Appendix 3

Geotechnical Assessment

Bayswater Vehicles Ltd

93 Carlyle Street, Napier

Cut Slope Design Report

17367
28 March 2019

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Bayswater Vehicles Ltd

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Cut Slope Design Report

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TABLE OF CONTENTS

Appendices –

1. INTRODUCTION

Cheal Consultants Ltd (Cheal) have been engaged by Rob Townshend on behalf of Bayswater Vehicles Ltd (Client) to carry out geotechnical investigations and design for earthworks associated with the proposed redevelopment of the sales court on the corner of Carlyle Street and Faraday Street (Figure 1).

The development will include a series of showrooms and a large 20 bay workshop with a mezzanine floor. The architectural design for the overall development has been carried out by Jeffrey Goodall, and a set of architectural plans (Proj. No: 4315 Sheets: SK3A – SK3G) are included in Appendix 1. Please note, these plans were circulated around the project team on 15 January 2019 and may not represent the final design.

The site is located on gently sloping ground that extends from the toe of Napier Hill (Scinde Island) to Carlyle Street (area shaded blue on Figure 1). However, to accommodate some of the new buildings, part of the existing hill slope will need to be excavated to extend the current building platform(s) and a small amount of fill will be required. The maximum height of the proposed cut profile is expected to be 27m (approx.) and the volume will be in the order of 22,500m^3. The fill will be wedge shaped with a maximum depth of 1.5m and a volume of 800m^3.

It is estimated that the earthworks will take approximately 12 weeks to complete depending on the Contractors methodology.

The intention is to carry out all earthworks within the client's legal boundary, and the existing slopes that will be modified by earthworks have been indicated on Figure 1 by a thick blue line. Note, this line is approximate only.

The aim of this report is to document the investigation, assessment and final design advice for the cut batter. The objective is to support application for resource consent.

2. DESKTOP STUDY

2.1 Geological Map

The main geology of the Napier Hill is shown on the New Zealand 1:250,000 Geological Map (GNS – ArcReader) as the Scinde Island Formation (Mangaheia Group). This formation is late Pliocene (2Ma yrs B.P.) in age and comprises calcareous, cross-bedded sandstone and limestone. No structural geological features are shown associated with the Napier Hill, but the Napier Fault runs northeast – southwest and cuts through the Ahuriri inlet approximately 700m to the northwest of the hill (opposite side of hill to the site). This fault is described as inactive (>250,000yr AEP) and is concealed.

Figure 1: Location Plan – Bayswater Vehicles

2.2 Masters Thesis – Scinde Island Formation

The Scinde Island Formation was subdivided into 5 separate members by Boyle (1987)[1], and labelled A to E in terms of decreasing age (i.e. A = oldest, E = youngest). Figure 2 below, is copied from Boyle's thesis (Figure 6.6) and is a graphical representation of the 5 separate parts of the formation. Information from tables in the thesis outlining the composition (i.e. proportion of calcium carbonate, sand and mud – Table 6.1) and mean grain size (Table 3.2) from different samples taken by Boyle. Note, the grain size in the original table is expressed in terms of Phi (Φ). These have been converted to diameter (mm). The samples represent different "Facies", which is a geological term that describes a type of depositional environment, rather than just the rock type itself. The following is a brief summary of each member based on information from the thesis.

[1] Boyle S. F., 1987. Master's Thesis (M.Sc Earth Science); Scinde Island Formation, Napier. University of Waikato.

The Scinde Island Formation is thought to have been deposited at a time when there was an open seaway between Hawke's Bay and Whanganui, and the oldest member (A) comprises uniformly graded (well sorted) coarse sands with some silt and clay. The sands are partly formed from skeletal remains of shellfish, and calcium carbonate ($CaCO_3$) level are typical over 50%. This material has a large-scale cross-bedded structure, which was formed due to the migration of large sand waves (linear sand ridges) under high strength tidal currents in water depths up to 40m (mid to inner shelf depth). The individual cross-bedding ranges from 0.5m thick (with average wavelengths of 5.5m) to 15m thick.

Member B is finer grained and described as massive sandy siltstone. This material would have been deposited in deeper water (outer shelf depths), below the normal wave base where the depositional environment was quieter and there was less carbonate sources (shellfish). Typically the siltstone contains less than 30% $CaCO_3$.

There is an unconformity (time gap where material was eroded/removed) between Member B and the overlying Member C. This member is dominated by cemented skeletal debris with minor amounts of intact shells (70 – 80% $CaCO_3$), and minor amounts medium to coarse sized sands. The base of this member contains 4 (minimum) repeated cycles where shelly sandstone is initial medium to coarse, then grades to fine silty sand. Each repetition is 0.2m to 0.3m thick before grading into a band of finely laminated (stratified) sandstone and sandy mudstone. The upper part of Member C contains a well-graded pebbly greywacke conglomerate.

Member D is characterised by a thick sequence of bedded sandstone and mudstone. These display wavy (or lenticular) bedding and the sandstone includes ripples and cross-lamination on a 2cm scale. Typically, the silt content increases towards the top of the member and this material represents deposition in a shallow/tidal flats environment. At the top of Member D is a 10m thick band of pumiceous sandstone which was most likely deposited when the site was above sea level by fluvial (river) means.

Member E represents a relative increase in sea – level and is a barnacle-rick limestone with minor amounts of sand content. This material would have been deposited in shallow intertidal conditions.

Boyle logged the cliff face overlooking the Napier Port (Bluff Hill) and indicates this section comprises mostly rock from Member A, with a part of Member B exposed at the top (Figure 3). Pandora Point (Corunna Bay) is located at the opposite end of the hill and Members B - E are exposed, indicating that the north-eastern end of the hill has been uplifted more than the southwestern end, and at Bluff Hill the younger members have been eroded off.

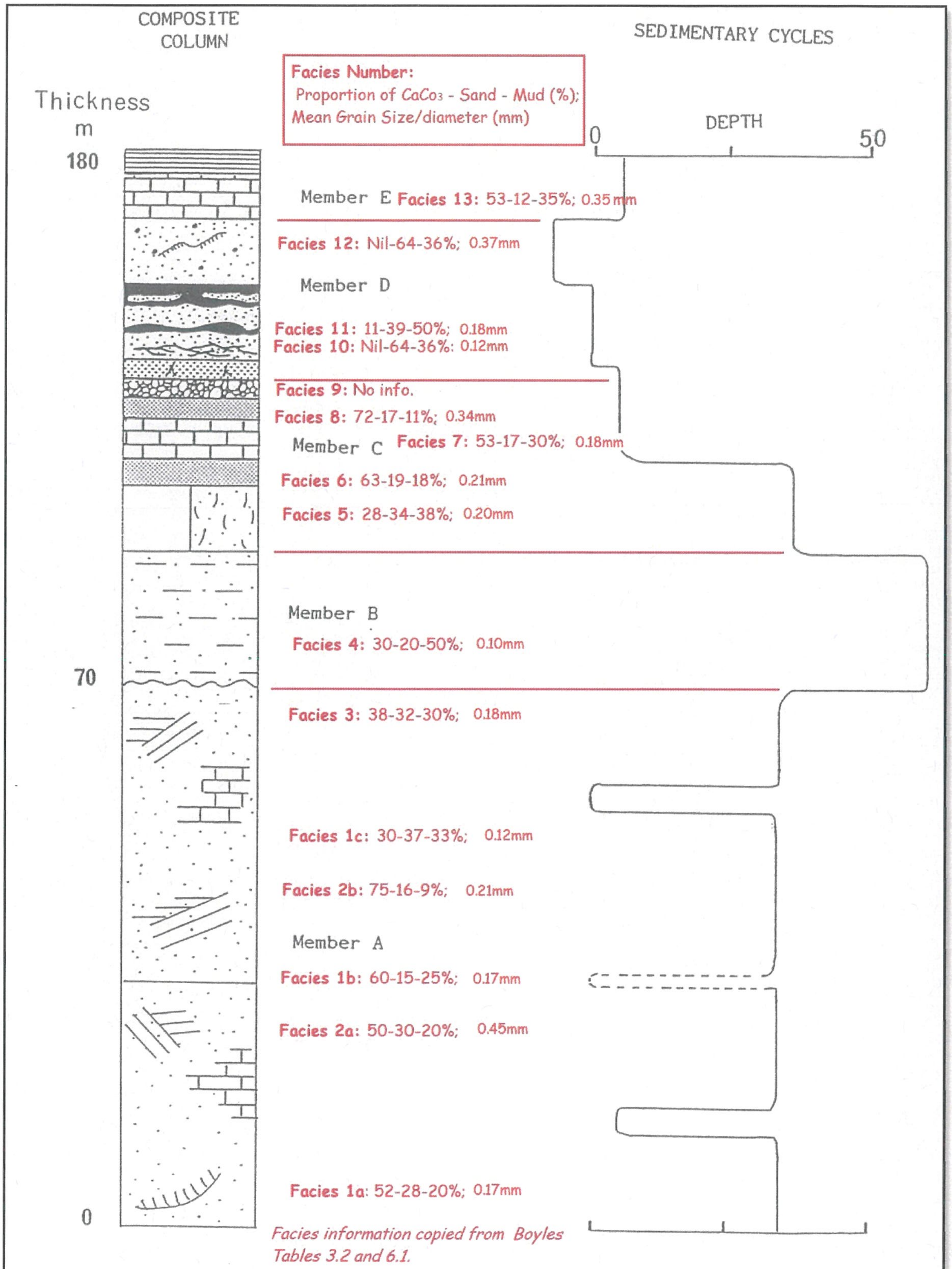

Figure 6.6 Composite stratigraphic column for the Scinde Island Formation showing a cyclic sedimentary sequence relative to changes in water depth.

Figure 2: Graphical representation of Scinde Island Formation (after Boyle 1987)

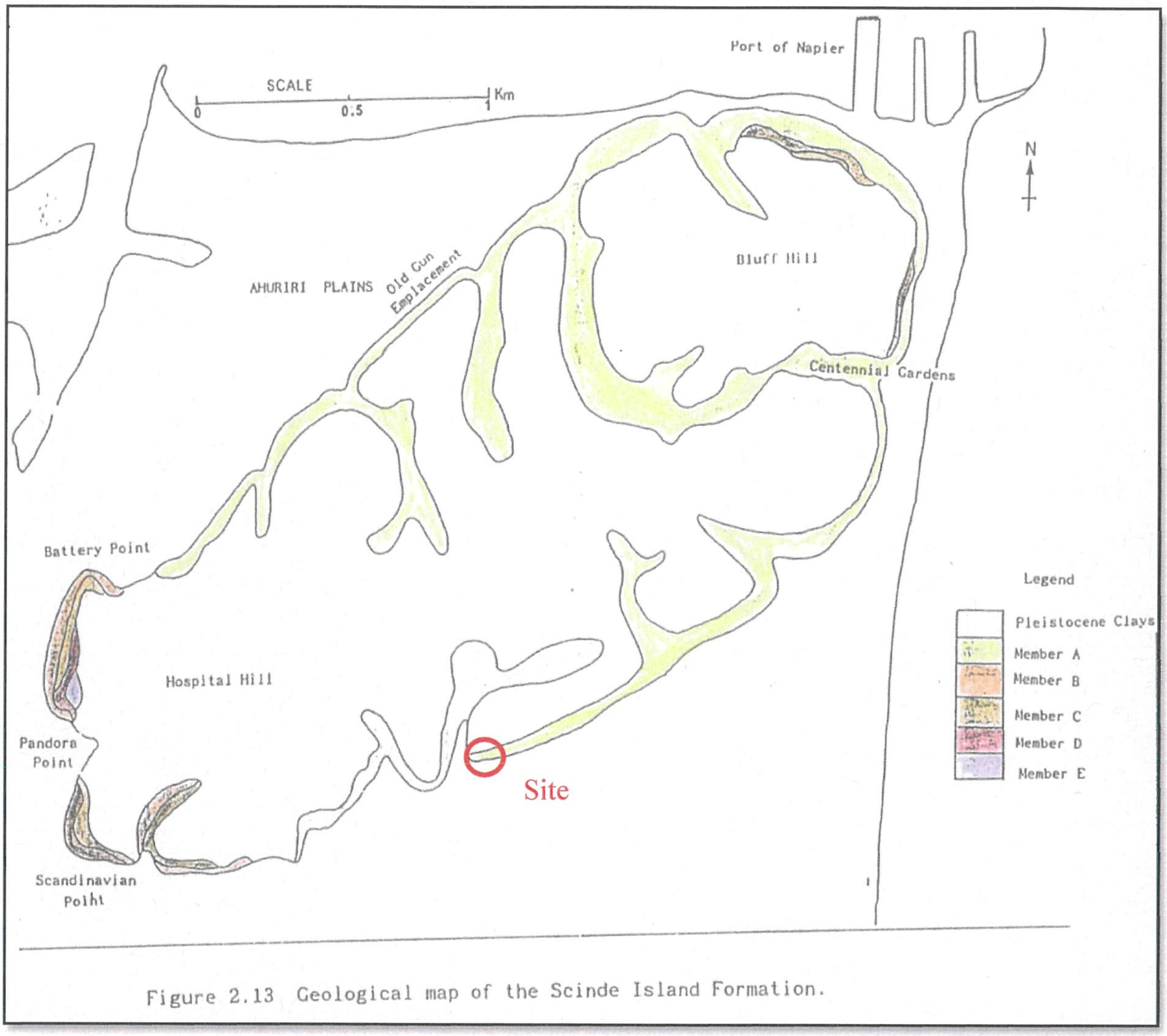

Figure 3: Plan of Napier Hill showing distribution of 5 Separate Members (after Boyle 1987)

Boyle's thesis focuses on the Scinde Island Formation. However, the sequence of limestones and sandstone described in the 5 member are capped by a thick layer of soil that was deposited relatively recently (Pleistocene; <50,000yrs B.P.). This unit is mostly Loess, and includes various pumice and ash deposits erupted from the Taupo Volcanic Zone. Figure 4 below is a copy of a diagram from Boyle's thesis (Figure 2.12) that provides the main sub division of the soils and indicates that in places this material can be up to 10m thick.

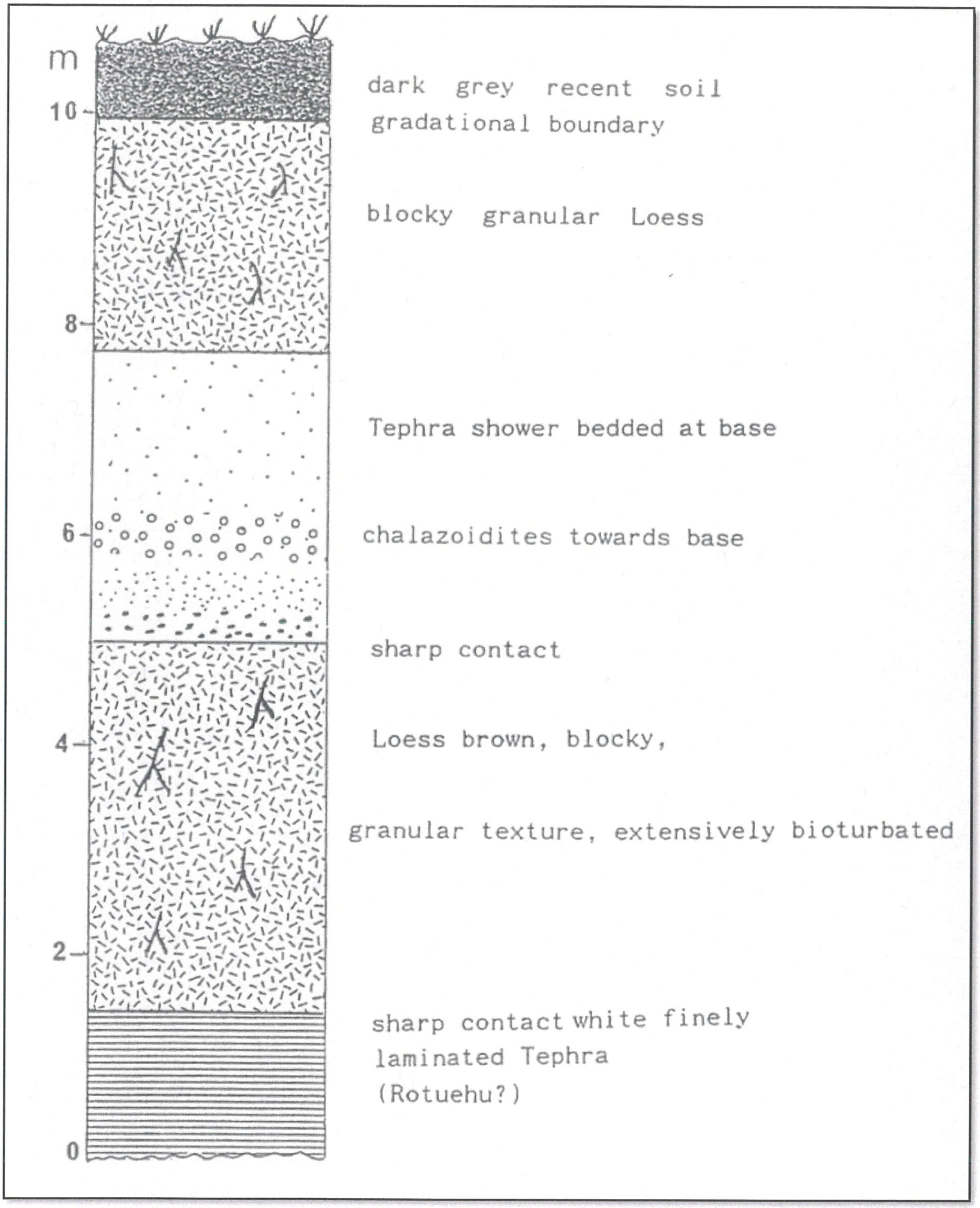

Figure 4: Typical Section of Loess (Pleistocene cover beds – after Boyle 1987)

3. HISTORIC INVESTIGATION

The geotechnical engineer has been involved in (or is aware of) a number of geotechnical investigations at sites around the Napier Hill. The following is a summary of factual data that has been collected as part of these.

3.1 Napier Girls High School

Geotechnical investigations were carried out by the Ministry of Works for development of Napier Girls High School (NGHS) in 1972. Two areas were investigated. These were:

1. Hostel and Classroom Blocks located on Clyde Road and above Cory Ave (November 1972)
2. Audio-Visual Block located below Clyde Road (August 1972)

3.1.1 Hostel and Classroom Block

The November 1972 investigation included 2 mechanically drilled, fully core Bore Holes (BH-A & BH-B). Standard Penetration Tests (SPT) were carried out in the BHs to measure the strength of the ground, and samples of the core were taken for laboratory testing.

This testing included:
1. 1 x Particle Size Distribution (Sieve Analysis)
2. 2 x Unconfined Compression Strength (UCS) Tests
3. 2 x Unconsolidated Undrained Triaxle Tests
4. 2 x Consolidated, Undrained Triaxle Tests

Copies of the BH logs, lab test reports (Table and Graphs) and a sketched plan showing the BH locations is included in Appendix 2. Please note, the BH's were originally logged in imperial units, but these have been converted to S.I. units for the key depths by the geotechnical engineer.

The BH logs indicate that at NGHS there is approximately 10m of Loess overlying limestone rock. The upper portion of the Loess is described as sandy (or including a trace of sand), and in both logs a band of pumice is recorded at 1.5m – 1.80m (approx.). The symbols used in the graphic column indicate that below the pumice layer the Loess is entirely silt with no sand. The results from a sieve analysis are included in Appendix 2, however this report doesn't identify the BH or sample depth, but indicates the sample contains approximately 45% sand, 36% silt and 19% clay.

SPT N-Values in the Loess range from 10 – 59, but are typically 15 – 31 indicating this soil is stiff – very stiff. There are simplistic correlations between SPT and Undrained Cohesion (C_u) and typically for a low plastic clay this would be 5 - 8 times the N-Values. Based on the lower values above, C_u = 75kPa – 155kPa.

A UCS was carried out on core from each of the BH's at a depth of 2.29m (8 ft). These are reported in Pounds per Square Foot (p.s.f) and the test carried out in BH-A converts to 1.6MPa. The test in BH-B is reported as 2,604 p.s.f, which converts to 0.12MPa.

The Unconsolidated Undrained (UU) Triaxle tests are recorded in tabular form (Table 1) and on a graph labelled FIG.2 in Appendix 2. The 2 test points show an Undrained Shear Strength (S_U) of 85kPa and 152kPa.

The C_U values derived from the SPTs and UU Triaxle test results are in good agreement and indicate a range of undrained strengths from 75kPa to 152kPa.

Effective Stress parameters were measured using Consolidated, Undrained (CU) Triaxial testing and plots are shown on FIG.3 and FIG.4 in Appendix 2. These plots indicate that for the Loess the Friction Angle (ø') is likely to be 35.8° to 37.3° at low confining pressures (<150kPa). As the Loess is typically limited to 10m thick this would generally be the case and under these conditions the plots indicate the Loess has Cohesion (c') of 62kPa to 85kPa.

The BH's reached a maximum depth of approximately 13.7m, and each hole extended about 3m into the limestone. SPT's carried out through this zone (5) all refused after 2 – 3 inches of penetration, and if the maximum N-Value (50) was extrapolated to the full 12 inches of penetration this would indicating the rock is very weak with an C_U of 1,200kPa. Assuming UCS is equivalent to 2 x C_U then this indicates a UCS of 2.4MPa.

3.1.2 Audio-Visual Block

The August 1972 investigation carried out for the Audio-Visual Class Room Block comprised 5 fully core BH's with SPTs (BH 1 - 5) and laboratory UCS testing. A location plan and BH logs are included in Appendix 2.

These BH's were drilled to a maximum of 15.2m depth, but unlike BH-A & BH-B these holes encountered limestone at a shallow depth (3.2m max.) and the logs and laboratory testing is almost entirely of limestone. The logs indicate the rock is fractured and contains some bands of siltstone and minor amounts of sandstone.

As with the limestone encountered at the base of BH-A & BH-B, the SPT's in BH 1 – 5 all reached refused (N = 50). A total of 5 UCS's were carried out on core from BH 1 with an additional test from BH 2. The UCS's in BH 1 ranged from 0.86MPa to 3.4MPa and generally got stronger with depth. The UCS from BH 2 is 0.71MPa and was carried out at a similar depth to the first (and lowest) test from BH 1 (0.86MPa).

Based on the mapping carried out by Boyle (Figure 3), it is most likely the Limestone encountered at NGHS is part of Member A of the Scinde Island Formation.

3.2 12 Kowhai Road

The geotechnical engineer investigated a site at the top end of Kowhai Road in April 2010 and a sketched profile of Loess exposed in a free standing bank is included in Appendix 2. The bank was approximately 1m high and standing at 80° (measured), and the soil profile was divided into 4 separate units (A – D). Unit A was topsoil, but below this the Loess was described as light yellowish brown, silty fine Sand. It was noted that the lower part of the unit contained a high pumice content, and at the base it was separated from the next unit (Unit C) thin layer of hard (very fine grained siliceous material, which is possibly an ash deposit. Unit C was a Silt/Sand mix and as above it was separated from the unit below by a thin siliceous layer. The lowest unit (Unit D) was similar to Unit B, although with slightly less silt content (fine Sand some silt).

The whole sequence was relatively dry and described as dense.

3.3 Simla Terrace

Photos 1, 2 & 3 were taken of a road cutting on Simla Terrace, opposite the start of Kavanagh Road (Hospital Hill), and shows a typical exposure of the Pleistocene aged Loess. This face is approximately 2.5m - 3m high and is standing sub-vertical. The material shown in Photo 2 has a weak columnar structure formed from vertical jointing. This is characteristic of the Loess seen in Napier and is likely to correspond to the "blocky" material described in Boyle's section of the Pleistocene cover beds (Figure 4).

At a finer scale, the Loess structure typically include a fine porous texture, and in Photo 3 a thin white band of very fine grained material can be seen running across the face. This is likely to be the same feature as described as siliceous material in Section 3.2.

Photo 1: Loess exposed in near vertical cut batter

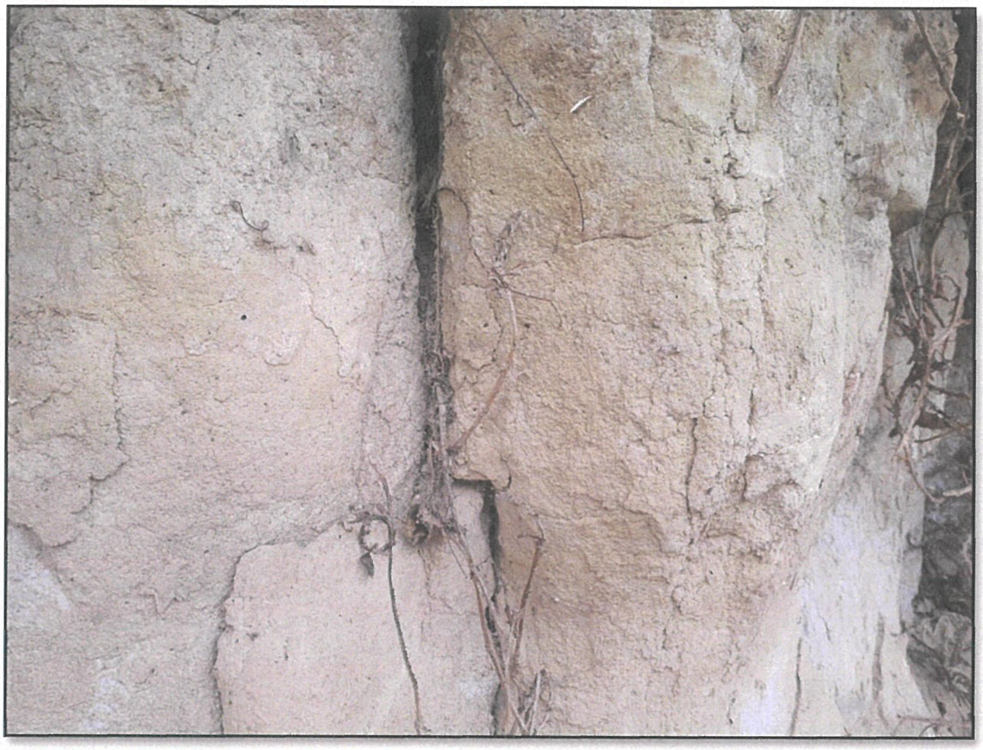

Photo 2: Natural jointing in Loess forming weak columnar structure

Photo 3: Typical exposure of Loess including a thin sub horizontal siliceous layer

3.4 Corunna Bay

The geotechnical engineer investigated a site at the corner of Corunna Bay and Hyderabad Road (SH 2/50) in August 2013. At the time the client was proposing to trim back the existing limestone to form a 10m high cut batter and as part of the investigation a total of 5 core samples were taken from the base of the existing slope and these were tested in a laboratory for UCS. The strengths of the rock samples ranged from 0.65MPa to 1.32MPa. A copy of the test report is included in Appendix 2.

In relation to Boyle's Geological Map of the Scinde Island Formation (Figure 3) the core would have been taken from the areas labelled as Scandinavian Point and the map indicates the rock at the base of the hill slope would be would belong to Member B, which is one of finer grained sandy siltstone units.

The test results from the Corunna Bay site are consistent with the UCS values measured on core from the NGHS BH's, which is most likely rock from Member A and indicates a degree of consistency in strength throughout the wider Scinde Island Formation.

4. SITE SPECIFIC INVESTIGATION

4.1 Field Investigation

One fully cored BH was carried out at the site (BH01). The hole was drilled by Drillcore Ltd using a tractor mounted drilling rig and triple-tube wire-line coring system with fresh water circulation (Photo 4). The hole was drilled from the top of the slope on ground next to 32 May Ave approximately 11.5m from edge of road (see red circle – Figure 1), and was drilled at an angle of 65° from the horizontal in a direction of 162° (south southeast). The orientation of the hole was set so as to be sub-parallel with the proposed cut profile. A ground profile through the line of the BH and including a sketched preliminary cut slope design is included as Figure 5.

A small working platform was constructed to support the drill rig at a Reduced Level (RL) of 45.5m, and the hole extended 40m in length to a final depth at RL9.25m (an equivalent vertical depth of 36.25m).

The BH was supervised and the core logged by the geotechnical engineer in general accordance with the New Zealand Geotechnical Society (NZGS) Guidelines on the Field Description of Rock and Soil (Dec. 2005). The BH log and core box photos are included in Appendix 3.

The scale on the log is in terms of BH length (m), although incremental RL's are included next to the graphic column. In addition, a graph relating BH length to depth is included.

Photo 4: Drilling Rig set up at top of existing hill slope

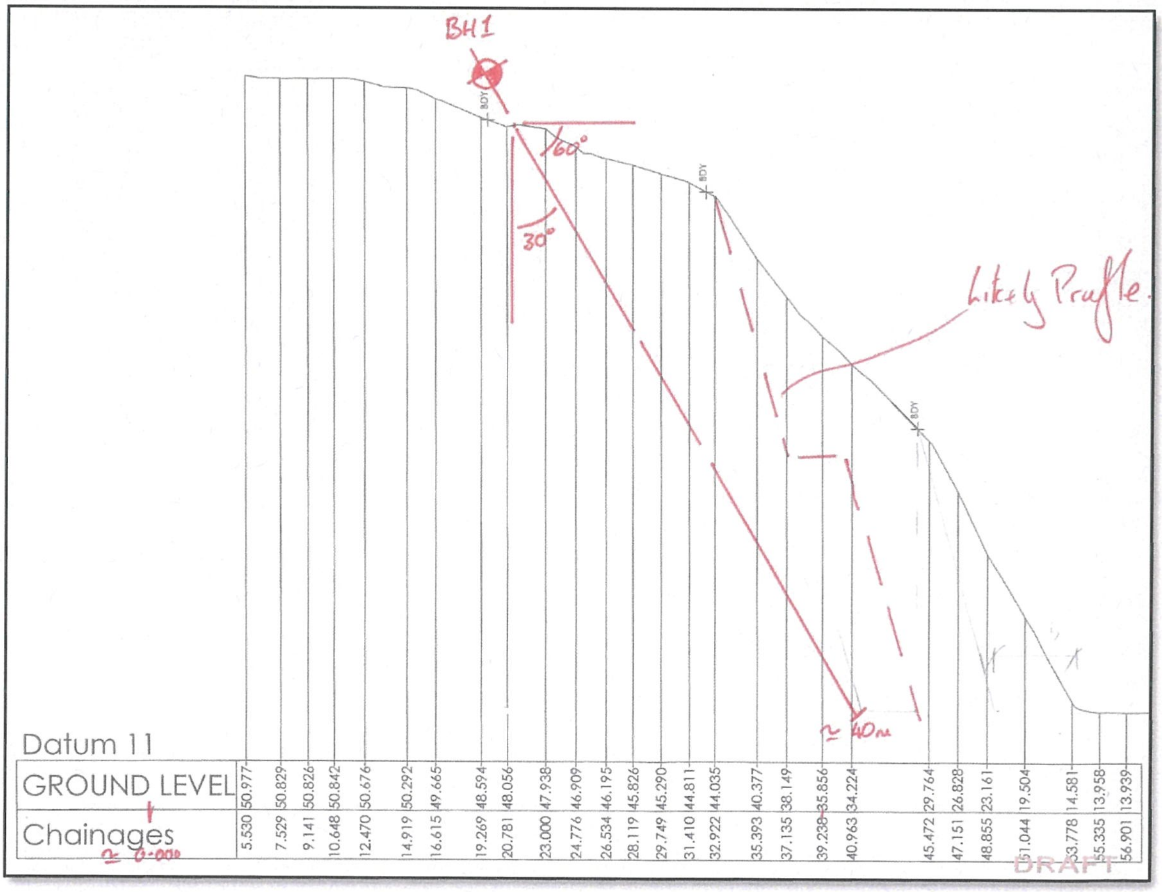

Figure 5: Slope profile with line of BH and preliminary cut profile

4.2 Laboratory Testing

A total of 32 samples were selected from the core following drilling and these were tested for Point Load strength by the geotechnical engineer using the Opus Laboratory (Napier) facilities. Size-corrected point load strength index was determined ($I_{s(50)}$) following the method outlined in ASTM D 5731-95 *Standard test method for Determination of the Point Load Strength Index of Rock (ATMS)*.

The $I_{s(50)}$ values were converted to UCS (MPa) using the formula $\delta_{UC} = C\ I_{s(50)}$ obtained from ASTM, where δ_{UC} = UCS (MPa) and C = factor that depends on site-specific correlation between δ_{UC} and $I_{s(50)}$. The adopted C value was taken from ASTM Table 1 (page 6) which states that for a core size of 60mm a generalised value of 24.5 shall be used for C.

Test results along with critical dimensions and the orientation of the sample (Diametral, Axial or Irregular) are included in a datasheet in Appendix 4. Included are photos of each of the core samples. A total of 42 tests were carried out and the results are summarized in Table 1.

Table 1: Summary of Point Load Testing

Statistical Values		
Parameter	$I_{s(50)}$ (MPa)	UCS (MPa)
Minimum	0.01	0.2
Maximum	3.08	75.6
Median	0.19	4.7

Table 2: UCS Range from Point Load Testing

UCS Ranges	Number of Values	Proportion of Data Set
<1MPa	6	14%
1 – 5 MPa	16	38%
5 – 20 MPa	12	29%
20 – 50 MPa	8	19%

Table 2 includes 4 different UCS ranges that coincide with the Rock Strength Terms outlined in the NZGS Guidelines for Field Description of Soil and Rock and the number and proportion of the test set that fall within each range. These categories correspond to a colour formatting in the test sheet included in Appendix 4.

5. GEOTECHNICAL UNITS

The rock profile has been subdivided into 6 individual geotechnical units based on the BH log and Point load (PL) test results. These Units are listed in Table 3 along with soil strength parameters that have been used for the stability modelling. A typical description of each unit is outlined in the following section. The 6 geotechnical units have been defined in terms of their expected engineering behaviour and are not equivalent to the geological (Facies) subdivision carried out by Boyle (1987) outlined in Section 2.2.

We consider that over the extent of the Bayswater site the geotechnical units outlined below can be considered horizontal and are continuous. The strength parameters included in Table 3 are based on the lower range values derived from testing, and we consider that these strength values are conservative.

Table 3: Geotechnical Units and Strength Parameters

Unit	Description	Thickness (m)	Composite UCS (MPa)	Mohr-Coulomb Parameters (Effective Stress)		
				Υ' (kN/m³)	ø' (°)	c' (kPa)
1	Loess	6.4	1.6	16	38	15
2	Weak Limestone	8.3	4.9	18	40	147
3	Mod. strong Limestone	4.7	6.9	20	42	169
4	Poorly Cemented Sandstone	3.9	0.5	20	38	56
5	Very weak Limestone	10.7	2.1	22	42	100
6	Weak Limestone	2.2+	5.5	18	40	154

The subdivision of the geotechnical units and the final strength parameters were determined based on a combination of the recovery of the drilled core and the distribution and magnitude of the UCS test results as summarised in Table 1 and Table 2. In terms of recovery the following measurements were used to qualitatively estimate the degree of cementation, and strength of the localised rock mass:

- Total Core Recovery (TCR%)
- Solid Core Recovery (SCR%)
- Rock Quality Designation (RQD%)

Table 4 outlines a site specific criteria that was developed and used as a guide for the subdivision of the geotechnical units.

Table 4: Criterial for Cementation

Core Measurements	TCR (%)	SCR(%)	RQD(%)
Poorly Cemented	<25	<25	<25
Cemented	>25	>25	<25
Well Cemented	>50	>25	>25

The UCS ranges included in Table 2 correspond to:

- Extremely Weak (<1MPa)
- Very Weak (1 - 5MPa)
- Weak (5 - 20MPa)
- Moderately Strong (>20MPa)

A hand drawn summary log which graphs the TCR, SCR and RQD, and the individual UCS values at their correct depth is included in Appendix 4. The final Composite UCS's included in Table 3 for each geotechnical unit was calculated by proportioning the range of actual UCS values in relation to the corresponding length of core represented by a consistent cementation criteria as outlined in Table 4.

5.1 Loess

The material seen in the BH was described as light yellowish brown silty Sand. Typically the sands are fine to medium sized and the unit is described as very dense. It was noted that parts of the core are weakly cemented with iron oxidation to form hard layers. The description is typical of the Loess described in Section 3.3, and testing from NGHS (Section 3.1) indicates the following soils strength parameters:

- UCS = 1.6MPa
- Effective stress parameters:
  - ø' = 35.8° - 37.3°
  - c' = 62kPa - 85kPa

We have used these values along with our experience with this material (and engineering judgement) to determine our final strength parameters outlined in Table 3. It should be noted that we have reduced the cohesion (c') to approximately one quarter (25%) of the lower end value to allow for future near surface weathering and possible variation within the soils.

5.2 Weak Limestone

Unit 2 represents rock described in the core as slightly - moderately weathered, interbedded Limestone. The bedding is moderately thick and typically comprises alternating bands of:

1. Uniformly graded, well cemented ($CaCO_3$) coarse Sand. The sand grains are rounded but also included are minor amounts of fine gravel sized shell fragments.
2. Poorly graded, weakly cemented fine - medium Sand. Sand grains are typically rounded to sub-angular.

Based on the description from the BH log it is likely this material corresponds with Member A, which is described by Boyle (1987) as *uniformly graded (well sorted) coarse sands with some silt and clay. The sands are partly formed from skeletal remains of shellfish, and calcium carbonate ($CaCO_3$) level are typical over 50%* (Section 2.2). Unit 2 is approximately 8.3m thick and drilling achieved just over 25% TCR, but typically less than 25% SCR with RQD (%) recorded for one drill-run only. Seven individual UCS tests were carried out and ranged from 8MPa - 27MPa.

Assuming all the core loss represented poorly cemented/extremely weak material it is determined that:

- 10% of the rock mass is weak
- 40% is very weak
- 50% is extremely weak

We consider that a composite UCS of 4.9MPa represents the rock mass of the unit as a whole. We have used this UCS and the website based version (RocLab) of the Hoek-Brown Classification to determine equivalent Mohr-Coulomb failure criteria parameters. Assuming:

1. GSI = 60
2. mi= 20
3. Disturbance factor = 0.7 (Cut slope)

The following parameter were calculated: ø'=35°; c'=220kPa and based on these the final strength parameters in Table 3 have been determined and are considered appropriate for design. Printouts from RocLab are included in Appendix 4 (Unit 2 – 6).

5.3 Moderately Strong Limestone

The rock described for Unit 2 continues through to Unit 3, and although the core recovery (TCR) is similar the SCR (%) increases to approximately 25% and RQD was measured throughout the length of the unit indicating more cementation of the rock mass. UCS test results were significantly stronger (23MPa to 38MPa) and assuming all the core loss represented poorly cemented/extremely weak material (as above) it was determined that:

- 25% of the rock mass is moderately strong
- 15% is very weak
- 60% is extremely weak

We consider that a composite UCS of 6.9MPa is appropriate for Unit 3 and using RocLab the Hoek-Brown parameters are: ø'=38°; c'=252kPa. As above we have adjusted these and consider that the values in Table 3 are appropriate for design.

5.4 Poorly Cemented Sandstone

Unit 4 is approximately 4.5m thick, and over a single drill run (1.5m) almost no core was recovered. Although TCR (%) was relatively good over the runs above and below there was no significant SCR (%) or RQD (%) recorded, indicating the rock is either relatively weakly cemented or highly fractured. Only 2 individual UCS tests were carried out and these were 4.7MPa and 9.3MPa respectively.

Assuming most of the rock in Unit 4 is poorly cemented/extremely weak a composite UCS of 0.5MPa was used and Hoek-Brown parameters of ø'=19° and c'=86kPa were calculated. It is considered that this friction angle (ø') is underestimated for a sandy material and we have increased this while distressing the cohesion (c') for the final values in Table 3.

5.5 Very Weak Limestone

Core recovery through Unit 5 was very good with greater than 75% TCR achieved with for approximately half the length of the Unit. SCR (%) is mostly above 25%, and RQD of 25% was achieved for over half of the unit. However, although a large number of UCS tests were carried out (15) these were mostly less than 5MPa. This indicates most of the rock (80%) is very weak, and assuming all the core loss represents poorly cemented/extremely weak material (20%) a composite UCS of 2.1MPa was calculated for the unit.
Using the composite UCS the Hoek-Brown parameters are ø'=29° and c'=156kPa. As above, it is considered that this friction angle (ø') is underestimated and we have adjusted this up while adjusting the cohesion (c') down in Table 3.

5.6 Weak Limestone

This unit represents the last 2m at the base of the BH, and the recovery and quality of the core is very similar to that described for Unit 2 (Weak Limestone). Only 4 UCS tests were carried out (3.3MPa, 3.8MPa, and 13MPa, 14MPa) indicating alternating layers of weakly cemented and well cemented sands as described in Section 5.2.

Considering the degree of core loss we consider that:

- 50% of the unit is extremely weak rock
- 10% is very weak
- 40% is weak

indicating a composite UCS of 5.5MPa and Hoek-Brown parameters of ø'=36° and c'=230kPa. We have adjusted these values and consider the final strength parameters in Table 3 are appropriate for design.

6. EARTHWORKS DESIGN

6.1 Cut Design

The development of the site has been designed by Jeffrey Goodall (Architect) and includes a series of showrooms and a large 20 bay workshop (Appendix 1). To accommodate the proposed buildings the toe of the existing hillside needs to be cut back in 3 areas to create sufficient building platform. We have determined that the existing hillside can be cut back and have designed a single cut batter of 0.25H:1.0V that will have a good level of stability and can be utilised in all areas of the site.

A *Draft* Earthworks Plan show the extent of the cut profiles is included in Appendix 5. The following sections outline the design criteria and stability modelling used to verify the cut design.

6.1.1 Design Criteria

Earthworks and geotechnical requirements are covered in Section 2 of NZS 4404:2010 *Land Development and Subdivision Infrastructure*. However, very little detail is provided and Section 2.3.4 *Stability Criteria* states that the "geo-professional shall use acceptable criteria and analysis methods", and that applicable criteria has been "published or recommended by the NZGS". The references included in NZS 4404:2010 is the EQC Research Project 95/183, (*Design of Permanent Slopes for Residential Building Development* - Crawford & Millar 1998), which was published by NZGS (June 1998), and recommends:

- A minimum Factor of Safety (FOS) of 1.5 for design conditions,
- A lesser minimum FOS of 1.2 under extreme conditions.

In addition, we have reviewed the New Zealand Transport Agency's (NZTA) Bridge Manual (SP/M/022) 3rd Ed. (2014), which includes a section on *Site Stability, Foundations, Earthworks and Retaining Walls* (Section 6). Section 6.4.2 covers *Design of Cuttings* and requires:

- A long-term FOS of 1.5,
- A FOS of 1.2 during construction,
- A FOS ≥ 1.0 for seismic events.

Table 2.3 of the Bridge Manual includes Importance Levels (IL) and Annual Probabilities of Exceedance (AEP) for earthquake loading of earth slopes and shows that only Ultimate Limit State (ULS) needs to be considered. For *Earth slopes providing protection to adjacent properties* but not having a post disaster function as listed in AS/NZS 1170.0:2002 *Structural Design Actions* the IL is 2, and the design earthquake has an AEP of 1/500. However, based on the size of the slope we are assuming a design earthquake with an AEP of 1/1,000 (equivalent to IL 3).

6.1.2 Seismic Loading

It is recommended that for slope stability the Unweighted Peak Ground Acceleration (PGA) for horizontal loading is calculated using the NZTA Bridge Manual; Section 6.2 *Design Loading and Analysis*. We have calculated a PGA of 0.38g based on:

- Site subclass is "B" (Rock), and factor (*f*) is1.0
- 1000yr return period PGA coefficient for Napier is 0.38
- 1/1,000 AEP has an R_u of 1.3 (IL 3)

Hand calculation outlining the determination of PGA using the NZTA Bridge Manual method are included in Appendix 6.

6.1.3 Global Stability Analysis

Global stability analysis has been carried out using the commercial software Slide 2018 (roc science) and methods of slices (i.e. Bishops Simplified, Janbu Simplified and GLE/Morgenstern-Price etc.). The existing ground profile was determined from Napier City Council's publicly available (GIS page) LiDAR and contour data and was supplemented by topographic survey carried out by Cheal in July 2018.

Geotechnical Units and strength parameters are as outlined in Section 5 and Table 3. We have not included ground water or seepage pressures in the stability model as our BH investigation did not encounter ground water, and observations at the site over time have not indicated any issues with localised springs or seepage. Figure 6 is the printout from Slide 2018 for the static model and shows that the FOS under normal long-term working conditions is 1.7. Under seismic conditions (Figure 7) the FOS reduces to 1.2.

Both these analyses show that the recommended criteria of 1.5 (Static) and >1.0 (seismic) as outlined in Section 6.1.1 are exceeded and we consider the cut design will provide a good level of security to the development and the properties at the top of the slope.

Larger copies of Figures 6 and 7, along with stability modelling of the existing hillside are included in Appendix 6.

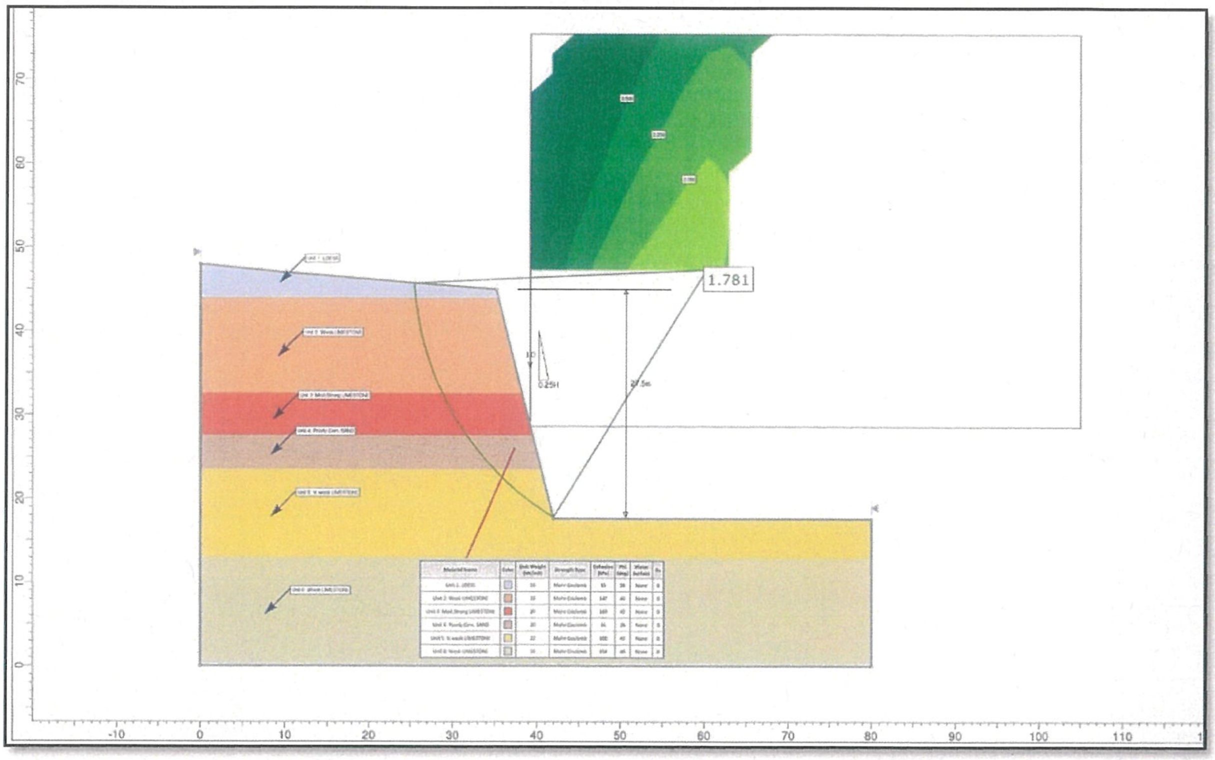

Figure 6: Slide 2018 Stability Model for Static Case

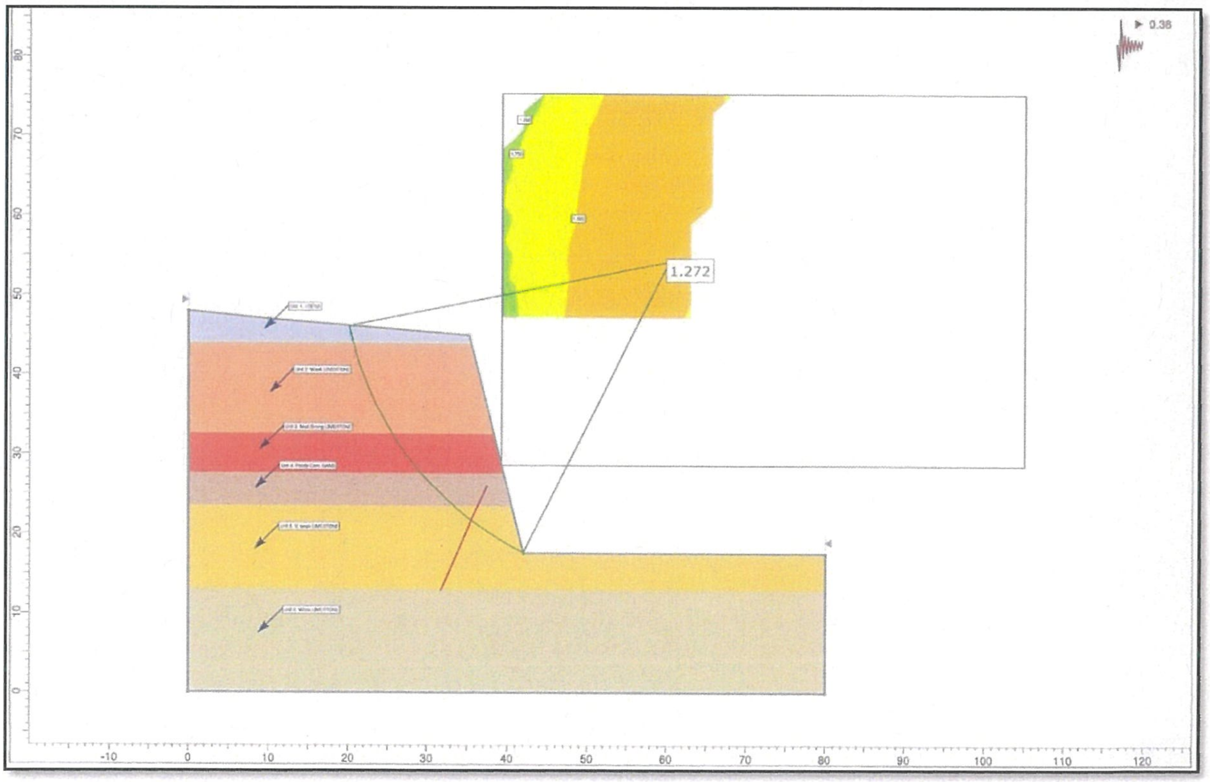

Figure 7: Slide 2018 Stability Model for Seismic Case (PGA=0.38g)

6.2 Precedent Based Verification

NZS 4431:1989 is a *Code of Practice for Earthfill for Residential Development*, and in Section 6.1.4.1 *Slope Stability* it states "in many cases it is impractical to measure quantitatively the FOS against shear failure, but the maximum slope of fills may be determined by the inspecting engineer by observation of slopes of fills which have a longstanding history of stability and which consists of similar material in similar geological groundwater and drainage conditions." Although this standard focuses on *Earthfill* it is considered that this advice is also applicable (if not more so) to cut slopes.

We have compared our cut design to existing slopes seen within the vicinity of the site, and in similar ground conditions. Many areas around the Napier Hill have been quarried in the past and these works have typically left relatively steep cut slopes.

Three examples have been presented. These are:

1. 15 Craven Terrace
2. Amner Place
3. 48 Burns Road

Based on the mapping by Boyle (Figure 3) the 3 examples are located in the Scinde Island Formation – Member A, same as the site. The cutting above Craven Terrace has the same aspect as the site and is located approximately 500m to the east (Photo 5). This cutting is standing sub-vertical with a height of approximately 20m, then supports a steep slope (>45°) which is a further 10m - 15m high.

Amner Place comes off the northern side of Milton Road and used to be the location of a series of large fuel tanks. The limestone face at the back of the platform (now housing) is a single continuous cut slope that extends up to Roslyn Road and is approximately 35m high (Photo 6).

Photo 7 shows an exposed rock face at the base of Burns Road. The face is sub-vertical and approximately 30m high.

In addition to the 3 examples presented there is the notable limestone bluff (Bluff Hill) that overlooks the Napier Port. This face is a natural feature and was mapped by Boyle (1986) as representing Member A, with part of Member B exposed at the top. The total height of the Bluff is approximately 100m, but there are sub-vertical sections of rock face that are at least 75m high (Photo 8).

Photo 5: Cut Slope above Craven Terrace

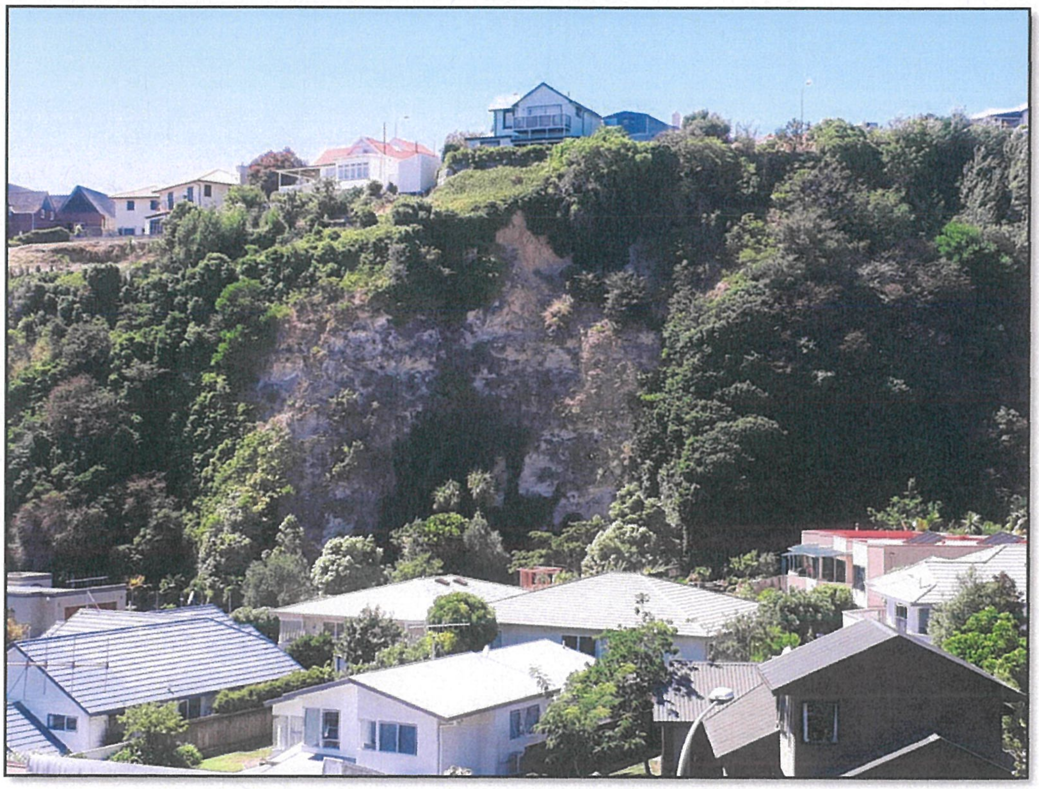

Photo 6: Cut Slope above Amner Place

Photo 7: Cut Slope above lower Burns Road

Photo 8: Natural Slopes of Bluff Hill (approx. 100m high)

6.3 Landscape Benching

The client has engaged a Landscape Architect to assess the visual impact of the proposed cut face (reported separately) and to mitigate the visual effects of the cut it has been agreed that a series of narrow benches will be formed into the upper part of the cut batter.

The final profile will include 3 separate benches spaced at 4m vertical intervals from the top of the slope. The main purpose of the benches is to enable planting of small shrubs and draping type plants. The width of each bench will be approximately 1m wide and will be formed and planted during the "top – down" construction of the cut profile. Once earthworks are complete it is not intended that there will be ongoing access to the benches. However, based on similar cut slopes constructed on the hill over the last 5 – 10 years it is anticipated that the weak limestone rock will fully vegetate itself over time, and the intention of the planting is simply to establish a level of vegetation cover in the short term that will encourage further vegetation establishment.

The landscape benching has been superimposed over the designed batter and the proposed slope profile is shown on the Cheal Earthworks plans (Appendix 6). Please note, the final bench profile may need to be adjusted once the current vegetation cover has been cleared and the existing ground surface verified.

In addition to allowing planting of the upper slope to occur, the landscape benching reduces the overall slope of the upper 12m of the cut profile to 0.35H:1.0V (approx. 70°). This will effectively increase the overall stability and FOS of the slope, as outlined in Section 6.1.3, and in particular the top of the slope and the neighbouring garage.

6.4 Slump Mitigation

We have carried out stability modelling to determine the global FOS of the proposed cut batter and this has achieved greater than 1.5 for the static long-term load case, and greater than 1 for the short-term seismic case. This fulfils the requirements for earthworks under NZS4404:2010 and supporting documents.

Although this analysis shows that the overall strength of the rock mass will resist large scale rotational type failure it does not consider the fretting which could occur at the scale of individual limestone (or sand) beds, which could result in slumping in the order of a few cubic meters of material, or the loss of individual boulders.

It is impossible, based on the current level of information, to predict where in the proposed cut profile such slumping could occur, or how frequently this may happen. Therefore 2 levels of mitigation are proposed.

6.4.1 *Structural Mitigation*

The issue of possible fretting has been outlined to the client and the architect designing the buildings, and it has been recommended that along the upper edges of the building(s) that are below the larger cut batters the top of the wall is extended above the roof and/or parking level to form a solid barrier similar to a "New Jersey/F-Barrier" type road safety barrier. The intention is that this barrier will stop debris from spilling onto the roof areas or across the upper parking space of the workshop.

6.4.2 *Slope Treatment*

It will be a requirement of the final cut slope design that the designer/geotechnical engineer monitors construction of all earthworks. It is anticipated that this will include visual inspection of the excavated face at regular intervals. The intention is that the designer/geotechnical engineer continuously checks the limestone rock mass being exposed against the descriptions and assumption made in the ground model (Section 5) and used in the stability modelling (Section 6.1.3).

Where there are parts of the rock mass that are significantly weaker/more friable, and it is considered that they represent an increased risk of future slumping then these localised part of the face will be covered with a combination of erosion control and face stabilisation matting. This mitigation will be directed by the designer/geotechnical engineer during construction.

A sketched detail illustrating the matting concept is included as Figure 6. Please note, these details have been used successfully by the designer/geotechnical engineer on a number of natural and engineered slope. However, this detail is only an example and will be modified for the final earthworks design.

6.5 Retaining Walls

A Number of retaining walls will be required as part of the development. In most cases these will form part of the building structure and based on the preliminary architectural plans (Appendix 1) retaining walls will be required along the northern side and the eastern end of the proposed workshop. The northern side of the structure will encroach into the toe of cut slope over a length of approximately 10m at the eastern end, and possibly up to 4.5m of the building will need to act as a retaining wall.

At the eastern end of the workshop there will be a ramp that will extend down from the upper parking level and across the west facing cut batter. The ramp will be in the order of 6m wide and formed onto the instu limestone. At its northern extent the ramp is expected to be 5.5m above the finished platform level and will reduce in height at a grade of 5% (approx.). The rock below the ramp will be cut vertical and will built up against by the end of the building.

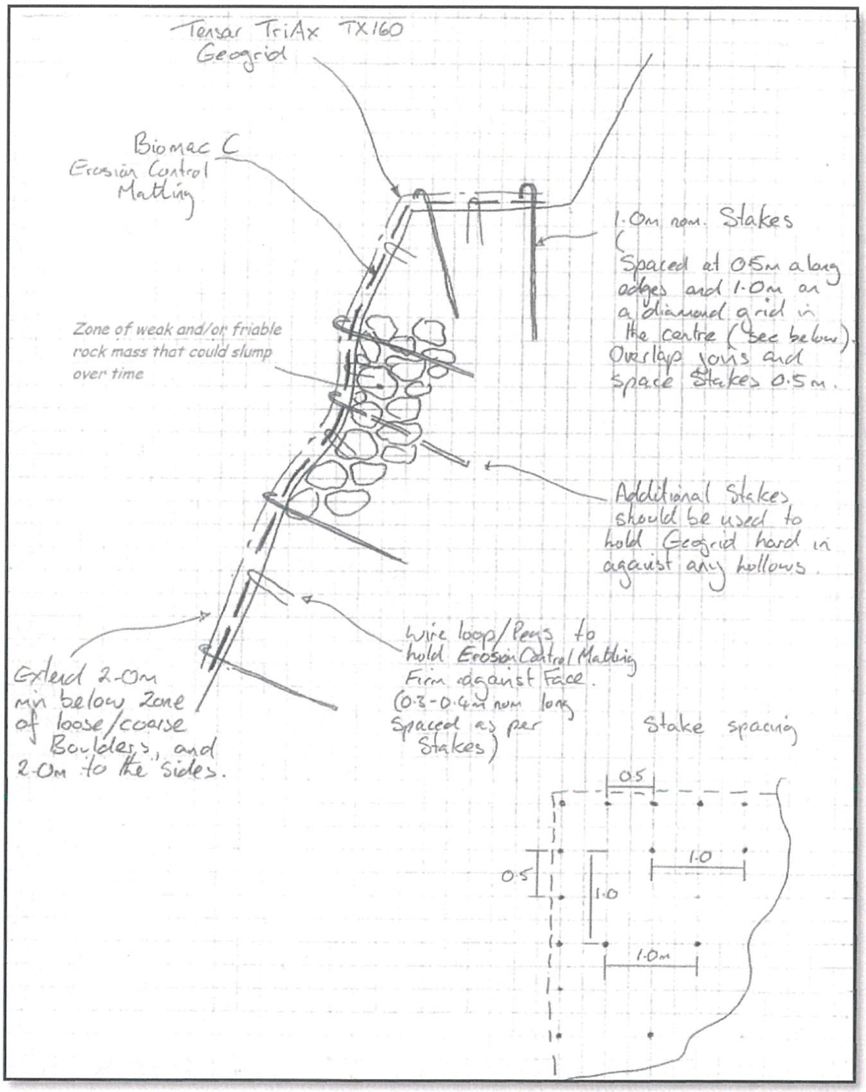

Figure 8: Face Stabilisation & Erosion Control Treatment

6.5.1 *Active Earth Pressure*

Assuming Rankine's theory of earth pressure the active pressure coefficient (K_a) for Unit 5 ($ø'=42°$; Table 3) is 0.20 under static conditions.

Seismic coefficients have been determined using Mononobe-Okabe, and under Serviceability Limit State (SLS1) the PGA for a 1/25 year AEP earthquake is 0.07g, and the $K_{a(SLS1)} = 0.24$. For the ULS earthquake the $K_{a(ULS)} = 0.57$ based on a PGA of 0.38g.

We have carried out preliminary calculation for a 5.5m high wall (H) built into the toe of the cut batter under static loading. We have assumed the rock slope above the wall will act as a Surcharge (q) and this has been estimated at 505.2kPa using the soil profile and Densities (γ') listed in Table 3. Hand calculations outlining the methodology are included in Appendix 6.

The Active Effective Stress (σ_a') behind the wall has been determined for the wall height only, and uses the soil strength parameters for Unit 5 (Table 3). Due to the amount of Cohesion (c') assumed for the rock mass the Rankine's equation for Active Earth Pressure (p_a) returns a negative value (-5.8kPa), indicating that if the rock didn't extend above the top of the wall then there would be more than enough resistance in the rock mass to be self-supporting. This positive resistance is shown in the global stability analysis (Section 6.1.3) which indicates a FOS well above 1.5 (static case), and although this model doesn't include the wedge rock removed at the toe of the batter we consider that the cut slope is sufficiently steep that the relatively minor change in geometry would have very little overall effect.

Once the surcharge load from the slope above the wall height (101kPa) is added to the Rankine equation the net pressure behind the wall increases to 35.2kPa. The Active Thrust (P_a) resulting from the surcharge load (rock mass above the wall) becomes 2,779kN per metre length of wall, and acts at a point half way up the wall ($^1/_2$H). Theoretically, part of this thrust would be concealed out by the negative earth pressure (resistance from cohesion) which equates to a thrust of -995kN (per metre) and results in a total thrust acting against the back of the wall of 1,784kN (per meter). The thrust resulting from the earth pressure will act at a point one third of the wall height from the base of the wall ($^1/_3$H).

Please note, these values are preliminary only and will need to be verified once detail of any walls are provided. The purpose of providing this information is to give the structural engineer a feel for the magnitude of the additional strength that will be required along the parts of the building that will be in contact with the cut faces.

Where the loads on the walls are considered to be too high to resist with internal structural elements ground anchors (or similar) may be considered, but would require additional input from the geotechnical engineer.

7. SUMMARY

Cheal have been engaged by Bayswater Vehicles Ltd (Client) to carry out a geotechnical investigation and develop an earthworks design that can be used to cut back the existing hill side and enlarge the sales court area at the corner of Carlyle Street and Faraday Street. This work is to enable the development of the site, which will include a series of new showrooms and a large 20 bay workshop.

The earthworks will include a series of large cuts, which will have an overall batter of 0.25H:1.0V and will be up to 27m high. The total volume of cut is expected to be 22,500m^3. Some fill will be required and will form a wedge with a maximum thickness of 1.5m and have an approximate volume of 800m^3.

It is estimated that the earthworks will take approximately 12 weeks to complete depending on the Contractors methodology.

Geotechnical investigation included a desktop study, site specific geotechnical drilling and laboratory testing of the rock core was carried out. Geological Maps show the hill side behind the site is part of the Scinde Island Formation, which is late Pliocene in age and comprises calcareous, cross-bedded sandstone and limestone. Capping the sandstone/limestone rock is a thick band of loess that was deposited during the last glaciation (Pleistocene age), and includes various pumice and ash deposits erupted from the Taupo Volcanic Zone.

A Masters thesis carried out by Boyle (1986) subdivided the Scinde Island Formation into 5 separate members (Members A – E) and mapping shows that the rock exposed in the hill side behind the site is part of Member A. This member is the oldest part of the formation and is described as uniformly graded (well sorted) coarse sands with some silt and clay. The sands are partly formed from skeletal remains of shellfish and are cemented with calcium carbonate ($CaCO_3$), which typically makes up over 50% of the rock mass.

Review of historic/existing geotechnical investigations from Napier Hill show that the limestone described in Boreholes is broadly consistent with Boyle's description, and both insitu and laboratory testing shows that the upper loess material is stiff – very stiff, and the limestone is a very weak rock.

A fully cored geotechnical bore hole was carried out from a property at the end of May Avenue for the Bayswater project and was drilled a total of 40m at an angle of 65°, resulting in a vertical depth of 36.25m from the top of the hill side. The core was logged by the geotechnical engineer and point load testing was carried out on a total of 42 samples. Following this the combination of core quality and rock strength was used to categories the rock profile into 6 separate geotechnical units. Geotechnical strength values (ϒ', ø' & c') were determined for each unit based on the Hoek-Brown Classification and a final geotechnical ground model was developed. In terms of the site specific model the geotechnical units are considered to be homogenous and have a consistent horizontal orientation.

Slope stability analysis was carried out for a single cut batter formed at 0.25H:1.0V using the ground model. Factors of safety of 1.7 and 1.2 were determined for the static long-term case and the seismic short-term case respectively. It is considered that these values are above the minimum criteria outlined in supporting documentation for national earthworks standards.

Following discussion with the client's landscape architect a series of narrow benches have been incorporated into the final cut profile for the cutting that faces Faraday Street. These benches will be approximately 1m wide and spaced at 4m vertical intervals to enable planting to mitigate the visual effects of the cut slope and encourage ongoing and long-term vegetation. This benching will effectively increase the FOS of the upper part of the slope and will provide additional stability to the neighbouring garage.

Further mitigation against small scale fretting over time will include solid barriers along the upper edge of the car yard buildings to protect from material falling from the face, and face stabilisation and erosion control matting, which will be applied to areas identified during construction.

A number of retaining walls will be incorporated into the main buildings structure, and we have provided active earth pressure coefficients, and calculated a profile of vertical effective stress with depth. An example of the active thrust that would be applied to the back of a 5.5m high wall has been provided, but this is intended for preliminary design and to evaluate if additional ground anchoring of the walls is likely to be required. We anticipate that additional geotechnical input will be required in relation to the detail design of any walls.

8. DISCLAIMER

This Report has been prepared solely for the use of our client with respect to the particular brief given to Cheal Consultants.

No liability is accepted in respect of its use for any other purpose or by any other person or entity. All future owners of this property should seek professional geotechnical advice to satisfy themselves as to its ongoing suitability for their intended use.

The opinions, recommendations and comments given in this Report are the result from the application of accepted industry methods of site investigation. As factual evidence over much of the site has been obtained solely from a single borehole and observation of existing slope, which by their nature only provide information about that exact location, there may be special conditions pertaining to this site which have not been identified by the investigation and which have not been taken into account in the report. Any groundwater levels measured during the investigation may change over time.

If variations in the subsoils occur from those described or assumed to exist then the matter should be referred back to Cheal Consultants immediately.

CHEAL CONSULTANTS LIMITED
28 March 2019