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## Geotechnical Feasibility Investigation at 39 Launch Road, Hobsonville Point

Rev A

26 November 2021

Job No. 211053



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**GEOTECHNICAL INVESTIGATION  
FOR FEASIBILITY OF FUTURE DEVELOPMENT  
39 LAUNCH ROAD, HOBSONVILLE**

<b>Job Number:</b>	211053
<b>Name of Project:</b>	39 Launch Road, Hobsonville Point
<b>Client:</b>	Kainga Ora
<b>Author:</b>	Martin Williams, Geotechnical Engineer, MEngNZ
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<b>Document Version:</b>	A
<b>Printed:</b>	26 November 2021
<b>Author Signature:</b>	
<b>Reviewer / Authoriser Signature:</b>	

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Appendix B: Investigation Logs (Augerholes, Scala Penetrometer Results)

Appendix C: Slope Stability Results

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## Report Summary

The following summarises the findings of this report however is not to be taken in isolation. It is a requirement that any user of this report review the document in its entirety, including all appendices.

Feature	Commentary
RMA: Section106	No <i>geotechnical</i> natural hazards were identified (as listed in this Act) that are considered an undue impediment to future development or that cannot be reasonably addressed by typical engineering design and construction
Fill	Encountered to a maximum depth of 0.8m bpgl
Natural Soils	Soft to hard alluvial deposits and weathered Waitemata Group soils. Holocene 'gully alluvium' may be present in the gully base.
Unduly Weak, Sensitive, or Compressible Soils	Isolated zones of weaker soils were encountered, however the ground conditions encountered were typically of favourable strength. The near surface soils are considered to be moderately sensitive to disturbance
Groundwater	Encountered between 2.0m and 2.5m bpgl within elevated ground. A near-permanent spring is present approximately mid-site and shallow groundwater is considered likely in the gully area.
Seismic Site Class	Site Class C
Expansive Soils	Expected to be Moderately Expansive to Highly Expansive, in accordance with B1/AS1. Adopt Class H for preliminary design.
Slope Stability	In-situ soil is considered globally stable. Structures near areas of moderate to steeper slopes will require specific assessment.
Foundations	Conventional shallow and piled foundations without undue design requirements are likely suitable. The required foundation system will depend on the future development proposal
Construction Constraints	Sandy soils are present, where saturated these soils could be susceptible to localised collapse within foundation excavations and bored pile holes. The gully requires permanent drainage and localised gully base alluvium ('mullock') may have to be discarded.
Additional Investigation	Additional geotechnical investigation is required to support future design and consenting of specific development proposals. The scale of investigation required will depend on the scale and nature of the development

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## **1.0 Introduction**

Soil & Rock Consultants (S&RC) were engaged by Kainga Ora to carry out a geotechnical investigation at 39 Launch Road, Hobsonville Point with respect to geotechnical feasibility for future development.

Our investigation has been informed by Section 106 of the Resource Management Act which lists 'Natural Hazards' that must be considered by Council when assessing a Resource Consent application, however this report alone is not suitable for support of a consent application and additional proposal specific assessment is required. Our assessment has also extended to consideration of the following:

- Provision of a seismic site class in accordance with NZS1170.5:2004
- Provision of preliminary geotechnical recommendations related to foundation and retaining design requirements

The primary purpose of this reporting is to identify the issues discussed above and provide comment on the feasibility of the site for future development, from a geotechnical perspective.

### **1.1 Limitations**

This report has been prepared by Soil & Rock Consultants for the sole benefit of Kainga Ora (the client) with respect to 39 Launch Road, Hobsonville Point and the brief given to us. We understand that Kainga Ora will forward this report to prospective purchasers to assist with decision making however Soil & Rock Consultants has no relationship with, and no liability to, those parties. The data and/or opinions contained in this report may not be used in other contexts, for any other purpose without our prior review and agreement. This report may only be read or transmitted in its entirety, including the appendices.

The recommendations given in this report are based on data obtained from discrete locations and soil conditions between locations are inferred only. Our geotechnical models are based on those actual and inferred conditions however variations between test locations may occur and Soil & Rock Consultants should be contacted in this event.

## **2.0 Site Description**

The subject site, legally described as Lot 8 DP 523085, is irregular in shape, covers an area of 2.1065ha (see Figure 1 - Aerial Image 2017) and is located at the corner of Launch Road and Bomb Point Drive with street frontages of approximately 115m and 250m respectively.

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The property is void of permanent built development; however, the site formerly contained the house of the Base Commander of the Hobsonville Air Base, which has been demolished leaving only parts of the asphalt driveway remaining.

The northern part of the site is currently occupied by a gravel-surfaced carpark and boat storage area.

Ground surfaces are typically gently sloping with a steeper slope of approximately 20° present at the northern edge of the site, sloping down toward Launch Rd. A gully crosses the central part of the site, with ground slopes of up to 18° immediately east of the site boundary. An overland flow path is present within the gully area and at the upper end has an ephemeral spring – water flows at most times of the year but appears to cease or markedly decrease to become seepage-only at the height of summer.

The southern part of the site is near level and has recently been used as a fill stockpile, however at the time of our investigation the majority of the fill had been removed with remaining fill primarily limited to a perimeter bund around the stockpile area.

An area of bush is present along the northern part of the site, south of the current gravel carparking. The northern slope (down to Launch Rd) is vegetated by landscaped planting and the majority of the rest of the site is grassed.

The Council GeoMaps system shows underground services within the site include a herringbone array of field-tile stormwater pipes (possibly private assets) within the northern part of the site, and a series of stormwater pipes and manholes within/adjacent to the central gully.

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Figure 1: Aerial Image - 2017 (Source: Auckland Council GeoMaps Website)

## 2.1 Proposed Development

It is understood the site is most likely to be used for future residential development comprising a mixture of apartment buildings and terraced housing.

## 3.0 Geology

Reference to the GNS New Zealand Geological Web Map 1:250,000 Geology map, indicates the site is underlain by Puketoka Formation soil and soils and rocks of the Waitemata Group (See Figure 2).

Both geological units were encountered during our field investigation.

It is also to be expected that Holocene (current geological age) alluvium will be found where watercourses are present.

### Holocene Alluvium

This material is usually wet-to-saturated, weak and muddy and is typically localised, usually along stream channels or present along deeper overland flow paths. This material requires removal if the area it occupies is to be filled and is not suited to support of any kind of structure.

### Puketoka Formation

Alluvial soils are often susceptible to consolidation (resulting in settlement) when subjected to foundation or fill loads, particularly where organic soils (e.g. Peats) are present. In addition, these soils shrink and swell with soil moisture content changes and can be sensitive, often rapidly losing strength in response to disturbance by construction plant and/or exposure to the elements.

### Waitemata Group

Waitemata Group soils are derived from weathering of the parent sedimentary sandstones and siltstones to form a mantle of residual soils typically comprising firm to very stiff clays, silts, and sands of variable plasticity. These soils are prone to shrinking and swelling with variations in soil moisture content.



**Figure 2: Geological Map** (Source: GNS WebMaps Website)



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## **4.0 Field Investigation**

The field investigation carried out on 3 November 2021 comprised the following components:

- Visual appraisal of the site
- Drilling of six hand augerholes (AH01 – AH06 inclusive) – Appendix B
- Measurement of two cross sections (A-A' and B-B') using a measuring tape and clinometer – Appendix A

The test locations are shown on the Site Plan, Drawing No 211053/1 (Appendix A). The locations were measured from existing site features using a hand-held tape or determined from hand-held GPS and are therefore approximate only.

Measurements of undrained shear strength were undertaken in the augerholes at intervals of depth using a handheld shear vane in accordance with the New Zealand Geotechnical Society Guidelines for Handheld Shear Vane Tests, dated August 2001. Peak and remoulded vane shear strengths shown on the attached augerhole logs represent dial readings off the shear vane adjusted using the BS 1377 calibration correction factor.

A visual-tactile field classification of the soils encountered during drilling was carried out in accordance with “Guidelines for the Field Classification and Description of Soil and Rock for Engineering Purposes”, issued by the New Zealand Geotechnical Society Inc. (2005).

Dynamic Cone (Scala) Penetrometer testing was carried out in-lieu of shear vane testing where soils became sand-dominated and from the base of the augerholes until refusal was reached. Refusal is defined as five consecutive blow counts of 10 or greater per 50mm penetration. The results are given on the attached sheet (Appendix B).

### **4.1 Subsurface Conditions**

Subsurface conditions have been interpolated between the test locations and localised variations between and away from the test locations will exist.

In general, the soils encountered comprised topsoil/fill underlain by Alluvial deposits and/or weathered Waitemata Group soils. An outline of the soil conditions and investigation results is given below and summarised in Table 1, and detailed descriptions of the soils are given on the attached logs (Appendix B).

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- 
- **Topsoil/Fill.** Topsoil and/or non-engineered fill were encountered at each test location, typically to depth of up to 0.3m, with a maximum depth of 0.8m below present ground level (bpgl) at the location of augerhole AH02. These materials are unsuitable for the support of permanent structures (i.e. building foundations, floor slabs, pavements etc.).

Cross Section BB' on Drawing 211053/2 is representative of fill stockpiles at the southern end of the site. These are easily visible as they are stockpiles however fill as described in the handauger holes is not obvious and requires investigation to quantify volumes and spread.

The depth, lateral extent, and composition of the fill material will vary across the site.

- **Holocene Alluvium.** This material was not encountered within any investigation augerhole however will be present at the spring location and along the associated watercourse and gully sides. This deposit is likely to be up to 1.0m thick centred on the watercourse however without specific investigation there is no guarantee in this regard.
- **Tauranga Group.** Puketoka Formation alluvial deposits or Holocene alluvial deposits were encountered at the location of augerholes AH02 to AH06 (inclusive) underlying the topsoil/fill to depths ranging between 1.7m and 2.3m bpgl, or to the base of the augerholes at 5.0m bpgl. The alluvial soils comprised soft to hard silty clay, clayey silt, silt, sandy silt or loose silty sand. Vane shear strengths ranged from 20kPa to greater than 200kPa where the soil strength was in excess of the shear vane dial capacity.

No deposits of unduly weak or organic material were found within our handauger holes.

- **Waitemata Group.** Weathered Waitemata Group soils were encountered underlying the topsoil/fill and Tauranga Group deposits (where encountered) to the termination depths of augerholes AH01, AH03 and AH04. The Waitemata Group soils comprised firm to hard clayey silt or sandy silt or loose to medium dense silty sand. Vane shear strengths ranged from 50kPa to greater than 200kPa where the soil strength was in excess of the shear vane dial capacity.
  - **Scala Penetrometer Testing.** Scala Penetrometer testing was carried out from the base of each augerhole. Refusal, inferred to be contact with the transition zone into an underlying dense stratum, was encountered at depths ranging between 5.1m and 6.3m bpgl. The upper surface of this dense stratum (inferred sandstone) is most likely several metres below the refusal depth encountered above.
  - **Groundwater.** Groundwater measurements were carried out within the hand augerholes on the day of drilling (3 November 2021) and are summarised in Table 1.
-

Groundwater measurements taken during drilling are not always an accurate portrayal of the actual long-term groundwater table as groundwater levels can take time to stabilise within the augerhole following drilling. Groundwater will be near-surface level in the vicinity of the spring and at shallow depth within the central gully area.

**Table 1 – Summary of Subsurface Conditions**

Test ID	Termination Depth	Depth to the base of Topsoil/Fill	Vane Shear Strength Range (kPa)	Scala Penetrometer Termination	Groundwater Depth
All depths measured in (m) below present ground level. (Rounded to 1 DP)					
AH01	4.5	0.2	119-192	5.1	2.2
AH02	5.0	0.8	20-128	6.3	2.5
AH03	5.0	0.3	45-200+	6.1	2.3
AH04	5.0	0.3	50-200+	6.2	2.1
AH05	5.0	0.2	55-181	6.0	2.2
AH06	5.0	0.3	99-200+	5.9	2.0

## 5.0 Expansive Soils

The soils present (Holocene alluvial deposits, Puketoka Formation deposits and Waitemata Group deposits) can range between Expansive Soil Class S - Slightly Expansive and Expansive Soil Class E - Extremely Expansive, however Hobsonville Point natural soils usually lie within the range of Expansive Soil Class M - Moderately Expansive and Expansive Class H - Highly Expansive. Class M to Class H soils experience surface movements of 44 to 78mm.

We recommend the expansive soil class be determined, in accordance with B1/AS1, as part of future proposal specific geotechnical investigation. Adoption of Site Class H would be the conservative course.

## 6.0 Sensitive Soils

The ratio of peak to remoulded vane shear strength values recorded during our investigation ranges approximately between 2 and 4, indicative of a 'moderately sensitive' subgrade. These soils are potentially susceptible to mechanical disturbance and/or exposure to the elements and soils that test well in-situ can perform poorly when construction is underway. Care is therefore required during construction to ensure the soils are protected to ensure favourable short and long-term subgrade and foundation performance.

Practical means of protecting the soils include avoidance of vibration-based compaction equipment, protecting the subgrade following initial site clearance, minimising the passage of heavy or vibrating construction plant, and extra care during foundation excavations, particularly any pile excavations.

## **7.0 Seismic Design Parameters**

The site is considered a Class C – ‘Shallow Soil Site’ as defined by NZS 1170.5:2004.

The Peak Ground Acceleration (PGA) value for a structure of Importance Level 2, adopted for stability analysis of the site is 0.153g (ULS) with an effective earthquake magnitude of 5.9. Depending on the nature of the final development, a different Importance Level, and hence a different PGA, may apply.

## **8.0 Slope Stability**

### Qualitative Assessment

The site comprises two large near-level to gently sloping areas with a gully between. Adjacent to the northern site boundary a short steeper slope of 18° to 20° is present, sloping down toward Launch Rd. Immediately south of the current carparking area is a declivity (as indicated on the Council GIS System), approximately 2m lower than the surrounding ground, that formerly comprised the Base Commanders Pond, this area is currently moderately to heavily vegetated. The pond bank slopes are inferred to be in the order of 20°. Construction within the pond area or adjacent to the pond banks may require proposal specific assessment of slope stability.

A gully is present within the central part of the site. The gully head is broad with short moderate slopes, typically up to 15° inclination on either flank and the gully steepens as it extends out of the site boundary, with inclinations of up to 18° recorded to the immediate north-east of the former fill stockpile.

The southern part of the site is typically gently sloping.

There were no visual signs of deep-seated instability during our site walkover, and no signs of any stability-related constraints that could not be addressed through suitable design and assessment. Future proposal specific assessment is recommended, in particular for any future construction proposed immediately adjacent to or on the gully slopes or Base Commanders Pond banks, or on or in close proximity to the northern site slope.

Other points of note relating to stability are as follows:



- Rain-shedding characteristics of the site are variable. It is expected the former Base Commanders Pond will be saturated through winter and following heavy rain events, at the time of our field investigation, water appeared to be seeping out of the bank south of the pond, flowing toward the gully area.

The current carparking area and fill stockpile area are unlikely to have favourable rain shedding characteristics. Parts of the gully area are likely to have favourable rain shedding characteristics due to the sloping ground present.

- The potential for groundwater recharge is moderate to high, particularly in the areas with unfavourable rain shedding characteristics.
- A spring-fed watercourse (likely associated with the nearby 'Base Commanders Pond') is present within the gully area, and an area of saturated surface seepage was encountered, flowing toward the watercourse from the northern bank of the gully.
- No obvious visual signs of previous slips or scarps were identified.
- Vegetation cover varies from moderate to dense bush (typically on the steeper parts of the site) to grass cover on the gently sloping parts of site. Vegetation cover is typically beneficial with regards to slope stability.
- No retaining was observed within the site area.

At the time of our investigation no visual evidence of major, deep-seated instability was identified.

Soil Creep is likely to be operating on any of the slopes inclined at 1V:4H (~14°) or steeper. Soil creep is the slow downslope movement of upper soil horizons, usually confined to the uppermost 1.0m to 1.5m of soil and generally in the order of millimetres per year. Soil creep is exacerbated by slope length, slope angle, inundation, groundwater fluctuations, soil expansivity, vegetation, and various surcharge loads.

### Quantitative Assessment

To quantitatively check the stability of the site, stability analyses have been undertaken for the existing topography through cross sections A-A' and B-B' as indicated on the Site Plan, Drawing No. 211053/1.

The computer program SLIDE Version 2 for slope stability analysis, developed by RocScience Inc. was used for stability calculations. Stability of theoretical circular surfaces was assessed using the GLE / Morgenstern-Price method.

Stability analyses have been undertaken for the measured groundwater, extreme (worst credible) groundwater, and seismic conditions. The measured groundwater condition has been adopted for the

seismic condition. Peak Ground Acceleration (PGA) values for the Auckland Region have been determined as per Section 7.0 of this report.

Lower-bound effective stress shear strength parameters used for our analyses are summarised in Table 2. These have been developed from the soil description, in-situ strength testing, limited back analysis, and our experience with these soil types in both the immediate area and the wider region.

**Table 2 – Effective Shear Stress Parameters**

Soil Type	Estimated Unit Weight $\gamma$ (kN/m <sup>3</sup> )	Effective Cohesion on the Failure Plane $c'$ (kPa)	Effective Angle of Internal Friction $\phi'$ (°)
Non-Engineered Fill	18	2	26
Puketoka Formation Soils	18	4	28
Weathered Waitemata Group Soils	18	5	30
Dense Waitemata Group Soils	19	10	36

The ratio of resisting forces to disturbing forces is presented as a 'Factor of Safety' (FOS) against slope instability occurring. A FOS of 1 indicates a slope near or at equilibrium.

Table 2.C.1 of the "Auckland Council Code of Practice for Land Development and Subdivision, Section 2, Earthworks and Geotechnical Requirements", Version 1.6, dated 24 September 2013 lists the minimum Factor of Safety acceptable to Auckland Council. These are provided in the 'Required' column in Table 3 alongside the calculated FOS results.

**Table 3 – Stability Analysis Results**

Section	Modelled Conditions	Global Factor of Safety		Compliant
		Required	Calculated	
A-A'	Measured Groundwater	1.5	2.3	Yes
	Extreme (Worst Credible) Groundwater	1.3	1.8	Yes
	Seismic Loading	1.2	1.5	Yes
B-B'	Measured Groundwater	1.5	2.4	Yes
	Extreme (Worst Credible) Groundwater	1.3	2.0	Yes
	Seismic Loading	1.2	1.6	Yes

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### Stability Conclusions

The global minimum FOS results given in Table 3 are greater than the Council requirements for all modelled conditions.

We therefore consider the site to currently be globally stable and suited to residential development from a global land stability point of view, however we recommend proposal-specific stability assessment/comment be sought during future development design phases.

In particular, the gully area requires future consideration. It will either be fully or partly filled, which will improve the stability of the slopes that lead up to Bomb Point Drive. Conversely, any excavation to capitalise on the gully topography (for example, to construct basement carparking) must first consider the temporary and permanent stability situation, again with particular reference to Bomb Point Drive.

## **9.0 Geotechnical Discussion**

We consider the site to be geotechnically suited to development provided the recommendations given in this report are observed.

Key points are as follows:

- Slope inclinations are, in the main, gentle to moderate. The exceptions are the gully area and the northern frontage, and some parts of the eastern boundary above the bushline. We recommend proposal-specific stability assessment/comment be sought for these locations as part of future design phases.
  - The proximity of buildings to unretained slopes will require consideration – further discussion is given in Section 10 below.
  - The soil varies between recent Holocene alluvial deposits, Puketoka Formation (ancient) alluvial deposits and Waitemata group soils. In places the soils were soft to firm, saturated and sandy; soils of this nature are susceptible to localised collapse in foundation or pile bores or where temporary excavations are made. These weaker soils are generally surficial.
  - Vane shear strengths measured in the augerholes were typically favourable for residential construction, however isolated lower strength results were encountered and in places the soil was described as ‘soft to firm’ in consistency. It is possible that during future investigation or construction that additional areas of lower strength soil could be encountered and may require remediation or undercutting in order to be a suitable foundation stratum.
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- Groundwater was encountered between 2.0m and 2.5m bpgl within all augerholes, however water seepage at the ground surface was noted in places.
  - The localised Holocene material expected in the gully area is not suited to the support of earthworks or buildings and will have to be removed. It may be suited to re-use as an engineered fill but will require removal of any organic content and space to dry.
  - The Puketoka Formation soils generally exhibited favourable vane strengths and material types however some caution is recommended due to the sparse information and the tendency of Puketoka material to vary across a site.
  - Waitemata Group soils are typically favourable for construction and generally offer low risk of undue settlement under typical terraced and low-level apartment building loads.
  - For larger apartment buildings, terraced buildings founded entirely on alluvial deposits or that cross geological boundaries, or that are supported on an engineered fill which has significant variation in depth across short distances, settlement (both total and differential) should be considered and accommodated by appropriate foundation design.
  - The site soils are likely to be suitable for cut to fill, however where weaker near surface soils are present, undercutting may be required prior to fill placement. Sandy soils were encountered in places and could be challenging to place and compact to an engineered standard. We have noted above the likely suitability and difficulties with the soils likely to be found in the gully.
  - Base Commanders Pond: the nature of the ground within the pond area and the associated pond banks is unknown. Both should be assessed as part of future geotechnical investigation. It should be expected that some degree of loose soil within the pond base will have to be mucked out and replaced with suitably compacted material, should Earthfilling or permanent structures be proposed in the area.

Proposal-specific investigation and assessments are required to support future design phases.

## **10.0 Foundation Design Discussion - Preliminary**

The site soils are generally suited to conventional spread (shallow) foundations or piled foundations exceptions being as described above. The required foundation system would depend on the nature of the proposed development, especially where Puketoka material overlies the Waitemata soils – the issue is not one of bearing capacity but of settlement characteristics.

It should be expected that larger apartment buildings (indicative: +5 levels) will require piles, whilst spread foundations are likely to be suitable for typical terraced dwellings up to and including 3 levels.

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Depending on the proximity to unretained slopes, buildings may require downslope perimeter piles capable of resisting lateral soil creep pressures. Buildings constructed hard against a slope crest may require a barrier-pile type of foundation configuration, either as part of the building or free-standing.

The gully represents an area where the depth of future engineered fill placed to create a construction subgrade could change markedly over a short distance. Settlement characteristics related to the effect of the fill mass on the underlying natural ground should be considered in foundation design - wherever significant change in fill depth is present. This consideration reduces with increasing time period between fill placement and foundation construction.

Bridging piles may be required where underground services are present within or adjacent to building footprints.

Foundation design requirements and design parameters should be provided in future proposal specific Geotechnical investigation.

## **11.0 Cuts and Fills**

All fills, regardless of depth, must be placed in accordance with NZS 4431:1989 with respect to subgrade preparation and standard of compaction.

Any proposal to create cuts or fills greater than 600mm in height should be the subject of specific design advice.

Proposals to fill the gully area must consider the Holocene deposits ('mullock') likely to be present. There is a requirement to install spring drainage and possibly a wider herring-bone underfill drainage system. That drainage should be permanent.

## **12.0 Retaining Structures**

Factors of safety and surcharge loadings appropriate to the conditions should be in accordance with 'Limit State Design of Retaining Walls and Foundations for Geotechnical and Structural Engineers' SESOC Seminar Series 2005.

We recommend retaining systems be Engineer-designed and consider both the local and global stability of the site, and any surcharge applicable to the wall. Particular attention should be paid to the influence of building surcharges above, and sloping ground above and below, any retaining wall.

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**13.0 Stormwater**

Concentrated stormwater flows must not be allowed to run onto or over slopes or saturate the ground as this could adversely affect slope stability or foundation conditions. Flows from all impermeable areas must be collected and carried in sealed pipes to a disposal point approved by Council.

**14.0 Underground Services**

Public underground wastewater services are present within the site. Additional underground services, public or private, mapped or unmapped, of any type (gas, pipelines, fibre, electricity etc) could be present. A thorough service-search should be carried out prior to commencement of excavations.

**15.0 Additional Investigation**

This report has been prepared for feasibility purposes only.

A proposal specific geotechnical investigation must be carried out to support initial and developed designs. The nature and scale of the required assessments will depend on the proposed development.

End of Report Text – Appendices Follow

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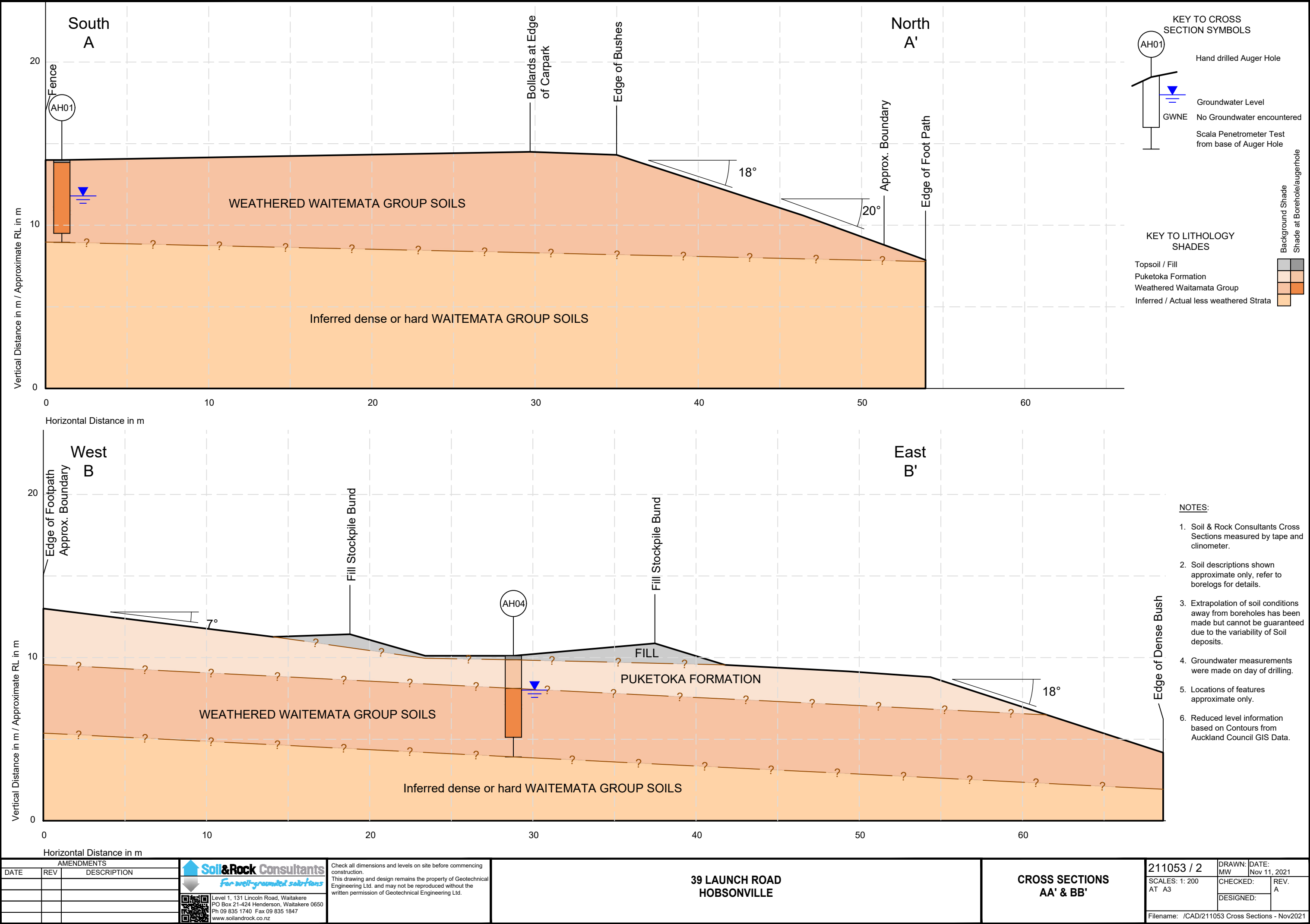
## Appendix A

### Drawings – Soil & Rock Consultants Site Plan & Geological Sections









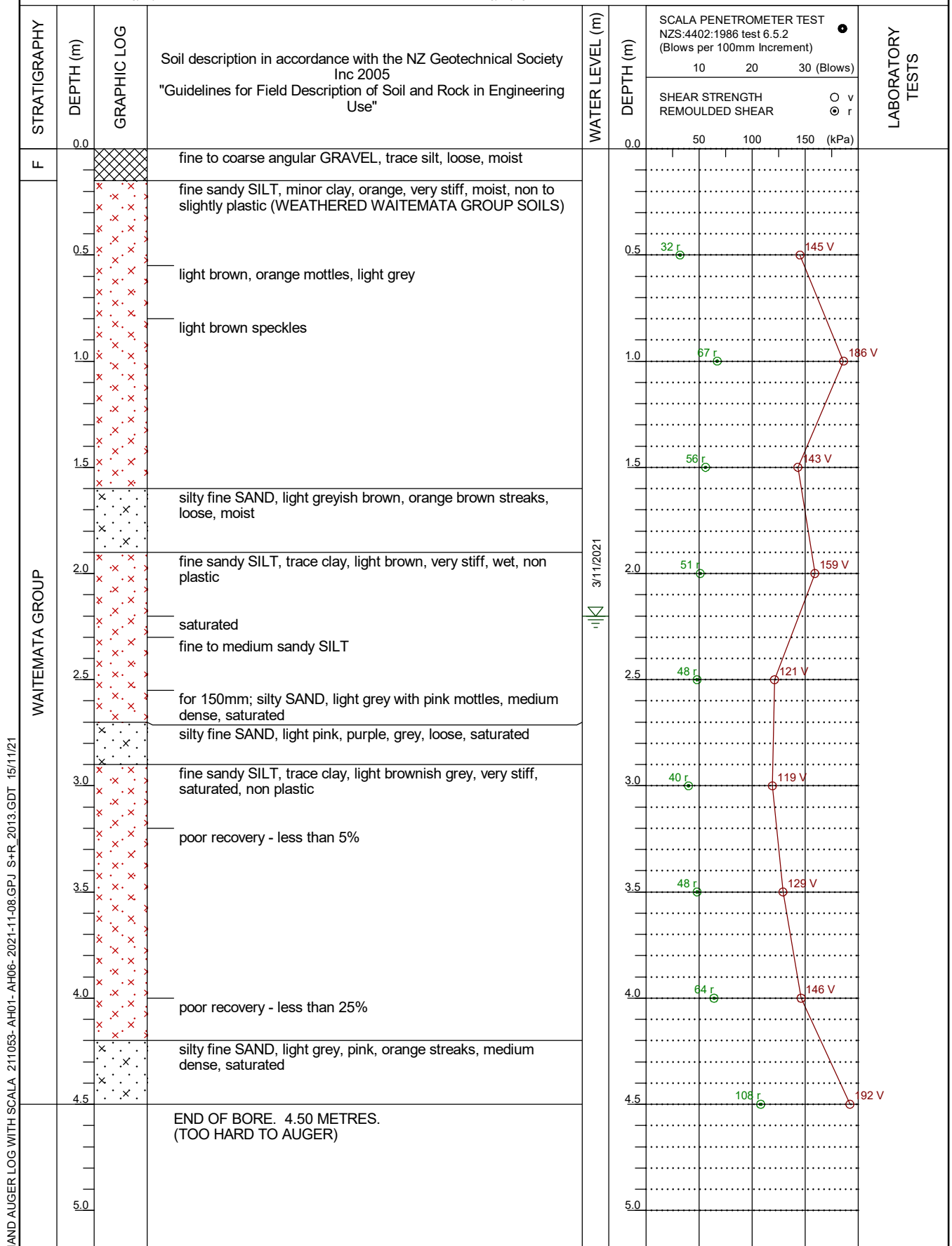
## Appendix B

### Investigation Logs (Augerholes, Scala Penetrometer Results)

Drill Type: 50mmØ Hand Auger  
 Drilled By: NN  
 Date Started: 3/11/21  
 Date Finished: 3/11/21

Project No: 211053  
 Coordinates:  
 Ground Elevation:  
 Water Level: 2.2m 3/11/2021

Logged By: NN  
 Shear Vane No - Calibration Date: GEO604 - 24/11/2020  
 Surface Conditions: Near Level, Gravel

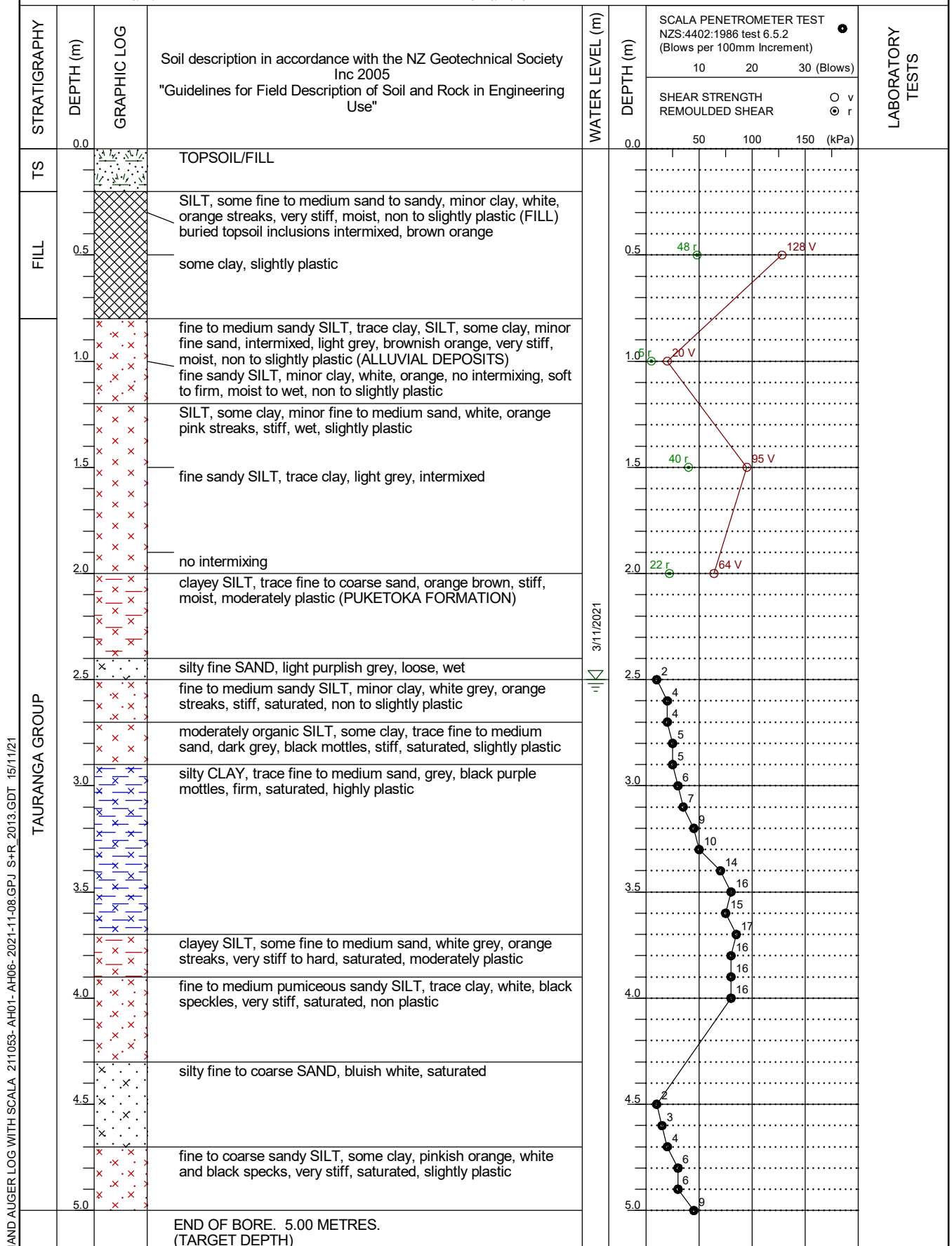


HAND AUGER LOG WITH SCALA 211053-AH01-AH06-2021-11-08 GPJ S+R 2013.GDT 15/11/21

Drill Type: 50mmØ Hand Auger  
 Drilled By: RH  
 Date Started: 3/11/21  
 Date Finished: 3/11/21

Project No: 211053  
 Coordinates:  
 Ground Elevation:  
 Water Level: 2.5m 3/11/2021

Logged By: RH  
 Shear Vane No - Calibration Date: GEO765 - 10/03/2021  
 Surface Conditions: Near Level, Grass



HAND AUGER LOG WITH SCALA 211053-AH01-AH06-2021-11-08 GPJ S+R 2013.GDT 15/11/21



**Soil & Rock Consultants**  
Your responsive & cost-effective engineers

CLIENT: Kainga Ora

PROJECT: Geotechnical Investigation, 39 Launch Road, Hobsonville

Auger Hole No: AH03

Sheet 1 of 1

Drill Type: 50mmØ Hand Auger

Drilled By: JT

Date Started: 3/11/21

Date Finished: 3/11/21

Project No: 211053

Coordinates:

Ground Elevation:

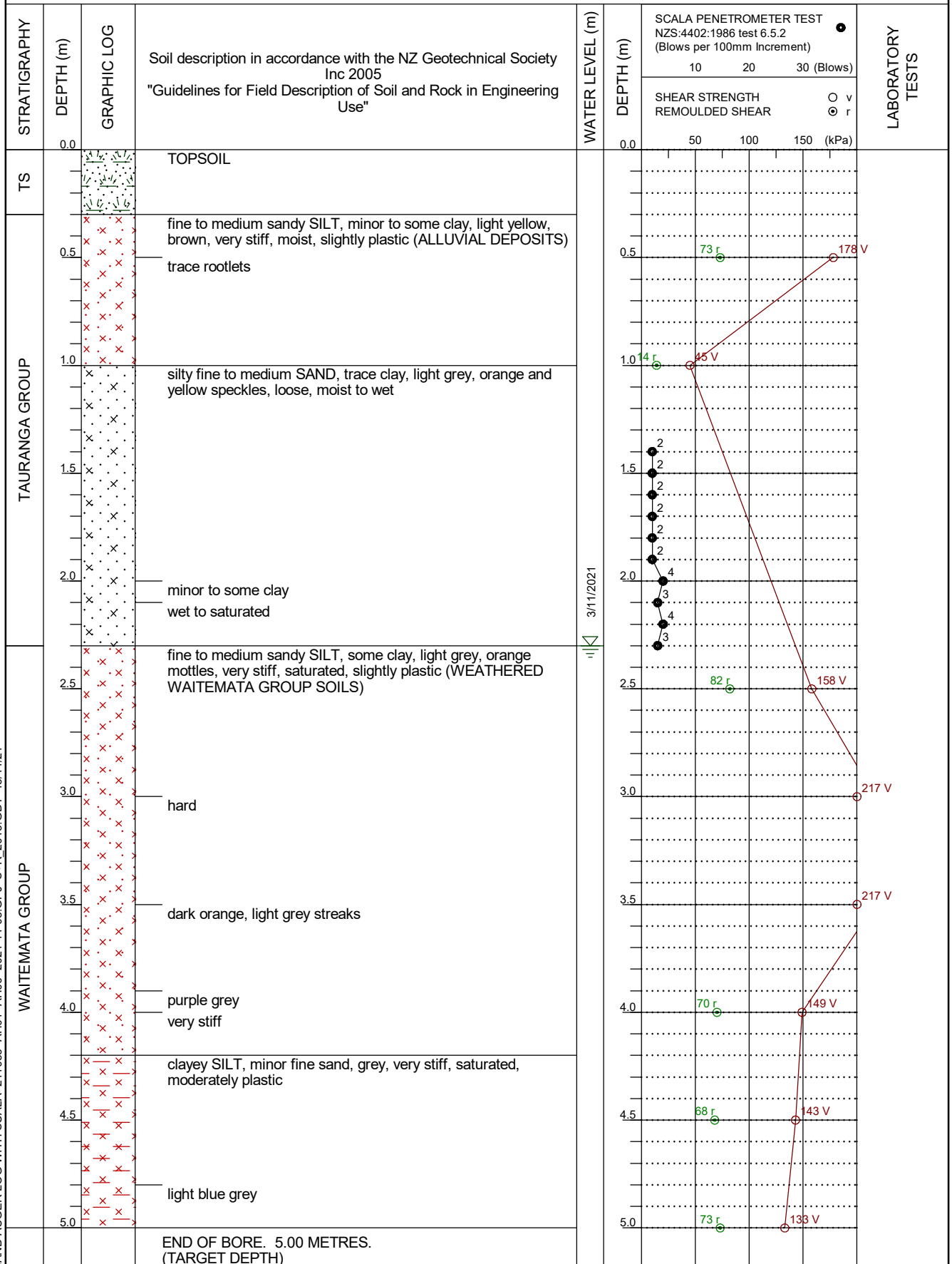
Water Level: 2.3m 3/11/2021

Logged By: JT

Shear Vane No - Calibration Date: GEO1591 - 5/03/2021

Surface Conditions: Near Level, Gravel

HAND AUGER LOG WITH SCALA 211053-AH01-AH06-2021-11-08.GPJ S+R 2013.GDT 15/11/21





**Soil&Rock Consultants**  
Your responsive & cost-effective engineers

CLIENT: Kainga Ora

PROJECT: Geotechnical Investigation, 39 Launch Road,  
Hobsonville

Auger Hole No: AH04

Sheet 1 of 1

Drill Type: 50mmØ Hand Auger

Drilled By: JT

Date Started: 3/11/21

Date Finished: 3/11/21

Project No: 211053

Coordinates:

Ground Elevation:

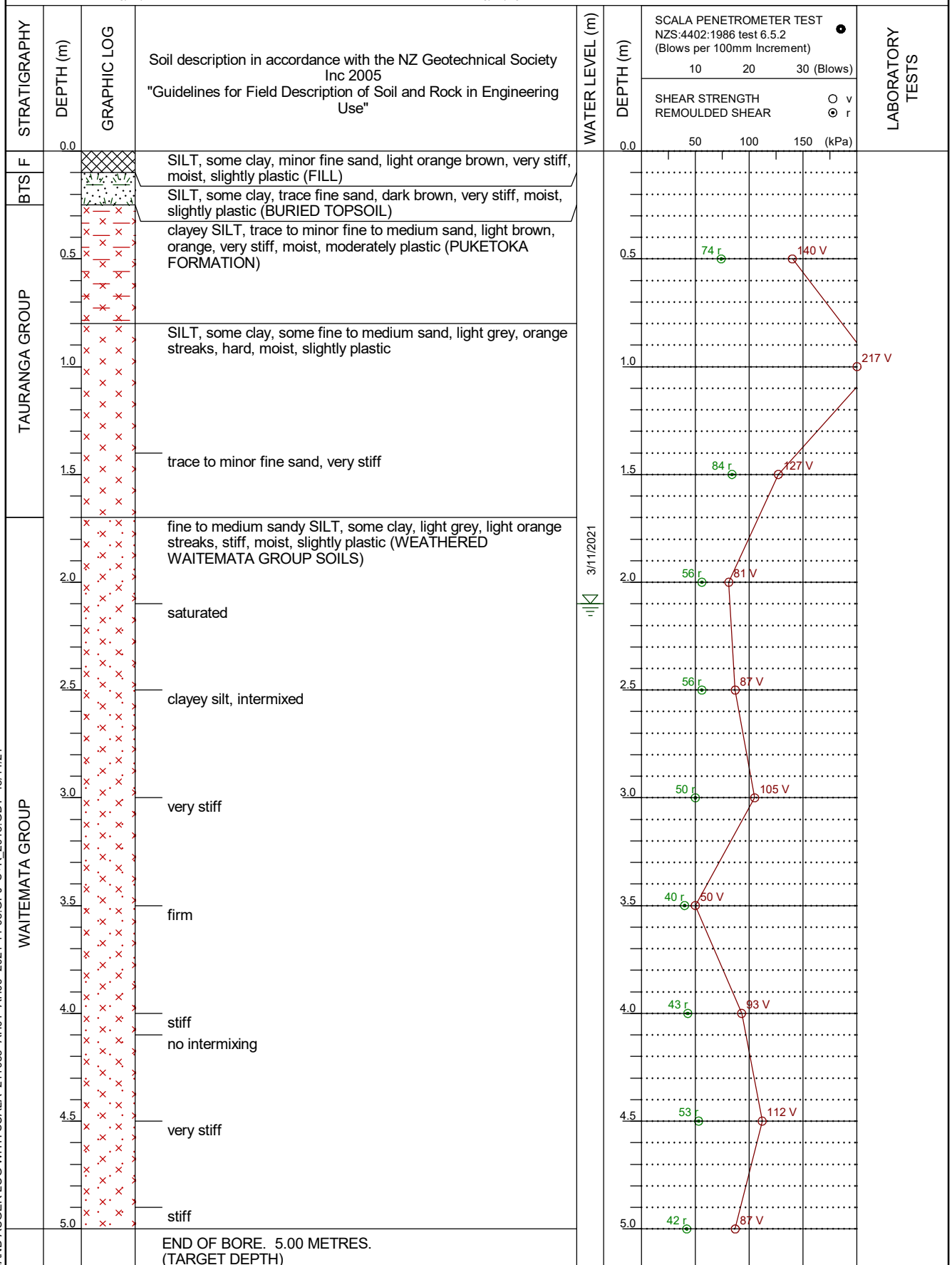
Water Level: 2.1m 3/11/2021

Logged By: JT

Shear Vane No - Calibration Date: GEO1591 - 5/03/2021

Surface Conditions: Near Level, Soil

HAND AUGER LOG WITH SCALA 211053-AH01-AH06-2021-11-08.GPJ S+R 2013.GDT 15/11/21





**Soil & Rock Consultants**  
Your responsive & cost-effective engineers

CLIENT: Kainga Ora

PROJECT: Geotechnical Investigation, 39 Launch Road,  
Hobsonville

Auger Hole No: AH05

Sheet 1 of 1

Drill Type: 50mmØ Hand Auger

Drilled By: RH

Date Started: 3/11/21

Date Finished: 3/11/21

Project No: 211053

Coordinates:

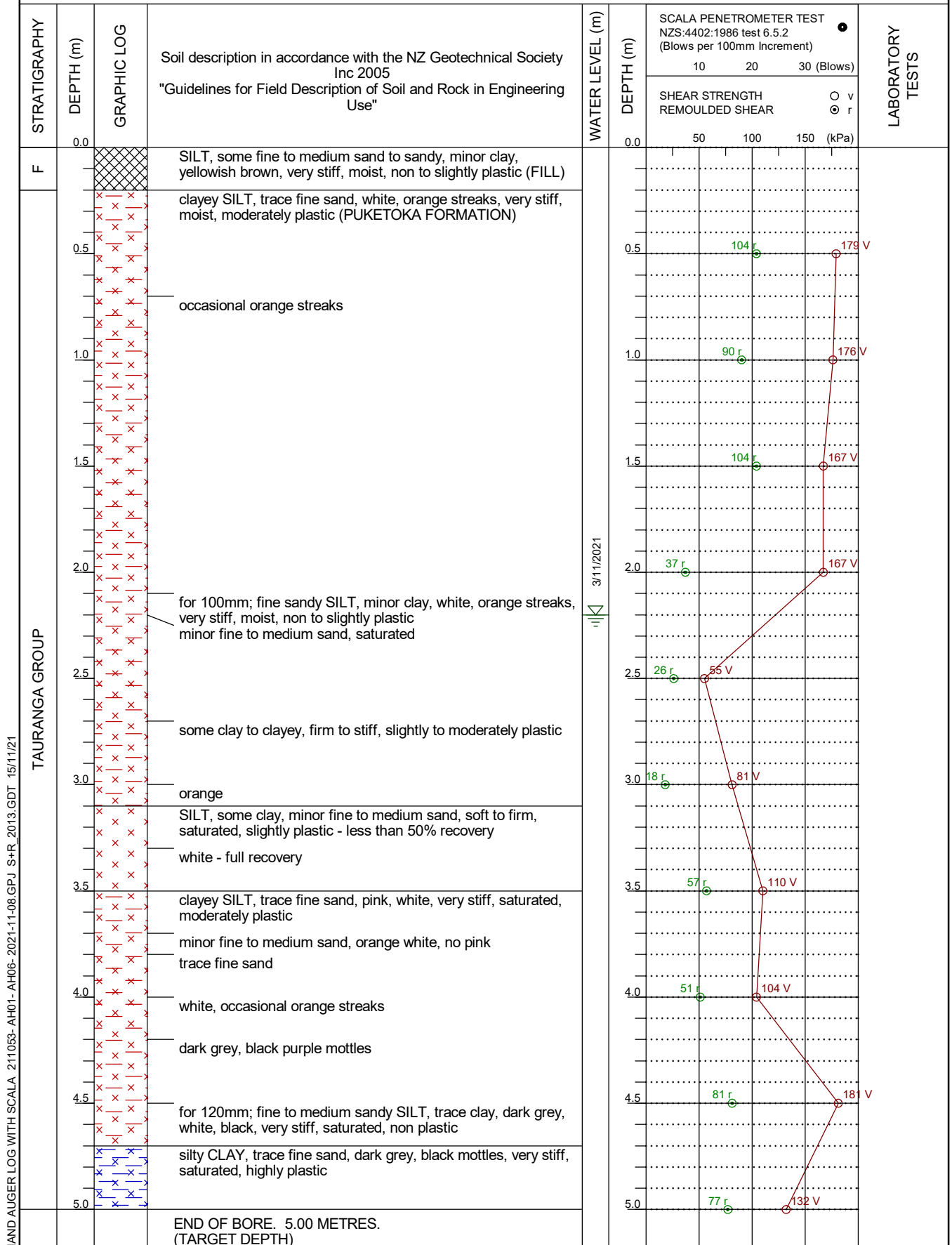
Ground Elevation:

Water Level: 2.2m 3/11/2021

Logged By: RH

Shear Vane No - Calibration Date: GEO765 - 10/03/2021

Surface Conditions: Near Level, Soil



HAND AUGER LOG WITH SCALA 211053-AH01-AH06-2021-11-08 GPJ S+R 2013.GDT 15/11/21





**Soil & Rock Consultants**  
Your responsive & cost-effective engineers

CLIENT: Kainga Ora

PROJECT: Geotechnical Investigation, 39 Launch Road, Hobsonville

Auger Hole No: AH06

Sheet 1 of 1

Drill Type: 50mmØ Hand Auger

Drilled By: NN

Date Started: 3/11/21

Date Finished: 3/11/21

Project No: 211053

Coordinates:

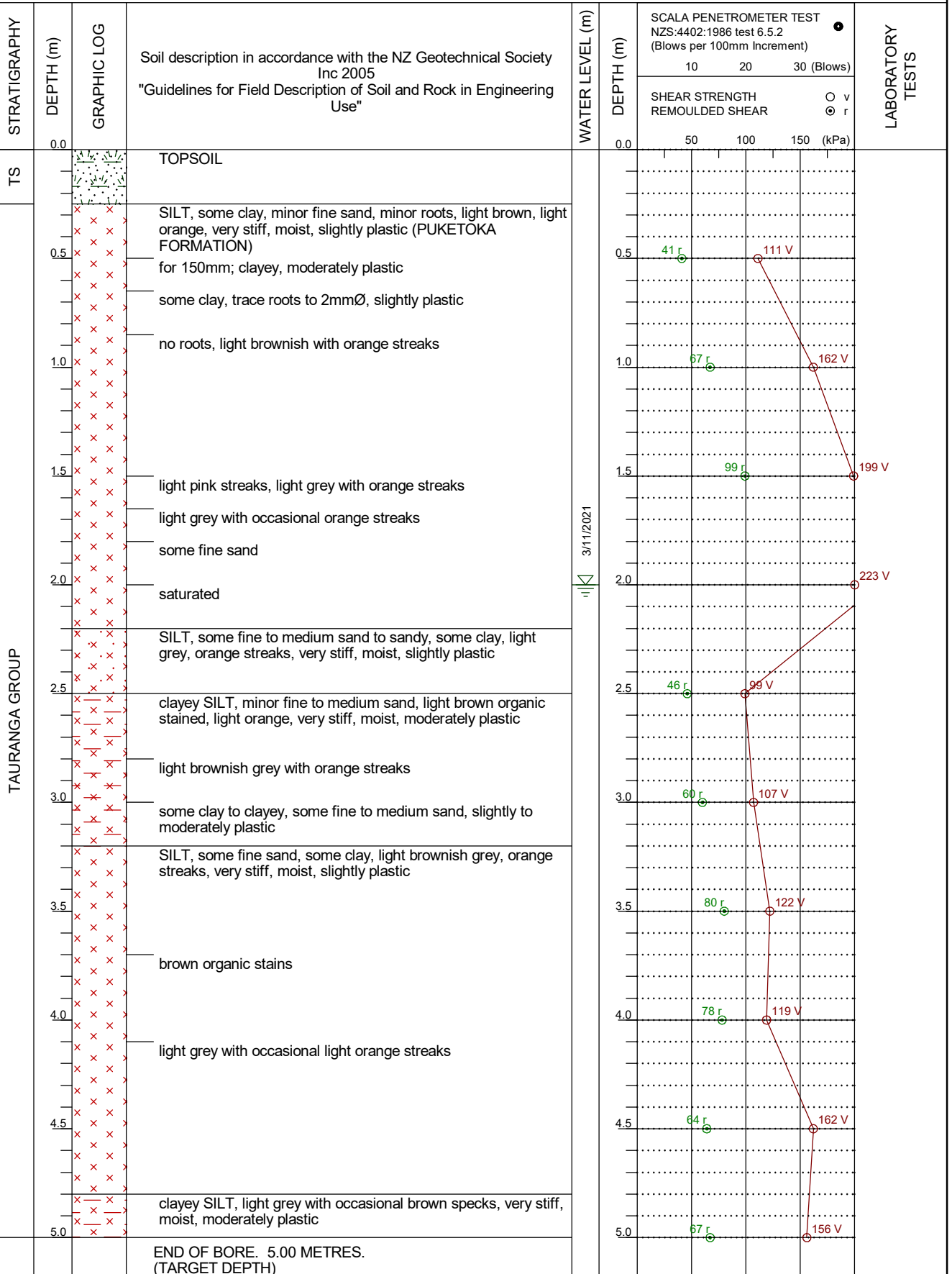
Ground Elevation:

Water Level: 2.0m 3/11/2021

Logged By: NN

Shear Vane No - Calibration Date: GEO604 - 24/11/2020

Surface Conditions: Slightly Sloping, Grass



HAND AUGER LOG WITH SCALA 211053-AH01-AH06-2021-11-08 GPJ S+R 2013.GDT 15/11/21



## SCALA PENETROMETER SHEET - TABLE OF BLOWS PER INCREMENT

**JOB NO:** 211053

**TESTED BY:** RH/JT/NN

**JOB NAME:** 39 Launch Rd, Hobsonville

**DATE:** 3/11/2021

Depth of Penetration [mm]	AH01		AH02		AH03		AH04		AH05		AH06	
DEPTH START[m] ➡	4.50		5.00		5.00		5.00		5.00		5.00	
50 mm	6		5		1		1		2		1	
100	7		5		1		1		3		2	
150	8		5		1		2		2		2	
200	10		5		2		3		3		2	
250	9		5		2		6		4		3	
300	10		5		3		3		4		3	
350	10		6		4		3		4		5	
400	10		6		4		4		5		6	
450	10		6		5		3		6		6	
500	11		7		5		3		7		6	
550	12		7		6		3		7		8	
600			8		6		6		7		9	
650			7		7		6		8		10	
700			6		8		6		9		11	
750			5		9		7		10		12	
800			7		10		7		10		12	
850			7		11		7		10		13	
900			7		12		8		10			
950			7		12		9		11			
1000			7		14		10					
1050			9		11		10					
1100			10				11					
1150			10				11					
1200			10				11					
1250			10									
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1850												
1900												
1950												
2000												
DEPTH END [m] ➡	5.05		6.30		6.05		6.20		5.95		5.85	

Testing Method: NZS 4402:1988 Test 6.5.2 Dynamic Cone Penetrometer

## Appendix C

### Slope Stability Results

