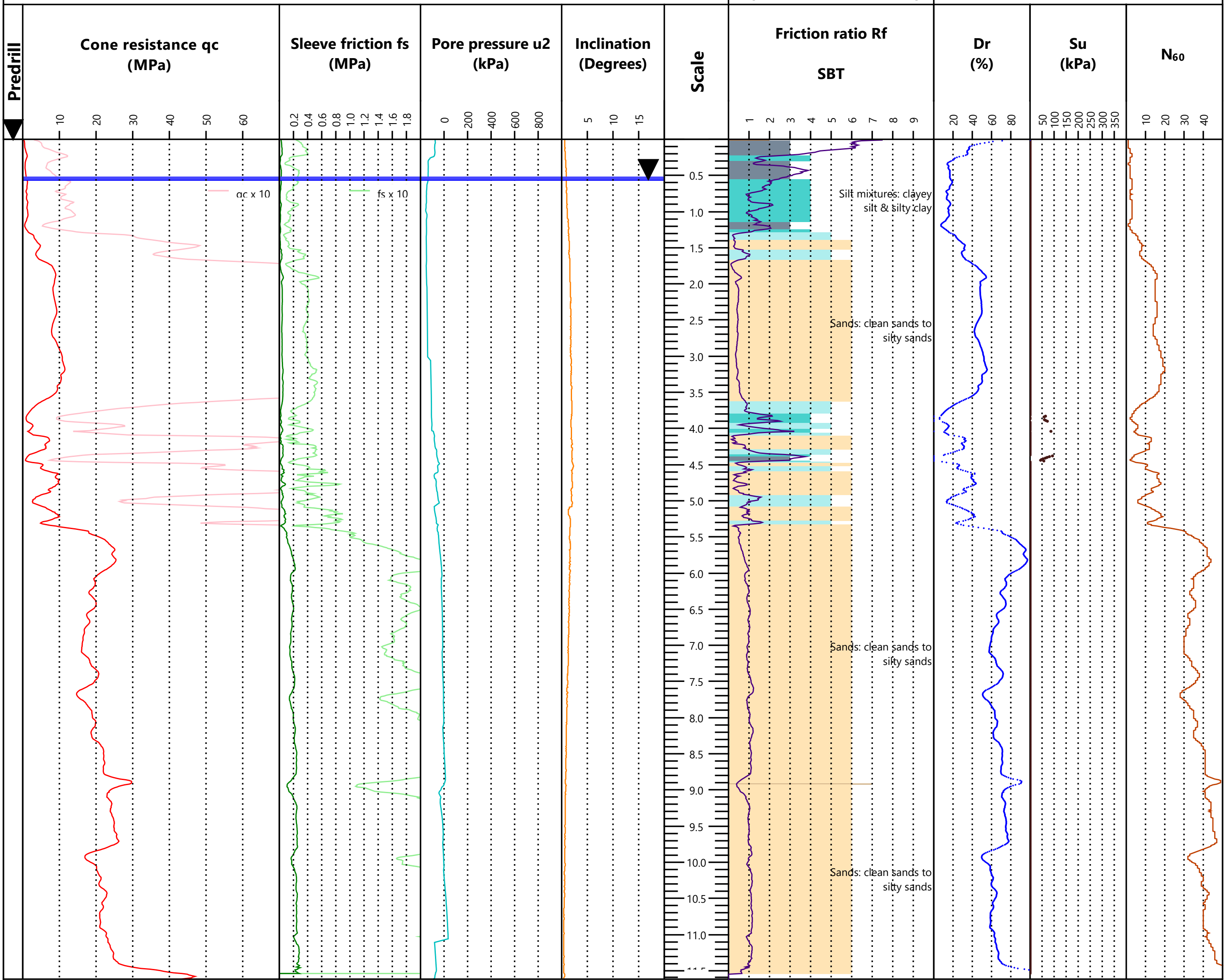
McMILLAN Drilling	Client: Tonkin and Taylor Ltd	Bore No.: CPTu211
	Project: Beach Grove Subdivision	Job No.: 20872

Site Location: 280 Beach Road, Kaiapoi **Date:** 2/5/2022

Grid Reference: 1573587.05m E, 5197637.4m N (NZTM) - Map or aerial photograph **Rig Operator:** B. Wilson

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

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



EOH: 11.62m

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

Predrill: -
Water Level: 0.55m
Collapse: 0.85m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.9558	0.8832
Local Friction	0.0260	0.0262
Pore Pressure	0.0486	0.0477

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

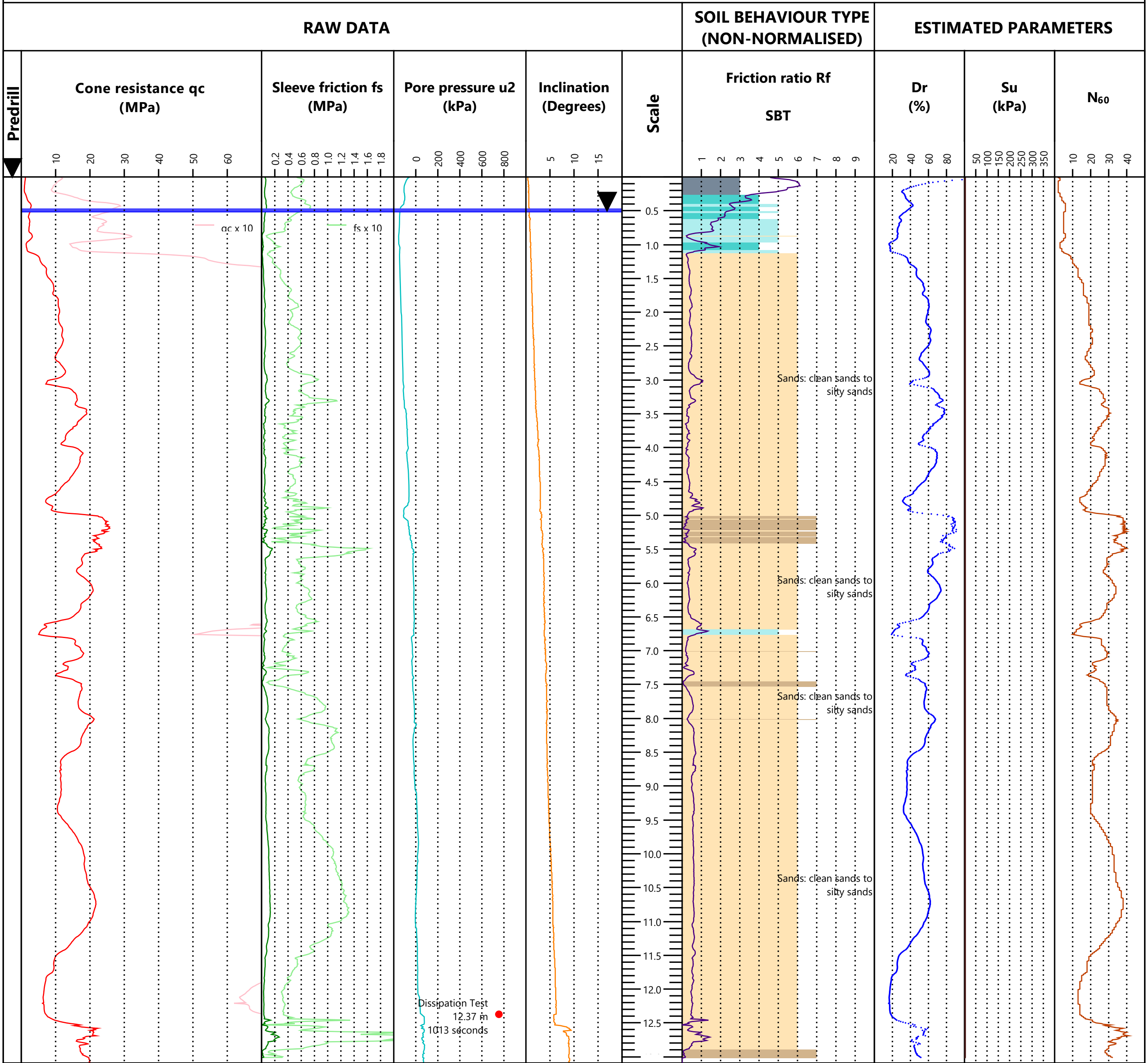
McMILLAN Drilling

Client: Tonkin and Taylor Ltd	**Bore No.:** CPTu212
Project: Beach Grove Subdivision	**Job No.:** 20872

Site Location: 280 Beach Road, Kaiapoi **Date:** 3/5/2022

Grid Reference: 1573591.42m E, 5197456.23m N (NZTM) - Map or aerial photograph **Rig Operator:** B. Wilson

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



Cone Type: I-CFXYP20-10 - Compression

Cone Reference: 111007

Cone Area Ratio: 0.75

Standards: ISO 22476-1:2012

Predrill: -

Water Level: 0.50m

Collapse: 0.8m

Termination

Target Depth ✓

Effective Refusal

Tip ☐

Gauge ☐

Inclinometer ☐

Other ☐

Zero load outputs (MPa)	**Before test**	**After test**
Tip Resistance	0.0839	0.1483
Local Friction	0.0037	0.0026
Pore Pressure	-0.0098	-0.0077

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: CPTu201
Sounding: 1

Operator:	B. Wilson	**Date:** 3/5/2022	**Termination**	
Cone Type:	I-CFXYP20-10 - Compression	**Predrill:** 0.00m		
Cone Reference:	100992	**Water Level:** 0.70m	**Target Depth**	☐
Cone Area Ratio:	0.75	**Collapse:** 1.2m	**Effective Refusal**	

Zero load outputs (MPa)	**Before test**	**After test**		
Tip Resistance	0.8609	0.8792	Tip	☑
Local Friction	0.0287	0.0259	Gauge	☐
Pore Pressure	0.0382	0.0409	Inclinometer	☐
			Other	☐

PointID: CPTu202
Sounding: 2

Operator:	B. Wilson	**Date:** 3/5/2022	**Termination**	
Cone Type:	I-CFXYP20-10 - Compression	**Predrill:** 0.00m		
Cone Reference:	100992	**Water Level:** 0.40m	**Target Depth**	☐
Cone Area Ratio:	0.75	**Collapse:** 2.4m	**Effective Refusal**	

Zero load outputs (MPa)	**Before test**	**After test**		
Tip Resistance	0.8834	0.8694	Tip	☑
Local Friction	0.0258	0.0263	Gauge	☐
Pore Pressure	0.0453	0.0524	Inclinometer	☐
			Other	☐

PointID: CPTu203
Sounding: 3

Operator:	B. Wilson	**Date:** 3/5/2022	**Termination**	
Cone Type:	I-CFXYP20-10 - Compression	**Predrill:** 0.00m		
Cone Reference:	111007	**Water Level:** 0.75m	**Target Depth**	☐
Cone Area Ratio:	0.75	**Collapse:** 1.0m	**Effective Refusal**	

Zero load outputs (MPa)	**Before test**	**After test**		
Tip Resistance	0.1044	0.0632	Tip	☑
Local Friction	0.0030	0.0038	Gauge	☐
Pore Pressure	-0.0075	-0.0101	Inclinometer	☐
			Other	☐

PointID: CPTu204
Sounding: 4

Operator:	B. Wilson	**Date:** 2/5/2022	**Termination**	
Cone Type:	I-CFXYP20-10 - Compression	**Predrill:** 0.00m		
Cone Reference:	111007	**Water Level:** 0.75m	**Target Depth**	☐
Cone Area Ratio:	0.75	**Collapse:** 1.1m	**Effective Refusal**	

Zero load outputs (MPa)	**Before test**	**After test**		
Tip Resistance	0.1182	0.0917	Tip	☑
Local Friction	0.0022	0.0024	Gauge	☐
Pore Pressure	-0.0092	-0.0088	Inclinometer	☐
			Other	☐

PointID: CPTu205
Sounding: 5

Operator:	B. Wilson	**Date:** 2/5/2022	**Termination**	
Cone Type:	I-CFXYP20-10 - Compression	**Predrill:** 0.00m		
Cone Reference:	100992	**Water Level:** 0.40m	**Target Depth**	☐
Cone Area Ratio:	0.75	**Collapse:** 3.95m	**Effective Refusal**	

Zero load outputs (MPa)	**Before test**	**After test**		
Tip Resistance	0.8467	0.8864	Tip	☑
Local Friction	0.0269	0.0258	Gauge	☐
Pore Pressure	0.0512	0.0553	Inclinometer	☐
			Other	☐

TEST DETAIL

PointID: CPTu206
Sounding: 6

Operator: B. Wilson	**Date:** 2/5/2022	**Termination**	
Cone Type: I-CFXYP20-10 - Compression	**Predrill:** 0.00m		
Cone Reference: 111007	**Water Level:** 0.60m	**Target Depth**	☐
Cone Area Ratio: 0.75	**Collapse:** 2.0m	**Effective Refusal**	

Zero load outputs (MPa)	Before test	After test		
Tip Resistance	0.0843	0.0916	Tip	☑
Local Friction	0.0040	0.0017	Gauge	☐
Pore Pressure	-0.0074	-0.0051	Inclinometer	☐
			Other	☐

PointID: CPTu207
Sounding: 7

Operator: B. Wilson	**Date:** 3/5/2022	**Termination**	
Cone Type: I-CFXYP20-10 - Compression	**Predrill:** 0.00m		
Cone Reference: 111007	**Water Level:** 0.85m	**Target Depth**	☐
Cone Area Ratio: 0.75	**Collapse:** 1.8m	**Effective Refusal**	

Zero load outputs (MPa)	Before test	After test		
Tip Resistance	0.1333	0.1115	Tip	☑
Local Friction	0.0018	0.0028	Gauge	☐
Pore Pressure	-0.0014	-0.0055	Inclinometer	☐
			Other	☐

PointID: CPTu208
Sounding: 8

Operator: B. Wilson	**Date:** 4/5/2022	**Termination**	
Cone Type: I-CFXYP20-10 - Compression	**Predrill:** 0.00m		
Cone Reference: 100992	**Water Level:** 0.55m	**Target Depth**	☐
Cone Area Ratio: 0.75	**Collapse:** 1.1m	**Effective Refusal**	

Zero load outputs (MPa)	Before test	After test		
Tip Resistance	0.8976	0.8616	Tip	☐
Local Friction	0.0264	0.0267	Gauge	☑
Pore Pressure	0.0429	0.0389	Inclinometer	☐
			Other	☐

PointID: CPTu209
Sounding: 9

Operator: B. Wilson	**Date:** 4/5/2022	**Termination**	
Cone Type: I-CFXYP20-10 - Compression	**Predrill:** 0.00m		
Cone Reference: 111007	**Water Level:** 0.40m	**Target Depth**	☐
Cone Area Ratio: 0.75	**Collapse:** 3.0m	**Effective Refusal**	

Zero load outputs (MPa)	Before test	After test		
Tip Resistance	0.1589	0.1093	Tip	☑
Local Friction	0.0032	0.0029	Gauge	☐
Pore Pressure	-0.0050	-0.0053	Inclinometer	☐
			Other	☐

PointID: CPTu210
Sounding: 10

Operator: B. Wilson	**Date:** 4/5/2022	**Termination**	
Cone Type: I-CFXYP20-10 - Compression	**Predrill:** 0.00m		
Cone Reference: 100992	**Water Level:** 0.15m	**Target Depth**	☐
Cone Area Ratio: 0.75	**Collapse:** 2.3m	**Effective Refusal**	

Zero load outputs (MPa)	Before test	After test		
Tip Resistance	0.9305	0.9338	Tip	☑
Local Friction	0.0263	0.0265	Gauge	☐
Pore Pressure	0.0150	0.0427	Inclinometer	☐
			Other	☐

TEST DETAIL

PointID: CPTu211
Sounding: 11

Operator: B. Wilson
Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 100992
Cone Area Ratio: 0.75

Date: 2/5/2022
Predrill: 0.00m
Water Level: 0.55m
Collapse: 0.85m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.9558	0.8832
Local Friction	0.0260	0.0262
Pore Pressure	0.0486	0.0477

Termination

Target Depth ☐

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

PointID: CPTu212
Sounding: 12

Operator: B. Wilson
Cone Type: I-CFXYP20-10 - Compression
Cone Reference: 111007
Cone Area Ratio: 0.75

Date: 3/5/2022
Predrill: 0.00m
Water Level: 0.50m
Collapse: 0.8m

Zero load outputs (MPa)	Before test	After test
Tip Resistance	0.0839	0.1483
Local Friction	0.0037	0.0026
Pore Pressure	-0.0098	-0.0077

Termination

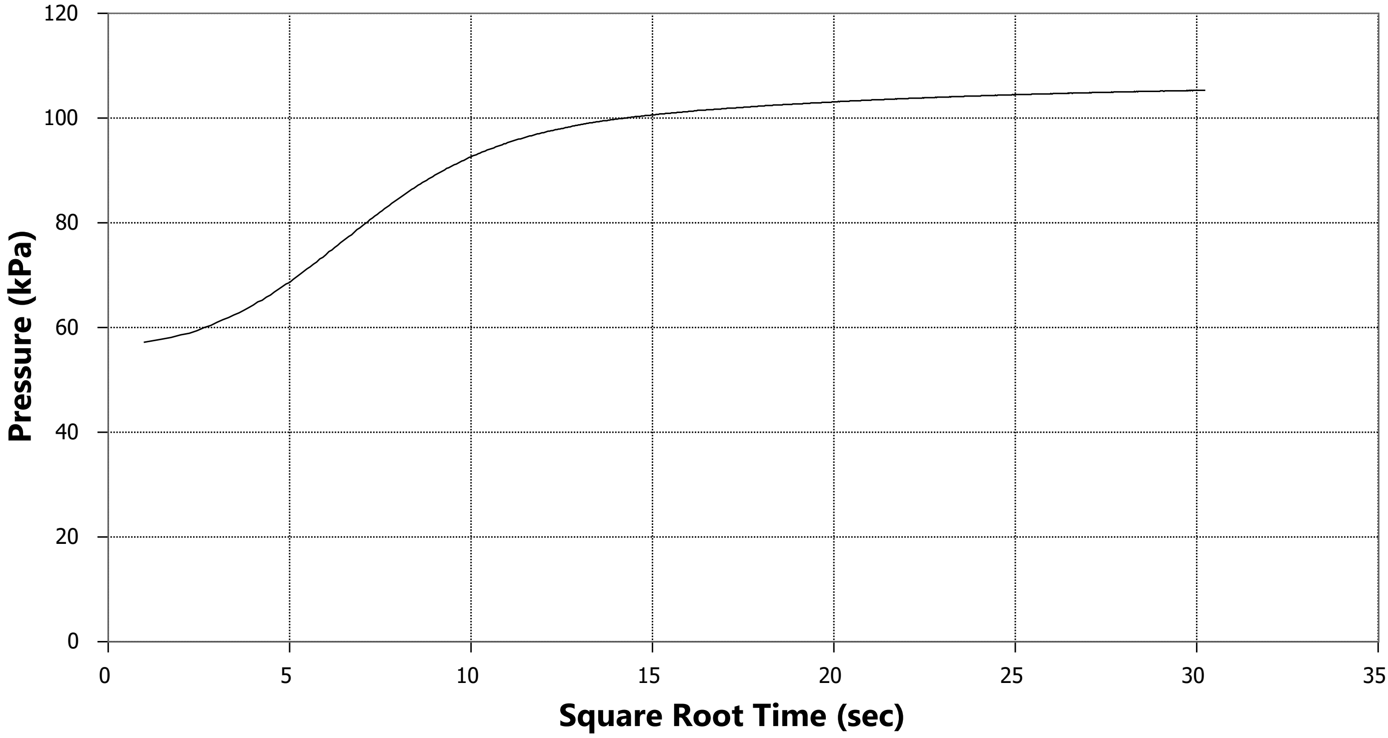
Target Depth ☑

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

DISSIPATION TESTS

PointID: CPTu209

Test Depth: 10.80 **Duration:** 913 seconds

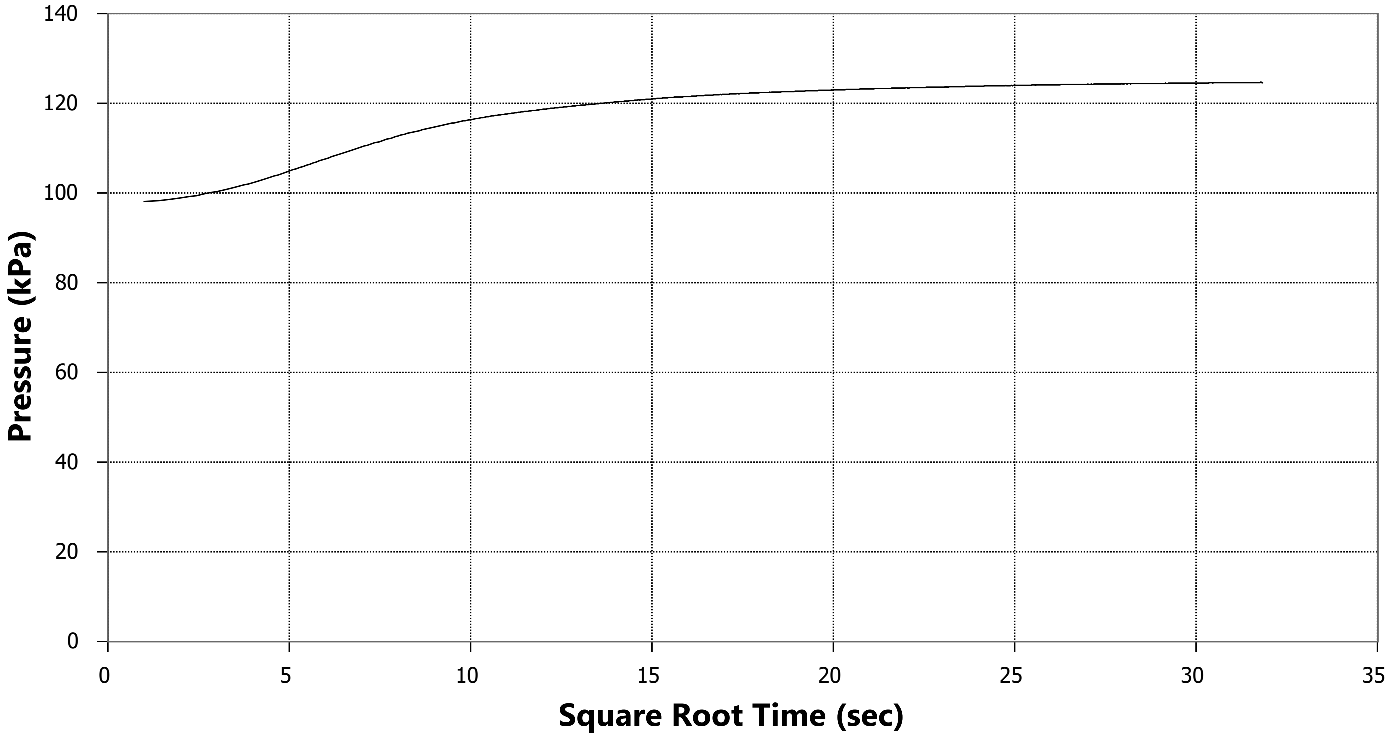


McMILLAN Drilling

DISSIPATION TESTS

PointID: CPTu212

Test Depth: 12.37 **Duration:** 1013 seconds



McMILLAN Drilling

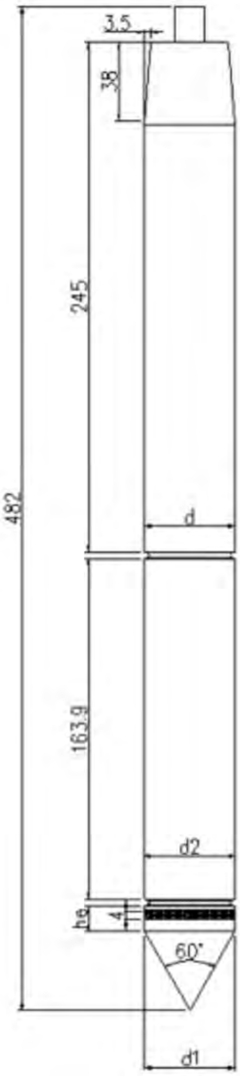
CPT CALIBRATION AND TECHNICAL NOTES

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.

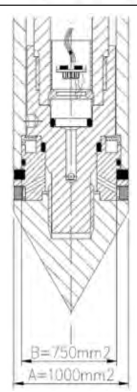
A.P. van den Berg Machinefabriek tel.: +31 (0)513-631355 info@apvandenberg.com	**DEVIATION of Straightness + MINIMUM Dimensions tip, friction jacket, cone adapter**	Standards: EN ISO 22476-1 APB-standard
Type of cone:	Icone 10 cm^2	Icone 15 cm^2
ALLOWABLE SIZE VARIATION		
Diameter of tip:	$35{,}3 \leq d1 \leq 36{,}0$	$43{,}2 \leq d_1 \leq 44{,}1$
Diameter of centering ring CFP	$35{,}3 \leq d1 \leq 36{,}0$	$43{,}2 \leq d_1 \leq 44{,}1$
Diameter of friction jacket:	$d_1 \leq d_2 < d_1 + 0{,}35$	$d_1 \leq d_2 < d_1 + 0{,}43$
Height dimension of tip edge:	$7 \leq h_e \leq 10$	$9 \leq h_e \leq 12$
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:	$d_2 \leq 35{,}65$	$d_2 \leq 43{,}7$
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:

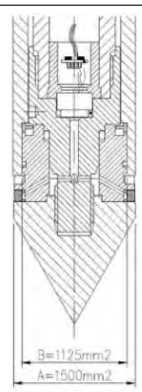
- $10cm^2$ cones: 80mm
- $15cm^2$ cones: 100mm



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.

Certificate version 1.20 Certificate number: 100992-2 Page 1/6

Calibration Certificate

CALIBRATION RvA K 178

a.p. van den berg

1.1 General

Probe number:	111007
Probe type:	I-CFXYP20-10
Description:	Tip 75 MPa Sleeve 1.00 MPa Inclinometer 20° Pore 2MPa
Part number:	0100277B
Certificate number:	111007-5
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 00E80F	Aug 2021 (HBM: 96998 2021-08)
DAQ MX440B 00FCAB	Aug 2021 (HBM: 97005 2021-08)
Loadcell 100kN 201330120	August 2021 (HBM: 96532 2021-08)
Loadcell 20kN 210230193	Aug 2021 (HBM: 96418 2021-08)
Sensor 20 Bar 240310135	Sept 2020 (ZKM: 02-1193 2020-09)
ACS-080-SC00-HP2-PM 02/18 2610439	April 2021 (Trescal: 2103-24005)
Temperature logger: 620-2326 SN: 170800285	June 2021 (AVANTOR: 219003177)

1.3 Laboratory conditions:

Ambient temperature: 22.5 ± 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:	22 February 2022
Signature:	
QA Manager:	N.R.E. de Jong
Date:	22 February 2022
Signature:	

Expiration date according to EN ISO 22476-1: 23 August 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 – Blocks 2 - 4, 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
- C. Ruegg (MSc), Senior Geophysicist
- A. Aspinwall (MASt), Geophysical Technician
- T. Vanderkley (BSc), Geophysical 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 1 (Issued May 26 2022)

Internally reviewed by:
M. Finnemore (PhD), Senior Geophysicist

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 Blocks 2 to 4 of Stage 3, Beach Grove Subdivision, Kaiapoi. The MASW was undertaken from April 11th to 13th and the GPR on April 26th, 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 lines have been investigated. If more detailed information on any part of the site is required, additional geophysical investigations could be conducted with closer line spacing.

Methodology:

Site Description:

The terrain was undeveloped grass paddocks with livestock fencing, power lines and gravel driveways (Figures 1 and 10). Some areas were saturated with large standing bodies of water and muddy areas. The site had no significant topography and weather conditions were fine, some occasional light rain, 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 lines 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 hammer 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 to 9). 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 Blocks 2 to 4, a shielded GSSI 200 MHz HyperStacking® GPR system was used.

The GPR acquisition parameters used at Beach Grove Blocks 2 to 4, Kaiapoi were:

- Antenna frequency – 200 MHz
- Trace increment – 2.5 cm
- Sample per trace - 1024
- Time increment - 0.3759 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 0.045 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 11 to 13. 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 lines have not been corrected for elevation.

Results:

A total of eight MASW lines and 63 GPR lines were acquired at the site, with a total MASW survey length of approximately 3,000 m (Figure 1) and a total GPR survey length of 3175 m (Figure 10).

The MASW results were correlated against Cone Penetrometer Test (CPT) logs provided by Tonkin & Taylor, CPTs within 20 m to the MASW survey lines were plotted on the site map and MASW profiles. CPTs farther than 10 m from the MASW survey lines should be interpreted with caution. The depth at which most CPT's refuse correlates with approximately 220 m/s to 260 m/s shear-wave velocity (Figures 2 to 9).

The GPR data were of average quality and imaged from the subsurface to 4 m depth (Figures 11 to 13). Some larger buried objects were seen in the GPR radargrams up to 2 m depth. Some channel features were seen in radargrams; however, these features were difficult to interpolate due to the large distance between lines. 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 or similar organic material e.g. logs.

Regions of near low shear-wave velocity (< 100 m/s) in the top 3 m are plotted against the GPR (Figure 10). Some of the low shear-wave velocity areas, especially in the south and west survey areas, correlated well with anomalous areas found in the GPR, reinforcing the indication of these areas being infilled with peat. Low shear-wave velocities that did not overlap with GPR features and where the GPR had poor penetration could be indicative of clay-rich sediments.

MASW Limitations:

The MASW survey was considered to be of good quality, with modelled shear-wave velocities to 15 to 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 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.

[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.

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: Blocks 2 - 4, MASW Survey Site Map**	NOTES -	Coordinates projected in NZTM2000. Aerial photograph sourced from LINZ, Crown Copyright © <u>CPT & Borehole</u> CPT locations labelled by cyan triangles <u>MASW</u> MASW lines marked with red circles, chainage marked at every 6th shot point
LOCATION -	**Beach Grove, Kaiapoi**		

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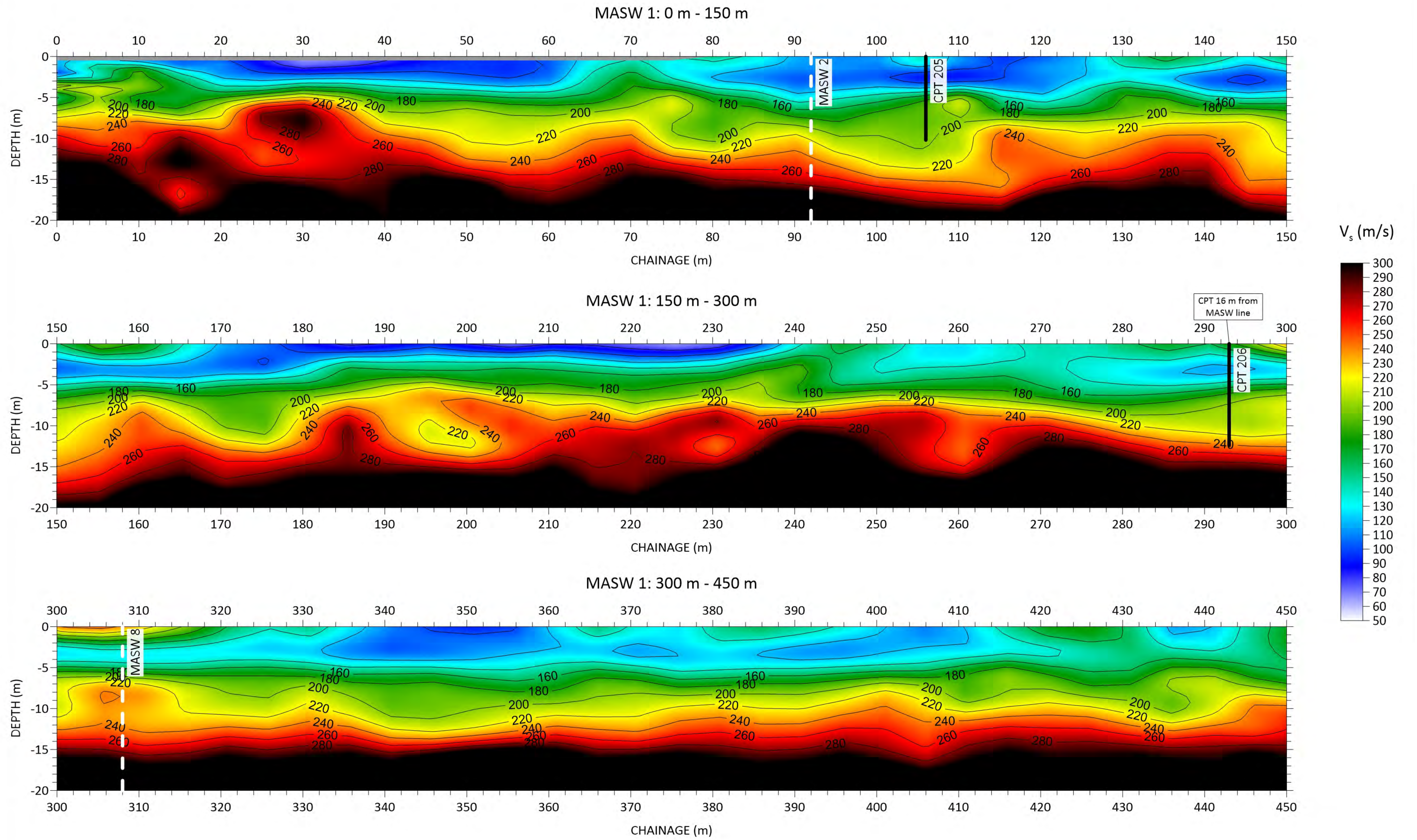

DRAWING -	**Figure 2: Blocks 2 - 4, MASW 2D V_s Profiles Line 1**
LOCATION -	**Beach Grove, Kaiapoi**

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.

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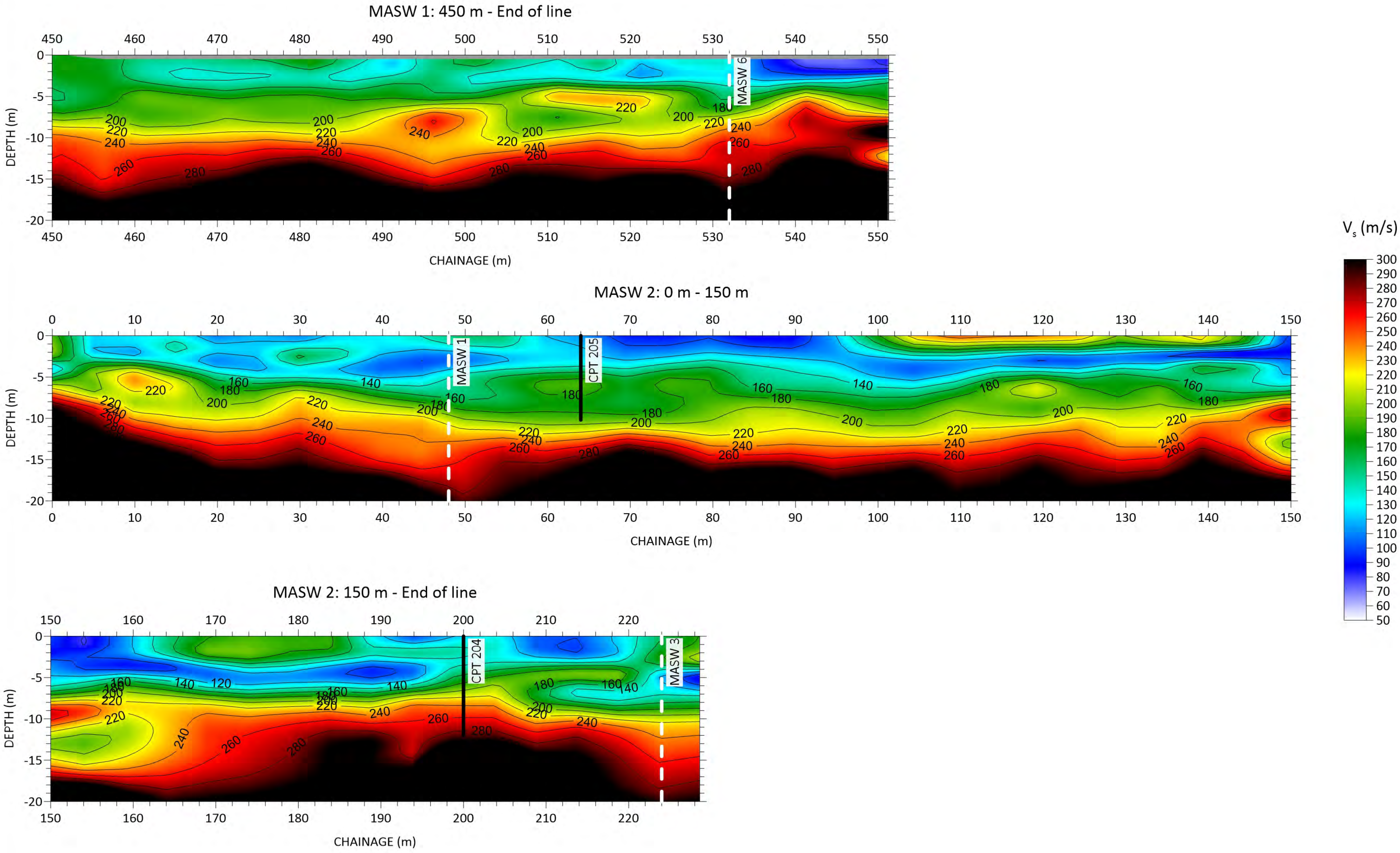

DRAWING - **Figure 3: Blocks 2 - 4, MASW 2D V_s Profiles Line 1 cont'd and Line 2**	NOTES - MASW V_s profile has contour intervals of 20 m/s (V_s). CPTs are drawn to refusal depths provided by Tonkin & Taylor. See site map for location of points.	A3
LOCATION - **Beach Grove, Kaiapoi**		

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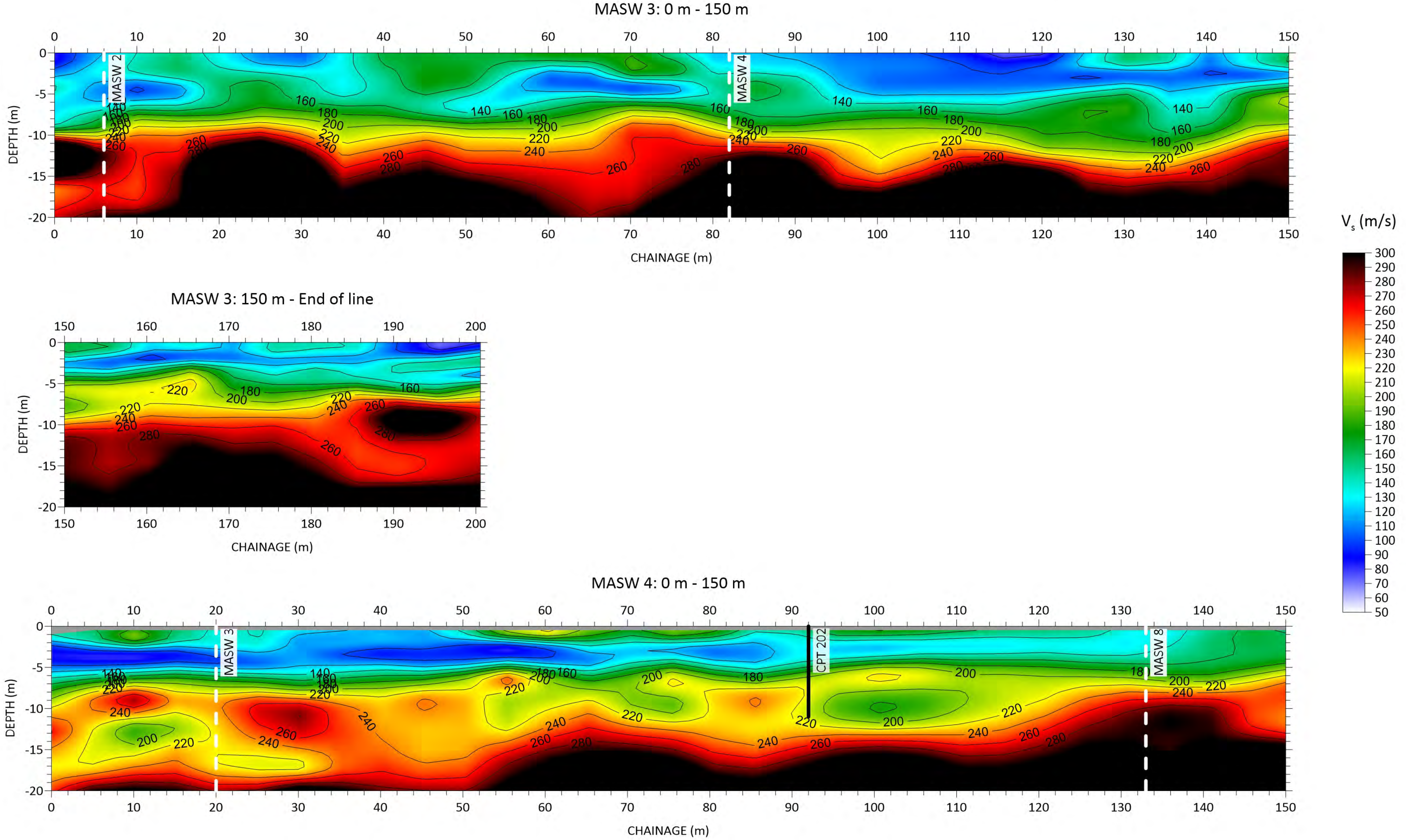

DRAWING - **Figure 4: Blocks 2 - 4, MASW 2D V_s Profiles Line 3 and Line 4**	NOTES - MASW V_s profile has contour intervals of 20 m/s (V_s). CPTs are drawn to refusal depths provided by Tonkin & Taylor. See site map for location of points.
LOCATION - **Beach Grove, Kaiapoi**	A3

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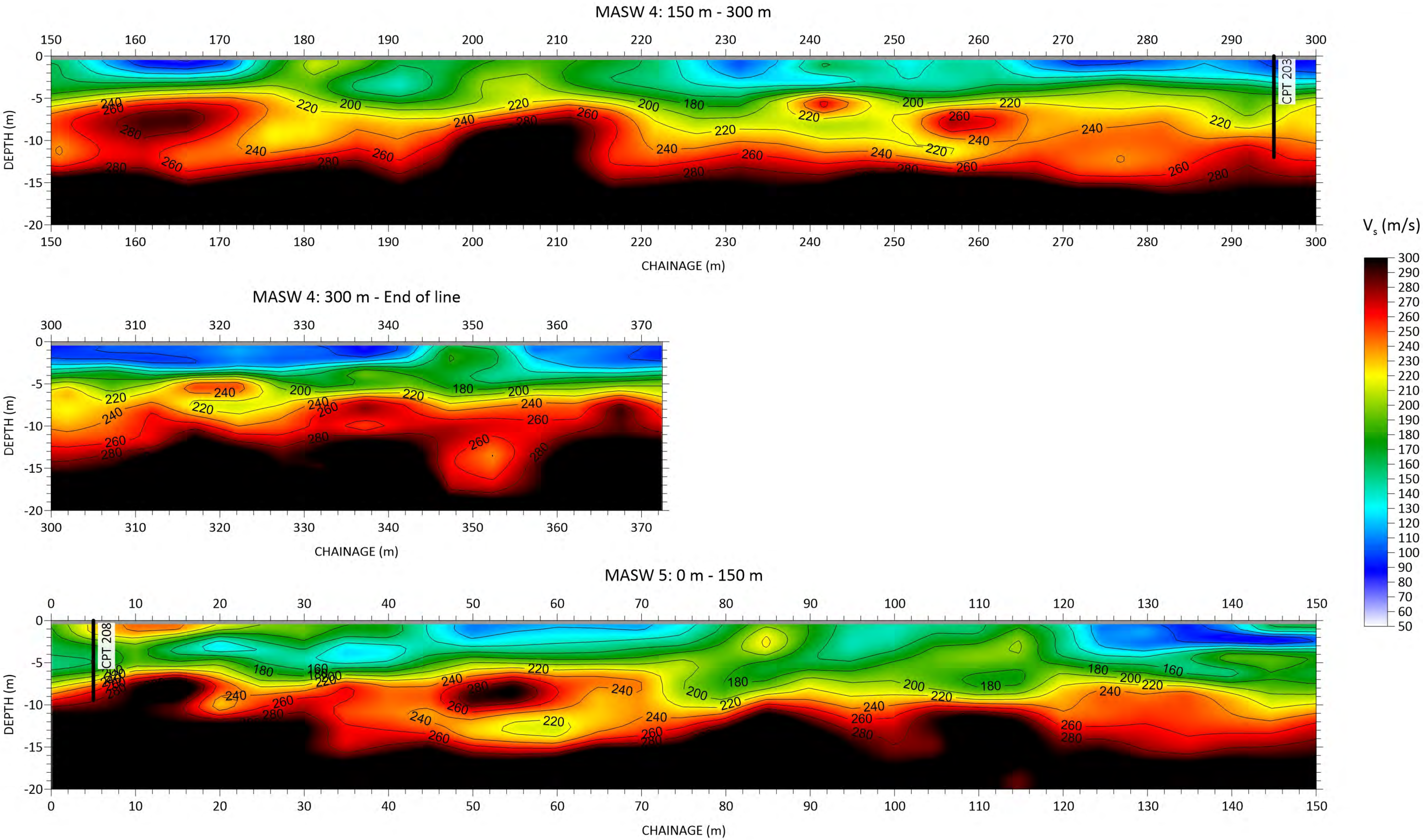

DRAWING - **Figure 5: Blocks 2 - 4, MASW 2D V_s Profiles Line 4 cont'd and Line 5**

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.

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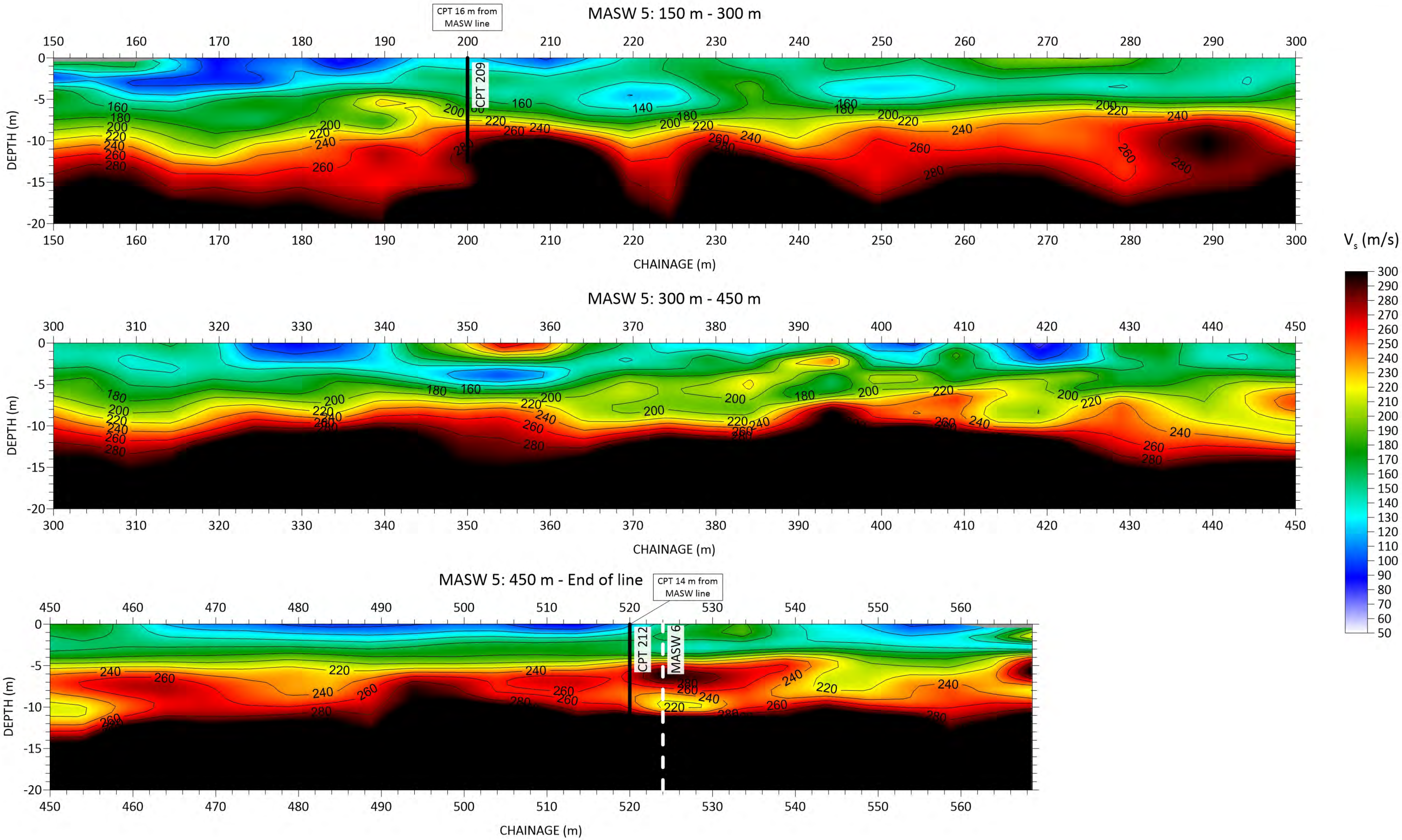

DRAWING - **Figure 6: Blocks 2 - 4, MASW 2D V_s Profile Line 5 cont'd**

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.

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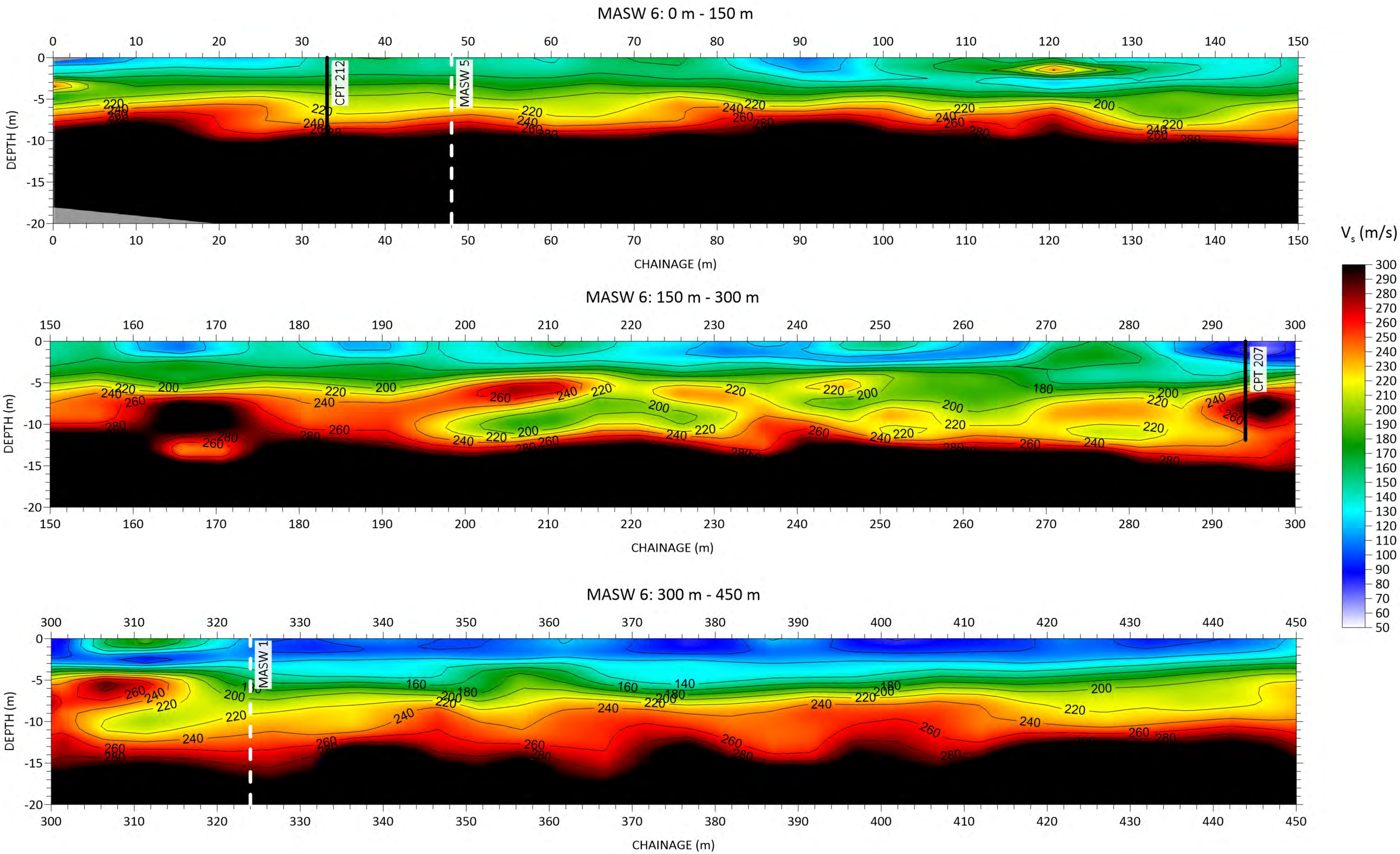

DRAWING - **Figure 7: Blocks 2- 4, MASW 2D V_s Profile 6 Line 6**

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.

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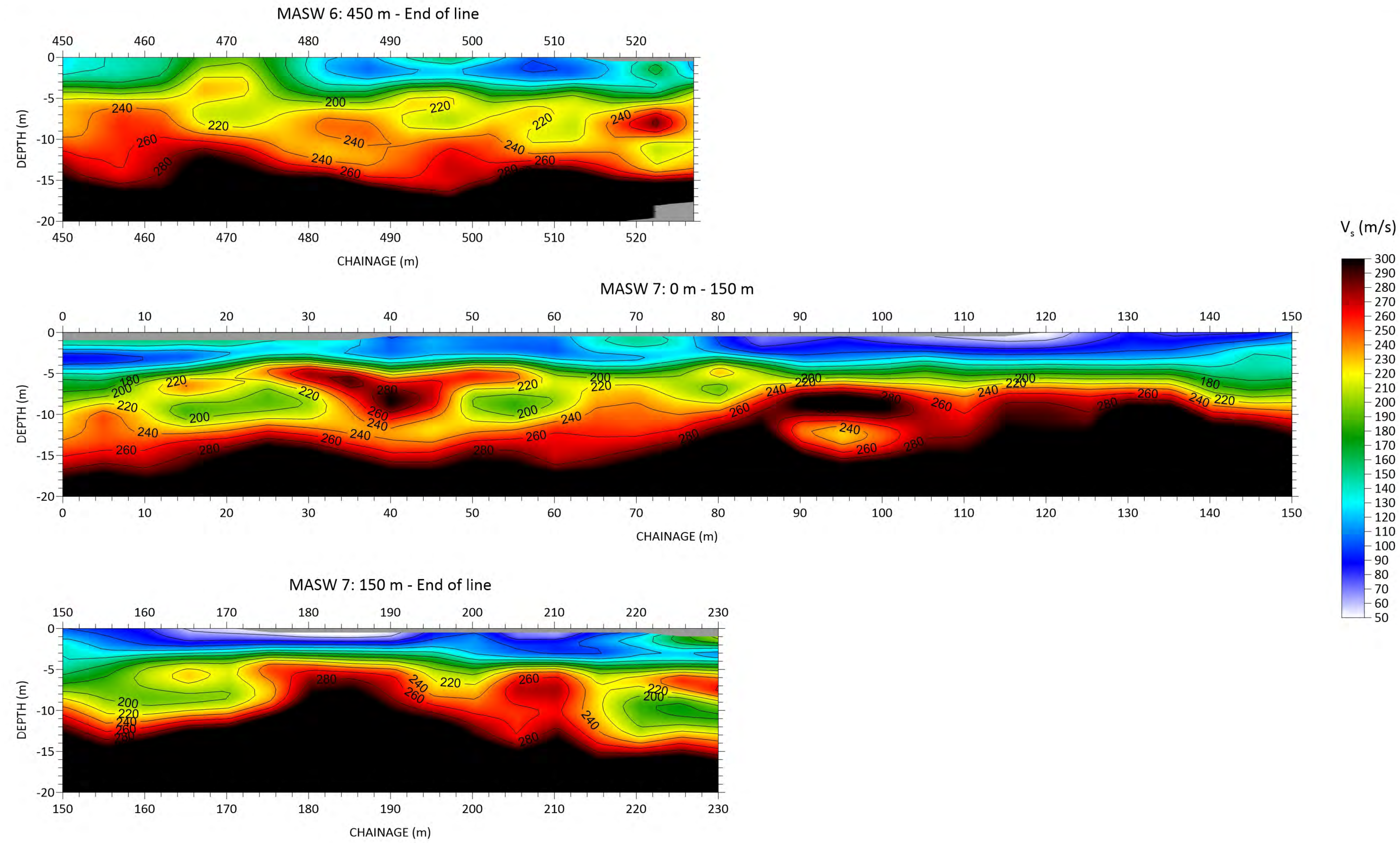

DRAWING -	**Figure 8: Blocks 2 - 4, MASW 2D V_s Profiles 6 & 7 Line 6 cont'd and Line 7**
LOCATION -	**Beach Grove, Kaiapoi**

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.

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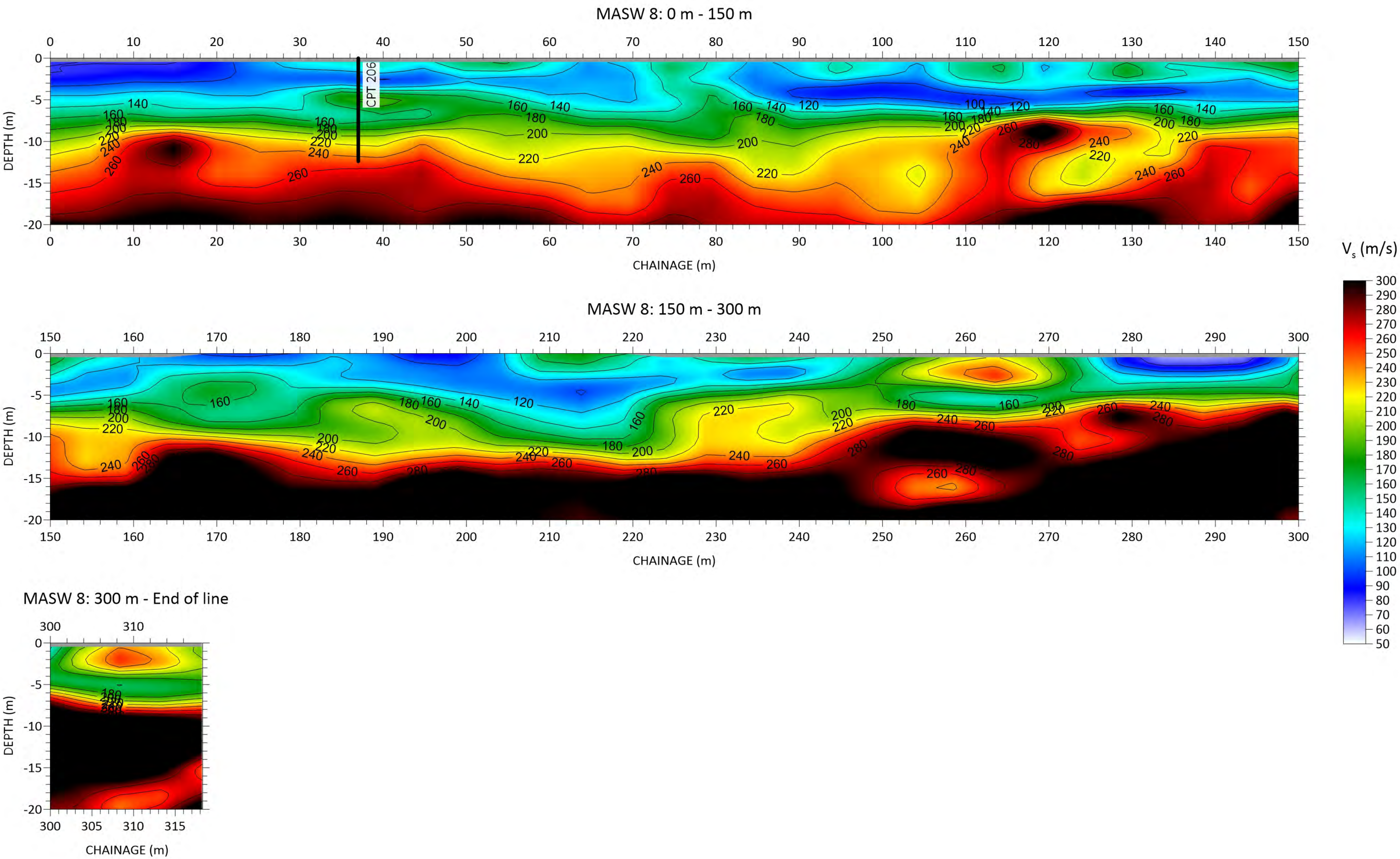

DRAWING - **Figure 9: Blocks 2 - 4, MASW 2D V_s Profile 8 Line 8**

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.

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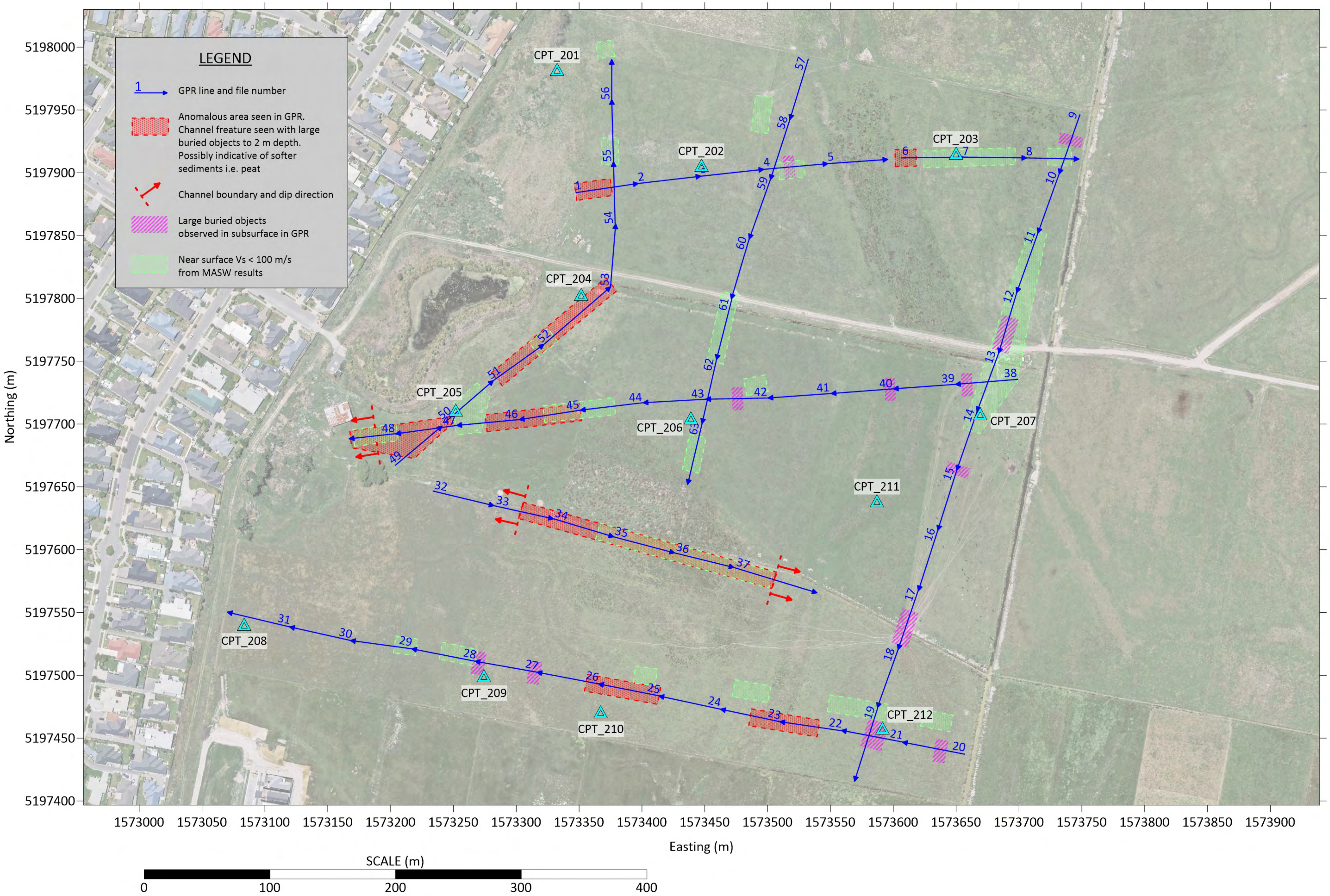

DRAWING - **Figure 10: Blocks 2 - 4, GPR Survey Site Map**

LOCATION - **Beach Grove, Kaiapoi**

NOTES -
Coordinates projected in NZTM2000.
Aerial photograph sourced from LINZ, Crown Copyright ©
CPT & Borehole
CPT locations labelled by cyan triangles
GPR
GPR lines marked by blue arrows, and are labelled at the start.

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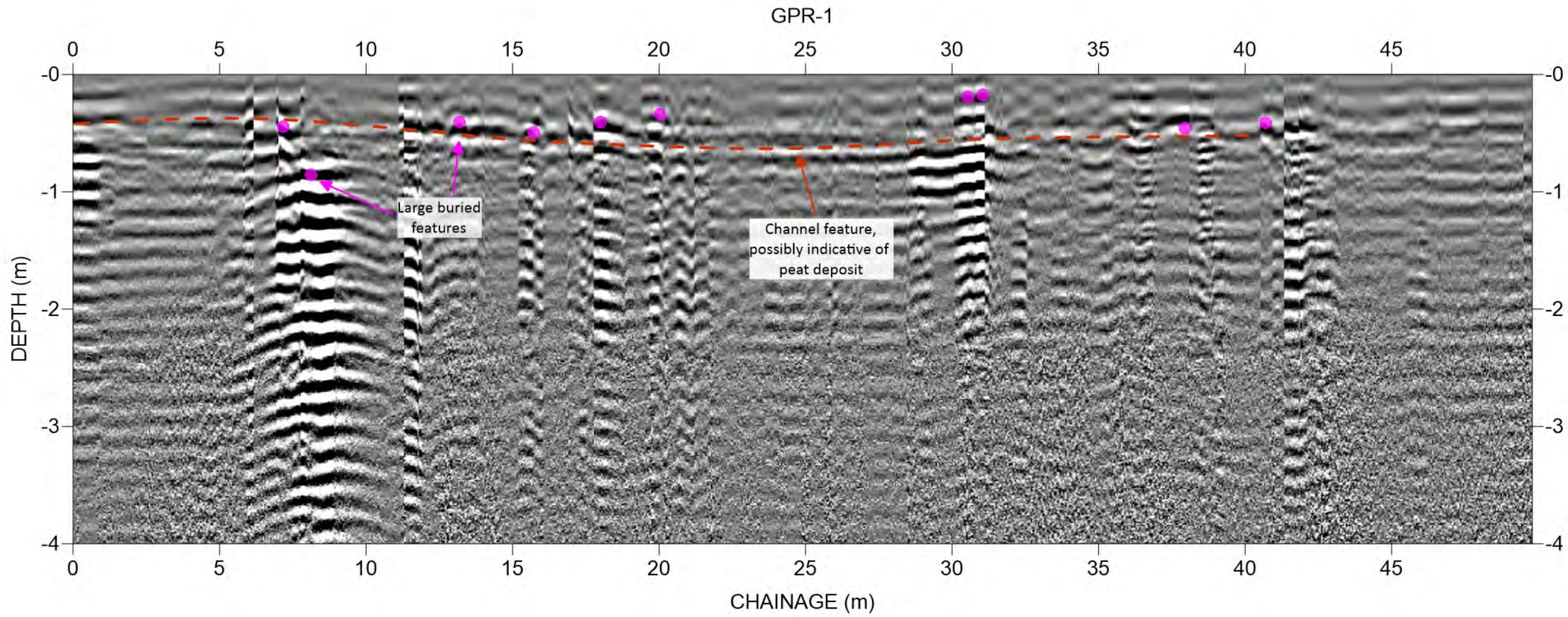

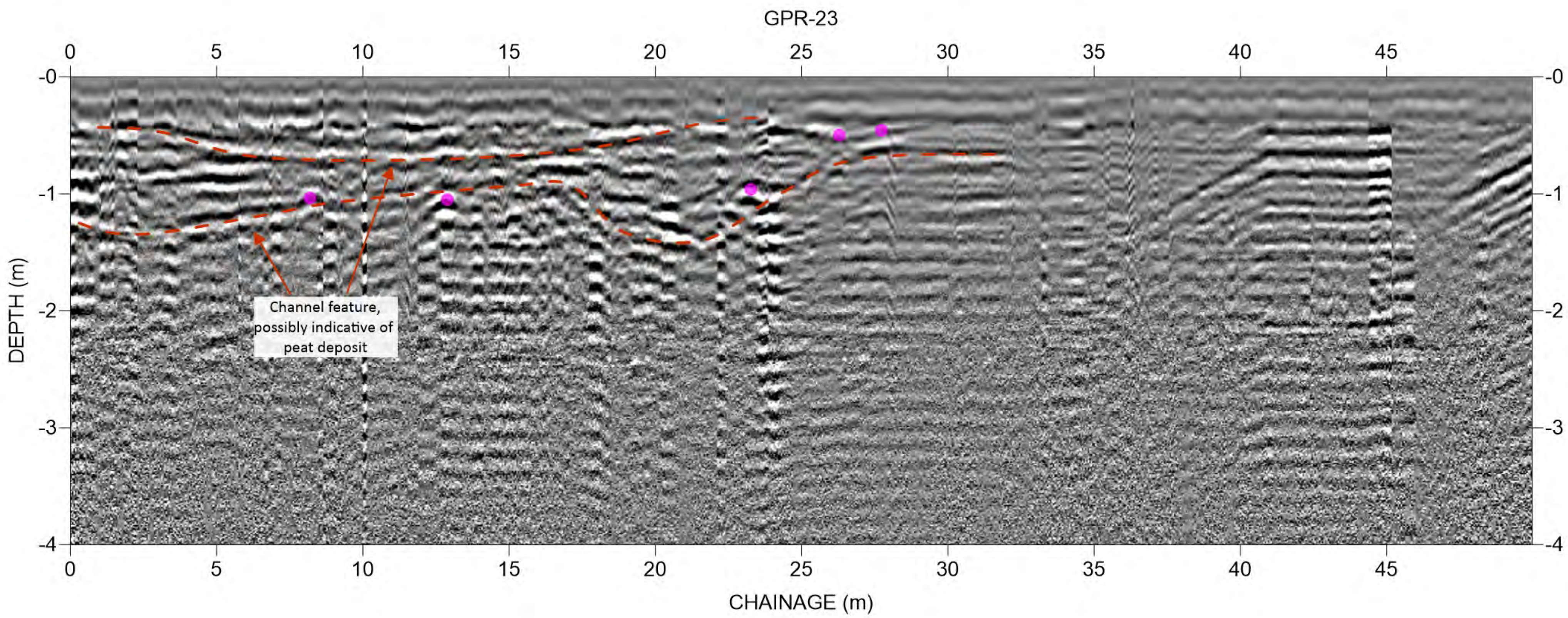

DRAWING -	**Figure 11: Blocks 2 - 4 Example Radargrams Lines 1 and 23**
LOCATION -	**Beach Grove, Kaiapoi**

NOTES -

The image scales are:
GPR: 4:1

See site map for location of lines.

A3

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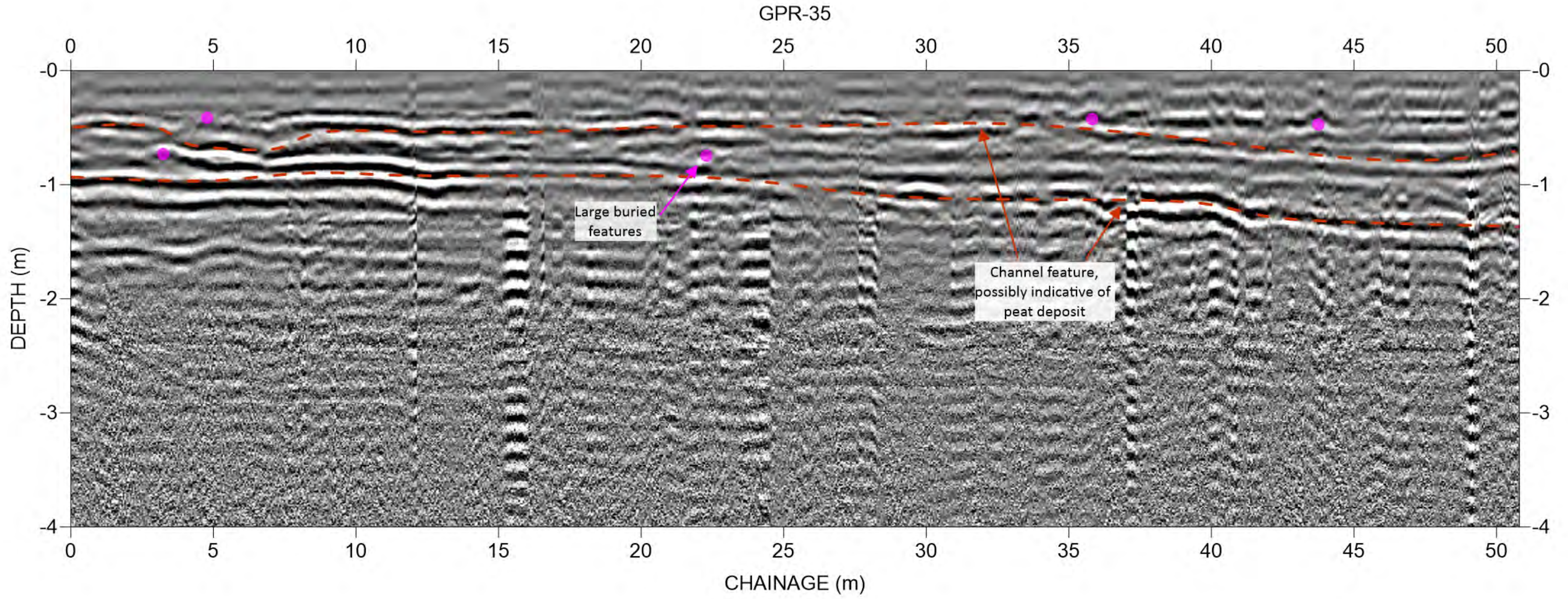

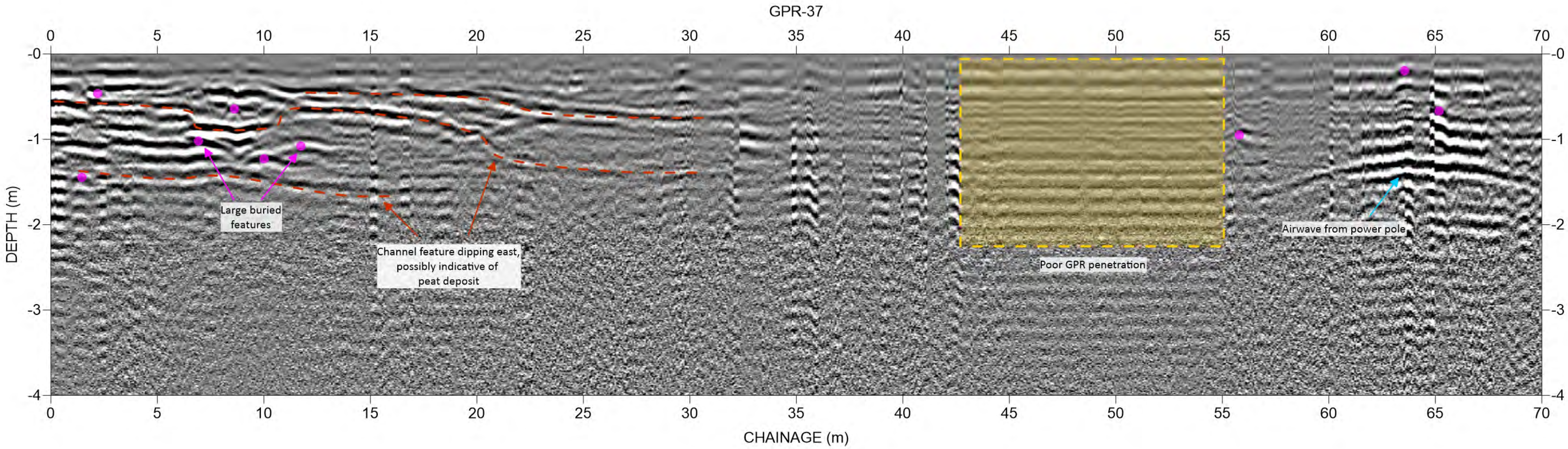

DRAWING - **Figure 12: Blocks 2 - 4 Example Radargrams Lines 35 and 37**

LOCATION - **Beach Grove, Kaiapoi**

NOTES -

The image scales are:
GPR: 4:1

See site map for location of lines.

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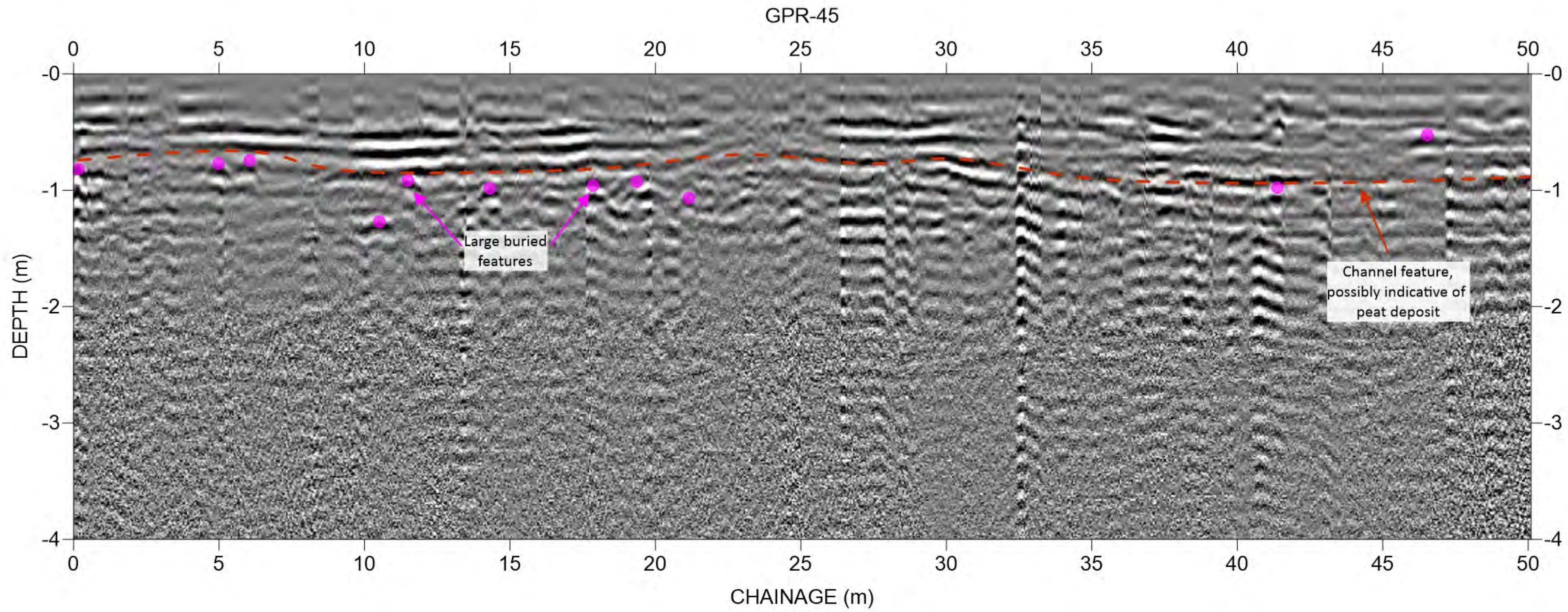
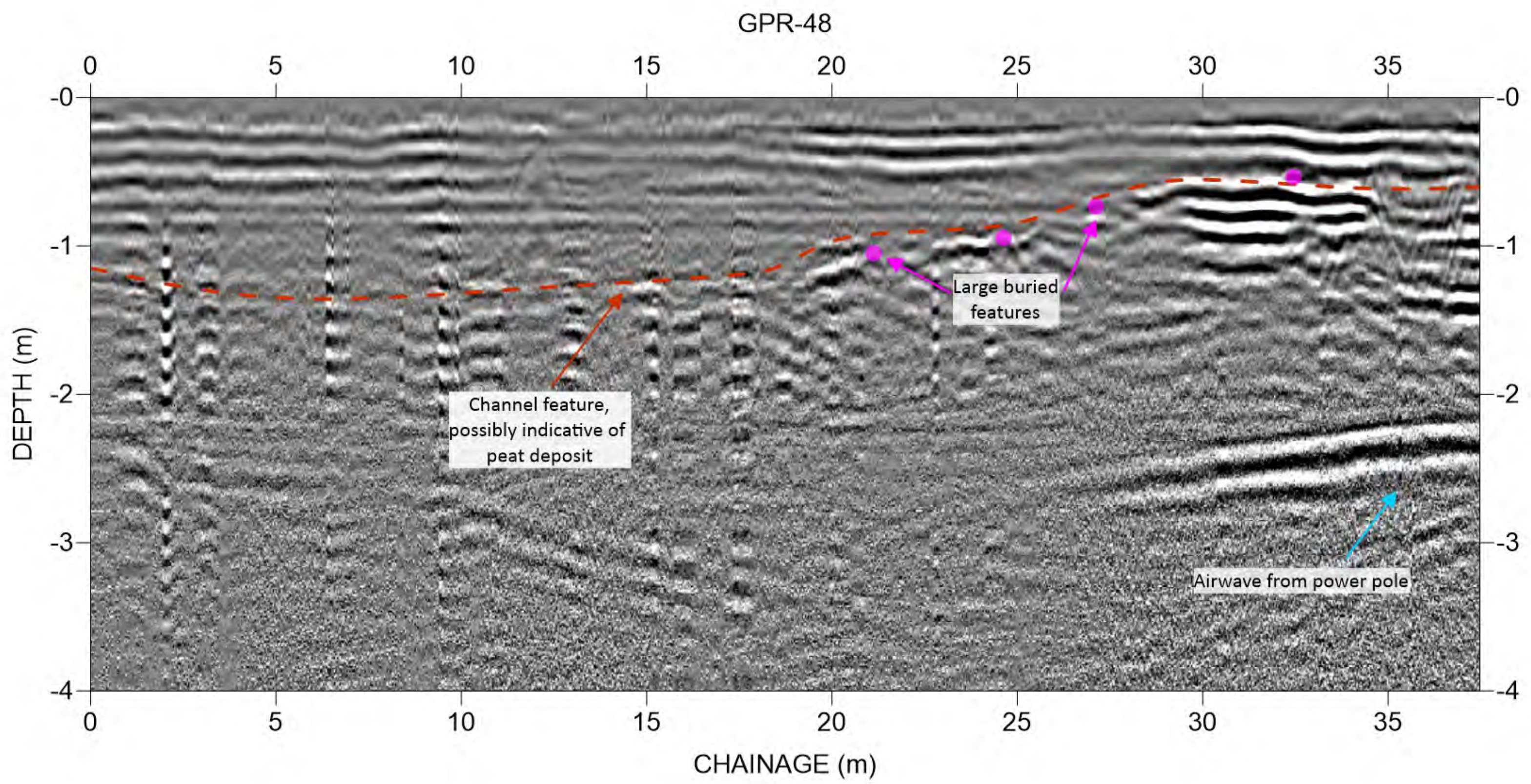
Southern Geophysical
www.southerngeophysical.com





DRAWING - **Figure 13: Blocks 2-4 Example Radargrams Lines 45 and 48**

LOCATION - **Beach Grove, Kaiapoi**

NOTES -

The image scales are:
GPR: 4:1

See site map for location of lines.

A3

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Appendix A: Field Photographs

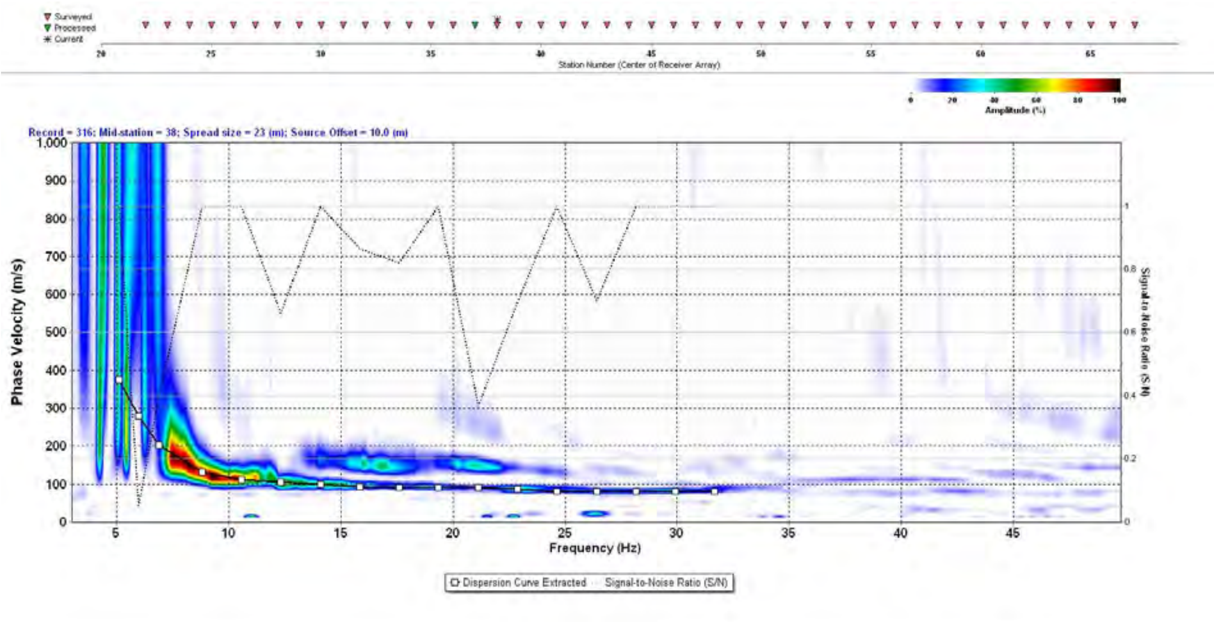
MASW data acquisition on Line 2.

MASW data acquisition during swampy hand-towed section of Line 4.

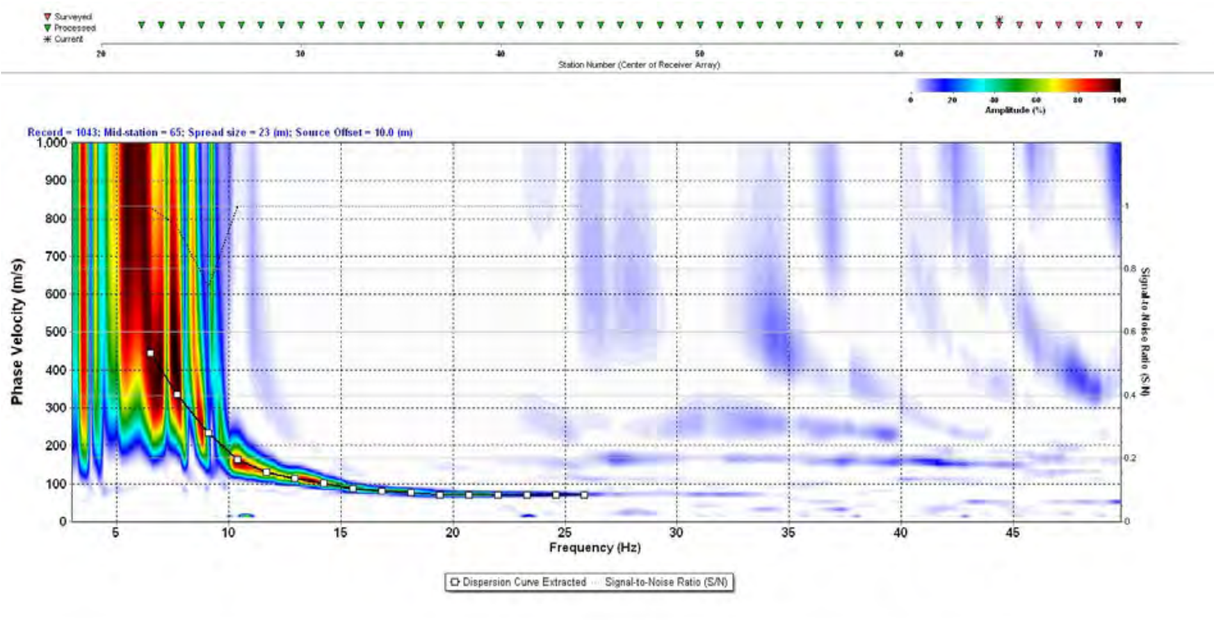
GPS acquisition on northern end of Line 6. MASW line passing over small saturated surface depression.

Muddy patch on Line 7.

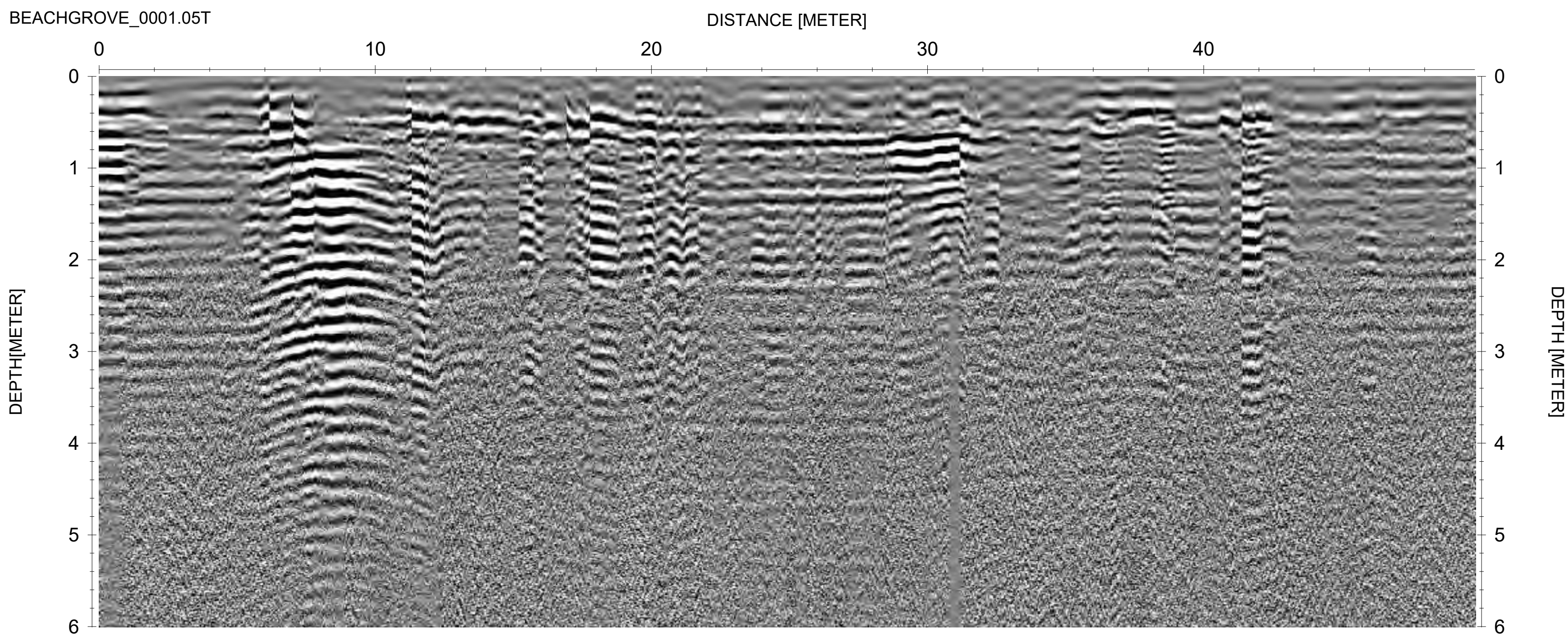
Appendix B: MASW Dispersion Curve Examples



MASW dispersion curve pick from Line 2, chainage 82 m.



MASW dispersion curve pick from Line 7, chainage 193 m.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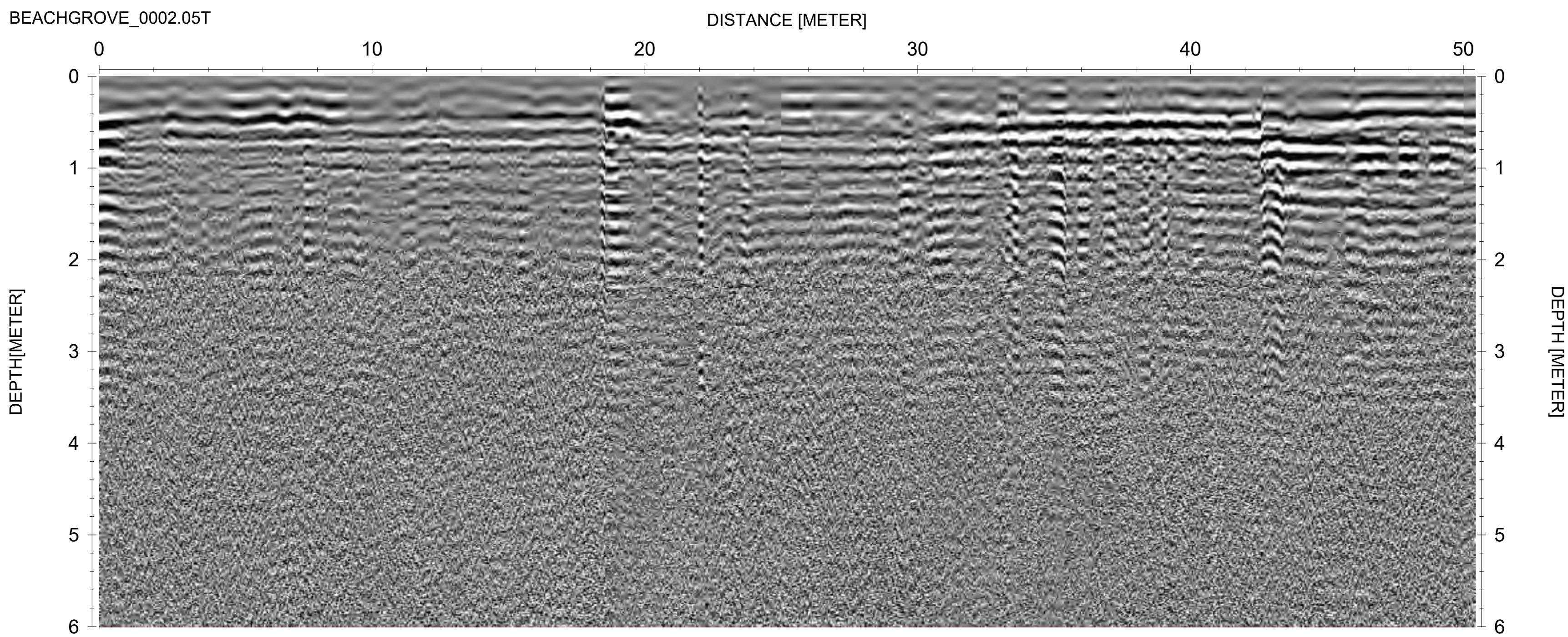
DISTANCE [METER]
0
10
20
30
40
DEPTH[METER]
0
1
2
3
4
5
6
DEPTH [METER]
0
1
2
3
4
5
6



BEACHGROVE_0002.05T
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DEPTH[METER]
DEPTH [METER]
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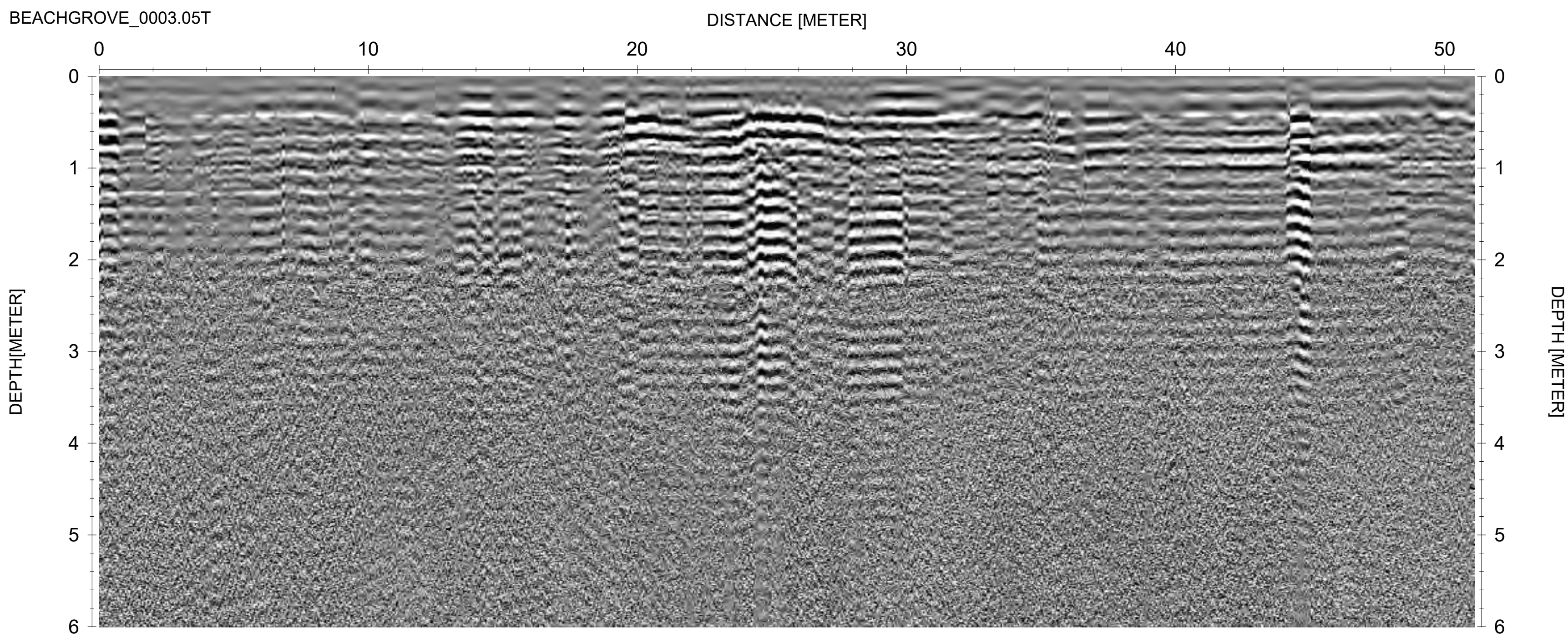
3
4
5
6



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DEPTH[METER]
DEPTH [METER]
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3
4
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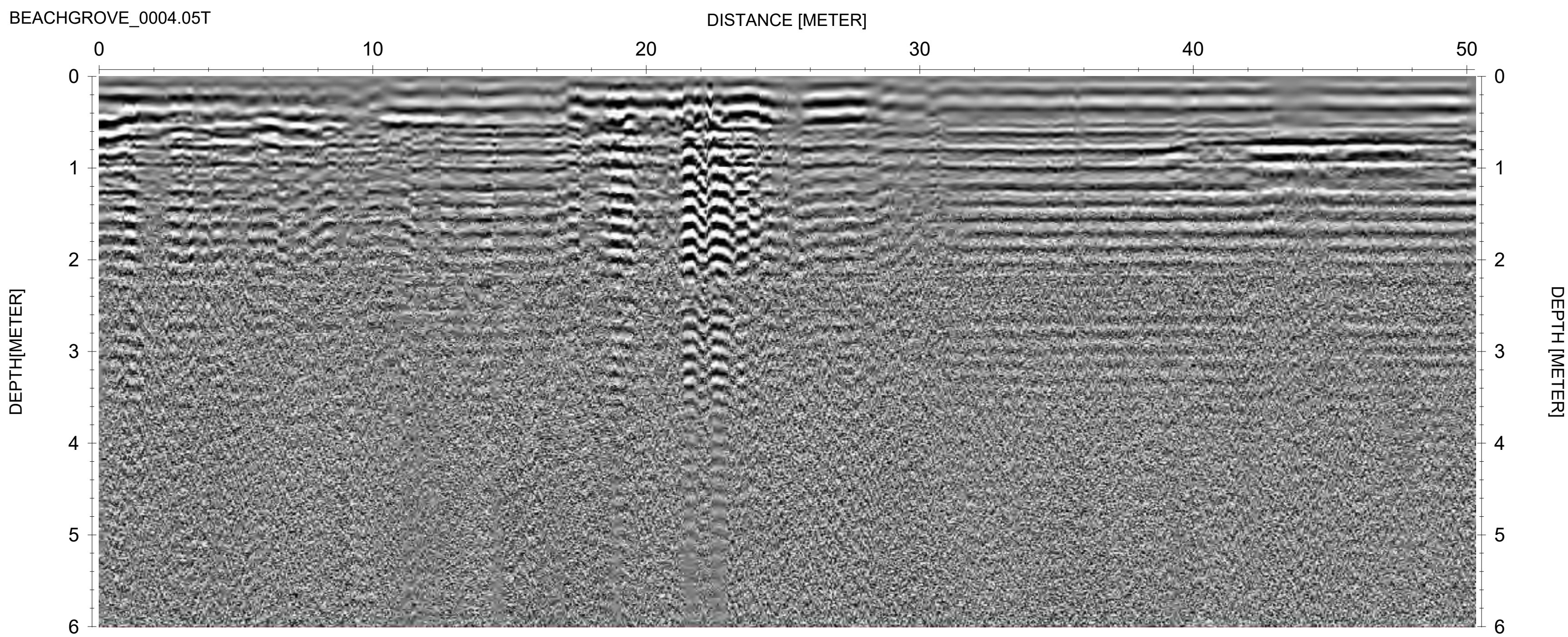
6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DEPTH [METER]
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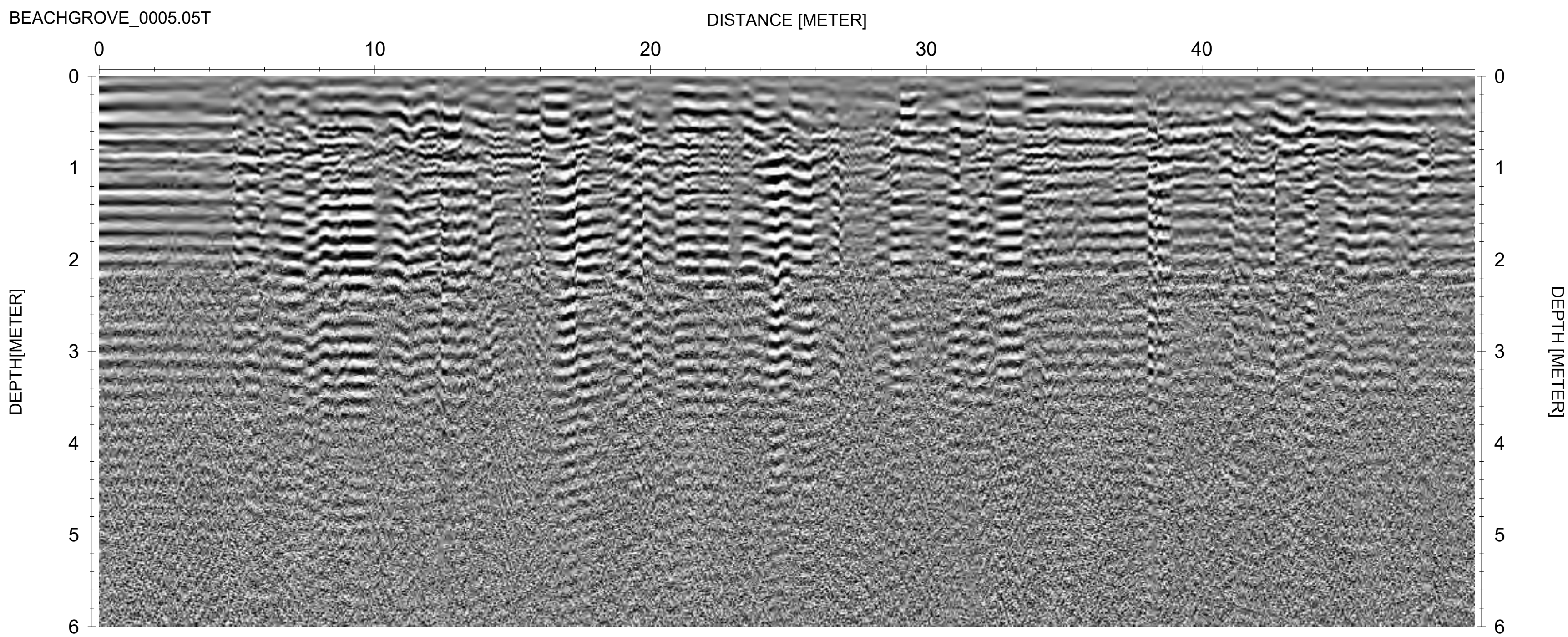
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DEPTH[METER]
DEPTH [METER]
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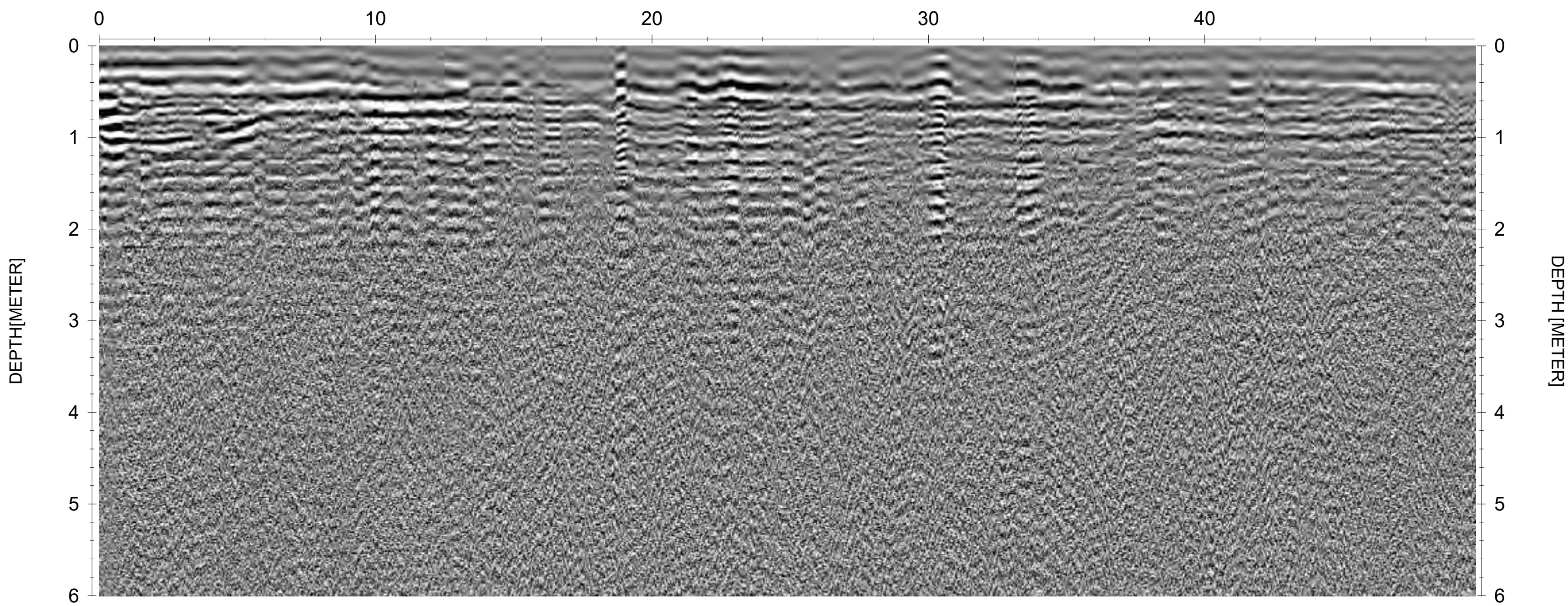
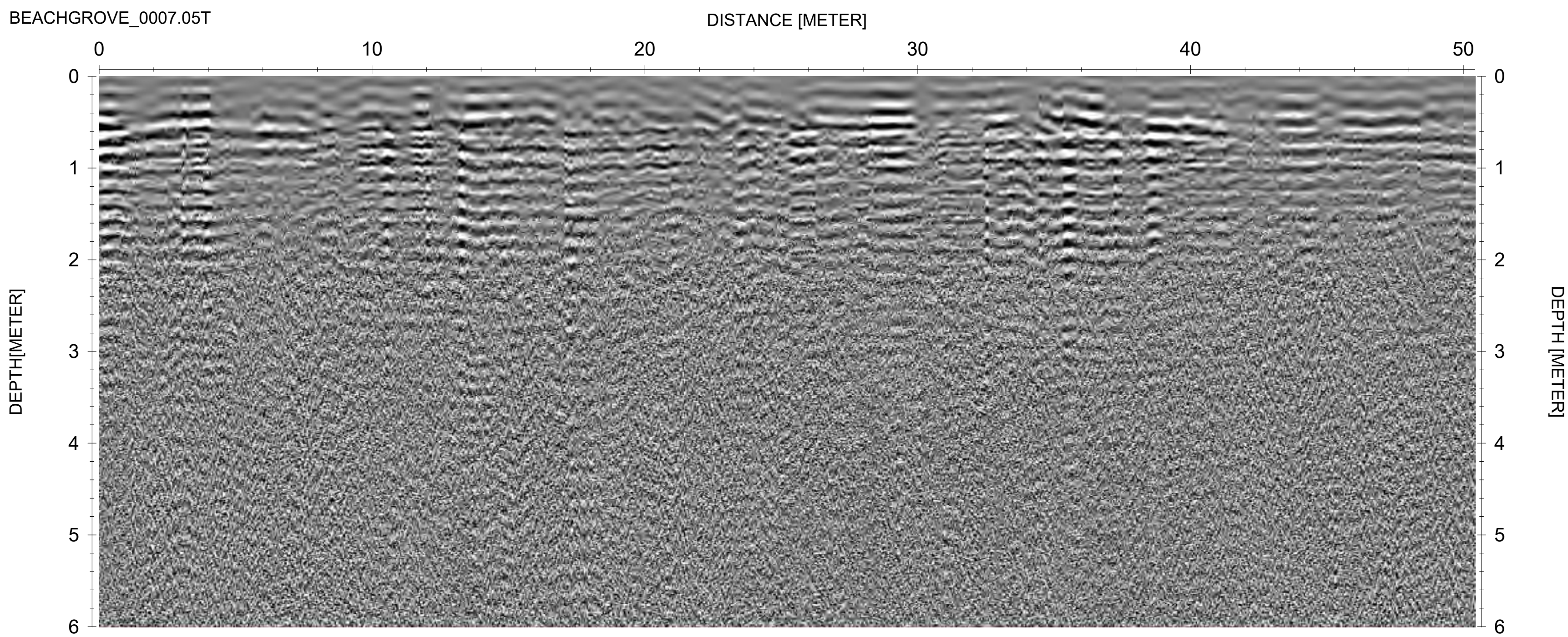
6

BEACHGROVE_0006.05T

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DEPTH[METER]
DEPTH [METER]
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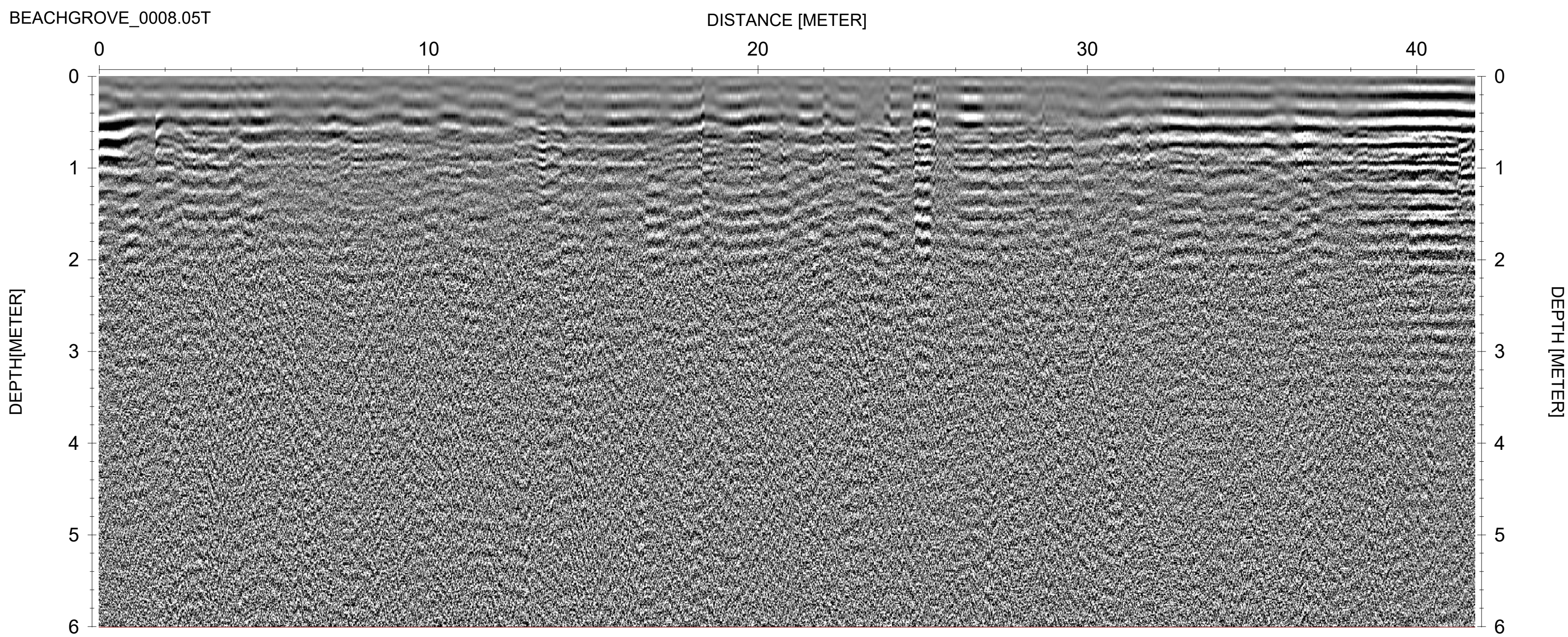
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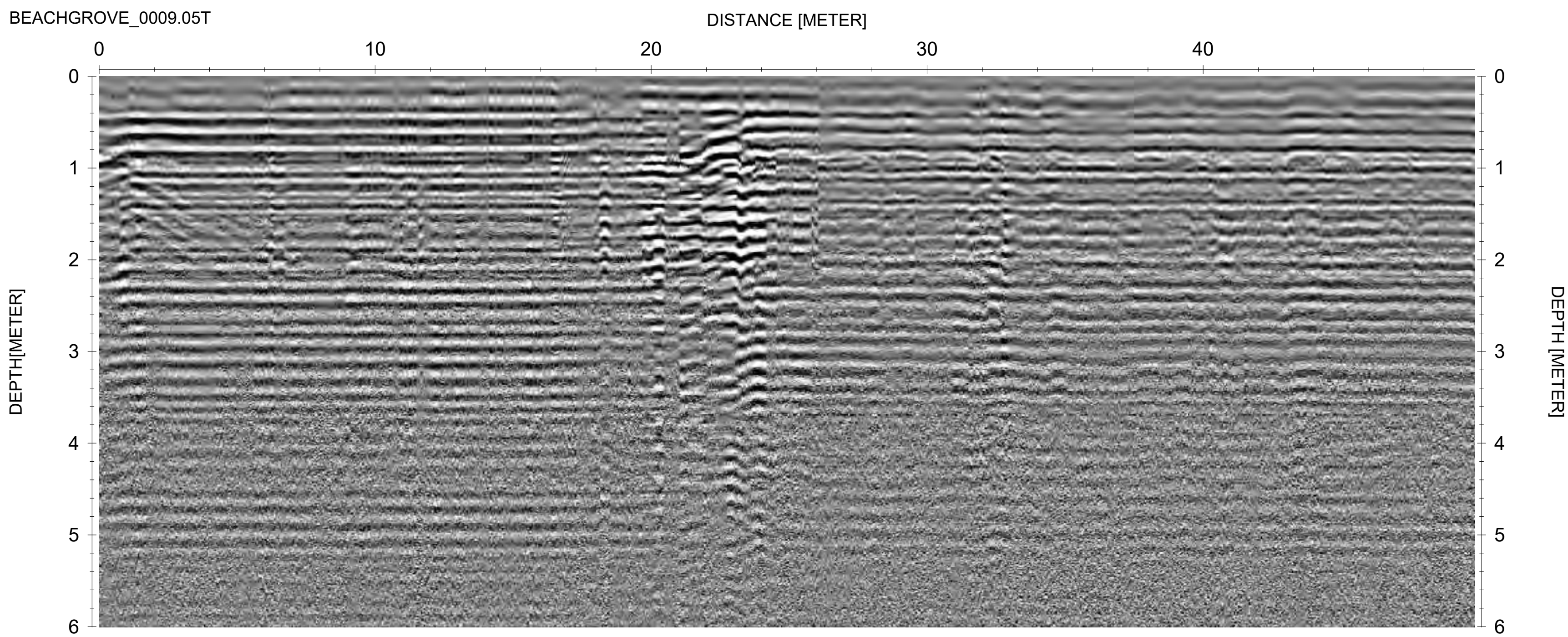
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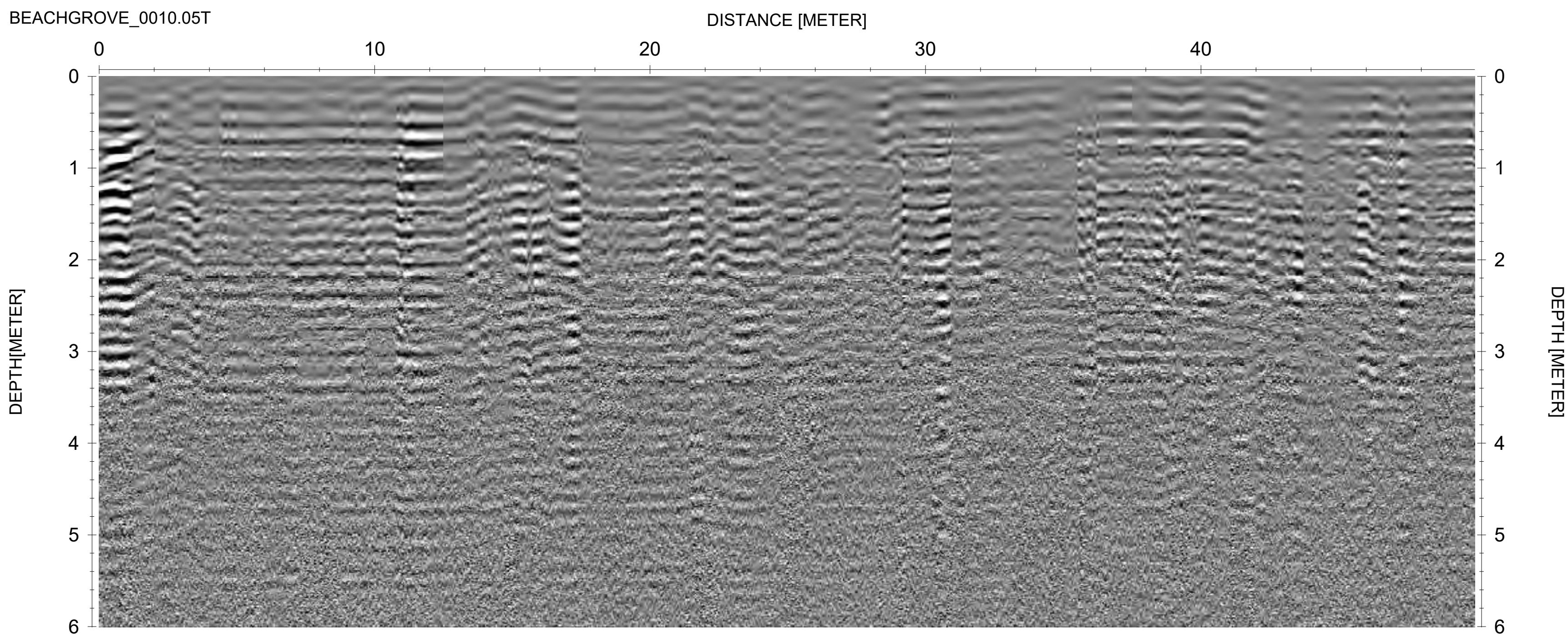

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DEPTH [METER]
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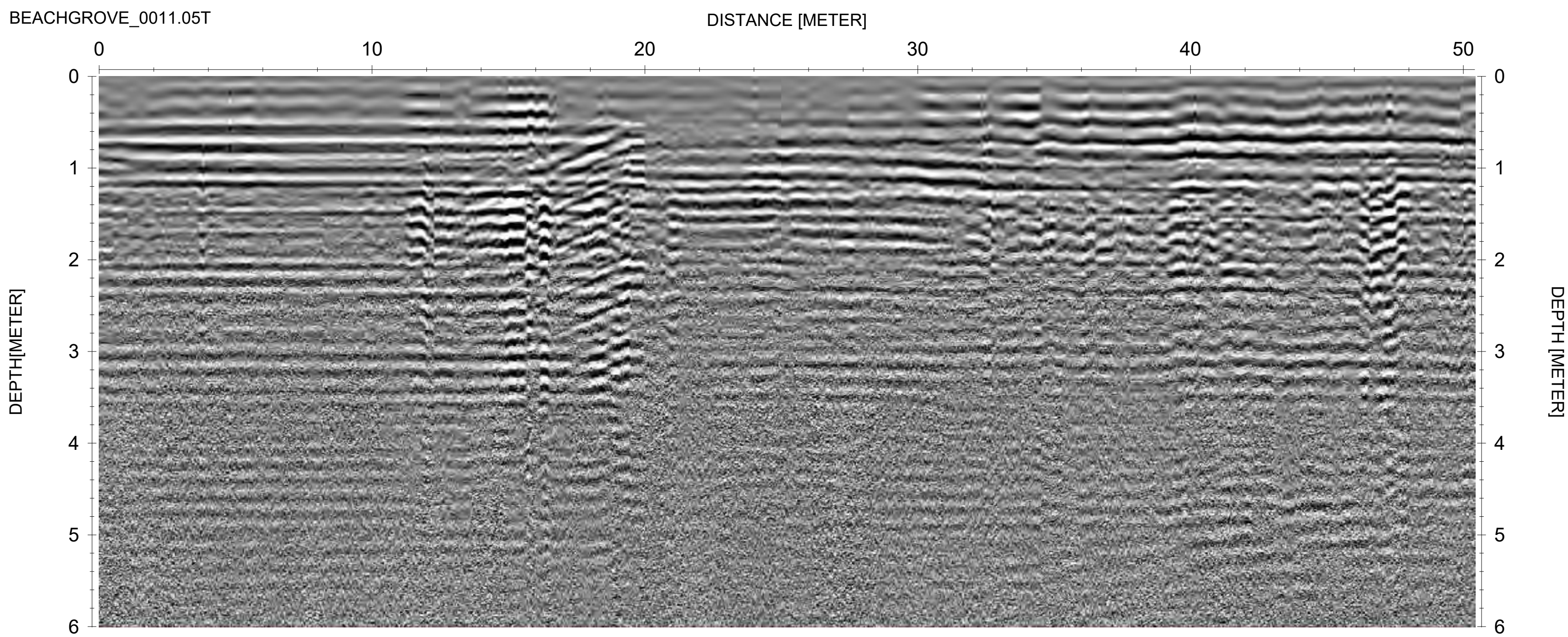
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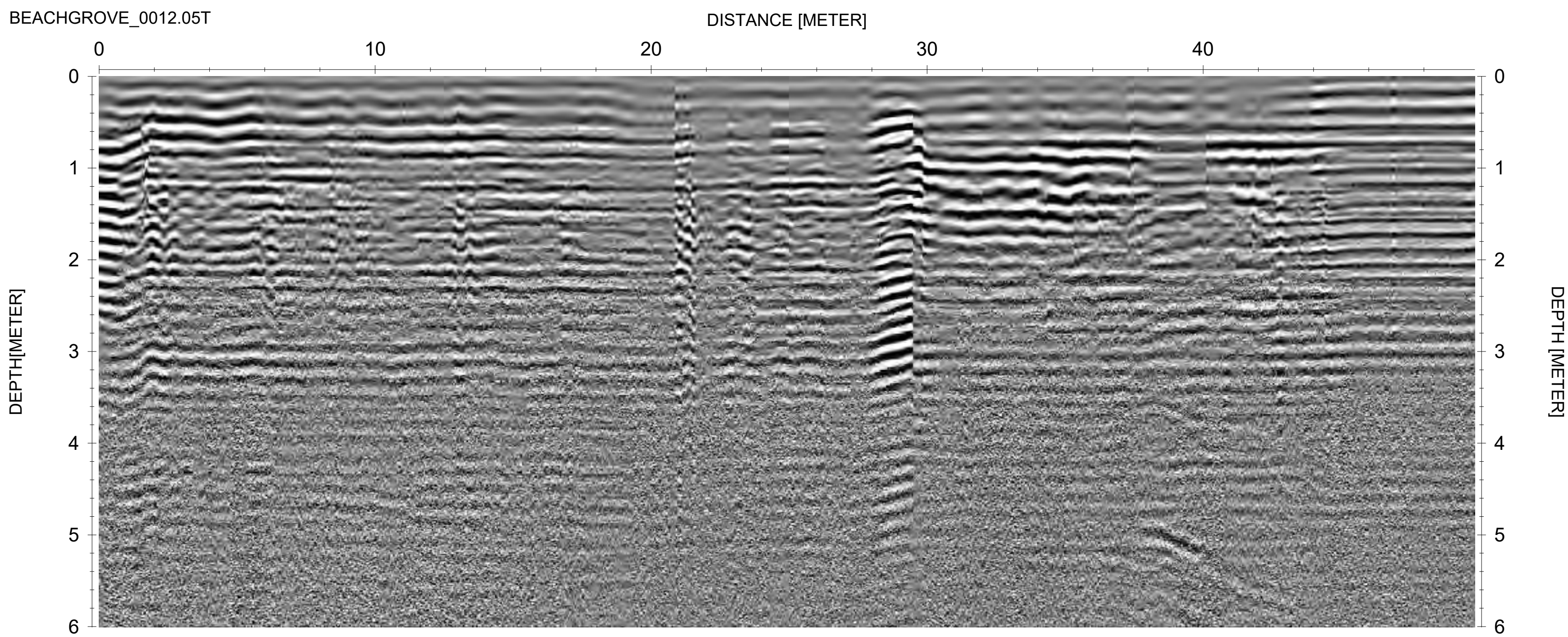
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DEPTH [METER]
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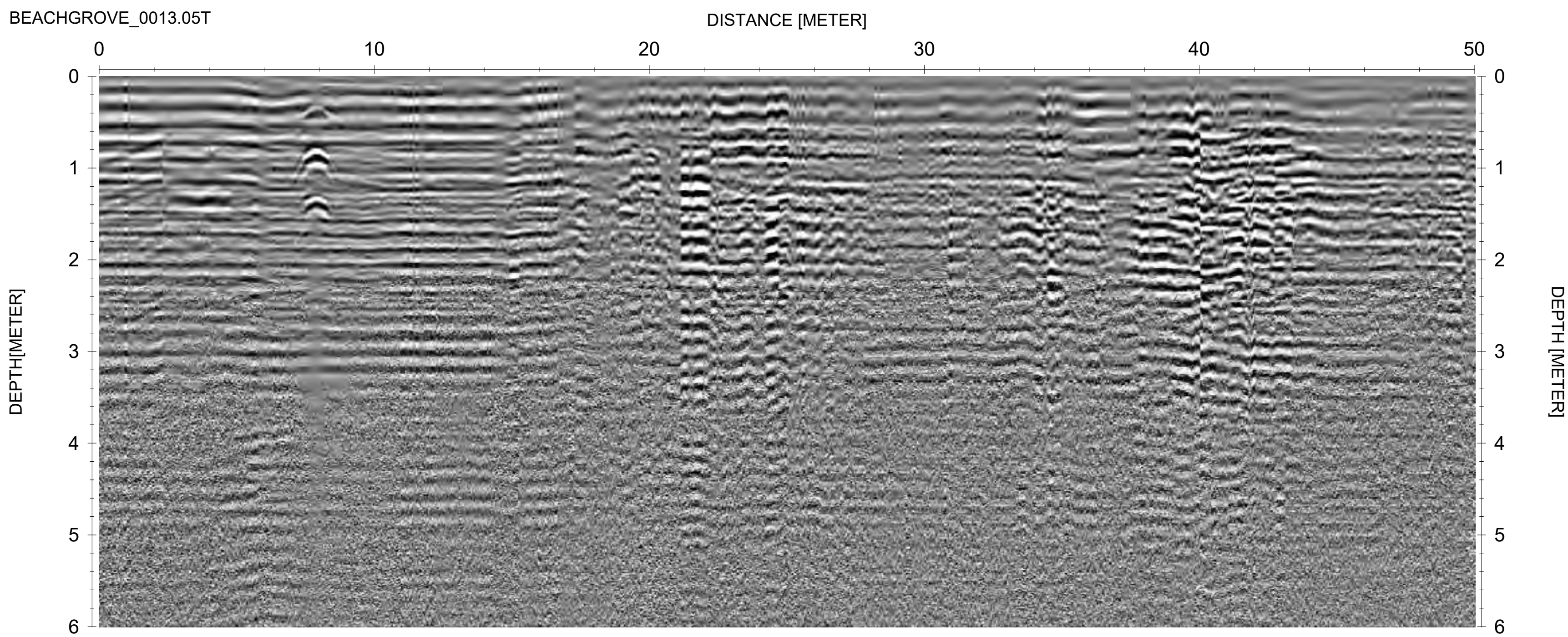
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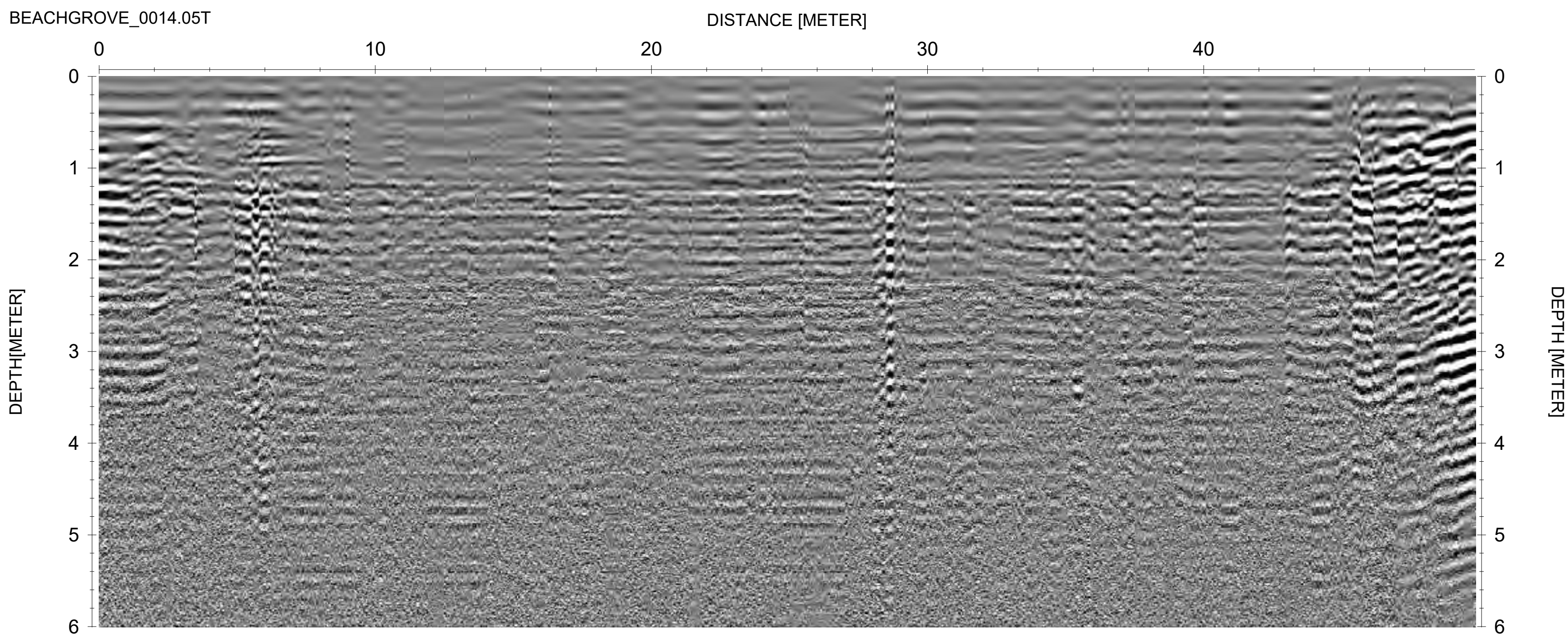
6
DEPTH [METER]



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DEPTH [METER]
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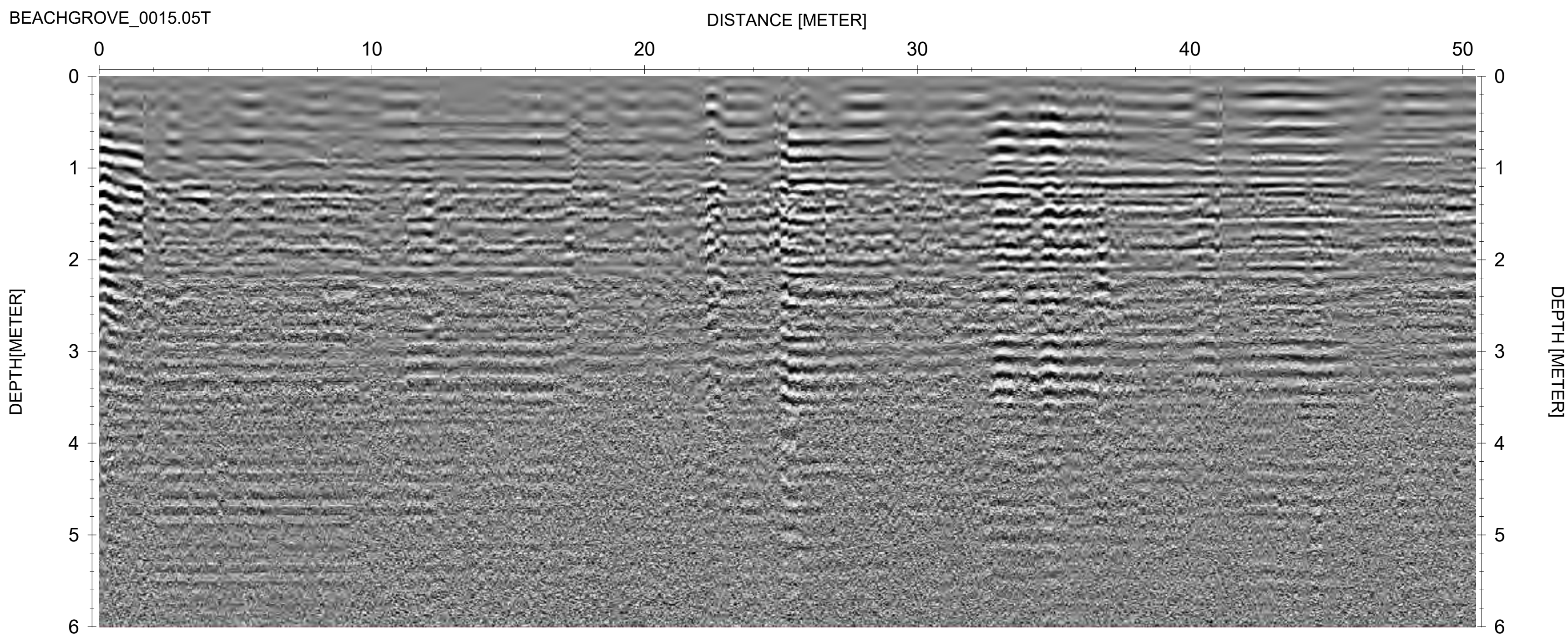
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DEPTH [METER]
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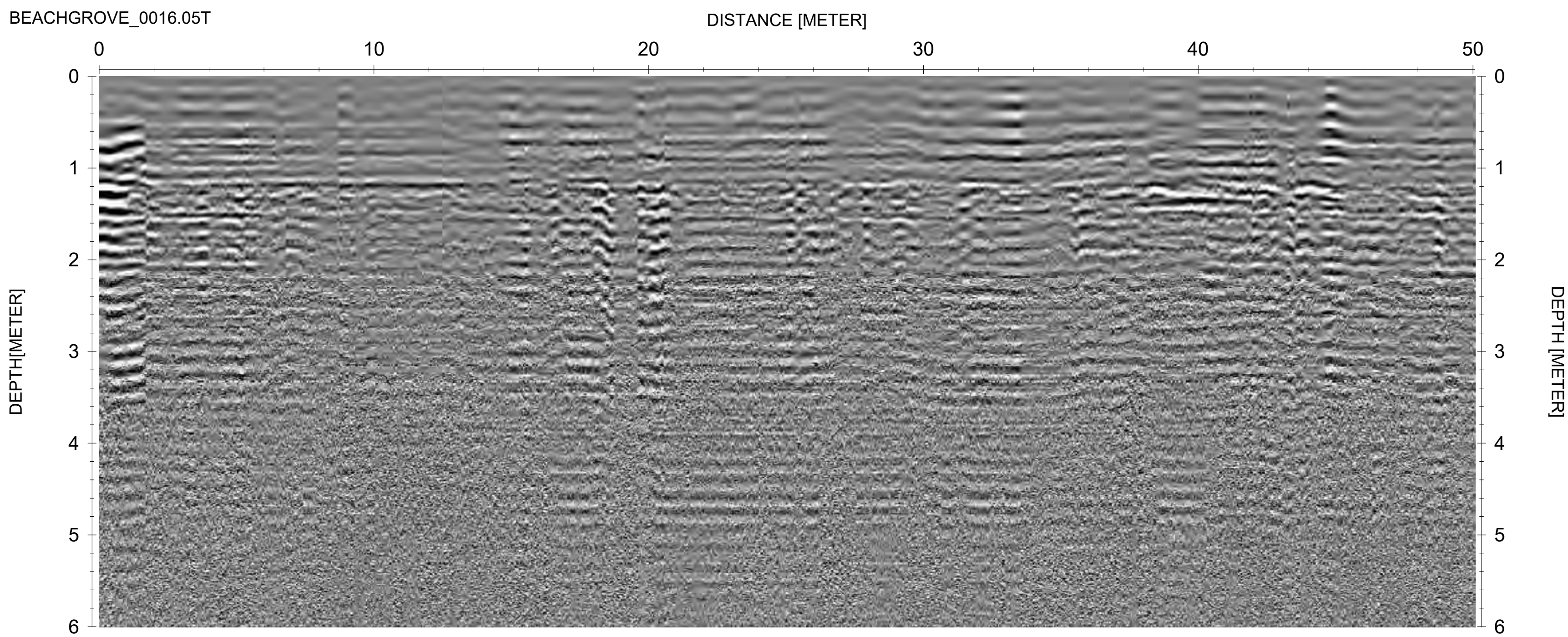
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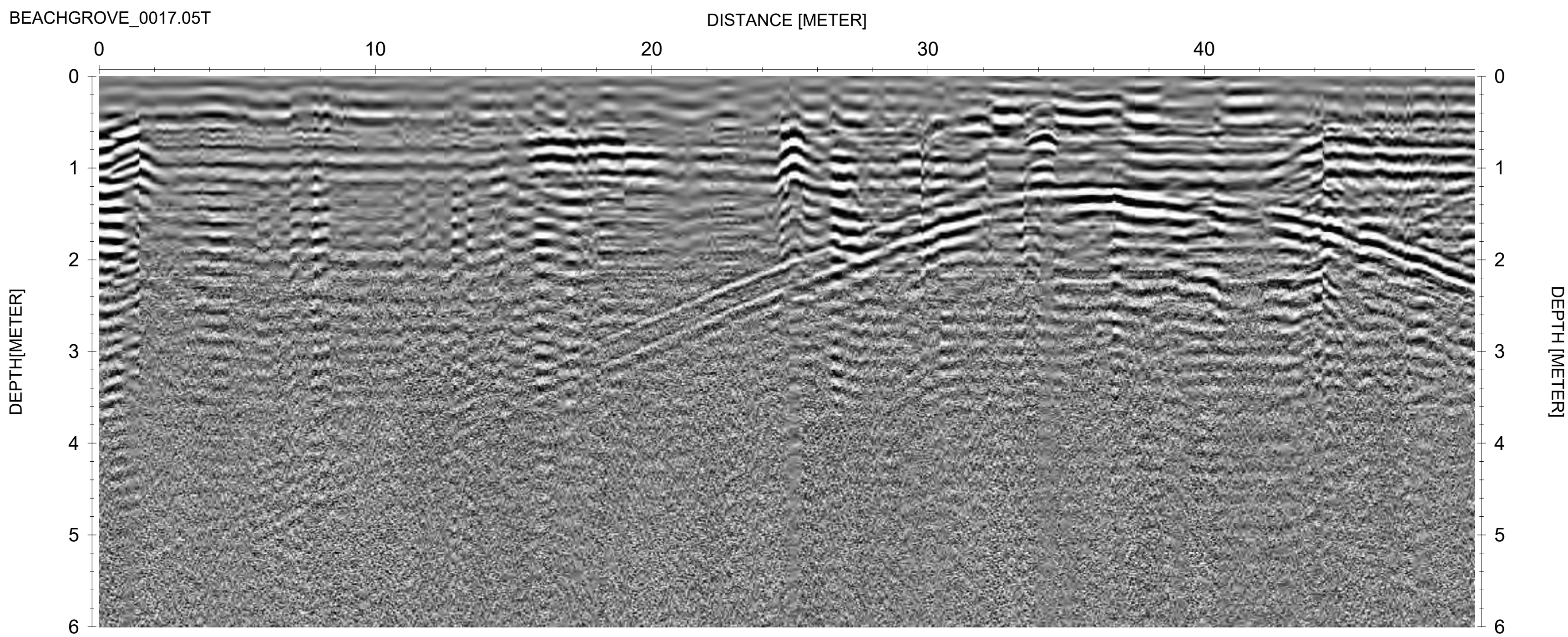
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DEPTH [METER]
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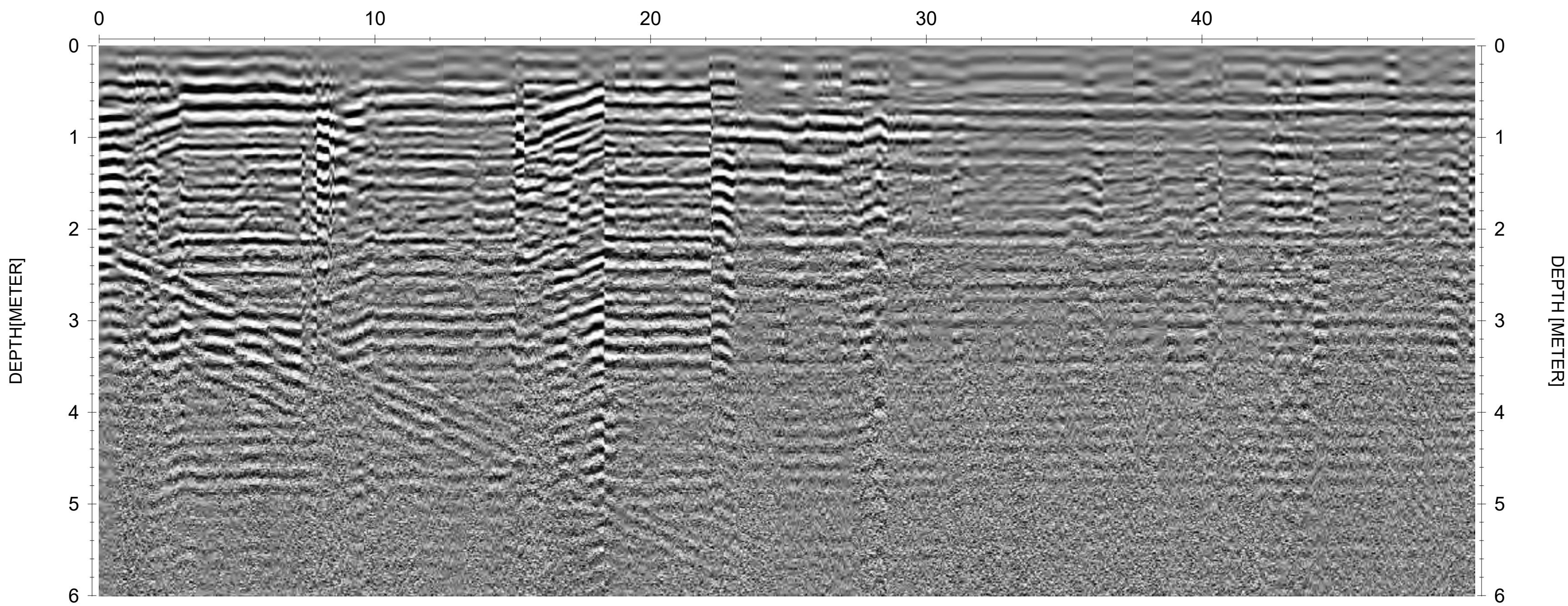
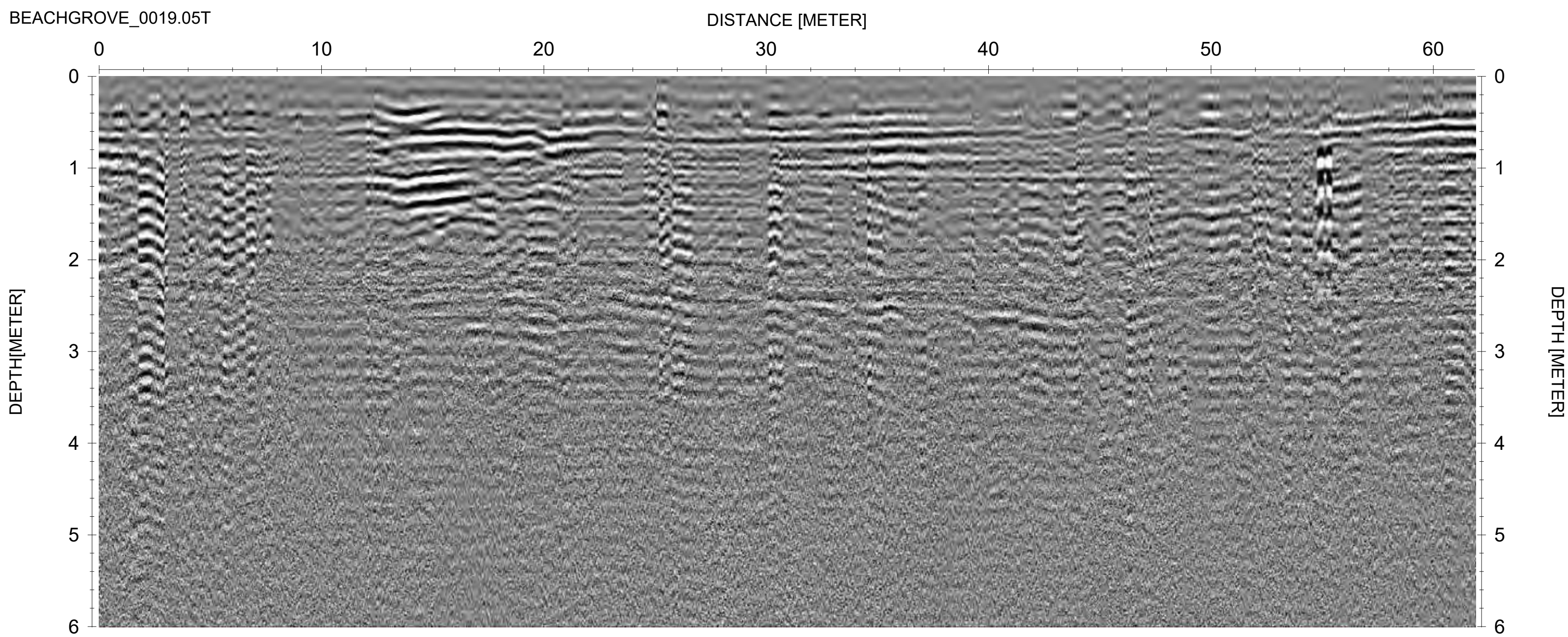
6

BEACHGROVE_0018.05T

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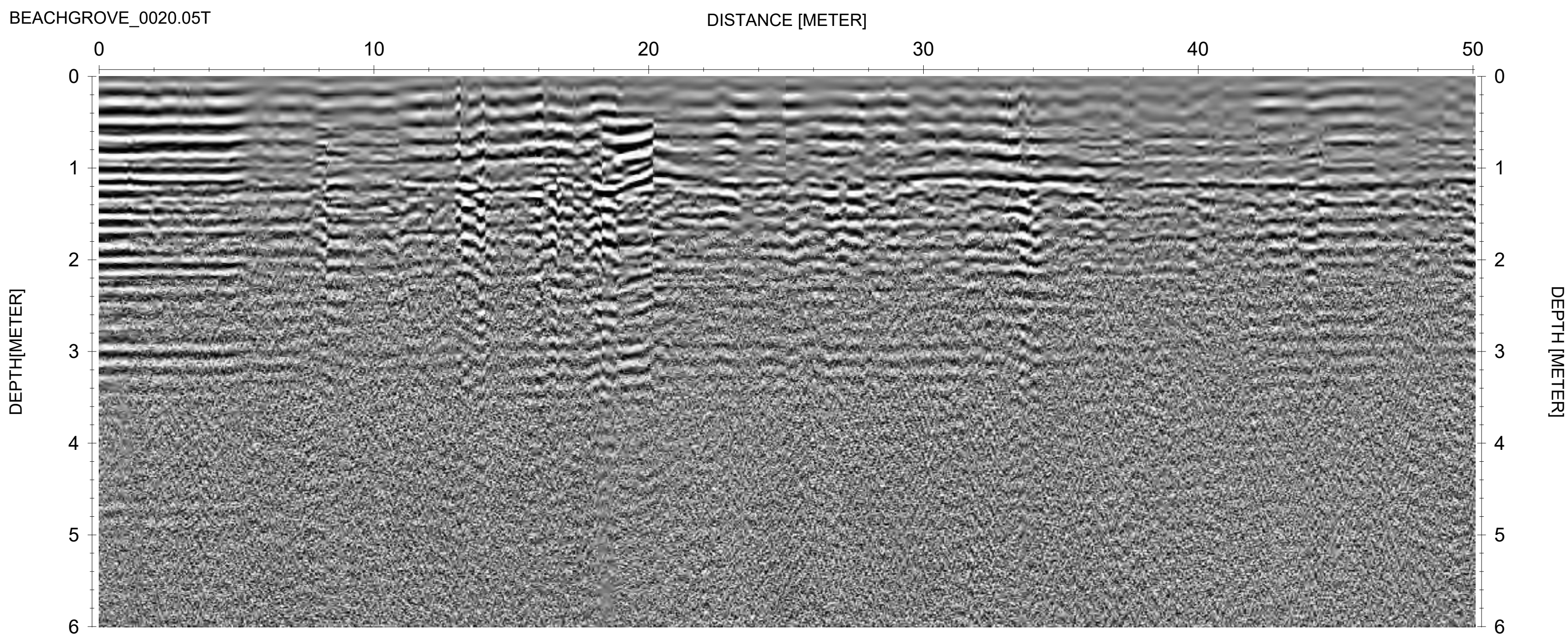
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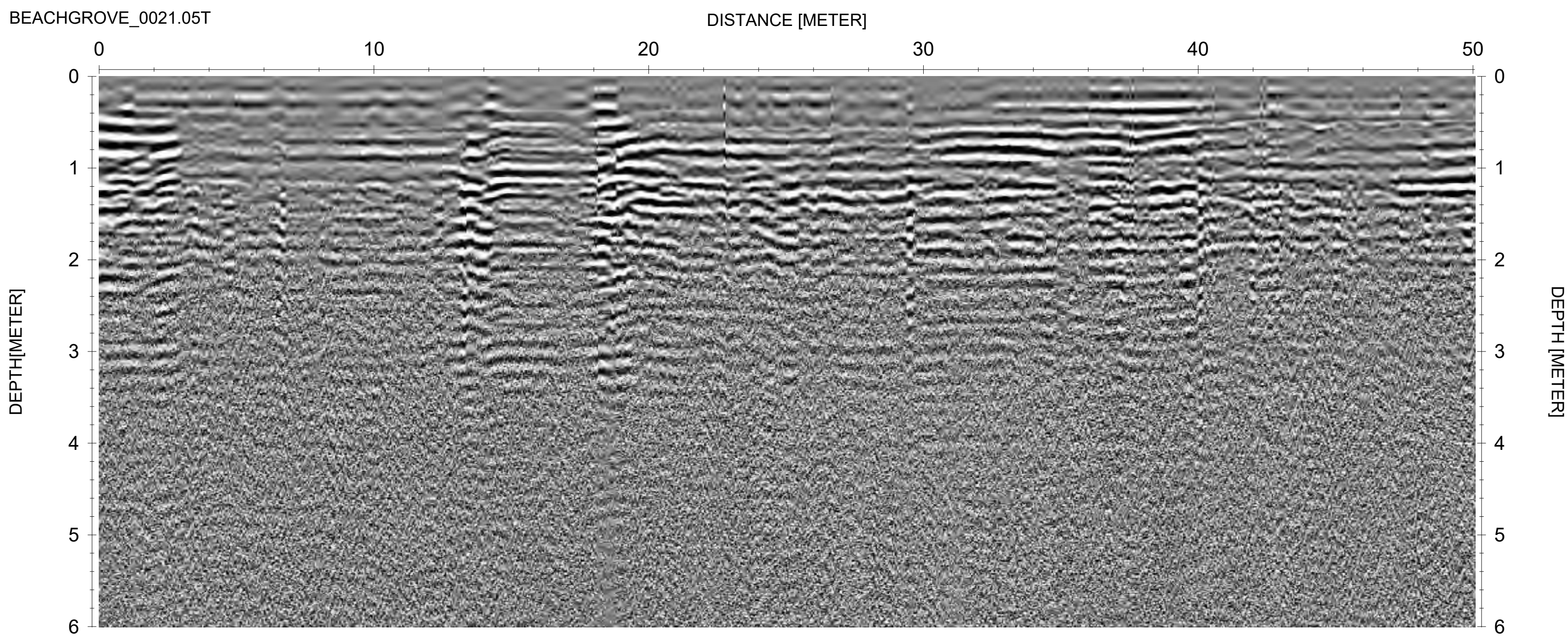

BEACHGROVE_0021.05T
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DEPTH[METER]
DEPTH [METER]
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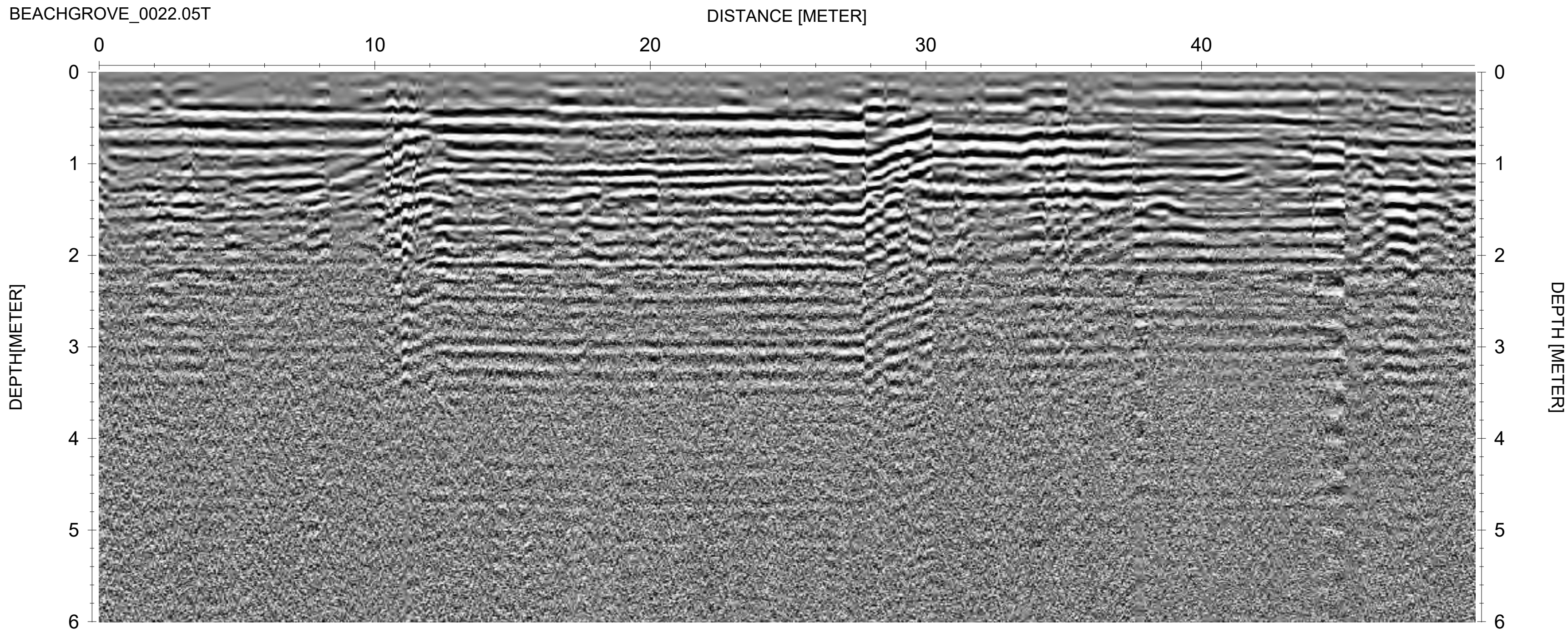

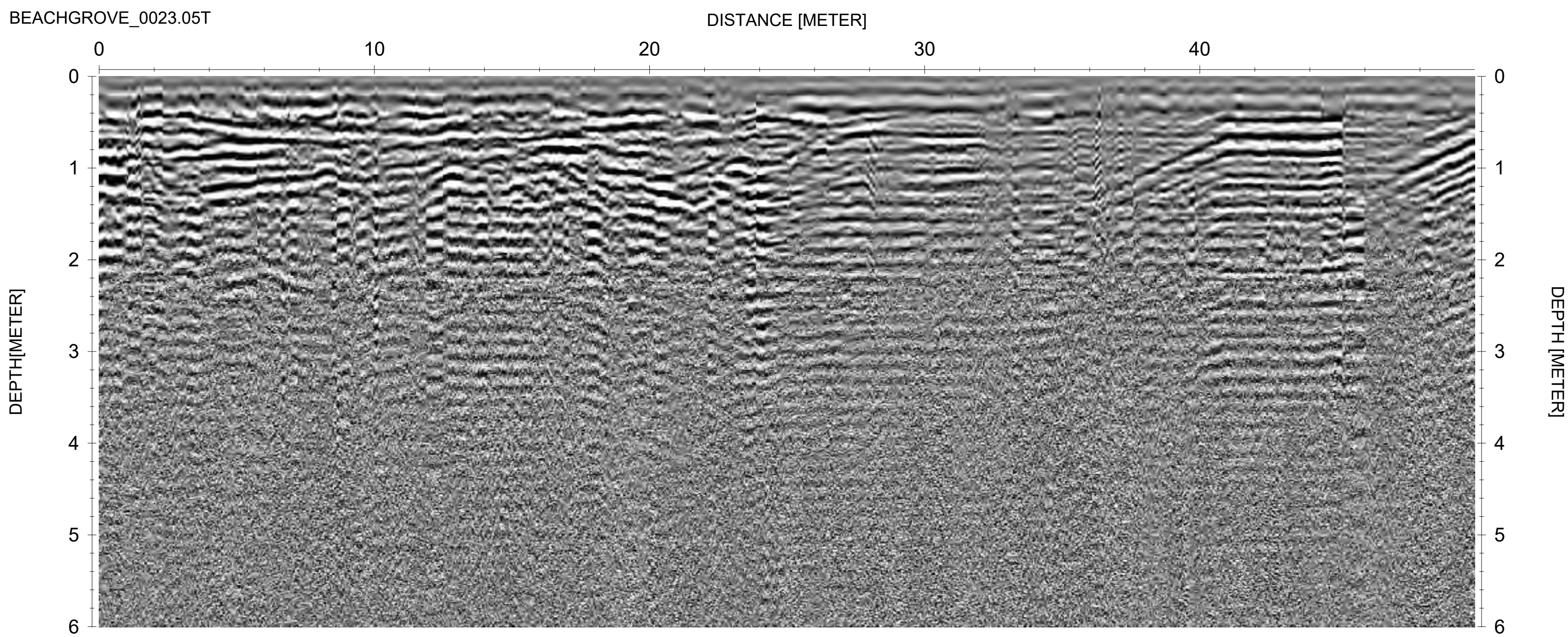
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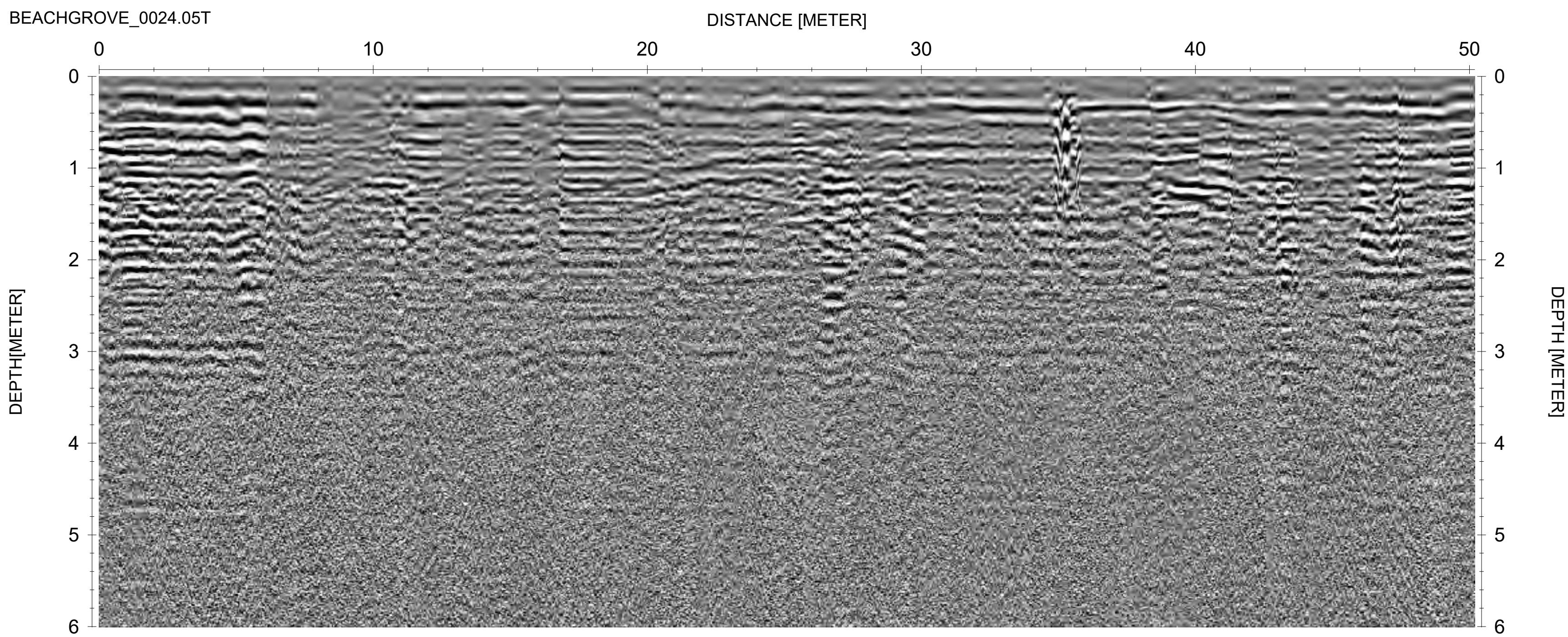
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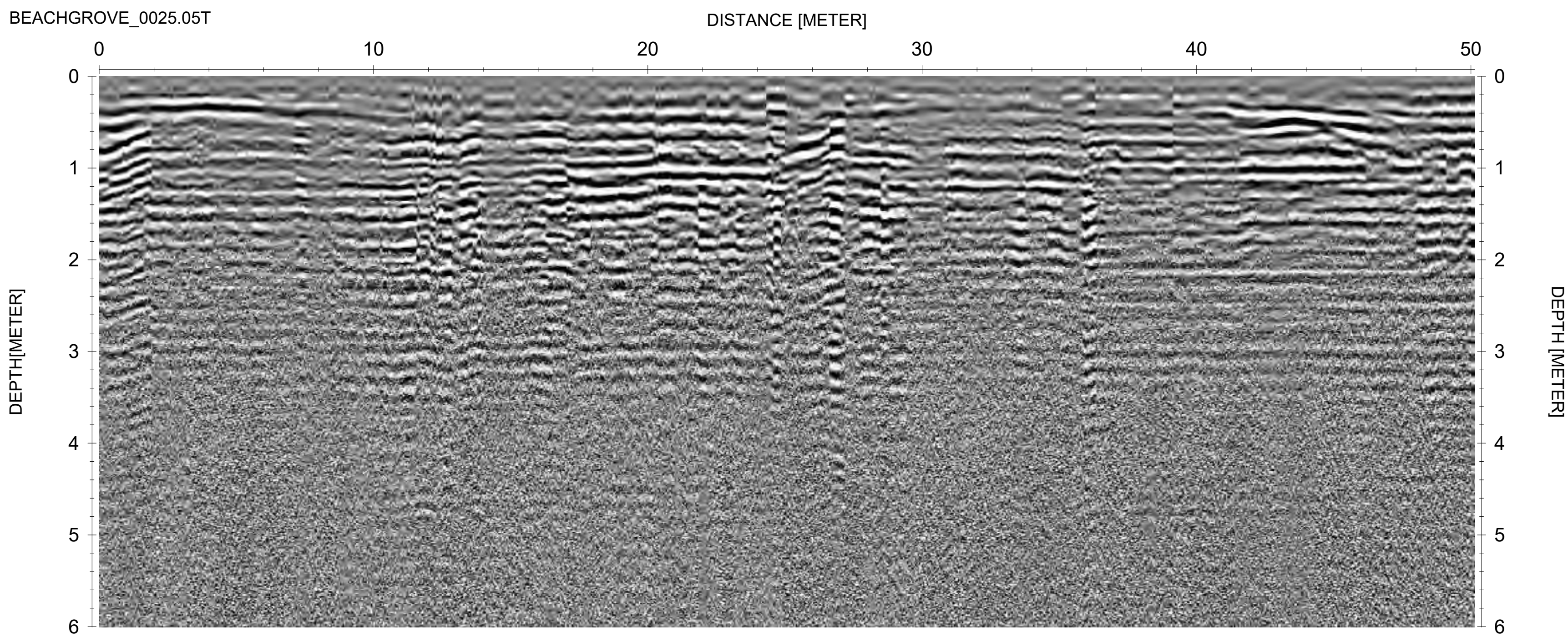
6
DEPTH [METER]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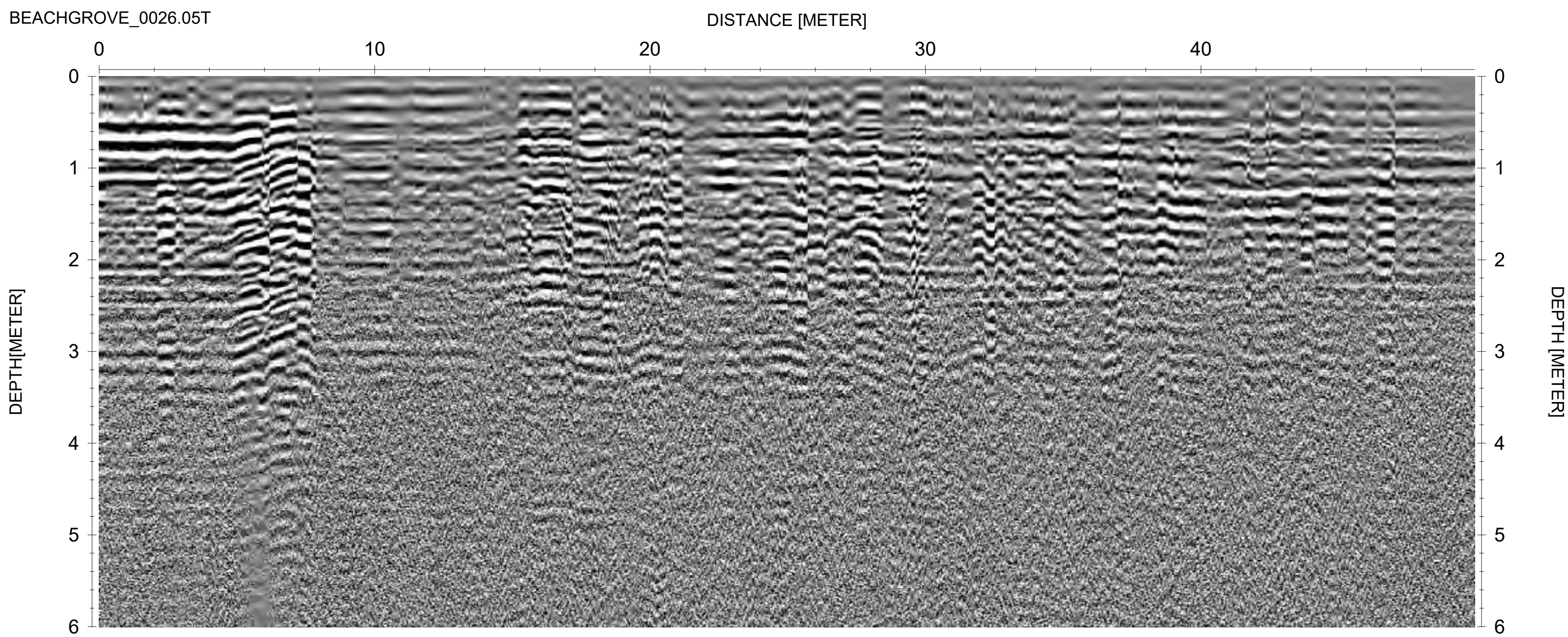

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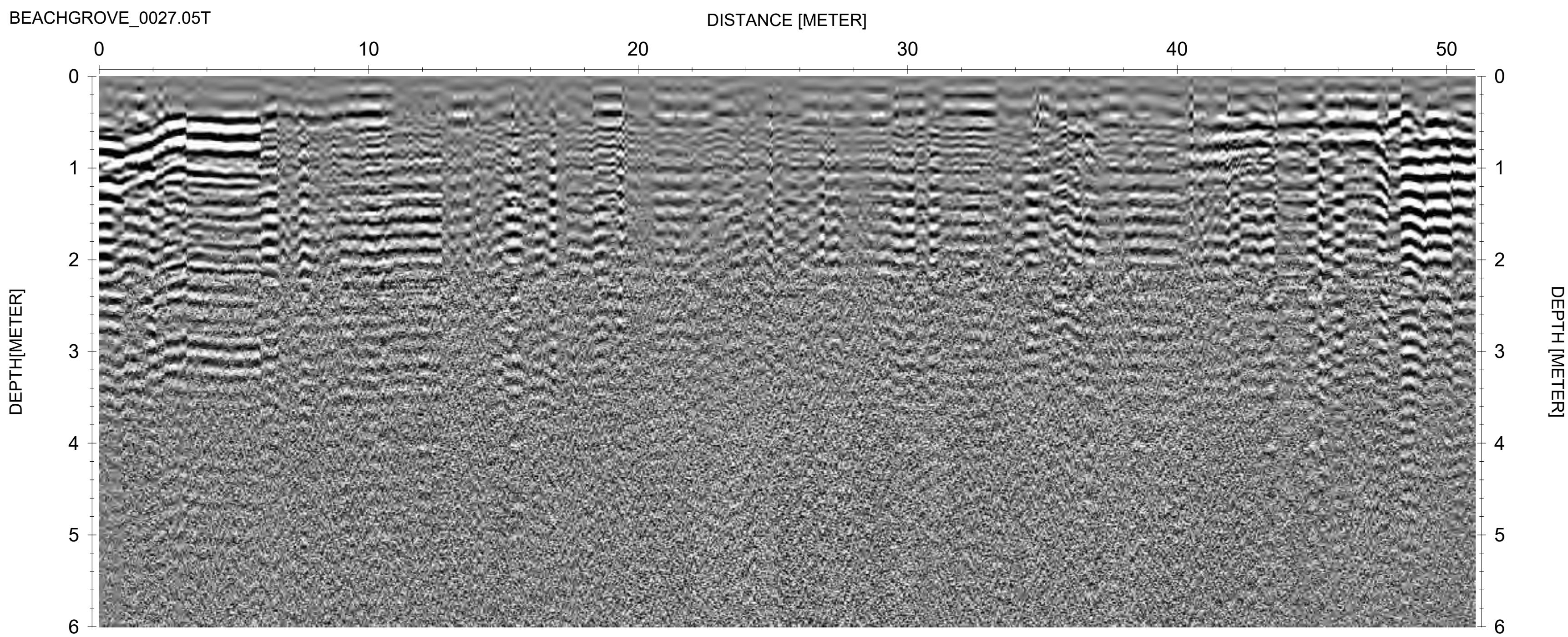

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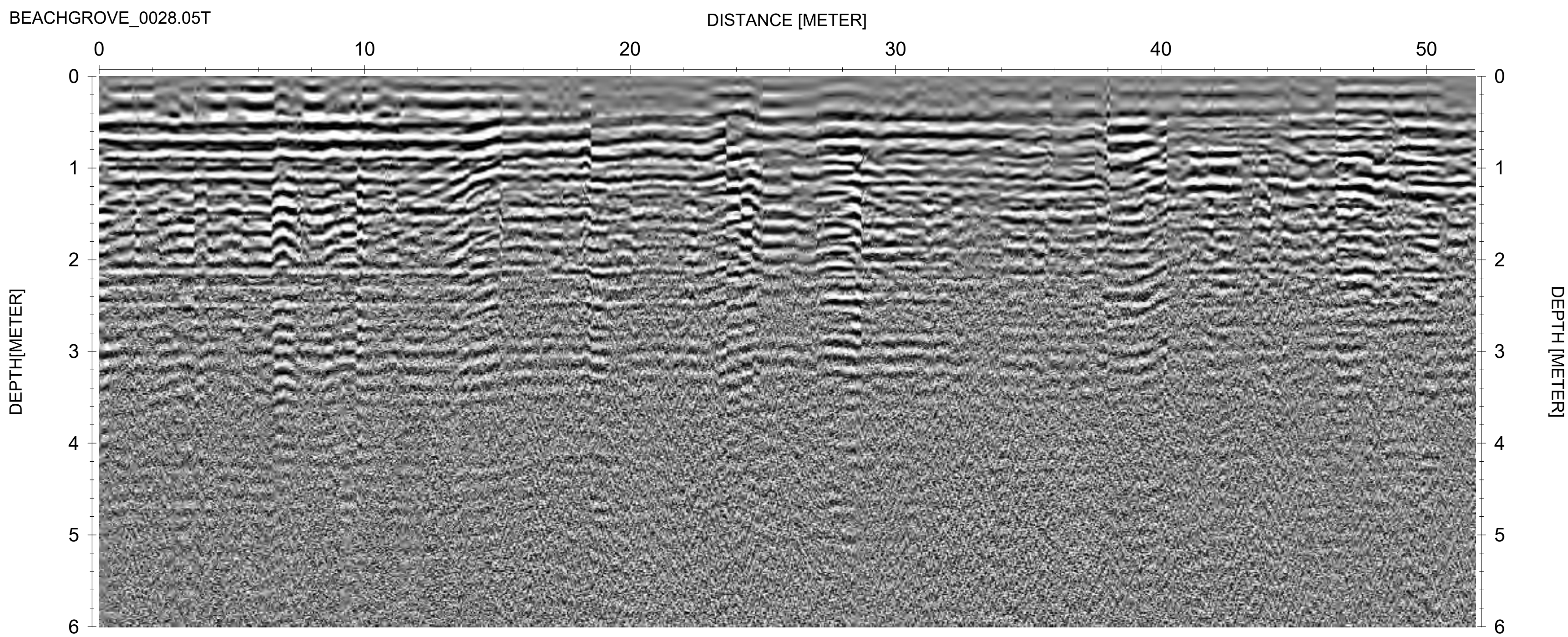
DEPTH[METER]
DEPTH [METER]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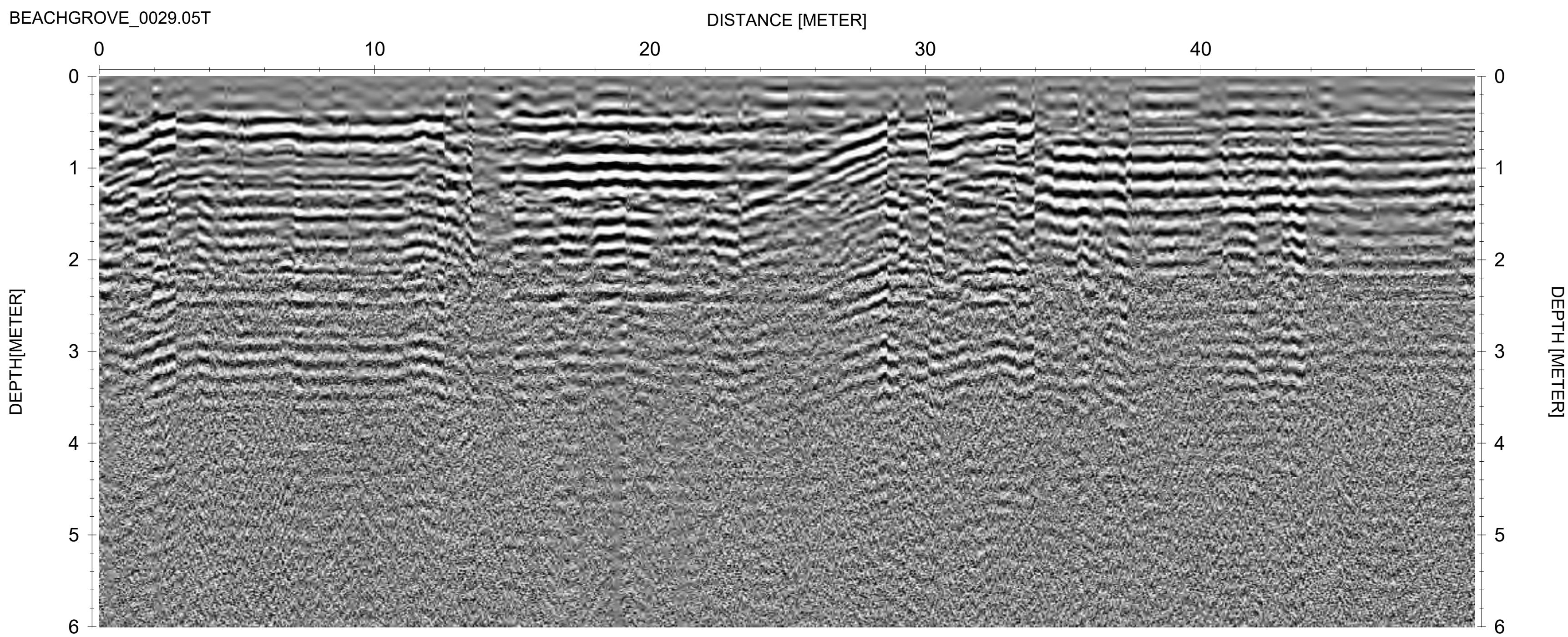
DEPTH[METER]
DEPTH [METER]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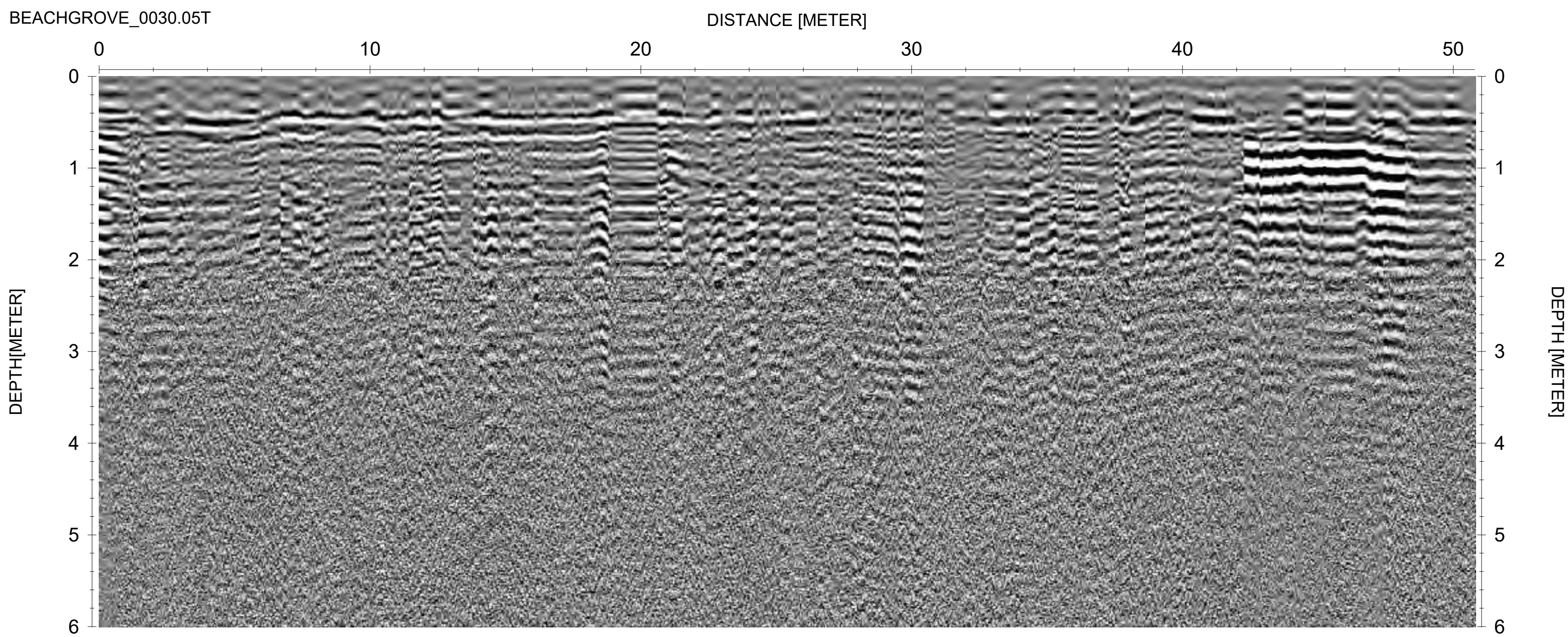
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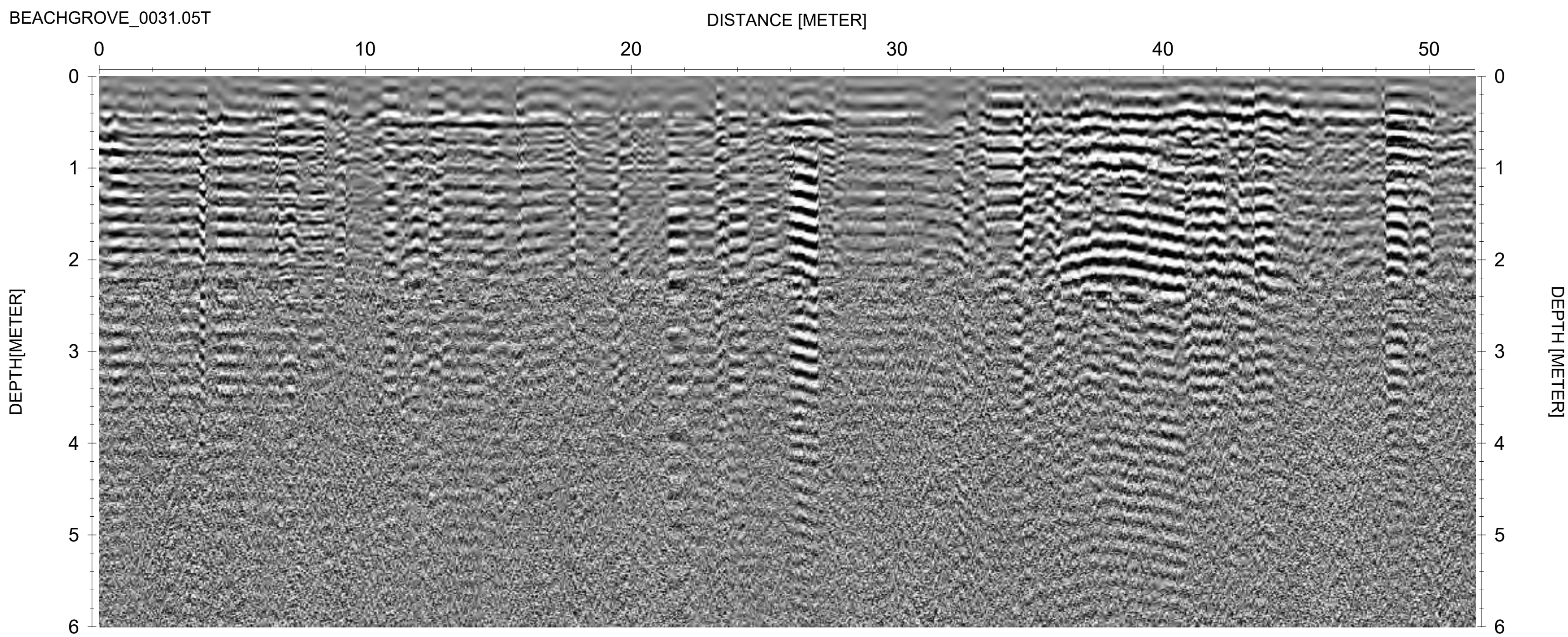
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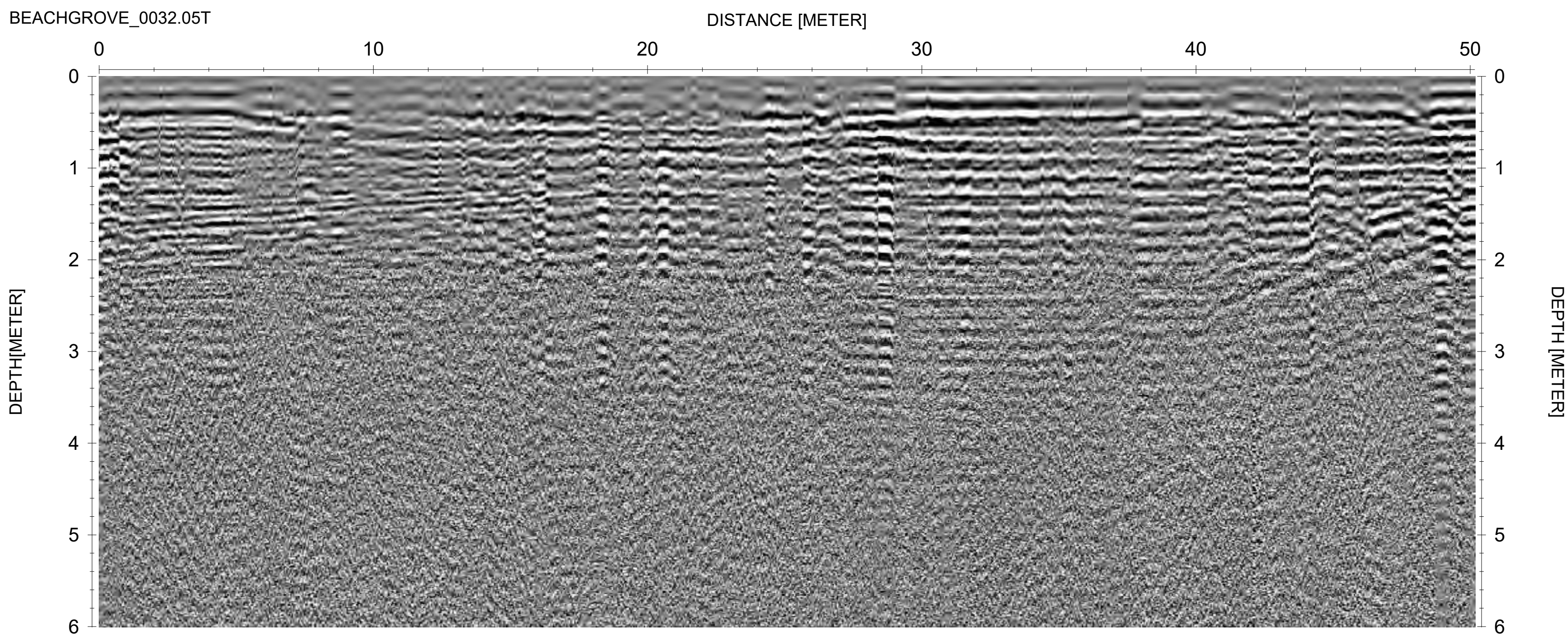
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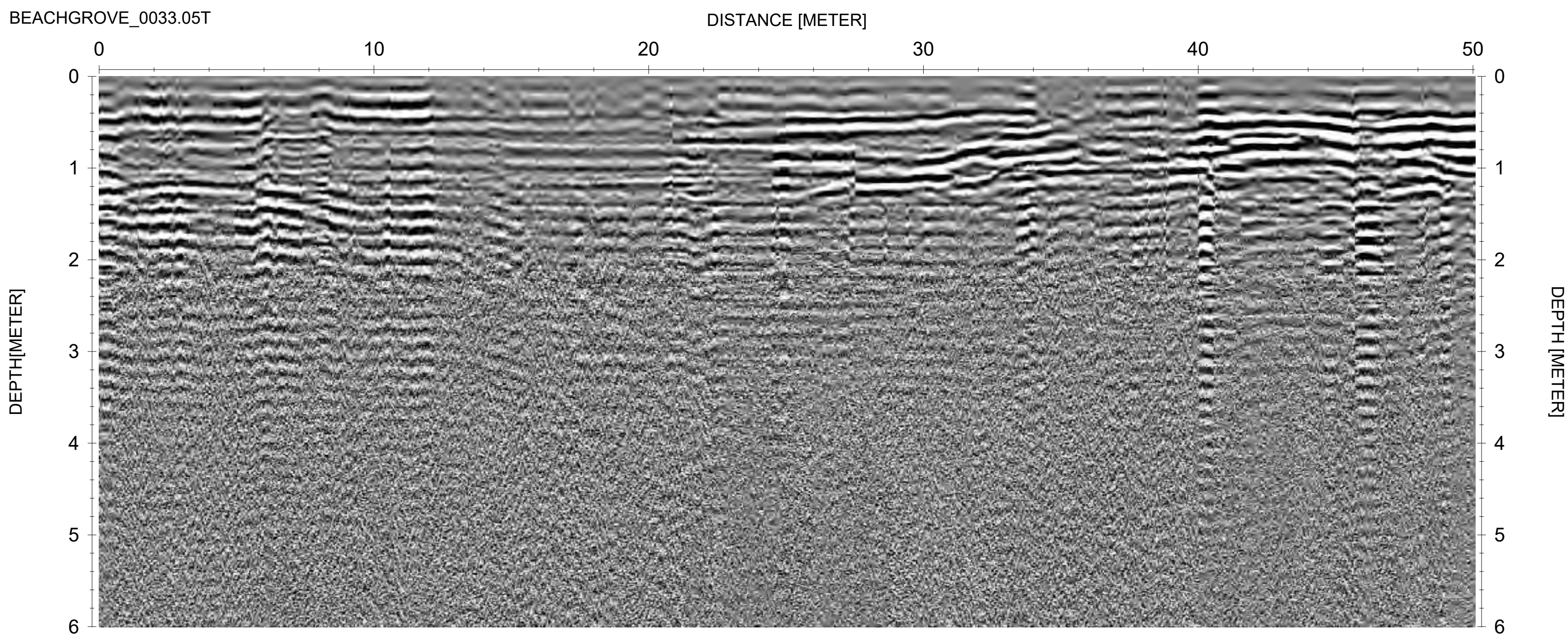
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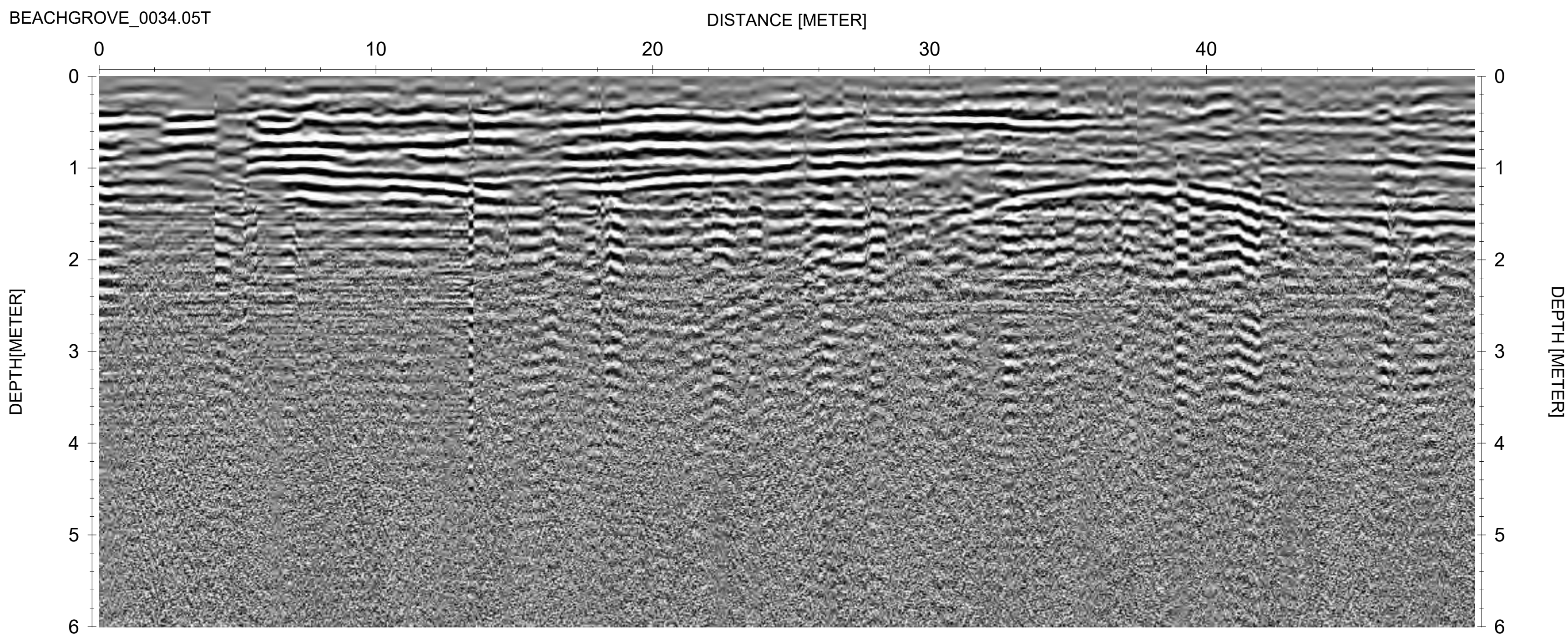

BEACHGROVE_0034.05T
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DEPTH[METER]
DEPTH [METER]
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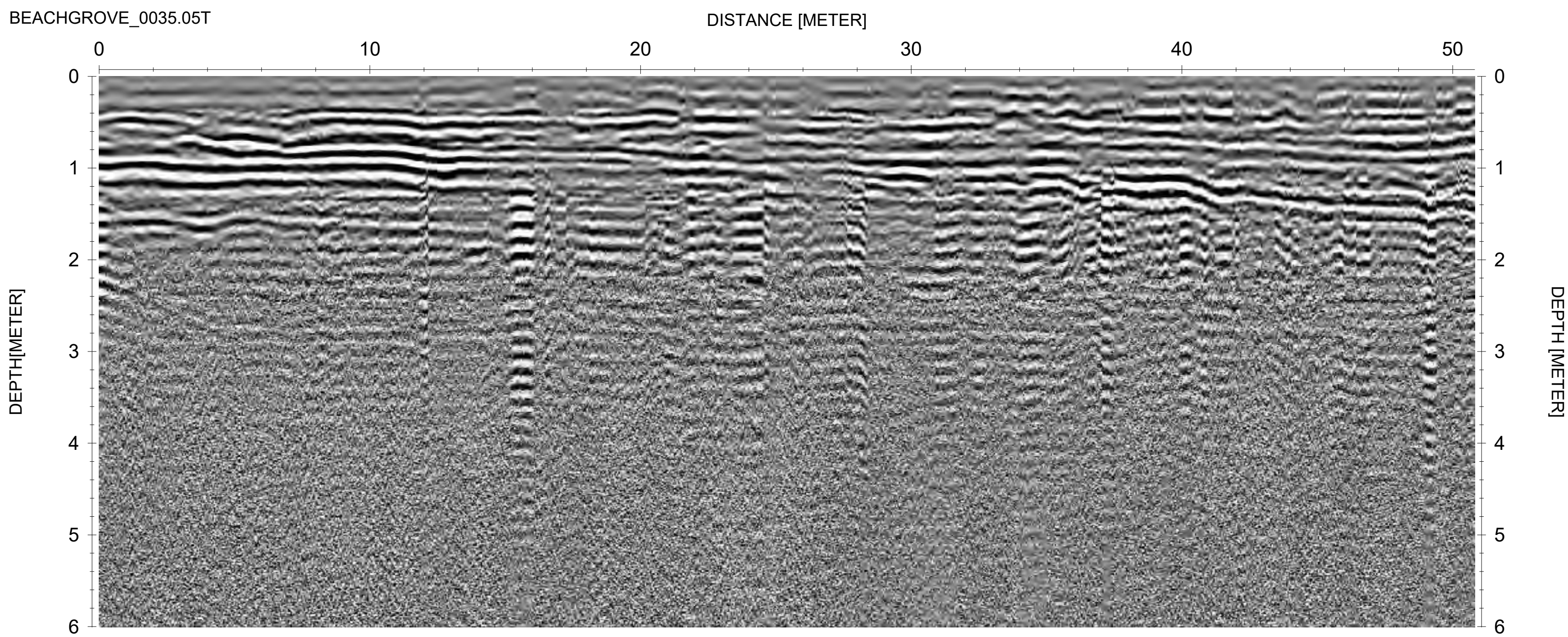

BEACHGROVE_0035.05T
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DEPTH[METER]
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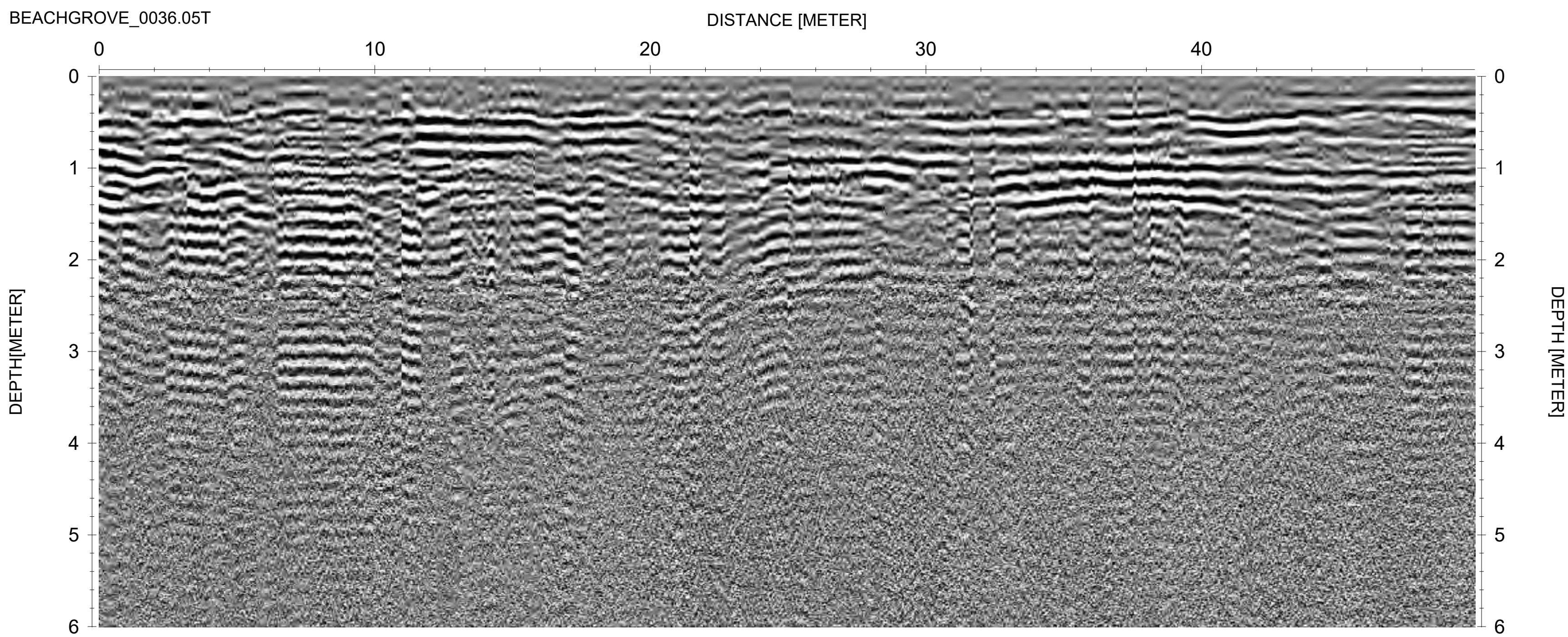

BEACHGROVE_0036.05T
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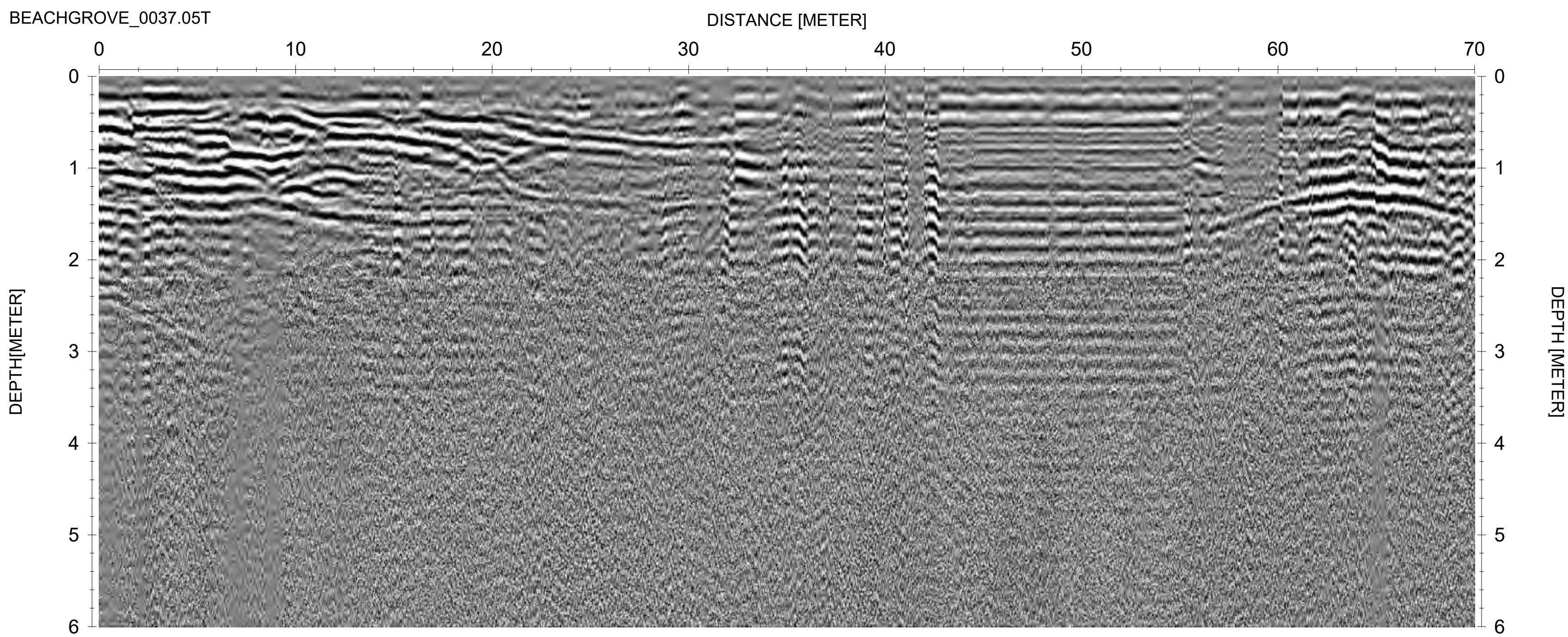
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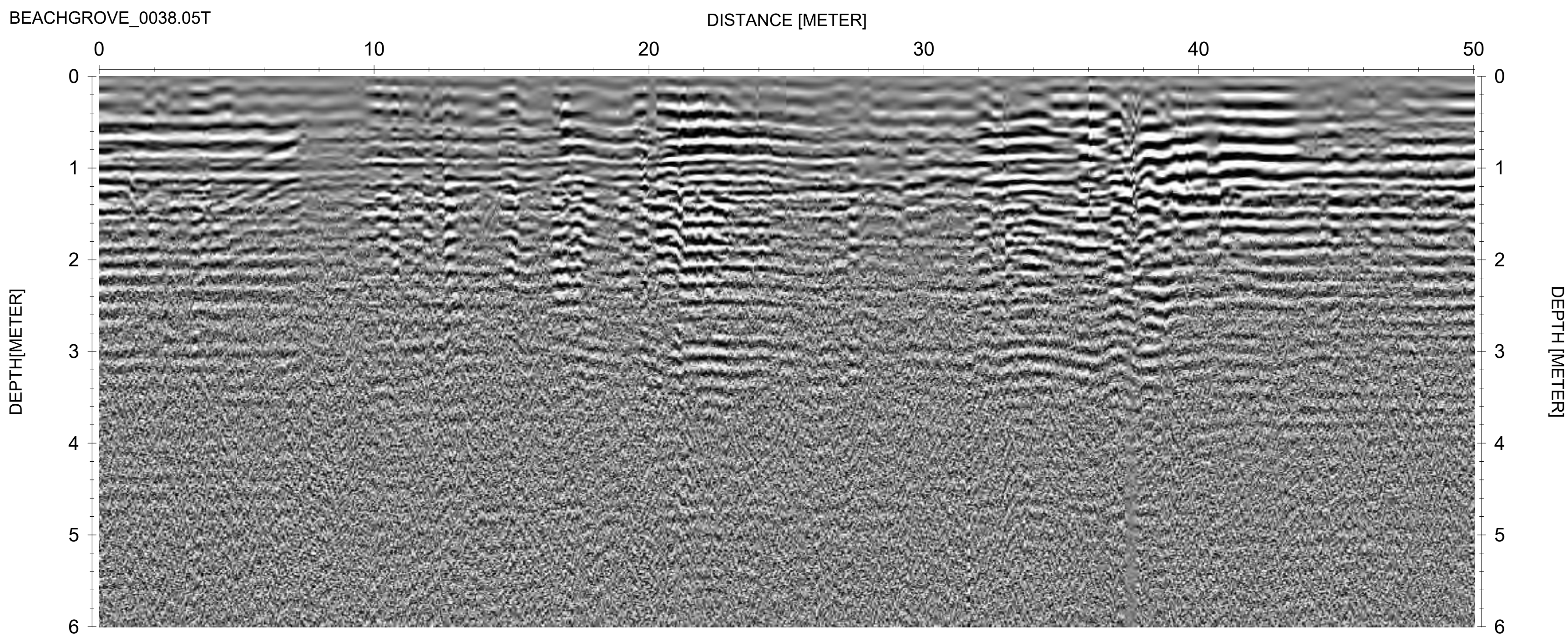
6
DEPTH [METER]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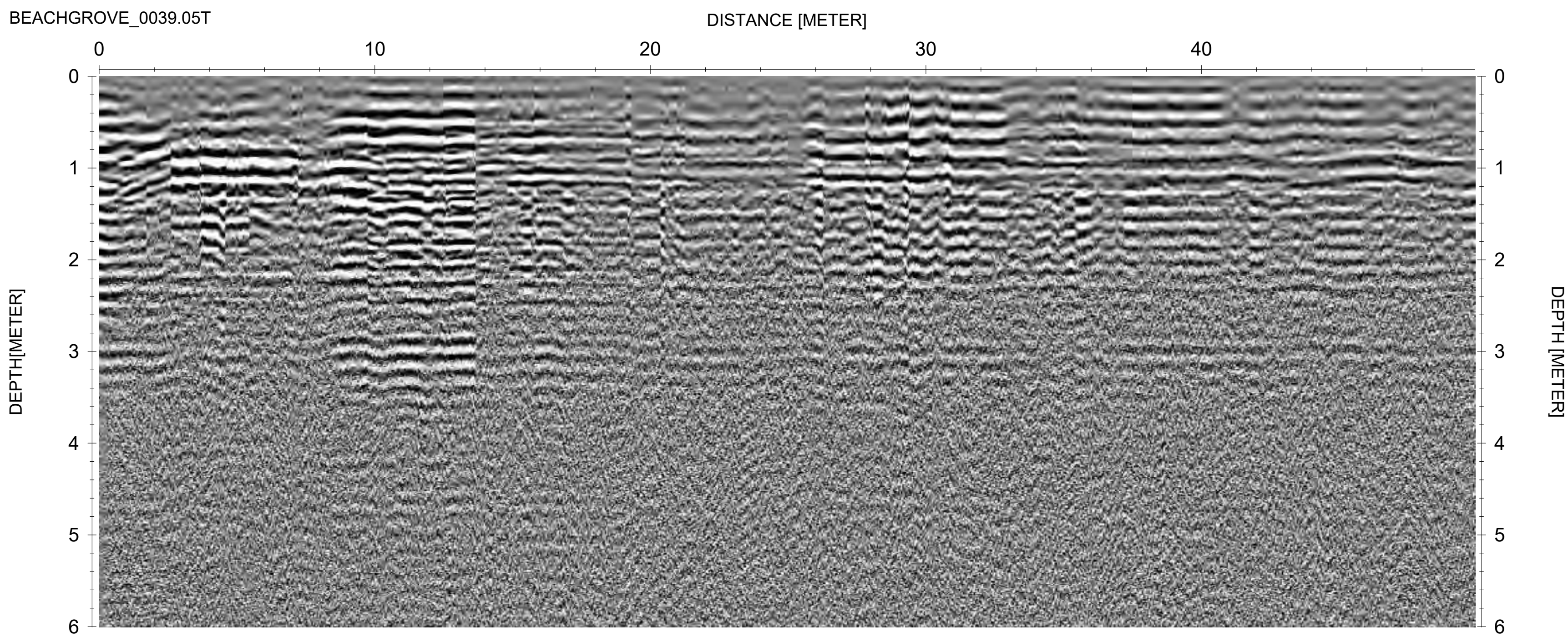
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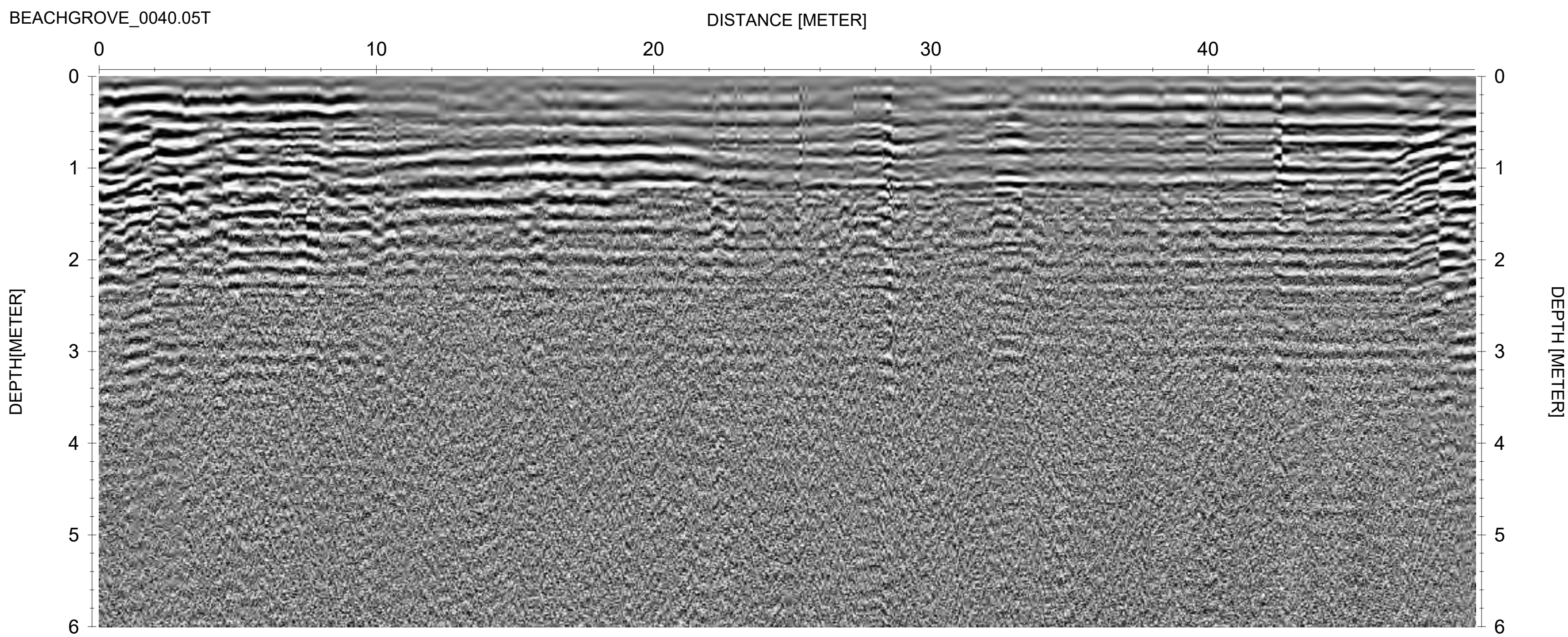
DEPTH [METER]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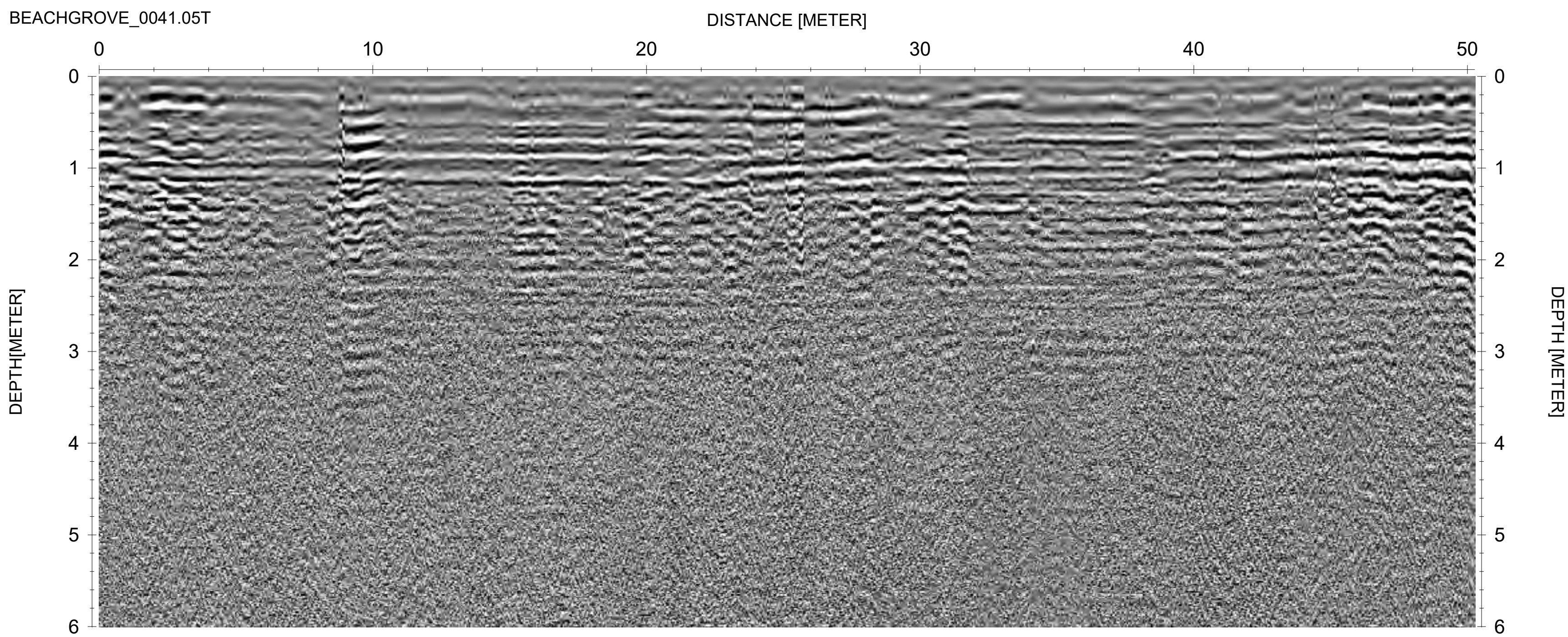
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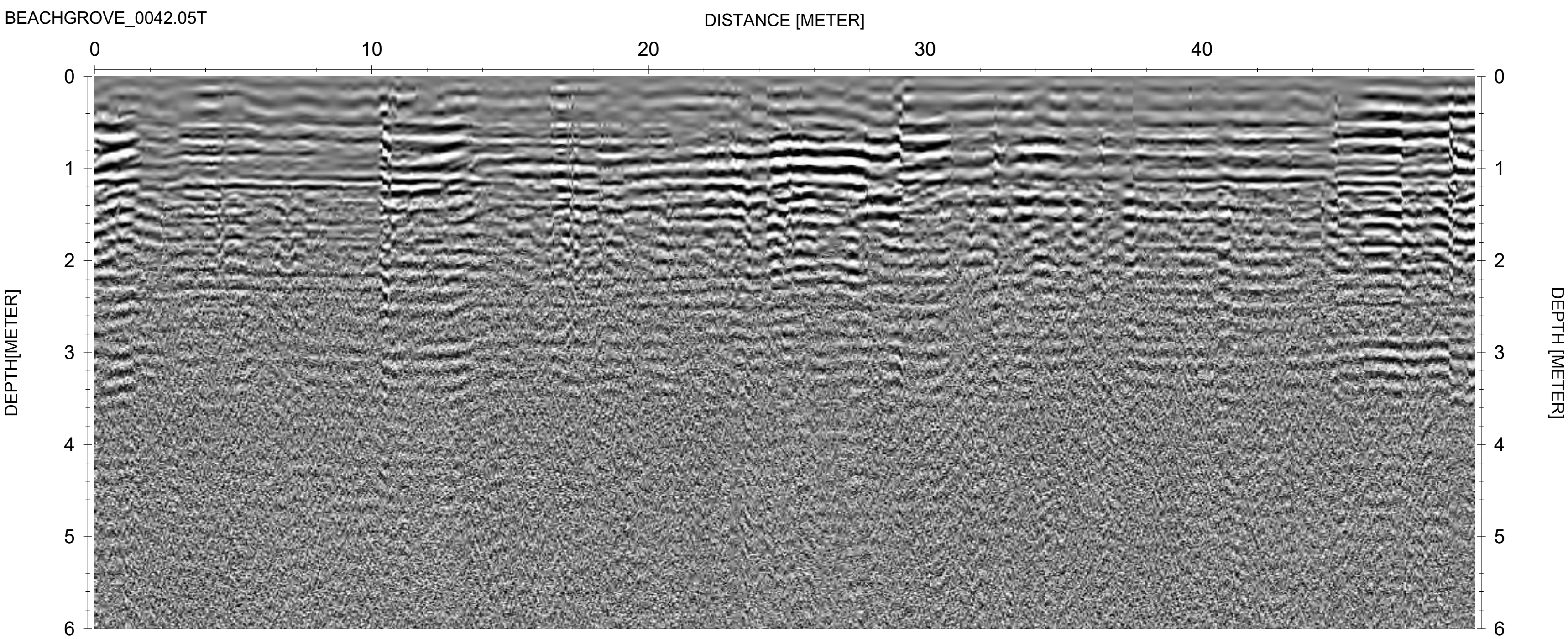
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DEPTH [METER]
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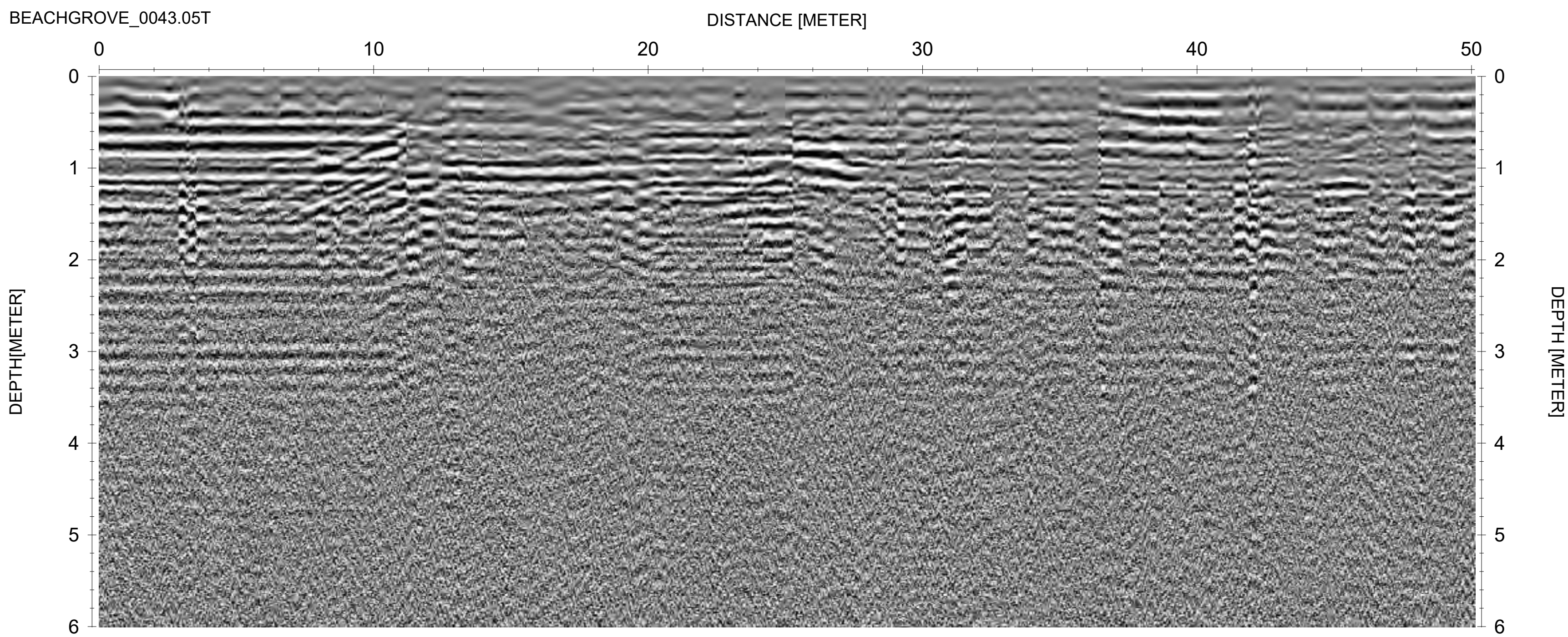
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DEPTH [METER]
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3
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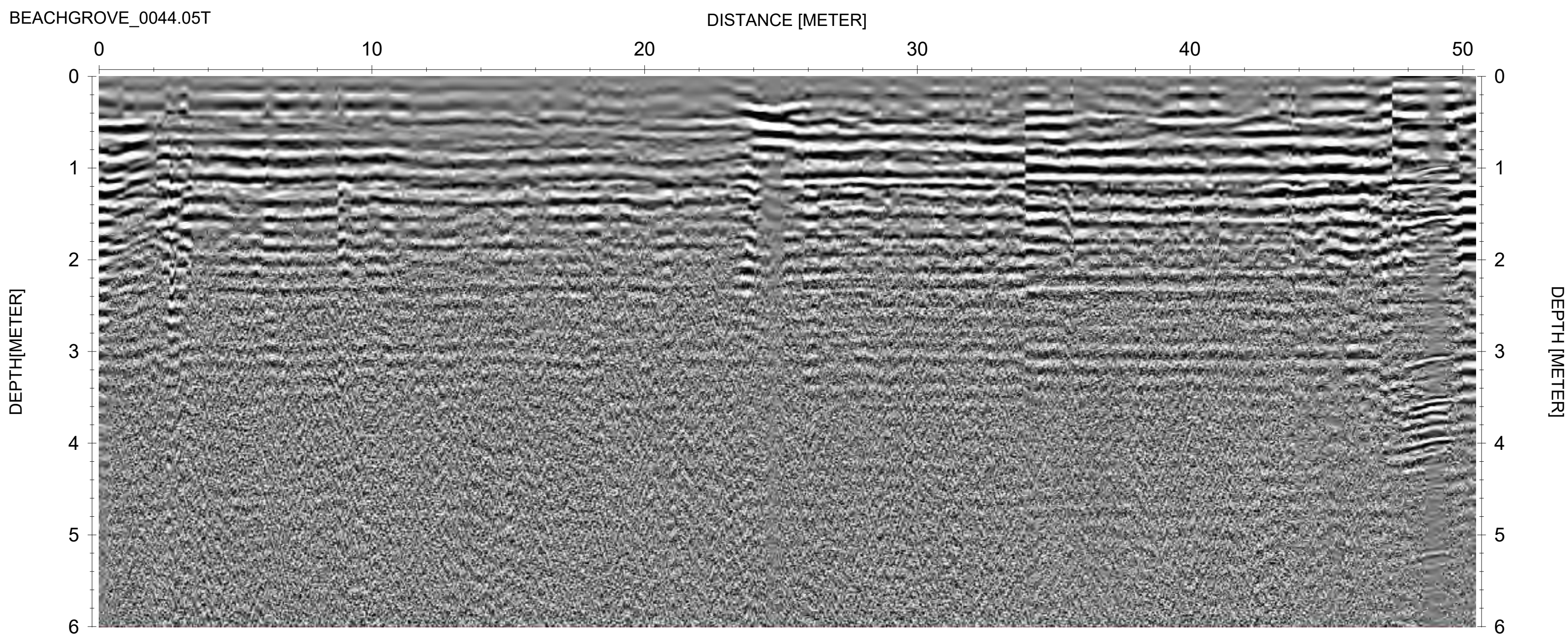
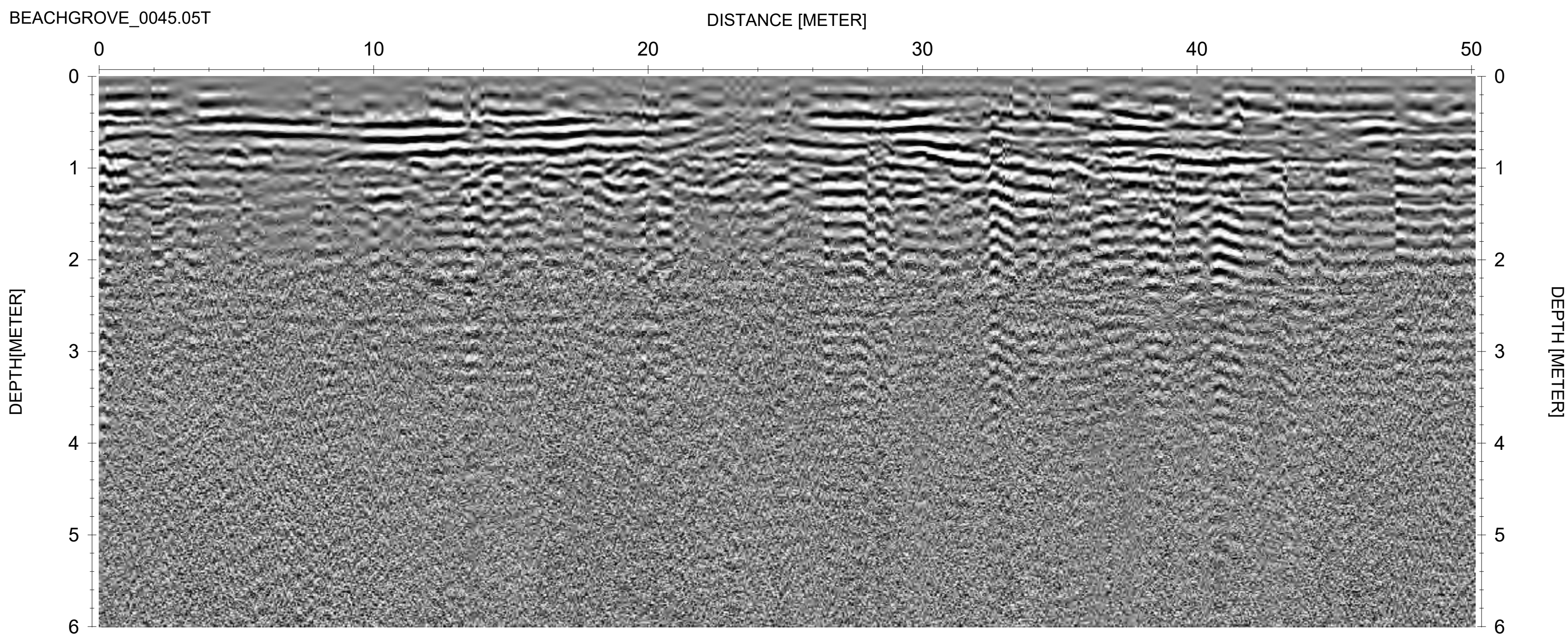
6


BEACHGROVE_0044.05T
DISTANCE [METER]
0
10
20
30
40
50
DEPTH[METER]
0
1
2
3
4
5
6
DEPTH [METER]



BEACHGROVE_0045.05T
DISTANCE [METER]
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50
0
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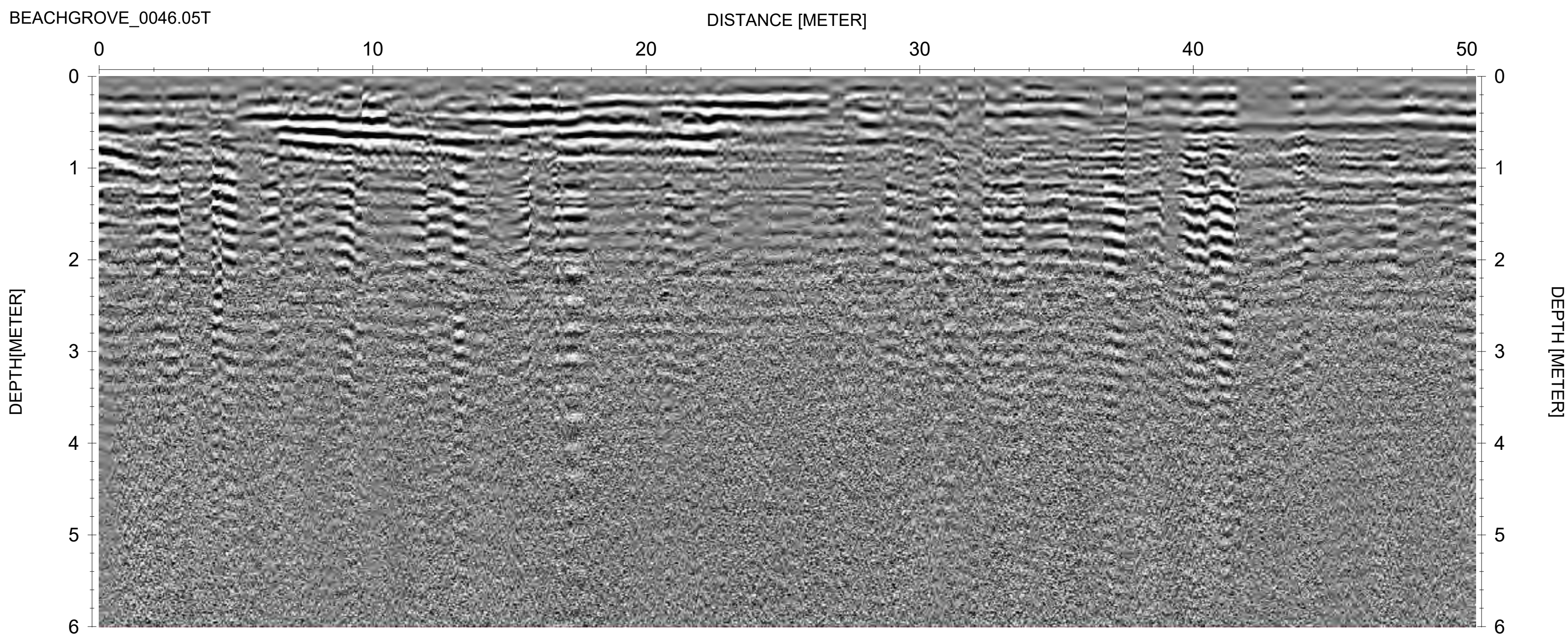
2
3
4
5
6
DEPTH[METER]
DEPTH [METER]



BEACHGROVE_0046.05T
DISTANCE [METER]
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30
40
50
DEPTH[METER]
DEPTH [METER]
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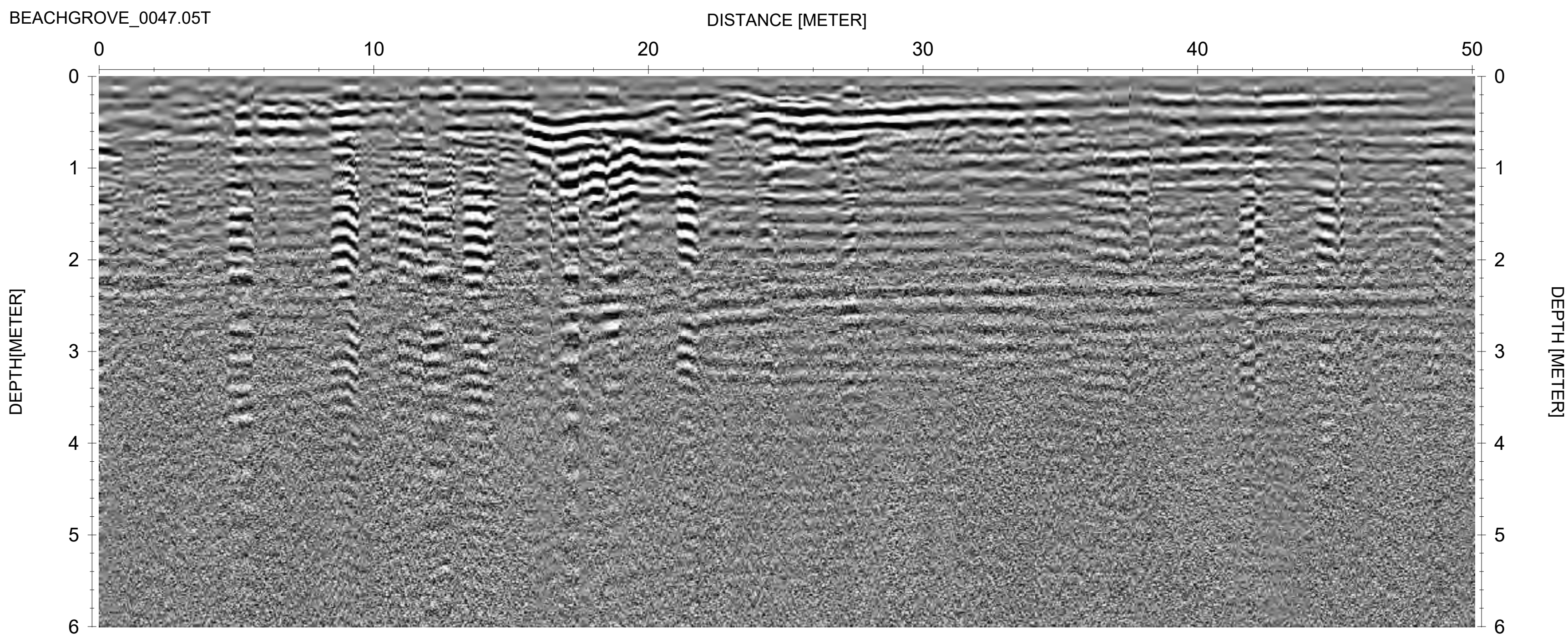
1
2
3
4
5
6



BEACHGROVE_0047.05T
DISTANCE [METER]
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50
0
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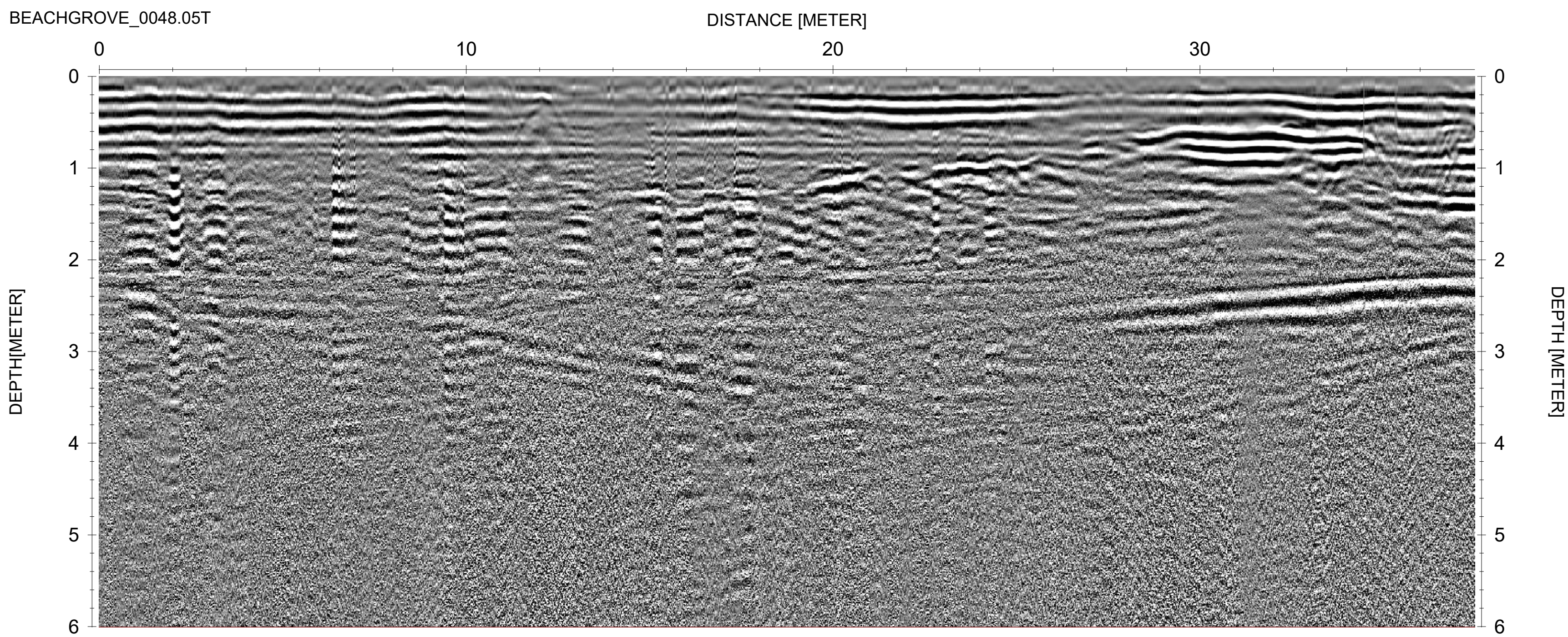
2
3
4
5
6
DEPTH[METER]
DEPTH [METER]


BEACHGROVE_0048.05T
DISTANCE [METER]
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30
DEPTH[METER]
DEPTH [METER]
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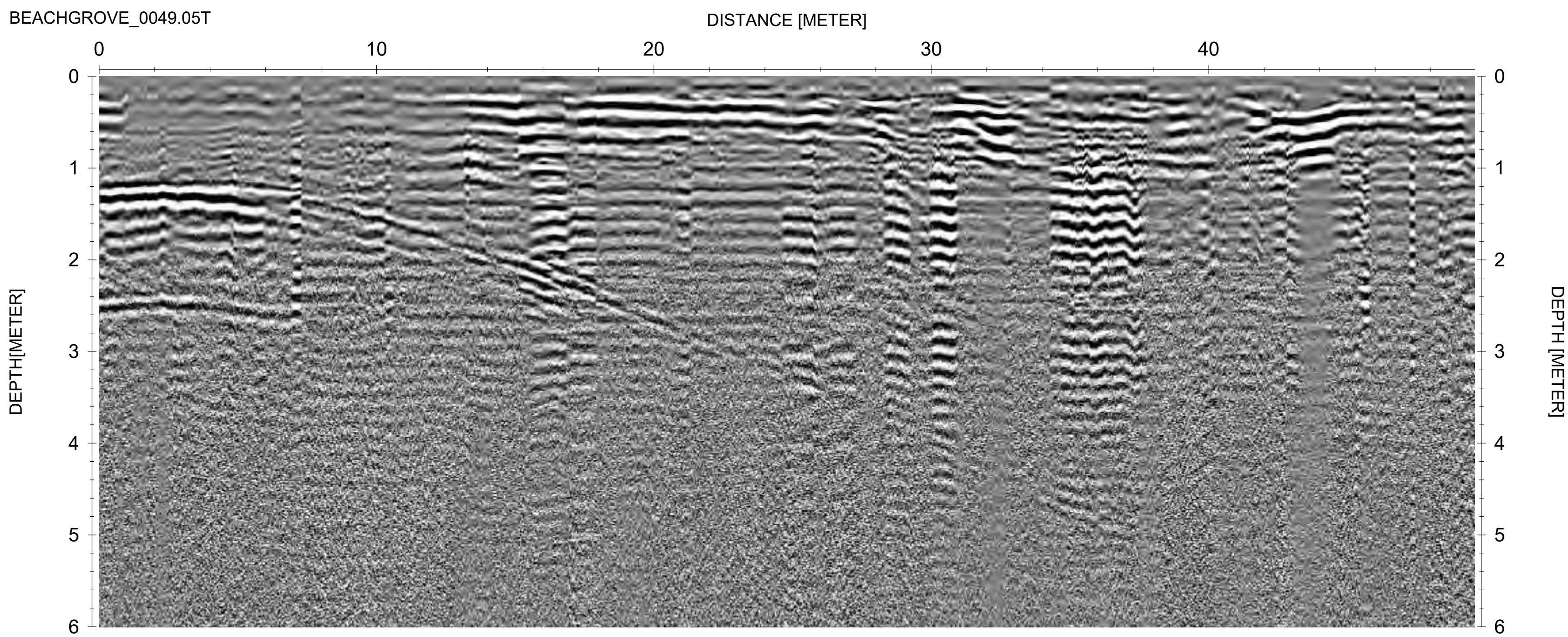
1
2
3
4
5
6


BEACHGROVE_0049.05T
DISTANCE [METER]
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DEPTH[METER]
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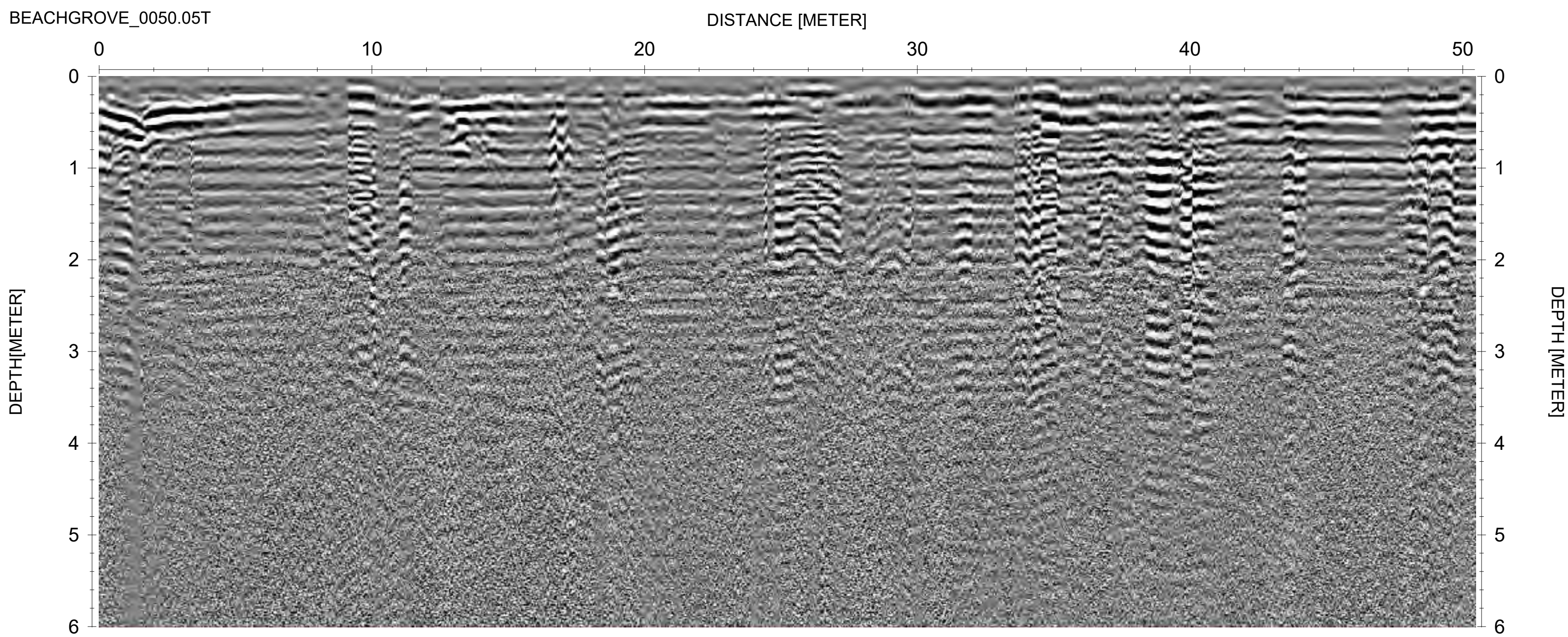
2
3
4
5
6
DEPTH [METER]


BEACHGROVE_0050.05T
DISTANCE [METER]
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DEPTH[METER]
DEPTH [METER]
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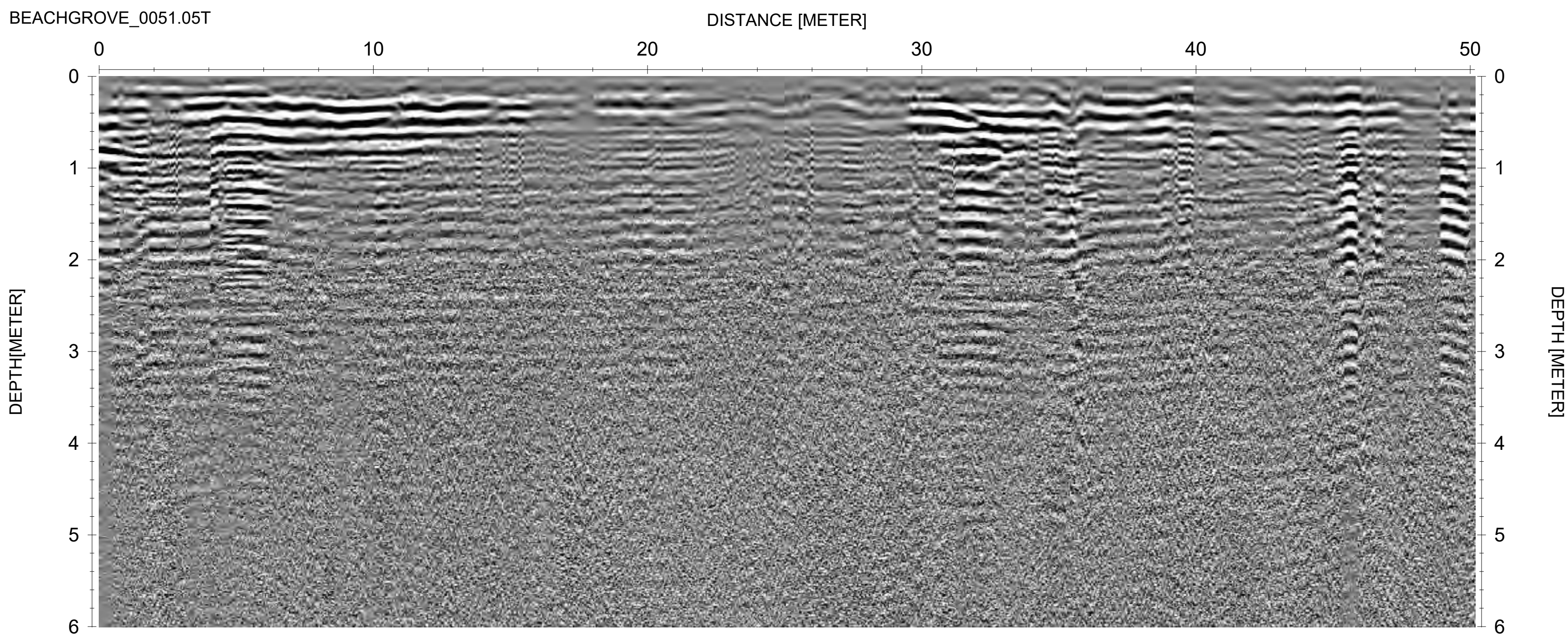
3
4
5
6



BEACHGROVE_0051.05T
DISTANCE [METER]
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DEPTH[METER]
DEPTH [METER]
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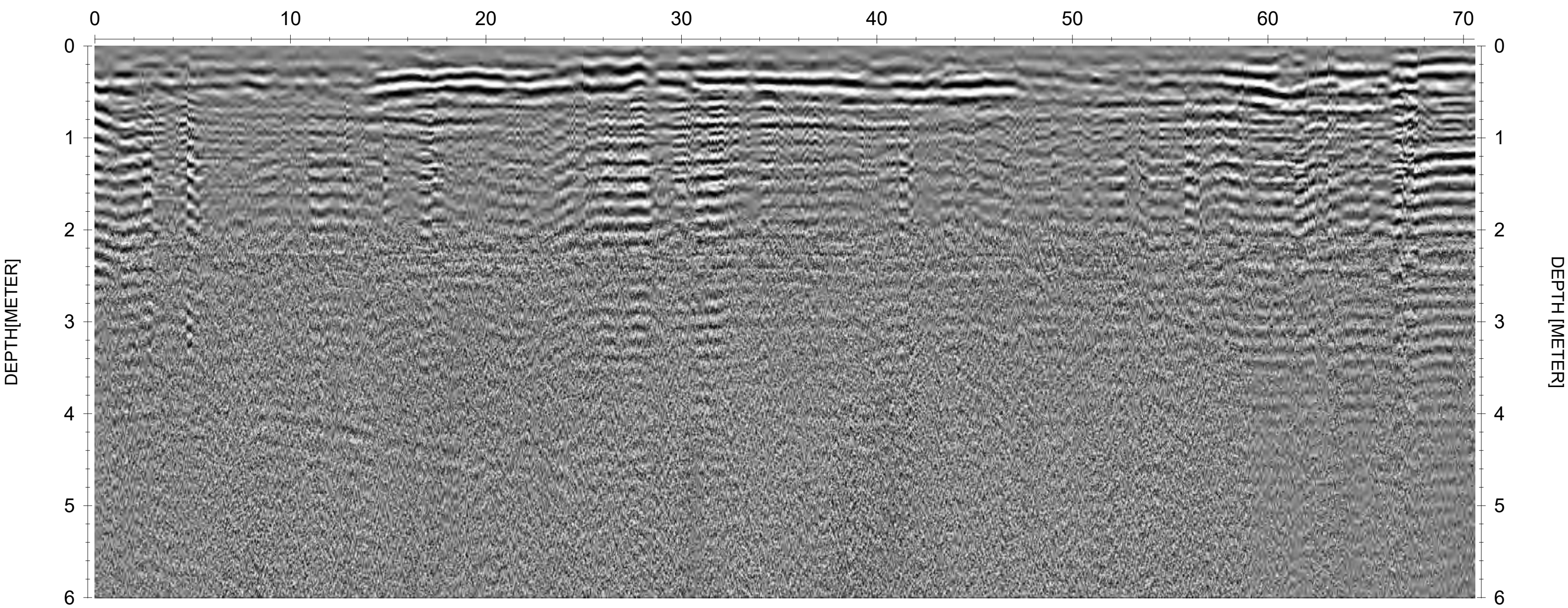
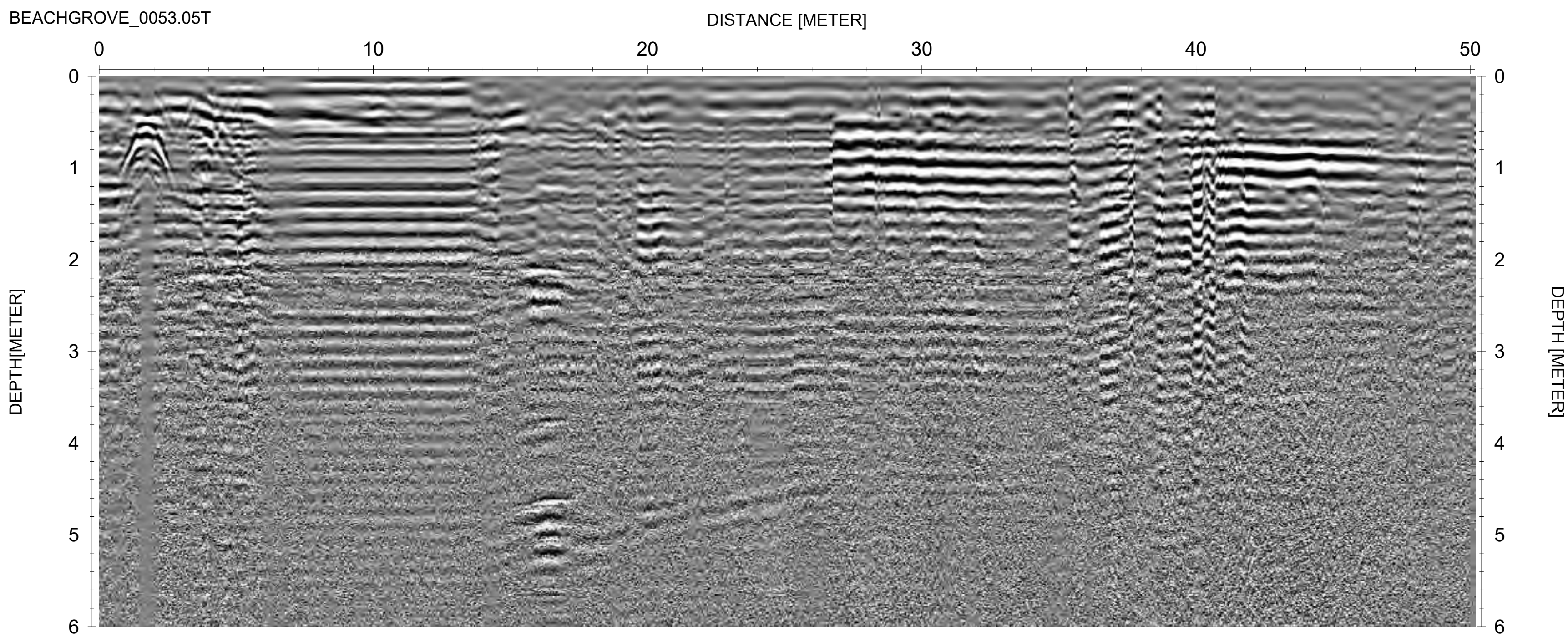
4
5
6

BEACHGROVE_0052.05T




BEACHGROVE_0053.05T
DISTANCE [METER]
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30
40
50
DEPTH[METER]
DEPTH [METER]
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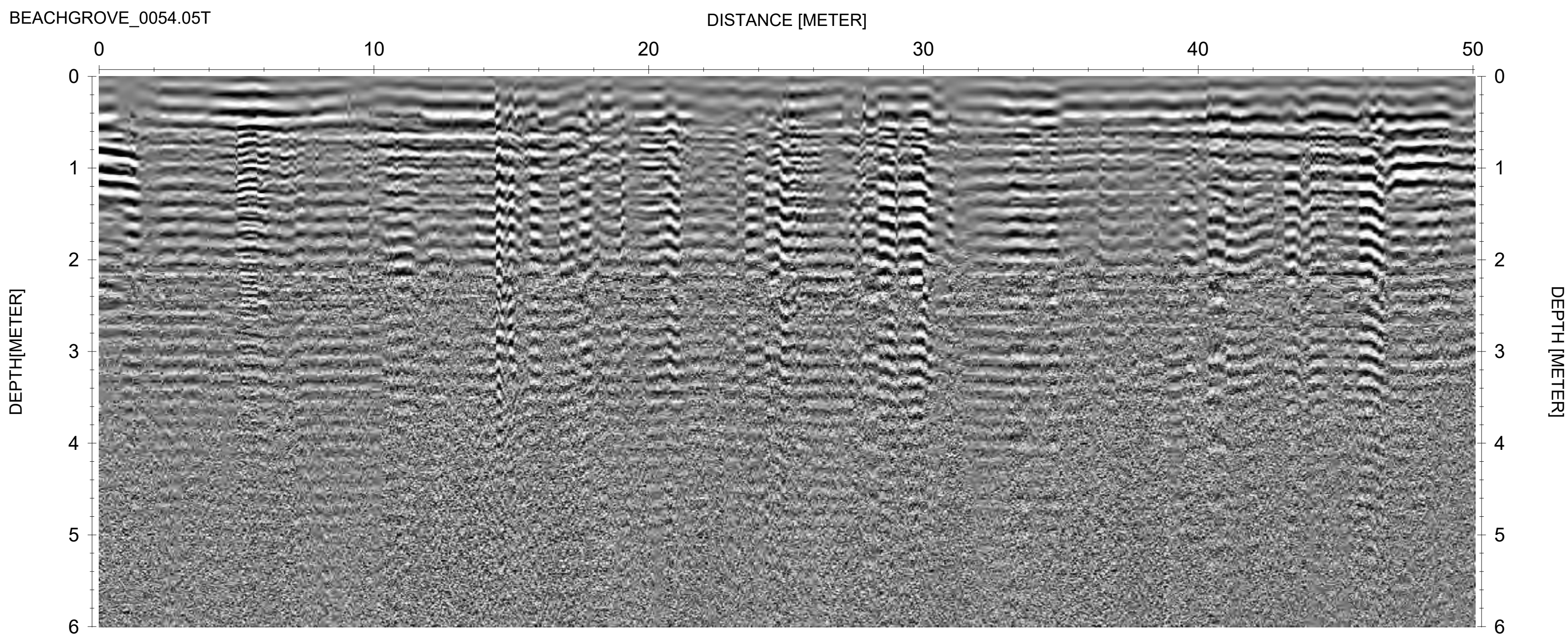
1
2
3
4
5
6


BEACHGROVE_0054.05T
DISTANCE [METER]
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50
DEPTH[METER]
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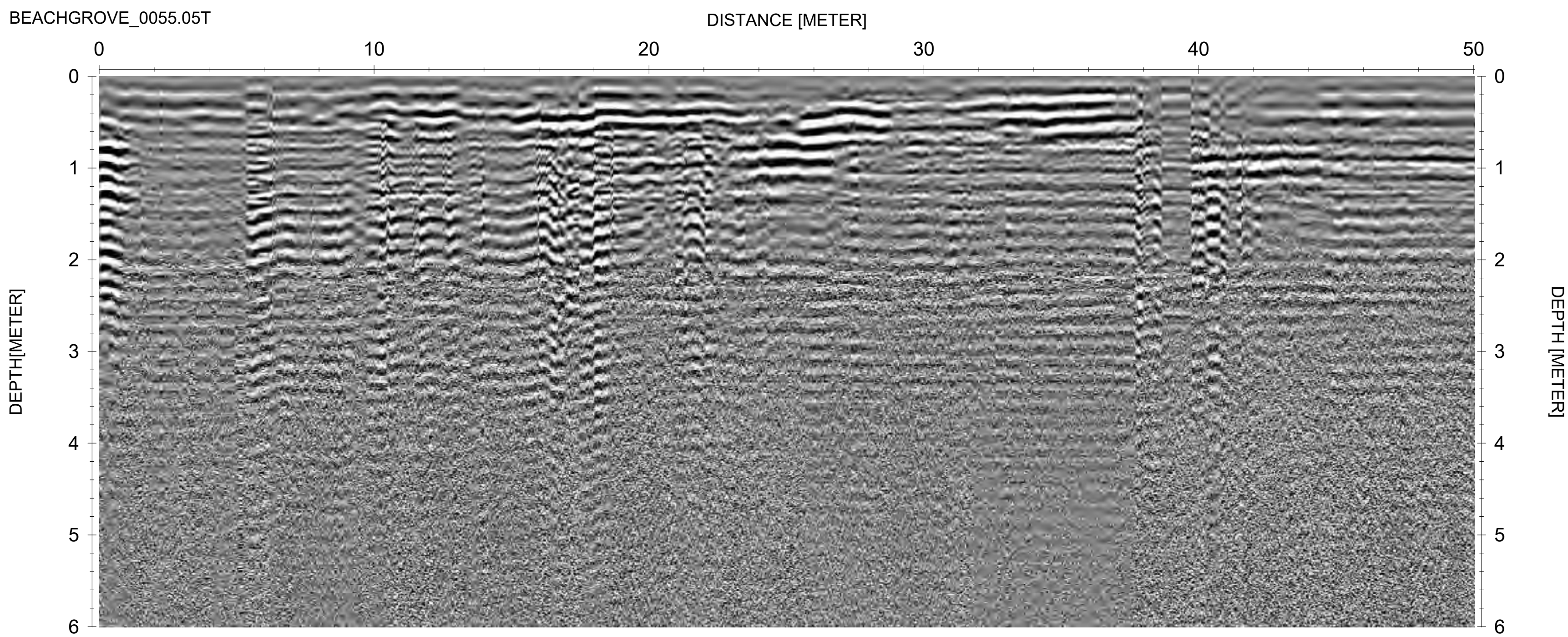
2
3
4
5
6
DEPTH [METER]


BEACHGROVE_0055.05T
DISTANCE [METER]
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30
40
50
DEPTH[METER]
DEPTH [METER]
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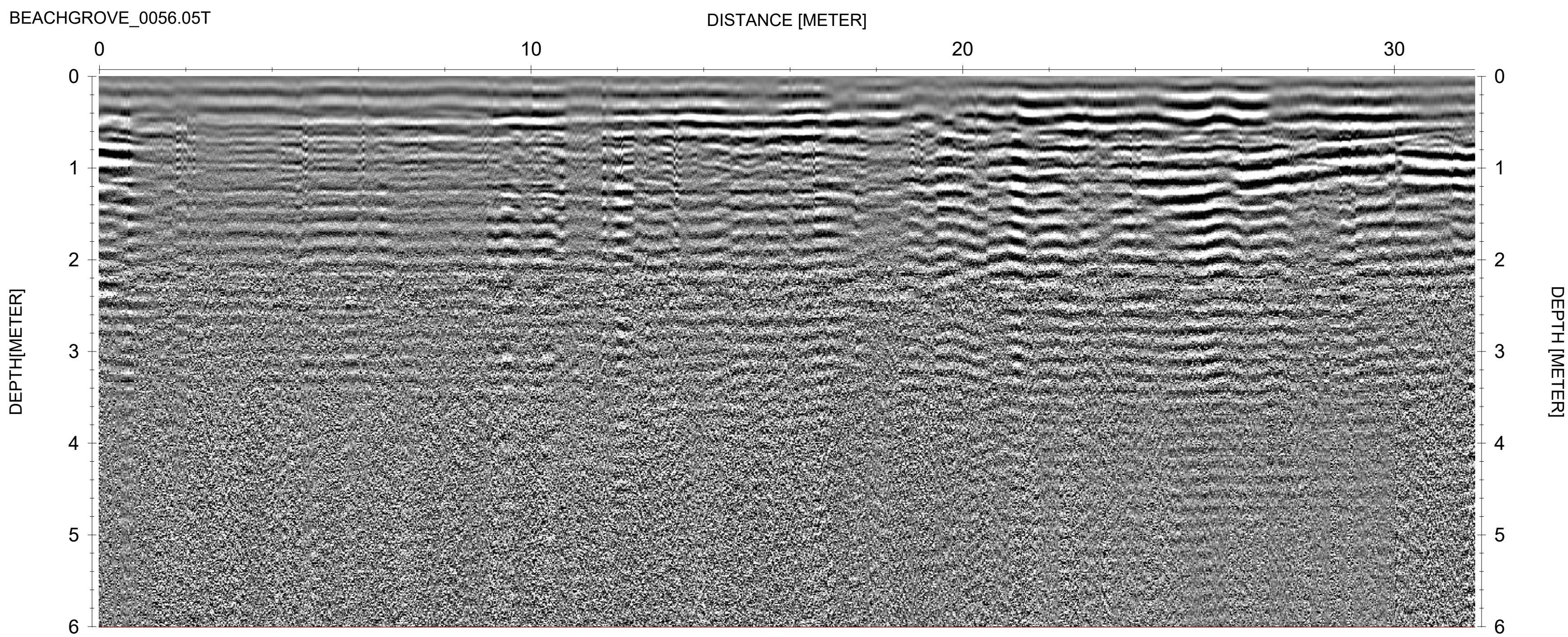
2
3
4
5
6


BEACHGROVE_0056.05T
DISTANCE [METER]
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30
0
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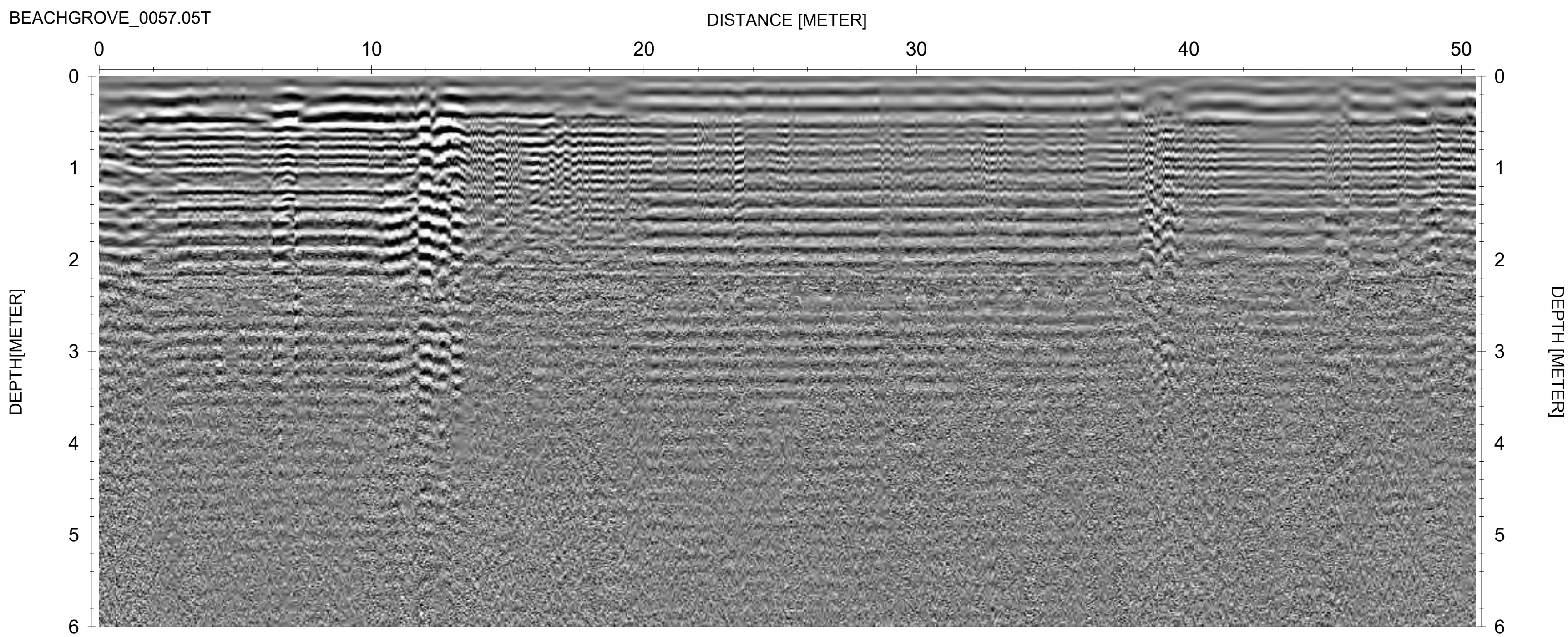
2
3
4
5
6
DEPTH[METER]
DEPTH [METER]


BEACHGROVE_0057.05T
DISTANCE [METER]
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30
40
50
DEPTH[METER]
DEPTH [METER]
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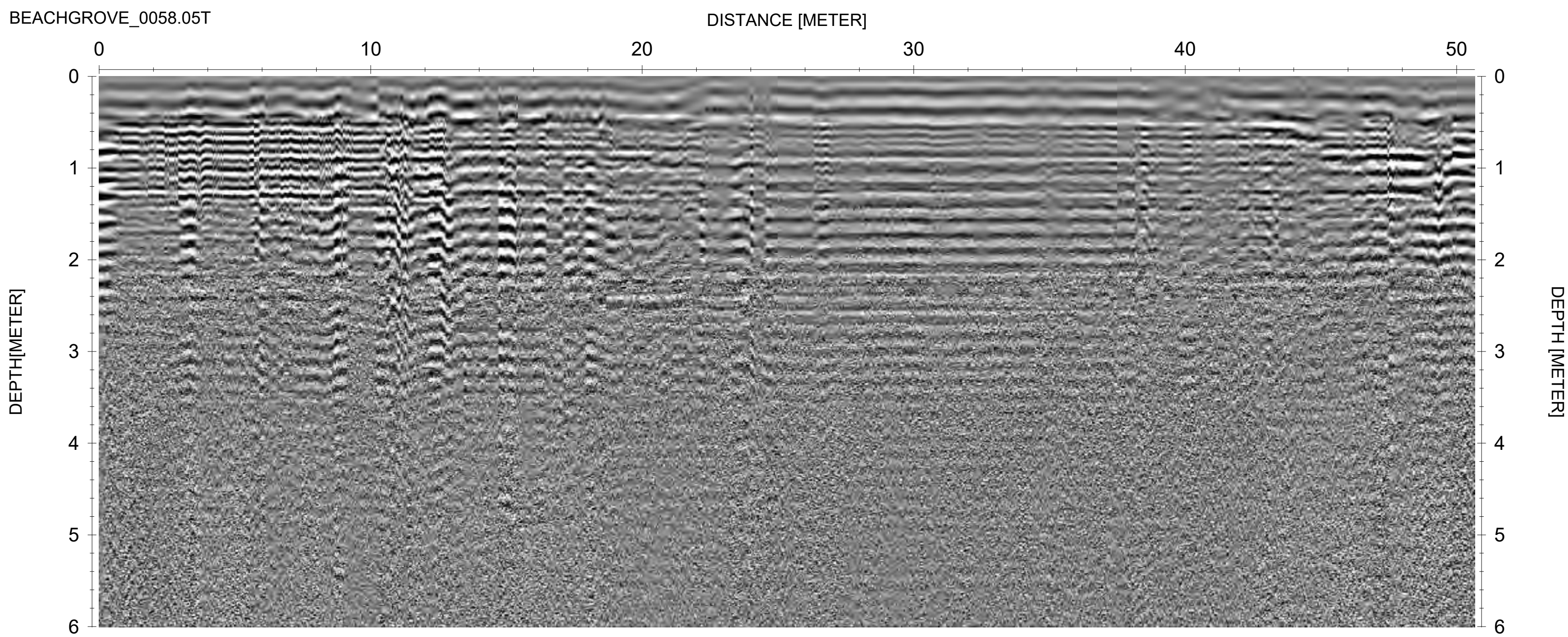
1
2
3
4
5
6


BEACHGROVE_0058.05T
DISTANCE [METER]
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50
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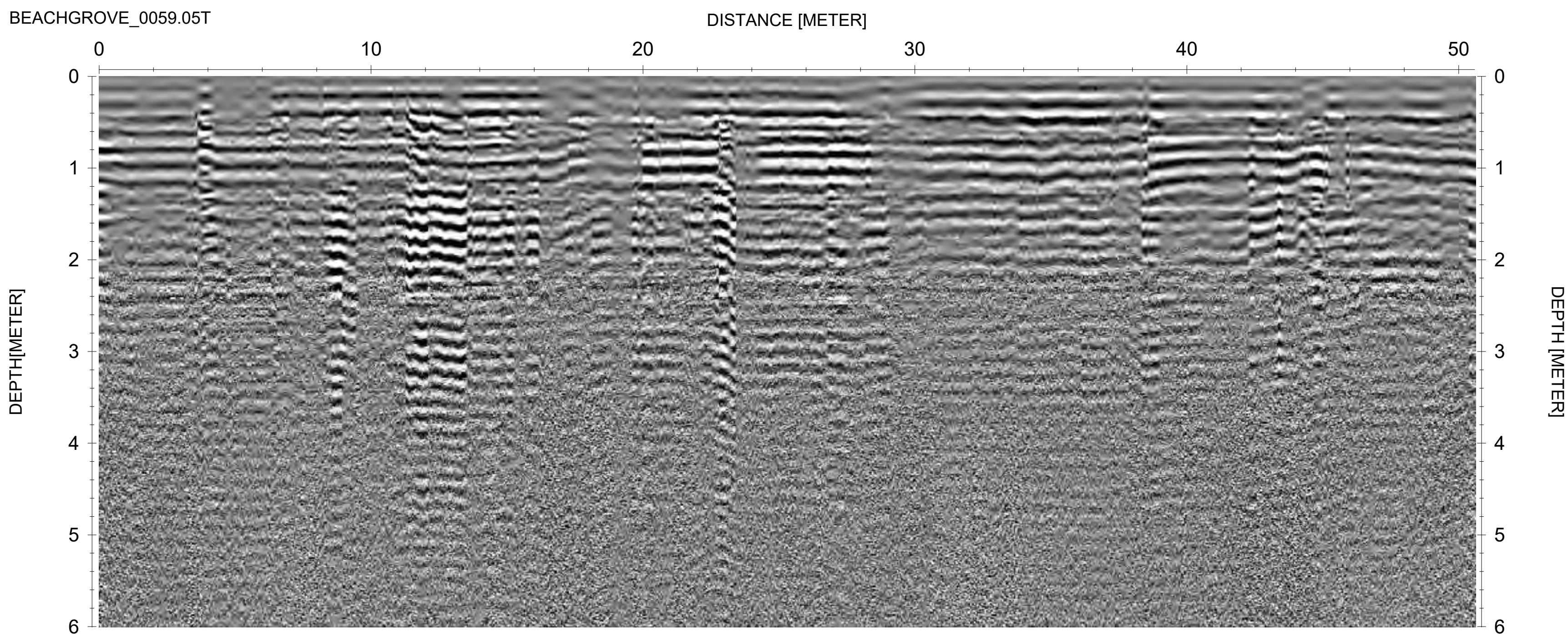
2
3
4
5
6
DEPTH[METER]
DEPTH [METER]


BEACHGROVE_0059.05T
DISTANCE [METER]
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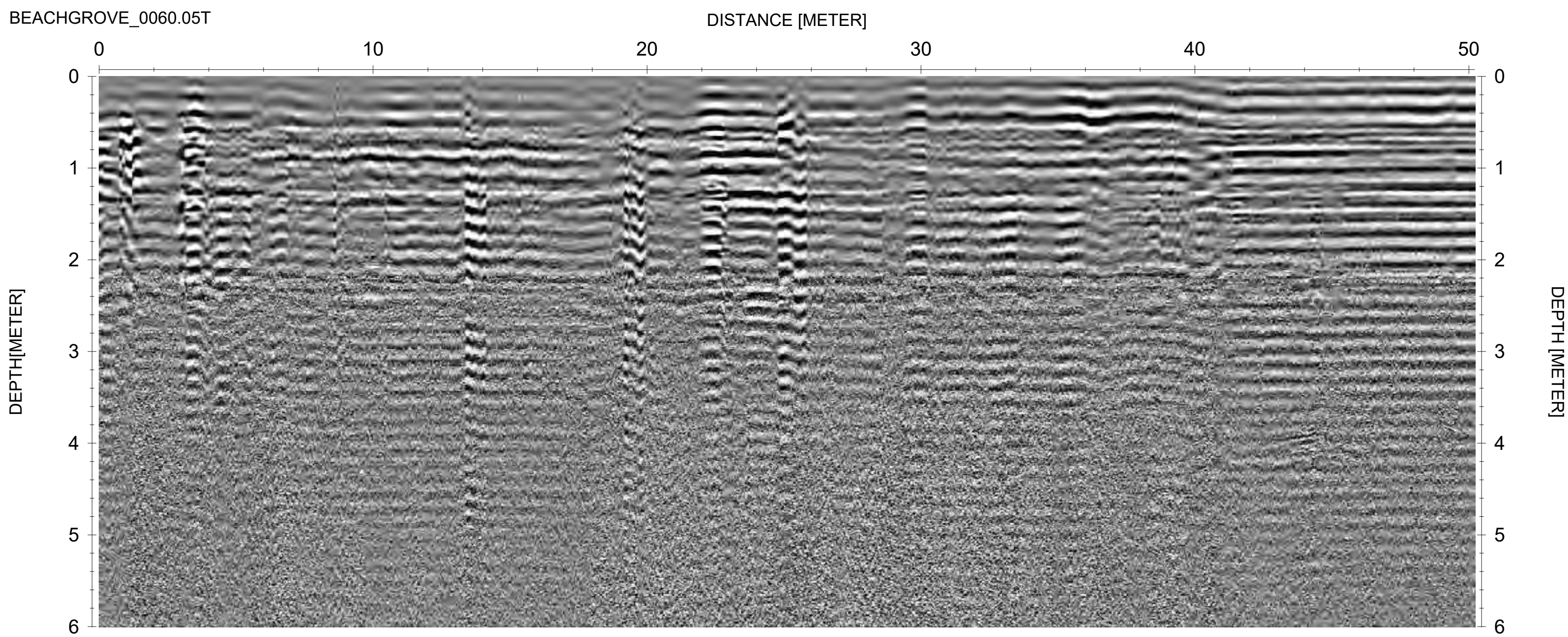
40
50
0
1
2
3
4
5
6
DEPTH[METER]
DEPTH [METER]


BEACHGROVE_0060.05T
DISTANCE [METER]
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20
30
40
50
DEPTH[METER]
DEPTH [METER]
0
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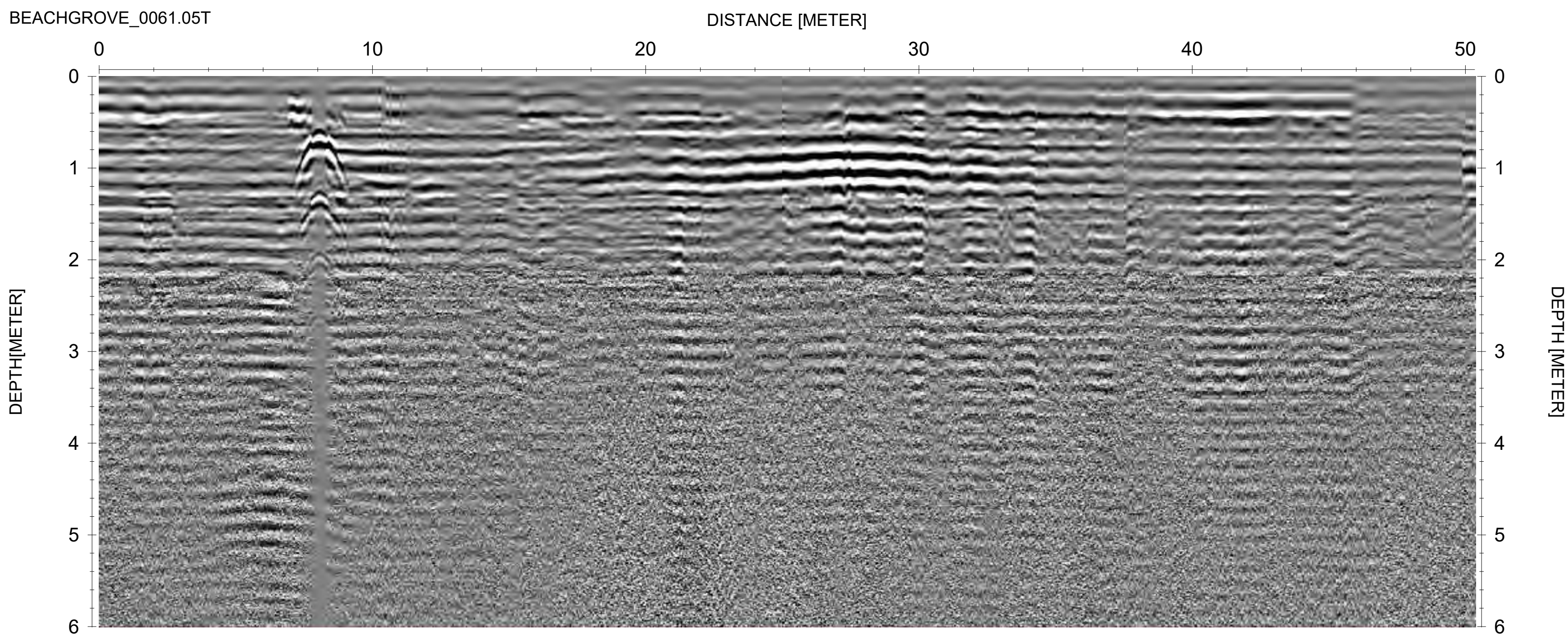
2
3
4
5
6



BEACHGROVE_0061.05T
DISTANCE [METER]
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30
40
50
DEPTH[METER]
DEPTH [METER]
0
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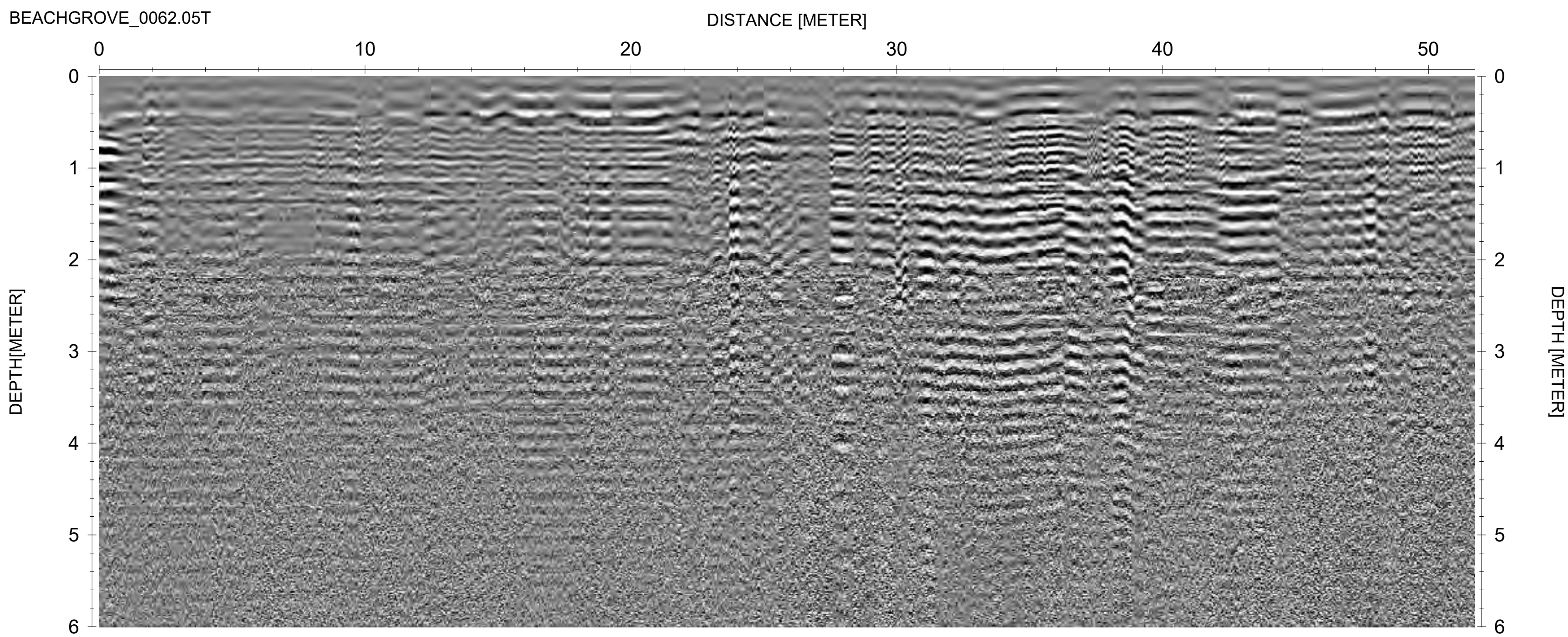
2
3
4
5
6



BEACHGROVE_0062.05T
DISTANCE [METER]
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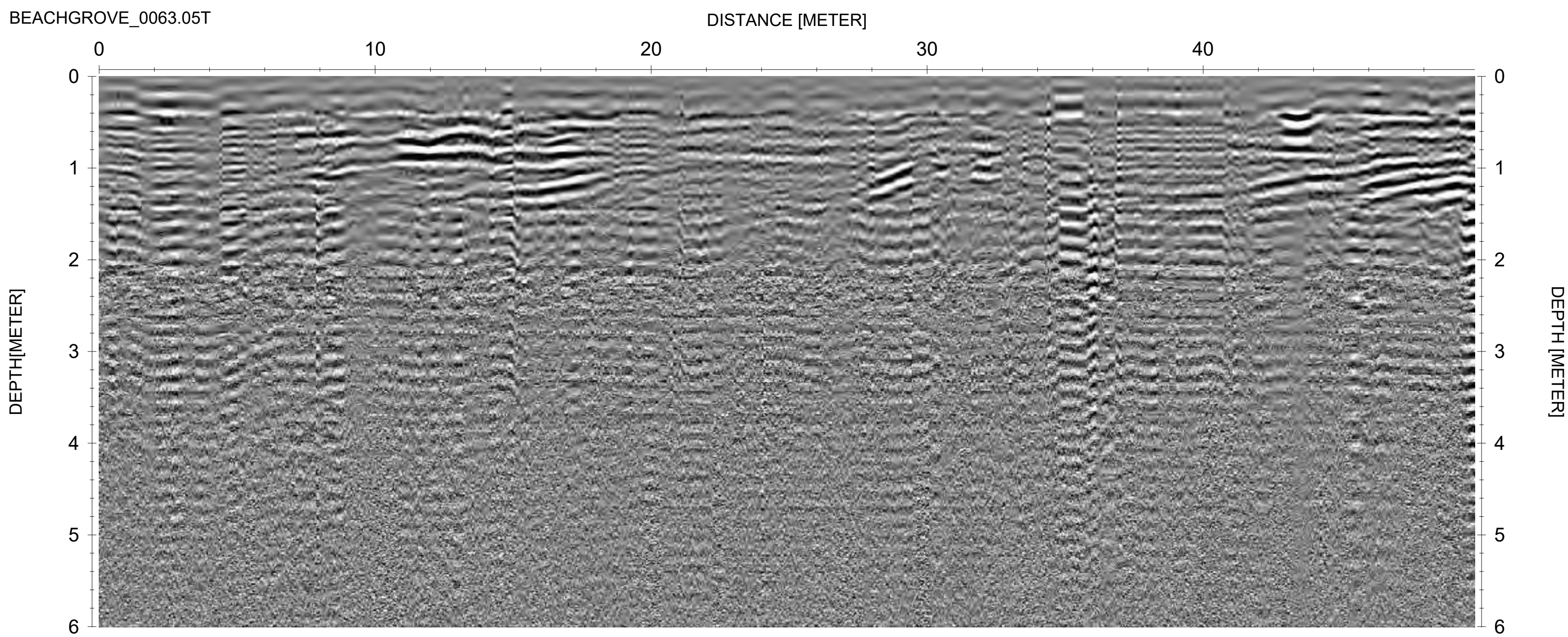
20
30
40
50
DEPTH[METER]
DEPTH [METER]
0
1
2
3
4
5
6


BEACHGROVE_0063.05T
DISTANCE [METER]
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10
20
30
40
DEPTH[METER]
0
1
2
3
4
5
6
DEPTH [METER]

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