
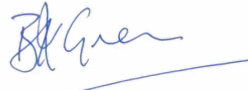





**GEOTECHNICAL INVESTIGATION:  
401-403 PARNELL ROAD, PARNELL  
PROPOSED MULTI LEVEL APARTMENT / RETAIL DEVELOPMENT**

<b>Job Number:</b>	15761
<b>Name of Project:</b>	Geotechnical Investigation - Proposed Apartments
<b>Client:</b>	Skirmantis Saltiss
<b>Author:</b>	Sean Berry (MSc - Geology) Hydrogeologist / Engineering Geologist
<b>Reviewer :</b>	Bruce Green (NZCE, CPEng) Senior Geotechnical Engineer
<b>Approved:</b>	Zeljko Viljevac (MSc - Geology) Senior Hydrogeologist / Engineering Geologist
<b>Document Version:</b>	R001
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**(i) REPORT SUMMARY**

The currently proposed development at 401-403 Parnell Rd is to comprise the construction of a new four storey commercial / residential apartment building. The proposed development also includes excavation of up to 2.5m for installation of vertical vehicle stackers at the rear of the site.

Following the investigation and analysis we provide a summary of our Geotechnical Constraints and Recommendations as described below:

- The new structure can be supported on a bored pile foundation embedded a minimum of 1.5D into the sandstone rock mass, encountered from an approximate depth of 11.7m below present ground level;
- Bored pile foundations should be designed for an Ultimate End Bearing Capacity of 3.0 MPa below 12.0m bpgl;
- Piles should be designed for the following range of depth-dependant Ultimate Skin Friction values as follows;
  - 0.0 - 1.0 m bpgl: Negligible
  - 1.0 - 5.0m bpgl: 15 kPa
  - 5.0 - 7.5m bpgl: 20 kPa
  - 7.5 - 12.0m bpgl: 50 kPa
  - 12.0+ m bpgl: 200 kPa
- Dependable Bearing Capacity and Dependable Skin Friction values may be determined based on a Strength Reduction Factor of  $\phi = 0.5$  (Ultimate End Bearing Capacity);
- Any organic or deleterious material, topsoil or weak fill should be removed from floor slab areas prior to placement of sub-base and basecourses. Any backfill should be by means of placement of good quality granular fill;
- Backfill for buried services should be placed to an engineered standard. Care should be undertaken during excavations to ensure services are protected and/or disconnected prior and during construction;
- Due to the presence of shallow groundwater beneath the site temporary or permanent casing support may be required for piled foundations;
- The temporary and permanent groundwater conditions shall be considered by the designer and contractor, in particular the retained voids associated with vehicle stacker construction;
- Given the limited excavation in terms of plan area and depth, and that piezometer measurements have shown the groundwater table to be at approximately 6.4m below present ground level, we infer no negative effects on neighbouring properties from groundwater drawdown;
- A geotechnical review of the finalised design is recommended prior to application for Building Consent;
- The foundation excavations should be inspected by a geotechnical engineer prior to placement of any filling or concrete blinding or reinforcing.

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## 1.0 Introduction

Soil & Rock Consultants were requested by Skirmantis Saltiss to carry out a geotechnical site investigation for a proposed four storey retail and residential apartment block at 401-403 Parnell Rd, Parnell (The Site). This report summarises our findings and recommendations and may be used to support a Resource or Building Consent application to Auckland Council.

### 1.1 Scope and Brief

The principal objective of the investigation is to develop a geotechnical model of the site so that recommendations with regard to geotechnical constraints to the proposed development be identified. The scope included provision of geotechnical recommendations and foundation design parameters for the proposed 4 storey structure.

The proposed development also includes minor excavation at the rear (southwest) of the site for installation of a hydraulic vehicle stacker. Our scope and brief includes recommendations regarding excavation and foundations for this equipment.

### 1.2 Limitations

This report has been prepared for the sole benefit of our client Skirmantis Saltiss with respect to the particular brief given to us. This report may be used by our Clients appointed Consultants and may be relied upon by Auckland Council when considering any proposed application in association with the proposed development. The data and/or opinions contained in this report may not be used by any other party or for any other purpose without our prior review and agreement.

The recommendations given in this report are based on investigation data from discrete locations. We have inferred a geotechnical model for our analyses however variations in ground conditions from those described in this report could exist across the site. Should ground conditions encountered differ to those outlined in this report we should be advised in order to review the continued applicability of our recommendations.

## 2.0 Site Description & Proposed Development

### 2.1 General

The subject property (the site) is situated on the southwestern side of Parnell Road, adjacent to the intersection with St Stephens Ave, and is legally described as Lot 10 DP 1967, being rectangular with a plan area of approximately 336m<sup>2</sup>.

The site is generally near level at an approximate elevation of RL 65m. Ground level across the immediate surrounding area is gently sloping down to the northwest.

At the time of our investigation the site is occupied by an existing single storey retail structure, which occupies the front (northeast) half of the property. The rear (southwest) half of the property is entirely paved. Access to the paved area is via a sealed driveway located outside the south eastern boundary of the site, being within the adjacent property at 405-411 Parnell Rd. A narrow (<1m) service corridor is located along the north western side of the site, between the existing structure and the adjacent building at 381-395 Parnell Rd.

The site is bounded to the north by a multi-storey retail and residential complex, to the east by Parnell Rd, to the south by 1-2 storey retail buildings, and to the west by residential properties.

A private stormwater catch pit is located in the centre of the rear paved area. A search of underground services (Auckland Council GIS and B4UDig) has not shown any public services extending beneath the site. **Public stormwater and wastewater services are located within the Parnell Rd road reserve.**

## 2.2 Proposed Development

The proposed development comprises demolition of the existing single storey building and construction of a new four storey retail and residential building. The proposed structure is to comprise ground level retail areas at the front of the site, with three residential floors above. The rear of the site will comprise ground level car parking, with three residential floors above, and will be connected to the front half of the structure via a covered stairwell. The proposed car stackers will be located beneath the car parking areas. Vehicle access to the proposed parking area at the rear of the site will be via a new driveway along the northwest boundary. A vehicle turntable will be located in the centre of the parking area to assist with vehicle manoeuvring.

The proposed development will entail **cuts of up to approximately 2.5m below present ground level** (bpgl).

## 3.0 Previous Geotechnical Investigations

Soil & Rock Consultants are in receipt of a previous geotechnical investigation report for the adjacent property at 381-397 Parnell Rd (Tonkin & Taylor, Ref. 14629, February 1998) which indicated volcanic air-fall (tuff and ash) to a depth of between approximately 5m and 7m bpgl beneath the site.

The volcanic deposits were reported as being underlain by between 3m and 4m of Pleistocene sediments, with residual Waitemata Group soils present at depths between approximately 9m and 10m bpgl. The depth to dense Waitemata Group strata beneath the site was inferred to be between 11m and 13m bpgl.

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#### 4.0 Published Geology

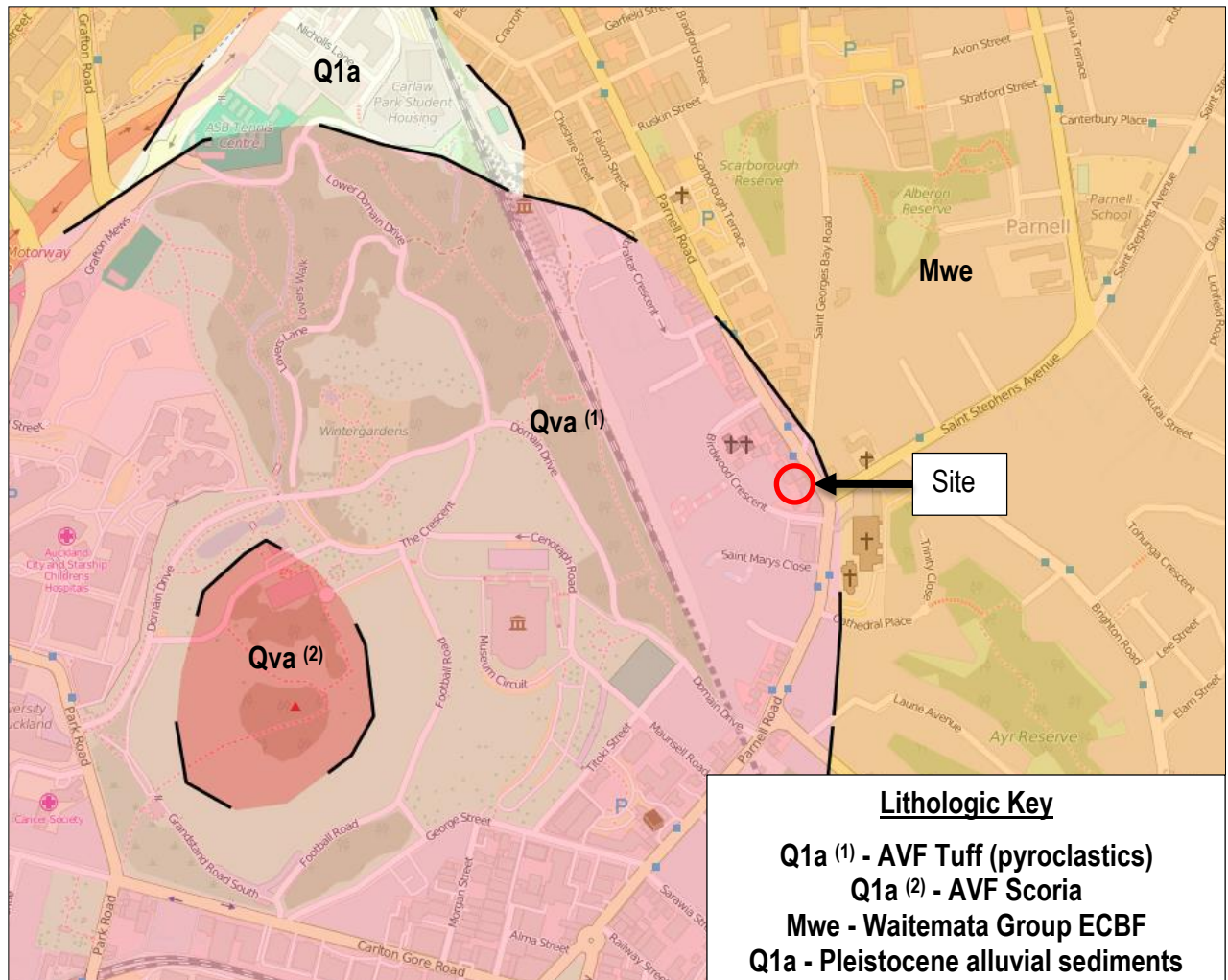
Reference has been made to the Auckland Urban Area Geology Map, Sheet R11, Scale 1:50,000, published 1992, indicating that the site is underlain by basalt tuff of the Auckland Volcanic Field (AVF; Kerikeri Volcanic Group), originating from the Auckland Domain volcanic centre.

These air-fall volcanic materials range in age from 140 ka to recent. This unit maybe very soft, compacted or cemented. Lithic tuff forms annular mounds or tuff rings around explosion craters that range between 150 m to 1000 m around 34 of the Auckland explosion craters. Tuff can be 50 m thick near eruption centres but rapidly thin with distance. The deposits can persist as a very thin mantle for several kilometres downwind of the volcano.

Tuff consists of very thin (2 mm – 60 mm) graded beds of light grey to grey, angular to rounded, well sorted silt and sand sized ejecta. These deposits may contain fragments of comminuted country rock which range from alternating mudstone and sandstone, alluvium, micaceous sand, greywacke, with basaltic or basanitic fragments between ash. These fragments range from 0.06 mm to 600 mm diameter. Lithic Tuff weathers to a soft to very soft, light grey to light brown clay to depths less than 1m.

East Coast Bays Formation (ECBF) deposits of the Waitemata Group of Miocene Age are shown as being present immediately east of the site, on the far side of Parnell Rd. These rocks are described as turbidite or inter-turbidite Flysch (alternating sandstone and mudstone) deposits, comprising greenish-grey, alternating muddy sandstone and mudstone. The sandstone beds are generally moderately thinly to extremely thickly bedded, usually gently dipping, but in places convoluted and often graded (coarse becoming finer upwards). Alternating mudstone beds are typically thinly to moderately thinly bedded. In some areas interbedded lenses of grit may be present. These deposits are generally greenish grey or grey when fresh, often light grey, then weathering to light brown or brown. Moderately widely to very widely spaced fractures may be present within the rock mass with the rock strength variable between extremely weak and strong.

The presence of alluvial deposits (Pleistocene sediments) is noted to the north-east of the site, in the vicinity of Carlaw Park.

**Figure 1: Local Geology (IGNS, Sheet R11, 1992)**

According to the GNS 'New Zealand Active Faults Database' the closest active fault line to the site is the Wairoa North Fault, approximately 30km east of the site. Slip rate, event displacement and recurrence interval have not been reliably established.

## 5.0 Subsurface Investigation

Investigation fieldwork comprised the drilling of one machine borehole, MB01, to depth of 13.8m bpgl. The machine borehole location is shown on the attached Site Plan, Drawing No. 15761/1 (Appendix C). The location of the borehole was measured in by tape from existing site features without survey instrument control and is therefore approximate only.

Measurements of undrained shear strengths were undertaken in the cohesive soils at the end of the sample barrel at intervals of depth by means of a hand held shear vane. The test method was in accordance with the New Zealand Geotechnical Society Guidelines for Hand Held Shear Vane Tests, dated August 2001. The peak vane shear strengths

and the remoulded vane shear strength values shown on the attached Machine Bore logs represent dial readings off the shear vane, adjusted using the BS 1377 calibration.

A visual-tactile field classification of the soil encountered during drilling was carried out in accordance with the "Guidelines for the Field Description of Soils and Rocks in Engineering Use", issued by the New Zealand Geotechnical Society Inc. (2005).

Standard Penetration Testing (SPT) was undertaken at selected depths within the machine boreholes. SPT "N" blow-count values or, if refusal was encountered the depth penetrated for 60 blows, are recorded on the borehole logs. Measurements of the groundwater table were carried out on the days of drilling.

## 6.0 Subsurface Conditions

Subsurface conditions encountered at the locations of our machine holes are summarised below and a detailed description of the soils is given on the attached logs (Appendix B). Subsurface conditions have been interpolated away from the test location and therefore localised variations across the site will exist.

In general the subsurface ground profile is comprised of a layer of asphalt pavement underlain by fill and volcanic air-fall (tuff). The presence of Puketoka Formation alluvial soils underlying the volcanic soils has been confirmed during this investigation. Miocene age East Coast Bays Formation of the Waitemata Group has been confirmed as underlying the Puketoka Formation at depth beneath the site.

The subsurface conditions encountered in the machine holes are summarised as follows:

- **Fill.** Non-cohesive granular fill (pavement base coarse) was encountered to a depth of 0.4m bpgl and was underlain by clayey SILT non-engineered fill to a depth of 0.8m bpgl.
- **Volcanic Tuff.** Volcanic deposits (i.e. Volcanic Tuff) were encountered underlying the fill to a depth of 5m bpgl. The deposits were described as being predominantly SILT with varying clay and sand content, and silty CLAY with trace fine grained sand. Measurements of undrained shear strength of the cohesive soils varied from 56kPa (stiff) to 99kPa (stiff/very stiff).

SPT testing within the volcanic soils ranged between 3 and 8 blows for the final 300mm of penetration, to a depth of 5.0m bpgl, corresponding to a loose to very loose soil mass.

- **Puketoka Formation.** Alluvial deposits of the Puketoka Formation were encountered at a depth of 5.0m bpgl. The deposits were described as being predominantly SILT with varying clay and fine grained sand content, and minor organic content (fibrous and amorphous inclusions). Measurements of undrained shear strength of the cohesive soils varied from 85 kPa (stiff) to 138kPa (very stiff).



Two SPT tests within Puketoka Formation soils both recorded 11 blows for the final 300mm of penetration, at depths of 6.0m and 7.5m bpgl, corresponding to a medium dense soil mass.

- **Waitemata Group Soils.** Residually weathered soils of the East Coast Bays Formation were encountered below the Puketoka Formation soils extending from 9.0m bpgl to 11.7m bpgl. The deposits were described as being predominantly SILT with varying clay and sand content. Measurements of undrained shear strength of the cohesive soils were 197kPa (very stiff/hard) at a depth of 9.0m bpgl and greater than 200kPa at a depth of 10.5m bpgl (i.e. UTP; soils were unable to be penetrated by hand held test equipment).

Two SPT tests within the Waitemata Group Soils recorded 5 blows for the final 300mm of penetration at a depth of 9.0m bpgl (loose soil mass) and 11 blows for the final 300mm of penetration at a depth of 10.5m bpgl (medium dense soil mass).

- **Waitemata Group Rock.** Extremely weak to very weak rock of the East Coast Bays Formation (ECBF) was encountered below the Waitemata Group soils at a depth of 11.7m bpgl, extending to the borehole termination depth of 13.8m bpgl. The ECBF rock was described as grey fine grained SANDSTONE with trace siltstone.

Two SPT tests within the Waitemata Group Rock, at depths of 12.0m and 13.5m bpgl refused at N values of 50+ for between 300mm to 450mm of total penetration, indicating a very dense material (N Value > 50).

- **Groundwater Table.** The depth of the groundwater table was recorded on the day of drilling at a depth of 3.5m bpgl. Post drilling groundwater measurements in the installed piezometer reports a stabilised groundwater depth of 6.4m bpgl, indicating the potential for shallow groundwater seepage at approximately 3.5m bpgl.

A summary of subsurface conditions is presented in Table 1.

**Table 1 – Summary of Subsurface Conditions**

Borehole	Total Depth	Fill	Volcanic Soils	Alluvial Soils	Waitemata Soils	Waitemata Rock	Groundwater
	Depth (m bpgl)						
MB01	13.8	0.8	5.0	9.0	11.7	13.8+	3.5 <sup>(1)</sup> 6.4 <sup>(2)</sup>

Table Notes: <sup>(1)</sup> Groundwater seepage on day of drilling (15-Feb-16)

<sup>(2)</sup> Post-drilling stabilised groundwater measurement 22-Feb-16 & 24-Mar-16

## 7.0 Site Seismicity

For the purposes of applying requirements of NZS 1170.5:2004, the site subsoil is **Class C – ‘Shallow soil site’**. This classification is based on depth to rock less than the maximum limits reported in Table 3.2 of the standard.

According to the GNS 'New Zealand Active Faults Database' the closest active fault line to the site is the Wairoa North Fault, approximately 30km east of the site. Slip rate, event displacement and recurrence interval have not been reliably established.

The nature of the soils at the site (i.e.: high silt content with organic inclusions) and the potential for an elevated groundwater table indicates the site has moderate to low potential for liquefaction during a seismic event.

## 8.0 Settlement Assessment

The upper foundation soils comprise stiff to very stiff volcanic soils, in turn underlain by Puketoka Group soils with noted organic lenses and inclusions. As such these materials are generally regarded as being of high potential compressibility.

Based on the findings of our fieldwork and the proposed development comprising, we recommend the proposed four storey structure is supported on piles taken down to the Waitemata Group rock encountered at 11.7m bpgl in the location of MB01.

## 9.0 Geotechnical Design Recommendations

It is our opinion that the site is suitable for the proposed development provided the design recommendations given are observed. Due to the near-level nature of the site, global slope stability is not considered to be of concern for the development.

We understand bulk excavations to form the building platform are not proposed in close proximity to neighbouring buildings. Notwithstanding this even modest excavations against a building can compromise the foundation performance of the existing building, particularly during the construction period when minor unsupported cuts could be present, allowing foundation soil relaxation or undue drying, leading to soil shrinkage. The designer and contractor should consider these issues as part of the construction methodology.

### 9.1 Building Foundations

It is our opinion that shallow foundations could allow unacceptable levels of total and differential settlement to develop across the building. We therefore recommend that piled foundations support the structure.

Based on the reported undrained shear strengths and SPT N values, the following design parameters are recommended for foundation design in accordance with AS/NZS 1170:2002 for bored, steel reinforced concrete pile foundations embedded a minimum of 1.5D into the weak to very weak sandstone rock of the East Coast Bays Formation.

**Table 2 – Pile Foundation Design Parameters**

Depth (m-bpgl)	Ultimate (Rupture) End Bearing Capacity (MPa)	Ultimate Skin Friction (kPa)
0.0 - 1.0	---	---
1.0 - 5.0	---	15
5.0 - 7.5	---	20
7.5 - 12.0	---	50
12.0 +	3.0	200

The minimum expected depth to the sandstone rock mass is 11.7m bpgl however could be deeper in places (i.e. minimum embedment depth for 1.0m Ø pile of 13.2m bpgl). We recommend that bored piles be used in order that foundation soils at depth can be confirmed. Structural design may require a deeper embedment.

We recommend that all piled foundation excavations be inspected by a Geotechnical Engineer prior to placement of concrete to determine that satisfactory founding conditions have been achieved.

Given the predominantly silty nature of the soils at depth, and peat and soft organic soils in places from ground level to 9m bpgl, and shallow depth to groundwater, temporary or permanent casing to support the sides of the bored pile excavation at the time of drilling may be required to prevent collapse of unsupported materials.

Dependable Bearing Capacity and Dependable Skin Friction may be determined based on a Strength Reduction Factor of  $\phi = 0.5$  for static calculations and  $\phi = 0.8$  for short-term seismic conditions.

The proposed vehicle stackers should be founded on a minimum of one pile in each corner (i.e. point loads).

## 9.2 Floor Slabs

Any organic or deleterious material, topsoil and non-engineered fill should be removed from floor slab areas prior to placement of sub-base and basecourses. Any remediation of weak fill should take the form of placement of good-quality granular fill (GAP-type hardfill).

Following removal of unsuitable materials we consider that stiff natural ground and engineered fill should be suitable for a reinforced concrete slab-on-grade (floor slab). Prior to placement of basecourse the subgrade should be proof-rolled to detect any significant deflection or soft spots which should be excavated and backfilled with compacted granular fill

Following preparation of the subgrade a basecourse comprising free-draining aggregate should be placed and compacted. The compaction of the basecourse should be carried out with a vibratory roller of appropriate static weight and energy.

### 9.3 Retaining

Retaining will be required to create space to site the vehicle stacker lift mechanisms. Retaining design may be carried out using the following parameters:

- Unit Weight ( $\gamma$ ) = 18 kN/m<sup>3</sup>
- Friction Angle ( $\Phi'$ ) = 30°
- Minimum  $K_0$  = 0.5
- Appropriate vehicle surcharges

Consideration is also required with respect to temporary and permanent groundwater conditions.

#### Permanent Groundwater Conditions

It may not be practical to establish the preferred option of permanent groundwater drainage within the vehicle stacker excavation. In that case the designer must consider hydrostatic loads on the retaining. In addition the designer and contractor must ensure adequate tanking is applied to structures.

#### Temporary Groundwater Conditions

Considerations regarding temporary (i.e. during construction) groundwater conditions are given in Section 9.4 below.

### 9.4 Vehicle Stacker Excavations

The designer and contractor should be cognisant of the reported depth to groundwater beneath the site (3.5m to 6.4m bpgl). The reported groundwater depth has been recorded during summer months, and consequently may be expected to rise during winter.

Reported groundwater depths are equivalent to a maximum approximate elevation of RL 61.5m, being 1m below the proposed FFL of the vehicle stackers.

Groundwater levels may rise to depths less than that of the vehicle stacker excavations during periods of prolonged or heavy rainfall. Appropriate drainage should be specified by the designer.

### 9.5 Underground Services

Underground services checks indicate a private stormwater line extending across the rear of the site. Backfill for buried services should be placed to an engineered standard. Care should be undertaken during excavations to ensure services are protected and/or disconnected prior and during construction.

A thorough search for all types of underground services should be carried out prior to commencement of excavations.

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The proposed foundations are recommended to be piled to approximately 12.7m bpgl, and we therefore consider the piles will be below the zone of influence of the buried services. Pile foundations are to be located to not intercept any buried services beneath the proposed development area.

### **10.0 Stormwater Disposal**

Concentrated stormwater flows from roofed areas must be collected and carried in sealed pipes to the reticulated Council system. Stormwater should not be allowed to saturate the subgrade around the site in order that current founding conditions are maintained.

### **11.0 Inspections and Certification**

The recommendations given above are based on data from discrete test locations and general knowledge of the area. Variations in ground conditions could exist across the site. It is in the interests of all parties that we be retained to inspect excavations and foundation conditions exposed during construction, so that ground conditions can be compared with those assumed in formulating this report.

We recommend that excavations be inspected by a Geotechnical Engineer prior to placement of concrete or fill. Any foundations (including piles) or excavations not inspected will be excluded from future completion certification (PS4) to be issued by Soil & Rock Consultants.

In any event, where doubt exists regarding the quality of the founding material, we should be contacted for geotechnical advice.

End of Report Text - Appendices Follow

#### Appendices

Appendix A – Producer Statement Advisory Note

Appendix B – Machine Borehole Logs

Appendix C – Machine Borehole core photos

Appendix D – Drawings

# APPENDIX A

## Producer Statement Advisory Note



## **PRODUCER STATEMENT – CONSTRUCTION REVIEW (PS4)** **IMPORTANT ADVISORY NOTE**

The Building Consent Authority (BCA) frequently requires Producer Statements–Construction Review (PS4) to be submitted to the BCA in order for a Code of Compliance Certificate (CCC) to be issued. A PS4 is usually required for each specialist area. The requirement for a consultant to issue a PS4 related to their area of work will appear as a condition in the Building Consent documents.

It is the consent holder's responsibility to notify Soil & Rock Consultants for geotechnical construction inspections and testing required for subsequent issue of a PS4. An initial inspection of stripped or excavated ground must take place before any fill or blinding concrete is placed. Retrospective inspection of completed or partially completed geotechnical work is not possible and a PS4 will not be issued without all the required inspections.

In order to secure our inspection services and avoid delays on site, Soil & Rock Consultants require at least 24 hours' notice prior to the time the inspection is required. Our inspections are limited to items that have been recommended, designed and detailed by Soil & Rock Consultants. We are unable to inspect non-consented or unauthorised work. Soil & Rock Consultants do not carry out inspections or issue PS4's for work that has been recommended, designed or detailed by other consultants. Soil & Rock Consultants will not issue a PS4 where inspections and/or testing have been carried out by any other consultant. The PS4 must be sought from the consultant who carried out those inspections.

The full Building Consent, with stamped plans with consent numbers (or a legible copy of the same) including all amendments, shall be made available to us during inspections. We will not commence any inspections until the documentation is available or provided to us prior to our inspection.

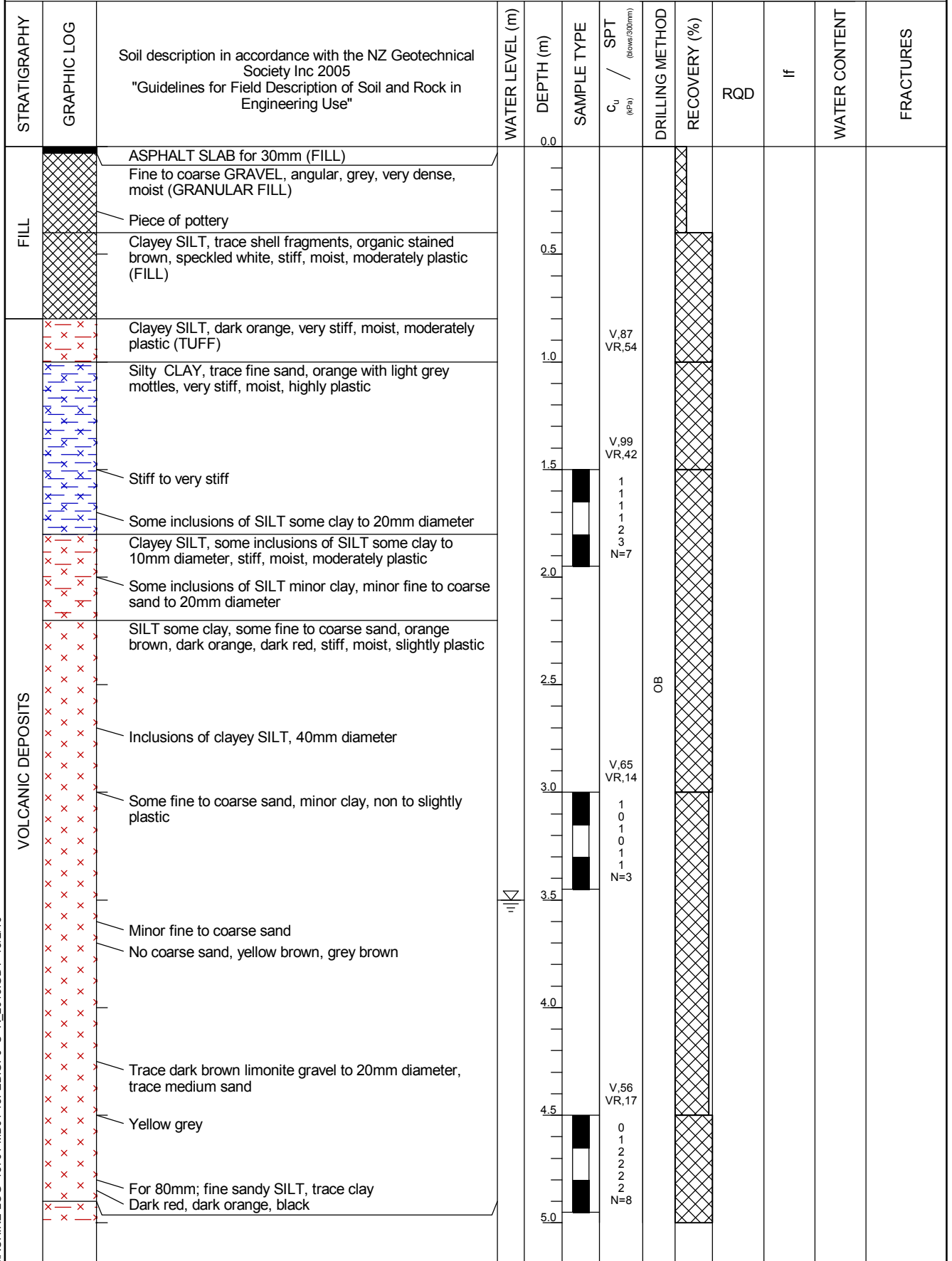
Unless stated otherwise in our terms of engagement, the fees associated with construction inspections and the issue of PS4's are separate from any work carried out prior to commencement of construction. We are able to provide a fee estimate for this work if required. We cannot provide a quote because the quantum of work required frequently depends on the construction program and the performance of others. These things are not known to us in advance of construction. Our normal terms of trade require payment of fees monthly during the inspection period and full settlement prior to release of any PS4.

## **APPENDIX B**

### **Machine Borehole Logs**

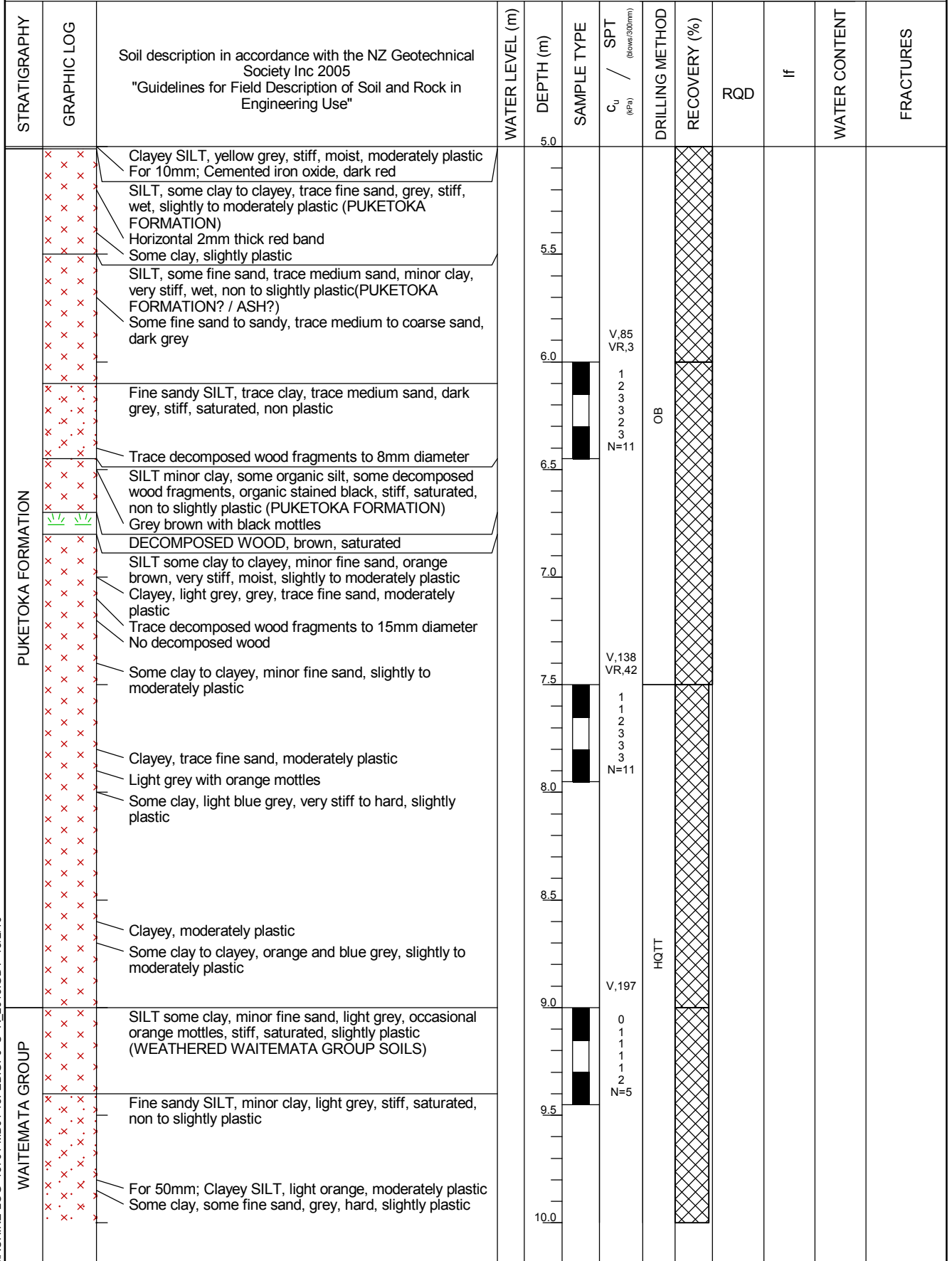


Drill Type: Rotary Machine Borehole Project No: 15761 Logged By: DEG  
 Drilled By: DCN Coordinates: Shear Vane No - Calibration Date: GEO122 - 17/11/2015  
 Date Started: 15/2/16 Ground Elevation: Surface Conditions: Asphalt, Near Level  
 Date Finished: 15/2/16 Water Level: 3.5m



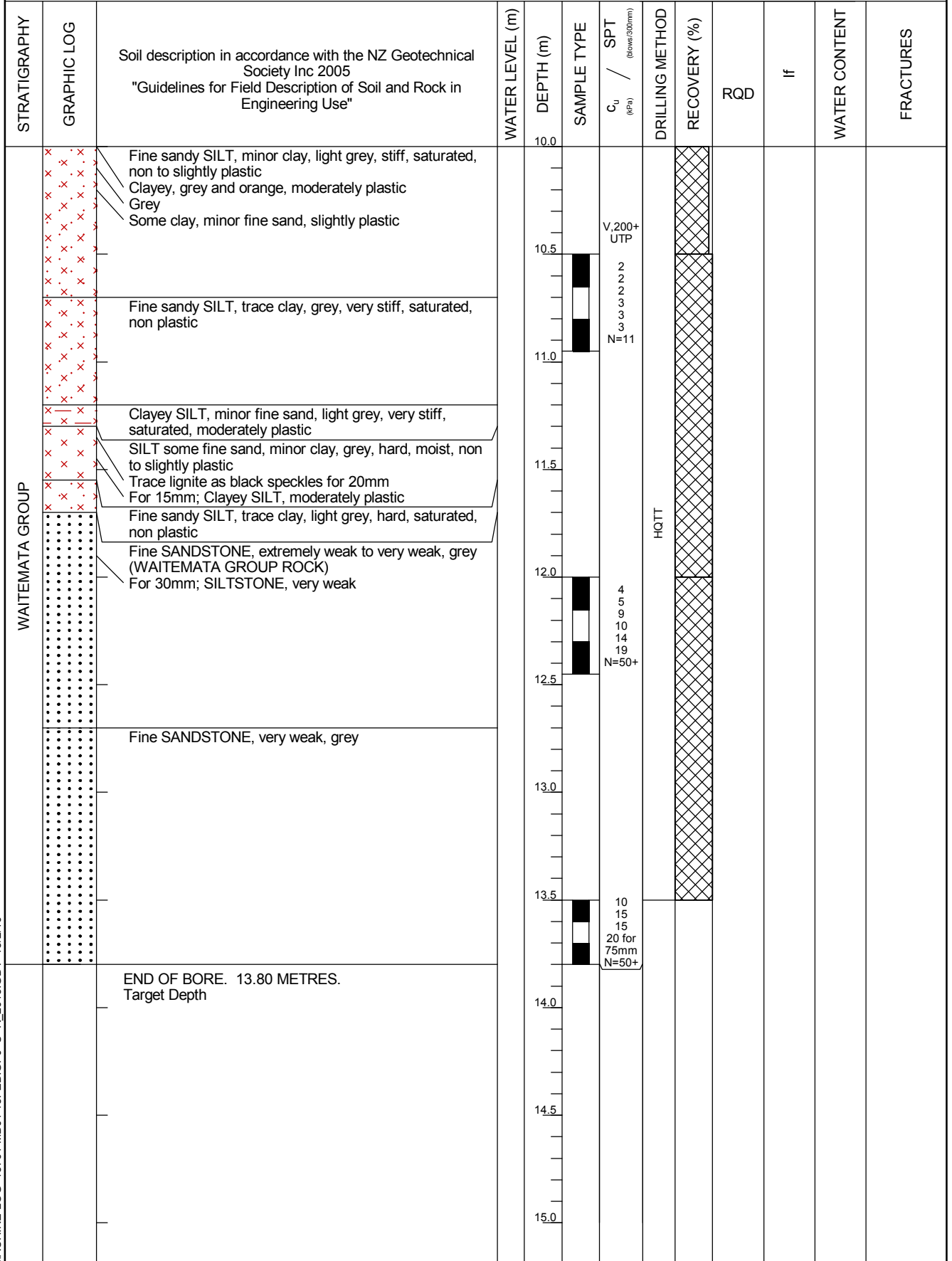
MACHINE LOG 15761 MB01 15FEB.GPJ S+R 2013.GDT 15/2/16

Drill Type: Rotary Machine Borehole Project No: 15761 Logged By: DEG  
 Drilled By: DCN Coordinates: Shear Vane No - Calibration Date: GEO122 - 17/11/2015  
 Date Started: 15/2/16 Ground Elevation: Surface Conditions: Asphalt, Near Level  
 Date Finished: 15/2/16 Water Level: 3.5m



MACHINE LOG: 15761 MB01 15FEB.GPJ S+R 2013.GDT 15/2/16

Drill Type: Rotary Machine Borehole Project No: 15761 Logged By: DEG  
 Drilled By: DCN Coordinates: Shear Vane No - Calibration Date: GEO122 - 17/11/2015  
 Date Started: 15/2/16 Ground Elevation: Surface Conditions: Asphalt, Near Level  
 Date Finished: 15/2/16 Water Level: 3.5m



MACHINE LOG: 15761 MB01 15FEB.GPJ S+R 2013.GDT 15/2/16

## **APPENDIX C**

### **Machine Borehole core photos**

# APPENDIX D

## Drawings





**Locality Plan**  
Scale: N.T.S.

**NOTES:**

1. Locations of features approximate only
2. Location of all buried services to be verified prior to construction
3. Boundary information on this Site plan adapted from information on Auckland Council GIS
4. Soil & Rock Consultants Site plan adapted from Site Plan sheet RC05 dated 20.09.2015 by Pattersons

**LEGEND:**

-  Site Boundary
-  MB01  
Approximate Cross Section Locations, Soil and Rock Consultants, February 9, 2016

AMENDMENTS		
DATE	REV	DESCRIPTION

Check all dimensions and levels on site before commencing construction.  
This drawing and design remains the property of Geotechnical Engineering Ltd. and may not be reproduced without the written permission of Geotechnical Engineering Ltd.

**Soil&Rock Consultants**  
*For well-grounded solutions*

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**401 - 403 PARNELL ROAD  
PARNELL**

**SITE PLAN**

15761/1	DRAWN: C Wang	DATE: 16.02.2016
SCALES: 1: 200 AT A3	CHECKED: S BERRY	DESIGNED:
Filename: s:\15-701-800\15761\cad\15761-1	REV.	