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GEOTECHNICAL REPORT

FOR PROPOSED NEW RESIDENTIAL DEVELOPMENT

AT

No.1803- No.1803A PITTWATER ROAD, MONA VALE, NSW

Prepared For

EDK Garfield Pty Ltd

Project No.: 2025-096
October 2025

Document Revision Record

Issue No	Date	Details of Revisions
0	09th July 2025	Original Issue
01	10th July 2025	Revision 1
02	23rd October 2025	Revision 2

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for___ EDK Garfield Pty Ltd ________________________

Name of Applicant

Address of site __No.1803 – No.1803a Pittwater Road, Mona Vale, NSW

Declaration made by geotechnical engineer or engineering geologist or coastal engineer (where applicable) as part of a geotechnical report

I, ***__Troy Crozier___________*** *on behalf of* ***___Crozier Geotechnical Consultants*** *23rd* October 2025 certify that I am a ~~geotechnical engineer or~~ engineering geologist ~~or coastal engineer~~ as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above ~~organisation/~~company to issue this document and to certify that the ~~organisation/~~company has a current professional indemnity policy of at least $2million.

I:

- ☐ have prepared the detailed Geotechnical Report referenced below in accordance with the Australia Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ am willing to technically verify that the detailed Geotechnical Report referenced below has been prepared in accordance with the Australian Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009
- ☐ have examined the site and the proposed development in detail and have carried out a risk assessment in accordance with Section 6.0 of the Geotechnical Risk Management Policy for Pittwater - 2009. I confirm that the results of the risk assessment for the proposed development are in compliance with the Geotechnical Risk Management Policy for Pittwater - 2009 and further detailed geotechnical reporting is not required for the subject site.
- ☐ have examined the site and the proposed development/alteration in detail and I am of the opinion that the Development Application only involves Minor Development/Alteration that does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.
- ☐ have examined the site and the proposed development/alteration is separate from and is not affected by a Geotechnical Hazard and does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.
- ☐ have provided the coastal process and coastal forces analysis for inclusion in the Geotechnical Report

Geotechnical Report Details:

Report Title: Geotechnical Report for Proposed new Residential Development No. 1803 – No.1803A Pittwater Road, Mona Vale, NSW

Report Date: 23rd October 2025 **Project No.:** 2025-096

Author: Marvin Lujan

Author's Company/Organisation: Crozier Geotechnical Consultants

Documentation which relate to or are relied upon in report preparation:

Architectural Drawings– by Studio McCue, Draft Pre DA Issue, Project No: 25004, Date: 9/10/2025, Drawing No.: DA-000, DA011, DA012, DA-101 – DA-109, DA-201 to DA-204, DA-301 to DA-303, DA-701 to DA-705, DA-901 to DA-904.
Survey Drawing – by DP Surveying, Reference No.: 3609, Dated: 25/02/2025

I am aware that the above Geotechnical Report, prepared for the abovementioned site is to be submitted in support of a Development Application for this site and will be relied on by Pittwater Council as the basis for ensuring that the Geotechnical Risk Management aspects of the proposed development have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature ....

Name ...**Troy Crozier**..

Chartered Professional Status...**RPGeo (AIG)** ....

Membership No.: ...**10197**.......................................

Company... **Crozier Geotechnical Consultants**

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1(a) - Checklist of Requirements For Geotechnical Risk Management Report for Development Application

Development Application for_ EDK Garfield Pty Ltd
Name of Applicant
Address of site ___No.1803 – No.1803a Pittwater Road, Mona Vale, NSW

The following checklist covers the minimum requirements to be addressed in a Geotechnical Risk Management Geotechnical Report. This checklist is to accompany the Geotechnical Report and its certification (Form No. 1).

Geotechnical Report Details:

Report Title: Geotechnical Report for Proposed new Residential Development No.1803-No.1803a Pittwater Road, Mona Vale, NSW
Report Date: 23rd October 2025 **Project No.:** 2025-096
Author: Marvin Lujan
Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

- [x] Comprehensive site mapping conducted ____5th June 2025___________
- [x] Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
- [x] Subsurface investigation required
  - [ ] No Justification …… minor works only.
  - [x] Yes Date conducted ……5th June 2025……………………………………………
- [x] Geotechnical model developed and reported as an inferred subsurface type-section
- [x] Geotechnical hazards identified
  - [ ] Above the site
  - [ ] On the site
  - [x] Below the site
  - [ ] Beside the site
- [ ] Geotechnical hazards described and reported
- [x] Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
  - [x] Consequence analysis
  - [ ] Frequency analysis
- [ ] Risk calculation
- [x] Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- [x] Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- [x] Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
- [x] Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved.
- [x] Design Life Adopted:
  - [x] 100 years
  - [ ] Other ….
    specify
- [x] Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
- [x] Additional action to remove risk where reasonable and practical have been identified and included in the report.
- [ ] Risk assessment within Bushfire Asset Protection Zone.

I am aware that Pittwater Council will rely on the Geotechnical Report, to which this checklist applies, as the basis for ensuring that the geotechnical risk management aspects of the proposal have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated, and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature ……………………………………………………

Name …**Troy Crozier**……………………………………

Chartered Professional Status….**RPGeo (AIG)**……………………

Membership No. …**10197**……………………………………

Company… **Crozier Geotechnical Consultants**

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Date: 23rd October 2025
Project No: 2025-096

GEOTECHNICAL REPORT FOR PROPOSED NEW RESIDENTIAL DEVELOPMENT
No.1803- No.1803A PITTWATER ROAD, MONA VALE, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for a proposed residential development at No.1803 - No.1803a Pittwater Road, Mona Vale, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of Capital One Funds Management Pty Ltd on behalf of the client EDK Garfield Pty Ltd.

The site is located on the higher western side of Pittwater Road and contains two lots (front (No. 1803A) and rear (No. 1803)) with a two-storey dwelling in each block. Both properties contain front and rear lawn with front concrete driveways.

The site is located within a Class 5 Acid Sulfate Soils (ASS) area and is within 100m of a Class 3 area.

The proposed works comprise the demolition of the site structures and the construction of a six-storey residential building, Class 2 development with likely a two-storey basement garage below. It is understood that bulk excavation down to a maximum of 8.70m depth, extending 1.10m from the north boundary will be required, reducing south east to 5.80m depth, extending to 6m form the south-eastern boundary.

The site is located within Northern Beaches Council (Pittwater) jurisdiction, according to the Geotechnical Risk Management Policy for Pittwater – 2009 and Northern Beaches mapping portal the site is not located within a landslip risk zone. Due to the proposed works and proposed excavation depth, this Report has been prepared to meet the requirement of Paragraph 6.5 of the Geotechnical Risk Management Policy for Pittwater -2009 and include a landslide risk assessment to the method of AGS 2007.

This report also includes a description of site and sub-surface conditions, a plan, a geological section, an assessment of acid sulfate soils hazards, provides recommendations for construction to ensure stability is maintained for a design life of 100 years and provides recommendations on groundwater assessment and required documentation.

The investigation and reporting were undertaken as per the Proposal No.: P25-243, Dated: 21st May 2025.

The investigation comprised:

a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Geotechnical Engineer.
b) DBYD plan request and onsite service clearance of borehole locations by accredited contractor.
c) Drilling of four boreholes (BH1 to BH4) using a restricted access drill rig along with Dynamic Cone Penetrometer (DCP) testing to investigate the subsurface geology, depth to bedrock and identification of ground water conditions.
d) Drilling of one borehole (BH5) using a track mounted rig (Hanjin 8D, 5 tons).
e) Installation of three groundwater wells within BH2, BH3 and BH5, to allow assessment of groundwater.
f) Test analysis at NATA accredited chemical and geotechnical laboratories to determine classification/soil reactivity (Moisture content) and Aggressivity (to AS2159).
g) A detailed geotechnical inspection and mapping of the entire site and adjacent land, with identification of geotechnical conditions by a Geotechnical Engineer.
h) Photographic record of site conditions.

The following plans and drawings were supplied for the work:

- Architectural Drawings – Studio McCue, Draft Pre DA Issue, Project No: 25004, Date: 9/10/2025, Drawing No.: DA-000, DA011, DA012, DA-101 – DA-109, DA-201 to DA-204, DA-301 to DA-303, DA-701 to DA-705, DA-901 to DA-904.
- Survey Drawing – DP Surveying, Reference No.: 3609, Dated: 25/02/2025.

2. PROPOSED DEVELOPMENT:

It is understood that the proposed works involve demolition of two existing dwellings and the construction of a new six-storey unit development with two basement levels for carparking. The proposed basements will require an excavation of approximately 8.70m depth (1.10m from the north boundary) reducing east to 5.80m depth (3.20m from the east boundary) and partly extending to the south-western boundary at some locations.

The proposed Basement Bulk Excavation Level (BEL) is estimated to be approximately RL 5.5.

3. SITE FEATURES:

3.1. Description:

The site is a combination of two blocks. The front block (No.1803a) and rear block (No. 1803) form an irregular shaped site with a front east boundary sum of 18.59m, a north boundary of 68.86m, a south-east boundary of 39.28m, a south-west boundary of 51.035m and a west boundary of 1.505m, as referenced from the supplied survey plan.

Ground Surface Levels (GSL) within the site reduce from a high of approximately RL 15.15m at the western end of the block, reducing northeast to RL 11.08m. The site is within gently east dipping topography on the eastern face of a low north plunging ridge adjacent to a wide valley floor infilled with sediments

An aerial photograph of the site and its surrounds is provided below, as sourced from NSW Government Six Map spatial data system, as Photograph-1. A general view of the site at the time of investigation are provided in Photograph-2 to Photograph-4 below.

Photograph-1: Aerial photo of site and surrounds.

Photograph-2: Front of existing dwelling at No.1803a Pittwater Road. View looking west.

Photograph-3: Front of existing dwelling at No.1803 Pittwater Road. View looking west.

Photograph-4: Rear grass lawn of No.1803. View looking southwest.

3.2. Geology:

Reference to the Sydney 1:100,000 Geological Series sheet (9130) indicates that the site is underlain by Newport Formation (Upper Narrabeen Group) rock (Rnn) which is of middle Triassic Age. The Newport Formation typically comprises interbedded laminite, shale and quartz to lithic quartz sandstones and pink clay pellet sandstones.

Extract of Sydney (9130 Geology Series Map): 1:100,000 – Geology underlying the site

4. FIELD WORK:

4.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on the 5th June 2025 by a Geotechnical Engineer. It included a photographic record of site conditions with examination of existing structures and soil slopes. It also included the drilling of four auger boreholes (BH1, BH2, BH3 and BH4) using a restricted access drill rig employing solid stem spiral flight augers and a tungsten carbide bit. The investigation also included the drilling of one borehole (BH5 also referenced as BH101 in the laboratory testing) using a drill rig (Hanjin 8D) employing solid stem spiral flight augers and a tungsten carbide bit, to investigate sub-surface geology.

Dynamic Cone Penetrometer (DCP) testing was carried out from ground surface adjacent to boreholes BH1 to 4 and through the base of the boreholes when they had progressed, in accordance with AS1289.6.3.2 – 1997, "Determination of the penetration resistance of a soil – 9kg Dynamic Cone Penetrometer" to estimate near surface soil conditions and depth to bedrock.

BH5 utilised regularly spaced Standard Penetration Testing (SPT) in accordance with AS1289.6.3.1 – 1997 to assess soil strengths and collect samples.

Strata identification was undertaken on material recovered from the boreholes with samples collected as per "AS1726: 2017 Geotechnical Site Investigation" for logging purposes and submission to NATA accredited laboratories.

The investigation included a second site visit to monitor the groundwater wells installed within the site on the 24th June 2025.

Explanatory notes are included in Appendix: 1. Mapping information and test locations are shown on Figure: 1, along with detailed bore log and DCP sheets in Appendix: 2. A geological model/section is provided as Figure: 2, Appendix 2.

4.2. Field Observations:

Pittwater Road is formed with a gently south-east dipping bitumen pavement where it passes the site, with low concrete gutter and kerbs along the sides. Between the bitumen pavement and the front boundary is a concrete footpath, grass lawns and two concrete crossover driveways that allow vehicular access to the site. Within the road reserve in between the two concrete crossover driveway is a service pit (approximately ≤1.5m depth below GSL), approximately 1.0m east from the sites front boundary. The road pavement and road reserve generally appeared in good condition, with only two cracks observed across the concrete driveways. However, this not does not appear to be due to underlying geotechnical issues.

Photograph-5: Service at the road reserve. View looking down west.

The site comprises two blocks. The front block (No.1803a Pittwater Road) contains a two-storey brick and timber dwelling located within the centre. This block also contains a concrete driveway and grass lawn within the front and a grass lawn within the rear. To the south of the dwelling, along the south boundary is a garden (≤0.5m high) retained by a brick retaining wall (Photograph-6) and a concrete pathway (≤1.5m wide) along the southern side of the building. The front block is bounded by a timber fence.

The site-dwelling appeared in good condition with no signs of cracking. However, the retaining wall along the south boundary contained tilting into the site and the concrete pathway directly adjacent to the building is dipping south. Based on experience, this structural defect appears to be construction related. The building in this block appeared in good condition and signs of underlying geotechnical issues were not observed within the front block of the site.

Photograph-6: Tilted brick retaining wall along the south boundary. View looking west.

The site also contains a rear property, being a battleaxe block (No. 1803) which contains a two-storey brick dwelling that broadly occupies the centre of the block. The block contains grass lawns at the western and southern sides and a long front concrete driveway allowing vehicular access to the dwelling. Along the rear boundary are relatively tall palm trees (approximately ≤6m high and approximately 2.5m separated). The site dwelling appeared in good condition and signs of underlying geotechnical issues were not observed within the rear of the site.

A sewer inspection pit was observed within the rear grass lawn of No.1803. Information obtained from available Sydney Water (SW) sewer plans indicates that a SW sewer main is located at the rear of the site and it extends from the inspection pit continuing south-west and south-east into the neighbouring property (No. 44 – No.48 Park Street) and continues south-east parallel and directly adjacent to the common boundary. The sewer main within the site appears to be approximately 1.70m below the existing GSL and it comprises Vitrified Clay (VC) pipe (225mm diameter). The sewer outside the site is concrete encased pipe (150mm diameter) and approximately 1.49m below the neighbouring GSL.

In between the rear and front blocks, adjacent the north-western corner of the front block is a water tap which CGC has been informed has been broken for approximately one year, and is constantly leaking water downslope. Due to the water leakage, the rear grass of the front block was wet.

Photograph-7: Water tap leakage. View looking north front the front of the rear block.

The neighbouring property to the north (No.1805 Pittwater Road) contains a one to two storey brick dwelling within the eastern side of the block. The front of the block contains a grass lawn and concrete driveway within the southern and northern sides, respectively. The rear of the block contains a grass lawn. A pathway is located between the dwelling and common boundary. The lawn, pathway and rear lawn are directly adjacent to the boundary and the dwelling is located 0.50m from the common boundary. The neighbouring property contains a similar GSL to the site along the common boundary. Excessive ground movement or underlying geotechnical issues were not observed within the neighbouring property and visible aspects of the structures appeared in good condition.

The neighbouring property to the west (No.50 Park Street) contains a concrete driveway directly adjacent to the common boundary. The block contains a four-storey brick unit building within the centre of the block, with single storey (two) brick garages to the rear north. The building and garage are located approximately 4.50m from the common boundary. The neighbouring property contains a similar GSL to the site. Excessive ground movement or underlying geotechnical issues were not observed within the neighbouring property and visible aspects of the structures appeared in good condition.

The neighbouring property to the southeast (No.1801 Pittwater Road) contains a brick driveway and shed on the northern side of the block, a single storey weatherboard dwelling within the centre of the block and front and rear grass lawns. The driveway and shed are located directly adjacent to the common boundary and the dwelling is located approximately 7.0m from the common boundary. The neighbouring property contains a similar GSL to the site along the common boundary. Excessive ground movement or underlying geotechnical issues were not observed within the neighbouring property and the visible aspects of structures appeared in good condition.

The neighbouring property to the south (No.42b Park Street) is a battleaxe block and contains a two-storey brick building broadly occupying the centre of the block. The rear of the block contains a grass lawn and access to the dwelling is via a concrete driveway. The rear lawn is located directly adjacent to the common boundary and the building is located 5.0m from the common boundary. The property contains a similar GSL to the site along the common boundary. Excessive ground movement or underlying geotechnical issues were not observed within the neighbouring property and visible aspects of the structure appeared in good condition.

The neighbouring property to the southwest (No.44-No.48 Park Street) contains a block with two-storey townhouses with a basement carpark, broadly occupying the centre of the block. Between the townhouses and the common boundary is a pathway that surrounds the townhouses. The pathway is located directly adjacent to the common boundary and is approximately 1.0m above the common boundary supported by timber retaining wall which appeared to be in good condition. The townhouses are located approximately 2.50m from the common boundary. The location of the basement carpark is unknown at time of reporting; however. it is estimated to be at least 3.0m below the townhouse Ground Floor Level (GFL). The neighbouring property rear grass lawn is approximately 1.0m above the site along the common boundary.

Excessive ground movement or underlying geotechnical issues were not observed within the neighbouring property and visible aspects of structures appeared in good condition.

The neighbouring buildings and properties were only inspected from within the site or from the road reserve however the visible aspects did not show any significant signs of instability or other major geotechnical concerns which would impact the site or the proposed development.

4.3. Field Testing:

Based on the field borehole logs and DCP test results the subsurface conditions at the project site can be classified as follows:

- **TOPSOIL/FILL –** this layer was ecountered in all boreholes down to varying depths between 0.20m and 0.80m. It was classified as either soft, dark brown moist silty CLAY or loose, dark grey, fine to medium grained, moist, silty SAND with plant roots.
- **SILTY CLAY (ALLUVIUM DEPOSITS) –** this layer was encountered in all boreholes apart from BH5 to depths of between 0.80m and 1.20m. This stratum typically comprised firm (locally soft and stiff) brown to grey, low to medium plasticity, moist silty CLAY. Occasional organic material was encountered within this stratum.
- **SILTY CLAY (RESIDUAL SOIL)** – this layer was encountered below the topsoil, fill and/ or material interpreted as Alluvium Deposits, in all exploratory holes. This stratum was encountered to depths of between 1.80m and 3.00m and typically comprised stiff to very stiff silty CLAY with trace of sand.
- **CLAYEY SAND (RESIDUAL SOIL) –** In BH01, granular material, also interpreted as Residual soil was encountered to a depth of 2.80m. This material comprised dense, grey to pale red, clayey SAND with a trace of ironstone gravel.
- **SILTY CLAY (NEWPORT FORMATION) –** In all exploratory holes, material interpreted as extremely weathered material of the Newport Formation was encountered to depths of between 3.75m and 15.30m. This material typically comprised interbedded very stiff to hard silty CLAY with occasional very low to low strength ironstone, siltstone, and sandstone beds. In BH1, BH2, BH3, and BH4, the drilling rig refused on these beds at depths between 3.75 m and 5.50 m. In BH5, the larger drilling rig allowed penetration through this horizon, and encountered further interbedded very stiff to hard silty CLAY beneath to the maximum investigated test depth of 15.30m. During drilling, beds of ironstone, siltstone, and sandstone were encountered at approximate depths of 3.00 m, 8.00 m, and 15.00 m.

During drilling groundwater was observed at the bottom of BH5 i.e. 15.30m depth, overlying the interpreted siltstone bed. In BH3, water was encountered at a depth of 0.90m, this was due to a nearby broken water tap and did not represent the water table, nor a perched water table. Seepage was found in the ironstone/siltstone/sandstone beds and interpreted as perched water table and isolated and minor.

Well Groundwater Level Measurements

Groundwater monitoring wells were installed within BH2, BH3 and BH5 at depths specified below. The well within BH5 was installed to a deeper level to meet the requirements of the Department of Planning and Environment 'Minimum requirements for building site groundwater investigation and reporting'. In BH2 and BH3, on extraction of the drilling rods, parts of the hole collapsed, and the well could not be installed to the full drilling depth. In BH5, the base of the borehole was filled up to the base of the monitoring well with inert crushed sandstone.

Preliminary seepage inflow rates were monitored in BH2, BH3, and BH5 by removing (bailing) the encountered groundwater and measuring the time required for the groundwater level to recover (rebound).

Table-2: Groundwater monitoring well details

	BH2		**BH3**		**BH5**	
	Depth (m)	RL (m)	Depth (m)	RL (m)	Depth (m)	RL (m)
Screened Zone	2.80m – 4.30m	7.90m – 9.40m	3.30m – 4.80m	4.97m – 6.47m	6.13m – 7.63m	3.52m – 5.02m
Bottom of the well	4.30m	9.40m	4.80m	6.47m	7.63m	5.02m

Test results obtained on the 24th June 2025:

Table-3: Groundwater monitoring within BH2

TIME	**WATER DEPTH (m) from existing GSL**	**RL (m)**
1:10 PM	1.57m	12.13m
BH2 - After the seepage was bailed down to the bottom of the well from existing GSL		
TIME	**WATER DEPTH (m) from existing GSL**	**RL (m)**
1:20 PM	4.30m	9.40m
2:20 PM	3.80m	9.90m
Average seepage inflow rate (L/hr)		0.98 L/hr

Table-4: Groundwater monitoring within BH3

TIME	WATER DEPTH (m) from existing GSL	RL (m)
1:30 PM	3.75m	7.55m
BH3 - After the seepage was bailed down to the bottom of the well from existing GSL		
TIME	**WATER DEPTH (m) from existing GSL**	**RL (m)**
1:37 PM	4.83m	6.47m
2:20 PM	4.70m	6.60m
Average seepage inflow rate (L/hr)		0.36 L/hr

Table-5: Groundwater monitoring within BH5

TIME	WATER DEPTH (m) from existing GSL	RL (m)
1:42 PM	1.56m	11.09m
BH3 - After the seepage was bailed down to the bottom of the well from existing GSL		
TIME	**WATER DEPTH (m) from existing GSL**	**RL (m)**
2:13 PM	7.63m	5.02m
2:30 PM	7.28m	5.37m
Average seepage inflow rate (L/hr)		1.04 L/hr

<u>Local Rainfall Data</u>

Information available from the nearest bureau rainfall station (i.e. 066059 Terry Hills 7.2km away) was obtained from the Australian Bureau of Meteorology website and it is provided below. Chart-1 shows the long term statistics with respect to 2025 monthly values and Table-a shows the daily values from the two week period prior to the groundwater monitoring visit.

Chart-1: Monthly Rainfall data Terry Hills AWS

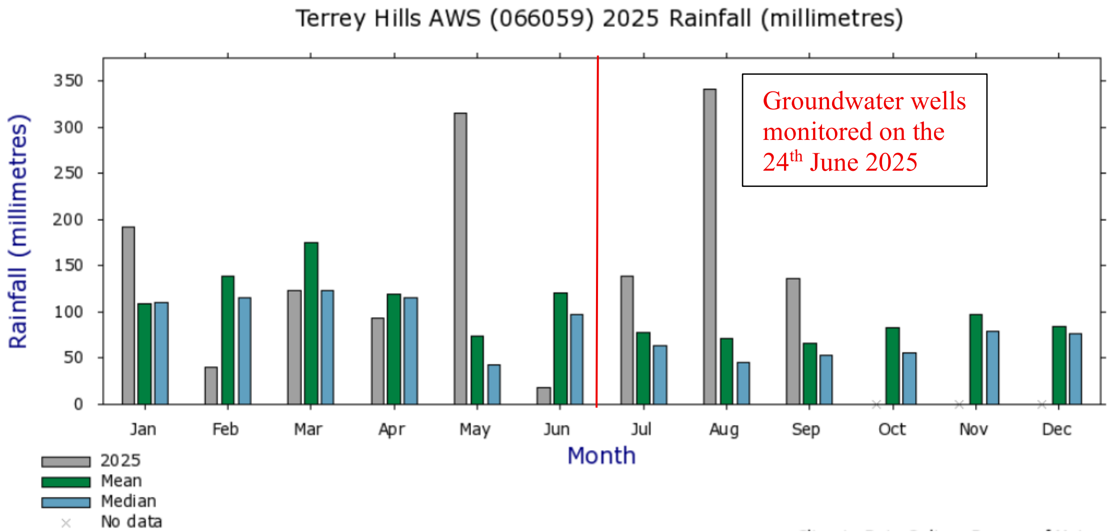

Table-a: Daily rainfall data (two weeks prior to groundwater monitoring)

Date	*Daily Rainfall (mm)*	*Accumulated Rainfall (mm)*
9th June	*0*	*0*
10th June	*1.0*	*1.0*
11th June	*0*	*1.0*
12th June	*0*	*1.0*
13th June	*0*	*1.0*
14th June	*0*	*1.0*
15th June	*0*	*1.0*
16th June	*7.6*	*8.6*
17th June	*0*	*8.6*
18th June	*0*	*8.6*
19th June	*0*	*8.6*
20th June	*0*	*8.6*
21st June	*0.2*	*8.8*
22nd June	*0*	*8.8*
23rd June	*0*	*8.8*

4.4. Laboratory Testing:

Geotechnical and chemical testing has been undertaken at NATA accredited Geotechnical/Chemical laboratories and the results are summarised and discussed in the following sections. The laboratory test report sheets are included in Appendix: 3

It should be noted that the sample descriptions provided on the summary tables are individual laboratory sample descriptions. No allowance has been made in the sample descriptions for sampling, sub-sampling or test methodology. The mass material properties are provided on the Borehole Logs, as such the laboratory test results should be read in conjunction with the relevant borehole log.

4.4.1 Moisture Content

Six soil samples were tested for measurement of field moisture content in accordance with Australian Standard AS 1289 2.1.1 and the results are summarized in Table-6 below.

Table-6: Summary of Reported Moisture Content Results

Sample Location and Depth (m)	Sample Description	Moisture Content (%)
BH5 1.00m -1.45m	Sandy Silty CLAY	20.7
BH5 3.00m - 3.45m	Sandy CLAY, with Gravel	17.5
BH5 7.00m - 7.45m	Silty CLAY, with Gravel	12.1
BH1 1.60m - 1.70m	Sandy Silty CLAY	8.1
BH3 1.20m - 1.30m	Silty CLAY	8.5
BH3 3.80m - 4.00m	Sandy Silty CLAY	19.2

4.4.2 Acid Sulfate Soils Testing

Four samples were tested to determine pH, pHFox and to assess reaction rates to a hydrogen peroxide solution, the results/observations are provided in the tables below. Two of the samples were also analyzed using the Chromium method to provide quantitative data on ASS based on the recommendations of the Acid Sulfate Soils Laboratory Methods Guidelines, Version: 2.1, June 2004 and National Acid Sulfate Soils Guidance (June 2018).

Samples were kept on ice and transported to a NATA accredited laboratory (Envirolab) for analysis under standard chain of custody protocol. A summary of the 'field' testing using hydrogen peroxide is given in. Table-7 below, together with the reaction observed during the test. A summary of the Chromium test results is provided in the *'Chromium Test Results'* Table-8 below.

Table-7: pH and pH FOX Test Results

Location	Depth (m)	R.L. (m)	pH (Field)	pH (FOX)
BH1	0.20m – 0.30m	14.10	6.6	5.7
BH1	0.50m – 0.60m	13.80	6.1	4.6
BH1	0.80m – 0.90m	13.50	6.0	5.0
BH3	1.20m – 1.30m	10.10	6.5	4.7
BH5	5.00m – 5.45m	7.65	5.5	3.9

Table-8: Scr Test Results

Borehole	Depth (m)	RL (m)	pH (kcl)	Titratable Actual Acidity (%w/w S)	Chromium Reducible Sulphur – Scr (% w/w)	Net Acidity (% w/w S)	Calculated Liming Rate (kg $CaCO_3$ / t)
BH3	0.35m – 0.40m	10.95m – 10.90m	7.6	<0.01	0.008	0.22	<0.75

5. COMMENTS:

5.1. Geotechnical Assessment:

The site investigation identified fill, underlain by Alluvium Deposits consisting of silty clay to a maximum depth of 1.20 m. These were underlain by Residual soil, typically comprising stiff silty clay, extending to depths of approximately 3.00 m. Below this, very stiff silty clay associated with the Newport Formation was encountered to the maximum investigated depth of 15.30 m bgl. This material was noted to be interbedded with beds of sandstone, siltstone, and ironstone.

Rainfall in the month prior the borehole drilling was significantly above average, yet the boreholes were effectively dry until completion, including down to 15.00m depth. Subsequent to rainfall, prior to monitoring, seepage inflow was low with values of ≤ 1L/hr in all three wells.

Groundwater was only encountered during drilling in BH5 at a depth of 15.30 m (RL -2.65 m). No groundwater was observed in the shallower boreholes (BH1 to BH4) during drilling, except for seepage recorded in BH3 at 0.90 m depth (RL 10.40 m), which is interpreted to be associated with a nearby leaking water pipe. During a subsequent groundwater monitoring visit, water was measured to be at depths of between 1.56m and 3.75m below current ground levels. It is considered that these water levels represent ponding of seepage inflow along the thin ironstone/siltstone/sandstone bands, which tend to contain minor perched aquifers, and does not represent a more static water table.

It is understood that the proposed works could comprise the demolition of the existing site structures and the construction of a new six-storey building with a two-basement carpark. Bulk excavation of approximately 8.7m depth, decreasing in the east to 5.8m depth, will be required for the proposed new basement.

Based on the investigation test results, the excavation is expected to intersect fill, Alluvial soils, Residual soil, and silty clay of the Newport Formation interbedded with beds of sandstone, siltstone and ironstone. Minor groundwater seepage into the excavation is expected, potentially increasing with depth of excavation, however, it is currently considered that a permanent large volume groundwater table will not be intersected (as evidence by the open boreholes to maximum of 15.30m depth before dry unit completion, though this should be confirmed via long term groundwater monitoring and further investigation.

The site excavation can be undertaken using an excavator with bucket and ripper and the use of a rock breaking equipment (e.g. rock hammer) will not be required. However, extreme care must be required during the site works (e.g. demolition and excavation) not to create ground vibrations that can damage the nearby SW sewer. Crozier Geotechnical Consultants (CGC) should be consulted regarding the size and type of demolition/excavation equipment proposed and demolition/excavation methodology prior to the works.

Due to the variability and interbedded nature of the material encountered, and the depth of the proposed excavation, very stiff or hard silty clays are expected to be present at the base of the basement excavation. Due to the interbedded nature of the Newport Formation, it is not possible to reliably predict which of these two soil types will be present at the base of the proposed footings. In light of this uncertainty, a conservative design approach is recommended.

Both soil types are considered suitable for the construction of traditional strip footings, shallow trench fill at the base of the excavation level, or piered foundations. To accommodate potential variability in the shear strength of the underlying materials, an allowable bearing capacity of **200 kPa** is recommended for shallow footing design.

If this bearing capacity is deemed inadequate for the structural loads associated with the proposed development, a piled foundation system may be required. In such cases, further subsurface investigation will likely be necessary to confirm the depth to competent bedrock within the Newport Formation to inform pile design.

Further preliminary bearing capacity values suitable for the site are provided in Section 5.2.1 of this report.

Based on the proposed excavation depth and distances to the site boundaries, the safe batter slopes as recommended in Section 5.2.2 of this report is only achieved between the basement excavation and part of the south-eastern boundary (where safe batter slopes are achieved). However, batter slopes ≥3.0m in height are not recommended in the soil condition encountered. Therefore, the construction of support prior to excavation will be required in most of the basement excavation. Where safe batter slopes are achieved, the construction of support post excavation is a viable option, however for simplicity in construction it is recommended that the construction of support prior to excavation be constructed along all the perimeter of the basement.

Where support prior to bulk excavation is required, driven piles, sheet piles or methodologies likely to generate significant vibrations to the adjacent structures are not recommended. The construction of a soldier to contiguous pile wall would be a viable option. All retaining structures must be constructed as per *Earth-retaining structures AS 4678-2002*.

Based on the groundwater monitor assessment via usage of installed groundwater wells, a maximum seepage inflow rate of 1.04 L/hr was measured in BH5 and a minimum seepage inflow rate of 0.36 L/hr was measured in BH3. Based on the effective area of the basement excavation, it is estimated that the water inflow volume varies between a maximum of 1.89 ML/year (BH2) and a minimum of 0.70 ML/year (BH3). It is recommended that a Hydrogeologist be contacted to assess the groundwater inflow rates estimated in this report and determine if long term groundwater monitoring will be required. Where a drained basement is proposed, it is anticipated that a dewatering management plan will be required to be prepared by a professional Hydrogeologist. Where a drained basement is proposed, the application of a Water Access License (WAL) may be required. Where a tanked basement is proposed, then a WAL may not be required and may require less applications/licenses making the development process simpler. More detailed information on the required applications will be provided by a professional Hydrogeologist.

It is anticipated that the proposed works will require an "Application for Water Supply Works Approval (AWSWA)", despite the low groundwater seepage inflow measured from the installed wells. However, this application can only be lodge post the issue of a Development Consent. It is recommended that Water NSW be contacted at early stage to assess the proposed works and issue General Terms of Approval (GTAs) which must be included in the development consent.

Based on the location of the proposed new basement, it will intersect the SW sewer manhole and part of the sewer line. In addition, the proposed basement excavation zone of influence (taken as 1.0V:1.0H from the base of the excavation location) includes the existing sewer line that extends into the neighbouring properties to the south. Therefore, it is anticipated that relocation of the existing SW sewer will be required along with a Specialist Engineering Assessment (SEA).

The site is also classified as being within an Acid Sulphate Soils (ASS) Class 5 Zone and located within 100m of a Class 3 area. Based on the laboratory test results indicators of Potential ASS and ASS were not encountered in the investigation, whilst the water table won't be encountered or lowered (though this needs to be confirmed with further investigation and monitoring works). Therefore, as per the Preliminary Assessment guidelines of the Acid Sulfate Soils Manual an ASS Management Plan (ASSMP) is not required.

The proposed works are considered suitable for the site and may be completed with negligible impact to existing, nearby structures within the site or neighbouring properties provided the recommendations of this report are implemented in the design and construction phases.

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and isolated boreholes. Therefore, minor variation to the interpreted sub-surface conditions is possible, especially between test locations. However, the results of the investigation provide a reasonable basis for the Development Application analysis and subsequent initial design of the proposed works.

5.2. Site Specific Risk Assessment:

There were no signs of existing or previous landslip instability within the site or adjacent land whilst the existing house structures show no signs of settlement or cracking. The proposed works require a large deep excavation that has potential to result in instability where not properly supported.

Based on our site investigation and the proposed works, it is considered that the stability hazards associated with the proposed works are limited to:

A. Landslip (earth slide >20m^3) from excavation sides.

A qualitative assessment of risk to life and property related to this hazard is presented in Table A and B, Appendix: 3, and is based on methods outlined in Appendix: C of the Australian Geomechanics Society (AGS) Guidelines for Landslide Risk Management 2007. AGS terms and their descriptions are provided in Appendix: 4.

The Risk to Life from Hazard A was estimated to be up to ***3.00 x 10^{-2}****, whilst the Risk to Property was considered to be* ***up to 'Very High'****.*

The hazards were therefore considered to be '**Unacceptable**' when assessed against the criteria of the Councils Policy. **However**, it should be noted that this assessment considers the excavations permanently unsupported, therefore actual risk levels will be significantly lower through construction of engineered pre-excavation support systems that will ensure "Acceptable" risk criteria will be achieved and maintained.

The entire site and surrounding slopes have therefore been assessed as per the Council Geotechnical Risk Management Policy 2009 and the site is considered to meet the 'Acceptable' risk management criteria for the design life of the development, taken as 100 years, provided the development is undertaken and the property is maintained as per the recommendations of this report.

5.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

5.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Not applicable
Type of Footing	Strip/Pad or Slab at base of excavation, or piers/piles external to excavation or where high point loads are required
Maximum Allowable Bearing Capacity for Shallow Footings	- Very Stiff Clay: 200kPa - Hard Clay: 400kPa
Piled Foundations	- To be confirmed through additional ground investigation works.
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	Ce – 'Shallow' soil site (based upon the guidance provided)
Remarks: * Subject to confirmation by geotechnical professional including further investigation. Where possible, all footings should be founded off material of similar strength to prevent differential settlement. All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata. This is mandatory to allow them to be 'certified' at the end of the project.	

5.3.2. Excavation:

Property Separation

The tables below shows the properties potentially affected by the proposed excavation and the separation distances to the shared property boundary and structure.

Basement Excavation

Table 1: Property Separation Distances

Boundary	Adjacent Property	Structure	Bulk Excavation Depth (m bgl)	Separation Distances (m)	
				Boundary (m)	Structure
North	No.1805 Pittwater Road	Driveway, terrace & pathway, and dwellings	8.70m depth, decreasing east to 5.8m depth at the eastern end	1.1m	- Lawn, pathway and rear lawn are located directly adjacent to the common boundary and the dwelling is located 0.50m from the common boundary.

South	No.1801 Pittwater Road	Driveway, house and shed	5.8m depth	6.0m	-Driveway and shed directly adjacent to the boundary, dwelling another 7.0m from the boundary.
	No.42B Park Street	Rear lawn and dwelling	7.5m depth	2.0m to 9.0m	- Rear lawn directly adjacent to the boundary and dwelling is another 5.0m from boundary.
	No.44- No.48 Park Street	Rear garden and buildings	7.9m depth	0.0m	- Rear lawn directly adjacent to the boundary and buildings are 2.5m from the common boundary.
West	No.50 Park Street	Driveway, garage and building	8.7m depth	16.0m	-Driveway directly adjacent to the boundary and building 4.50m from the boundary.
East	Pittwater Road	Footpath and road pavement	5.8m depth	3.2m	- Footpath is 6.0m from the boundary and road is another 8.5m.

Type of Material to be Excavated	Fill (to 0.8m)
	Alluvium – Silty Clay (to 1.2 m depth)
	Residual soil - Silty Clay (to 3.0 m depth)
	Newport Formation – Silty Clay (to base of excavation (proven to a depth of 15.30 m)).

Guidelines for batter slopes for this site are tabulated below:

Material	**Safe Batter Slope (degree)**	
	Short Term/ Temporary	**Long Term/ Permanent**
Fill	33	27
Alluvial Deposits – Silty Clay	40	30
Residual soil – Silty Clay	45	34
Newport Formation – Silty Clay	45	34

*Dependent on seepage and assessment by engineering geologist

Remarks:

Seepage along defects in the soil can also reduce the stability of batter slopes and invoke the need to implement additional support measures. Where safe batter slopes are not implemented the stability of the excavation cannot be guaranteed until the installation of permanent support measures. This should also be considered with respect to safe working conditions.

Equipment for Excavation	Topsoil/Fill Alluvial Deposits – Silty Clay Residual soil – Silty Clay Newport Formation – Silty Clay	Excavator with bucket and ripper
Recommended Vibration Limits	Sewer 8 mm/s (subject to SW requirements)	

(Maximum Peak Particle Velocity (PPV))	Residential structures 5mm/s on nearby properties.
Vibration Calibration Tests Required	Pending assessment of the proposed demolition equipment prior to the start of the demolition,
Full time vibration Monitoring Required	Pending the results from the Vibration Calibration Test
Geotechnical Inspection Requirement during construction	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • Prior to site demolition to assess size of machinery and methodology proposed, • During installation of the excavation support system, • For assessment of batter slopes, • Where unexpected ground conditions are identified or any other concerns are held, • At completion of the excavation, • Following footing excavations to confirm founding material strength.
Dilapidation Surveys Requirements	Recommended on neighbouring structures or parts thereof within 10m of the excavation perimeter prior to site work to allow assessment of the recommended vibration limit and protect the client against spurious claims of damage.

5.3.3. Retaining Structures:	
Required	New retaining structures will be required as part of the proposed basement
Types	-Where support prior to excavation is required, the construction of a soldier pile wall with shotcrete infill panels is a viable option, subject to further monitoring and investigation works. -Where support post excavation can be constructed, the construction of steel reinforced concrete/concrete block wall is a viable option. Designed in accordance with Australian Standard AS 4678-2002 Earth Retaining Structures

Parameters for calculating pressures acting on retaining walls for the materials likely to be retained:

Material	Unit Weight (kN/m³)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Topsoil/fill	18	ϕ' = 29°	0.35	0.52	N/A
Alluvium Deposits – Silty Clay	19	ϕ' = 30°	0.33	0.50	N/A
Residual Soil – Silty Clay	20	ϕ' = 34°	0.33	0.50	N/A
Newport Formation – Silty Clay	21	ϕ' = 34°	0.30	0.48	N/A

Remarks:

In suggesting these parameters it is assumed that the retaining walls will be fully drained with suitable subsoil drains provided at the rear of the wall footings. If this is not done, then the walls should be designed to support full hydrostatic pressure in addition to pressures due to the soil backfill. It is suggested that the retaining walls should be back filled with free-draining granular material (preferably not recycled concrete) which is only lightly compacted in order to minimize horizontal stresses.

Retaining structures near site boundaries or existing structures should be designed with the use of at rest (K_0) earth pressure coefficients to reduce the risk of movement in the excavation support and resulting surface movement in adjoining areas. Backfilled/ retaining walls within the site, away from site boundaries or existing structures, that may deflect can utilize active earth pressure coefficients (Ka).

5.3.4. Drainage and Hydrogeology

Groundwater Table or Seepage identified in Investigation		Groundwater table encountered within BH5 at 15.30m depth (RL -2.65m). Though further investigation and monitoring is required to confirm this value.
Excavation likely to intersect	Water Table	No* (Though further investigation and monitoring is required to confirm this)
	Seepage	Yes
Site Location and Topography		On the higher western side of the road, within gently east dipping topography
Impact of development on local hydrogeology		Limited inflow of seepage to bulk excavation
Onsite Stormwater Disposal		Not recommended

Remarks:

As the excavation faces are expected to encounter some seepage, an excavation trench should be installed at the base of excavation cuts to below floor slab levels to reduce the risk of resulting dampness issues. Trenches, as well as all new building gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which preferably discharges to the Council's stormwater system off site.

5.4. Conditions Relating to Design and Construction Monitoring:

To comply with Councils conditions and to enable us to complete Forms: 2b and 3 required as part of construction, building and post-construction certificate requirements of the Councils Geotechnical Risk Management Policy 2009, it will be necessary for Crozier Geotechnical Consultants to:

1. Conduct additional deep boreholes to determine geotechnical conditions, groundwater conditions and to meet structural engineering design requirements.
2. Review and approve the structural design drawings for compliance with the recommendations of this report prior to construction,
3. Inspection of site and works as per Section 5.3.2 of this report
4. Inspect all new footings and earthworks to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and the stability prior to the placement of steel or concrete,
5. Inspect completed works to ensure construction activity has not created any new hazards and that all retention and stormwater control systems are completed.

The client and builder should make themselves familiar with the Councils Geotechnical Policy and the requirements spelled out in this report for inspections during the construction phase. Crozier Geotechnical Consultants <u>cannot</u> sign Form: 3 of the Policy if it has not been called to site to undertake the required inspections.

5.5. Design Life of Structure:

We have interpreted the design life requirements specified within Council's Risk Management Policy to refer to structural elements designed to support the existing structures, control stormwater and maintain the risk of instability within acceptable limits. Specific structures and features that may affect the maintenance and stability of the site in relation to the proposed and existing development are considered to comprise:

- stormwater and subsoil drainage systems,
- retaining walls and instability,
- maintenance of trees/vegetation on this and adjacent properties.

Man-made features should be designed and maintained for a design life consistent with surrounding structures (as per AS2870 – 2011 (100 years)). It will be necessary for the structural and geotechnical engineers to incorporate appropriate design and inspection procedures during the construction period. Additionally, the property owner should adopt and implement a maintenance and inspection program.

If this maintenance and inspection schedule are not maintained the design life of the property cannot be attained. A recommended program is given in Table: C in Appendix: 3 and should also include the following guidelines.

- The conditions on the block don't change from those present at the time this report was prepared, except for the changes due to this development.
- There is no change to the property due to an extraordinary event external to this site
- The property is maintained in good order and in accordance with the guidelines set out in;
    a) CSIRO sheet BTF 18
    b) Australian Geomechanics "Landslide Risk Management" Volume 42, March 2007.
    c) AS 2870 – 2011, Australian Standard for Residential Slabs and Footings

Where changes to site conditions are identified during the maintenance and inspection program, reference should be made to relevant professionals (e.g. structural engineer, geotechnical engineer or Council). Where the property owner has any lack of understanding or concerns about the implementation of any component of the maintenance and inspection program the relevant engineer should be contacted for advice or to complete the component. It is assumed that Council will control development on neighbouring properties, carry out regular inspections and maintenance of the road verge, stormwater systems and large trees on public land adjacent to the site so as to ensure that stability conditions do not deteriorate with potential increase in risk level to the site. Also, individual Government Departments will maintain public utilities in the form of power lines, water and sewer mains to ensure they don't leak and increase either the local groundwater level or landslide potential.

6. CONCLUSION:

The site investigation identified fill, underlain by Alluvium Deposits consisting of silty clay to a maximum depth of 1.20 m. These were underlain by Residual soil, typically comprising stiff silty clay, extending to depths of approximately 3.00 m. Below this, very stiff and hard silty clay of the Newport Formation was encountered to the maximum investigated depth of 15.30 m bgl. This material was noted to be interbedded with beds of sandstone, siltstone, and ironstone.

Groundwater was encountered only in BH5 at a depth of 15.30 m (RL -2.65 m). No groundwater was observed in the shallower boreholes (BH1 to BH4) during drilling, except for seepage recorded in BH3 at 0.90 m depth (RL 10.40 m), which is interpreted to be associated with a nearby leaking water pipe. During a subsequent groundwater monitoring visit, water was measured to be at depths of between 1.56m and 3.75m below current ground levels. It is not clear if this water level represents groundwater levels or seepage levels and further investigation and monitoring should be undertaken to confirm this. The findings of the further monitoring/ assessment will determine the requirement for more substantial excavation support and tanking.

Based upon the ground conditions encountered during the investigation, very stiff or hard silty clays are expected to be present at the base of the basement excavation. Due to the interbedded nature of the Newport Formation, it is not possible to reliably predict which of these two soil types will be present at the base of the proposed footings. In light of this uncertainty, a conservative design approach is recommended.

Both soil types are considered suitable for the construction of traditional strip footings, shallow trench fill at the base of the of the excavation level, or piered foundations. To accommodate potential variability in the shear strength of the underlying materials, an allowable bearing capacity of **200 kPa** is recommended for shallow footing design.

If this bearing capacity is deemed inadequate for the structural loads associated with the proposed development, a piled foundation system may be required. In such cases, further subsurface investigation will likely be necessary to confirm the depth to competent bedrock within the Newport Formation to inform pile design.

It is recommended that CGC inspect the proposed demolition and excavation equipment prior to its use to assess the potential to create ground vibrations that can damage the nearby SW sewer and neighbouring properties. An onsite vibration calibration test is recommended.

Based on the identified sub-surface conditions excavation support measures will need to be implemented prior to bulk excavation to protect the boundaries, adjacent properties/structures and road reserves. This is expected to require a soldier or contiguous piled wall, subject to the findings of the additional monitoring and investigation. The construction of support prior to excavation is required along most of the perimeter of the basement excavation, except between the basement excavation and south-eastern boundary. However, for simplicity of construction, it is recommended that the construction of support prior to excavation be constructed along the sides of the proposed new basement.

Based on the location of the proposed basement, it is anticipated that the relocation of the existing SW sewer main will be required along with an SEA.

Based on the laboratory test results, the presence of PASS and ASS were not encountered within the site and an ASS Management Plan will not be required.

Based on the groundwater monitoring assessment via usage of the installed wells, it is estimated that the basement excavation will experience a maximum volume of 1.89 ML/year (BH2). It is recommended that these values be assessed by a professional Hydrogeologist Engineer and determine if a long term ground water monitoring will be required. Where a drained basement is proposed, it is anticipated that a WAL will be required. It is anticipated that where a tanked basement is proposed, less approvals/licenses will be required (making the development process faster).

It is anticipated that an application for water supply works approval will be required, however this will be post the issue of a Development Consent. It is recommended that during the DA lodgement process, Water NSW be contacted to assess the proposed site works and issue a General Term of Approval.

Provided the recommendations of this report are implemented in the design and construction phases of the development, it is considered that the works can be carried out with negligible impact to the site and neighbouring properties and as such are considered suitable for the site.

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Appendix 1

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NOTES RELATING TO THIS REPORT

Introduction

These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the Discussion and Comments section. Not all, of course, are necessarily relevant to all reports.

Geotechnical reports are based on information gained from limited subsurface test boring and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Description and classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigation Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. Sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination.
The strength terms are defined as follows:

Classification	Undrained Shear Strength kPa
Very soft	Less than 12
Soft	12 - 25
Firm	25 – 50
Stiff	50 – 100
Very stiff	100 - 200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT "N" Value (blows/300mm)	CPT Cone Value (Qc – MPa)
Very loose	less than 5	less than 2
Loose	5 – 10	2 – 5
Medium dense	10 – 30	5 -15
Dense	30 – 50	15 – 25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling to allow information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Drilling Methods

The following is a brief summary of drilling methods currently adopted by the company and some comments on their use and application.

Test Pits – these are excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descent into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) – the hole is advanced by a rotating plate or short spiral auger, generally 300mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling – the hole is advanced by pushing a 100mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers – the hole is advanced using 90 – 115mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPT's or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling - the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

Rotary Mud Drilling – similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. From SPT).

Continuous Core Drilling – a continuous core sample is obtained using a diamond-tipped core barrel, usually 50mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedures is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" – Test 6.3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150mm of say 4, 6 and 7
  as 4, 6, 7 then N = 13
- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm then as 15, 30/40mm.

The results of the test can be related empirically to the engineering properties of the soil. Occasionally, the test method is used to obtain samples in 50mm diameter thin wall sample tubes in clay. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch Cone – abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australia Standard 1289, Test 6.4.1.

In tests, a 35mm diameter rod with a cone-tipped end is pushed continually into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separte 130mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected buy electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) their information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: -

- Cone resistance – the actual end bearing force divided by the cross-sectional area of the cone – expressed in MPa.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio - the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0 – 5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0 – 50 MPa) is less sensitive and is shown as a full line. The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios 1% - 2% are commonly encountered in sands and very soft clays rising to 4% - 10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range: -

$$Q_c \text{ (MPa)} = (0.4 \text{ to } 0.6)\ N \text{ blows (blows per 300mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range: -

$$Q_c = (12 \text{ to } 18)\ C_u$$

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculations of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Dynamic Penetrometers

Dynamic penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150mm increments of penetration. Normally, there is a depth limitation of 1.2m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer – a 16mm diameter flattened rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test 6.3.3). The test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS 1289, Test 6.3.2). The test was developed initially for pavement sub-grade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is generally carried out in accordance with Australian Standard 1289 "Methods of Testing Soil for Engineering Purposes". Details of the test procedure used are given on the individual report forms.

Borehole Logs

The bore logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable, or possible to justify on economic grounds. In any case, the boreholes represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes, the frequency of sampling and the possibility of other than 'straight line' variations between the boreholes.

Details of the type and method of sampling are given in the report and the following sample codes are on the borehole logs where applicable:

D	Disturbed Sample	E	Environmental sample	DT	Diatube
B	Bulk Sample	PP	Pocket Penetrometer Test		
U50	50mm Undisturbed Tube Sample	SPT	Standard Penetration Test		
U63	63mm " " " " "	C	Core		

Ground Water

Where ground water levels are measured in boreholes there are several potential problems:

- In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made. More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be interference from a perched water table.

Engineering Reports

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. A three-storey building), the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty-storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface condition, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- unexpected variations in ground conditions – the potential for this will depend partly on bore spacing and sampling frequency,
- changes in policy or interpretation of policy by statutory authorities,
- the actions of contractors responding to commercial pressures,

If these occur, the Company will be pleased to assist with investigation or advice to resolve the matter.

Site Anomalies

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some later stage, well after the event.

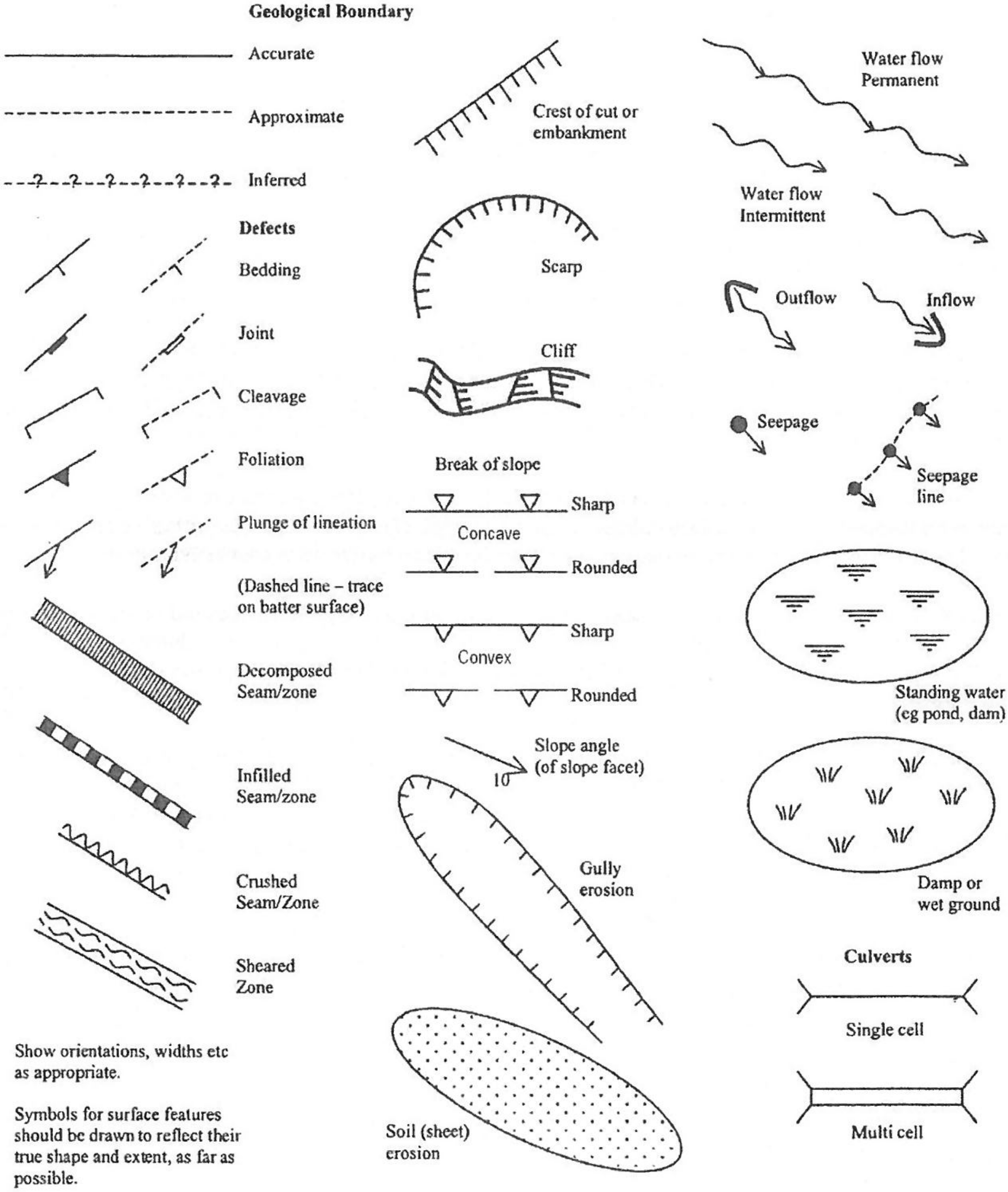
Reproduction of Information for Contractual Purposes

Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institution of Engineers Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a special ally edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

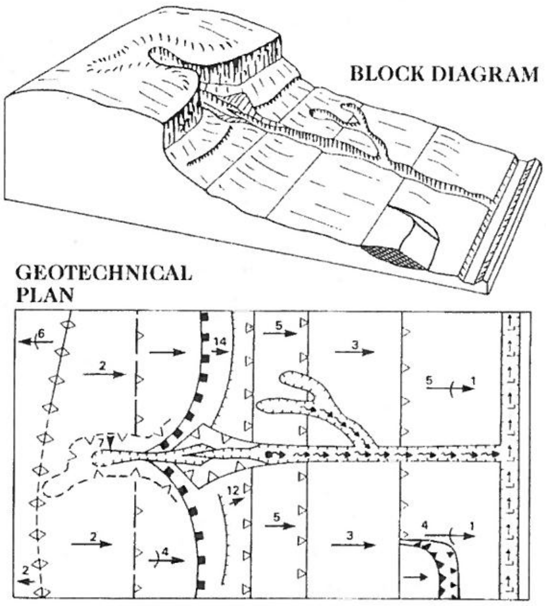
Site Inspection

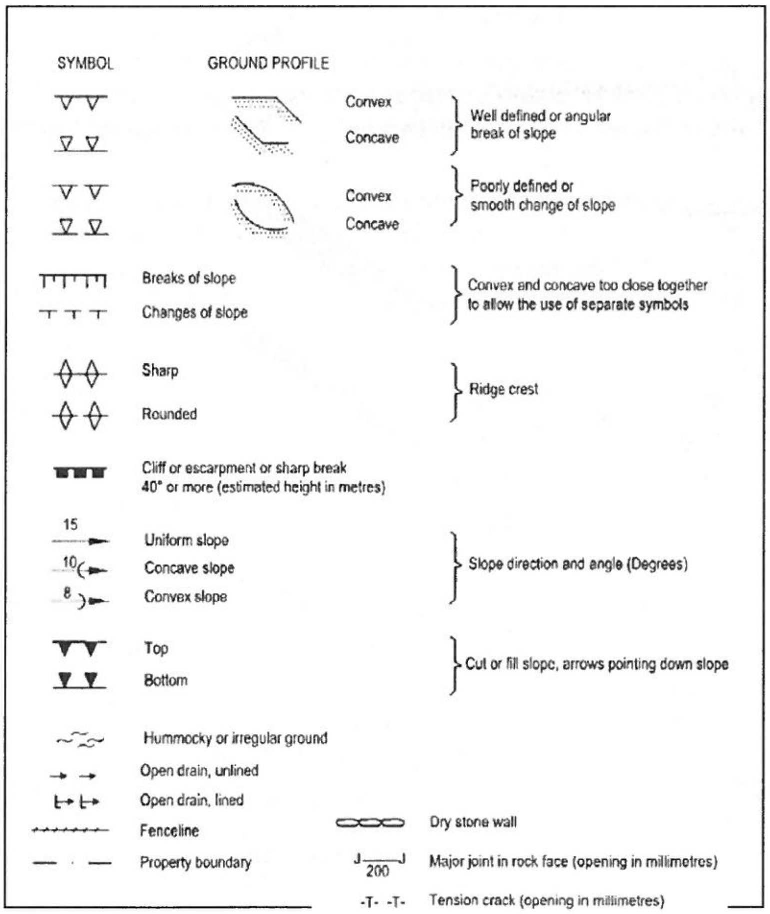
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



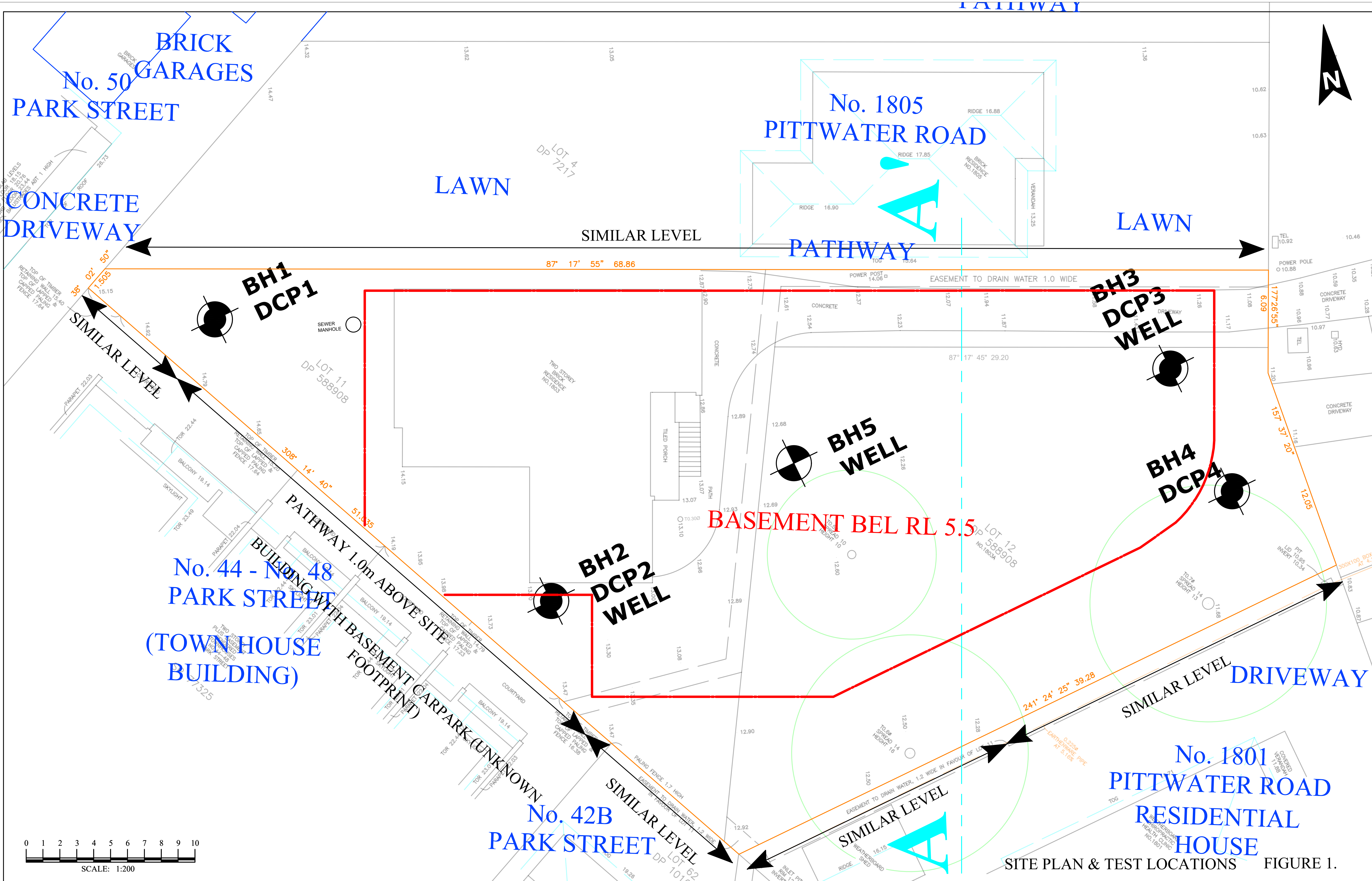
Examples of Mapping Symbols (after Guide to Slope Risk Analysis Version 3.1 November 2001, Roads and Traffic Authority of New South Wales).





Example of Mapping Symbols
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

Appendix 2



SITE PLAN & TEST LOCATIONS FIGURE 1.

CROZIER GEOTECHNICAL CONSULTANTS

Crozier Geotechnical ABN: 96 113 453 624
Unit 12, 42-46 Wattle Road Phone: (02) 9939 1882
Brookvale NSW 2100 Fax: (02) 9939 1883
Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

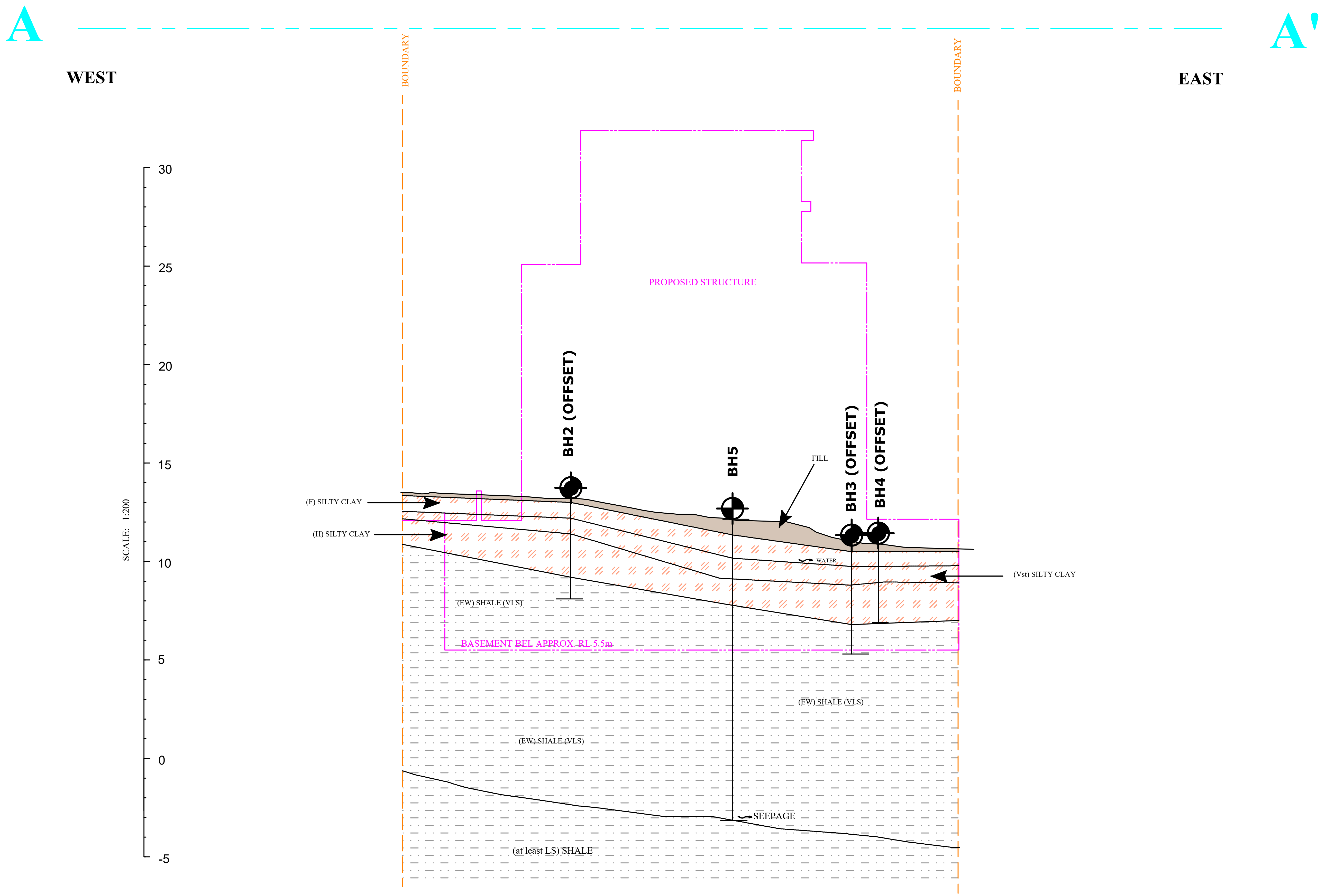
LEGEND

A — A'	CROSS-SECTION REFERENCE LINE
	PROPOSED BASEMENT
BH	AUGER LOCATIONS
	PROPERTY BOUNDARY
BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION

SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	16/10/2025
APPROVED BY:	TMC
DRAWN BY:	GL
PROJECT:	2025-096

PREPARED FOR: EDK Garfield Pty Ltd

ADDRESS: 1803 & 1803a PITTWATER ROAD, MONA VALE



0 1 2 3 4 5 6 7 8 9 10

SCALE: 1:200

VL - Very Loose	VS - Very Soft	ELS - Extremely Low Strength	EW - Extremely Weathered	fg - Fine Grained
L - Loose	S - Soft	VLS - Very Low Strength	HW - Highly Weathered	mg - Medium Grained
MD - Medium Dense	F - Firm	LS - Low Strength	DW - Distinctly Weathered	cg - Coarse Grained
D - Dense	St - Stiff	MS - Medium Strength	MW - Moderately Weathered	MAS - Massive
VD - Very Dense	VSt - Very Stiff	HS - High Strength	SW - Slightly Weathered	BD - Bedded
	H - Hard	VHS - Very High Strength	FR - Fresh	OC - Outcrop

NB. FOR LOCATION OF SECTION A-A', PLEASE REFER TO FIGURE 1. SITE PLAN AND TEST LOCATIONS

GEOLOGICAL MODEL FIGURE 2.

CROZIER
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LEGEND

A — A' CROSS-SECTION REFERENCE LINE

BH DCP AUGER / DYNAMIC CONE PENETROMETER LOCATION

BH AUGER LOCATIONS

FILL

PROPERTY BOUNDARY

PROPOSED WORKS

SILTY CLAY

SHALE BEDROCK

SCALE:	1:200 @ A3
DRAWING:	FIGURE 2
DATE:	16/10/2025
APPROVED BY:	TMC
DRAWN BY:	GL
PROJECT:	2025-096

PREPARED FOR:
EDK Garfield Pty Ltd

ADDRESS:
1803 - 1803A PITTWATER ROAD, MONA VALE

BOREHOLE LOG

CLIENT: **EDK Garfield Pty Ltd** **DATE:** 5/06/2025 **BORE No.:** BH01

PROJECT: **1803 & 1803a Pittwater Road** **PROJECT No.:** 2025-096 **SHEET:** 1 of 1

LOCATION: **Mona Vale NSW** **SURFACE LEVEL:** 14.3 m

Depth (m)	RL (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
0.00				Type	Tests	Type	Results
0.20	14.10	F	Grass FILL: SILTY CLAY Soft, dark brown, moist, silty clay.				
0.80	13.50	OL	SILTY CLAY with trace of sand. Soft to firm, grey brown, low to medium plasticity, moist. Sand is fine to coarse. Rare organics. (ALLUVIUM DEPOSITS) … from 0.70m, fine grained sandstone and ironstone gravels	D, PH, Phfox D, PH, Phfox	0.2-0.3 0.5-0.6		
2.20	13.10	CL	SILTY CLAY with trace of sand. Stiff, orange red, low plasticity, moist. (RESIDUAL SOIL) … from 1.30m hard … between 1.60-1.70m, gravel band, medium to coarse sandstone and ironstone gravel	D, PH, Phfox D ,MC 8.1%	0.8-0.9 1.6-1.7		
2.80	11.50	SC	Clayey SAND with trace of ironstone gravel. Dense, grey pale red, dry. Sand Is fine to coarse (RESIDUAL SOIL)	D	2.3-2.5		
3.75	10.55	CL	SILTY CLAY with sand. Stiff becoming very stiff, grey red, low plasticity, dry. (NEWPORT FORMATION)				

Dingo refusal @ 3.75m depth within interpreted VLS bedrock

RIG: Dingo restricted access rig DRILLER: AC

METHOD: Solid stem spiral flight auger, tungsten carbide bit LOGGED: JC

GROUND WATER OBSERVATIONS: No groundwater encountered.

REMARKS: CHECKED: JW

BOREHOLE LOG

CLIENT: EDK Garfield Pty Ltd **DATE:** 5/06/2025 **BORE No.:** BH02

PROJECT: 1803 & 1803a Pittwater Road **PROJECT No.:** 2025-096 **SHEET:** 1 of 1

LOCATION: Mona Vale NSW **SURFACE LEVEL:** 11.3

Depth (m)	RL (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
0.00				Type	Tests	Type	Results
0.30	11.00	F	Grass FILL: SILTY CLAY Soft, dark brown, moist, silty clay.	D	0.2-0.3		
1.20	10.10	OL	SILTY CLAY with trace of sand. Firm, grey brown, low to medium plasticity, moist. Sand is fine to coarse. With organics. (ALLUVIUM DEPOSITS) … from 0.70m, fine grained sandstone and ironstone gravels … from 0.80m, light brown mottled grey	D	0.5-0.6		
1.80	9.50	CL	SILTY CLAY with trace of sand. Stiff becoming very stiff, orange red, low plasticity, moist. (RESIDUAL SOIL) … between 1.60-1.70m, gravel band, medium to coarse sandstone and ironstone gravel				
5.10	6.20	CL	SILTY CLAY with trace of sand. Hard, orange red, low plasticity, moist. (NEWPORT FORMATION) … from 1.80m, dark red mottled pale grey with trace of fine sub rounded and sub angular gravels … at 3.10m, gravel band, pale grey mottled dark red. … between 3.50m and 3.80m, very low strength siltstone.	D	3.0-3.3		

Dingo refusal @ 5.10m depth within VLS shale

RIG: Dingo restricted access rig DRILLER: AC

METHOD: Solid stem spiral flight auger, tungsten carbide bit LOGGED: JC

GROUND WATER OBSERVATIONS: No groundwater encountered.

REMARKS: CHECKED: JW

BOREHOLE LOG

CLIENT:	**EDK Garfield Pty Ltd**	**DATE:**	5/06/2025	**BORE No.:**	BH03
PROJECT:	**1803 & 1803a Pittwater Road**	**PROJECT No.:**	2025-096	**SHEET:**	1 of 1
LOCATION:	**Mona Vale NSW**	**SURFACE LEVEL:**	11.3		

Depth (m) 0.00	RL (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.30	**11.00**	F	Grass FILL: SILTY CLAY Soft, dark brown, moist, silty clay.				
1.10	**10.20**	OL	SILTY CLAY with trace of sand. Firm, grey brown, low to medium plasticity, moist to wet. Sand is fine to coarse. With organics. (ALLUVIUM DEPOSITS) … from 0.70m, occasionally mottled red … from 0.90m, wet.	D D	0.3-0.4 0.8-0.9		
2.00	**9.30**	CL	SILTY CLAY with gravel. Stiff, grey mottled red, low plasticity, moist. Gravel is subangular to subrounded fine ironstone. (RESIDUAL SOIL) … between 1.10m and 1.80m, wet. … from 1.80, red mottled white	D, MC: 8.5%	1.2-1.3		
5.50	**5.80**	CL	SILTY CLAY with trace of sand. Hard, orange red, low plasticity, moist. (NEWPORT FORMATION) … between 3.50m and 3.70m, soft zone. … from 3.80m, pale grey mottled red with gravel of ironstone and sandstone	D, MC: 19.2%	3.8-4.0		

Dingo refusal @ 5.50m depth within VLS shale

RIG: Dingo restricted access rig — DRILLER: AC

METHOD: Solid stem spiral flight auger, tungsten carbide bit — LOGGED: JC

GROUND WATER OBSERVATIONS: No groundwater encountered.
Water encountered at 0.90m from nearby broken tap.

REMARKS: — CHECKED: JW

BOREHOLE LOG

CLIENT: EDK Garfield Pty Ltd **DATE:** 5/06/2025 **BORE No.:** BH04

PROJECT: 1803 & 1803a Pittwater Road **PROJECT No.:** 2025-096 **SHEET:** 1 of 1

LOCATION: Mona Vale NSW **SURFACE LEVEL:** 11.6

Depth (m) 0.00	RL (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.60	11.00	F	Grass FILL: SILTY CLAY Soft, dark brown, moist, silty clay.				
1.10	10.50	OL	SILTY CLAY with trace of sand and gravel. Stiff, dark brown, medium plasticity, moist. Sand is fine to coarse. With organics. (ALLUVIUM DEPOSITS)	D	0.7-0.8		
1.80	9.80	CL	SILTY CLAY with trace of gravel. Very stiff, grey mottled red, low plasticity, moist. Gravel is subangular to subrounded fine ironstone. (RESIDUAL SOIL) … between 1.10m and 1.80m, wet. … from 1.80, red mottled white				
4.00	7.60	CL	SILTY CLAY with trace of sand. Hard, grey mottled red, low plasticity, dry. (NEWPORT FORMATION)				

Dingo refusal @ 4.00m depth within VLS shale bedrock.

RIG: Dingo restricted access rig DRILLER: AC

METHOD: Solid stem spiral flight auger, tungsten carbide bit LOGGED: JC

GROUND WATER OBSERVATIONS: No groundwater encountered.

REMARKS: CHECKED: JW

BOREHOLE LOG

CLIENT: EDK Garfield Pty Ltd | **DATE:** 5/06/2025 | **BORE No.:** BH05/101

PROJECT: 1803 & 1803a Pittwater Road | **PROJECT No.:** 2025-096 | **SHEET:** 1 of 1

LOCATION: Mona Vale NSW | **SURFACE LEVEL:** 12.65

Depth (m) 0.00	RL (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.20	12.45	F	Grass FILL: SILTY CLAY Soft, dark brown, moist, silty clay.				
0.80	11.65	F	FILL: SILTY SAND. Loose, grey brown, moist. Sand is fine to coarse. Trace of organics. … from 0.50m, soft, grey clayey silt.				
		CL	SILTY CLAY with trace of sand. Firm orange red, medium plasticity, moist. Sand is fine to coarse. (RESIDUAL SOIL) … from 1.45m, orange red mottled grey, with some ironstone gravels.	D	1.00-1.45	SPT @ 1.00m	2,3,4 **N=7**
3.00	9.65		… from 2.00m, very stiff, orange red.	D	2.00-2.45	SPT @ 2.00m	5,8,9 **N=17**
		CL	SILTY CLAY with trace of sand. Hard, orange red, low plasticity, moist. Interbedded very stiff and hard beds of silty clay. (NEWPORT FORMATION) … at 3.30m, SPT hammer bouncing on interpreted siltstone/ sandstone /ironstone bed.	D	3.00-3.45	SPT @ 3.0m	5,20 (B) **N = Refusal**
			… between 4.00m and 5.00m, very stiff.	D	4.00-4.45	SPT @ 4.00m	6, 15,15 **N = 30**
			… between 5.00m and 6.00m, very stiff.	D	5.00-5.45	SPT @ 5.00m	1,8,11 **N=19**
			… between 6.00m and 7.00m, very stiff.	D	6.00-6.45	SPT @ 6.00m	13,12,20 **N = 32**
				D	7.00-7.45	SPT @ 7.00m	9,15,13 **N=28**
				D	8.00-8.45	SPT @ 8.00m	13,30 **N = Refusal**
				D	9.00-9.45	SPT @ 9.00m	2,10,18 **N=28**
				D	10.00-10.45	SPT @ 10.00m	5,9,11 **N=20**
				D	11.00-11.45	SPT @ 11.00m	9,16,21 **N=37**
				D	13.00-13.45	SPT @ 13.00m	5,9,31 **N=40**
15.30	-2.65		SPT bouncing - refusal @15.30m depth on interpreted siltstone/ sandstone/ ironstone bed	D	15.00-15.30	SPT @ 15.00m	2,20 **N = Refusal**

RIG: Rig 24 (Han-jin 8D) BG Drilling | DRILLER: BG Drilling

METHOD: Solid stem spiral flight auger, tungsten carbide bit | LOGGED: ML

GROUND WATER OBSERVATIONS: No Groundwater encountered

REMARKS: | CHECKED: JW

DYNAMIC PENETROMETER TEST SHEET

CLIENT: EDK Garfield Pty Ltd　　**DATE:** 5/06/2025

PROJECT: 1803 & 1803a Pittwater Road　　**PROJECT No.:** 2025-096

LOCATION: Mona Vale, NSW　　**SHEET:** 1 of 1

	Test Location									
Depth (m)	DCP1	DCP1a	DCP1b	DCP2	DCP2a	DCP2b	DCP3	DCP3a	DCP4	DCP4a
0.00 - 0.10	0	--	--	1	--	--	0	--	1	--
0.10 - 0.20	1	--	--	1	--	--	2	--	2	--
0.20 - 0.30	1	--	--	2	--	--	1	--	1	--
0.30 - 0.40	3	--	--	1	--	--	2	--	5	--
0.40 - 0.50	3	--	--	2	--	--	1	--	14	--
0.50 - 0.60	2	--	--	2	--	--	2	--	9	--
0.60 - 0.70	3	--	--	2	--	--	1	--	6	--
0.70 - 0.80	4	--	--	3	--	--	1	--	3	--
0.80 - 0.90	5	--	--	3	--	--	3	--	4	--
0.90 - 1.00	4	--	--	4	--	--	4	--	5	--
1.00 - 1.10	5	2	--	6	3	--	4	--	4	--
1.10 - 1.20	8	4	--	7	10	--	6	2	4	--
1.20 - 1.30	8	5	--	9	5	--	8	2	5	--
1.30 - 1.40	10	9	--	8	6	--	11	1	5	--
1.40 - 1.50	11	9	--	13	6	--	11	2	5	--
1.50 - 1.60	8	10	--	14	6	--	11	2	6	--
1.60 - 1.70	11	21 (B)	--	10	7	--	12	3	6	--
1.70 - 1.80	10		--	12	7	--	16	2	7	--
1.80 - 1.90	11		--	11	9	--	19	4	9	--
1.90 - 2.00	14		--	11	8	--		7	9	--
2.00 - 2.10	12		--	9	15	--		20	10	--
2.10 - 2.20	13		--	13	14	--		17	9	--
2.20 - 2.30	17 (B)		--	19	15 (B)	--			10	--
2.30 - 2.40			--	17		--			10	--
2.40 - 2.50			17 (B) @ 2.47m depth	24 (B)		--			10	--
2.50 - 2.60						7			10	7
2.60 - 2.70						17			15	9
2.70 - 2.80						20 (B) @ 2.80m depth			12	12
2.80 - 2.90									13	11
2.90 - 3.00										13
3.00 - 3.10										18 (B) @3.10m depth
3.10 - 3.20										
3.20 - 3.30										
3.30 - 3.40										
3.40 - 3.50										
3.50 - 3.60										
3.60 - 3.70										
3.70 - 3.80										
3.80 - 3.90										
3.90 - 4.00										

TEST METHOD: AS 1289. F3.2, CONE PENETROMETER

REMARKS:
(B) Test hammer bouncing upon refusal on solid object
-- No test undertaken at this level due to prior excavation of soils

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide		Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life	
A	Landslip (earth slide >20m³) from excavation sides		Silty Clay to base of excavation of varying stiffnesses. **Condition A:** Insufficient retention - Almost Certain **Condition B:** Engineer designed and implemented suport - Rare		a) Building 2.5m away from excavation edge b) Building 2.0m to 9.0m away from excavation edge c) Pavement 9.2m away from excavation edge d) Building 1.6m away from excavation edge		a) Person in building 16hrs per day. b) Persons on footpath 12hrs per day. c) Person in building 16hrs per day. d) Person in building 16hrs per day.	a) Likely to not evacuate b) Likely to not evacuate c) Likely to not evacuate d) Likely to not evacuate e) Likely to not evacuate	a) Person in building, collapse b) Person in open space, buried c) Person in building, collapse d) Person in building, collapse	Condition A	Condition B
			Almost Certain	**Rare**	**Prob. of Impact**	**Impacted**					
		a) 44-48 Park Street	0.1	0.00001	0.60	1.00	0.667	0.75	1.0	**3.00E-02**	**1.50E-07**
		b) 42B Park Street	0.1	0.00001	0.30	0.80	0.005	0.75	1.0	**9.38E-05**	**2.93E-12**
		c) Pittwater Road Footpath	0.1	0.00001	0.01	1.00	0.667	0.75	1.0	**5.00E-04**	**2.50E-09**
		d) 1805 Pittwater Road	0.1	0.00001	1.00	0.75	0.667	0.75	1.0	**3.75E-02**	**1.41E-07**

* hazards considered in for unsuitable/insufficient excavation support measures
* likelihood of occurrence for design life of 100 years
* Spatial Impact - Probaility of Impact referes to slide impacting structure/area expressed as a % (1.00 = 100% probability of slide impacting area if it occurs), Imapcted refers to % of area/strucure impacted if slide occurred
* considered for 1 person only, where multiple persons occupy locatoin at any one time risk levels increase accordingly
* where vehicles/persons travel past site, considered for slide impact during travel
* considered for adjacent premises/buildings founded via shallow footings unless indicated
* evacuation scale from Almost Certain to <u>not</u> evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knwoing of landslide and completely evacuating area prior to landslide impact.
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (earth slide >20m³) from excavation sides	a) 44-48 Park Street	Likely	Event will probably occur under adverse circumstances over the design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		b) 1801 Pittwater Road	Almost Certain	Event is expected to occur over design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High
		c) Pittwater Road Footpath	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Low
		d) 1805 Pittwater Road	Almost Certain	Event is expected to occur over design life.	Major	Extensive damage to most of site/structures with significant stabilising to support site or MEDIUM damage to neighbouring properties.	Very High

* hazards considered in current condition, without remedial/stabilisation measures and during construction works.
* qualitative expression of likelihood incorporates both frequency analysis estimate and spatial impact probability estimate as per AGS guidelines.
* qualitative measures of consequences to property assessed per Appendix C in AGS Guidelines for Landslide Risk Management.
* Indicative cost of damage expressed as cost of site development with respect to consequence values: Catastrophic : 200%, Major: 60%, Medium: 20%, Minor: 5%, Insignificant: 0.5%.

Appendix 4

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CERTIFICATE OF ANALYSIS 383206

Client Details

Client	Crozier Geotechnical Consultants
Attention	Marvin Lujan
Address	Unit 12/42-46 Wattle Rd, Brookvale, NSW, 2100

Sample Details

Your Reference	**2025-096 Mona Vale, 1803- 1803a Pittwater Road**
Number of Samples	9 Soil
Date samples received	13/06/2025
Date completed instructions received	13/06/2025

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Please refer to the last page of this report for any comments relating to the results.

Report Details

Date results requested by	20/06/2025
Date of Issue	20/06/2025

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Results Approved By
Diego Bigolin, Inorganics Supervisor
Nick Sarlamis, Assistant Operation Manager

Authorised By
Nancy Zhang, Laboratory Manager

Acid Sulphate Soil Suite		
Our Reference		383206-1
Your Reference	UNITS	BH3
Depth		0.35-0.40
Date Sampled		05/06/2025
Type of sample		Soil
Date prepared	-	13/06/2025
Date analysed	-	16/06/2025
pH $_{kcl}$	pH units	7.6
s-TAA pH 6.5	%w/w S	<0.01
TAA pH 6.5	moles H^+ /t	<5
a-Chromium Reducible Sulfur	moles H^+ /t	5
Chromium Reducible Sulfur	%w/w	0.008
S_{KCl}	%w/w S	[NT]
S_{HCl}	%w/w S	[NT]
S_{NAS}	%w/w S	[NT]
ANC_{BT}	% $CaCO_3$	0.70
s-ANC_{BT}	%w/w S	0.22
s-Net Acidity excluding ANC	%w/w S	0.0080
a-Net Acidity excluding ANC	moles H^+ /t	5.0
Liming rate excluding ANC	kg $CaCO_3$ /t	<0.75
s-Net Acidity including ANC	%w/w S	<0.005
a-Net Acidity including ANC	moles H^+ /t	<5
Liming rate including ANC	kg $CaCO_3$ /t	<0.75

sPOCAS field test						
Our Reference		383206-2	383206-3	383206-4	383206-5	383206-6
Your Reference	UNITS	BH3	BH5	BH1	BH1	BH1
Depth		1.2-1.3	5.00-5.45	0.2-0.3	0.5-0.6	0.8-0.9
Date Sampled		05/06/2025	05/06/2025	05/06/2025	05/06/2025	05/06/2025
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	13/06/2025	13/06/2025	13/06/2025	13/06/2025	13/06/2025
Date analysed	-	20/06/2025	20/06/2025	20/06/2025	20/06/2025	20/06/2025
pH_F (field pH test)	pH Units	6.5	5.5	5.7	6.1	6.0
pH_{FOX} (field peroxide test)	pH Units	4.7	3.9	4.7	4.6	5.0
Reaction Rate*	-	Low reaction	Low reaction	Low reaction	Low reaction	Low reaction

Soil Aggressivity				
Our Reference		383206-7	383206-8	383206-9
Your Reference	UNITS	BH1	BH2	BH3
Depth		0.2-0.3	3.0-3.20	3.8-4.0
Date Sampled		05/06/2025	05/06/2025	05/06/2025
Type of sample		Soil	Soil	Soil
Date prepared	-	18/06/2025	18/06/2025	18/06/2025
Date analysed	-	18/06/2025	18/06/2025	18/06/2025
pH 1:5 soil:water	pH Units	6.3	5.1	4.5
Electrical Conductivity 1:5 soil:water	µS/cm	52	50	68
Chloride, Cl 1:5 soil:water	mg/kg	20	<10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	10	58	93

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell.
Inorg-063	pH- measured using pH meter and electrode. Soil is oxidised with Hydrogen Peroxide or extracted with water. To ensure accurate results these tests are recommended to be done in the field as pH may change with time thus these results may not be representative of true field conditions.
Inorg-068	Determination of Acid Sulphate Soil analysis - a sample is analysed by traditional titration method and ICP-OES analysis. Based on Acid Sulfate Soils Laboratory Methods Guidelines, latest edition. Ideally samples should be received in the laboratory at <4oC. Please refer to SRA for sample temperature on receipt. Samples should also ideally be received within 24 hrs of sampling, otherwise there is the potential for oxidation to occur (as indicated by the lowering of the pH). Freezing the samples may help mitigate the potential for oxidation. There is no documented official holding time for frozen samples, we have assigned an arbitrary 180 days to frozen samples. Neutralising value (NV) of 100% is assumed for liming rate. Net Acidity with ANC calculation should only be used when corroborated by other data that demonstrates the soil material does not experience acidification during complete oxidation under field conditions. The recommendation that the SHCL concentration be multiplied by a factor of 2 to ensure retained acidity is not underestimated, has not been applied in the SHCL results reported.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Acid Sulphate Soil Suite					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			13/06/2025	[NT]	[NT]	[NT]	[NT]	13/06/2025	[NT]
Date analysed	-			16/06/2025	[NT]	[NT]	[NT]	[NT]	16/06/2025	[NT]
pH $_{kcl}$	pH units		Inorg-068	[NT]	[NT]	[NT]	[NT]	[NT]	98.0	[NT]
s-TAA pH 6.5	%w/w S	0.01	Inorg-068	<0.01	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
TAA pH 6.5	moles H^+/t	5	Inorg-068	<5	[NT]	[NT]	[NT]	[NT]	86	[NT]
a-Chromium Reducible Sulfur	moles H^+/t	3	Inorg-068	<3	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
Chromium Reducible Sulfur	%w/w	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	87	[NT]
S_{KCl}	%w/w S	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
S_{HCl}	%w/w S	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
S_{NAS}	%w/w S	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
ANC_{BT}	% $CaCO_3$	0.05	Inorg-068	<0.05	[NT]	[NT]	[NT]	[NT]	99	[NT]
s-ANC_{BT}	%w/w S	0.05	Inorg-068	<0.05	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
s-Net Acidity excluding ANC	%w/w S	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
a-Net Acidity excluding ANC	moles H^+/t	5	Inorg-068	<5	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
Liming rate excluding ANC	kg $CaCO_3$/t	0.75	Inorg-068	<0.75	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
s-Net Acidity including ANC	%w/w S	0.005	Inorg-068	<0.005	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
a-Net Acidity including ANC	moles H^+/t	5	Inorg-068	<5	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]
Liming rate including ANC	kg $CaCO_3$/t	0.75	Inorg-068	<0.75	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

QUALITY CONTROL: sPOCAS field test					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			13/06/2025	[NT]	[NT]	[NT]	[NT]	13/06/2025	[NT]
Date analysed	-			20/06/2025	[NT]	[NT]	[NT]	[NT]	20/06/2025	[NT]
pH_F (field pH test)	pH Units		Inorg-063	[NT]	[NT]	[NT]	[NT]	[NT]	99	[NT]
pH_{FOX} (field peroxide test)	pH Units		Inorg-063	[NT]	[NT]	[NT]	[NT]	[NT]	99	[NT]

QUALITY CONTROL: Soil Aggressivity					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			18/06/2025	[NT]	[NT]	[NT]	[NT]	18/06/2025	[NT]
Date analysed	-			18/06/2025	[NT]	[NT]	[NT]	[NT]	18/06/2025	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	101	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	98	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	92	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	92	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Air volumes are typically provided by customers (often as flow rate(s) and sampling time(s) and/or simply volumes) sampled or exposure times (determines 'volume' passive badges are exposed to)). Hence in such circumstances the volume measurement is inevitably not covered by Envirolab's NATA accreditation. An exception may occur where Envirolab Newcastle does the sampling where accreditation exists for certain types of sampling and hence volume determination(s). Note air volumes are often used to determine concentrations for dust and/or analyses on filters, sorbents and in impingers. For canister sampling, the air volume is covered by Envirolab's NATA accreditation.

Urine Analysis - The BEI values listed are taken from the 2022 edition of "TLVs and BEIs Threshold Limits" by ACGIH.

Report Comments

Samples were out of the recommended holding time for this analysis pH/EC.

MOISTURE CONTENT TEST REPORT

Client	Crozier Geotech	**Job #**	S25294-1
Address	Unit 12/ 42-46 Wattle Street Brookvale NSW 2100	**Report #**	S106881-MC
Project	New Building (2025-096)		

Test Procedure

- ☑ AS 1289 2.1.1 Determination of the moisture content of a soil - Oven drying method (Standard method).
- ☐ AS4133 1.1.1 Determination of the moisture content of rock - Oven drying method (standard method)
- ☐ RMS T120 Moisture content of road construction materials (Standard method)
- ☐ RMS T262 Determination of moisture content of aggregates (Standard method)

Sampling Sampled by Client - results apply to the sample as received — **Date Sampled** 5/06/2025

Preparation Prepared in accordance with the test method — **Date Tested** 17/06/2025

Sample #	Source	Sample Description	Moisture Content %
S106881	BH101 1.00-1.45m	Silty CLAY	**23.5**
S106882	BH101 3.00-3.45m	Silty CLAY	**20.2**
S106883	BH101 7.00-7.45m	Silty CLAY	**15.0**
S106884	BH1 1.60-1.70m	Silty CLAY	**19.4**
S106885	BH3 1.20-1.30m	Silty CLAY	**22.9**
S106886	BH3 3.80-4.00m	Silty CLAY	**22.2**

Notes

Accredited for compliance with ISO/IEC 17025 - Testing.

NATA Accredited Laboratory Number: 14874

Authorised Signatory:

Divye Grover

18/06/2025

Date:

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Results relate only to the samples tested.

Macquarie Geotechnical
14 Carter St
Lidcombe NSW 2141

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 5

APPENDIX A

DEFINITION OF TERMS

INTERNATIONAL UNION OF GEOLOGICAL SCIENCES WORKING GROUP ON LANDSLIDES, COMMITTEE ON RISK ASSESSMENT

Risk – A measure of the probability and severity of an adverse effect to health, property or the environment. Risk is often estimated by the product of probability x consequences. However, a more general interpretation of risk involves a comparison of the probability and consequences in a non-product form.

Hazard – A condition with the potential for causing an undesirable consequence (*the landslide*). The description of landslide hazard should include the location, volume (or area), classification and velocity of the potential landslides and any resultant detached material, and the likelihood of their occurrence within a given period of time.

Elements at Risk – Meaning the population, buildings and engineering works, economic activities, public services utilities, infrastructure and environmental features in the area potentially affected by landslides.

Probability – The likelihood of a specific outcome, measured by the ratio of specific outcomes to the total number of possible outcomes. Probability is expressed as a number between 0 and 1, with 0 indicating an impossible outcome, and 1 indicating that an outcome is certain.

Frequency – A measure of likelihood expressed as the number of occurrences of an event in a given time. See also Likelihood and Probability.

Likelihood – used as a qualitative description of probability or frequency.

Temporal Probability – The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.

Vulnerability – The degree of loss to a given element or set of elements within the area affected by the landslide hazard. It is expressed on a scale of 0 (no loss) to 1 (total loss). For property, the loss will be the value of the damage relative to the value of the property; for persons, it will be the probability that a particular life (the element at risk) will be lost, given the person(s) is affected by the landslide.

Consequence – The outcomes or potential outcomes arising from the occurrence of a landslide expressed qualitatively or quantitatively, in terms of loss, disadvantage or gain, damage, injury or loss of life.

Risk Analysis – The use of available information to estimate the risk to individuals or populations, property, or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification, and risk estimation.

Risk Estimation – The process used to produce a measure of the level of health, property, or environmental risks being analysed. Risk estimation contains the following steps: frequency analysis, consequence analysis, and their integration.

Risk Evaluation – The stage at which values and judgements enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental, and economic consequences, in order to identify a range of alternatives for managing the risks.

Risk Assessment – The process of risk analysis and risk evaluation.

Risk Control or Risk Treatment – The process of decision making for managing risk, and the implementation, or enforcement of risk mitigation measures and the re-evaluation of its effectiveness from time to time, using the results of risk assessment as one input.

Risk Management – The complete process of risk assessment and risk control (*or risk treatment*).

Individual Risk – The risk of fatality or injury to any identifiable (named) individual who lives within the zone impacted by the landslide; or who follows a particular pattern of life that might subject him or her to the consequences of the landslide.

Societal Risk – The risk of multiple fatalities or injuries in society as a whole: one where society would have to carry the burden of a landslide causing a number of deaths, injuries, financial, environmental, and other losses.

Acceptable Risk – A risk for which, for the purposes of life or work, we are prepared to accept as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks justifiable.

Tolerable Risk – A risk that society is willing to live with so as to secure certain net benefits in the confidence that it is being properly controlled, kept under review and further reduced as and when possible.

In some situations risk may be tolerated because the individuals at risk cannot afford to reduce risk even though they recognise it is not properly controlled.

Landslide Intensity – A set of spatially distributed parameters related to the destructive power of a landslide. The parameters may be described quantitatively or qualitatively and may include maximum movement velocity, total displacement, differential displacement, depth of the moving mass, peak discharge per unit width, kinetic energy per unit area.

Note: Reference should also be made to Figure 1 which shows the inter-relationship of many of these terms and the relevant portion of Landslide Risk Management.

APPENDIX C: LANDSLIDE RISK ASSESSMENT

QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

Approximate Annual Probability		Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
Indicative Value	**Notional Boundary**					
10^{-1}		10 years		The event is expected to occur over the design life.	ALMOST CERTAIN	A
	5×10^{-2}		20 years			
10^{-2}		100 years		The event will probably occur under adverse conditions over the design life.	LIKELY	B
	5×10^{-3}		200 years			
10^{-3}		1000 years		The event could occur under adverse conditions over the design life.	POSSIBLE	C
	5×10^{-4}		2000 years			
10^{-4}		10,000 years		The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
	5×10^{-5}		20,000 years			
10^{-5}		100,000 years		The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
	5×10^{-6}		200,000 years			
10^{-6}		1,000,000 years		The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

Note: (1) The table should be used from left to right; use Approximate Annual Probability or Description to assign Descriptor, not *vice versa*.

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

Approximate Cost of Damage		Description	Descriptor	Level
Indicative Value	**Notional Boundary**			
200%		Structure(s) completely destroyed and/or large scale damage requiring major engineering works for stabilisation. Could cause at least one adjacent property major consequence damage.	CATASTROPHIC	1
	100%			
60%		Extensive damage to most of structure, and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.	MAJOR	2
	40%			
20%		Moderate damage to some of structure, and/or significant part of site requiring large stabilisation works. Could cause at least one adjacent property minor consequence damage.	MEDIUM	3
	10%			
5%		Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
	1%			
0.5%		Little damage. (Note for high probability event (Almost Certain), this category may be subdivided at a notional boundary of 0.1%. See Risk Matrix.)	INSIGNIFICANT	5

Notes: (2) The Approximate Cost of Damage is expressed as a percentage of market value, being the cost of the improved value of the unaffected property which includes the land plus the unaffected structures.

(3) The Approximate Cost is to be an estimate of the direct cost of the damage, such as the cost of reinstatement of the damaged portion of the property (land plus structures), stabilisation works required to render the site to tolerable risk level for the landslide which has occurred and professional design fees, and consequential costs such as legal fees, temporary accommodation. It does not include additional stabilisation works to address other landslides which may affect the property.

(4) The table should be used from left to right; use Approximate Cost of Damage or Description to assign Descriptor, not vice versa

APPENDIX C: – QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY (CONTINUED)

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

LIKELIHOOD		CONSEQUENCES TO PROPERTY (With Indicative Approximate Cost of Damage)				
	Indicative Value of Approximate Annual Probability	1: CATASTROPHIC 200%	2: MAJOR 60%	3: MEDIUM 20%	4: MINOR 5%	5: INSIGNIFICANT 0.5%
A – ALMOST CERTAIN	10^{-1}	VH	VH	VH	H	M or **L** (5)
B - LIKELY	10^{-2}	VH	VH	H	M	**L**
C - POSSIBLE	10^{-3}	VH	H	M	M	**VL**
D - UNLIKELY	10^{-4}	H	M	**L**	**L**	**VL**
E - RARE	10^{-5}	M	**L**	**L**	**VL**	**VL**
F - BARELY CREDIBLE	10^{-6}	**L**	**VL**	**VL**	**VL**	**VL**

Notes: (5) For Cell A5, may be subdivided such that a consequence of less than 0.1% is Low Risk.

(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

Risk Level		**Example Implications** (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.
H	HIGH RISK	Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to Low. Work would cost a substantial sum in relation to the value of the property.
M	MODERATE RISK	May be tolerated in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as practicable.
L	LOW RISK	Usually acceptable to regulators. Where treatment has been required to reduce the risk to this level, ongoing maintenance is required.
VL	VERY LOW RISK	Acceptable. Manage by normal slope maintenance procedures.

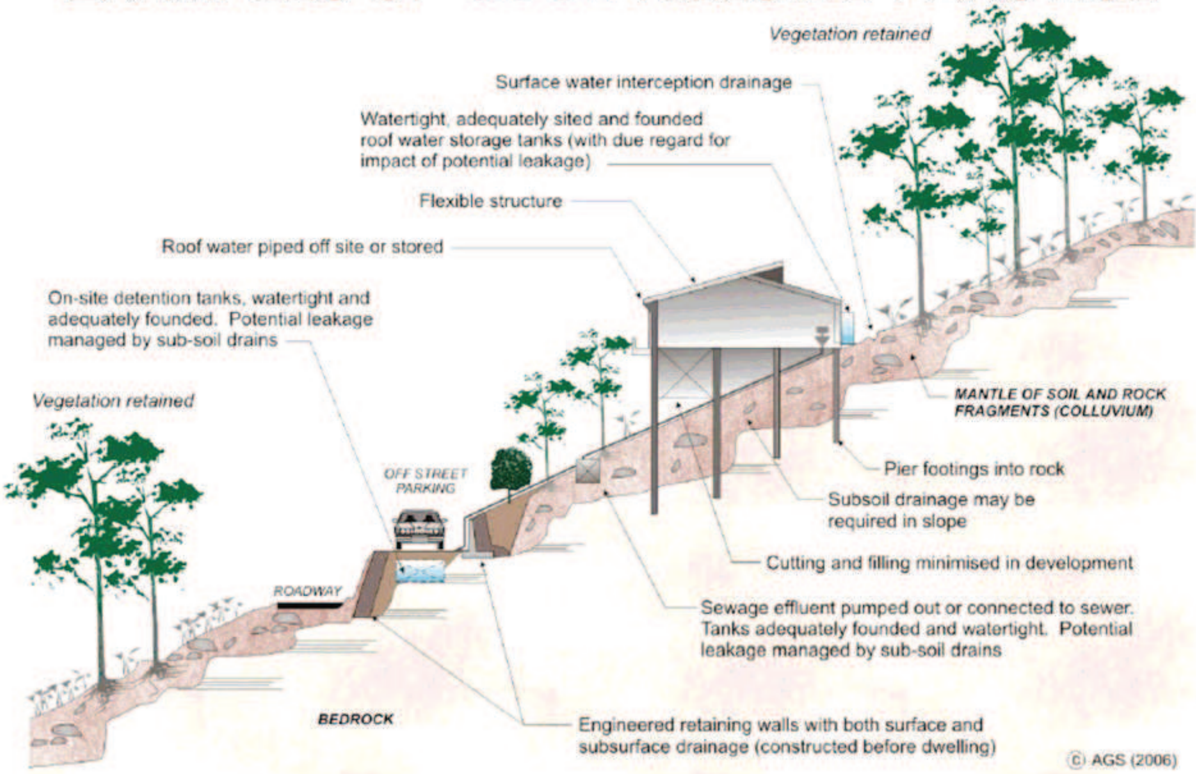
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

Appendix 6

APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

	GOOD ENGINEERING PRACTICE	*POOR ENGINEERING PRACTICE*
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical practitioner at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having obtained geotechnical advice, plan the development with the risk arising from the identified hazards and consequences in mind.	Plan development without regard for the Risk.
DESIGN AND CONSTRUCTION		
HOUSE DESIGN	Use flexible structures which incorporate properly designed brickwork, timber or steel frames, timber or panel cladding. Consider use of split levels. Use decks for recreational areas where appropriate.	Floor plans which require extensive cutting and filling. Movement intolerant structures.
SITE CLEARING	Retain natural vegetation wherever practicable.	Indiscriminately clear the site.
ACCESS & DRIVEWAYS	Satisfy requirements below for cuts, fills, retaining walls and drainage. Council specifications for grades may need to be modified. Driveways and parking areas may need to be fully supported on piers.	Excavate and fill for site access before geotechnical advice.
EARTHWORKS	Retain natural contours wherever possible.	Indiscriminatory bulk earthworks.
CUTS	Minimise depth. Support with engineered retaining walls or batter to appropriate slope. Provide drainage measures and erosion control.	Large scale cuts and benching. Unsupported cuts. Ignore drainage requirements
FILLS	Minimise height. Strip vegetation and topsoil and key into natural slopes prior to filling. Use clean fill materials and compact to engineering standards. Batter to appropriate slope or support with engineered retaining wall. Provide surface drainage and appropriate subsurface drainage.	Loose or poorly compacted fill, which if it fails, may flow a considerable distance including onto property below. Block natural drainage lines. Fill over existing vegetation and topsoil. Include stumps, trees, vegetation, topsoil, boulders, building rubble etc in fill.
ROCK OUTCROPS & BOULDERS	Remove or stabilise boulders which may have unacceptable risk. Support rock faces where necessary.	Disturb or undercut detached blocks or boulders.
RETAINING WALLS	Engineer design to resist applied soil and water forces. Found on rock where practicable. Provide subsurface drainage within wall backfill and surface drainage on slope above. Construct wall as soon as possible after cut/fill operation.	Construct a structurally inadequate wall such as sandstone flagging, brick or unreinforced blockwork. Lack of subsurface drains and weepholes.
FOOTINGS	Found within rock where practicable. Use rows of piers or strip footings oriented up and down slope. Design for lateral creep pressures if necessary. Backfill footing excavations to exclude ingress of surface water.	Found on topsoil, loose fill, detached boulders or undercut cliffs.
SWIMMING POOLS	Engineer designed. Support on piers to rock where practicable. Provide with under-drainage and gravity drain outlet where practicable. Design for high soil pressures which may develop on uphill side whilst there may be little or no lateral support on downhill side.	
DRAINAGE		
SURFACE	Provide at tops of cut and fill slopes. Discharge to street drainage or natural water courses. Provide general falls to prevent blockage by siltation and incorporate silt traps. Line to minimise infiltration and make flexible where possible. Special structures to dissipate energy at changes of slope and/or direction.	Discharge at top of fills and cuts. Allow water to pond on bench areas.
SUBSURFACE	Provide filter around subsurface drain. Provide drain behind retaining walls. Use flexible pipelines with access for maintenance. Prevent inflow of surface water.	Discharge roof runoff into absorption trenches.
SEPTIC & SULLAGE	Usually requires pump-out or mains sewer systems; absorption trenches may be possible in some areas if risk is acceptable. Storage tanks should be water-tight and adequately founded.	Discharge sullage directly onto and into slopes. Use absorption trenches without consideration of landslide risk.
EROSION CONTROL & LANDSCAPING	Control erosion as this may lead to instability. Revegetate cleared area.	Failure to observe earthworks and drainage recommendations when landscaping.
DRAWINGS AND SITE VISITS DURING CONSTRUCTION		
DRAWINGS	Building Application drawings should be viewed by geotechnical consultant	
SITE VISITS	Site Visits by consultant may be appropriate during construction/	
INSPECTION AND MAINTENANCE BY OWNER		
OWNER'S RESPONSIBILITY	Clean drainage systems; repair broken joints in drains and leaks in supply pipes. Where structural distress is evident see advice. If seepage observed, determine causes or seek advice on consequences.	

EXAMPLES OF GOOD HILLSIDE PRACTICE



EXAMPLES OF POOR HILLSIDE PRACTICE