REPORT

Te Ararata Flood Resilience Works - Walmsley Road Bridge Replacement

Geotechnical and Groundwater Assessment Report

Prepared for
Auckland Council
Prepared by
Tonkin & Taylor Ltd
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October 2024	1.0	Geotechnical Resource Consent Assessment Report	Z. Liu	P. Malan	P. Malan
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1 Introduction

Tonkin & Taylor Ltd (T+T) has been engaged by Auckland Council's Healthy Waters department (Healthy Waters) to undertake an assessment of geotechnical and groundwater effects for the proposed Te Ararata Walmsley Road bridge replacement works (the Project). The Project is flood resilience works, with the assessment prepared to support a resource consent application under the Severe Weather Emergency Recovery (Auckland Flood Resilience Works) Order 2024.

This report assesses the construction and operational geotechnical and groundwater effects of the Project based on an indicative construction methodology and concept design developed to support the resource consent application. In particular, the proposed works of relevance and outlined in this report includes:

- Construction of a new replacement Walmsley Road bridge over Te Ararata Creek.
- Construction of a temporary bailey bridge to the north of the existing Walmsley Road bridge, facilitating pedestrian and active mode diversions between Coronation Road and Walmsley Road'.
- Demolition and removal of the existing twin culvert (excluding the culvert base).
- Constructing retaining wall structures (up to 3.5 m high) and reshaping the creek channel at both upstream and downstream sides of the bridge to achieve the final creek profile, with works extending to tie into the existing riverbank slope.
- Relocation of the existing Watercare watermain pipe bridge foundations to achieve a wider clearance beneath the structure.
- Relocate or realign existing underground services.

1.1 Site description

The existing Walmsley Road bridge structure is located on Walmsley Road in Māngere, Auckland. It is a two-lane road structure that spans the Te Ararata Creek, located at the southern edge of both the Black Bridge Reserve and Walmsley Road Reserve, which lie to the west and east of the creek respectively. Residential neighbourhoods are situated to the east of Walmsley Road Reserve and south of the bridge.

The existing Walmsley Road bridge structure is located on Walmsley Road in Māngere, Auckland. It is a two-lane road structure that spans the Te Ararata Creek, located at the southern edge of both the Black Bridge Reserve and Walmsley Road Reserve, which lie to the west and east of the creek respectively. Residential neighbourhoods are situated to the east of Walmsley Road Reserve and south of the bridge.

Figure 1.1: Project location – Site plan.

The existing structure also comprises a concrete twin culvert beneath the road, featuring two rectangular openings, each 2.5 m in width and 3.7 m in height. A solid pier separates two openings. The culvert is oblique to the road.

1.2 Proposed works

A detailed description of the full project works can be found in the assessment of effects on the environment (AEE) report. Detailed design of the Walmsley Road Bridge Upgrade will be undertaken in accordance with NZTA bridge manual[1].

This report considers effects associated with the following construction activities that are identified to be relevant to geotechnical or groundwater related effects[2]:

- Installing driven pile foundations with an outer diameter not exceeding 1.5 m for the new bridge.
- Excavation from the existing road surface associated with the construction of the permanent bridge pile caps (comprising reinforced concrete beams spanning the width of the road), wingwalls and settlement slab footprints.
- Initial excavation to expose the existing culvert structure walls and the subsequent removal of the existing culvert after the permanent bridge abutments are constructed. This will be undertaken progressively and will occur with one half of the culvert removed first, followed by the other half. The culvert base and the side walls up to 0.5 m high on each side of the culvert opening will remain in place. Temporary slope retention may be installed close to the culvert headwalls during excavation and removal of the culvert structure.

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[1] NZ Transport Agency, *Bridge Manual SP/M/022*, Third edition, Amendment 4, Effective from May 2022

[2] A full description of the construction methodology is provided in the AEE.

- Recontouring of the stream banks (including a 3H:1V batter slope), installation of rock armouring and construction of permanent retaining walls, under the bridge and immediately upstream and downstream of the bridge to achieve the final stream profile. These works will tie into the existing creek bank slope.
- The piers supporting the existing Watercare watermain pipe, which is located north of the existing culvert structure, will be replaced by new foundations positioned behind the new retaining structures at either side of the creek.
- To enable the bridge construction, a temporary bailey bridge to accommodate pedestrian and active mode diversion movements is proposed north of the permanent bridge.

Figure 1.2 below presents the locations of main construction activities relevant to geotechnical or groundwater matters.

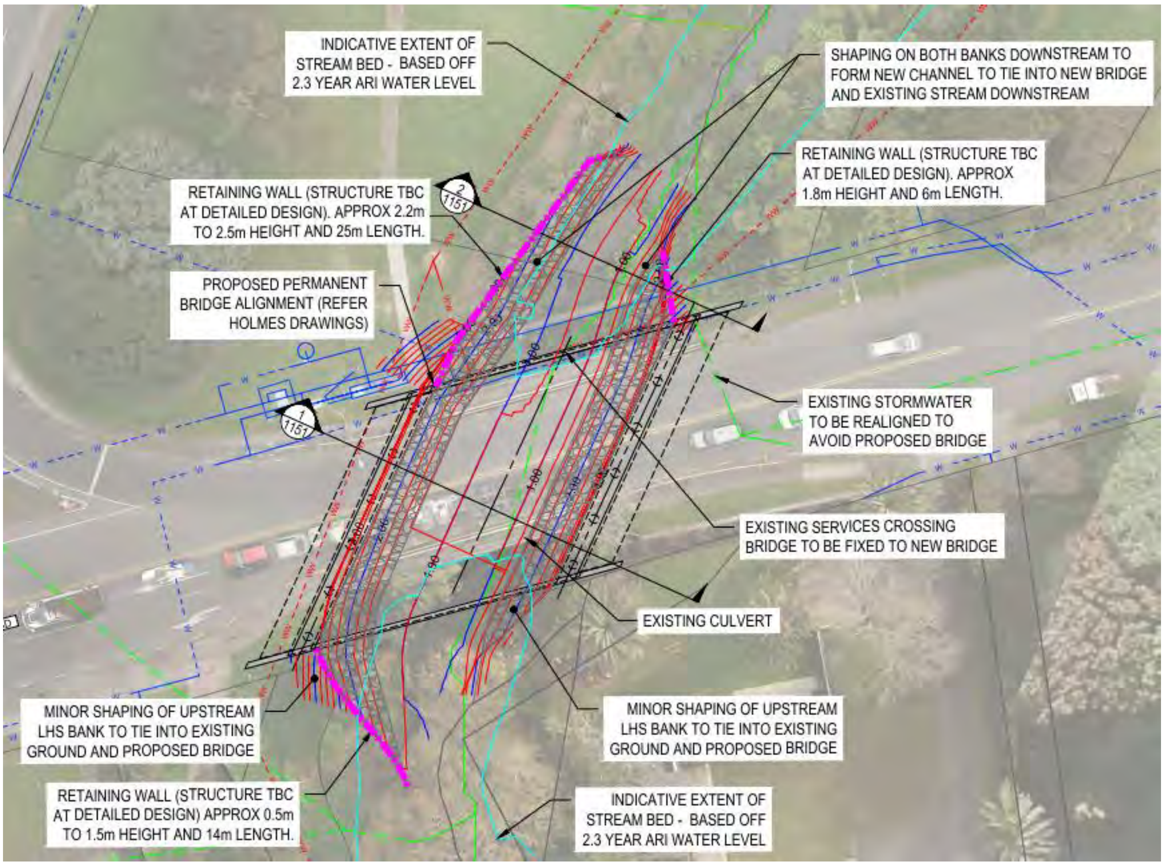

Figure 1.2: Construction works relevant to Walmsley Road bridge replacement relevant to geotechnical or groundwater matters.

2 Site conditions

2.1 Summary of geological conditions

The site of the proposed works is underlain by Auckland Volcanic Field (AVA) and Pliocene to Holocene Takaanini Formation.

Geotechnical investigations and a review of historical investigation reports and published geological information was undertaken to confirm the geological conditions of the site.

Site specific geotechnical investigations comprised:

- 2 cone penetrometer tests (CPTs) to refusal at depths of 20.9 to 23.0 m.
- 1 hand augered boreholes (HAs) to depth of 5.2 m.
- 2 machine drilled boreholes (BHs) with depths 34.8 and 6.0 m respectively, with down-hole shear wave velocity testing undertaken within one of the BHs.

Figure 2.1 presents the location of geotechnical investigations undertaken for the project, as well as the location of the historical investigations.

More information on the geotechnical investigations and testing is included in Appendix A.

Figure 2.1: Location of geotechnical investigations within the site.

A summary of the underlying ground conditions around the Walmsley Road bridge based on the available investigation data is presented in Table 2.1 and a simplified ground profile is shown on Figure 2.2.

Table 2.1: Summary of subsurface ground conditions around the Walmsley Road bridge

Geological unit	Soil description	Typical elevation of surface of layer (mRL)	Typical depth to surface of layer (mbgl)	Typical layer thickness (m)
Topsoil[1]	Dark brown firm SILT.	-	0	0.1 – 0.3
Fill	Dark greyish brown stiff SILT with some gravel or cobbles.	6.3 – 7.2	0 - 0.1	0.4-1.5
Auckland Volcanic Field	Stiff to very stiff CLAY or SILT.	3.7 - 6.7	0 – 1.5	0.6-3.7
	Slightly weathered BASALT (western abutment side only).	4.0 – 5.8	1.0 -1.7	2.5 - 3.3
Takaanini Formation	Dark grey firm to very stiff clayey or silty SILT with lens of organic clay.	2.1 – 5.4	1.5 – 4.5	5.4 – 8
	Dark grey silty SAND or sandy SILT, generally medium dense or very stiff to hard, occasionally interbedded with loose or firm lens.	-2.4 - -3.5	9 – 10.5	5 – 7.8
	Dark grey stiff to hard SILT, interbedded with medium dense silty sand.	-8.2 – -11.7	14 – 18.3	5 – 8
	Dark grey dense to very dense SAND.	-16.2 - -17.1	22 - 24	> 10

2.2 Groundwater

The groundwater assessment is set out in detail in Appendix C. Groundwater levels were measured at various locations around the bridge site. These measurements showed groundwater levels across the site ranging from 1.4 to 3.4 mbgl (+1.9 to +3.0 mRL) in July and August 2024 (i.e. during winter). Groundwater is measured to generally follow the ground surface and flows towards the Te Ararata creek. This is consistent with normal groundwater regimes around waterways. A more detailed assessment is presented in Appendix C.

For the purposes of this assessment, a groundwater level of + 3.0 mRL (3 mbgl) has been adopted at the bridge pile locations on both abutments, tapering down to +0.9 mRL at the creek. This is shown on Figure 2.2. We consider this to be a reasonably representative (winter) groundwater level at project site prior to the proposed development. We have used this for assessing whether the proposed works interact with groundwater, noting that in summer (and in droughts) the groundwater levels will be lower than modelled.

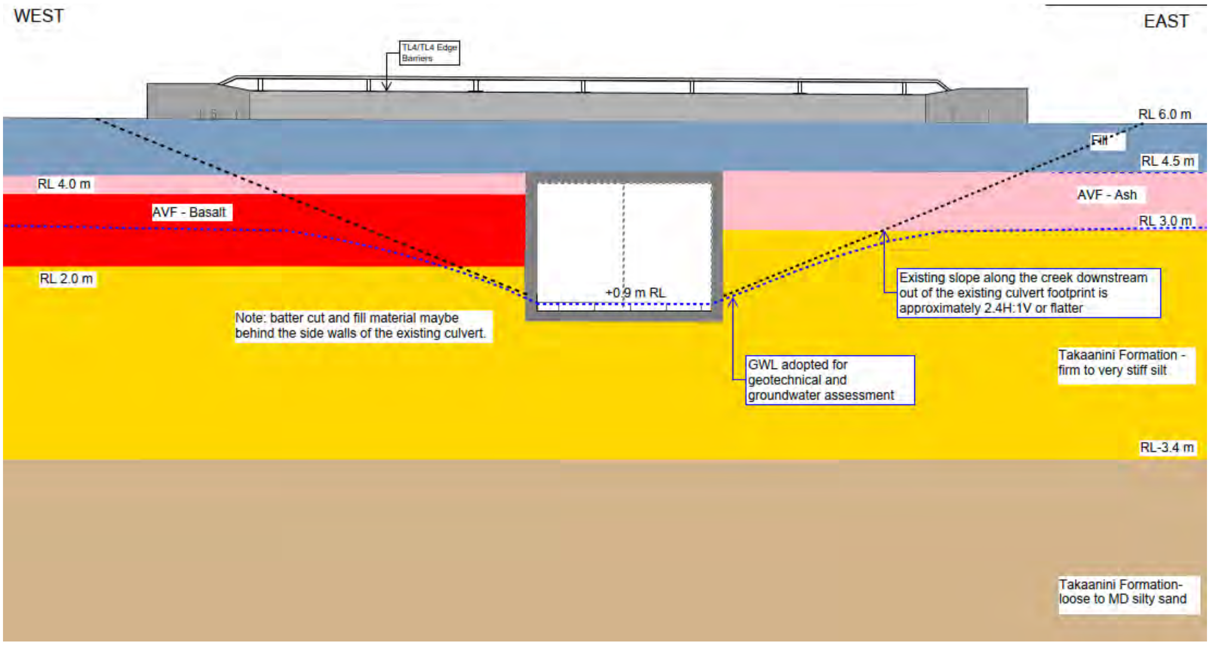

Figure 2.2: Simplified ground profile along bridge long section based on the available investigation information.

3 Effects assessment

3.1 General

Based on the current concept design and subsurface conditions, the key geotechnical and groundwater considerations and assessment for this site are:

- Seismic performance.
- Instability of abutment slopes.
- Potentially compressible soils and settlements.
- Groundwater.
- Excavation effects.

3.2 Seismic considerations

We have assessed the liquefaction susceptibility of the project site using the Boulanger & Idriss (2016) method and cone penetration test data. The assessment indicates that the soil layers from 10 m to 14 m beneath the site to be at risk of liquefaction during long (1000 year) return period seismic loads. Any potential adverse liquefaction effects on the bridge will be mitigated during the detailed design using normal design approaches. The proposed bridge itself will not affect the liquefaction potential of the soil or the surrounding sites.

3.3 Geotechnical assessment of land subject to instability

Chapter J1 of the Auckland Unitary Plan (AUP) defines land as being subject to instability if ground comprising Holocene or Pleistocene sediments are at a slope of 4H: 1V or steeper. The existing creek banks are underlain by Holocene to Pleistocene Takaanini Formation, with slopes of approximately 2.4H:1V or steeper in the proximity to the existing culvert structure per the current available Lidar data. That means the existing slopes trigger both elements, and therefore the stream banks are identified as 'land that may be subject to land instability' under the AUP.

The proposed infrastructure works located on land that may be subject to instability (defined above) comprise:

- Removal of the existing culvert structure.
- Construction of the permanent bridge abutments.
- Building retaining structures on both the upstream and downstream sides of the new bridge.
- Stream channel shaping extending to tie into the existing creek channel.
- Construction of new foundations to support the Watercare watermain.
- Construction of a temporary pedestrian bailey bridge.

To assess the effects of the works on the land, a quantitative slope stability assessment has been carried out.

We have analysed the stability of the east side (including both bridge abutment and retaining structure) which we consider worse in terms of land instability in the proposed development work (compared to the western side). If the eastern side is stable, the western will also be stable asthe excavation at the western side will be partially in basalt material which is self- supporting rock. Any local rock defects will be identified and removed during construction.

The new pile foundations of the Watercare watermain pipe will improve (and not adversely affect) the slope stability, but we have not allowed for this beneficial effect in our assessment.

The pedestrian bailey bridge is a temporary structure that will be designed to meet appropriate stability requirements for its purpose. The detailed design will consider the slope stability, geometry and foundation interaction. No adverse effects on instability are expected to be present while the structure is temporarily in place and after it is demolished.

The stability analysis has been conducted using the limit state equilibrium numerical software Slide 2 (version 9.034, developed by Rocscience). Material parameters for the stability analysis were selected based on site investigations and previous project experience with similar materials at adjacent locations. Further details of the material parameters, and model outputs are included in Appendix B.

The load cases and target factors of safety outlined in section 2.6.8 from Chapter 2 of the Auckland Code of Practice for Land Development and Subdivision (version 2.0, May 2023) were considered, as follows:

- Factor of safety of 1.5 for normal groundwater conditions.
- Factor of safety of 1.3 for the worst credible groundwater condition.
- Factor of safety of 1.0 for the pseudo-static seismic loading using ULS PGA (0.19g corresponding to 1000-year return period).

A summary of the stability analysis results is presented in Table 3.1. Each of the model scenarios meet the target factors of safety.

Table 3.1: Summary of slope stability modelling outputs

Design scenario	Factor of safety results	Factor of safety requirement
Bridge Abutment		
Long-term static	1.5	1.5
Worst credible groundwater level	1.4	1.4
Pseudo-static seismic	1.1	1.0
Retaining structure		
Long-term static	2.0	1.5
Worst credible groundwater level	2.0	1.3
Pseudo-static seismic	1.2	1.0

During construction, temporary excavations will be undertaken as part of the proposed development. The excavation depths will reach up to approximately 5.8 m below the existing road surface, and approximately 12.5 m and 24 m away from the nearest property boundary on east and west sides respectively. The contractor will undertake normal measures such as retention, benching or battering to mitigate the risk of instability during the temporary excavations.

Based on the analysis and our site observations, we consider the land meets the target factors of safety against slope instability. It is therefore not land subject to instability and does not adversely affect land potentially subject to instability.

3.4 Load-induced settlement

The permanent bridge is to be designed and constructed following the existing grade and levels along Walmsley Road. The fill placement behind the permanent bridge abutment is very minor and the associated settlement is therefore negligible. We note that the excavation for the bridge is a net unloading of the site.

3.5 Groundwater effects

3.5.1 General

The groundwater profile adopted for this assessment (set out in Section 2.2) and the proposed permanent bridge upgrade work are shown on Figure 3.1.

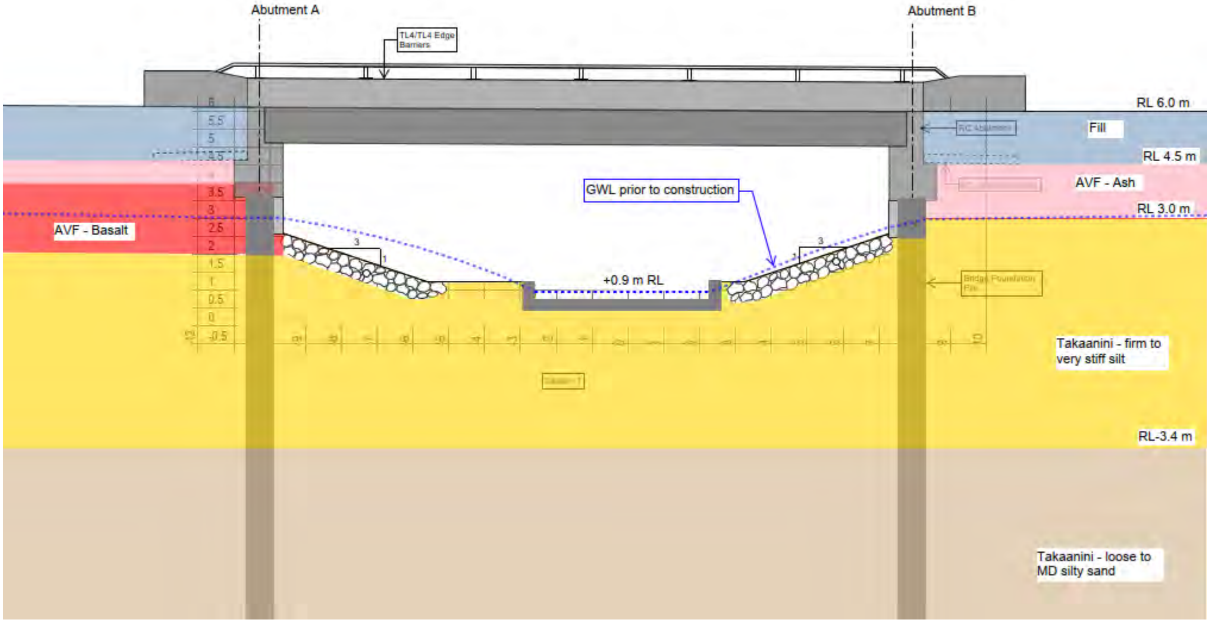

Figure 3.1: Proposed bridge upgrade work and groundwater profile adopted for assessment.

Based on adopting a typical winter groundwater level of +3 mRL (3 mbgl at proposed bridge abutment location, as discussed in section 2.2 above), Table 3.2 below sets out the proposed activities that will encounter groundwater as well as those that will not.

Table 3.2: Summary of construction activities and associated groundwater interception

Proposed construction activities	Indicative Duration	Above or below groundwater level adopted for assessment (3 mbgl)	Expected groundwater interception?
Piling for the permanent bridge structures, the replacement of the existing Watercare watermain bridge foundations (if required), and temporary bailey bridge (if required) - piles less than 1.5 m diameter.	3-4 weeks for each end of the permanent bridge 2-3 weeks for each end of the Watercare watermain bridge 2 – 3 weeks for each end of the temporary bailey bridge	Below groundwater level	No
Construction for the temporary bailey bridge (both sides) – shallow abutments.	2 – 3 weeks	Above groundwater level	No

Proposed construction activities	Indicative Duration	Above or below groundwater level adopted for assessment (3 mbgl)	Expected groundwater interception?
Construction for the permanent bridge abutment (one side below water table).	25 days (but for flexibility assume 30 days)	Below groundwater level	Yes
Construction for permanent bridge abutment (other side below water table).	25 days (but for flexibility assume 30 days)	Below groundwater level	Yes
Construction for the existing Watercare watermain bridge abutment replacement (both sides) – shallow foundations.	1 – 2 months	Above groundwater level	No
Excavation to the top of the culvert.	1-2 weeks	Above groundwater level	No
Excavation to initially expose culvert walls.	3- 4 weeks	Below groundwater level	No
Remove sections of culvert.	3 – 4 weeks	Below groundwater level	No
Cut for final stream profile, rip rap installation and retaining walls.	30 – 40 days (total over both sides of the stream)	Below Groundwater level	Yes

An assessment against the relevant AUP standards has been undertaken for the activities identified above likely to extend below groundwater level is provided in Appendix E. In summary:

- The proposed piles are less than 1.5 m in diameter. As such, they are exempt under Standard E7.6.1.10(1)(c) of the AUP from meeting Standards E7.6.1.10(2) – (6) relating to groundwater diversion.
- The excavation associated with constructing the permanent bridge abutments, excavation to expose the existing culvert wall and its removal, and final stream reshaping (including installation of rip rap and retaining walls) do not meet the exemption under Standard E7.6.1.10 (d) which states diversions for no longer than 10 days, therefore assessment against Standards 7.6.1.10 (2) - (6) is required. Assessment against Standards E7.6.1.10(2) – (6) demonstrates compliance with the standards.
- The groundwater interception associated with excavations for the permanent bridge abutments and final stream reshaping (including installation of rip rap and retaining walls) meet all requirements of Standard E7.6.1.6, except clause (2) as they each potentially exceed the maximum 30 day water take period. It is noted that each excavation can be considered a separate diversion.

Given the above, the following sections of assessment only consider any potential effects associated with dewatering required to construct the permanent bridge abutments and final stream reshaping.

3.5.2 Groundwater drawdown

A detailed discussion of groundwater profile in the project site is presented in Appendix C.

The proposed construction works in relation to the permanent bridge abutments and final stream profile involve excavations which will lower the permanent ground surface to around 0.5 m below the current winter groundwater profile (i.e. RL +2.5 m). The deepest drawdown is expected to be at the base of the rock riprap adjacent to the abutment during the excavation work. Making an allowance of 1 m thickness for undercut, drainage and riprap thickness means an excavation to around RL +1.5 m is expected to be required (i.e. a reduction of approximately 1.5 m on groundwater levels in the winter at the bridge abutment). The geometry is shown in Figure 3.2 below (noting the rip rap is shown to be 0.5 m thick on Figure 3.2).

In undertaking the groundwater effects assessment, we have taken an envelope of effects approach in order to allow for any uncertainty in estimating effects and groundwater levels. We have assessed effects based on a winter groundwater drawdown of 2 m to provide a conservative effects assessment (to RL +1 m, at least 0.5 m more than expected). The drawdown that the assessment is based on is shown on Figure 3.2 below.

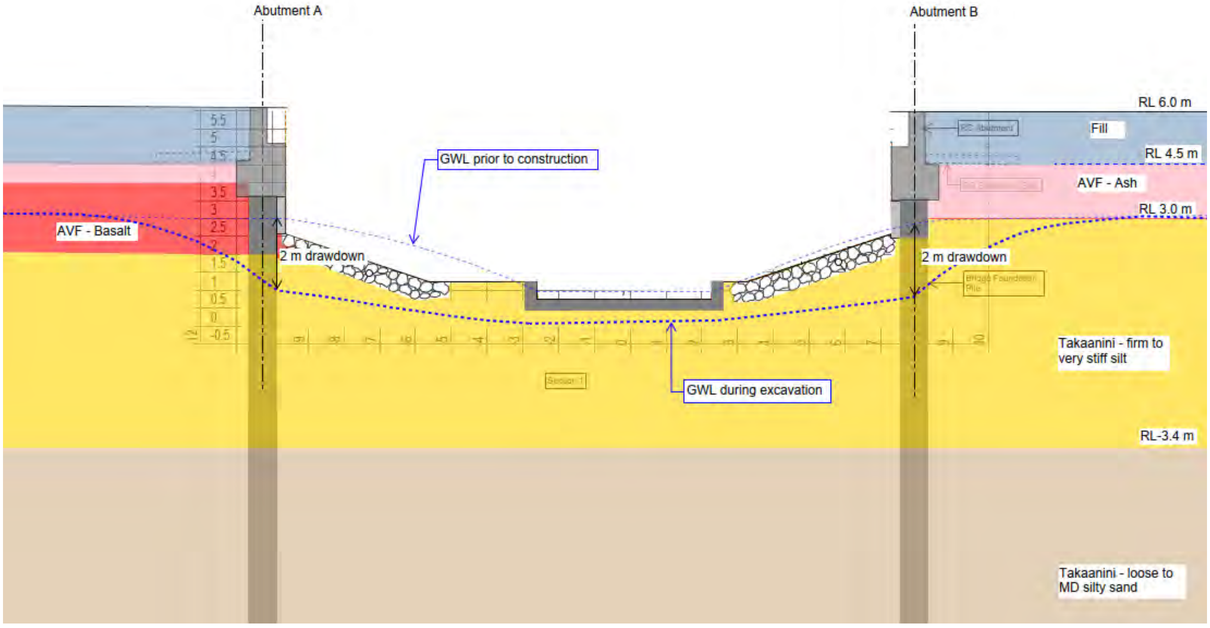

Figure 3.2: Maximum 2 m drawdown consideration during construction in the winter.

This drawdown assessment has significant conservatism built in:

- Allows permanent drainage to +1.0 mRL (when we expect it to be around +1.5 mRL or higher) in front of the abutment piles.
- In terms of drawdown effects, considers no beneficial effects from the seasonal reductions in groundwater table (i.e. conservatively models 2 m drawdown for settlement and radius of effect assessment in both winter and summer conditions).
- Does not consider any previous drawdown effects (for settlement assessment).
- Assumes the maximum drawdown applies to the entire soil profile (for settlement assessment).
- Does not consider the flow of groundwater towards the stream (for radius of effects assessment).
- Does not consider mitigation from unloading through excavation (for settlement assessment).

We therefore consider the assessment to be appropriately cautious for the extent of the proposed groundwater interception. The assessment concludes no consequential effects are identified based on the drawdown envelope adopted.

The influence of groundwater drawdown and the associated settlement effects are discussed in the following section.

3.5.3 Groundwater drawdown influence and induced settlement effects

The distances of groundwater drawdown influence have been assessed using the method outlined in Section 6, CIRIA C750[3] and the permeability of soil where groundwater level reduction occurs. The induced settlements from drawdown have been estimated using a linear elastic 1D settlement approach. The assessment results are presented in Table 3.3 below (refer Appendix C for calculation details). This shows 'worst case' drawdown settlements of 20 mm, reducing to negligible amounts within a few metres behind the bridge abutment and the proposed retaining structures. Note again the conservatism in the settlement estimates discussed in Section 3.5.2 above.

Table 3.3: Estimated maximum dewatering induced settlement

Location	Maximum GW level drawdown	Permeability of soil where groundwater drawn occurs	Distance of drawdown influence from bridge abutment and retaining structures(m)	Maximum drawdown settlement immediately behind retaining wall, mm
West side	2.0	1.0×10^{-4}	20	18
East side	2.0	2.0×10^{-6}	6	20

The drawdown settlement contours are presented in Figure 3.3 below. On this basis, the nearest property structures are located outside the assessed distance of groundwater drawdown influence. Refer to Appendix C for more discussion and analysis details.

With the potential settlement reducing with increasing distance, there is no credible mechanism to affect nearby structures with drawdown induced settlement.

The permanent drawdown influence zone after construction will also reduce. No further settlement will be induced by the permanent groundwater drawdown as we assess drawdown effects to be effectively completed during the construction period.

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[3] CIRIA, Groundwater Control: Design and Practice, 2nd ed. (London: CIRIA, 2016).

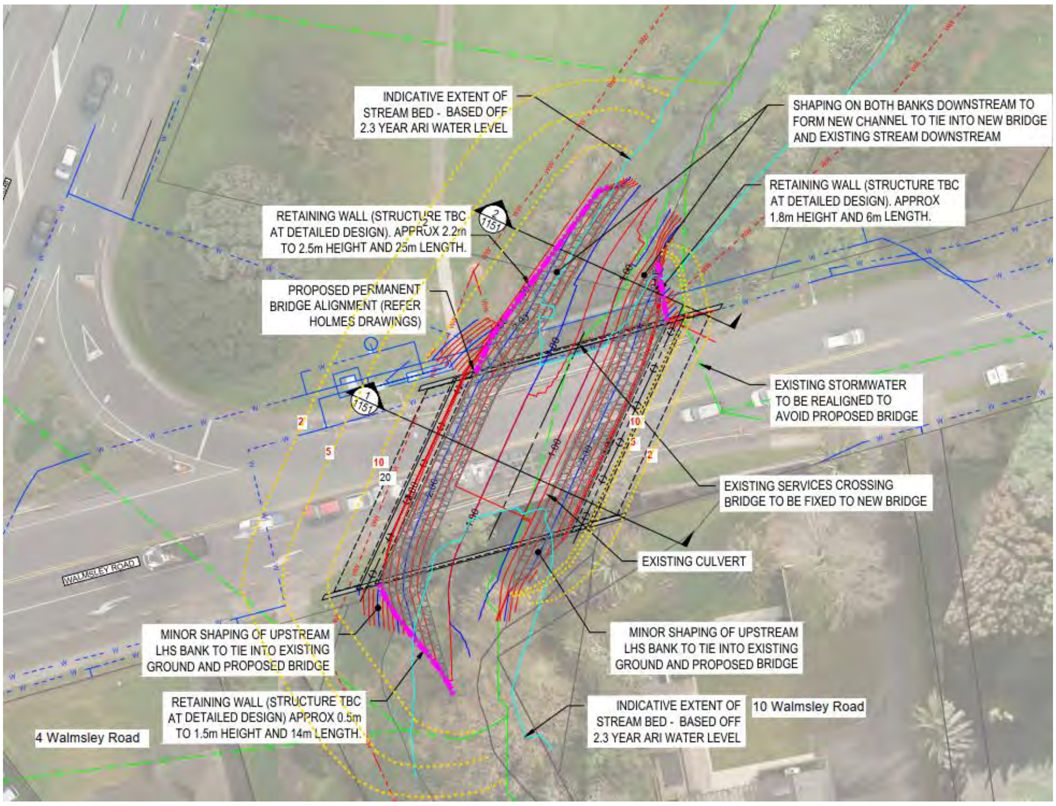

Figure 3.3: Drawdown settlement contours (mm, yellow contours with red notation) and nearby existing structures.

3.6 Mechanical deformation effects

Mechanical deformation effects may be caused by the following construction activities:

- The excavation in front of the bridge abutment prior to bridge deck installation.
- The final stream profile excavation works (including installation of rip rap and retaining structures) below the bridge and upstream and downstream of the bridge.

The mechanical settlements induced by these construction works have been estimated using the empirical methods outlined in CIRIA C760[4]. The assessment results are presented in Table 3.4 below (refer Appendix C for calculation details). This shows 'worst case' mechanical settlements of 20 mm, reducing to negligible amounts within about 15 m behind the bridge abutment and the proposed retaining structures.

Table 3.4: Estimated maximum mechanical settlement

Location	Maximum Excavation depth, m	Maximum calculated mechanical settlement immediately behind retaining wall, mm	Distance to assessed negligible mechanical settlement (m)
West side			
Bridge abutment	5.8	20	20

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[4]

Location	Maximum Excavation depth, m	Maximum calculated mechanical settlement immediately behind retaining wall, mm	Distance to assessed negligible mechanical settlement (m)
Downstream retaining structure (northwest)	4.2	15	15
upstream retaining structure (southwest)	3.2	11	11
East side			
Bridge abutment	5.8	20	20
Downstream retaining wall structure (northeast)	3.5	12	12

Given the setbacks (between four and seven times the maximum permanent retained height of 3.5 m), we assess the potential for mechanical deformation at a nearby structure as effectively nil, as presented on Figure 3.5 below.

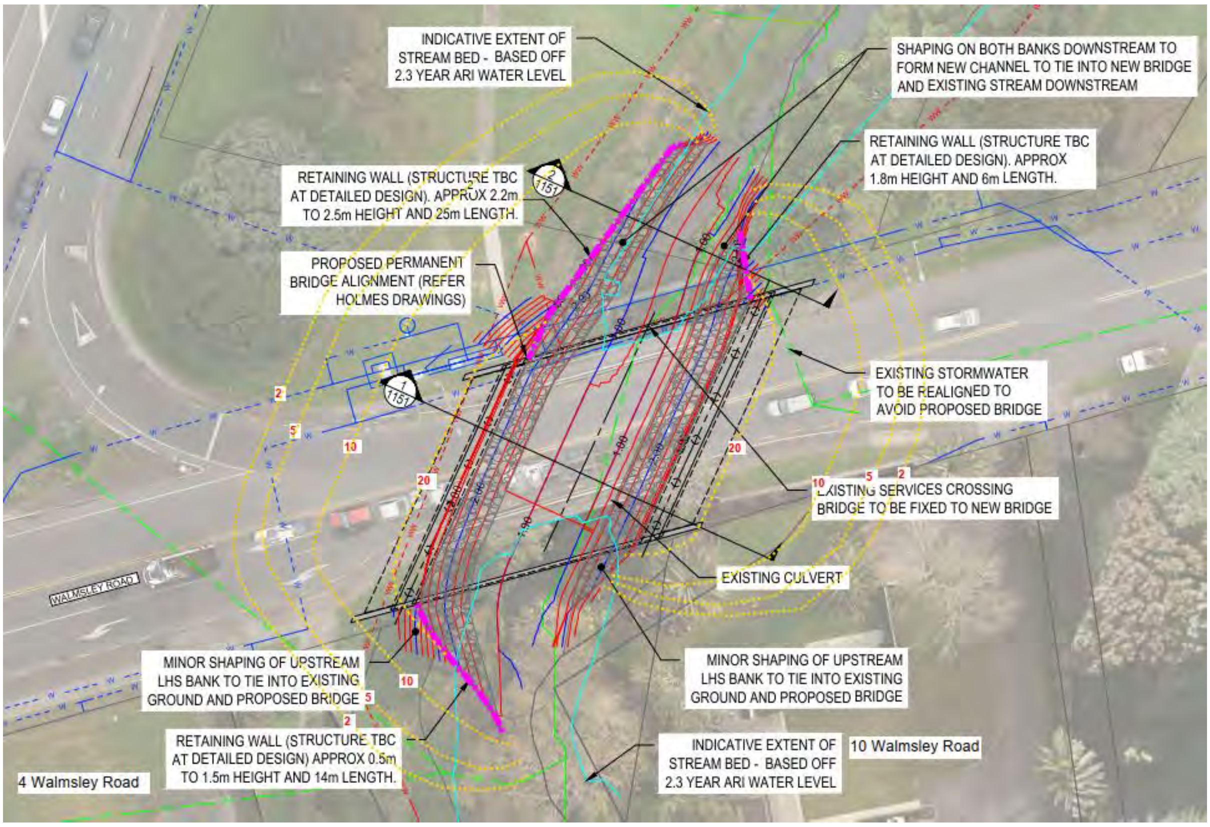

Figure 3.4: Mechanical settlement contours (mm, yellow contours with red notation) and nearby existing structures.

3.7 Combined settlement

Contours of the combined mechanical and consolidation settlement are presented in Figure 3.5 below. A set of settlement contour figures at A3 scale are included in Appendix D. These include drawdown, mechanical and the combined deformation contours.

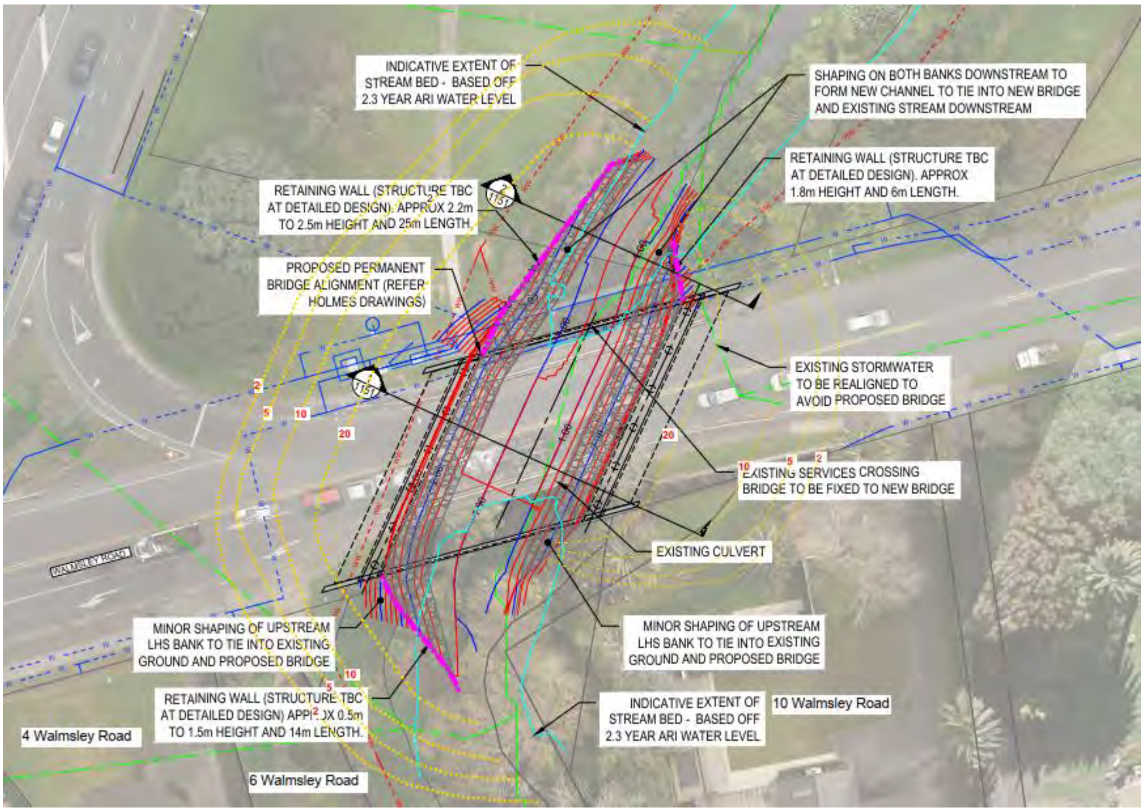

Figure 3.5: Combined drawdown and mechanical settlement contours (mm, yellow contours with red notation) and location of nearby structures.

The existing buried pipelines which are located within the proposed excavation area are to be either realigned or re-connected after construction. The grade change due to the proposed development work (settlement over distance) along the existing buried services behind the proposed bridge are assessed to not exceed the empirical damage criteria set out by O' Rourke and Trautmann (1982)[5]. Additional details on this are presented in Appendix D.

Watercare has a watermain that crosses the bridge and connects to buried infrastructure to the northwest of the bridge. The construction methodology includes interaction with Watercare to protect these assets as they are re-founded and the new bridge is constructed.

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[5] O'Rourke, T. D. and Trautman, C. H. (1982), Buried pipeline response to timnelling ground movements. Proc. Europipe '82 Conf., Basel, Switzerland.

4 Summary

This report has been prepared to support resource consent application for flood resilience under the Severe Weather Emergency Recovery (Auckland Flood Resilience Works) Order 2024. The works involve the upgrading of the Walmsley Road bridge to remove the blockage risk that resulted in significant flooding of properties following the significant rain events in January 2023.

This report assesses the construction and operational geotechnical and groundwater effects of the Project, given its location in an area that is assessed as land subject to instability (notably the banks of the river) and given that groundwater dewatering will be required during construction.

The assessment has considered the location of the works on land subject to instability and concluded that any potential risk during construction will be able to be appropriately managed through standard construction methodologies.

In relation to potential groundwater dewatering and settlement effects, the assessment has concluded that any potential effect on nearby structures is negligible, given the construction methodology proposed, the nature of the underlying geology and the distance from any nearby structures. Given no adverse effects are assessed, no monitoring (including a Groundwater and Settlement Monitoring & Contingency Plan) is proposed during or after construction. Once completed, there is not expected to be any ongoing groundwater or settlement effects. We note that building surveys are proposed related to potential vibration-related effects. These are scheduled for 4, 6, 8, 14 and 15 Walmsley Road, 2 McKenzie Road and 164 Coronation Road. These surveys will provide confirmation that no adverse effects have occurred.

5 Applicability

This report has been prepared for the exclusive use of our client Auckland Council, 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.

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:
....	....
Zhilong Liu Senior Geotechnical Engineer	Pierre Malan Project Director

6-Nov-24

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Appendix A Geotechnical Investigation Factual Report

REPORT

Tonkin+Taylor

Te Ararata - Walmsley Road Bridge Upgrade

Geotechnical investigation factual report

Prepared for
Auckland Council
Prepared by
Tonkin & Taylor Ltd
Date
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September 2024	1	Geotechnical Investigation Factual Report	ZHLI	Pierre Malan	Pierre Malan

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Table of contents

1 Introduction

Tonkin & Taylor Ltd (T+T) was engaged by Auckland Council to conduct a Geotechnical Investigation at Walmsley Road Bridge in Māngere, Auckland. The investigation was undertaken to support the upgrade design of the existing culvert bridge.

2 Site description

The existing Black Bridge Culvert is located on Walmsley Road in Māngere, Auckland. It is a two-lane structure that spans Te Ararata Creek, located at the southern edge of both the Black bridge Reserve and Walmsley Road Reserve, which lie to the west and east of the creek respectively. Residential neighbourhoods are situated to the east of Walmsley Road Reserve and south of the bridge.

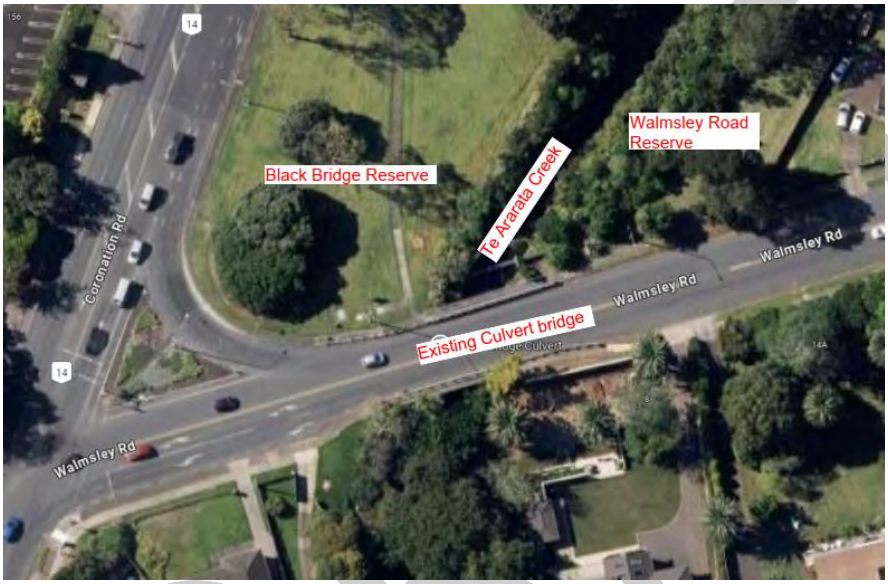

Figure 2.1: Project location – Site plan

The existing bridge was built with a concrete culvert, featuring two rectangular openings, each 2.5 m in width and 3.7 m in hight. A solid pier separates two openings.

3 Published geology

The published geological map of the area[1] and previous investigation indicates that the subject site is expected to be underlain by the following geological units:

- *Fill* – High variable human-placed material, containing recompacted clay to gravel-sized material.
- *Auckland Volcanic Field (AVF)* – Lithic tuff, comprising comminuted pre-volcanic materials with basaltic fragments, and unconsolidated ash and lapilli deposits of well-sorted basalt and basanite fragments.
- *Takaanini Formation* – Late Pliocene to Middle Pleistocene pumiceous river deposits

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[1] Heron, D.W. (custodian) 2020. Geological map of New Zealand 1:250,000. 3rd ed. Lower Hutt, NZ: GNS Science. GNS Science geological map 1. 1 USB; https://doi.org/10.21420/03PC-H178

4 Site investigations

Geotechnical investigations were carried out at the project site in July and August 2024. The investigations comprised:

- 2 machine boreholes.
- 1 hand augured boreholes.
- 2 cone penetrometer tests (CPTs).
- 1 downhole geophysical test.

Actual investigation locations were selected by T+T based on access, buried services and traffic management considerations.

The locations of the investigations were surveyed by handheld GPS and are presented on Figure A1 attached in Appendix A.

4.1 Previously completed investigations

NZGD shows there are some historical investigations in the proximity of the project site. These investigations have been reviewed in the context of the stratigraphy and their geological features of the project site. Details of all relevant investigations considered in this study are presented in Table 4.1 below.

Table 4.1: Summary of previously completed relevant investigations considered for development of a ground model

Investigation Name (NZGD ID)	Location (NZTM)		Ground Surface Elevation RL (m)	Depth (m)
	Easting (m)	Northing (m)		
Machine Drilled Boreholes				
BH_65035	1759469	5908682	7.5	8.0
BH_65038	1759504	5908678	6.0	6.0
BH_65049	1759482	5908635	6.3	6.5
BH_215507	1759567	5908665	6.9	36

4.2 Project specific investigations

The following sections describe and summarise the geotechnical investigations undertaken for this project.

4.2.1 Hand augered boreholes

The drilling of one (1) hand augered borehole was undertaken on 23 August 2024. The works were carried out by a geotechnical engineer from T+T. In situ shear strength testing was undertaken at 0.3 m intervals throughout the soil horizon.

Investigation locations are presented on Figure A1, Appendix A. Summary borehole logs are presented in Appendix B. A summary of the hand auger borehole details is presented in Table 4.2 below.

Table 4.2: Hand augered borehole summary

HA ID	Location (NZTM)		Ground Surface Elevation RL (m)	Depth (m)
	Easting (m)	Northing (m)		
HA01	1759666	5908688	5.8	5.2

4.2.2 Machine boreholes

The machine boring of two (2) vertical boreholes was undertaken over the period between 21 August 2024 and 26 August 2024. The works were carried out using a rotary coring drilling rig, supplied and operated by McMillan Drilling. The boreholes were advanced from ground level using a hand auger/hydro-vacuum technique to a depth of 1.2 m for service clearance, then PQ triple tube coring was undertaken down to the end of hole.

Hand shear vane tests were undertaken at the end of the core barrel sample/in situ down hole at the completion of each core run where fine-grained soil was present. Corrected shear vane values are presented on the borehole logs. In situ Standard Penetration Testing (SPT) was carried out at regular (1.5 m) intervals below depth of 25 m at BH01. All drilling works were completed under the full time supervision of a geotechnical engineer from T+T Geotechnics. The recovered drill core was photographed and logged to NZGS 'Field Description of Soil and Rock' guidelines.

The boreholes were drilled at the locations specified by T+T; the investigation locations are presented on Figure 1, Appendix A. Summary borehole logs and core photographs are presented in Appendix B. Summary borehole details are presented in Table 4.3 below.

Table 4.3: Machine borehole summary

BH ID	Location (NZTM)		Ground Surface Elevation RL (m)	Depth (m)
	Easting (m)	Northing (m)		
BH01	1759492	5908666	6.6	34.8
BH02	1759506	5908694	6.1	6.0

4.2.3 Cone penetration tests

The pushing of two (2) Cone Penetrometer Tests (CPTs) was undertaken by Ground Investigation on 8 July 2024. In all cases, the CPTs were taken to 'refusal' which occurred due to the cone terminating on or within a hard, impenetrable strata such as rock or a dense sand layer.

The CPT locations are presented on Figure A1, Appendix A. CPT logs are appended in Appendix B. A summary of the CPTs and termination depths is presented in Table 4.4 below.

Table 4.4: CPT summary

CPT ID	Location (NZTM)		Ground Surface Elevation RL (m)	Termination depth (m)	Reason for termination
	Easting (m)	Northing (m)			
CPT02	1759556	5908687	5.8	23.0	Refusal
CPT03	1759494	5908630	3.0	20.9	Refusal

4.2.4 Down hole seismic testing

Downhole seismic testing for shear wave velocity measurement has been carried out by Resource Development Consultant Limited (RDCL) at BH01 on 5 September 2024. A 50 mm internal diameter PVC pipe was installed in the machine borehole at the end of borehole drilling on 23 August 2024. The annular space between pipe and borehole was filled with bentonite/cement slurry. A copy of the downhole test report is included in Appendix C.

4.3 Groundwater level

Groundwater levels (GWL) were measured with a dip-meter following the completion of CPT investigations and hand auger excavation. GWL measurements were also undertaken following the machine borehole excavation but may be influenced by drilling fluid. A summary of GWL measurements is shown in Table 4.5. The groundwater measurement data presented on historical investigation logs were also reviewed and presented in Table 4.5.

Table 4.5: Summary of recorded groundwater levels at the Site

Investigation ID	Ground Level [mRL]	Ground water level [mbgl]	Ground water [mRL]	Date of measurement	Comment
Site-specific Investigation 2024					
CPT-02	5.8	2.6	3.2	8 July 2024	Approximately 26 m southeast of the creek. The soil behaviour type encountered at the GWL is inferred to be silt mixture
CPT-03	3.0	1.4	1.6	8 July 2024	Approximately 1 m west of the creek. The soil behaviour type encountered at GWL is inferred to be mixture of sand/silt mixture
HA-01	5.8	2.7	3.1	23 August 2024	Adjacent to CPT-02, silt material at measured GWL
BH-01	6.6	3.4	3.2	23 August 2024	Approximately 12 m northwest of the creek. Fractured basalt at measured GWL m
BH-02	6.1	2.1	4.0	26 August 2024	Approximately 11 m northwest of the creek. Silty boulders fill at measured GWL
Historical Investigation					
BH_65035	7.5	3.0	4.5	7 Nov 2013	Approximately 36 m northwest of the creek. Fractured basalt at measured GWL
BH_215507	6.9	2	4.9	3 May 2023	Approximately 26 m southeast to the creek. Measured after overnight GWL resuming. Clay silt at measured GWL

4.4 Laboratory testing

4.4.1 Geotechnical testing

Three disturbed bulk samples at different depths were collected from BH01 core boxes. The samples were tested at the Geotechnics laboratory for determination of water content, fines content and plasticity. The tests were undertaken in accordance with NZS 4402.

Results of the laboratory testing are presented in Appendix D.

5 Applicability

This report has been prepared for the exclusive use of our client Auckland Council, 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.

Tonkin & Taylor Ltd
Environmental and Engineering Consultants

Report prepared by:	Authorised for Tonkin & Taylor Ltd by:
....	....
Zhilong Liu Senior Geotechnical Engineer	Pierre Malan Project Director

ZHLI
\\ttgroup.local\corporate\auckland\projects\1017033\1017033.2003\workingmaterial\05 geotechnical\3. assessment\0.6 rc report\geotechnical investigation factual report_draft.docx

Appendix A Figures

- **Figure A1 – Geotechnical Investigation Layout**

COPYRIGHT ON THIS FIGURE IS RESERVED   DO NOT SCALE FROM THIS FIGURE.   Tonkin + Taylor Map Viewer (NZ)   Date: 18/07/2024 4:25 PM

NOTES:

CRS: NZGD 2000 New Zealand Transverse Mercator   Credits: Tonkin & Taylor Group , Esri Community Maps Contributors, LINZ, Stats NZ, Esri, TomTom, Garmin, Foursquare, METI/NASA, USGS, Earthstar Geographics, LINZ, LINZ, Stats NZ, Esri, TomTom, Garmin, Foursquare, METI/NASA, USGS

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

LOCATION PLAN

PROJECT No.	1017033.2003	
DESIGNED	ZHLI	JUL.24
DRAWN	-WEB-	JUL.24
CHECKED		
APPROVED	DATE	

CLIENT AUCKLAND COUNCIL
PROJECT TE ARARATA STAGE 2: DESIGN
TITLE TE ARARATA BRIDGE INVESTIGATION LAYOUT
SCALE (A3) 1:589   FIG No. FIGURE A1   REV 0

Appendix B Field Investigations

- **Appendix B1 – Machine borehole logs**
- **Appendix B2 – Hand auger borehole logs**
- **Appendix B3 – CPT plots**

BOREHOLE LOG

BOREHOLE No.:
BH01
SHEET: 1 OF 6

DRILLED BY: Craig Kennedy
LOGGED BY: SCZH
CHECKED: PRMM
START DATE: 21/08/2024
FINISH DATE: 23/08/2024
CONTRACTOR: McMillan

PROJECT: Te Ararata Stage 2: Design
JOB No.: 1017033.2003
LOCATION: 5R Walmsley Road, Mangere

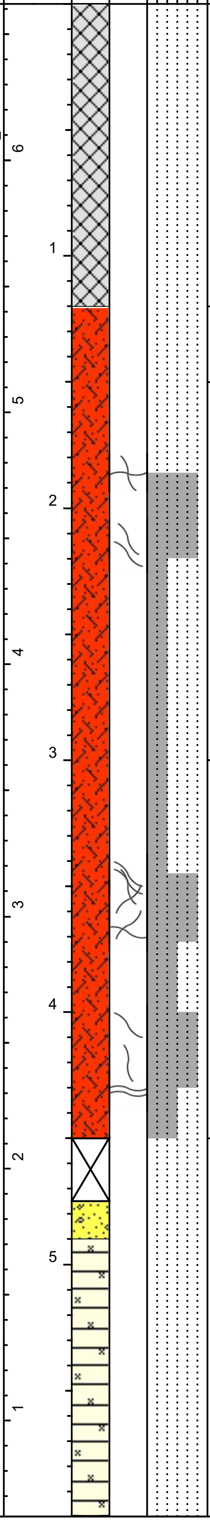
CO-ORDINATES: 5908666 mN
(NZTM2000) 1759492 mE
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND: 7m
R.L. COLLAR:
DATUM: NZVD2016
SURVEY: Handheld GPS

SOIL: Classification, colour, consistency / density, moisture, plasticity
ROCK: Weathering, colour, fabric, name, strength, cementation

Rock Weathering: UW SW MW HW CW RS
Rock Strength: VS+ S MS W VW EW
Fracture Spacing (mm): 2000 600 200 60 20
Water Level / Fluid Loss (%): 25 50 75

GEOLOGICAL UNIT	MATERIAL DESCRIPTION	Sampling Method	Core Recovery (%)	Testing	RQD (%)	ROCK MASS DISCONTINUITIES Description & Additional Observations	Water Level / Fluid Loss (%)	Casing	Core Box No
Fill	*0.00m:* Clayey gravelly SILT; dark greyish brown. Stiff, moist, medium to high plasticity. [FILL].								
	0.40m: Steel Waste								
	0.60m: Steel Waste	HVAC	0	● 51/6 kPa Insitu					
				● 176/51 kPa Insitu					
Auckland Volcanic Field	*1.20m:* Slightly to moderately weathered, dark grey, BASALT. Strong to very strong. [AVF].	PQTT	100		70		22/08/2024		
		PQTT	100		91	*1.86m:* J, PL, R, FeSt *1.86-2.00m:* J, 70°, PL, R, FeSt *2.12-2.20m:* J, 60°, PL, R, FeSt *2.18-2.20m:* J, 45°, PL, R, FeSt		35.0m	Box 0.00-3.45m
	3.40m: Becomes highly vesicular *3.45 - 3.80m* Crush Zone	PQTT	100		33	*3.45-3.48m:* J, 45°, UN, R, FeSt *3.48-3.50m:* J, 45°, UN, R, FeSt *3.50-3.75m:* J, 70°, PL, R, FeSt *3.55-3.60m:* J, 45°, UN, R, FeSt *3.65-3.68m:* J, 45°, PL, R, FeSt *3.68-3.72m:* J, 20°, PL, R, FeSt *4.05-4.10m:* J, 45°, UN, SM, FeSt *4.20-4.30m:* J, 80°, UN, SM, FeSt *4.30-4.32m:* J, 5°, PL, R, FeSt *4.32-4.34m:* J, 5°, PL, R, FeSt	23/08/2024		
	4.50m: Core Loss								
Takanini Formation	*4.75m:* Medium to coarse SAND, some gravel; dark grey. Loosely packed, moist. Gravel, fine. [TAKANINI].								
	4.90m: Silty CLAY; dark grey. Firm, wet, high plasticity.								
	5.20m: Becomes stiff to very stiff; Colour becomes light grey mottled orange	PQTT	83						Box 3.45-6.00m



COMMENTS:

Hole Depth
34.75m

BOREHOLE LOG

BOREHOLE No.:
BH01
SHEET: 2 OF 6

DRILLED BY: Craig Kennedy
LOGGED BY: SCZH
CHECKED: PRMM
START DATE: 21/08/2024
FINISH DATE: 23/08/2024
CONTRACTOR: McMillan

PROJECT: Te Ararata Stage 2: Design
JOB No.: 1017033.2003
LOCATION: 5R Walmsley Road, Mangere

CO-ORDINATES: 5908666 mN
(NZTM2000) 1759492 mE
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND: 7m
R.L. COLLAR:
DATUM: NZVD2016
SURVEY: Handheld GPS

GEOLOGICAL UNIT	MATERIAL DESCRIPTION SOIL: Classification, colour, consistency / density, moisture, plasticity ROCK: Weathering, colour, fabric, name, strength, cementation	Rock Weathering UW SW MW HW CW RS	Rock Strength VS+ S MS W VW EW	Sampling Method	Core Recovery (%)	Testing	RL (m)	Depth (m)	Graphic Log	Defect Log	Fracture Spacing (mm) 2000 600 200 60 20	RQD (%)	ROCK MASS DISCONTINUITIES Description & Additional Observations	Water Level / Fluid Loss (%) 25 50 75	Casing	Installation	Core Box No
Takanini Formation	*6.00m:* Pushtube			PT	70	PT_6.0m @ 6.00m 125/51 kPa Insitu 109/38 kPa In barrel											Box 6.00-10.10m
	6.50m: Sandy SILT, minor clay; light whitish grey. Soft to firm, wet, low plasticity. Sand, fine to medium. *6.80m:* becomes dark brownish grey			PQTT	80		0										
	6.90m: Organic clayey SILT; black. Firm, wet, medium plasticity.							7									
	7.30m: Core Loss																
	7.50m: Pushtube			PT	60	PT_7.5m @ 7.50m 83/0 kPa In barrel	-1	8									
	8.00m: Core Loss			PQTT	80												
	8.20m: Organic clayey SILT; black. Firm, wet, medium plasticity.																
	8.40m: Silty CLAY, minor organic; dark grey mottled black. Firm, moist, high plasticity.						-2										
	8.60m: Sandy SILT; whitish grey. Very stiff to hard, moist. Sand, fine to medium.							9									
	9.00m: Pushtube			PT	50	PT_9.0m @ 9.00m											
	9.50m: Sandy SILT; whitish grey. Very stiff to hard, moist. Sand, fine to medium. *9.90 - 10.10m:* Becomes saturated and soft *10.40m:* Colour becomes dark grey			PQTT	100	DS_9.6m @ 9.60m	-3	10									
	10.50m: Sandy SILT; dark grey. Soft to firm, wet. Sand, fine to medium.			PQTT	100		-4 -5	11									

COMMENTS:

Hole Depth
34.75m

Scale 1:30

Rev.: A

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BOREHOLE LOG

BOREHOLE No.:
BH01
SHEET: 3 OF 6

PROJECT: Te Ararata Stage 2: Design	CO-ORDINATES: (NZTM2000) 5908666 mN 1759492 mE
JOB No.: 1017033.2003	DIRECTION:
LOCATION: 5R Walmsley Road, Mangere	ANGLE FROM HORIZ.: -90°

R.L. GROUND: 7m
R.L. COLLAR:
DATUM: NZVD2016
SURVEY: Handheld GPS

DRILLED BY: Craig Kennedy
LOGGED BY: SCZH
CHECKED: PRMM
START DATE: 21/08/2024
FINISH DATE: 23/08/2024
CONTRACTOR: McMillan

GEOLOGICAL UNIT	MATERIAL DESCRIPTION SOIL: Classification, colour, consistency / density, moisture, plasticity ROCK: Weathering, colour, fabric, name, strength, cementation	Rock Weathering UW SW MW HW CW RS	Rock Strength VS+ S MS W VW EW	Sampling Method	Core Recovery (%)	Testing	RL (m)	Depth (m)	Graphic Log	Defect Log	Fracture Spacing (mm) 2000 600 200 60 20	RQD (%)	Description & Additional Observations	Water Level / Fluid Loss (%) 25 50 75	Casing	Installation	Core Box No
Takanini Formation	*12.00m:* Pushtube			PT	82	PT_12.0m @ 12.00m											
	12.50m: Sandy SILT; dark grey. Soft to firm, wet. Sand, fine to medium.			PQTT	100		-6										
	12.70m: Sandy SILT; dark grey. Very stiff to hard, moist. Sand, fine to medium. *13.50 - 13.90m* Lenses of clayey SILT						-7	13									Box 10.10-13.50m
				PQTT	100	DS_13.75m @ 13.75m	-8	14 15									
	15.30m: Silty fine to medium SAND; dark grey. Loosely packed, wet.			PQTT	100	DS_16.0m @ 16.00m	-9 -10	16 17									Box 13.50-15.90m
				PQTT	100		-11										
	17.60m: Silty fine to medium SAND; dark grey. Tightly packed, moist.																

COMMENTS:

Hole Depth
34.75m

Scale 1:30

BOREHOLE LOG

BOREHOLE No.: **BH01**
SHEET: 4 OF 6

DRILLED BY: Craig Kennedy
LOGGED BY: SCZH
CHECKED: PRMM
START DATE: 21/08/2024
FINISH DATE: 23/08/2024
CONTRACTOR: McMillan

PROJECT: Te Ararata Stage 2: Design
JOB No.: 1017033.2003
LOCATION: 5R Walmsley Road, Mangere

CO-ORDINATES: 5908666 mN 1759492 mE
(NZTM2000)
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND: 7m
R.L. COLLAR:
DATUM: NZVD2016
SURVEY: Handheld GPS

GEOLOGICAL UNIT	MATERIAL DESCRIPTION SOIL: Classification, colour, consistency / density, moisture, plasticity ROCK: Weathering, colour, fabric, name, strength, cementation
Takanini Formation	[CONT] *17.60m:* Silty fine to medium SAND; dark grey. Tightly packed, moist. *18.00m:* Becomes saturated and loosely packed
	18.30m: SILT, some clay and some sand; dark grey. Very stiff, moist, low plasticity. Sand, fine to medium. *19.20 - 19.35m* becomes soft
	21.00m: Sandy SILT, minor clay; dark grey. Very stiff, moist, low plasticity. Sand, fine to medium.
	21.45m: Fine to medium SAND, minor silt; dark grey. Tightly packed, moist.
	21.60m: Organic SILT, minor clay; black. Hard, moist, low plasticity.
	22.20m: Clayey SILT; dark brown. Hard, moist, medium plasticity. *22.70m:* Colour becomes light grey, Inclusion of trace SAND
	23.30m: Fine to medium SAND, minor silt; light grey. Dense to very dense, moist. *23.60 - 24.50m* Becomes saturated and loosely packed
	23.70m: Core Loss

Sampling Method	Core Recovery (%)	Testing
PQTT	100	
PQTT	100	● 138/51 kPa In barrel
PQTT	100	● 176/45 kPa In barrel
PQTT	80	● UTP In barrel

RL (m): -12, -13, -14, -15, -16, -17
Depth (m): 19, 20, 21, 22, 23

Graphic Log

ROCK MASS DISCONTINUITIES: Defect Log; Fracture Spacing (mm) 2000 600 200 60 20; RQD (%); Description & Additional Observations

Rock Weathering: UW SW MW HW CW RS
Rock Strength: VS+ S MS W VW EW
Water Level / Fluid Loss (%): 25 50 75
Casing
Installation

Core Box No:
Box 15.90-18.10m
Box 18.10-20.30m
Box 20.30-22.70m

COMMENTS:

Hole Depth
34.75m

Scale 1:30

Rev.: A

BOREHOLE LOG

BOREHOLE No.: BH01

SHEET: 5 OF 6

DRILLED BY: Craig Kennedy
LOGGED BY: SCZH
CHECKED: PRMM
START DATE: 21/08/2024
FINISH DATE: 23/08/2024
CONTRACTOR: McMillan

PROJECT: Te Ararata Stage 2: Design
JOB No.: 1017033.2003
LOCATION: 5R Walmsley Road, Mangere

CO-ORDINATES: (NZTM2000) 5908666 mN 1759492 mE
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND: 7m
R.L. COLLAR:
DATUM: NZVD2016
SURVEY: Handheld GPS

GEOLOGICAL UNIT: Takanini Formation

MATERIAL DESCRIPTION

SOIL: Classification, colour, consistency / density, moisture, plasticity
ROCK: Weathering, colour, fabric, name, strength, cementation

24.00m: Fine to medium SAND, minor silt; light grey. Dense to very dense, moist.

26.05m: SAND becomes medium to coarse

26.10m: Core Loss

27.00m: Fine to medium SAND, minor silt; light grey. Dense to very dense, moist.

27.90m: SILT, minor clay; dark brown. Hard, moist, low plasticity.

28.50m: Solid SPT - no recovery

28.95m: Fine to medium SAND, minor silt; dark grey. Very dense, moist.

29.35 - 29.60m Soils disturbed from drilling and becomes loosely packed

Sampling Method	Core Recovery (%)	Testing
		● UTP In barrel
PQTT	100	
SPT	100	3/5// 7/12/12/15 **N=46**
PQTT	10	
SPT	100	1/1// 6/8/8/12 **N=34**
PQTT	100	UTP In barrel
SPTC	0	4/4// 4/6/5/8 **N=23** Solid Cone
PQTT	100	

RL (m)	Depth (m)
-18	
	25
-19	
	26
-20	
	27
-21	
	28
-22	
	29
-23	

Core Box No: Box 22.70-25.20m; Box 25.20-29.00m

ROCK MASS DISCONTINUITIES

COMMENTS:

Hole Depth 34.75m

Scale 1:30

Rev.: A

BOREHOLE LOG

BOREHOLE No.: BH01
SHEET: 6 OF 6

DRILLED BY: Craig Kennedy
LOGGED BY: SCZH
CHECKED: PRMM
START DATE: 21/08/2024
FINISH DATE: 23/08/2024
CONTRACTOR: McMillan

PROJECT: Te Ararata Stage 2: Design
JOB No.: 1017033.2003
LOCATION: 5R Walmsley Road, Mangere

CO-ORDINATES: 5908666 mN
(NZTM2000) 1759492 mE
DIRECTION:
ANGLE FROM HORIZ.: -90°

R.L. GROUND: 7m
R.L. COLLAR:
DATUM: NZVD2016
SURVEY: Handheld GPS

MATERIAL DESCRIPTION
SOIL: Classification, colour, consistency / density, moisture, plasticity
ROCK: Weathering, colour, fabric, name, strength, cementation

Rock Weathering: UW SW MW HW CW RS
Rock Strength: VS S MS W VW EW
ROCK MASS DISCONTINUITIES: Defect Log; Fracture Spacing (mm) 2000 600 200 60 20; RQD (%); Description & Additional Observations
Water Level / Fluid Loss (%): 25 50 75
Casing; Installation; Core Box No

GEOLOGICAL UNIT	MATERIAL DESCRIPTION	Sampling Method	Core Recovery (%)	Testing	RL (m)	Depth (m)	Core Box No
Takanini Formation	[CONT] *28.95m:* Fine to medium SAND, minor silt; dark grey. Very dense, moist. *30.30 - 30.50m* Soils disturbed from drilling and becomes loosely packed	SPTC	0	6/4// 17/20/13 for 15 mm **N>=50** Solid Cone			Box 29.00-33.80m
		PQTT	100		-24	31	
	31.50m: Solid SPT - no recovery	SPTC	0	4/8// 10/13/16/ 11 for 45mm **N>=50** Solid Cone	-25		
	31.92m: Fine to medium SAND, minor silt; dark grey. Very dense, moist.	PQTT	51			32	
	32.47m: Core Loss				-26		
	33.00m: Solid SPT - no recovery	SPTC	0	5/15// 26/24 for 60 mm **N>=50** Solid Cone		33	
	33.29m: Fine to medium SAND, minor silt; dark grey. Very dense, moist.	PQTT	50		-27		
	33.90m: Core Loss					34	Box 33.80-34.75m
	34.50m: Solid SPT - no recovery	SPTC	0	7/20// 37/13 for 20 mm **N>=50** Solid Cone	-28		
	34.75m: END OF BOREHOLE. Target depth.				-29	35	

COMMENTS:

Hole Depth
34.75m

Scale 1:30

Rev.: A

CORE PHOTOS

BOREHOLE No.: **BH01**

Hole Location: 5R Walmsley Road, Mangere

SHEET: 1 OF 6

PROJECT: Te Ararata Stage 2: Design
LOCATION: Walmsley Road
JOB No.: 1017033.2003

CO-ORDINATES: 5908666 mN
(NZTM2000) 1759492 mE
R.L.: 7m
DATUM: NZVD2016

DRILL TYPE:
METHOD: Rotary cored

HOLE STARTED: 21/08/2024
HOLE FINISHED: 23/08/2024
DRILLED BY: McMillan
LOGGED BY: SCZH
CHECKED: PRMM

0.00-3.45m

3.45-6.00m

CORE PHOTOS

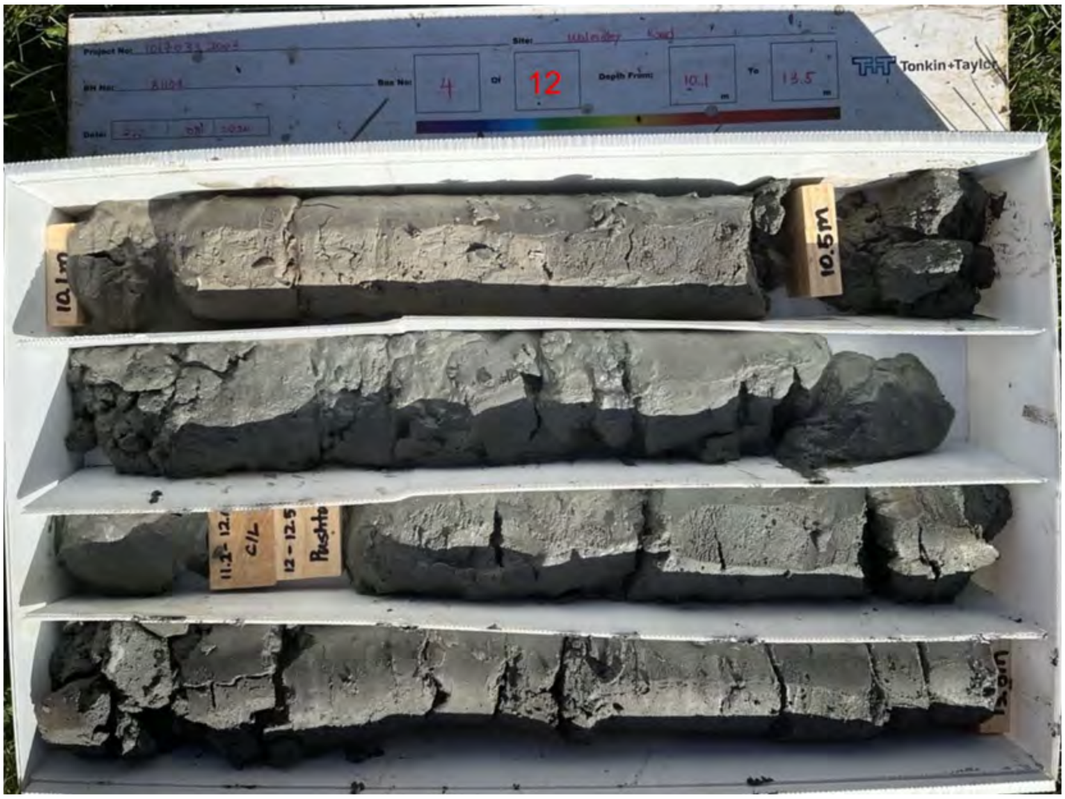
BOREHOLE No.: **BH01**

Hole Location: 5 R Walmsley Road, Mangere

SHEET: 2 OF 6

PROJECT: Te Ararata Stage 2: Design	LOCATION: Walmsley Road	JOB No.: 1017033.2003
CO-ORDINATES: (NZTM2000) 5908666 mN 1759492 mE R.L.: 7m DATUM: NZVD2016	DRILL TYPE: METHOD: Rotary cored	HOLE STARTED: 21/08/2024 HOLE FINISHED: 23/08/2024 DRILLED BY: McMillan LOGGED BY: SCZH CHECKED: PRMM

6.00-10.10m



10.10-13.50m

CORE PHOTOS

BOREHOLE No.: **BH01**

Hole Location: 5R Walmsley Road, Mangere

SHEET: 3 OF 6

PROJECT: Te Ararata Stage 2: Design	LOCATION: Walmsley Road	JOB No.: 1017033.2003
CO-ORDINATES: (NZTM2000) 5908666 mN 1759492 mE	DRILL TYPE:	HOLE STARTED: 21/08/2024
	METHOD: Rotary cored	HOLE FINISHED: 23/08/2024
R.L.: 7m		DRILLED BY: McMillan
DATUM: NZVD2016		LOGGED BY: SCZH CHECKED: PRMM

13.50-15.90m

15.90-18.10m

CORE PHOTOS

BOREHOLE No.: **BH01**

Hole Location: 5 R Walmsley Road, Mangere

SHEET: 4 OF 6

PROJECT: Te Ararata Stage 2: Design
LOCATION: Walmsley Road
JOB No.: 1017033.2003

CO-ORDINATES: (NZTM2000)	5908666 mN 1759492 mE	DRILL TYPE:	HOLE STARTED: 21/08/2024
R.L.:	7m	METHOD: Rotary cored	HOLE FINISHED: 23/08/2024
			DRILLED BY: McMillan
DATUM:	NZVD2016		LOGGED BY: SCZH CHECKED: PRMM

18.10-20.30m

20.30-22.70m

CORE PHOTOS

BOREHOLE No.: **BH01**

Hole Location: 5R Walmsley Road, Mangere

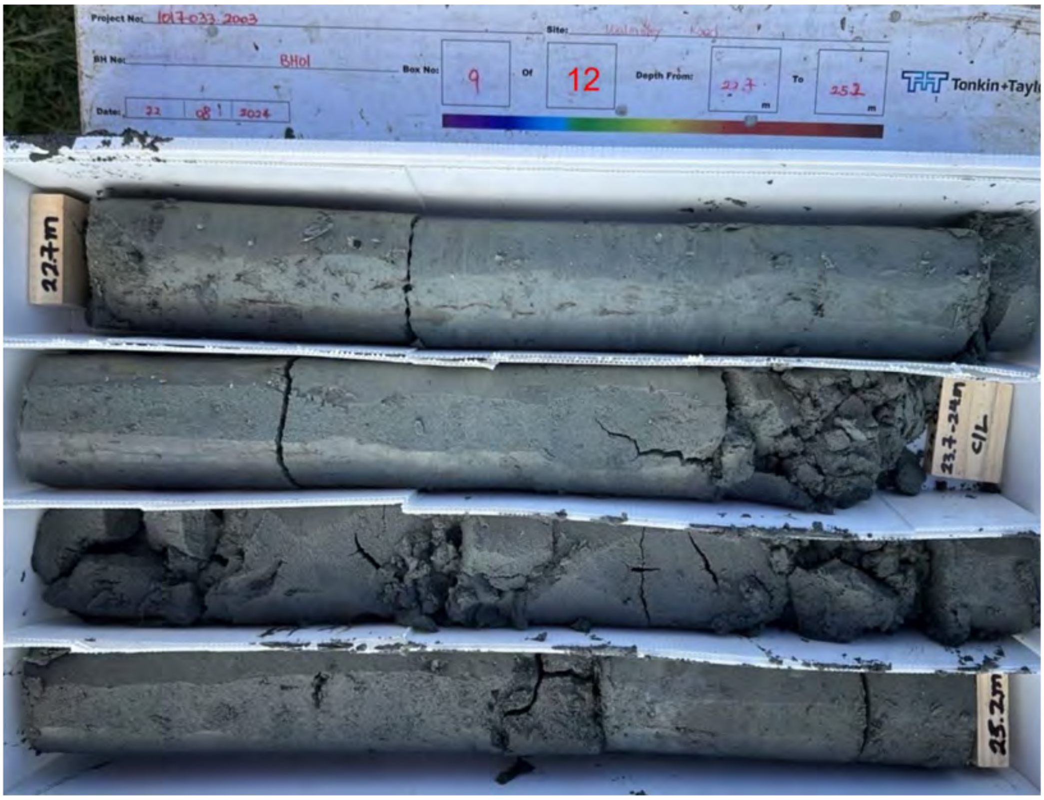
SHEET: 5 OF 6

PROJECT: Te Ararata Stage 2: Design
LOCATION: Walmsley Road
JOB No.: 1017033.2003

CO-ORDINATES: (NZTM2000) 5908666 mN 1759492 mE
R.L.: 7m
DATUM: NZVD2016

DRILL TYPE:
METHOD: Rotary cored

HOLE STARTED: 21/08/2024
HOLE FINISHED: 23/08/2024
DRILLED BY: McMillan
LOGGED BY: SCZH
CHECKED: PRMM



22.70-25.20m

25.20-29.00m

TTNZ_20240703 - GeneralLog - 4/09/2024 3:41:57 pm - Produced with Core-GS by GeRoc

CORE PHOTOS

BOREHOLE No.: **BH01**

Hole Location: 5R Walmsley Road, Mangere

SHEET: 6 OF 6

PROJECT: Te Ararata Stage 2: Design
LOCATION: Walmsley Road
JOB No.: 1017033.2003

CO-ORDINATES: 5908666 mN
(NZTM2000) 1759492 mE
R.L.: 7m
DATUM: NZVD2016

DRILL TYPE:
METHOD: Rotary cored

HOLE STARTED: 21/08/2024
HOLE FINISHED: 23/08/2024
DRILLED BY: McMillan
LOGGED BY: SCZH
CHECKED: PRMM

29.00-33.80m

33.80-34.75m

BOREHOLE LOG

BOREHOLE No.: BH02

SHEET: 1 OF 1

PROJECT: Te Ararata Stage 2: Design	DRILLED BY: Craig Kennedy
JOB No.: 1017033.2003	LOGGED BY: SCZH
LOCATION: 5R Walmsley Road, Mangere	CHECKED: PRMM
CO-ORDINATES: (NZTM2000) 5908694 mN 1759506 mE	START DATE: 26/08/2024
DIRECTION:	FINISH DATE: 26/08/2024
ANGLE FROM HORIZ.: -90°	CONTRACTOR: McMillan
R.L. GROUND: 6m	
R.L. COLLAR:	
DATUM: NZVD2016	
SURVEY: Handheld GPS	

MATERIAL DESCRIPTION

SOIL: Classification, colour, consistency / density, moisture, plasticity

ROCK: Weathering, colour, fabric, name, strength, cementation

Rock Weathering: UW SW MW HW CW RS — Rock Strength: VS+ S MS W VW EW — Sampling Method — Core Recovery (%) — Testing — RL (m) — Depth (m) — Graphic Log

ROCK MASS DISCONTINUITIES: Defect Log — Fracture Spacing (mm) 2000 600 200 60 20 — RQD (%) — Description & Additional Observations

Water Level / Fluid Loss (%) 25 50 75 — Casing — Installation — Core Box No

GEOLOGICAL UNIT	MATERIAL DESCRIPTION	Testing	Water Level	Core Box No
Top	*0.00m:* SILT, some rootlets, minor clay; dark brown. Firm, wet, low plasticity. [TS].			Box 0.00-3.25m
Fill	*0.10m:* SILT, some clay and some gravel; dark brown mottled orange. Stiff to very stiff, wet, low plasticity. Gravel, fine. [FILL]. *0.40m:* Some boulders. Becomes saturated. *1.00m:* Some boulders.			
Fill	*1.00m:* Core Loss			
Fill	*1.50m:* SILT, some rootlets, minor clay; light brown mottled grey. Soft, wet, low plasticity.	● 96/24 kPa Insitu		
Fill	*1.70m:* Silty BOULDERS; brown mottled grey. Moist. Boulders, sandstone.		26/08/2024	
Takanini Formation	*2.20m:* Silty fine to medium SAND; dark grey. Loosely packed, saturated.			
Takanini Formation	*2.30m:* Clayey SILT; dark grey. Firm, wet, medium to high plasticity. *2.50m:* Becomes very stiff to hard, Colour becomes light grey mottled orange *4.90m:* Cobur becomes purplish grey *5.00 - 5.10m* becomes firm	● 138/45 kPa In barrel ● 192/99 kPa Insitu ● 173/29 kPa Insitu 99/19 kPa In barrel		
Takanini Formation	*5.30m:* SILT, minor clay; light brownish grey. Hard, moist, low plasticity. *5.50m:* Inclusion of organic stain, Colour becomes black			Box 3.25-6.00m
Takanini Formation	*5.60m:* Silty fine to medium SAND; light brownish grey. Loosely packed, saturated.			

6m: END OF BOREHOLE. Target depth.

RL (m)	Depth (m)
6	0
5	1
4	2
3	3
2	4
1	5

COMMENTS:

Hole Depth 6m

Scale 1:30

Rev.: A

TTNZ_20240703 - GeneralLog - 4/09/2024 3:43:05 pm - Produced with Core-GS by GeRoc

CORE PHOTOS

BOREHOLE No.: **BH02**

Hole Location: 5R Walmsley Road, Mangere

SHEET: 1 OF 1

PROJECT: Te Ararata Stage 2: Design
LOCATION: Walmsley Road
JOB No.: 1017033.2003

CO-ORDINATES: (NZTM2000) 5908694 mN 1759506 mE
R.L.: 6m
DATUM: NZVD2016

DRILL TYPE:
METHOD: Rotary cored

HOLE STARTED: 26/08/2024
HOLE FINISHED: 26/08/2024
DRILLED BY: McMillan
LOGGED BY: SCZH
CHECKED: PRMM

0.00-3.25m

3.25-6.00m

HAND AUGER LOG

HOLE Id: **HA01**

Hole Location: 5R Walmsley Road, Mangere

SHEET: 1 OF 1

PROJECT: Te Ararata Stage 2: Design

LOCATION: Walmsley Road

JOB No.: 1017033.2003

CO-ORDINATES: (NZTM2000) 5908688 mN
1759556 mE

R.L.: 6m

DATUM: NZVD2016

DRILL TYPE: 50mm hand auger

METHOD: Hand auger

HOLE STARTED: 23/08/2024

HOLE FINISHED: 23/08/2024

DRILLED BY: T+T

LOGGED BY: SCZH

CHECKED: PRMM

GEOLOGICAL | METHOD OBSERVATIONS | ENGINEERING DESCRIPTION

WATER: 23/082024 ▼ (at approx. 2.7 m)

CORE RECOVERY (%): 100

METHOD: HA

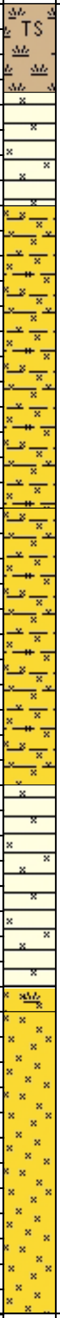
SCALA PENETROMETER (Blows/100mm): 0 1 2 3 4 5 6 7 8 9

TESTS
● 115/29 kPa Insitu
● 109/48 kPa Insitu
● 179/32 kPa Insitu
● >224 kPa Insitu
● UTP Insitu
● 86/19 kPa Insitu
● 64/19 kPa Insitu
● 134/19 kPa Insitu
● 221/67 kPa Insitu
● 99/35 kPa Insitu
● 224/32 kPa Insitu
● >224 kPa Insitu
● UTP Insitu
● UTP Insitu
● UTP Insitu
● UTP Insitu
● UTP Insitu

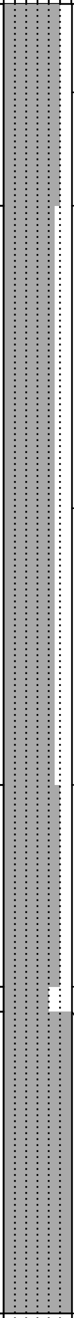
RL (m): 5 4 3 2 1 0

DEPTH (m): 1 2 3 4 5

GRAPHIC LOG



ESTIMATED SOIL SHEAR STRENGTH (Su, kPa): VS 12 S 25 F 50 St 100 VSt 200 H



STRATIGRAPHY / ENG GEOLOGICAL UNIT / ADDITIONAL OBSERVATIONS	MOISTURE CLASSIFICATION	CONSISTENCY / DENSITY CLASSIFICATION	DESCRIPTION
Topsoil	D	VSt	*0.00m:* SILT, some rootlets, minor clay; dark brown. Very stiff, dry, low plasticity. [TS].
Auckland Volcanic Field	M		*0.35m:* Silty CLAY; greyish brown. Very stiff, moist, high plasticity. [AVF].
		St-VSt	*0.80m:* SILT, minor clay; whitish brown. Stiff to very stiff, moist, low plasticity. *1.00m:* Colour becomes brownish grey
Takanini Formation			*2.00m:* SILT, some clay; brown. Stiff to very stiff, moist, medium plasticity. [TAKANINI]. *2.20m:* Colour becomes brownish grey *2.50m:* Becomes wet
	W	VSt	*3.10m:* Silty CLAY; grey mottled orange. Very stiff, wet, high plasticity. *3.40m:* Inclusion of some SAND
		St	*3.90m:* Organic silty CLAY; blackish brown. Stiff, wet, high plasticity.
	S	H	*4.00m:* SILT; light grey. Hard, saturated, dilatant rapid. *4.00m:* low recovery

5.2m: END OF BOREHOLE. Target depth.

0.0-5.2m

COMMENTS:

Hole Depth
5.2m

Scale 1:30

Rev.: A

HAND AUGER PHOTOS

BOREHOLE No.: **HA01**

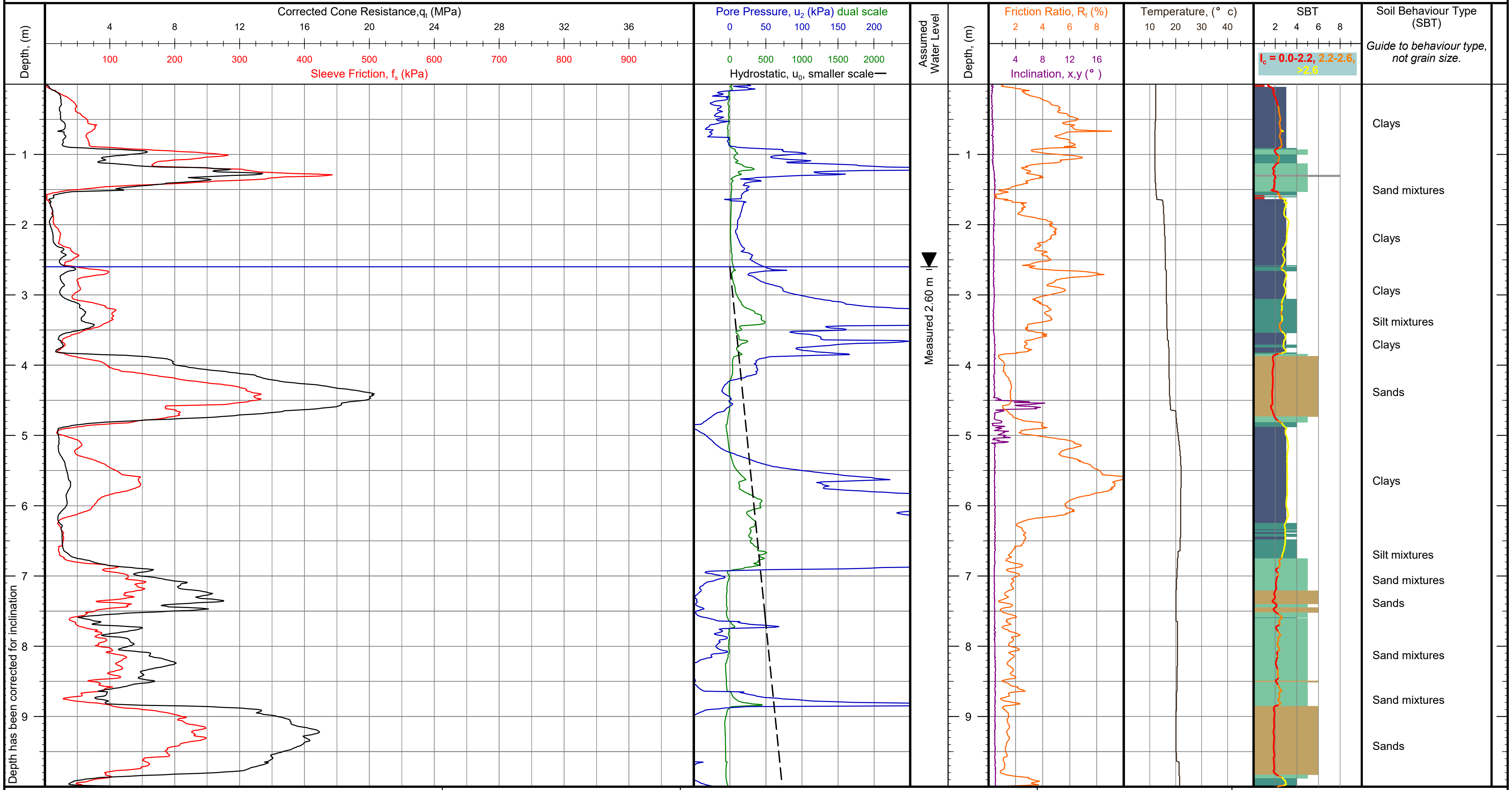
Hole Location: 5R Walmsley Road, Mangere

SHEET: 1 OF 1

PROJECT: Te Ararata Stage 2: Design		LOCATION: Walmsley Road		JOB No.: 1017033.2003
CO-ORDINATES: (NZTM2000)	5908688 mN 1759556 mE	DRILL TYPE: 50mm hand auger	HOLE STARTED: 23/08/2024	
R.L.:	6m	METHOD: Hand auger	HOLE FINISHED: 23/08/2024	
			DRILLED BY: T+T	
DATUM:	NZVD2016		LOGGED BY: SCZH	CHECKED: PRMM

0.00-5.20m

CONE PENETRATION TEST (CPT) LOG



Client: Tonkin + Taylor
Project: Black Bridge Culvert
Location: Mangere Bridge, Auckland
Engineer: Zhilong Liu
Contractor: Ground Investigation Ltd

Operator: Cesar Etchevarne
Cone Ref: 000335
Cone Type: 10cm^2 Compression
Area Ratio: 0.80
Filter Type: u_2

NZTM 2000 N, E (m): 5908687.65, 1759556.23
WGS84 (deg): -36.954383,174.792097
Location Method: Handheld GPS
Surveyor:
Termination Reason: Limit of reaction force

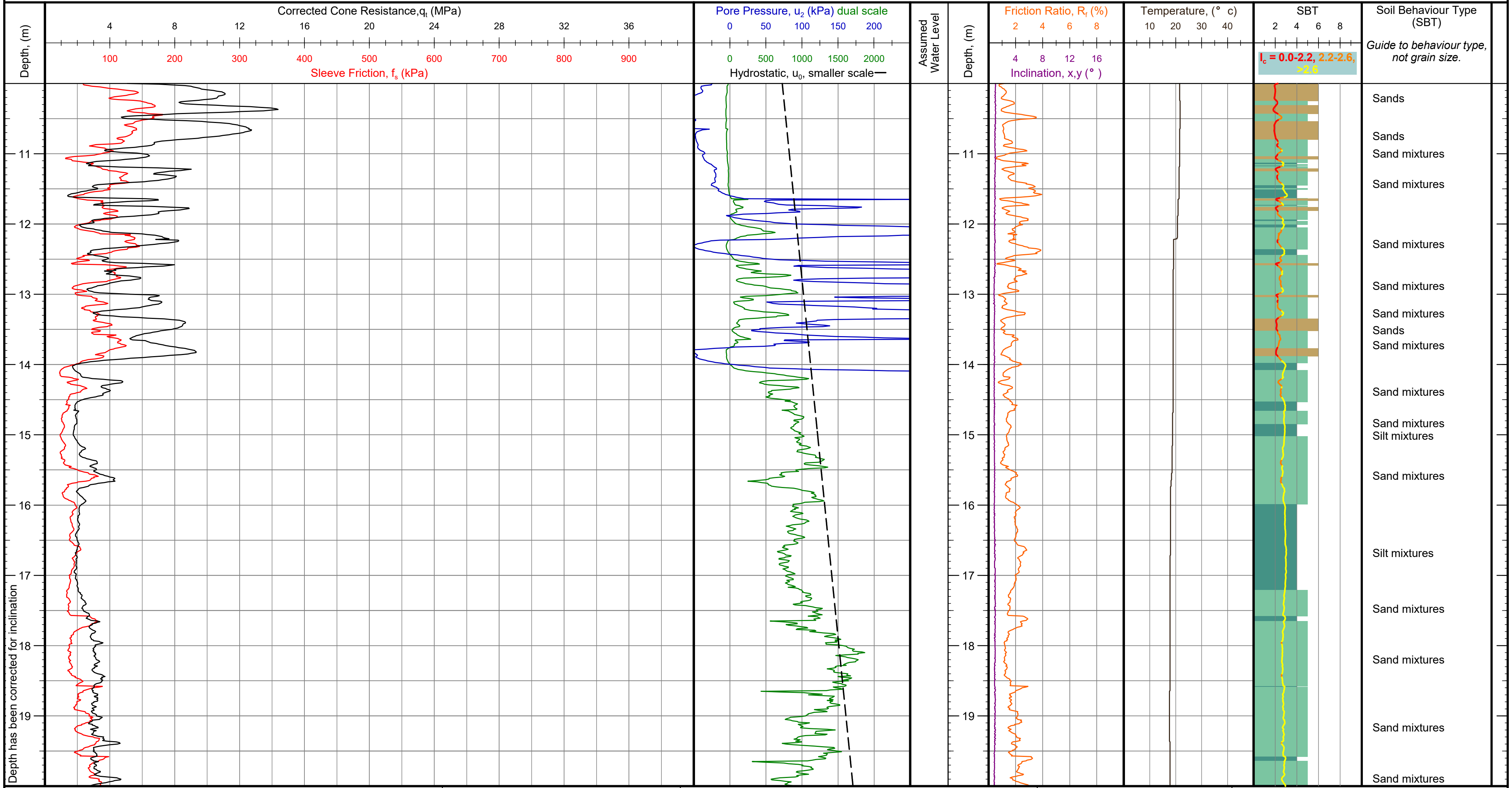
Elevation (m): Unknown
Date of Test: 8/07/2024
Depth (m): 22.98
Pre Drill (m): N/A

Client Reference:
Test Number: CPT-02
G.I. Job Ref: 240553

Comments:

Where possible GWL is measured after testing, or estimated in the office. This may not represent the true GWL

CONE PENETRATION TEST (CPT) LOG



Client: Tonkin + Taylor
Project: Black Bridge Culvert
Location: Mangere Bridge, Auckland
Engineer: Zhilong Liu
Contractor: Ground Investigation Ltd

Operator: Cesar Etchevarne
Cone Ref: 000335
Cone Type: 10cm^2 Compression
Area Ratio: 0.80
Filter Type: u$_2$

NZTM 2000 N, E (m): 5908687.65, 1759556.23
WGS84 (deg): -36.954383,174.792097
Location Method: Handheld GPS
Surveyor:
Termination Reason: Limit of reaction force

Elevation (m): Unknown
Date of Test: 8/07/2024
Depth (m): 22.98
Pre Drill (m): N/A

Client Reference:

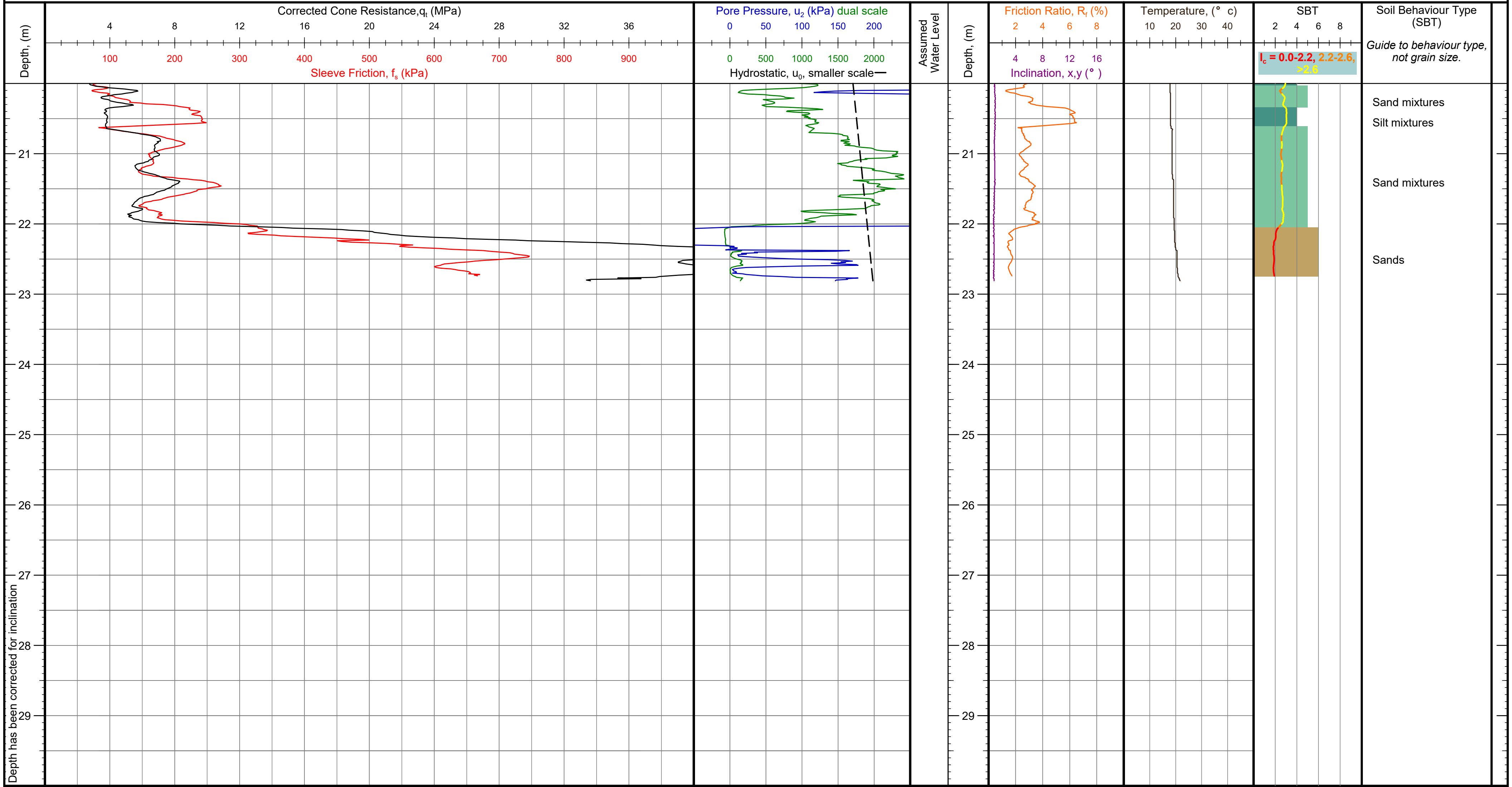
Test Number: **CPT-02**

G.I. Job Ref: 240553

Comments:

Where possible GWL is measured after testing, or estimated in the office. This may not represent the true GWL

CONE PENETRATION TEST (CPT) LOG



Client: Tonkin + Taylor
Project: Black Bridge Culvert
Location: Mangere Bridge, Auckland
Engineer: Zhilong Liu
Contractor: Ground Investigation Ltd

Operator: Cesar Etchevarne
Cone Ref: 000335
Cone Type: 10cm^2 Compression
Area Ratio: 0.80
Filter Type: u_2

NZTM 2000 N, E (m): 5908687.65, 1759556.23
WGS84 (deg): -36.954383,174.792097
Location Method: Handheld GPS
Surveyor:
Termination Reason: Limit of reaction force

Elevation (m): Unknown
Date of Test: 8/07/2024
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Pre Drill (m): N/A

Client Reference:
Test Number: **CPT-02**
G.I. Job Ref: 240553

Comments:

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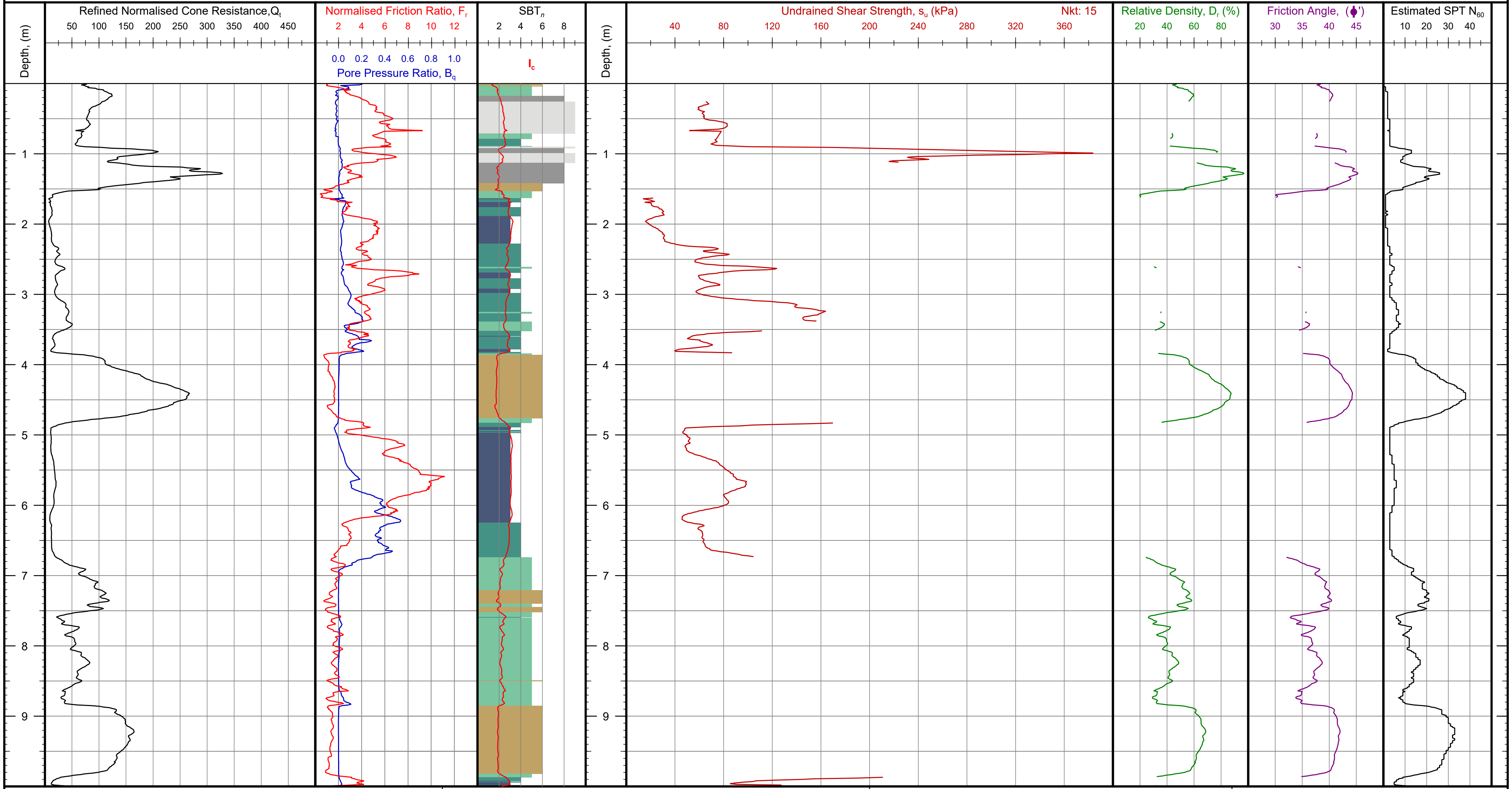
CPT PARAMETER LOG



Client: Tonkin + Taylor
Project: Black Bridge Culvert
Location: Mangere Bridge, Auckland
Engineer: Zhilong Liu
Contractor: Ground Investigation Ltd

Soil Behaviour Type SBT_n - Robertson et al. 1990

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

Notes and 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. Cabel (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 7th Edition. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed by the user. Ground Investigation Ltd. does not warrant the correctness or applicability of any of the geotechnical soil and design parameter 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.

Client Reference:

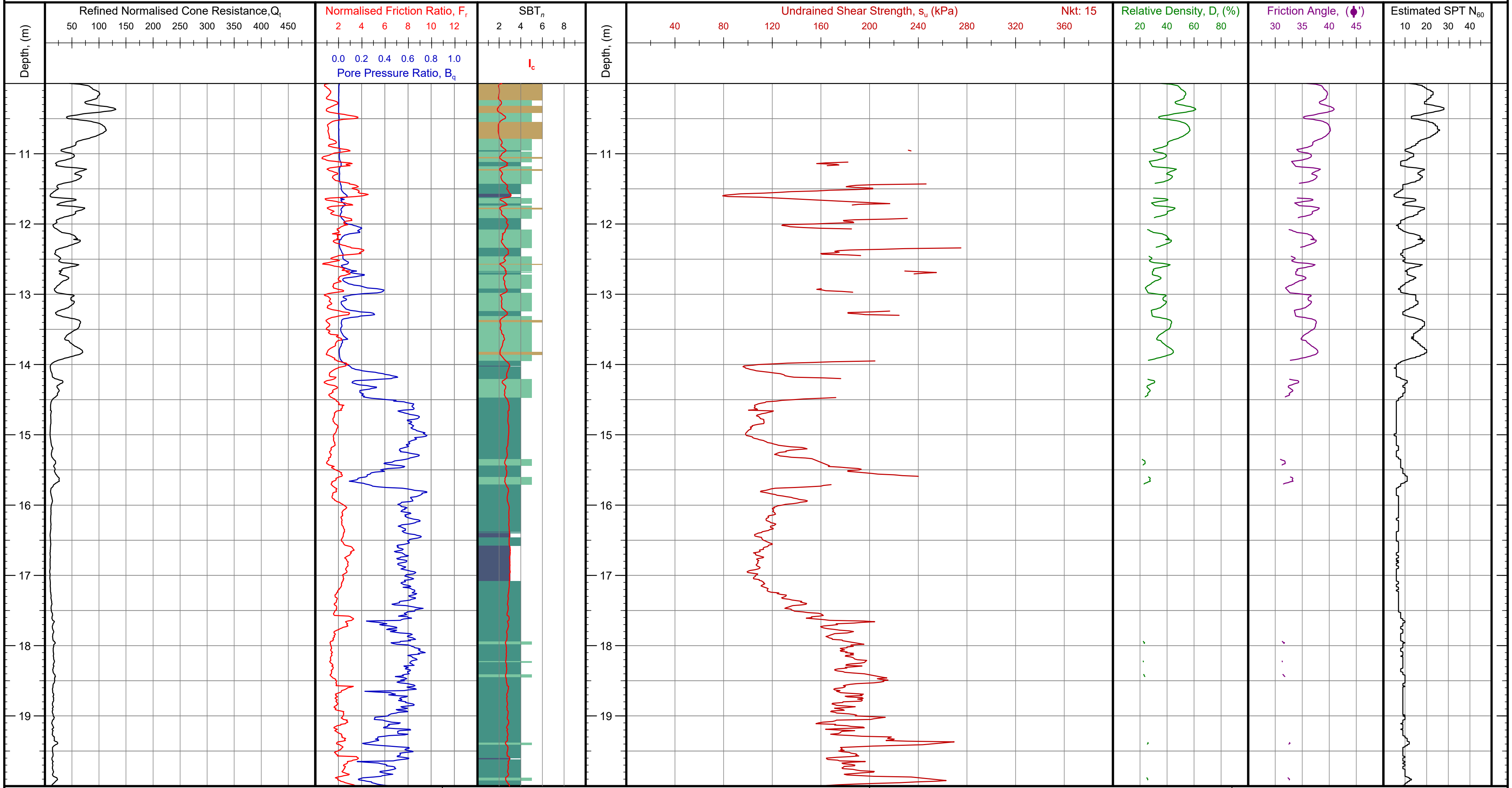
Test Number: **CPT-02**

G.I. Job Ref: 240553

Tonkin+Taylor

CPT PARAMETER LOG

GROUND INVESTIGATION



Client: Tonkin + Taylor
Project: Black Bridge Culvert
Location: Mangere Bridge, Auckland
Engineer: Zhilong Liu
Contractor: Ground Investigation Ltd

Soil Behaviour Type SBT_n - Robertson et al. 1990

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1	Sensitive fine grained	6	Sands: clean sands to silty sands
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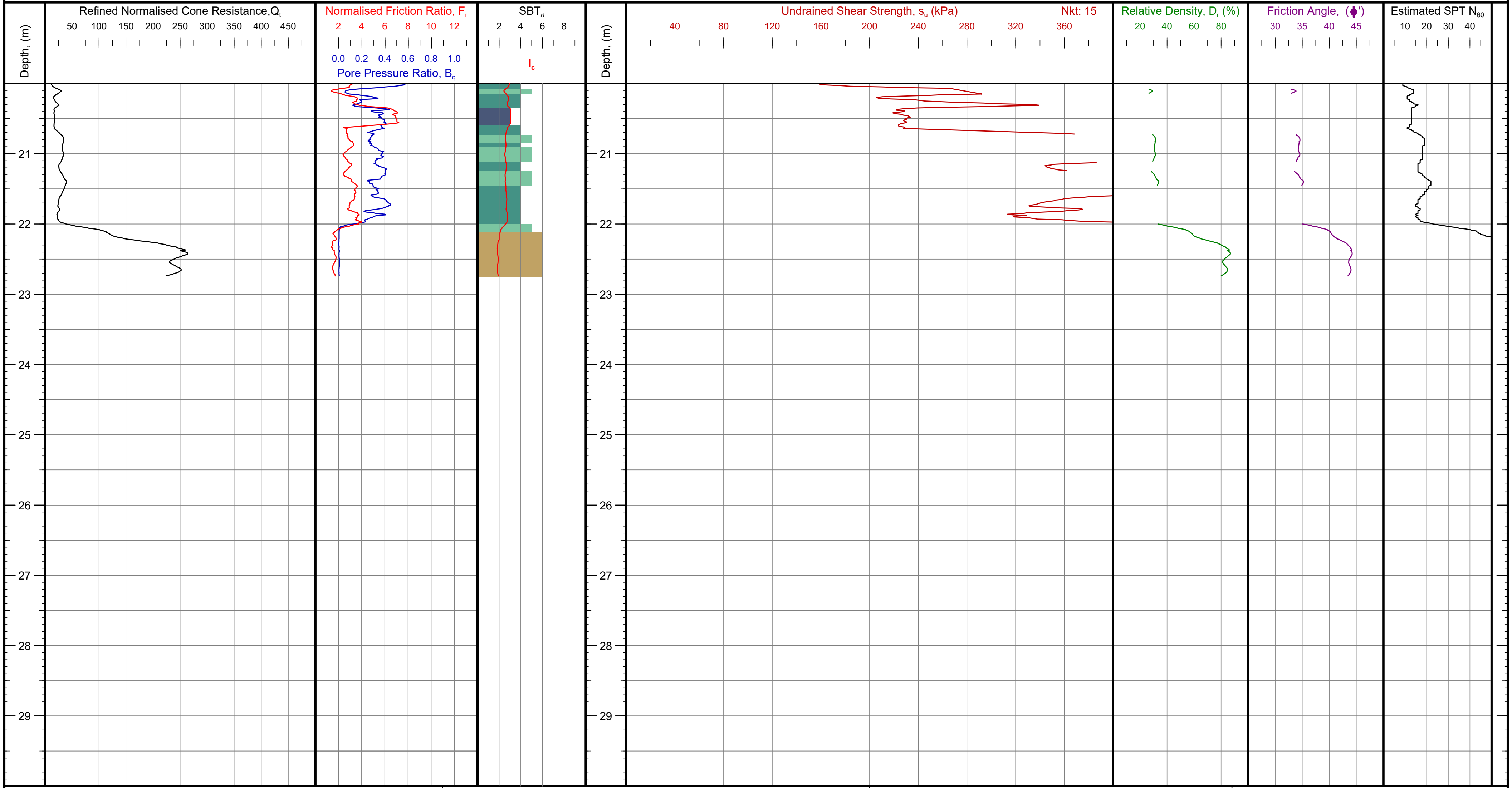
Client Reference:

Test Number: CPT-02

G.I. Job Ref: 240553

CPT PARAMETER LOG



Client: Tonkin + Taylor
Project: Black Bridge Culvert
Location: Mangere Bridge, Auckland
Engineer: Zhilong Liu
Contractor: Ground Investigation Ltd

Soil Behaviour Type SBT_n - Robertson et al. 1990

0	Undefined	5	Sand mixtures: silty sand to sandy silt
1	Sensitive fine grained	6	Sands: clean sands to silty sands
2	Organic: Organic clay/silt, peat	7	Dense sand to gravelly sand
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Client Reference:

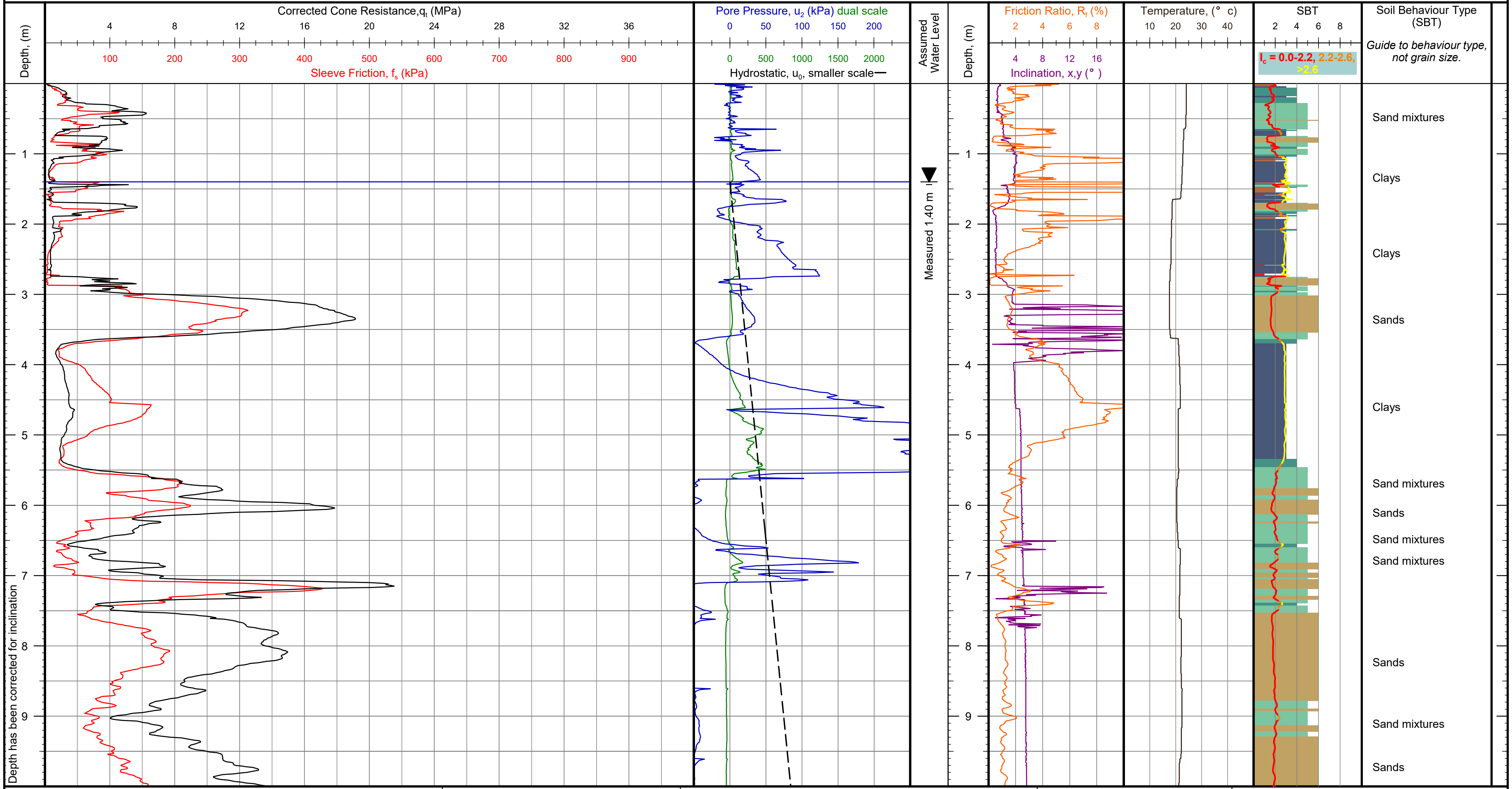
Test Number: **CPT-02**

G.I. Job Ref: 240553

Tonkin+Taylor

CONE PENETRATION TEST (CPT) LOG

GROUND INVESTIGATION



Client: Tonkin + Taylor
Project: Black Bridge Culvert
Location: Mangere Bridge, Auckland
Engineer: Zhilong Liu
Contractor: Ground Investigation Ltd

Operator: Cesar Etchevarne
Cone Ref: 000865
Cone Type: 10cm^2 Compression
Area Ratio: 0.80
Filter Type: u_2

NZTM 2000 N, E (m): 5908630.21, 1759494.58
WGS84 (deg): -36.954911,174.791417
Location Method: Handheld GPS
Surveyor:
Termination Reason: Limit of reaction force

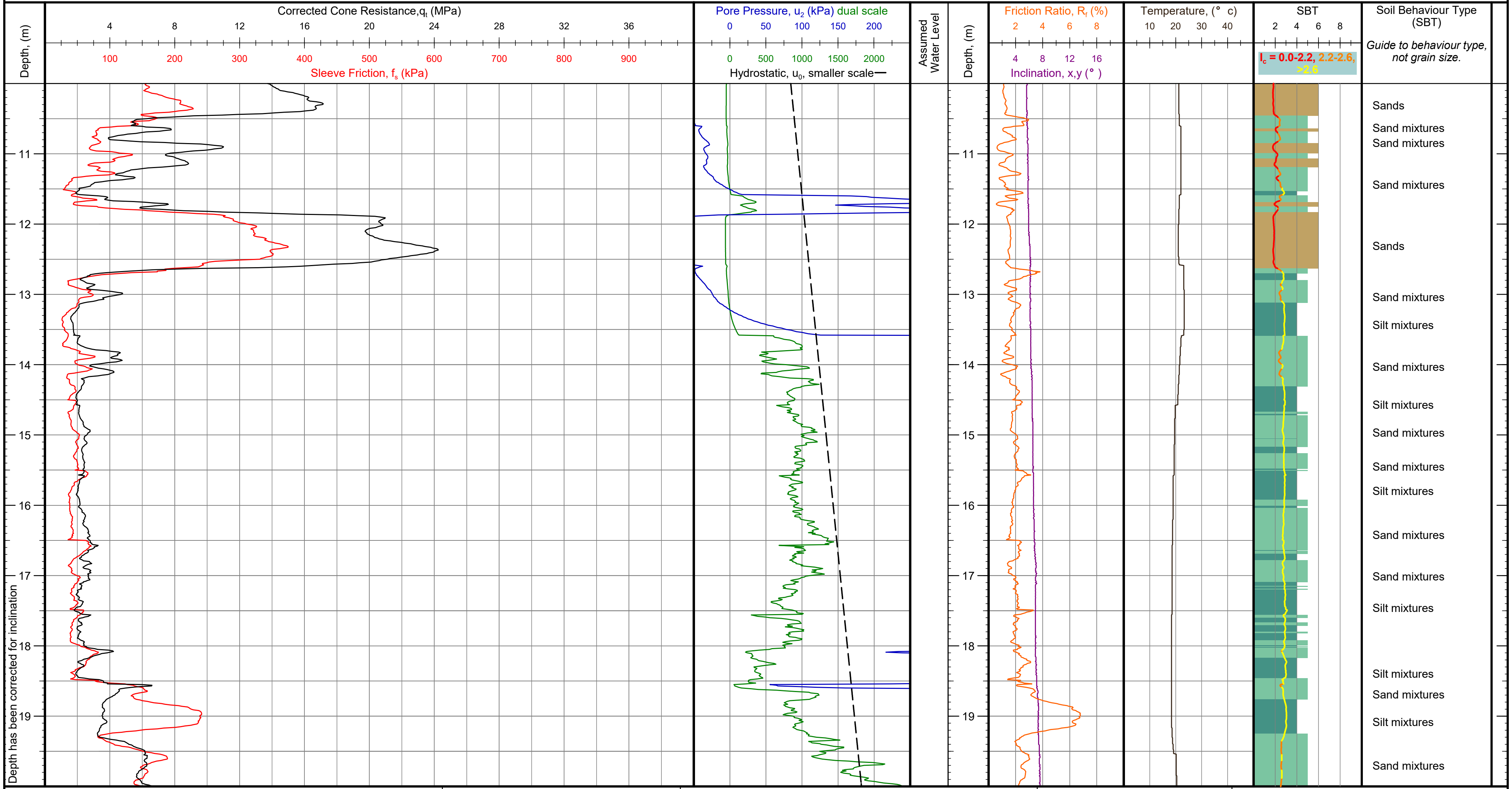
Elevation (m): Unknown
Date of Test: 8/07/2024
Depth (m): 20.86
Pre Drill (m): N/A

Client Reference:
Test Number: **CPT-03**
G.I. Job Ref: 240553

Comments:

Where possible GWL is measured after testing, or estimated in the office. This may not represent the true GWL

CONE PENETRATION TEST (CPT) LOG



Client: Tonkin + Taylor
Project: Black Bridge Culvert
Location: Mangere Bridge, Auckland
Engineer: Zhilong Liu
Contractor: Ground Investigation Ltd

Operator: Cesar Etchevarne
Cone Ref: 000865
Cone Type: 10cm^2 Compression
Area Ratio: 0.80
Filter Type: u_2

NZTM 2000 N, E (m): 5908630.21, 1759494.58
WGS84 (deg): -36.954911,174.791417
Location Method: Handheld GPS
Surveyor:

Elevation (m): Unknown
Date of Test: 8/07/2024
Depth (m): 20.86
Pre Drill (m): N/A

Termination Reason: Limit of reaction force

Client Reference:

Test Number: **CPT-03**

G.I. Job Ref: 240553

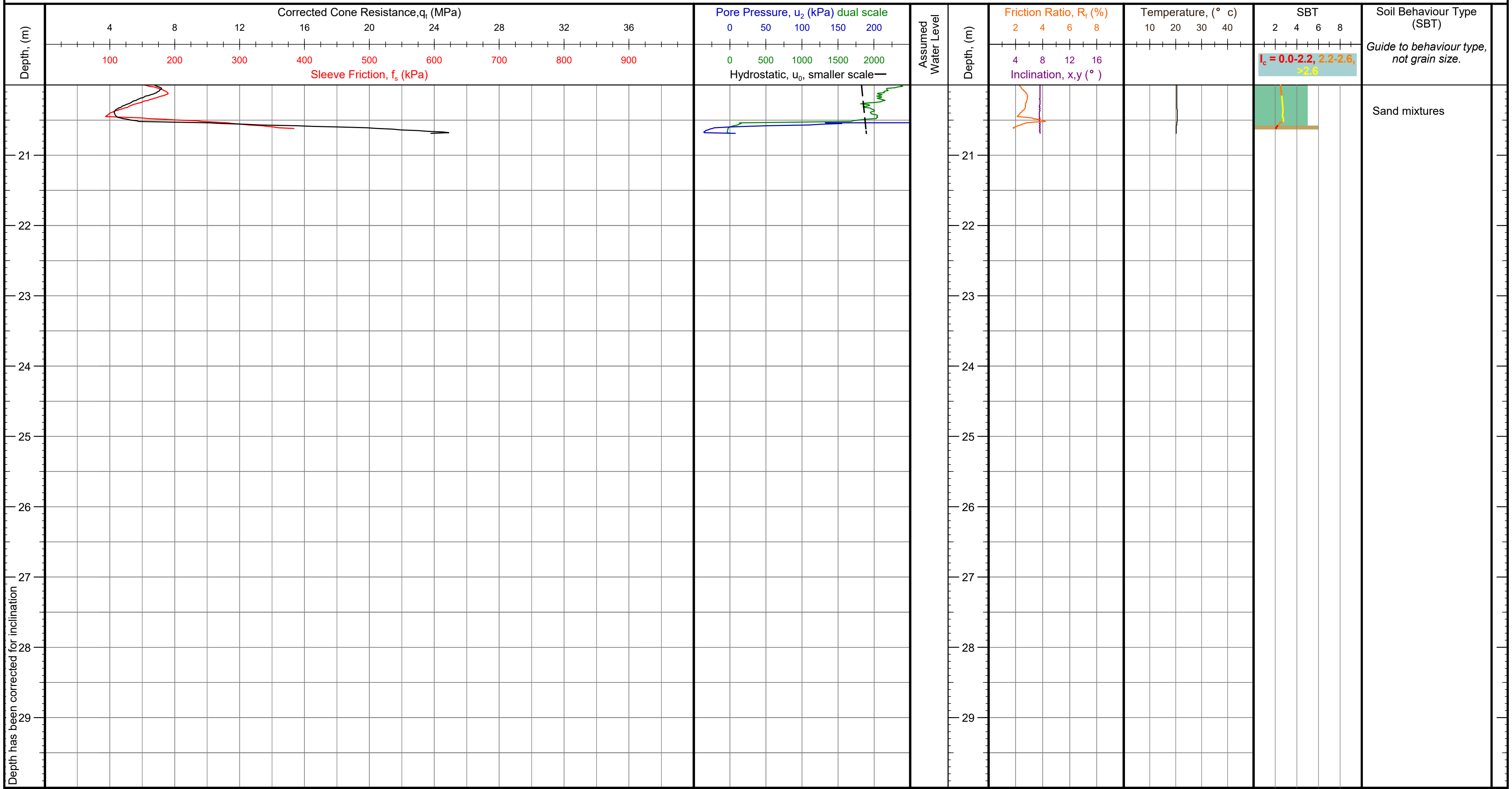
Comments:

Where possible GWL is measured after testing, or estimated in the office. This may not represent the true GWL

Tonkin+Taylor

CONE PENETRATION TEST (CPT) LOG

GROUND INVESTIGATION



Client: Tonkin + Taylor
Project: Black Bridge Culvert
Location: Mangere Bridge, Auckland
Engineer: Zhilong Liu
Contractor: Ground Investigation Ltd

Operator: Cesar Etchevarne
Cone Ref: 000865
Cone Type: $10cm^2$ Compression
Area Ratio: 0.80
Filter Type: u_2

NZTM 2000 N, E (m): 5908630.21, 1759494.58
WGS84 (deg): -36.954911,174.791417
Location Method: Handheld GPS
Surveyor:
Termination Reason: Limit of reaction force

Elevation (m): Unknown
Date of Test: 8/07/2024
Depth (m): 20.86
Pre Drill (m): N/A

Client Reference:

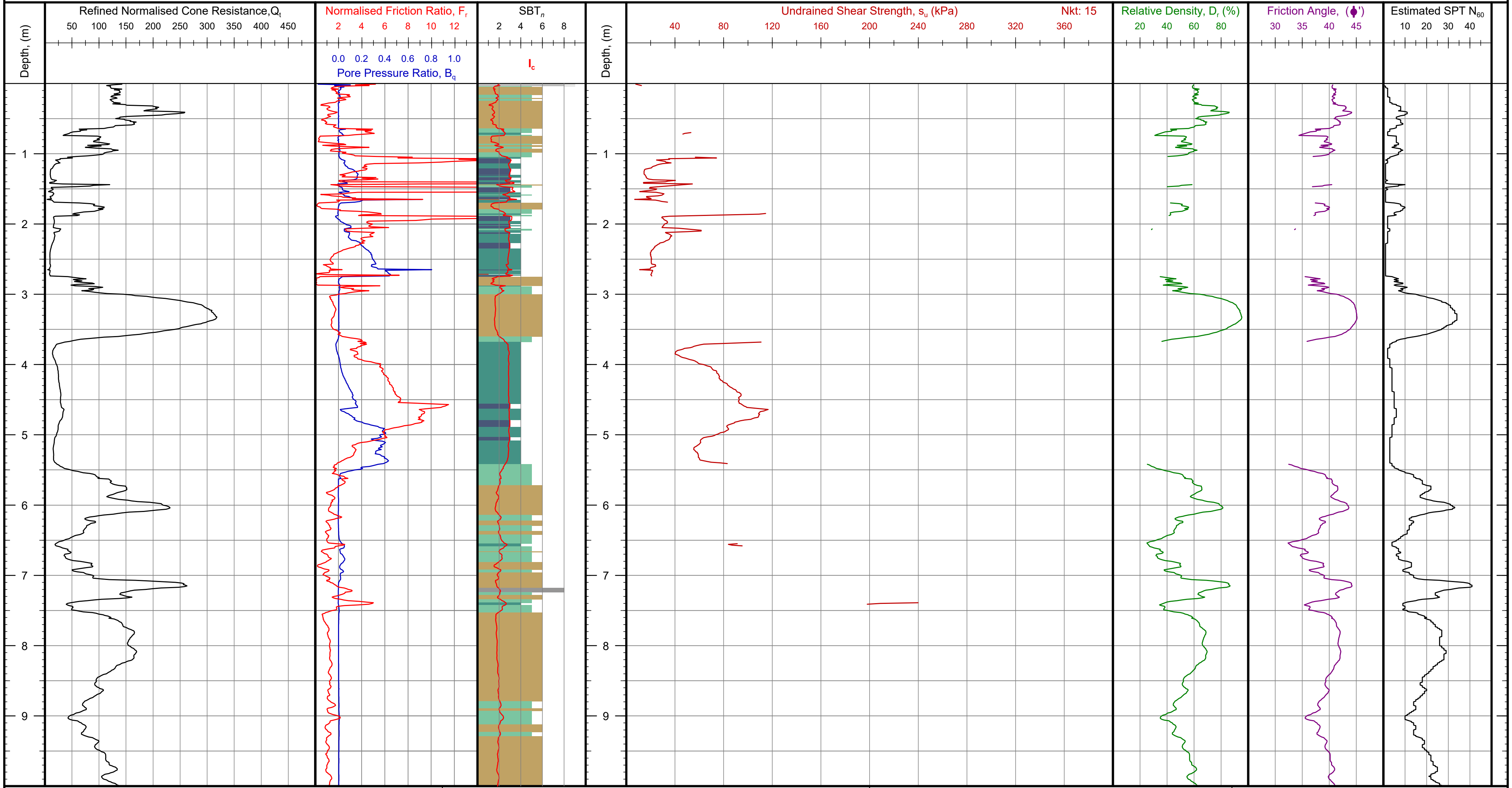
Test Number: **CPT-03**

G.I. Job Ref: 240553

Comments:

Where possible GWL is measured after testing, or estimated in the office. This may not represent the true GWL

CPT PARAMETER LOG



Client: Tonkin + Taylor
Project: Black Bridge Culvert
Location: Mangere Bridge, Auckland
Engineer: Zhilong Liu
Contractor: Ground Investigation Ltd

Soil Behaviour Type SBT_n - Robertson et al. 1990

0	Undefined	5	Sand mixtures: silty sand to sandy silt
1	Sensitive fine grained	6	Sands: clean sands to silty sands
2	Organic: Organic clay/silt, peat	7	Dense sand to gravelly sand
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4	Silt mixtures: clayey silt & silty clay	9	Stiff silt/clay

Notes and Limitations:

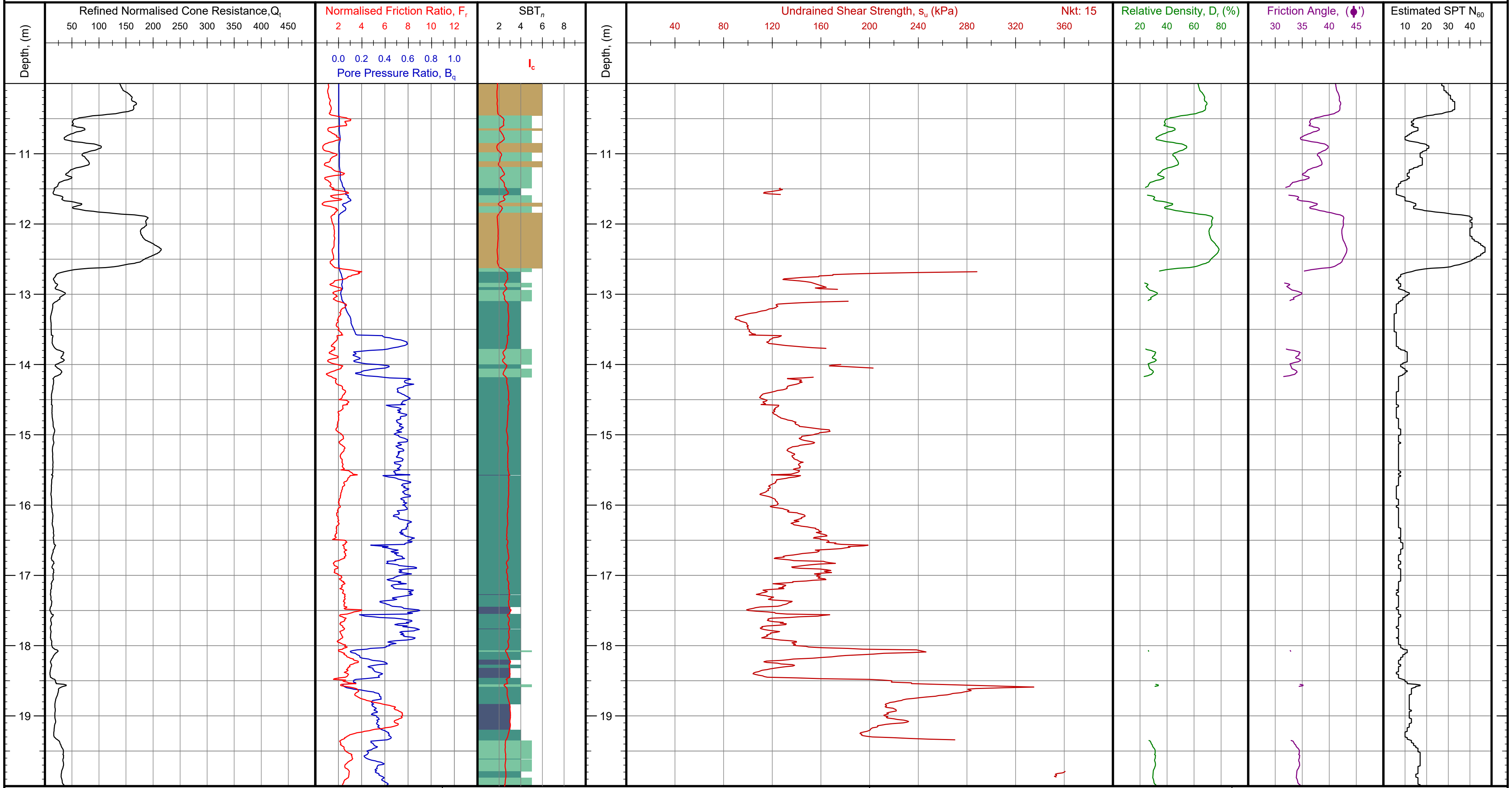
Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P.K. Robertson and K.L. Cabel (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 7th Edition. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed by the user. Ground Investigation Ltd. does not warrant the correctness or applicability of any of the geotechnical soil and design parameter 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.

Client Reference:

Test Number: **CPT-03**

G.I. Job Ref: 240553

CPT PARAMETER LOG



Client: Tonkin + Taylor

Project: Black Bridge Culvert

Location: Mangere Bridge, Auckland

Engineer: Zhilong Liu

Contractor: Ground Investigation Ltd

Soil Behaviour Type SBT$_n$ - Robertson et al. 1990

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

Notes and Limitations:

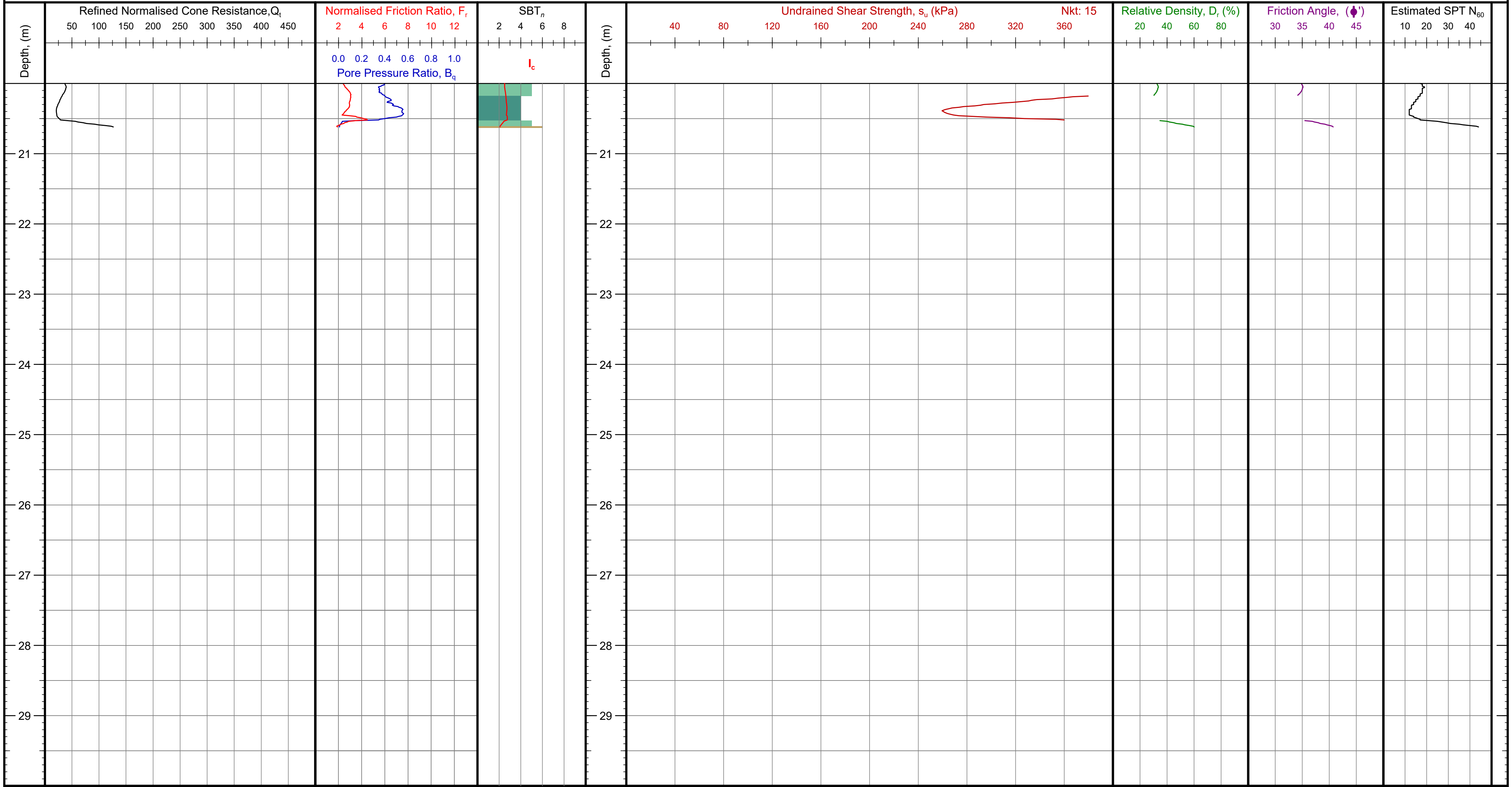
Data shown on this report has been assessed to provide a basic interpretation in terms of Soil Behaviour Type (SBT) and various geotechnical soil and design parameters using methods published in P.K. Robertson and K.L. Cabel (2010), Guide to Cone Penetration Testing for Geotechnical Engineering, 7th Edition. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed by the user. Ground Investigation Ltd. does not warrant the correctness or applicability of any of the geotechnical soil and design parameter 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.

Client Reference:

Test Number: CPT-03

G.I. Job Ref: 240553

CPT PARAMETER LOG



Client: Tonkin + Taylor

Project: Black Bridge Culvert

Location: Mangere Bridge, Auckland

Engineer: Zhilong Liu

Contractor: Ground Investigation Ltd

Soil Behaviour Type SBT_n - Robertson et al. 1990

0	Undefined	5	Sand mixtures: silty sand to sandy silt
1	Sensitive fine grained	6	Sands: clean sands to silty sands
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Client Reference:

Test Number: CPT-03

G.I. Job Ref: 240553

Appendix C RDCL Seismic Testing Report

REPORT ON:

GEOPHYSICAL SERVICES

PROJECT:

BLACK BRIDGE CULVERT
DOWNHOLE SEISMIC

CLIENT:

TONKIN + TAYLOR
1 FANSHAWE STREET
AUCKLAND CBD
AUCKLAND 1010
NEW ZEALAND

240669

RDCL®

CONTENTS

TABLES

FIGURES

APPENDICES

1 INTRODUCTION

Tonkin + Taylor (T+T, the client) engaged Resource Development Consultants Limited (RDCL) to undertake downhole (DH) seismic at Black Bridge Culvert in Auckland, New Zealand.

This report details results from the downhole seismic testing.

1.1 BACKGROUND INFORMATION

RDCL understands that T+T (or their client) require downhole seismic testing at Black Bridge Culvert in Mangere Bridge, Auckland.

As part of this investigation shear wave velocity (Vs) measurements were required in one existing drillhole (BH001, 32 m deep).

BH001 was drilled and logged by McMillan Drilling and had 50 mm internal diameter PVC grouted pipe installed.

The drillers log for the drillhole was provided to RDCL by the client (from McMillan Drilling) and are summarised in Table 1 below. These summaries were used in part to define geophysical velocity layers.

The drillhole's water level was observed near surface by RDCL. This was similar to what was observed in a nearby drillhole recorded in the New Zealand Geotechnical database (3.0 m bgl).

TABLE 1 - SIMPLIFIED SUMMARY OF DRILLERS LOG - BH001

From (m)	To (m)	Material
1	4.5	BASALT
4.5	9.5	SILT
13.5	17.5	Sandy organic SILT
17.5	22.5	SAND
22.5	27.5	Silty SAND with some organics
27.5	32	SAND

Table 2 defines the drill hole location and depth.

TABLE 2 - DRILLHOLE DEFINITIONS

Drill hole ID	NZTM		Drilled Depth (m)	Tested Depth (m bgl)
	Easting	Northing		
BH001	1759489.33	5908668.91	34	32

2 SCOPE OF WORK

The scope of work for the project included acquisition of DHS in BH001 as below:

- Tests at 1.0 m centres from 1 m bgl to 32 m bgl.
- Data processing.
- Delivery of a technical summary report.

Data was acquired on the 5th of September 2024.

2.1 TERMS OF REFERENCE

This investigation employed geophysical methods and therefore the findings presented here are the result of the measurement and interpretation of seismic (acoustic) signals. As such any results derived from the geophysical investigation should be taken in the context of and in reference to the complete ground investigation. Reasonable skill and care were taken to ensure that the results are accurate and reliable, including reference where appropriate to published date from this and/or other sites. However, as with other indirect methods there is a possibility of localised inconsistencies and inaccuracies within the results.

3 METHODOLOGY

Field acquisition and processing methodology are described in Appendix A.

4 RESULTS

Shear wave and compressional wave velocities were measured in BH001 to a depth of 32 m bgl.

4.1 DATA PRESENTATION

Velocity layers were assigned based on (in decreasing order of importance):

- Gradient changes observed in the S-wave t-x curves.
- Gradient changes observed in the P-wave t-x curves.
- Changes in geology observed in drill hole logs provided by the client.

4.2 SHEAR WAVE VELOCITY

Shear wave velocities were recorded in the range of 145 – 420 m/s. (Figures 01 – 03). Data are summarised in Table 3.

Signal to noise ratios (SNR) were average in the shear wave shots. Some picks were not possible in the data due to lack of polarisation in the recorded shots. This is potentially related to core loss providing poor coupling with the installed grout. This core loss is identified in the drillers log at depths of 10-12 m bgl and 27-29 m bgl.

Velocities calculated from the three different picking methods (Appendix A) show reasonable agreement.

4.3 COMPRESSIONAL (P) WAVE VELOCITY

Compressional wave velocities (Vp) were recorded in the range of 1198 – 2386 m/s (Figure 04). Data are summarised in Table 3.

SNR in the P wave data was reasonable.

TABLE 3 - BH001 TABULATED VELOCITIES

Depth (m)		Shear Wave Velocity (m/s)				P Wave Velocity (m/s)
From	To	First Break	First Peak	Max Peak	Average	First Break
1	4.5	300	282	198	260	1198
4.5	9.5	145	159	164	156	1342
9.5	13.5	162	251	194	202	1495
13.5	17.5	312	267	235	271	2386
17.5	22.5	224	218	209	217	1536
22.5	27.5	175	163	248	195	1250
27.5	32	420	378	370	389	2035

4.4 ARRIVAL TIMES

Arrival times are tabulated in Appendix D. These values may be used to calculate velocities over different intervals than that defined in this report.

4.5 COMMENT

Measured shear wave velocities fall within the expected ranges for most of the logged materials (Street R et al (2001)).

The layer of basalt from 1 to 4.5 m bgl had an average velocity of 260 m/s. Generally, basalt would be expected to return higher velocities but based on the data collected this is not the case for this particular testing. Lower than expected velocities may be related to the vesicular and weathered nature of the basalt.

The water level was observed in the drillers log between ~1.25 and ~3.5 m bgl.

5 REFERENCES

- Reynolds, J. M. (2011). An introduction to applied and environmental geophysics. John Wiley & Sons.
- Street, R., Woolery, E. W., Wang, Z., & Harris, J. B. (2001). NEHRP soil classifications for estimating site-dependent seismic coefficients in the Upper Mississippi Embayment. *Engineering Geology*, *62*(1-3), 123-135.

6 LIMITATIONS

- This report has been prepared for the particular purpose outlined in the project brief and no responsibility is accepted for the use of any part in other contexts or for any other purpose.
- This investigation employed geophysical methods and therefore the majority of the findings presented here are the result of the measurement and interpretation of seismic (acoustic) signals. As such any results derived from the geophysical investigation should be taken in the context of and in reference to the complete ground investigation. Reasonable skill and care were taken to ensure that the results are accurate and reliable, including reference where appropriate to published data from this and/or other sites. However, as with other indirect methods there is a possibility of localised inconsistencies and inaccuracies within the results.
- Ground conditions assessed in this report are inferred from data provided by the Client, published sources, site inspection and the investigations described. Variations from the interpreted conditions may occur, and special conditions relating to the site may not have been revealed by this investigation, and which are therefore not taken into account. No warranty is included either expressed or implied that the actual conditions will conform to the interpretation contained in this report.
- No responsibility is accepted by Resource Development Consultants Ltd for inaccuracies in data supplied by others. Where data has been supplied by others, it has been assumed that this information is correct.
- Groundwater conditions can vary with season or due to other events. Any comments on groundwater conditions are based on observations at the time.
- This report is provided for sole use by the Client and is confidential to the Client and their professional advisors. No responsibility whatsoever for the contents of this report shall be accepted for any person other than the Client.

7 CLOSURE

We trust this meets your current needs. Should you wish to discuss any aspect of the contents of this document please contact Ollie Gibson (ogibson@rdcl.co.nz) on +64 4 282 1564.

Prepared by:	Review by:
Edward Brim	O. Gibson
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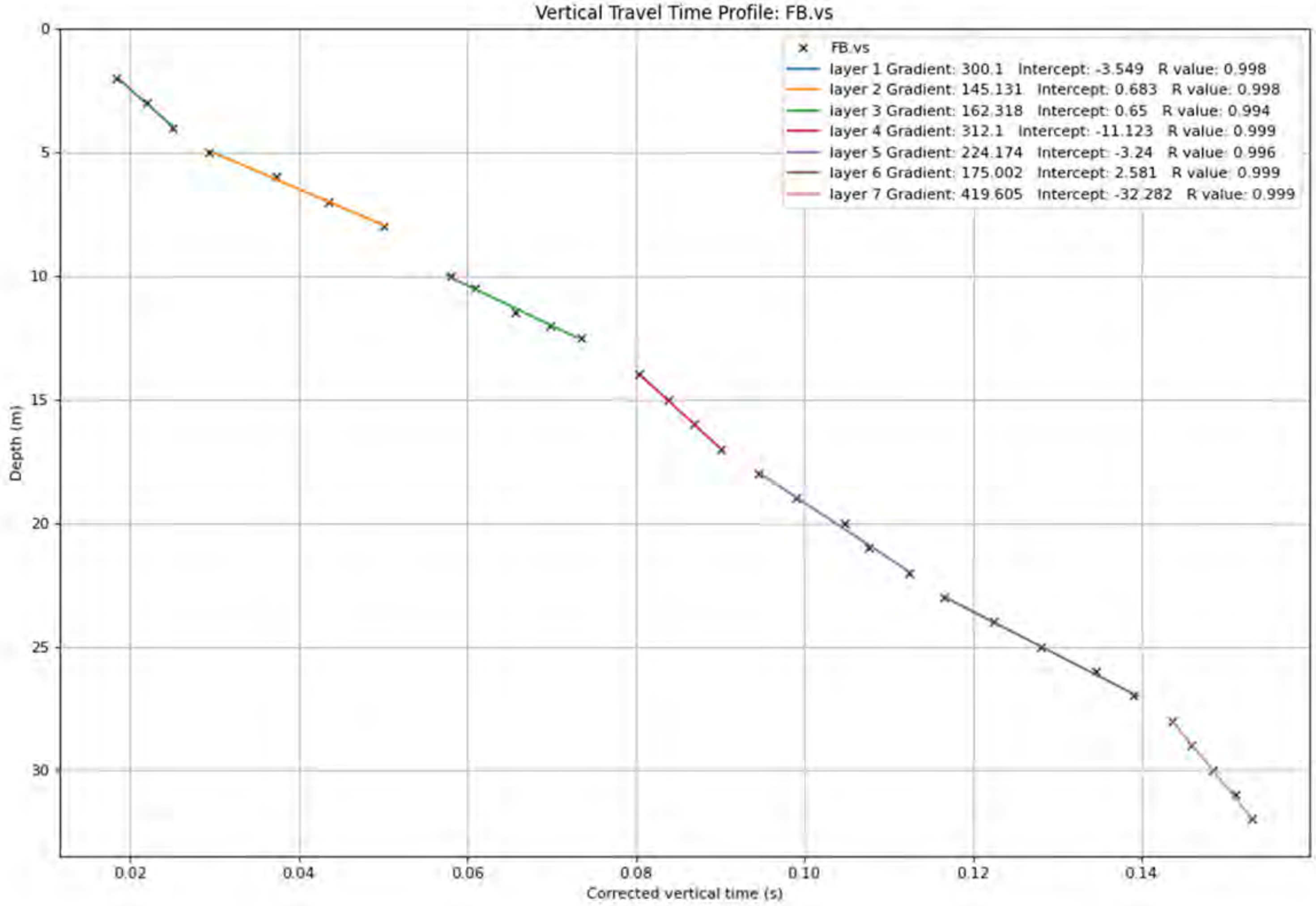
BEnvSc	BSc, MRes
Ground Investigation Technician	Principal Geophysicist

Attached:

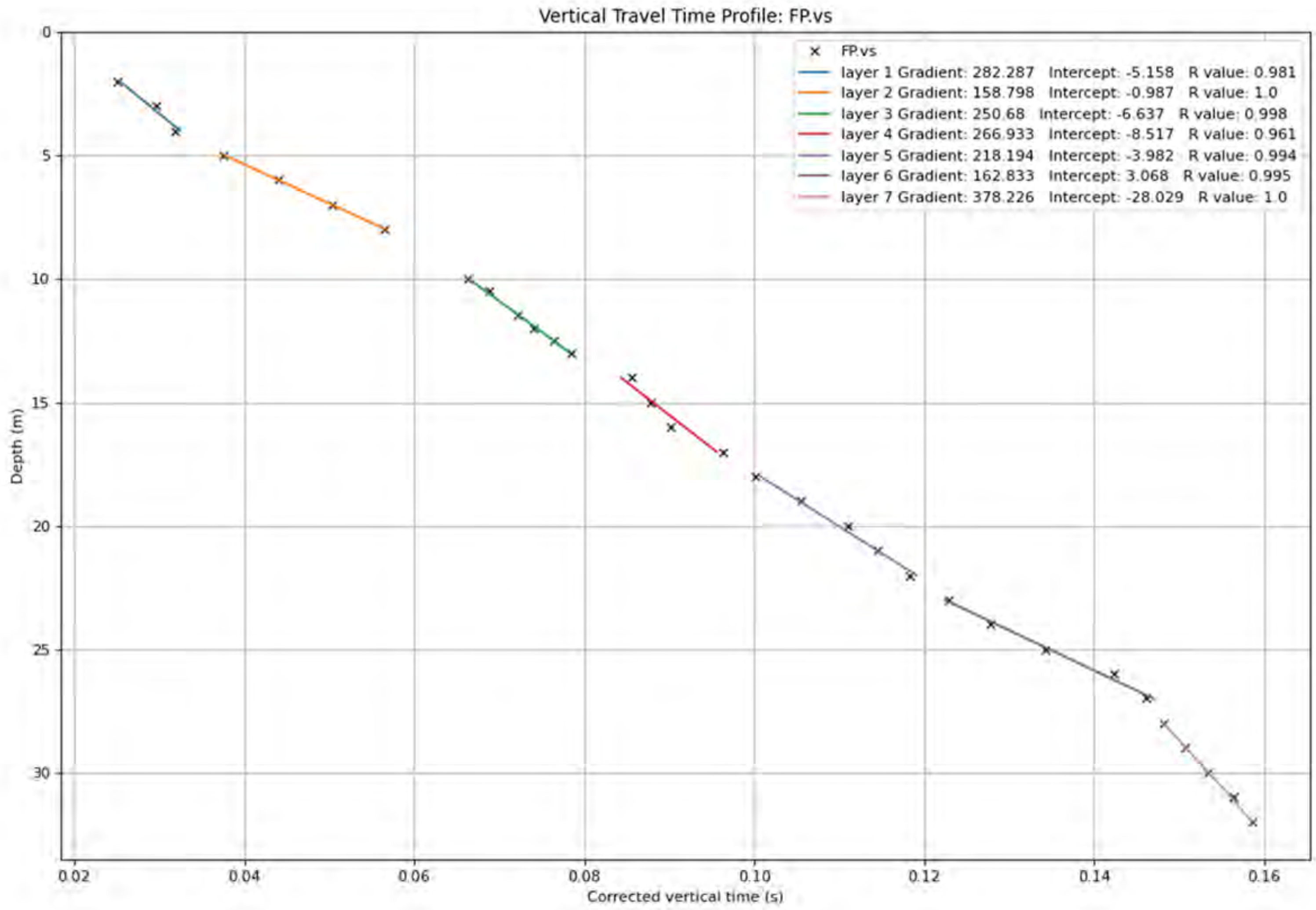
Figures	01 – 06
Appendix	A - D

FIGURES

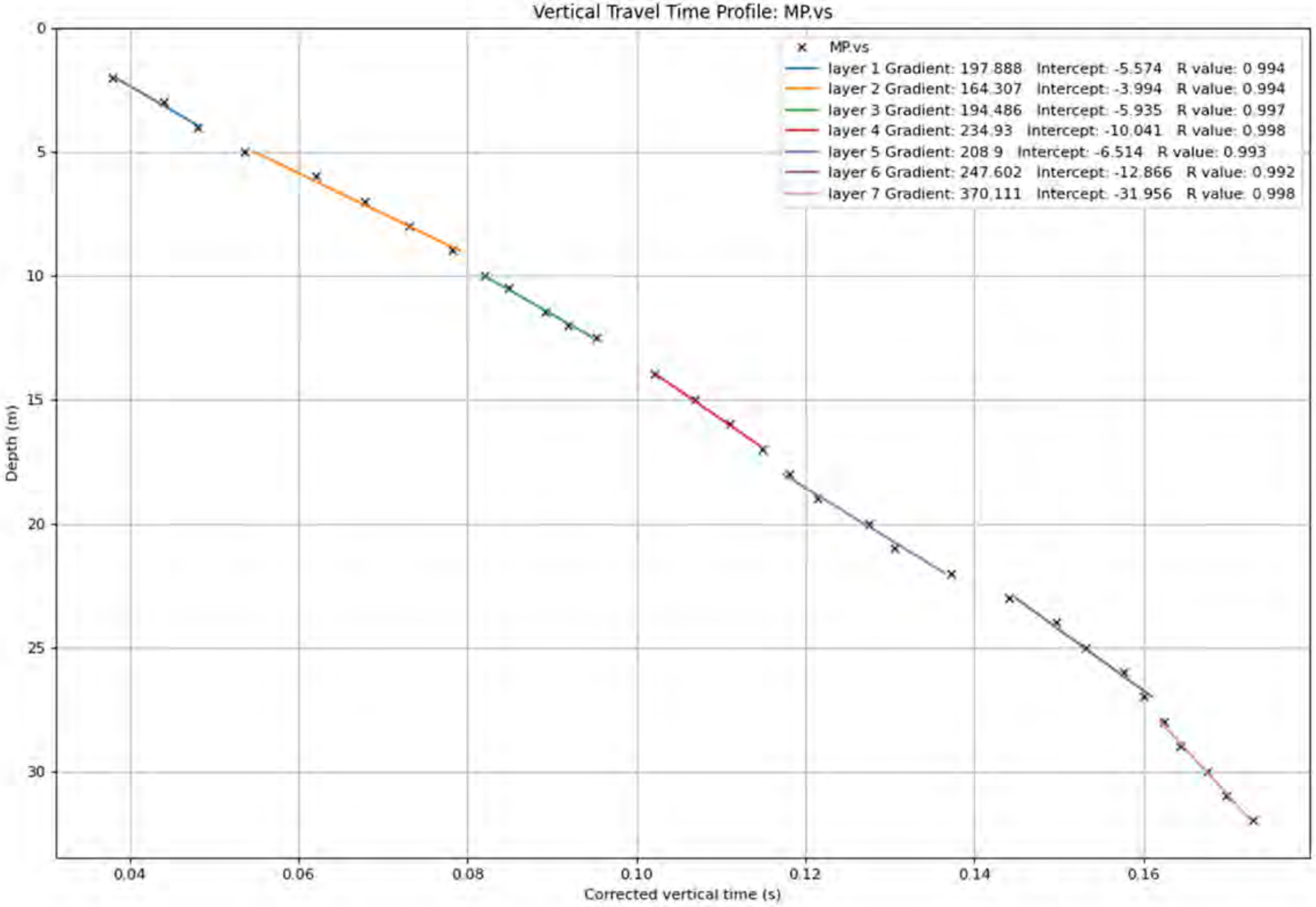


RDCL®

TITLE	BH001 Shear Wave Tx Curve – First Break	PROJECT	240669			
PROJECT	Black Bridge Culvert DH Seismic	DRAWN BY	EB	DATE	11/09/24	FIG
CLIENT	Tonkin + Taylor	CHECKED BY	OG	DATE	11/09/24	01

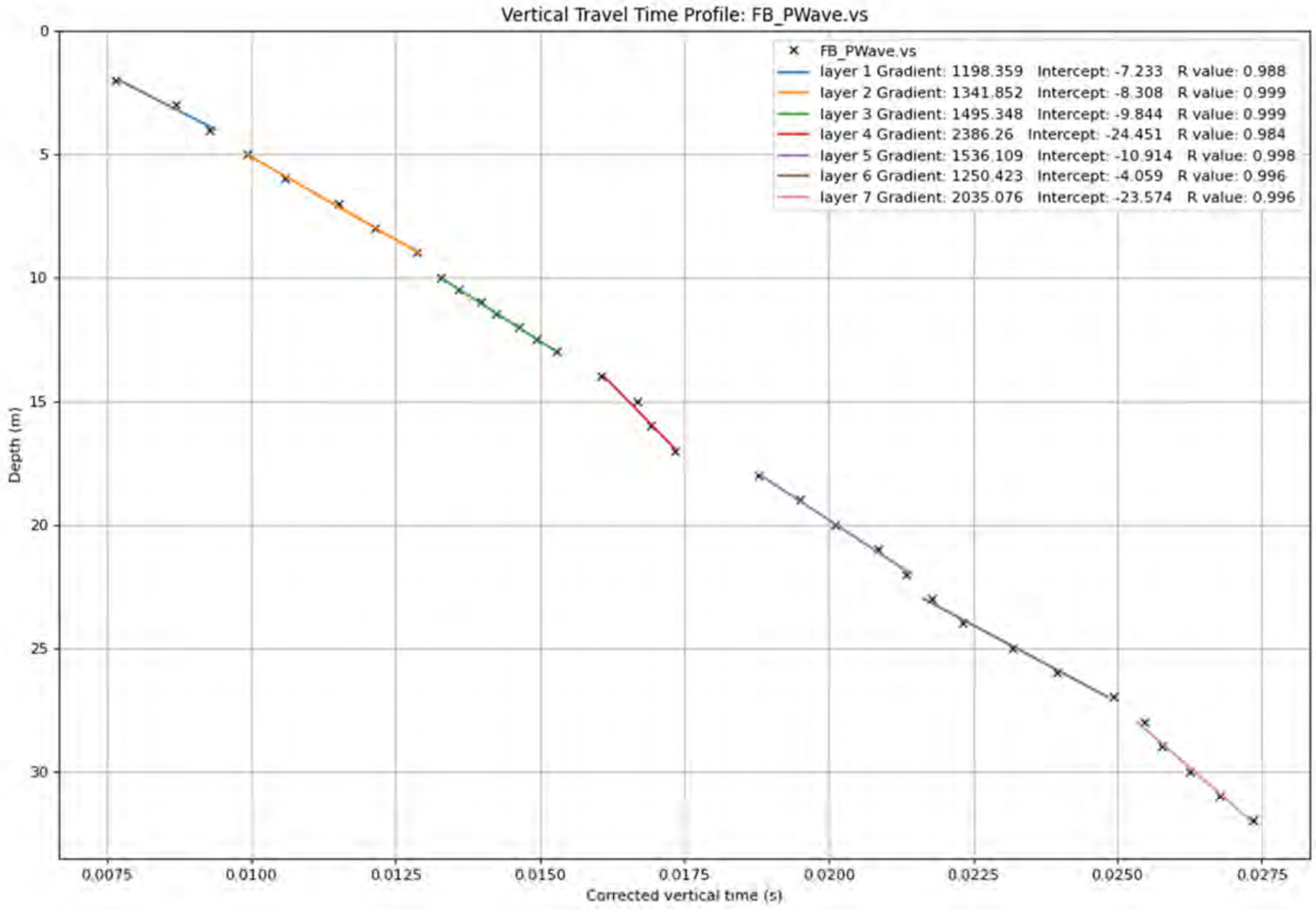


RDCL®	TITLE	BH001 Shear Wave Tx Curve – First Peak	PROJECT	240669			
	PROJECT	Black Bridge Culvert DH Seismic	DRAWN BY	EB	DATE	11/09/24	FIG
	CLIENT	Tonkin + Taylor	CHECKED BY	OG	DATE	11/09/24	02



RDCL®

TITLE	BH001 Shear Wave Tx Curve – Max Peak	PROJECT	240669			
PROJECT	Black Bridge Culvert DH Seismic	DRAWN BY	EB	DATE	11/09/24	FIG
CLIENT	Tonkin + Taylor	CHECKED BY	OG	DATE	11/09/24	03



RDCL®

TITLE	BH001 P Wave Tx Curve – First Break	PROJECT	240669			
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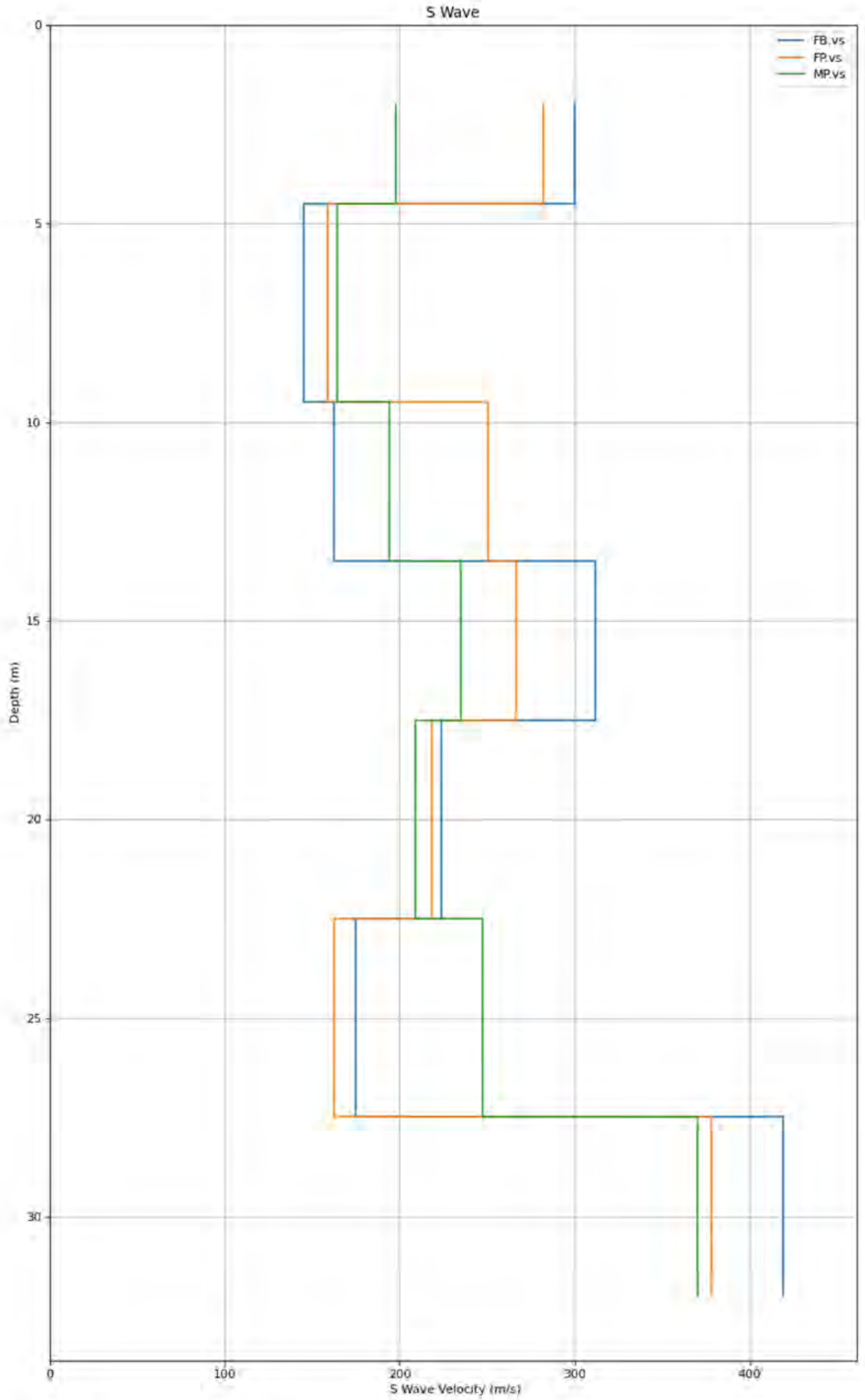
CLIENT	Tonkin + Taylor	CHECKED BY	OG	DATE	11/09/24	04



RDCL®				TITLE	BH001 Shear Wave Layer Velocities
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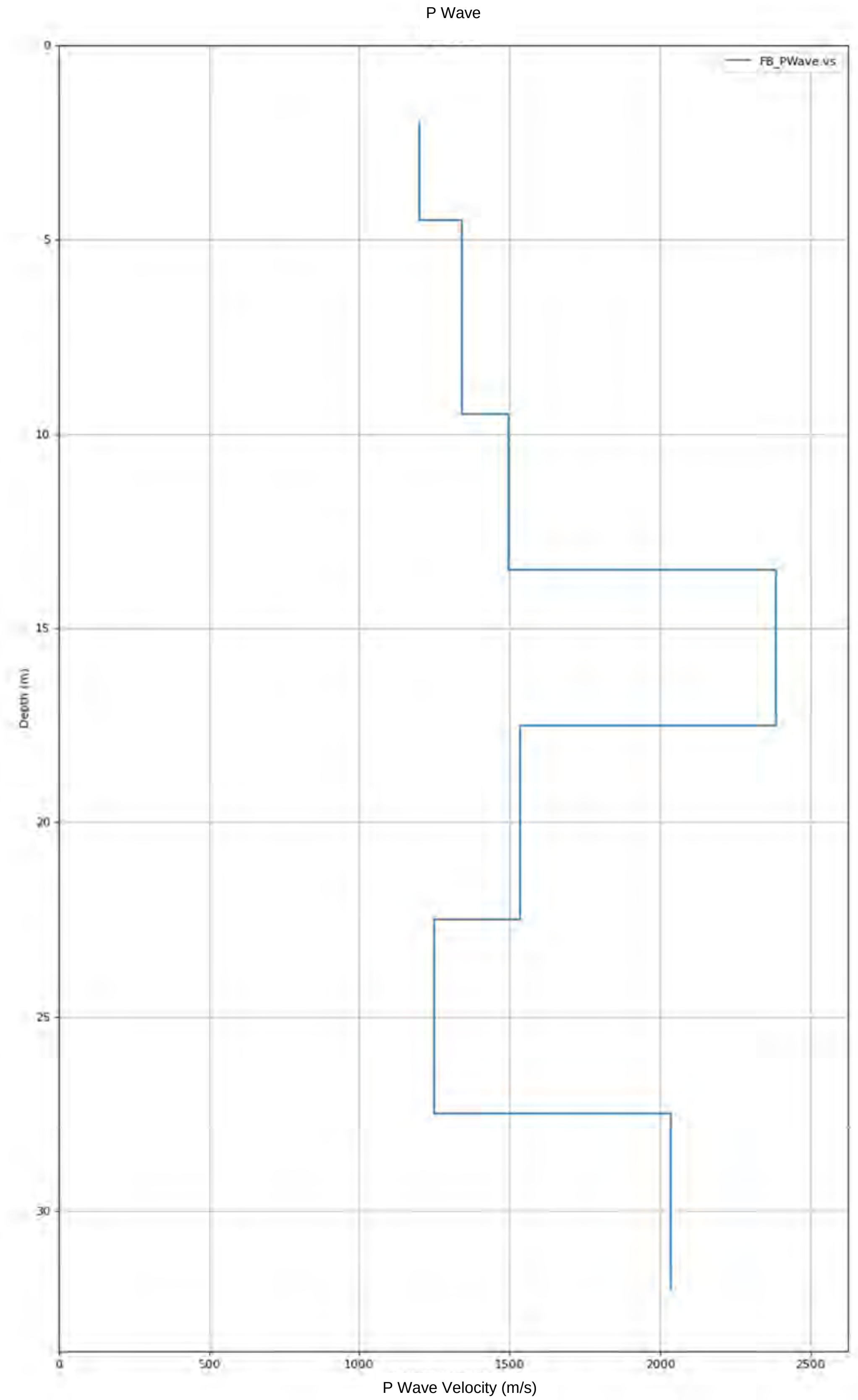
CHECKED BY	OG	DATE	11/09/24	PROJECT No.	240669 Figure No. 05



RDCL®				TITLE	BH001 Compressional Wave Layer Velocities		
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CHECKED BY	OG	DATE	11/09/24	PROJECT No.	240669	Figure No.	06

APPENDIX A – METHODOLOGY

APPENDIX A CONTENTS

A1 DOWNHOLE SEISMIC SURVEY

Data was acquired in BH001 on the 5th of September 2024.

The hole was bailed prior to testing and the first measurement was undertaken at 32 m bgl.

Downhole seismic testing was used to provide a measurement of variations in shear wave and compressional wave velocities (Vs and Vp) downhole. By measuring the difference in arrival times of the waves at known depths, the seismic wave velocities can be calculated.

Key to the survey is accurate identification of both waves.

P-wave identification relies on the fact that they are the first arriving (fastest) seismic waves, therefore the first observed signal in seismic trace is used for velocity inversion. Precise picking of P-wave signal is ensured by suited filtering of frequency range and trace waveforms.

Identification of shear waves was achieved by utilising the fact that shear waves can be polarised. By striking a shear beam in opposing directions, polarised shear waves are created. Seismic traces can then be superimposed allowing the polarisation to be observed, and shear waves to be therefore distinguished from the coda ("tail") of the earlier arriving P-waves.

A2 DOWNHOLE SEISMIC ACQUISITION METHOD

A2.1 ACQUISITION SYSTEM

Acquisition of downhole seismic data was conducted using a Geometrics Geode seismograph addressing a tri-axial Geostuff BHG-3 downhole clamping geophone.

Test intervals were spaced at 1.0 m intervals from surface to the bottom of the hole. Three (3) shots were taken with 0.5 m intervals at 12.5, 11.5, and 10.5 m depths.

A2.1.1 SEISMIC SOURCE

Shots were:

- Produced by a sledgehammer for seismic signal generation (“shots”).
- Stacked (five shots) to improve signal to noise ratio.
- Horizontally polarised “shear” wave shots were acquired in opposite directions by striking a shear beam weighted by a vehicle (to improve coupling). The shear beam was orientated to align with the orientation of one of the transverse components of the tri-axial geophone sensor.

A2.1.2 ACQUISITION PARAMETERS

Acquisition parameters were:

- Record Length - 500 ms
- Delay - 0 ms
- Sample Interval - 62.5 µs
- Acquisition Filters - OUT
- Source Offset (P & S wave) - 1.5 m
- Drillhole Stickup - 0 m

A2.1.3 POSITIONAL CONTROL

Positional control downhole was obtained using 0.5 and 1.0 m interval marks on the geophone cable. Downhole measurements were made relative to ground level during acquisition.

- Therefore, downhole positional accuracy is likely to be of the order of ±0.05 m.

Horizontal offsets to the shear beam were measured using a tape measure.

- Therefore, positional accuracy is likely to be of the order of ±0.05 m.

A2.1.4 VERTICALITY

Drill hole verticality was assumed to be vertical. A direct ray path method is used, and no travel time corrections are applied associated with variations in verticality.

A2.1.5 QUALITY ASSURANCE

There are three main field-testing QC steps.

- Before data acquisition seismic channels are checked for signal to noise interference. This is used to assess poor drill hole wall coupling or external noise on site (unwanted sound). This is commonly referred to as a noise shot.
    - A noise shot is recorded at surface, to assess signal to noise ratio and to confirm electronic systems are functioning and correct timing and trigger errors.
    - A noise shot is also recorded at depth, typically at a mid-point in the drill hole. This confirms in-hole signal to noise ratio and to ensure the tool is functioning after deployment.
- Every shot interval has stacked signal shots of three to five shots per test depth. Each shot is checked for timing errors and poor coupling before being saved to the data sets. Shots were stacked in field to improve signal to noise ratios.
- Shear wave seismic shots are recorded in opposing directions to establish a polarised shear wave. Polarised shear waves reduce the chances of tube waves being confused with shear wave arrivals.

A2.2 RESOLUTION LIMITATIONS

Layer velocities were defined based on changes in arrival time slopes and geological intervals detailed in drill hole logs provided to RDCL. Velocities are calculated from calculating the slope from the Tx curves (time depth). It is usually not appropriate to calculate interval velocities using just two points (over successive measurements) as large errors in velocity are likely (ASTM – D7400-08).

A3 DOWNHOLE SEISMIC PROCESSING

Data processing consisted of:

- Phase 1 – Initial processing and data filtering.
- Phase 2 – First break picking and wave amplitude picking.
- Phase 3 – Data presentation and calculation of interval velocities.

A3.1 PHASE 1 – INITIAL PROCESSING

Raw data files were imported into an RDCL proprietary Python script. The script undertook the following generalised processes:

- Split of channels 1, 2 and 3 (data orientated in the vertical, north and east components).
- Sort into left, right and vertically polarised shots. Vertically polarised shots were used for P-wave picking (Channel 1). Channel 2 was chosen for Shear wave arrival picking.
- Deletion of unrequired traces.
- Correct assignment of depth geometry from header.
- Merge of traces into separate gathers for left, right and vertical polarised shots.
- Bandpass filtering.
- Visual assessment of traces.

A3.2 PHASE 2 – ARRIVAL TIME 'PICKS'

Gathers were subsequently imported into the Seisimager Pickwin module software for picking.

P-wave arrival times were manually picked by identifying the first wave arrivals on gather after proper bandpass filtering was applied. Shear wave arrival times were manually picked by identifying waves that were polarised.

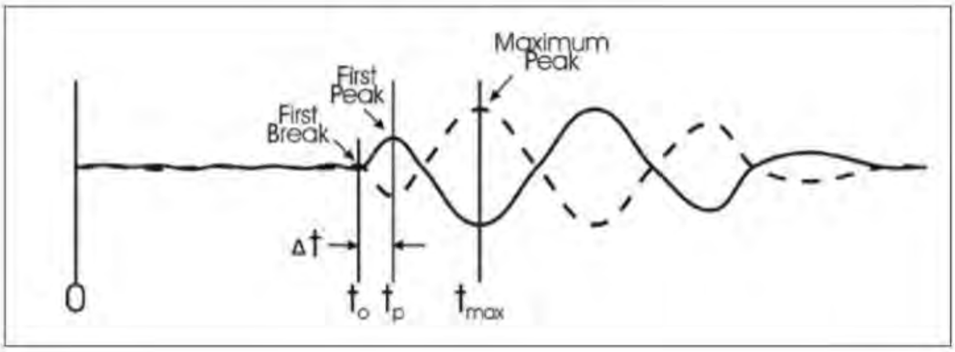
Where waves could not be reliably identified, a gap was left, and that data was not picked.

Shear wave identification may be affected by:

- Decreases in signal to noise ratio.
- Interference from tube waves.
- Disturbance by the coda of the P-wave arrivals.

In processing downhole seismic surveys RDCL usually makes picks at three separate locations on each trace (Schematic 1) and interval/layer velocities are calculated using each of the picking methods.

SCHEMATIC 1 - SHEAR WAVE VELOCITY PICKING



The three methods are called:

- First Break (FB)
- First Peak (FP).
- Maximum Peak (MP).

Three methods of shear wave picking are used to reduce and assess the uncertainty in the shear wave velocity interval/layer calculations.

P-wave arrivals were picked using only the "First Break" method, as there is usually minimal ambiguity when picking P-waves.

Arrival times were exported from Pickwin into Python code to create a velocity model.

A3.3 PHASE 3 – VELOCITY CALCULATIONS

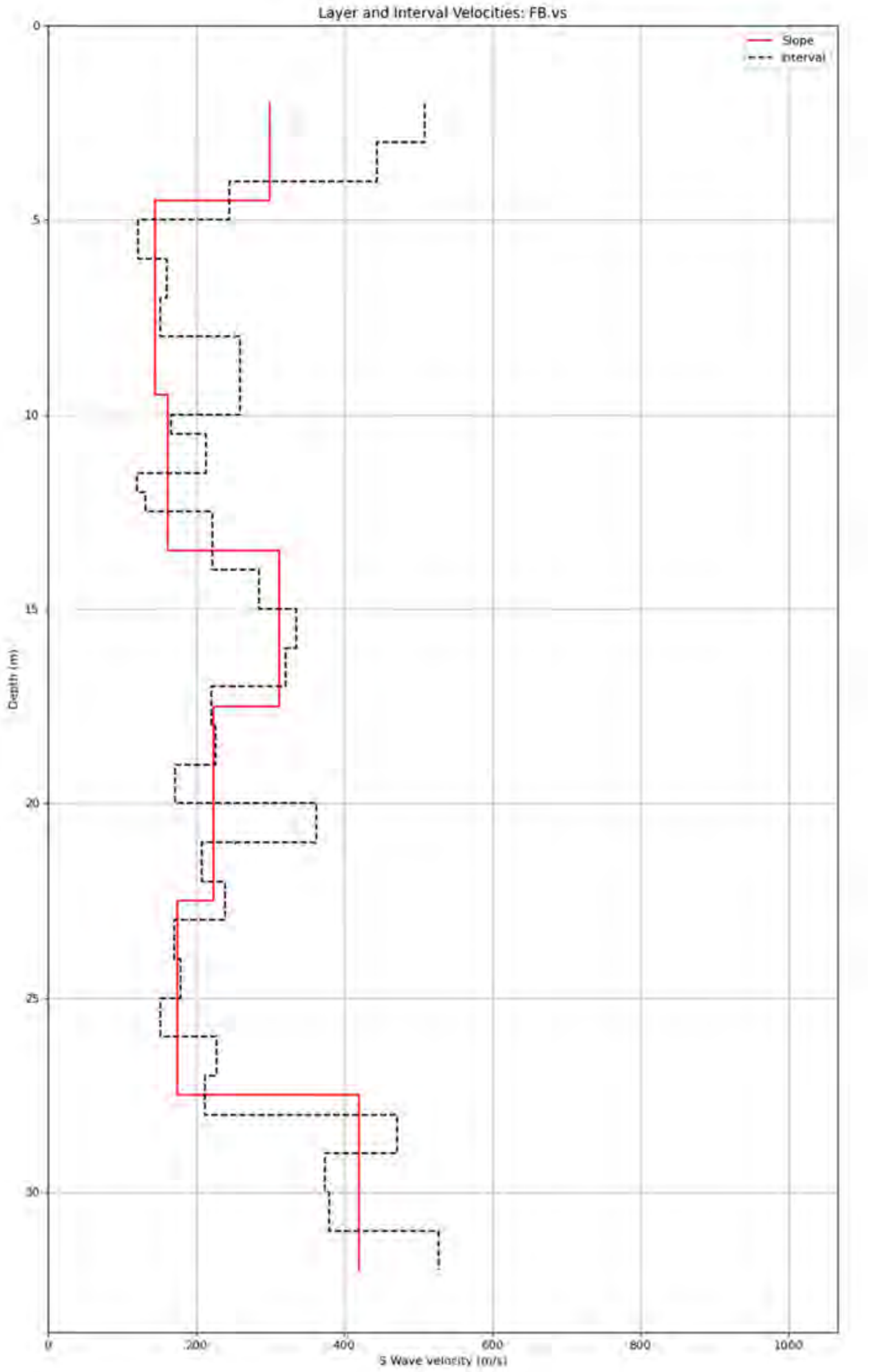
Arrival times were imported into an RDCL proprietary Python script for presentation and calculation of interval velocities and layer velocities. Intervals were defined from changes in slope in the arrival time data and the drill hole log provided to RDCL.

Geometric corrections were made to the travel times for shot/collar horizontal offsets and casing stick up.

A4 REFERENCES

Geometrics (2014). http://www.geometrics.com/geometrics-products/seismographs/download-seismograph-software/#SeisImager/2D

APPENDIX B – INTERVAL VELOCITIES



RDCL®				TITLE	Appendix B—Interval Velocities—First Break		
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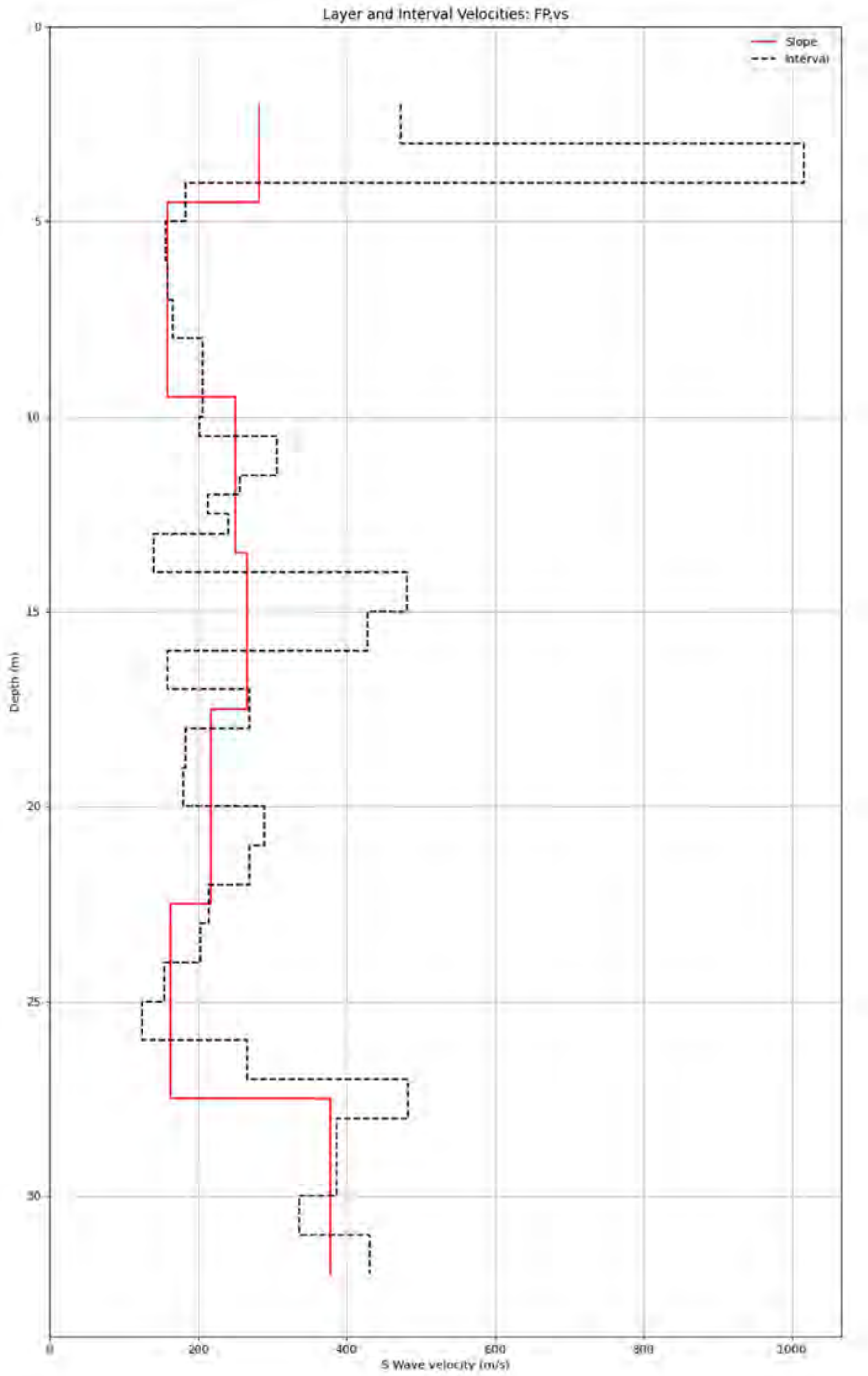
CHECKED BY	OG	DATE	11/09/24	PROJECT No.	240669	Figure No.	B01



RDCL®				TITLE	Appendix B—Interval Velocities—First Peak		
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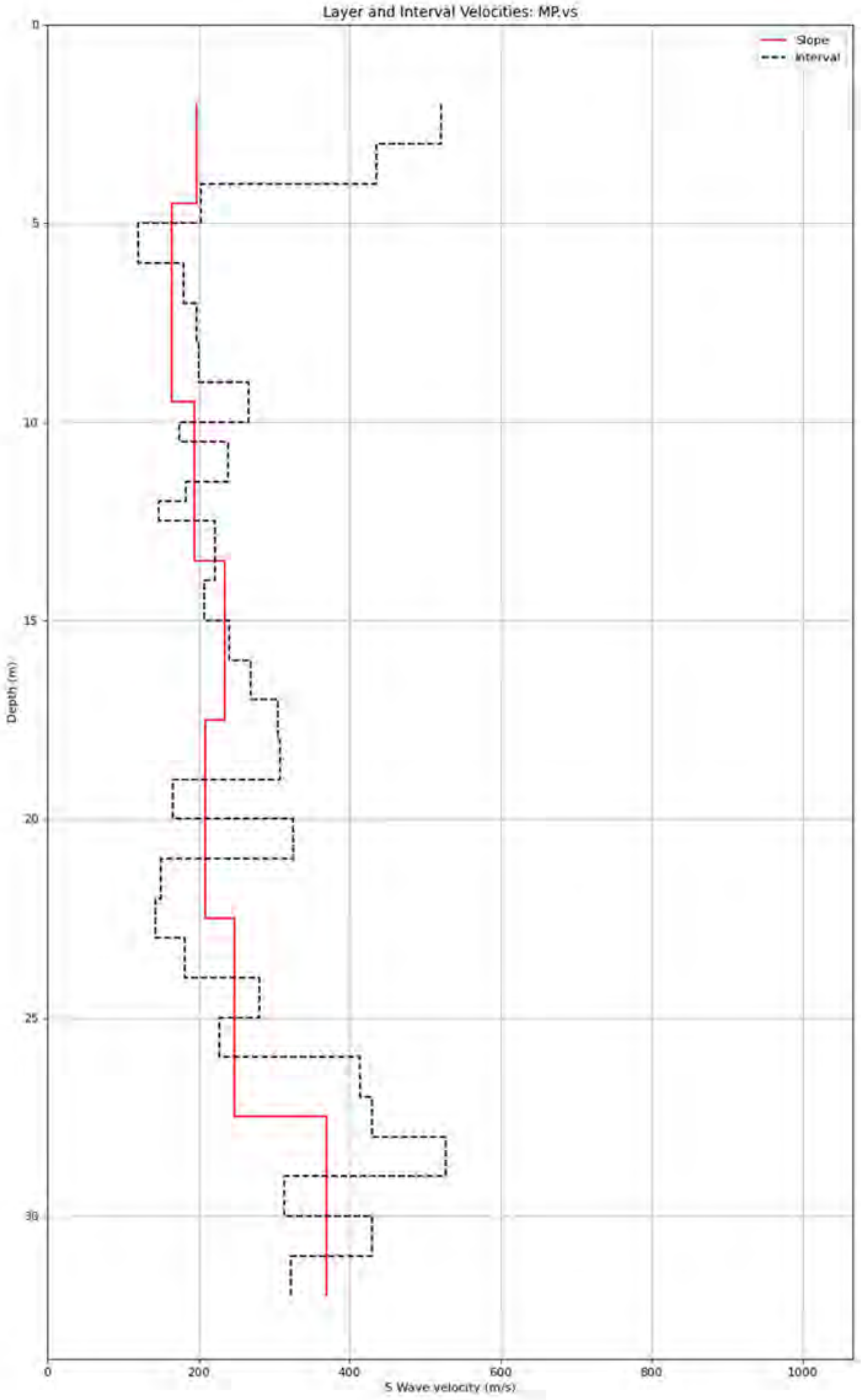

RDCL®				TITLE	Appendix B—Interval Velocities—Max Peak
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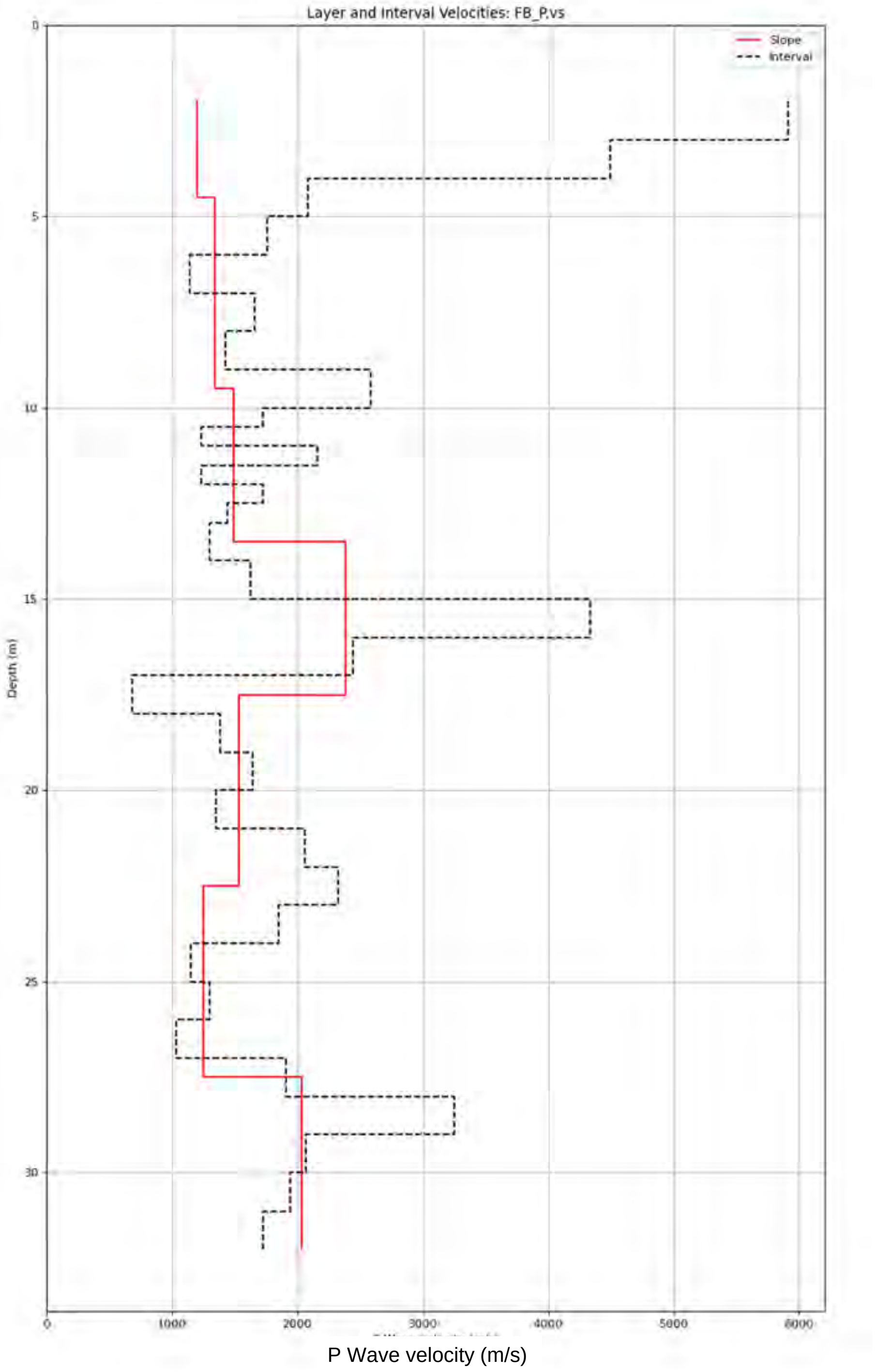

RDCL®				TITLE	Appendix B—Interval Velocities—Vp		
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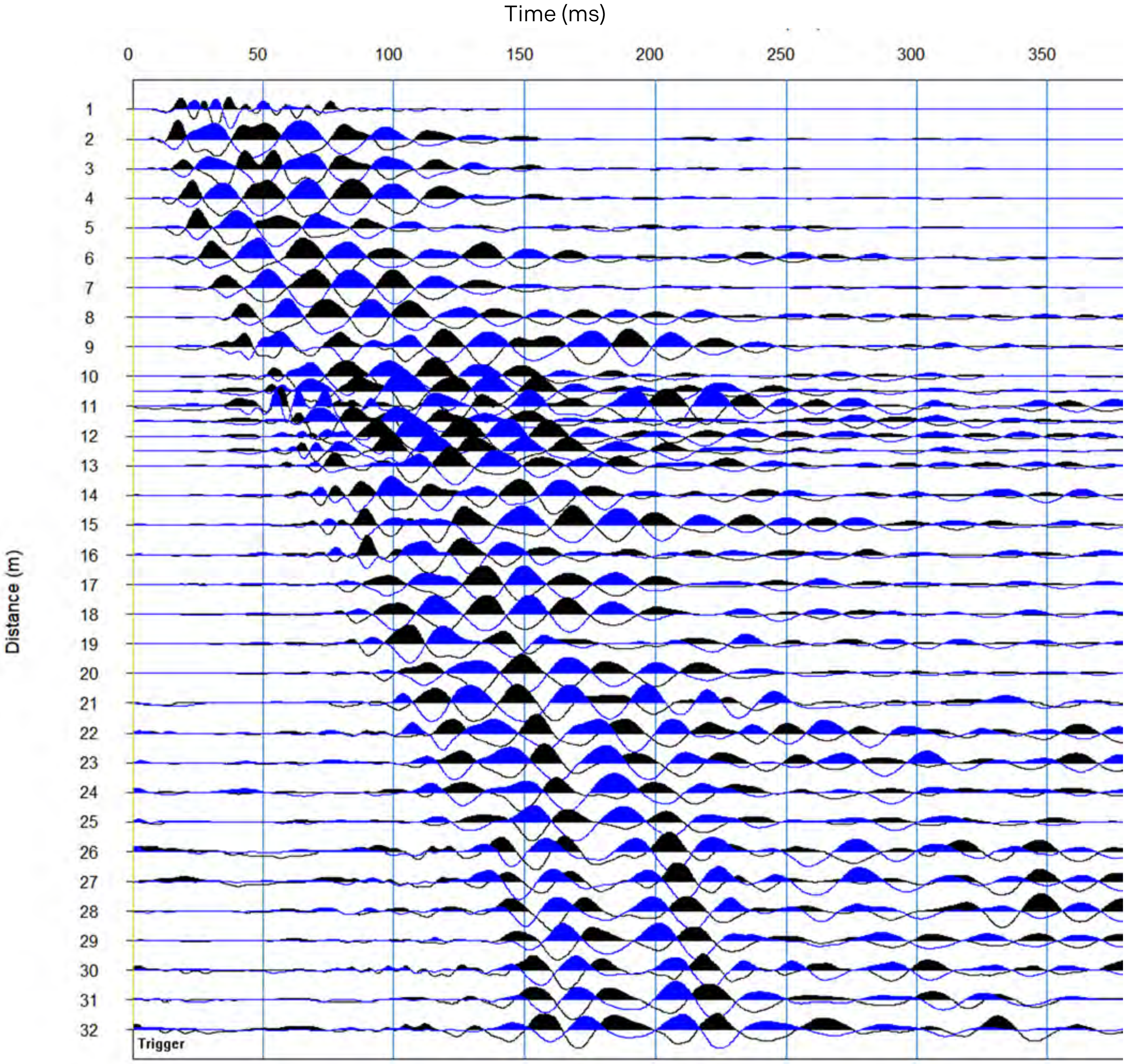
APPENDIX C – TRACE GATHERS



RDCL®

				TITLE	Appendix C—Vs Trace Gather
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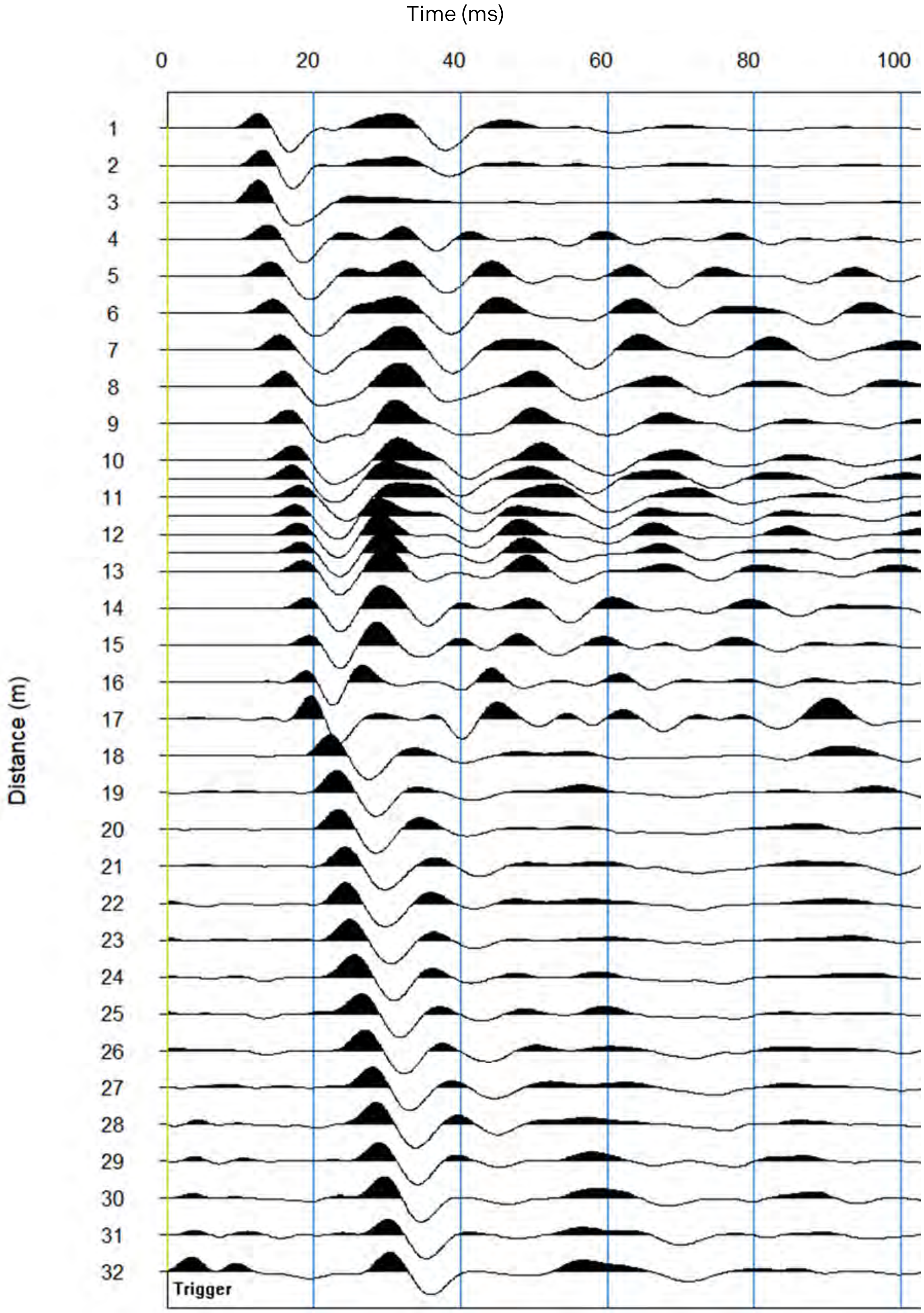

RDCL®

				TITLE	Appendix C—Vp Trace Gather	
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CHECKED BY	OG	DATE	11/09/24	PROJECT No.	240669	Figure No.	C02

APPENDIX D – TABULATED ARRIVAL TIMES

TABLE D1 –BH001 - VS & VP ARRIVAL TIMES

Vs – First Break		Vs – First Peak		Vs – Max Peak		Vp – First Break	
Depth	Arrival Time	Depth	Arrival Time	Depth	Arrival Time	Depth	Arrival Time
(m)	(s)	(m)	(s)	(m)	(s)	(m)	(s)
2.0	0.0184	2.0	0.0251	2.0	0.0380	2.0	0.0077
3.0	0.0221	3.0	0.0297	3.0	0.0440	3.0	0.0087
4.0	0.0250	4.0	0.0319	4.0	0.0480	4.0	0.0093
5.0	0.0293	5.0	0.0376	5.0	0.0536	5.0	0.0099
6.0	0.0373	6.0	0.0441	6.0	0.0620	6.0	0.0106
7.0	0.0436	7.0	0.0504	7.0	0.0678	7.0	0.0115
8.0	0.0501	8.0	0.0565	8.0	0.0730	8.0	0.0121
9.0	-	9.0	-	9.0	0.0781	9.0	0.0129
10.0	0.0580	10.0	0.0663	10.0	0.0820	10.0	0.0133
10.5	0.0610	10.5	0.0688	10.5	0.0849	10.5	0.0136
11.0	-	11.0	-	11.0	-	11.0	0.0140
11.5	0.0657	11.5	0.0721	11.5	0.0891	11.5	0.0142
12.0	0.0698	12.0	0.0741	12.0	0.0919	12.0	0.0146
12.5	0.0736	12.5	0.0764	12.5	0.0953	12.5	0.0149
13.0	-	13.0	0.0785	13.0	-	13.0	0.0153
14.0	0.0804	14.0	0.0856	14.0	0.1021	14.0	0.0161
15.0	0.0839	15.0	0.0878	15.0	0.1069	15.0	0.0167
16.0	0.0869	16.0	0.0901	16.0	0.1111	16.0	0.0169
17.0	0.0900	17.0	0.0964	17.0	0.1148	17.0	0.0173
18.0	0.0946	18.0	0.1001	18.0	0.1181	18.0	0.0188
19.0	0.0990	19.0	0.1056	19.0	0.1214	19.0	0.0195
20.0	0.1048	20.0	0.1111	20.0	0.1274	20.0	0.0201
21.0	0.1076	21.0	0.1145	21.0	0.1305	21.0	0.0209
22.0	0.1124	22.0	0.1183	22.0	0.1372	22.0	0.0213
23.0	0.1166	23.0	0.1229	23.0	0.1441	23.0	0.0218
24.0	0.1224	24.0	0.1278	24.0	0.1496	24.0	0.0223
25.0	0.1280	25.0	0.1343	25.0	0.1532	25.0	0.0232
26.0	0.1346	26.0	0.1423	26.0	0.1576	26.0	0.0240
27.0	0.1390	27.0	0.1461	27.0	0.1600	27.0	0.0249
28.0	0.1437	28.0	0.1482	28.0	0.1624	28.0	0.0255
29.0	0.1458	29.0	0.1507	29.0	0.1643	29.0	0.0258
30.0	0.1485	30.0	0.1533	30.0	0.1675	30.0	0.0263
31.0	0.1512	31.0	0.1563	31.0	0.1698	31.0	0.0268
32.0	0.1531	32.0	0.1586	32.0	0.1729	32.0	0.0273

Appendix D Laboratory Test Results

GEOTECHNICS

29 August 2024
Our Ref: 1096487.0000.0.0/Rep1
Customer Ref: 1017033.2003

Tonkin & Taylor Limited
PO BOX 5271
AUCKLAND 1141

Attention: Scott Zhang

Dear Scott

Te Ararata Stage 2: Design

Laboratory Test Report

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

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

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

This report may be reproduced only in full.

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

GEOTECHNICS LTD

Report approved by:

....
Kelsey Sanderson
Laboratory Technician
Key Technical Person

Authorised for Geotechnics by:

....
Steven Anderson
Project Director

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

29-Aug-24
T:\GeotechnicsGroup\Projects\1096487\IssuedDocuments – Report 1

Customer Ref: 1017033.2003

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

TEST DETAILS

LOCATION	**ID**	DS_9.6 m		
	Description	Te Aratata Stage 2: Design		
	Data	N/A		
SAMPLE	**Geotechnics ID**	A0762		
	Reference	-	**Depth**	9.6-9.9 m
	Description	silty SAND; dark grey. Soft, wet, quick		
SPECIMEN	**Reference**	-	**Depth**	-
	Description	-		

TEST RESULTS

Liquid Limit	**Not Obtainable**
Plastic Limit	**Non-plastic**
Plasticity Index	**Not Obtainable**

TEST REMARKS

• The material used for testing was natural, fraction passing a 425um sieve. • Both the final Liquid Limit and Plastic Limit results were unobtainable during the course of testing. • Date tested 28/08/2024

This test result is IANZ accredited.

Approved by KTP KESA **Date** 28/08/2024

Determination of the Water Content & Fines Content - GEO190-21 In-house Method

TEST DETAILS

LOCATION	**ID**	DS_13.75 m		
	Description	Te Ararata Stage 2: Design		
	Data	N/A		
SAMPLE	**Geotechnics ID**	A0763		
	Reference	-	**Depth**	13.75-14 m
	Description	Sitly SAND; dark grey. Moist		
SPECIMEN	**Reference**	-	**Depth**	-
	Description	-		

TEST RESULTS

Natural Water Content	**37.5%**
Fines Content <75µm Sieve	**53.0%**
Fines Content <63µm Sieve	**51.2%**

TEST REMARKS

- The material used for testing was natural, whole soil.
- Date tested 28/08/2024

This test is IANZ accredited.

Approved by KTP	KESA	**Date**	28/08/2024

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

TEST DETAILS

LOCATION	**ID**	DS_13.75 m		
	Description	Te Aratata Stage 2: Design		
	Data	N/A		
SAMPLE	**Geotechnics ID**	A0763		
	Reference	-	**Depth**	13.75-14 m
	Description	Silty SAND; dark grey. Wet		
SPECIMEN	**Reference**	-	**Depth**	-
	Description	-		

TEST RESULTS

Liquid Limit	**Not Obtainable**
Plastic Limit	**Non-plastic**
Plasticity Index	**Not Obtainable**

TEST REMARKS

• The material used for testing was natural, fraction passing a 425um sieve. • Both the final Liquid Limit and Plastic Limit results were unobtainable during the course of testing. • Date Tested 28/08/2024

This test result is IANZ accredited.

Approved by KTP	KESA	**Date**	28/08/2024

Determination of the Water Content & Fines Content - GEO190-21 In-house Method

TEST DETAILS

LOCATION	**ID**	DS_16 m		
	Description	Te Ararata Stage 2: Design		
	Data	N/A		
SAMPLE	**Geotechnics ID**	A0764		
	Reference	-	**Depth**	16.0-16.3 m
	Description	Silty SAND; dark grey. Wet, dilatant		
SPECIMEN	**Reference**	-	**Depth**	-
	Description	-		

TEST RESULTS

Natural Water Content	**28.0%**
Fines Content <75µm Sieve	**11.7%**
Fines Content <63µm Sieve	**11.3%**

TEST REMARKS

- The material used for testing was natural, whole soil.
- Date tested 28/08/2024

This test is IANZ accredited.

Approved by KTP	KESA	**Date**	28/08/2024

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

TEST DETAILS

LOCATION	**ID**	DS_16 m		
	Description	Te Aratata Stage 2: Design		
	Data	N/A		
SAMPLE	**Geotechnics ID**	A0764		
	Reference	-	**Depth**	16.0-16.3 m
	Description	Silty SAND; dark grey. Wet, dilatant		
SPECIMEN	**Reference**	-	**Depth**	-
	Description	-		

TEST RESULTS

Liquid Limit	**Not Obtainable**
Plastic Limit	**Non-plastic**
Plasticity Index	**Not Obtainable**

TEST REMARKS

• The material used for testing was natural, fraction passing a 425um sieve. • Both the final Liquid Limit and Plastic Limit results were unobtainable during the course of testing. • Date tested 28/08/2024

This test result is IANZ accredited.

Approved by KTP KESA **Date** 28/08/2024

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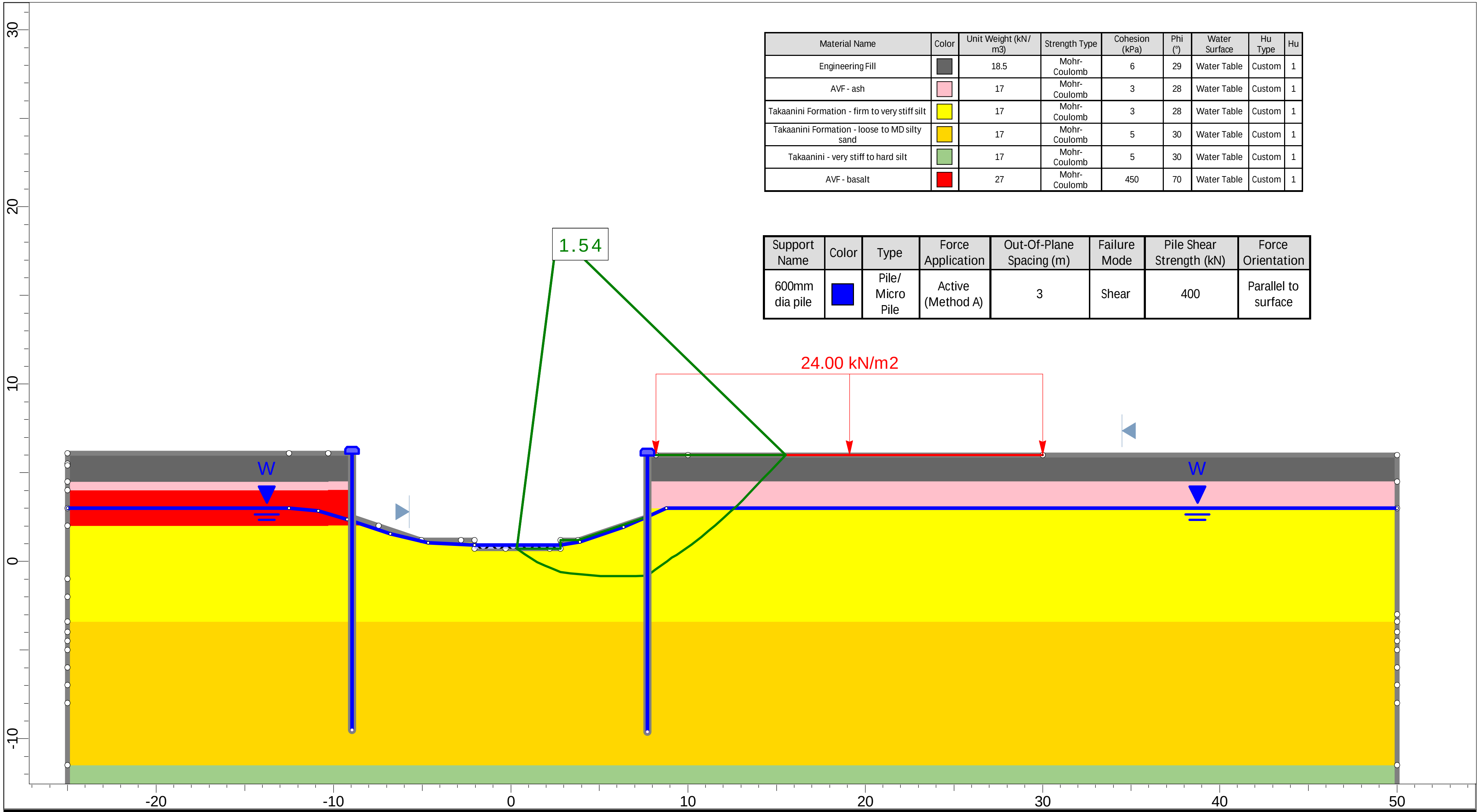
Appendix B Stability Assessment Outputs



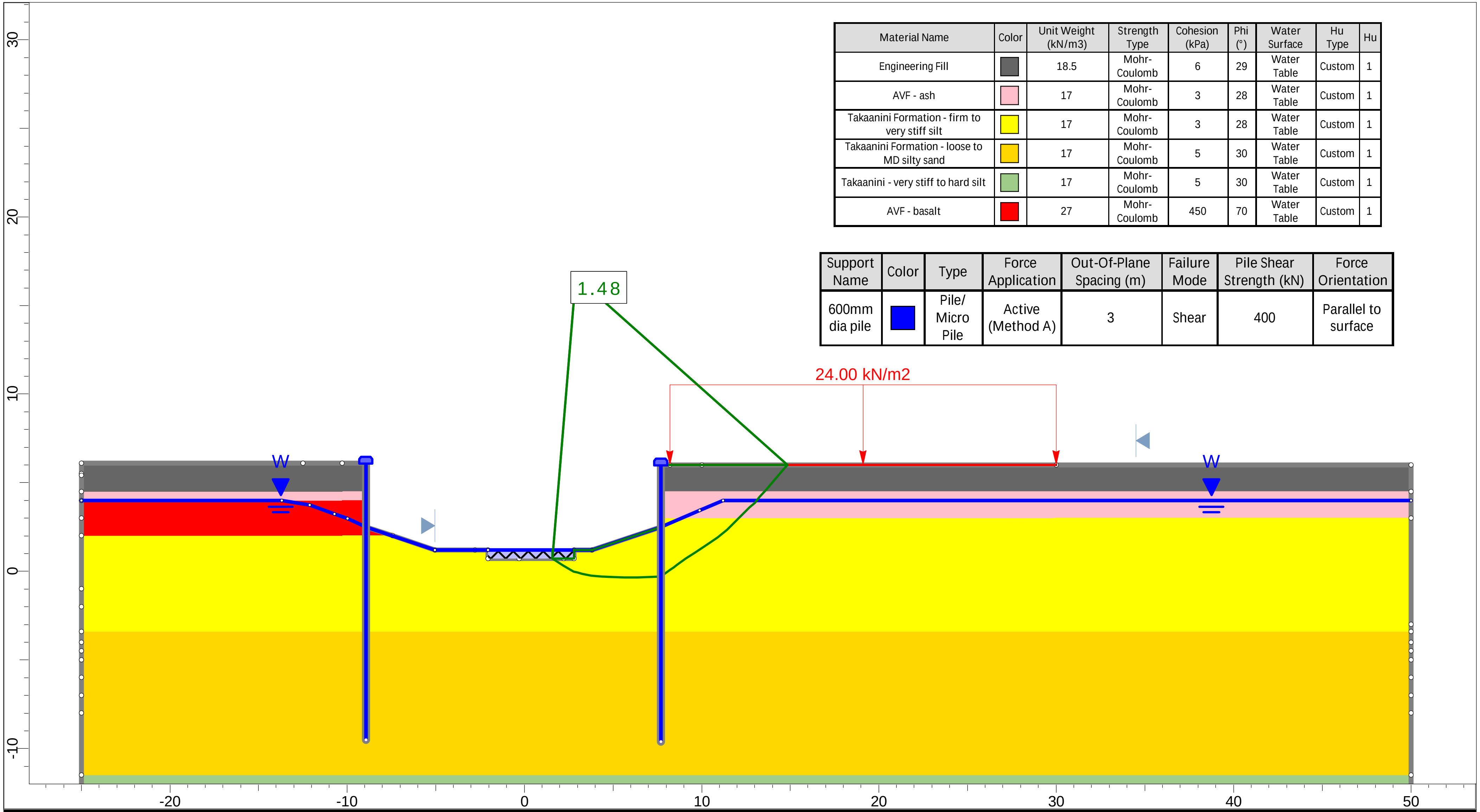
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type	Hu
Engineering Fill	⬛	18.5	Mohr-Coulomb	6	29	Water Table	Custom	1
AVF - ash	🟪	17	Mohr-Coulomb	3	28	Water Table	Custom	1
Takaanini Formation - firm to very stiff silt	🟨	17	Mohr-Coulomb	3	28	Water Table	Custom	1
Takaanini Formation - loose to MD silty sand	🟧	17	Mohr-Coulomb	5	30	Water Table	Custom	1
Takaanini - very stiff to hard silt	🟩	17	Mohr-Coulomb	5	30	Water Table	Custom	1
AVF - basalt	🟥	27	Mohr-Coulomb	450	70	Water Table	Custom	1

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Failure Mode	Pile Shear Strength (kN)	Force Orientation
600mm dia pile	🟦	Pile/ Micro Pile	Active (Method A)	3	Shear	400	Parallel to surface

rocscience

SLIDEINTERPRET 9.034

Project	Te Ararata - Walmsley Road Bridge Upgrade		
Group	Bridge Abutment	Scenario	LT static
Analysed By/Checked By	ZHLI/ABL	Company	Tonkin + Taylor
Date	11/10/2024	File Name	Abutment Stability 11Oct.slmd



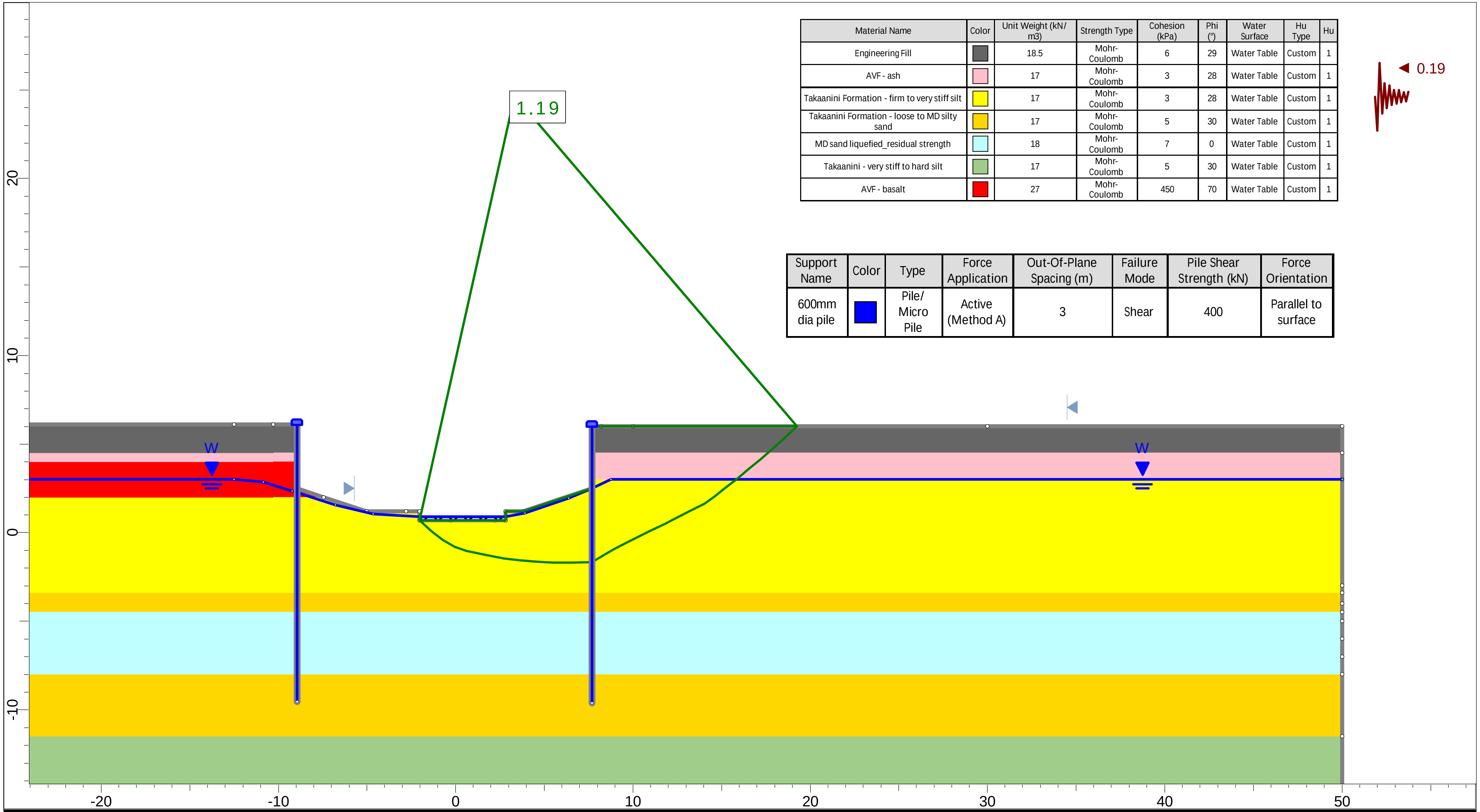
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type	Hu
Engineering Fill	⬛	18.5	Mohr-Coulomb	6	29	Water Table	Custom	1
AVF - ash	▨	17	Mohr-Coulomb	3	28	Water Table	Custom	1
Takaanini Formation - firm to very stiff silt	🟨	17	Mohr-Coulomb	3	28	Water Table	Custom	1
Takaanini Formation - loose to MD silty sand	🟧	17	Mohr-Coulomb	5	30	Water Table	Custom	1
Takaanini - very stiff to hard silt	🟩	17	Mohr-Coulomb	5	30	Water Table	Custom	1
AVF - basalt	🟥	27	Mohr-Coulomb	450	70	Water Table	Custom	1

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Failure Mode	Pile Shear Strength (kN)	Force Orientation
600mm dia pile	🟦	Pile/ Micro Pile	Active (Method A)	3	Shear	400	Parallel to surface

rocscience

SLIDEINTERPRET 9.034

Project	Te Ararata - Walmsley Road Bridge Upgrade
Group	Bridge Abutment
Scenario	Elevated GWL
Analysed By/Checked By	ZHLI/ABL
Company	Tonkin + Taylor
Date	11/10/2024
File Name	Abutment Stability 11Oct.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type	Hu
Engineering Fill	⬛	18.5	Mohr-Coulomb	6	29	Water Table	Custom	1
AVF - ash	🟪	17	Mohr-Coulomb	3	28	Water Table	Custom	1
Takaanini Formation - firm to very stiff silt	🟨	17	Mohr-Coulomb	3	28	Water Table	Custom	1
Takaanini Formation - loose to MD silty sand	🟧	17	Mohr-Coulomb	5	30	Water Table	Custom	1
MD sand liquefied_residual strength	🟦	18	Mohr-Coulomb	7	0	Water Table	Custom	1
Takaanini - very stiff to hard silt	🟩	17	Mohr-Coulomb	5	30	Water Table	Custom	1
AVF - basalt	🟥	27	Mohr-Coulomb	450	70	Water Table	Custom	1

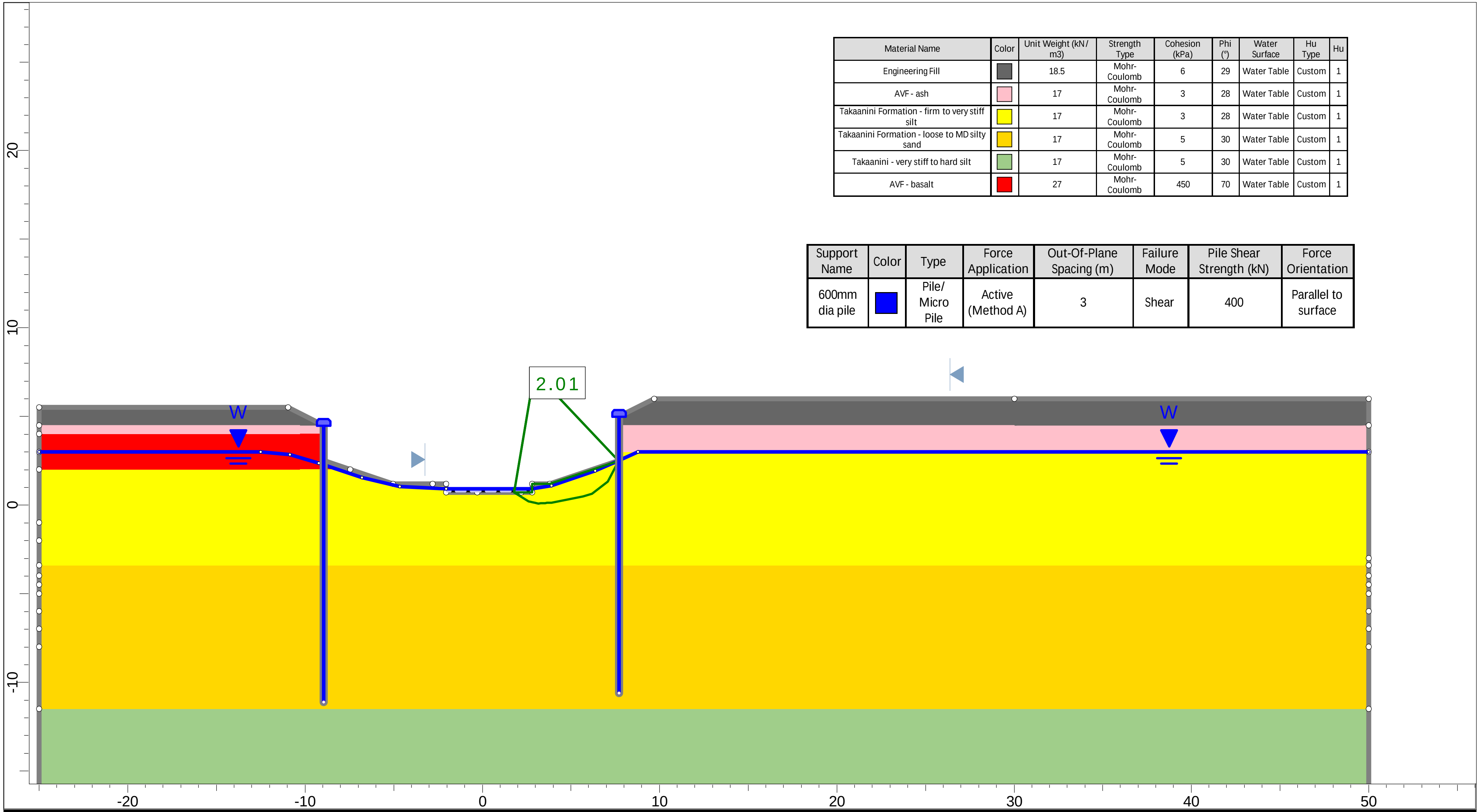
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Failure Mode	Pile Shear Strength (kN)	Force Orientation
600mm dia pile	🟦	Pile/ Micro Pile	Active (Method A)	3	Shear	400	Parallel to surface

rocscience

SLIDEINTERPRET 9.034

Project
Te Ararata - Walmsley Road Bridge Upgrade

Group	Bridge Abutment	Scenario	ULS seismic (1000-year event, liquefaciton)
Analysed By/Checked By	ZHLI/ABL	Company	Tonkin + Taylor
Date	11/10/2024	File Name	Abutment Stability 11Oct.slmd



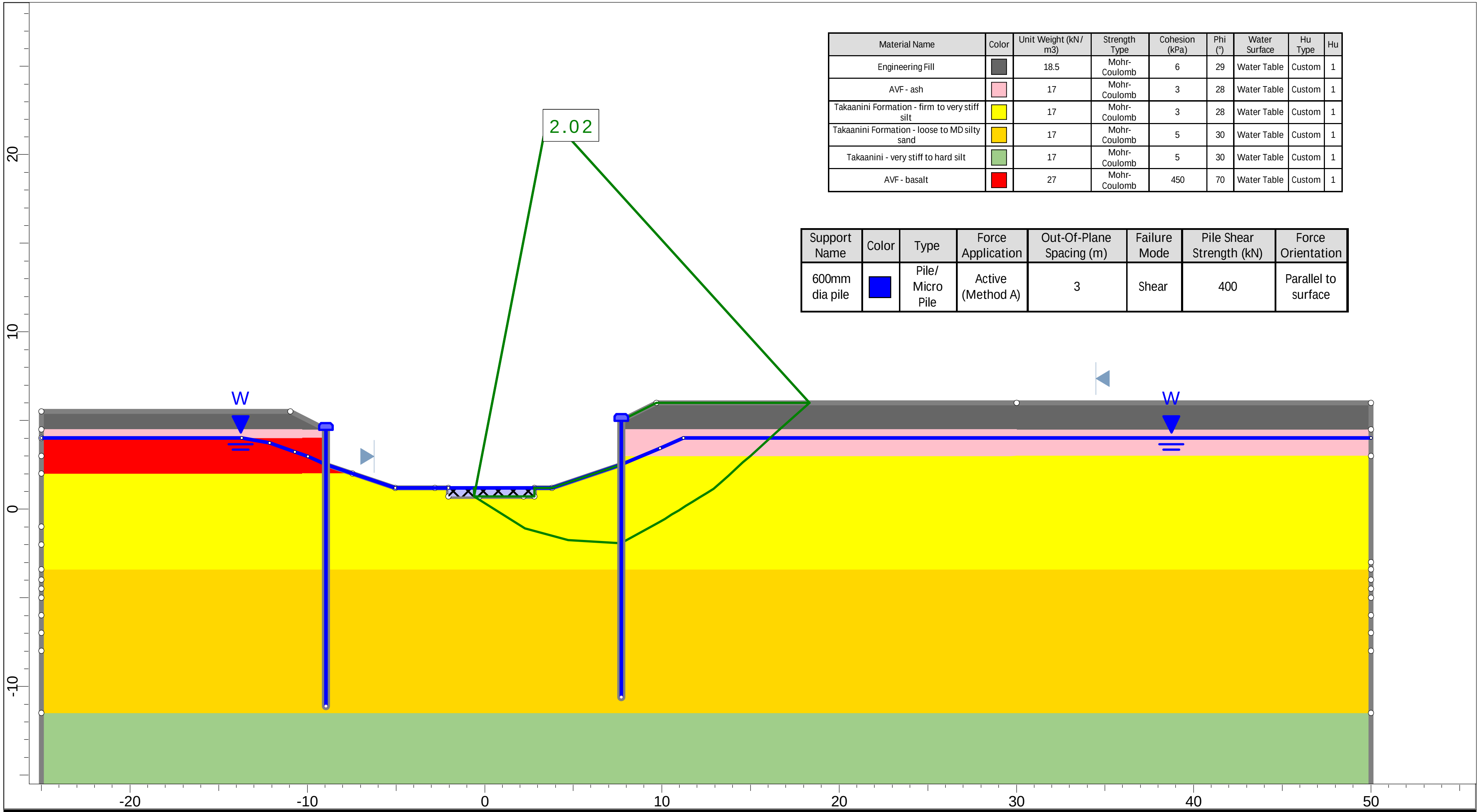
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type	Hu
Engineering Fill	⬛	18.5	Mohr-Coulomb	6	29	Water Table	Custom	1
AVF - ash	■	17	Mohr-Coulomb	3	28	Water Table	Custom	1
Takaanini Formation - firm to very stiff silt	🟨	17	Mohr-Coulomb	3	28	Water Table	Custom	1
Takaanini Formation - loose to MD silty sand	🟨	17	Mohr-Coulomb	5	30	Water Table	Custom	1
Takaanini - very stiff to hard silt	🟩	17	Mohr-Coulomb	5	30	Water Table	Custom	1
AVF - basalt	🟥	27	Mohr-Coulomb	450	70	Water Table	Custom	1

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Failure Mode	Pile Shear Strength (kN)	Force Orientation
600mm dia pile	🟦	Pile/ Micro Pile	Active (Method A)	3	Shear	400	Parallel to surface

rocscience

SLIDEINTERPRET 9.034

Project	Te Ararata - Walmsley Road Bridge Upgrade
Group	River bank retaining structure
Scenario	LT static
Analysed By/Checked By	ZHLI/ABL
Company	Tonkin + Taylor
Date	11/10/2024
File Name	Abutment Stability 11Oct.slmd



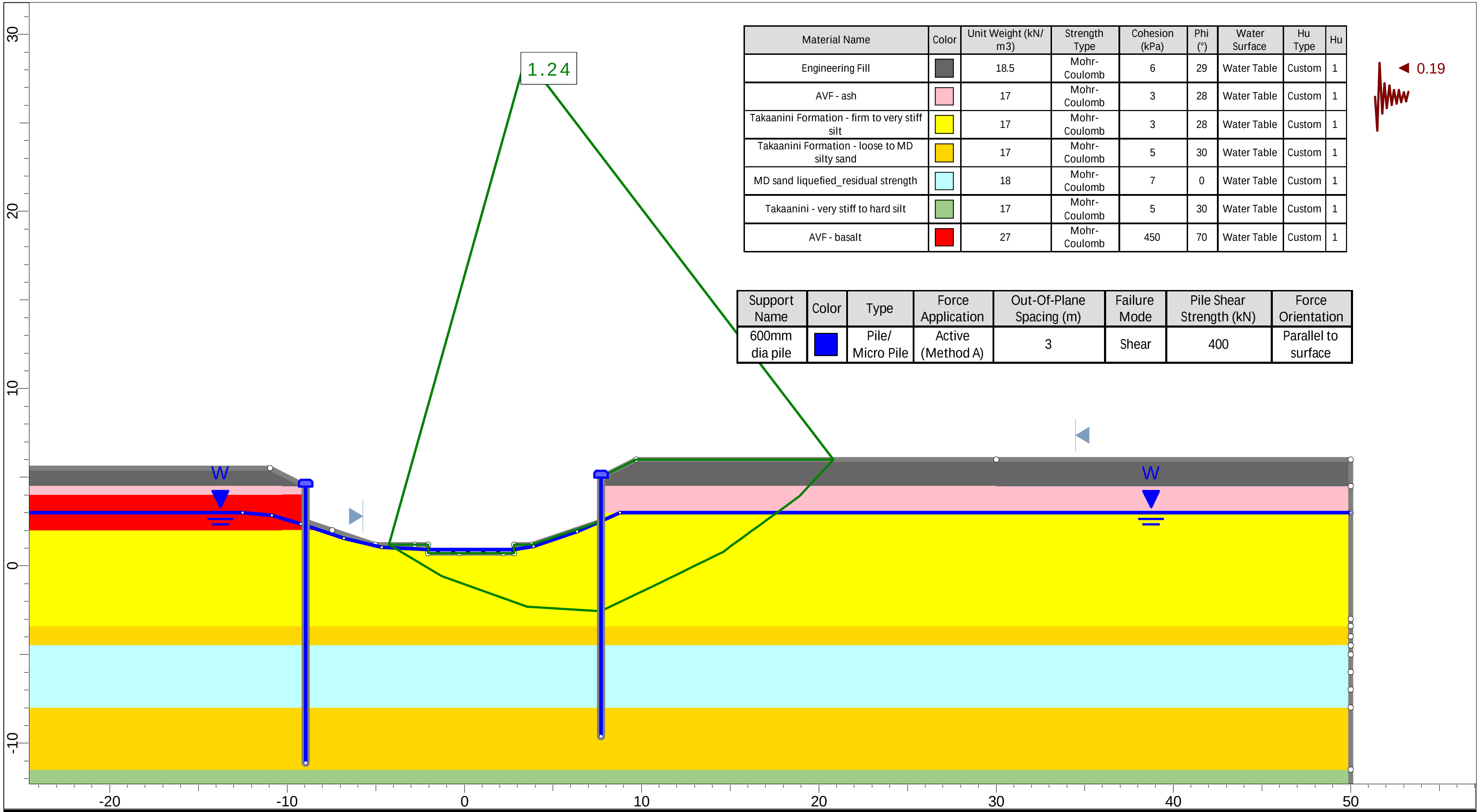
Material Name	Color	Unit Weight (kN/ m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type	Hu
Engineering Fill	■	18.5	Mohr-Coulomb	6	29	Water Table	Custom	1
AVF - ash	■	17	Mohr-Coulomb	3	28	Water Table	Custom	1
Takaanini Formation - firm to very stiff silt	🟨	17	Mohr-Coulomb	3	28	Water Table	Custom	1
Takaanini Formation - loose to MD silty sand	🟨	17	Mohr-Coulomb	5	30	Water Table	Custom	1
Takaanini - very stiff to hard silt	🟩	17	Mohr-Coulomb	5	30	Water Table	Custom	1
AVF - basalt	🟥	27	Mohr-Coulomb	450	70	Water Table	Custom	1

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Failure Mode	Pile Shear Strength (kN)	Force Orientation
600mm dia pile	🟦	Pile/ Micro Pile	Active (Method A)	3	Shear	400	Parallel to surface

rocscience

SLIDEINTERPRET 9.034

Project	Te Ararata - Walmsley Road Bridge Upgrade		
Group	River bank retaining structure	Scenario	Elevated GWL
Analysed By/Checked By	ZHLI/ABL	Company	Tonkin + Taylor
Date	11/10/2024	File Name	Abutment Stability 11Oct.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Hu Type	Hu
Engineering Fill	⬛	18.5	Mohr-Coulomb	6	29	Water Table	Custom	1
AVF - ash	🟪	17	Mohr-Coulomb	3	28	Water Table	Custom	1
Takaanini Formation - firm to very stiff silt	🟨	17	Mohr-Coulomb	3	28	Water Table	Custom	1
Takaanini Formation - loose to MD silty sand	🟧	17	Mohr-Coulomb	5	30	Water Table	Custom	1
MD sand liquefied_residual strength	🟦	18	Mohr-Coulomb	7	0	Water Table	Custom	1
Takaanini - very stiff to hard silt	🟩	17	Mohr-Coulomb	5	30	Water Table	Custom	1
AVF - basalt	🟥	27	Mohr-Coulomb	450	70	Water Table	Custom	1

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (m)	Failure Mode	Pile Shear Strength (kN)	Force Orientation
600mm dia pile	🟦	Pile/ Micro Pile	Active (Method A)	3	Shear	400	Parallel to surface

rocscience

SLIDEINTERPRET 9.034

Project	Te Ararata - Walmsley Road Bridge Upgrade		
Group	River bank retaining structure	Scenario	ULS seismic (1000-year event, liquefaciton)
Analysed By/Checked By	ZHLI/ABL	Company	Tonkin + Taylor
Date	11/10/2024	File Name	Abutment Stability 11Oct.slmd

Appendix C Groundwater Assessment

C1 General

This section sets out the assessment of the groundwater model, the results of which are presented in the main body. The groundwater measurements are presented and discussed, followed by an discussion of the western and eastern abutments. The groundwater model is then presented, followed by an assessment of radius of drawdown and the settlement potential.

This drawdown assessment has significant conservatism built in:

- Allows permanent drainage to +1.0 mRL (when we expect it to be around +1.5 mRL or higher) in front of the abutment piles.
- In terms of drawdown effects, considers no beneficial effects from the seasonal reductions in groundwater table (i.e. conservatively models 2 m drawdown for settlement and radius of effect assessment in both winter and summer conditions).
- Does not consider any previous drawdown effects (for settlement assessment).
- Assumes the maximum drawdown applies to the entire soil profile (for settlement assessment).
- Does not consider the flow of groundwater towards the stream (for radius of effects assessment).
- Does not consider mitigation from unloading through excavation (for settlement assessment).

We therefore consider the assessment to be appropriately cautious for the extent of the proposed groundwater interception. The assessment considers a reasonably robust envelope of possible effects and can be refined if required.

C2 Groundwater measurements

Groundwater levels (GWL) were measured around the site at the completion of CPT investigations and hand auger excavation. GWL measurements were also undertaken during and after machine borehole excavation (noting that these can potentially be influenced by the use of drilling fluid). A summary of GWL measurements is shown in Table Appendix C.1, along with the groundwater measurement data on historical investigation logs. This information has been used to determine the groundwater regime at the site.

In summary (refer Figure Appendix C.1), groundwater levels across the site range from 1.4 to 3.4 mbgl (+1.9 to 3.0 mRL) based on measurements taken across July and August 2024 and we consider these represent a typical winter groundwater level across the site.

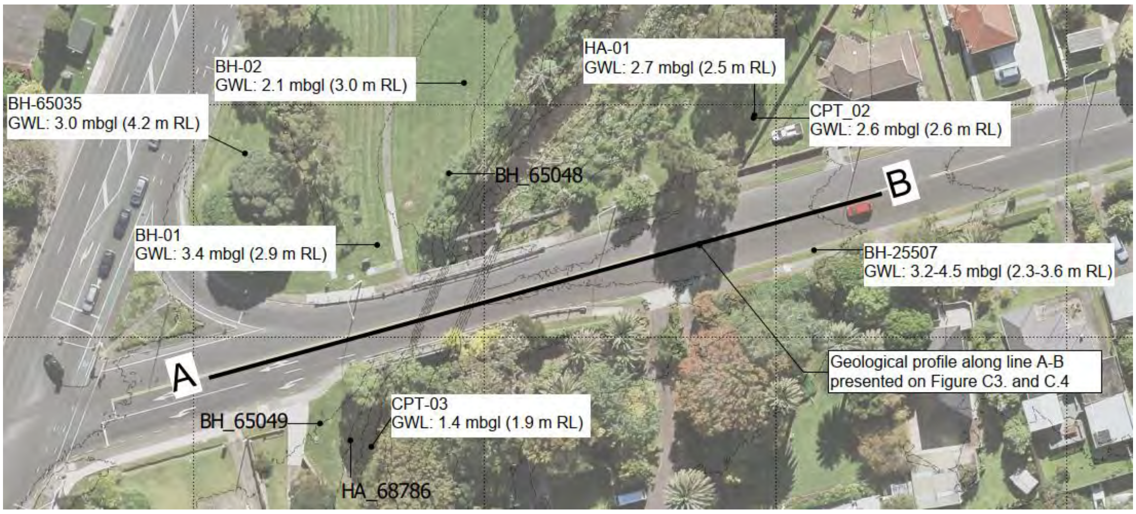


Figure Appendix C.1: Plan view of site with groundwater measurements at different investigation locations.

Water levels within Te Ararata Creek have been observed to be about 200 mm or less above the culvert invert level on 26 August 2024, as shown in Figure Appendix C.2. This is approximately +0.9 mRL and appears to be a low-flow scenario.

Figure Appendix C.2: Water level observed in Te Ararata on 26 August 2024 (approximately = 0.9 mRL).

The measurements at different investigation locations indicate that groundwater is generally dipping with ground surface and flows towards the Te Ararata creek. This is consistent with normal groundwater regimes around waterways and is discussed below.

C3 Western abutment

The geological profile at the western abutment comprises the edge of a basalt flow (Auckland Volcanic Field) with Takaanini Formation sediments (Figure Appendix C.3). Groundwater has been measured in BH-01 (Aug 2024) at a depth of 3.4 m (RL +2.9 m). The water level in this borehole was formed in fractured basalt and dipped the morning after drilling. We have a high degree of confidence in that particular measurement as we consider it will have reached equilibrium overnight and was surrounded by fractured basalt. We note that both BH-01 and BH-65035 measured groundwater in the basalt elevated a metre or so above the base.

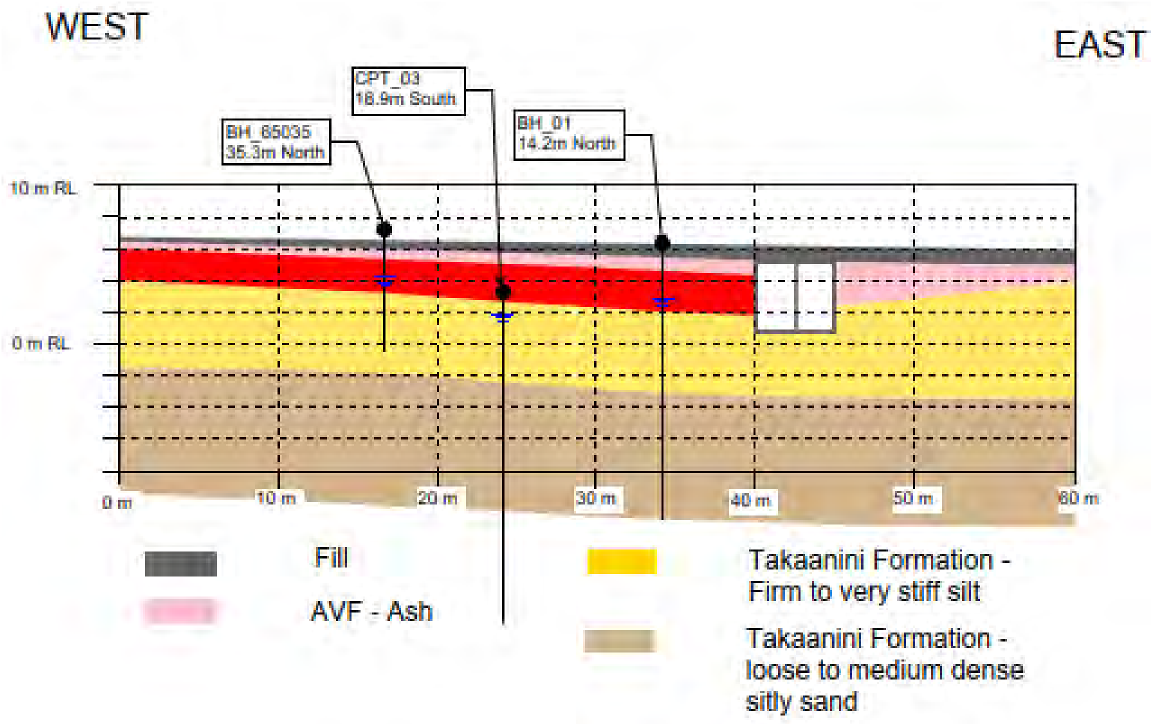


Figure Appendix C.3: Ground geological profile crossing the current culvert structure and GW level at investigation locations (West abutment side – Oblique to the Te Araata Creek).

That measurement is consistent with measurements in fractured basalt in BH_65035 (25 m northwest of the creek), at 3 mbgl (+4.2 mRL), measured in November 2013.

Finally, the depth of groundwater is consistent with observations and the dips in CPT-03. While those were directly on the stream banks, they show a groundwater profile dipping towards the creek.

Overall, this shows a profile where groundwater dips towards the Te Ararata creek. This is shown in Section C4. We note that the basalt flow is likely to be connected to other flows from Mangere Mountain. However, if this were a confined aquifer (i.e. more permeable than encapsulating ground), we would expect to see full saturation here as it is at the toe of the basalt

C4 Eastern abutment

The eastern abutment pre-construction groundwater profile is assessed as similar to the west. In July and August 2024 groundwater was measured at similar elevations at HA 01 (2.7 mbgl, RL +2.6 m) and CPT-02 (2.6 mbgl, RL +2.5 m), where investigation locations were accessible and away from existing underground utilities. These measurements were adjacent to each other (approximately 24 m southeast of Te Ararata Creek) but measured a month apart.

A similar reading of 3.2 to 4.5 mbgl (+2.3 to 3.6 mRL) was recorded at BH_215517 on 28 April and 3 May 2023, located approximately 47 m southeast of Te Ararata Creek. Please note that the geological section shown on Figure Appendix C.4 is along the existing Walmsley Road and oblique to the Te Ararata Creek (i.e. the setbacks from the stream are generally less than shown on the section).

On this basis, we assess groundwater to be similar to the western abutment, dipping towards the Te Ararata creek. This is shown in Section C4.

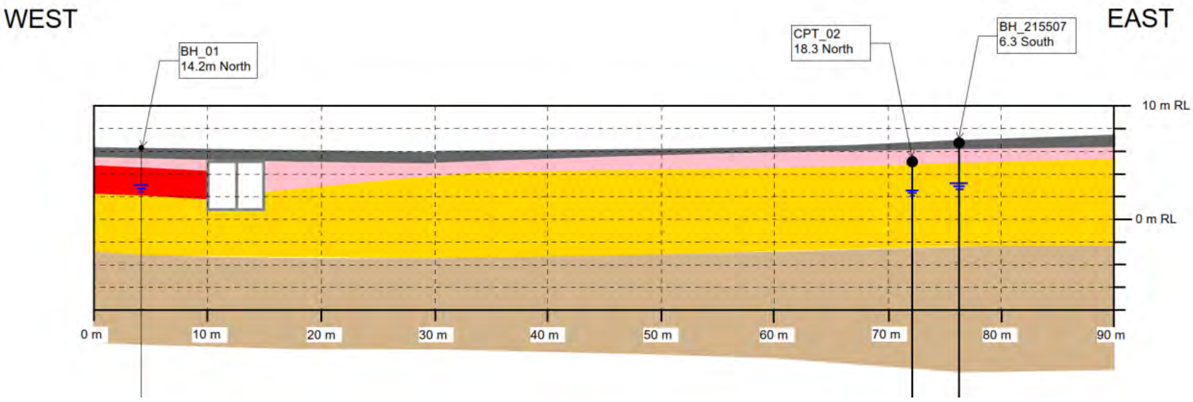


Figure Appendix C.4: Ground geological profile crossing the current culvert structure and GW level at investigation locations (Eastern abutment side – oblique to the Ter Ararata Creek).

C5 Groundwater model (winter)

For the purposes of this assessment, a simplified ground profile with groundwater level of + 3.0 mRL (3 mbgl) has been adopted on both sides of the abutments, gradually tapering down to +0.9 mRL at the creek, as shown on Figure Appendix C.5 and Figure Appendix C.6. We consider this to be a reasonably representative winter groundwater level at project site prior to the proposed development.

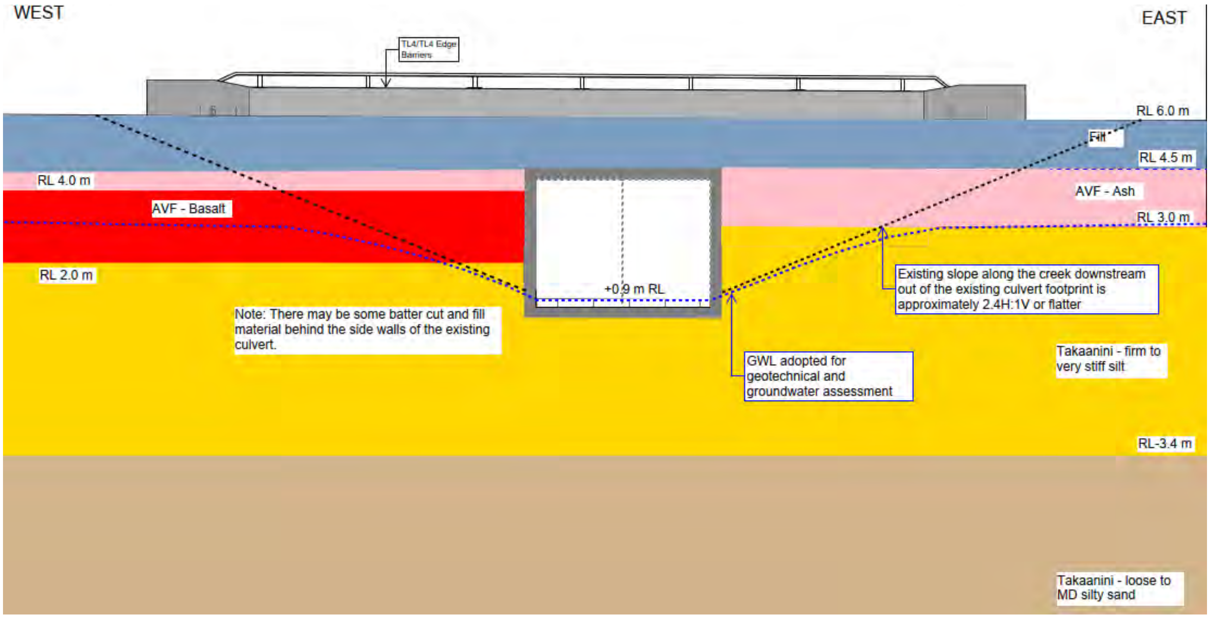


Figure Appendix C.5: Ground profile along bridge long section based on the available investigation information.

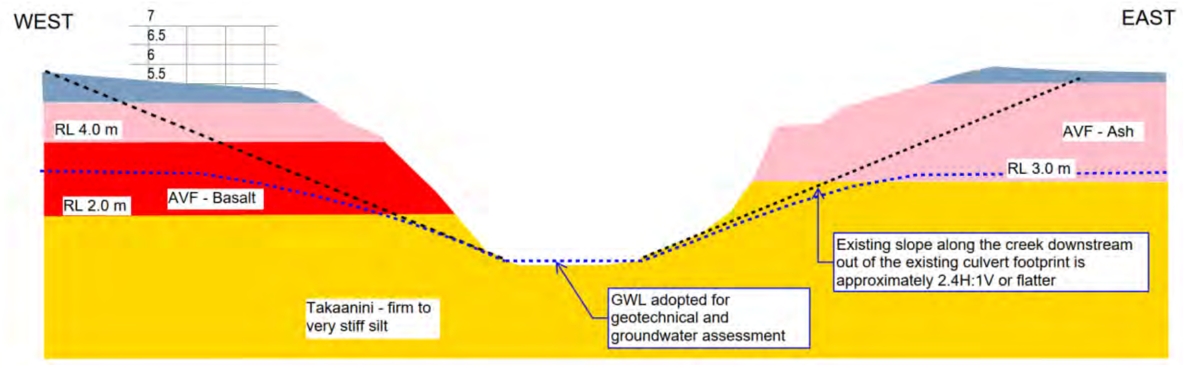


Figure Appendix C.6: Ground profile at retaining wall structure location and crossing Te Ararata Creek.

Table Appendix C.1 : Summary of recorded groundwater levels at the Site

Investigation ID	Location	Ground Level [mRL]	Groundwater level [mbgl]	Groundwater level [mRL]	Date of measurement	Comment on level of confidence in GWL measurement
Site-specific Investigation 2024						
CPT-02	Approximately 24 m southeast of the creek.	5.2	2.6	2.6	8 July 2024	High. No drilling fluid influence, soil behaviour type encountered at GWL is inferred to be silt mixture and the investigation penetration through an inferred sandy layer at depths of 4 to 5 m.
CPT-03	Approximately 2 m west of the creek.	3.3	1.4	1.9	8 July 2024	High. No drilling fluid influence, soil behaviour type encountered at GWL is inferred to be mixture of sand / silt mixture and penetration through an inferred sand layer at depths of 3 to 4 m.
HA-01	Adjacent to CPT-02	5.2	2.7	2.5	23 August 2024	High. No drilling fluid influence, silt material at measured GWL and similar measurement at CPT-02, Sand lens was encountered at depth of about 3.4 m.
BH-01	Approximately 10 m northwest of the creek	6.3	3.4	2.9	23 August 2024	High. Fractured basalt at measured GWL and less influenced by drilling fluid.
BH-02	Approximately 11 m northwest of the creek.	5.2	2.1	3.0	26 August 2024	Low. Probably impacted by drilling fluid (elevated).
Historical Investigations						
BH_65035	Approximately 36 m northwest of the creek.	7.2	3.0	4.2	7 Nov 2013	High. Fractured basalt at measured GWL and less influenced by drilling fluid, but well set back from the bridge (and a similar depth below ground level as BH-01 and BH-02).
BH_215507	Approximately 47 m southeast of the creek.	6.8	3.2 – 4.5	2.3 – 3.6	28 April to 3 May 2023	Medium to high. Multiple overnight GWL measurements from 28 April to 3 May, 2023. GWL ranged from 3.2 to 4.5 mbgl when drilling depths was 10.5 to 30.0 m. However, the last reading indicated GWL rise to 2 mbgl after overnight resuming when drilling depth was 31.5 m. We infer that the first few GWL readings reflected the shallow groundwater table, while the last reading was impacted by lower groundwater table.

C6 Groundwater drawdown extents

The estimated groundwater drawdown extent due to the proposed excavations has been estimated using available ground information and correlation with in-situ testing results. Based on the proposed development plan, the proposed excavation work is summarised below:

- Excavation plan view dimension: 50 (along creek flow direction) x 18 m (bridge span).
- Depth of excavation: 4.5 to 5.8 m below existing ground surface, including approximately 0.5 m undercut below riprap placement.
- Maximum dewatering during excavation: assessed to be 2.0 m below inferred winter groundwater table at bridge abutment and retaining structures on both upstream and downstream sides (see Figure Appendix C.7).

Permeability: See Table Appendix C.2 for details.

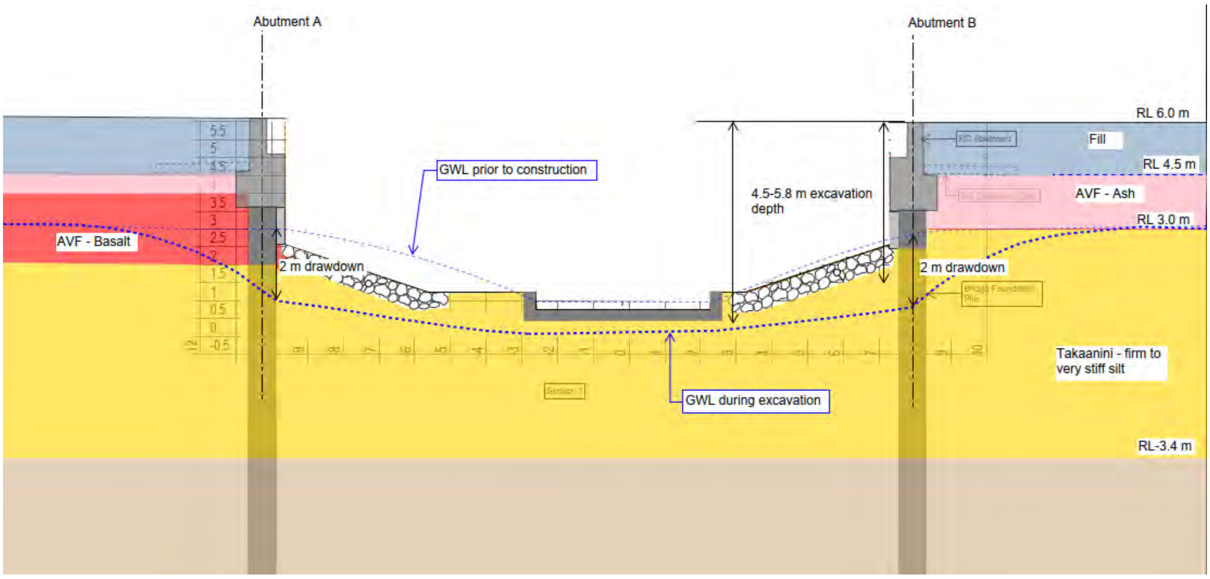


Figure Appendix C.7: Maximum 2 m drawdown condieration during construction in the winter.

Table Appendix C.2 : Permeability adopted in the groundwater effect assessment

Location	Aquifer layer	Permeability, m/s	Note
West side	AVF – basalt	1x10^{-4}	Groundwater was encountered about 1 m above the bottom of basalt layer where fractures exists. Both groundwater and the basalt bottom layer are dipping down towards the Te Ararata Creek. The permeability adopted represents a fractured, permeable mass. Given the elevated groundwater table observed in the basalt adjacent to the stream it appears likely to be less permeable than modelled (i.e. if this was more permeable and hydraulically connected to the stream, it would be likely to have drained the basalt flow already).
East side	Takaanini Formation – firm to stiff silt	2x10^{-6}	The correlation with CPT investigation indicates the permeability of Takaanini Formation within the potential drawdown depths, i.e. 6 m below ground surface is typically 1.0x10^{-9} to 1.0x10^{-7} m/s. It may

Location	Aquifer layer	Permeability, m/s	Note
			increase within the thin interbedded sandy lens, but the overall permeability is estimated based on the bulk performance (and is consistent with previous projects). We have modelled this as $2x10^{-6}$ m/s to test a more permeable situation than expected.

C7 Extent of drawdown

The distance of influence due to groundwater drawdown have been estimated using the method outlined in CIRIA C750 Section 6 for unconfined aquifer with partially penetration slots:

$$L_o = C\ (H - h_w) \times \sqrt{k}$$

Where:

L_o = distance of drawdown influence

C = empirical calibration factor between 1500 - 2000, adopting 2000 in this estimation.

$(H-h_w)$ = drawdown depth, maximum 2 m.

k = soil’s horizontal permeability.

Using the parameters discussed above, the distance of drawn influence has been estimated and presented in Table Appendix C.3.

Table Appendix C.3 : Estimated distance of drawdown influence

Location	Maximum GW level drawdown	Distance of drawdown influence, m
West side	1.0 (total 2 m drawdown but the lower Takaanini Formation unit has significant lower permeability compared, and therefore the drawdown in the lower part not considered in the estimation).	20
East side	2.0	6

C8 Drawdown induced settlement

Settlements induced by groundwater drawdown were estimated using a linear elastic 1D settlement approach:

$$s = \sum \frac{\Delta\sigma * H_i}{E'}$$

Where s = consolidation settlement due to groundwater drawdown

Δσ = effective vertical stress increasing due to groundwater drawdown. It is 19.8kPa (2 m x 9.8 kN/m^3) corresponding to maximum 2 m drawdown.

H_i = the thickness of each compressible soil layers

E' = constrained modulus of each compressible soil layers

The ground profile was divided into different geotechnical units presented in Section 2.1. It is anticipated that lowest geotechnical unit, Takaanini Formation – dense to very dense sand layer where CPTu get refusal forms a lower incompressible boundary. A constrained modulus based on the CPT data was assigned to each geotechnical unit. The values are consistent with other projects nearby and are presented in Table Appendix C.4

Table Appendix C.4 : Thickness and constrained modulus of compressible layer below groundwater table

Geotechnical units	Thickness, m		Constrained modulus, MPa
	West side	East side	
Takaanini Formation – Firm to very stiff silt	5.4	6.4	10.5
Takaanini Formation – loose to very dense sand	8.1	8.1	37.5
Takanini Formation – stiff to hard silt	5.5	5.5	33.5

The consolidation settlement has been calculated using equation above and the relevant inputs presented in Table Appendix C.4. The estimated settlement is summarised in Table Appendix C.5. We note this does not consider the effect of unloading (excavation) on the settlement, or previous drawdown events.

Table Appendix C.5 : Calculated consolidation settlement

Location	Maximum consolidation settlement, mm
West side	18
East side	20

The drawdown settlement contours are presented on Figure Appendix C.8.

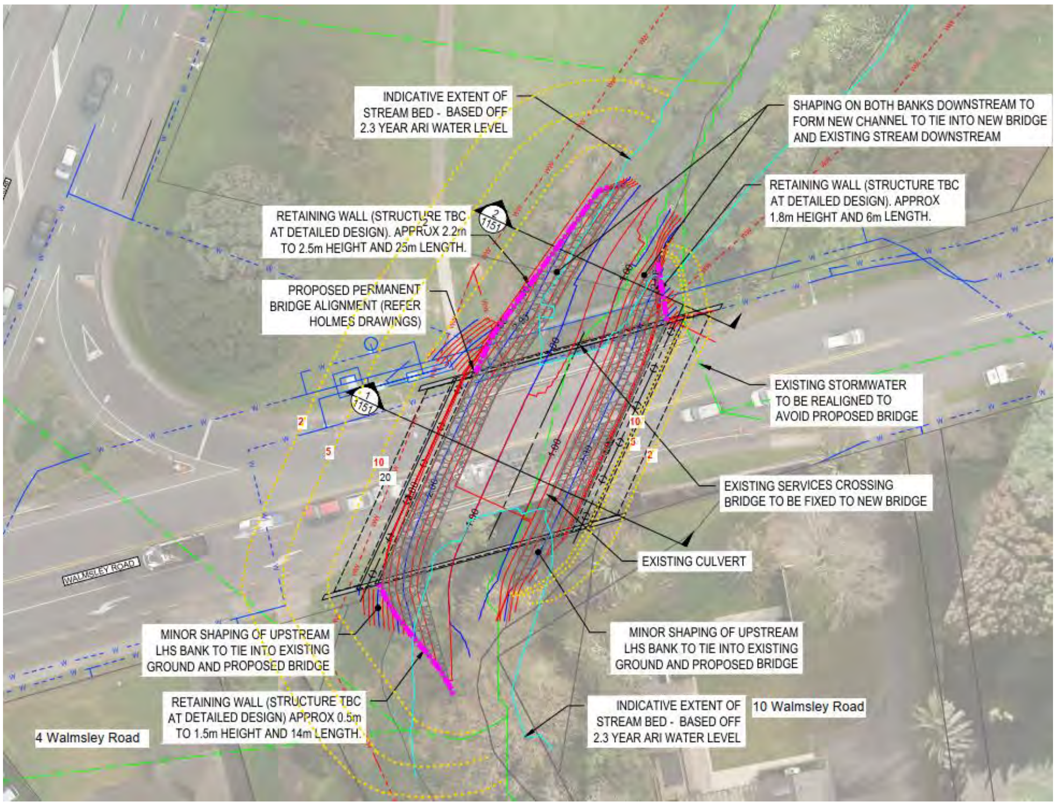


Figure Appendix C.8 Drawdown settlement contours behind bridge abutments and retaining structures.

Appendix D Mechanical Deformation Assessment

D1 General

This section presents the assessment of the mechanical deformation caused by the proposed development work, the results of which are presented in the main body.

The following construction work will induce mechanical deformation of the ground within the site includes:

- Excavation in front of the bridge abutment at each side prior to the bridge deck installation.
- Excavation in front of the retaining wall structures, including the upstream and downstream of the west abutment side and downstream of the east abutment side.

D2 Assessment methodology

The mechanical effects caused by the proposed development work were assessed using the empirical methods outlined in Section 6.2.1, CIRIA C760. The empirical methods were developed on the basis of case histories and setting up empirical relations among excavation depths, soil type, wall movements. The empirical relationship includes two parts:

1 The maximum vertical settlement is equal to some percentage of maximum excavation depth, depending on soil type and wall stiffness.

2 The distance of influence is equal to certain times maximum excavation depth.

We consider that the empirical method for 'stiff clays' discussed in CIRIA C760 is applicable to the project site. That recommends that in the retaining wall case with low support stiffness, i.e. cantilever retaining wall:

- Maximum vertical settlement at wall location is expected to be no more than 0.35% of maximum excavation depth.
- The distance behind wall with negligible settlement is about 3.5 times maximum excavation depth:

  $0 < d/H <= 0.8$, $\delta_v = \delta_{max}$;

  $0.8< d/H <3.5$, $\delta_v = \delta_{max}*(3.5 – d/H)/2.7$;

Where ***d*** is the distance from excavation, H is the maximum excavation depth, δ_v is the settlement at the location with distance ***d*** from excavation, $\boldsymbol{\delta_{max}}$ is the maximum settlement at wall location.

O' Rourke etc (1982) proposed a simplified method to assess the settlement effect on the existing buried pipeline that is commonly adopted by Council specialists for assessing services. The O' Rourke etc (1982) method was developed for assessing the performance of buried cast iron mains to soil displacement caused by tunnelling, which is considered to be in a normal distribution curve shape in the pipeline alignment direction. That method concludes that buried pipelines won't be damaged as long as the grade change (settlement over distance) does not exceed a maximum value of normalised settlement and curvature. The assessment relates pipe type and diameter, as shown below.

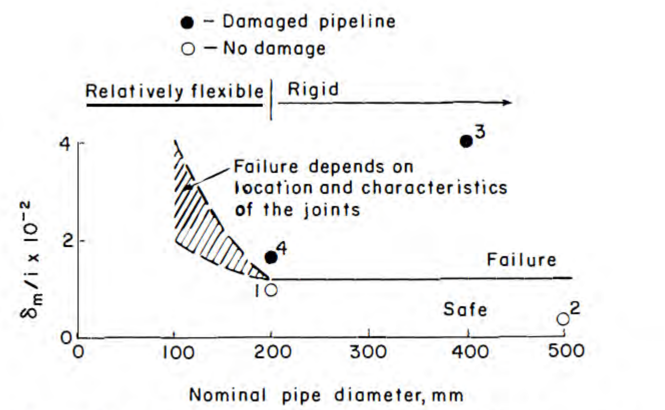


Fig. 7. Relationship between the dimensionless settlement associated with pipeline damage and pipe diameter

Figure Appendix D.1: A snapshot of the maxiumum acceptable grade chance in O'Rourke etc (1982) paper. The horizontal line is at 1.5%.

In Figure Appendix D.1 above, ***δ_m*** is the maximum settlement experienced by pipeline, and ***i*** is the distance from the centreline to the inflection point of the normal distribution curve.

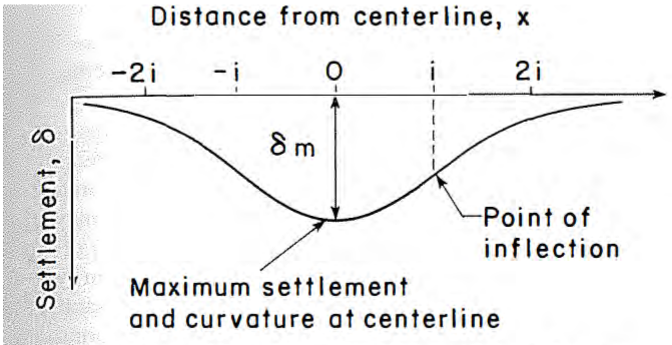


Fig. 2. Normal probability distribution of settlement

*Figure Appendix D.2: A snapshot of the definition of **δ_m** and **i** in O' Rourke etc (1982) paper.*

By simplifying the normal distribution curve which is originally with infinite distance in both directions to a limited width equal to 3****i*** as the settlement at the location with distance more than 3****i*** is negligible. Therefore, the parameter ***i*** is considered to be equal to **D**/3 where **D** is distance from no settlement to wall / retaining structure.

D3 Assessment results

The mechanical settlement caused by the proposed development work was assessed for both west and east sides. The results are presented in below table.

Table Appendix D.1 : Estimated maximum mechanical settlement

Location	Maximum Excavation depth, m	Maximum mechanical settlement immediately behind retaining wall, mm	Distance to assessed negligible mechanical settlement
West side			
Bridge abutment	5.8	20	20
Downstream retaining structure (northwest)	4.2	15	15
upstream retaining structure (southwest)	3.2	11	11
East side			
Bridge abutment	5.8	20	20
Downstream retaining wall structure (northeast)	3.5	12	12

Figure Appendix D.3 below present the contour of mechanical settlement behind bridge abutment and retaining structure at each side.

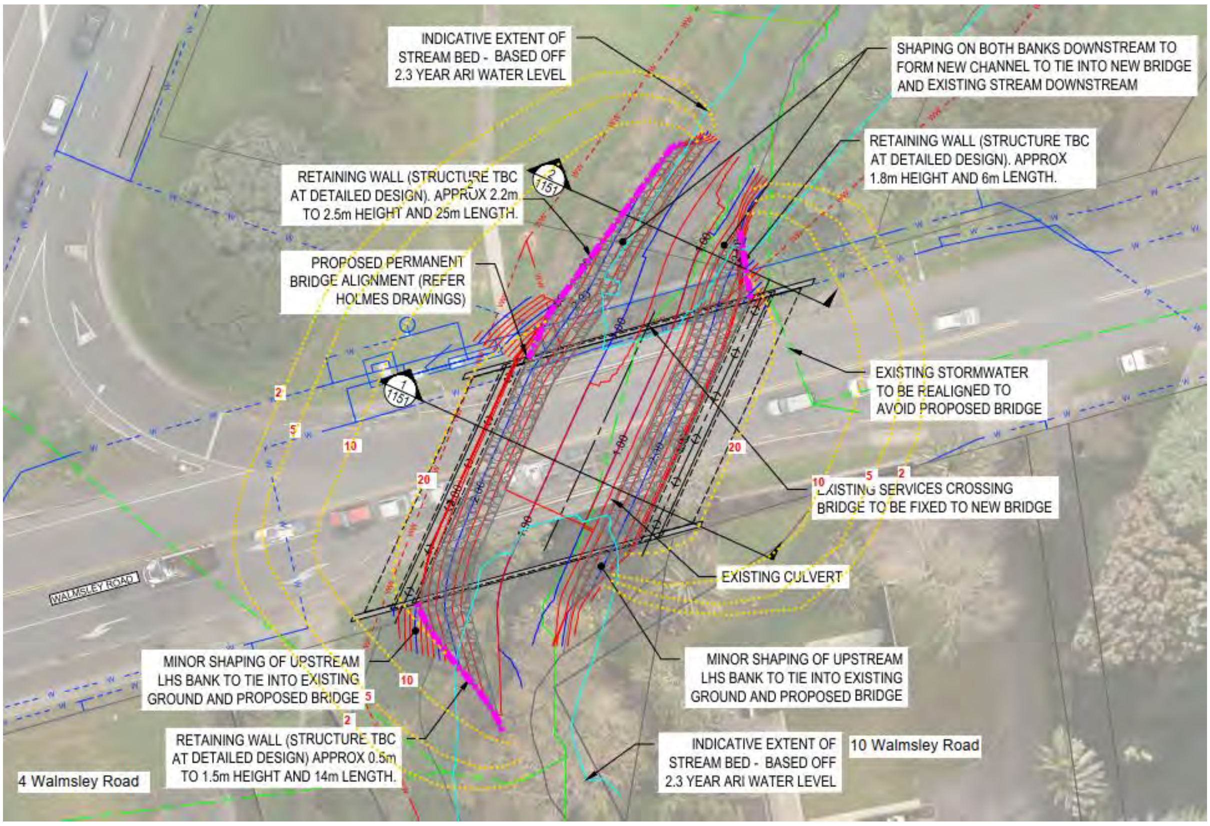


Figure Appendix D.3: Mechanical settlement contours behind bridge abutments and retaining structures.

To assess the impact of ground settlement caused by both mechanical deformation and groundwater drawdown, combined mechanical and consolidation settlement contours are prepared and presented in Figure Appendix D.4.

As demonstrated on Figure Appendix D.4, the nearest existing property buildings at 4 Walmsley Road (west side) and 10 Walmsley Road (east side) are anticipated to experience nil settlement caused by the proposed infrastructure work.

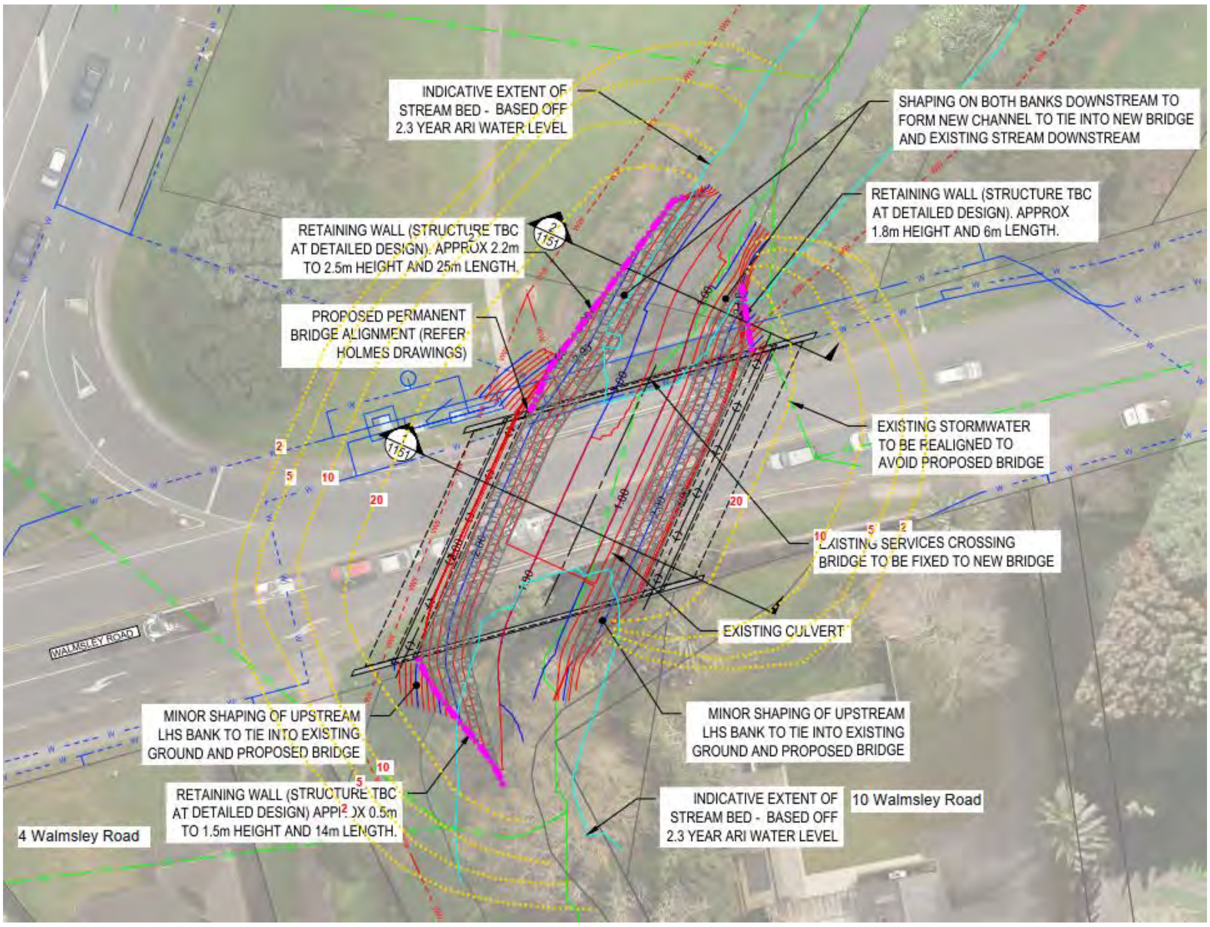


Figure Appendix D.4: Combined drawdown and mechanical settlement contours behind bridge abutments and retaining structures.

Table Appendix D.2 presents the maximum combined settlement and the estimated distance from the infection point to the centreline using the O' Rourke etc (1982) method. The maximum grade change is 0.78% which does not exceed the damage threshold of 1.5% shown on Figure Appendix D.2. The actual grade change along the individual pipeline is equal or less than the assessment presented in Table Appendix D.2 as the buried pipeline is generally in a diagonal direction to the combined settlement contours. It indicates the buried pipelines (except for the one directly disconnected due to the excavation work) will not be adversely impacted by the proposed development work.

Table Appendix D.2 : Estimated maximum mechanical settlement

Location Maximum Excavation depth, m	Maximum combined settlement immediately behind structure [(1)] (mm)	Distance to assessed negligible settlement m)	Distance from infection point to maximum settlement, i (m)	δm/I, %
West side				
Bridge abutment	38 (20+18)	20	6.7	0.56
Downstream retaining structure (northwest)	33 (15+18)	15	5	0.66
upstream retaining structure (southwest)	29 (11+18)	11	3.7	0.78
East side				
Bridge abutment	40 (20+20)	20	6.7	0.60
Downstream retaining wall structure (northeast)	12 (12+20)	12	4	0.3

(1) maximum combined settlement is the total settlement at structure location, i.e. at west abutment, the mechanical settlement and drawdown settlement is 20 and 18 mm respectively.

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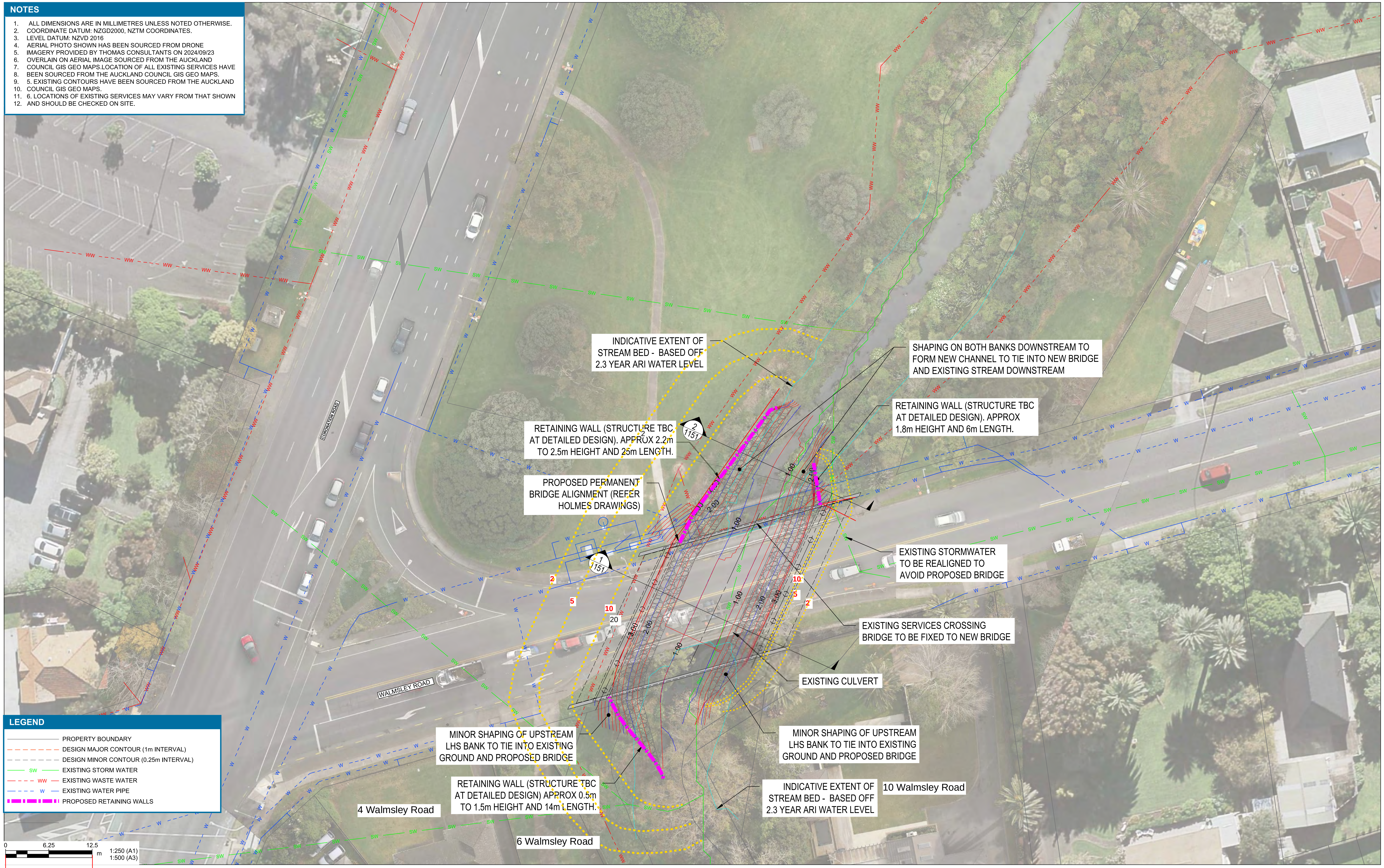

LEGEND

———	PROPERTY BOUNDARY
– – –	DESIGN MAJOR CONTOUR (1m INTERVAL)
– – –	DESIGN MINOR CONTOUR (0.25m INTERVAL)
— SW —	EXISTING STORM WATER
– – WW —	EXISTING WASTE WATER
– – W —	EXISTING WATER PIPE
▬ · ▬	PROPOSED RETAINING WALLS

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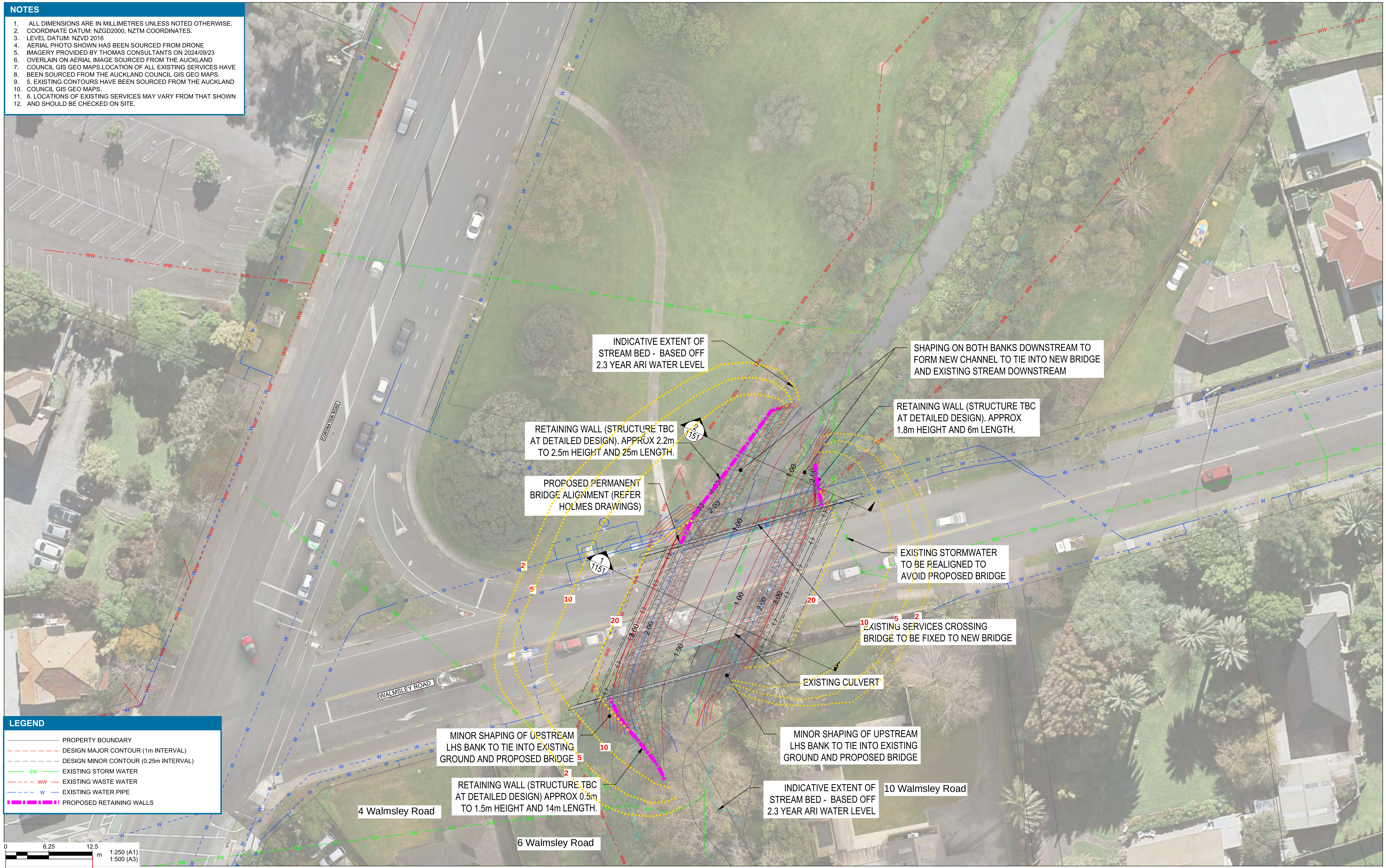

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– – WW — EXISTING WASTE WATER
– – W — EXISTING WATER PIPE
▬ ▪ ▬ PROPOSED RETAINING WALLS

0 6.25 12.5 m
1:250 (A1)
1:500 (A3)
12,568mm

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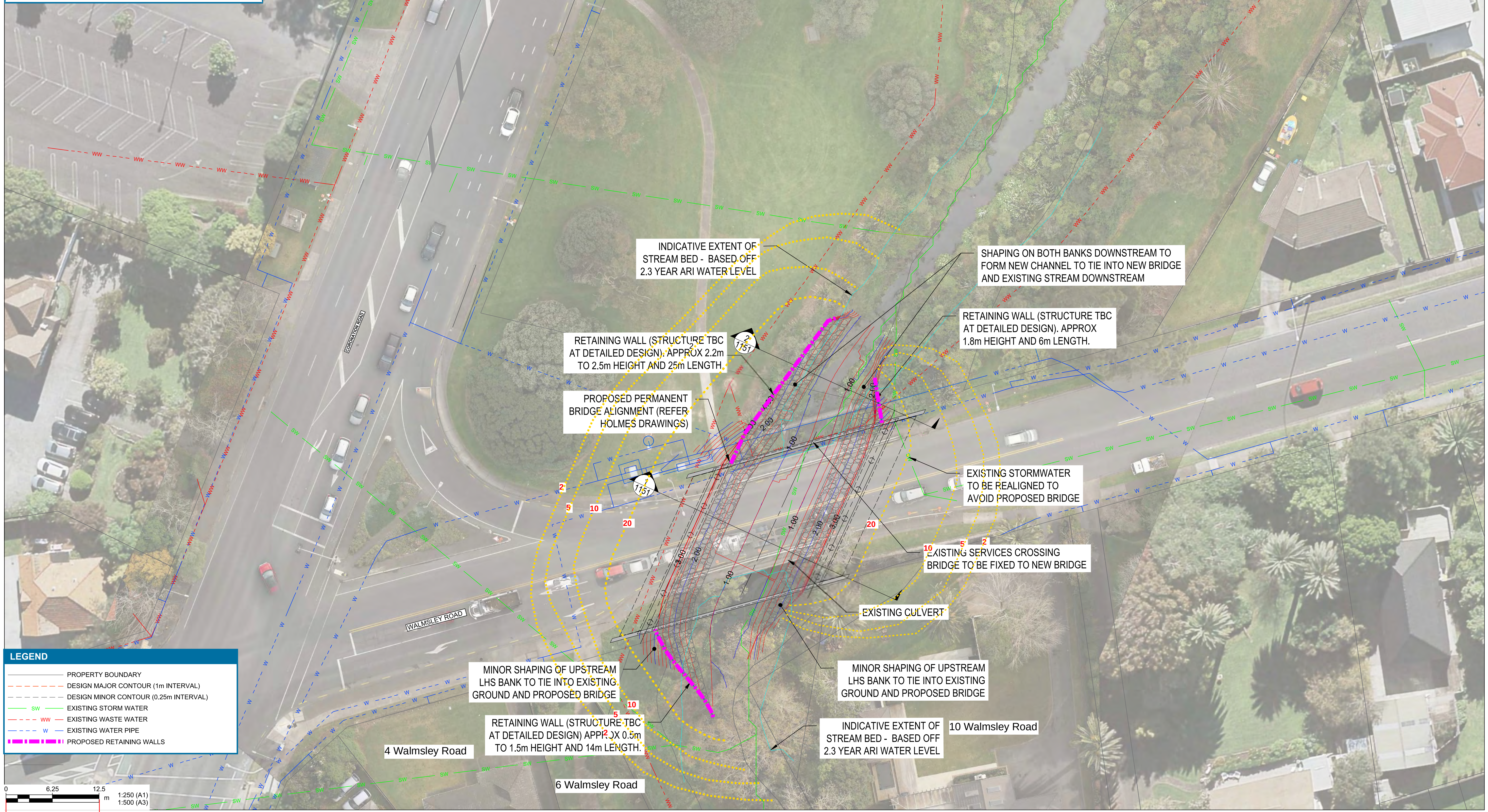

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TITLE COMBINED SETTLEMENT CONTOURS

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Appendix E Permitted Activity Standard (E.7.6.1) Assessment

Walmsley Road Bridge

Assessment of geotechnical aspects of proposed development with respect to the Auckland Unitary Plan Operative in Part

REGIONAL AND DISTRICT RULES - Chapter E: Auckland-wide rules - Natural resources

E7 Taking, using, damming and diversion of water and drilling - E7.4.1 Activity Table

Activity		Activity Status		
		All zones	High-Use Stream Management Areas Overlay	Wetland Management Areas Overlay
Take and use of groundwater				
(A17)	Dewatering or groundwater level control associated with a groundwater diversion permitted under the Unitary Plan	P	P	RD
(A20)	Dewatering or groundwater level control associated with a groundwater diversion authorised as a restricted discretionary activity under the Unitary Plan, not meeting permitted activity standards or is not otherwise listed.	RD	RD	RD
Diversion of groundwater				
(A27)	Diversion of groundwater caused by any excavation (including trench) or tunnel	P	P	RD
(A28)	The diversion of groundwater caused by any excavation (including trench) or tunnel that does not meet the permitted activity standards or is not otherwise listed.	RD	RD	RD

E7 Taking, using, damming and diversion of water and drilling - E7.6.1 PERMITTED ACTIVITY STANDARDS

E7.6.1 Diversion of groundwater - Rule (A27) Permitted Activity Standards	Geotechnical Interpretation of Compliance	Complies - Y/N
E.7.6.1.10. Diversion of groundwater caused by any excavation, (including trench) or tunnel (1) All of the following activities are exempt from the Standards E7.6.1.10(2) – (6): (a) pipes cables or tunnels including associated structures which are drilled or thrust and are less than 1.2m in external diameter; (b) pipes including associated structures up to 1.5m in external diameter where a closed faced or earth pressure balanced machine is used; (c) piles up to 1.5m in external diameter are exempt from these standards; (d) diversions for no longer than 10 days; or (e) diversions for network utilities and road network linear trenching activities that are progressively opened, closed and stabilised where the part of the trench that is open at any given time is no longer than 10 days.	IF THE PROPOSED WORK MEETS ANY OF A - E THEN IT IS PERMITTED UNDER THIS RULE. IF THESE DON'T APPLY THEN SIMPLY NOTE N/A AND MOVE TO (2) TO (6). (a) N/A - no works of this nature proposed (b) N/A - no work of this nature proposed (c) Foundation pile diameter is less than 1.0m (d) Excavation proposed will involve diversions for longer than 10 days (e) N/A - No works of this nature proposed	Y Y Y N Y
(2) Any excavation that extends below natural groundwater level, must not exceed: (a) 1ha in total area: and (b) 6m depth below the natural ground level	The excavation below groundwater level is approximately maximum 1500 m2 and max 6 m below natural ground level. Therefore, does not exceed (a) and (b)	Y
(3) The natural groundwater level must not be reduced by more than 2m on the boundary of any adjoining site.	The groundwater level will not be reduced by more than 2m on the boundary of the closest adjoining site	Y

E7.6.1 Diversion of groundwater - Rule (A27) Permitted Activity Standards	Geotechnical Interpretation of Compliance	Complies - Y/N
(4) Any structure, excluding sheet piling that remains in place for no more than 30 days, that physically impedes the flow of groundwater through the site must not: (a) impede the flow of groundwater over a length of more than 20m; and (b) extend more than 2m below the natural groundwater level.	The proposed structures (e.g., retaining structures and bridge abutments) will be over	Y
(5) The distance to any existing building or structure (excluding timber fences and small structures on the boundary) on an adjoining site from the edge of any: (a) trench or open excavation that extends below natural groundwater level must be at least equal to the depth of the excavation; (b) tunnel or pipe with an external diameter of 0.2 - 1.5m that extends below natural groundwater level must be 2m or greater; or (c) a tunnel or pipe with an external diameter of up to 0.2m that extends below natural groundwater level has no separation requirement.	(a) The maximum depth of excavation is 6m. The closest building or structure from the excavation is more than 6m (approximately 12.5m from property building on eastern abutment side, and 7.2m from the driveway on western abutment side). (b) N/A - No works of this nature proposed (c) N/A - No works of this nature proposed	Y
(6) The distance from the edge of any excavation that extends below natural groundwater level, must not be less than: (a) 50m from the Wetland Management Areas Overlay; (b) 10m from a scheduled Historic Heritage Overlay; or (c) 10m from a lawful groundwater take	The distance from edge of the excavation is greater than those required under (a) - (c)	Y

E7 Taking, using, damming and diversion of water and drilling - E7.6.1 PERMITTED ACTIVITY STANDARDS

E7.6.1 Dewatering or groundwater level control - Rule (A17) Permitted Activity Standards	Geotechnical Interpretation of Compliance	Complies - Y/N
E7.6.1. 6. Dewatering or groundwater level control associated with a groundwater diversion permitted under Standard E7.6.1.10, all of the following must be met: (1) the water take must not be geothermal water	(1) The groundwater dewatering associated with excavation does not involve geothermal water.	Y
(2) the water take must not be for a period of more than 10 days where it occurs in peat soils, or 30 days in other types of soil or rock	(2) Groundwater dewatering associated with excavation will be more than 30 days (approximately. 3- 4 months construction)	N
(3) the water take must only occur during construction.	(3) Groundwater dewatering is only during construction.	Y

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