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REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

CONSTRUCTION OF NEW DEVELOPMENT

at

94-96 PARK STREET AND 4 KUNARI PLACE, MONA VALE, NSW

Prepared For

Mona Vale Central Pty Ltd

Project No.: 2025-047

May, 2025

Document Revision Record

Issue No	Date	Details of Revisions
0	16 May 2025	DA issue

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for ____________________	
Name of Applicant	
Address of site	94-96 Park Street and 6 Kunari Road, Mona Vale

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*** __on this the __16 May 2025__ certify that I am a engineering geologist as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above company to issue this document and to certify that the 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 Demolition of existing structures and construction of a new three storey development

Report Date: 16 May 2025 **Project No.:** 2025-047

Author: S. Bohara and T. Crozier

Author's Company/Organisation: Crozier Geotechnical Consultants

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

Architectural: Walsh Architects; Drawing No.: DA 000, 010, 020, 030, 040, 050, 100 to 106, 200, 201, 300 to 302;
Revision No.: 01; Dated: 16.05.2025
Survey: Bee & Lethbridge Pty Ltd, Drawing No.: 23487, Dated: 18/03/2025

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**

REGISTERED PROFESSIONAL GEOSCIENTIST
AUSTRALIAN INSTITUTE OF GEOSCIENTISTS
TROY CROZIER
10,197

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1(a) - Checklist of Requirements For Geotechnical Risk Management Report for Development Application

Development Application for ____
Name of Applicant
Address of site ___94-96 Park Street and 6 Kunari Road, Mona Vale ____________________

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 Demolition of existing structures and construction of a new three storey development
Report Date: 16 May 2025 **Project No.:** 2025-047
Author: S. Bohara and T. Crozier
Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

- ■ Comprehensive site mapping conducted ___25.03.2025_________ (date)
- ■ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
- ■ Subsurface investigation required
  - ☐ No Justification ……………………………………………………...
  - ■ Yes Date conducted ……Preliminary conducted 25.03.2025………………
- ■ Geotechnical model developed and reported as an inferred subsurface type-section
- ■ Geotechnical hazards identified
  - ☐ Above the site
  - ■ On the site
  - ☐ Below the site
  - ☐ Beside the site
- ■ Geotechnical hazards described and reported
- ■ Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
  - ■ Consequence analysis
  - ■ Frequency analysis
- ■ Risk calculation
- ■ Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved.
- ■ Design Life Adopted:
  - ■ 100 years
  - ☐ Other …………………………………………… specify
- ■ Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
- ■ 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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TABLE OF CONTENTS

CROZIER
GEOTECHNICAL CONSULTANTS

APPENDICES

1	Notes Relating to this Report
2	Figure 1 – Site Plan, Figure 2 - Section
	Borehole Log sheets and Dynamic Penetrometer Test Results
3	Risk Assessment Tables
4	Laboratory Test Results
5	AGS Terms and Descriptions
6	Hillside Construction Guidelines

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GEOTECHNICAL REPORT FOR PROPOSED DEMOLITION OF EXISTING STRUCTURES AND CONSTRUCTION OF NEW THREE STOREY DEVELOPMENT 94-96 PARK STREET, AND 4 KUNARI PLACE, MONA VALE, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for a proposed new three storey residential structure at 94-96 Park Street and 4 Kunari Place, Mona Vale, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the written request of Walsh Architects on behalf of the client Mona Vale Central Pty Ltd.

It is understood that the proposed works consist of the demolition of existing structures and the construction of two new three storey apartment buildings formed over a double basement level carpark. The proposed structure will require bulk excavation to a maximum of approximately 9.75m depth.

The site is not located within a geotechnical landslip hazard zone as identified within Northern Beaches Councils precinct (Geotechnical Risk Management Policy for Pittwater – 2009). However, due to the depth of proposed bulk excavation a full geotechnical investigation and subsequent reporting will be required for Council submission of a Development Application, as per the policy.

To meet the Councils Policy requirements, a detailed Geotechnical Report which meets the requirements of Paragraph 6.5 of that policy must be submitted. This report includes a landslide risk assessment to the methods of AGS 2007 for the site and proposed works, plans, geological sections and provides recommendations for construction and to ensure stability is maintained for a preferred design life of 100 years to meet the policy requirements. It is recommended that the client make themselves aware of the Policy and its requirements.

The site is also classified under Northern Beaches Council (Pittwater) LEP 2014 as being within 'Class 3' Acid Sulfate soils hazard zone.

The assessment and reporting were taken as per the Fee Proposal No.: P25-119, Dated: 17th March 2025. The investigation comprised:

a) Before You Dig Australia (BYDA) plan review and onsite clearance of test locations by an accredited service locating sub-contractor.
b) A detailed geotechnical inspection and mapping of the site and inspection of adjacent properties by a Geotechnical Engineer.
c) Drilling of four auger boreholes to identify sub-surface geology using a restricted access drill rig employing solid stem spiral flighted auger and hand tools due to site access limitation.
d) Dynamic Penetrometer (DCP) testing at seven locations to investigate subsurface conditions.
e) Soil sample collection and logging as per “AS1726: 2017 Geotechnical Site Investigation”
f) Test analysis at NATA accredited chemical laboratories (Envirolab) to determine pH, pHFOX, Scr/SPOCAS and aggressivity (to AS2159).
g) Test analysis at NATA accredited chemical laboratories (Macquire Geotech) to determine moisture content and Atterberg limits.

The following documents, plans and drawings were supplied and were relied on for the proposal, investigation and reporting:

- Survey Drawing – Bee & Lethbridge Pty Ltd, Drawing No.: 23487, Dated: 18/03/2025
- Architectural Drawings – Walsh Architects; Drawing No.: DA 000, 010, 020, 030, 040, 050, 100 to 106, 200, 201, 300 to 302; Revision No.: 01; Dated: 16.05.2025.

2. PROPOSED WORKS

The proposed works involve the demolition of existing structures and the construction of two new three storey apartment buildings formed over a double basement level carpark with a main swimming pool and several plunge pools.

The lower basement level of the proposed development is proposed with a Finished Floor Level (FFL) of RL2.75. As such it is anticipated to require bulk excavation to RL2.50 resulting in a depth of up to approximately 9.75m at the south end of the development.

To the east, excavation will reach depths between 9.75m reducing to 6.10m in the north adjacent to No. 92 and 92A Park Street and No. 167 Darely Street West.

On the western side of the site, excavation of up to 8.00m depth will occur adjacent to the intersection between Kunari Place and Park Street, with the cut depth reducing to ≤2.0m in the north.

The proposed excavation will be located approximately 4.00m from the east boundary, with the nearest dwelling structures situated a further 1.30m and 1.73m from the boundary. The works will be positioned at separation distances of approximately 6.50m from the south boundary with Parks Street, with a footpath/nature strip located immediately beyond the boundary, and the road pavement a further 4.3m away. The bulk excavation will be approximately 6.50m from the neighbouring property to the north (No. 6) where a residential dwelling is 1.0m from the boundary and >12.0m from the property to the north-east.

A swimming pool and the plunge pools will also be constructed along the west and north sides, requiring additional minor excavations.

3. SITE FEATURES:

3.1. Description:

The site (94-96 Park Street & 4 Kunari Place, Mona Vale, Lot 42 DP11108, Lot 2 DP 222636, & Lot 13 DP226681) is an irregular shaped block situated on the north side of Park Street and the east side of Kunari Place within gentle north-west dipping topography. The survey indicates ground surface levels within the site vary from a high of RL12.25 along the southern boundary to a low of RL3.39 at the north-east corner.

From the provided plans, the site has front southern boundaries of 20.62m and 6.215m, stepped rear boundary of 53.37m, eastern boundary of 80.41m, and western boundary of 54.52m. An aerial photograph of the site and its surrounds is provided in Photograph 1, as sourced from NSW Government Six Maps Spatial Data.

Photograph 1: Aerial view of site and surrounds (NSW Government Six Map Spatial Data)

3.2. Geology:

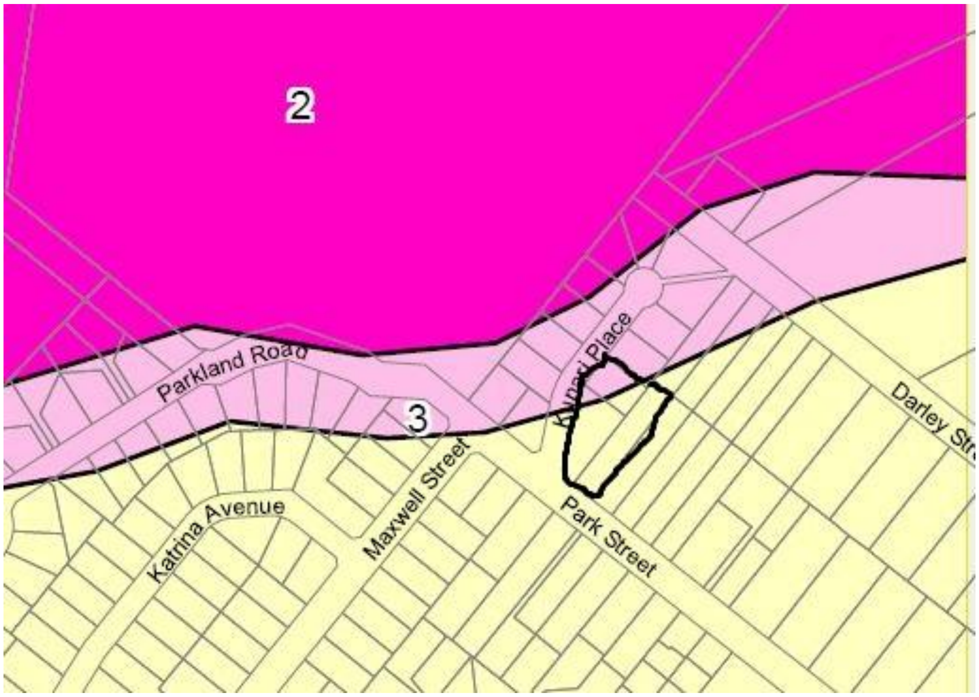
Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is underlain by Quaternary aged alluvial and estuarine sediments (Qha) which typically consist of silty to peaty quartz sands, silt and clay with common shell layers. The site is also close to a boundary between Newport Formation rocks (Rnn) just to the south of the site. However, the topography of the site indicates the Quatenary sediments are located to the north-west of the site.

An extract from the Sydney 1:100,000 Geological Series sheet 9130 provided below (extract 1) indicates the geology underlying the site and surrounding area.

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

3.3 Acid Sulfate Soils

Reference to the Pittwater Council Local Environment Plan 2014 Acid Sulfate Soils Map – Sheet ASS_012 (excerpt shown below) indicates the site is within Class '3' and Class '5' zoning. The higher classed zoning is to the north of the site associated with the Cahill Creek.



Extract 2: Pittwater Council LEP 2014, Acid Sulfate Soils Map Sheet ASS_012 with site indicated

4. FIELD WORK:

4.1. Methods:

The field investigation comprised a geotechnical inspection, mapping of the site and adjacent properties on 25th March 2025 by a Geotechnical Engineer. It included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of existing features and ground conditions.

The subsurface investigation included the drilling of four auger boreholes (BH1 – BH4) throughout the site to investigate the sub-surface geology using a restricted access drill rig operating solid stem, spiral flighted augers and a tungsten carbide blade bit and hand tools where access restrictions existed.

Geotechnical logging of the subsurface conditions was undertaken by a Geotechnical Engineer by inspection of disturbed soil recovered from the augers. Logging was undertaken in accordance with AS1726:2017 'Geotechnical Site Investigations'. Soil samples were collected from the boreholes and directly transferred to sterile glass jars for submission to a NATA accredited laboratory for Acid Sulphate Soils and aggressivity testing.

Dynamic Cone Penetrometer (DCP) testing was carried out from the ground surface adjacent to the boreholes 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.

Explanatory notes are included in Appendix: 1. Mapping information and test locations are shown on Figure: 1, along with detailed borehole logs and DCP sheets in Appendix: 2. A geological model/section is provided as Figure: 2, Appendix: 2. Laboratory test results are provided in Appendix 3.

4.2. Field Observations:

Park Street and Kunari Place are both formed with a bituminous sealed pavement that is gentle west dipping for Park Street and north dipping for Kunari Place and separated from the site by a concrete kerb, concrete walkway as well as a nature strip on Park Street whilst the site is separated from Kunari Place by a nature strip. The roadways showed some signs of cracking but did not exhibit any obvious signs of significant cracking or excessive settlement to indicate any underlying geotechnical concerns that may impact the site.

The site comprises three lots with containing, a separate one and two storey residential dwelling which are of brick masonry and timber construction. The one storey timber dwelling (No.94 Park Street) is situated in the southeastern portion of the site, the two storey rendered dwelling (No. 96 Park Street) is situated on the southwestern portion of the site and the one and two storey brick masonry dwelling (No. 4 Kunari Place) is situated on the northern portion of the site.

The one storey rendered dwelling (No. 94 Park Street) is estimated to be fairly new. The structure appears to be in good condition, with no signs of notable cracking or differential settlement. The front portion of the site comprises a concrete driveway and a combination of gravel and grassed surfaces towards the southeast. The remainder of the lot is predominantly occupied by the dwelling and a detached garage. A swimming pool and landscaped lawn area are located at the rear of the property.

The two-storey rendered dwelling (No. 96 Park Street) is also of new construction and appears to be in good condition, with no signs of cracks or settlement issues observed. The front south end of the property contains a concrete driveway along with a grass lawn near the southwestern boundary. The central portion of the property is occupied by the main structure, which is situated at a slightly lower elevation compared to the adjoining property at No. 94 Park Street. The rear section of the property comprises a grassed lawn and an in-ground swimming pool, both of which appear to be in good condition with no visible signs of structural distress. This property is retained above the level of Kunari Place by a steep vegetated embankment and a timber retaining wall.

No. 4 Kunari Place contains a one and two storey brick house is estimated to be approximately 80 years old that appears to be in good condition, with no signs of significant cracks or settlement issues observed. The front portion of the property features a concrete driveway exhibiting minor cracking, though no significant cracking was observed. Grass lawns are present on both sides of the driveway. The central part of the property is occupied by the structure, which is generally at a similar elevation to the adjoining properties, with a grassed area extending along the northern boundary. The rear of the site also comprises a gently sloping grass lawn.

Photograph 2: View of No. 96 Park Street front, facing northeast from Park Street.

Photograph 3: View of No. 96 Park Street rear, facing southeast from Park Street

Photograph 4: View of No. 94 Park Street front, facing northeast from Park Street

Photograph 5: View of No. 94a Park Street, facing broadly northeast

Photograph 6: View of No. 4 Kunari Place front, facing southeast from Kunari Place

Photograph 7: View of No. 4 Kunari Place rear, facing northwest from Kunari Place

The neighbouring properties to the east (No. 92 & 92A Park Street) contain one to two storey brick-masonry dwelling structures. The existing buildings are set back approximately 1.30 m and 1.73 m from the shared boundary with the site. Ground surface levels appear broadly consistent with those at the site along the boundary, and visible elements of the neighbouring structures appeared in good condition, with no obvious signs of significant cracking or settlement that would suggest existing geotechnical issues.

The neighbouring property to the north (No. 6 Kunari Place) contains a one and two storey brick masonry dwelling with a garage below the north end, similar to that in No. 4. The building is setback about 1.0 m from the shared boundary. Ground surface levels appear to be slightly lower than the site surface level (No. 4 Kunari Place) along the boundary, and the visible portions of the structure were observed to be in good condition, with no evident signs of significant cracking or settlement that would indicate any underlying geotechnical concerns.

The neighbouring property to the north (No. 167 Darley Street West) contains a one and two storey brick and timber dwelling with a garage below at the north end of the block with undeveloped slopes and bushland to the rear. The visible portions of the property showed no evidence any underlying geotechnical concerns.

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 large-scale slope instability or other major geotechnical concerns which would impact the site.

4.3. Ground Conditions:

Four boreholes (BH1 – BH4) were undertaken across site broadly in the envelope of proposed works with BH1 and BH2 drilled within the rear and front grass lawn of No. 96 Park Street, whilst BH3 was drilled within the front grass lawn of No. 94 Park Street and BH4 was drilled within rear grass lawn of No. 4 Kunari Place. The boreholes extended through a layer of topsoil/fill before intersected natural clay for the remainder of the investigation to a maximum of 2.95m depth before terminating on interpreted sandstone or interbedded sandstone-siltstone bedrock. Bedrock was encountered at variable depths across the site, reaching a maximum depth of 2.95m in the west, 2.10m in the south-east corner and 1.0m in the north-east.

Dynamic Cone Penetrometer (DCP) tests were undertaken from the ground surface adjacent to the borehole locations and three additional locations with solid refusal on interpreted bedrock of at least very low strength encountered at between 1.05m increasing to 2.70m, with the exception of DCP5, which terminated due to practical refusal in interpreted extremely weathered material.

For a description of the ground conditions encountered at the individual borehole/DCP test locations, the Borehole Log and DCP results sheets should be consulted however the subsurface conditions at the site can be summarised as follows:

- **TOPSOIL/FILL** – This layer was encountered in all boreholes from ground surface level extending to a maximum depth of 2.10m (BH1) and generally comprised a loose, dark brown, fine to medium grained, dry to moist silty sand fill near surface becoming dark grey silty clay with depth.
- **SILT –** This layer was encountered in two boreholes BH2 and BH3 beneath the overlying topsoil/fill layer, extending from approximately 0.15m to a maximum depth of 0.50m (BH3). The layer is characterized as light grey in colour, fine-grained, dry, and with ironstone gravel.
- **Silty CLAY –**Silty clay layer was encountered in all boreholes, generally underlying the silt or fill materials. In BH1, silty clay was recorded from 2.10m to 2.80m depth, described as hard, orange to brown, with medium plasticity and trace sand. BH2 and BH3 encountered silty clay starting from 0.50m and extending to depths of 1.4m and 2.10m respectively. These were described as very stiff, orange/red to grey mottled clay, with low plasticity and trace ironstone gravels. Mottling was common, with transitions from grey to orange/red indicating variable oxidation states and weathering.
- **Clayey SAND/Sandy CLAY –** Clayey sand and sandy clay layers were encountered above the bedrock in BH1 and BH4 indicating a sandstone composition. In BH1, clayey sand was recorded from 2.80m to 2.95m, described as orange/red, medium to coarse-grained, dry, with trace silt—indicative of residual soil over bedrock. In BH4, sandy clay occurred from 0.60 m to 1.0 m, noted as hard, orange/red, moist, with low plasticity and trace silt, grading into weathered rock. These layers mark the interface between overburden soils and underlying sandstone, reflecting partial weathering and mixed soil composition.

- **SANDSTONE/SILSTONE** –Bedrock of at least very low strength was interpreted in all boreholes, at depths ranging from 1.0 m (BH4) to 2.95 m (BH1). The bedrock was interpreted from refusal or termination conditions. It is expected to increase in strength with depth, grading from low to potentially medium strength. In BH4, weathered rock was observed as shallow as 0.95 m.

A free-standing groundwater table or significant seepage was not encountered in any of the boreholes.

5. LABORATORY TESTING:

5.1 Corrosion Potential:

Soil samples recovered from BH1, BH3 and BH4 were tested to determine the corrosion potential of the site soils to provide durability classification for new steel and concrete structures as per AS2159. The reported results are summarised below in Table 1 and Certificate of Analysis is attached in Appendix 3.

Table 1: Aggressivity Laboratory Test Results

Location	Depth (m)	pH	Electrical Conductivity (µS/cm)	Chloride Cl (mg/kg)	Sulphate SO4 (mg/kg)
BH1	2.5 - 2.6	5.4	70	58	82
BH3	0.5 – 0.6	5.7	96	58	95
BH4	0.4 – 0.5	6.7	51	10	20

*By calculation

The results of the soil chemical testing undertaken on the soil samples were compared against the Australian Standard AS 2159 – 2009 Pile Design and Installation.

The results were compared against Table 6.4.2 (C) Exposure Classification for Concrete Piles – Piles in Soil. The results indicate that the soils at 2.50m depth within BH1 are '**non-aggressive'** to concrete from sulphate and chlorides, and **'mild'** from pH. The results indicate that the soils at 0.50m depth within BH3 are **'non-aggressive'** to concrete from sulphate and chlorides and pH. The results indicate that the soils at 0.40m depth within BH4 are **'non-aggressive'** to concrete from sulphate and chlorides and pH.

The results were also compared against Table 6.5.2 (C) Exposure Classification for Steel Piles – Piles in Soil. The results indicate that the soil at 2.50m depth within BH1 are **'non-aggressive'** to steel with regards to pH, and chloride. The results that indicate that the soils at 0.50m depth within BH3 is **'non-aggressive'** to steel with regards to pH and chlorides. The results indicate that the soil at 0.40m depth within BH4 are **'non-aggressive'** to steel with regards to pH, and chloride.

5.2. Acid Sulphate Soils Testing:

Six samples were tested to determine pH, pHFox and to assess reaction rates to a hydrogen peroxide solution, the results/observations are provided in the table below. Two of the samples were also analysed 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 the table below, together with the reaction observed during the test. A summary of the Chromium test results is provided in the *'Chromium Test Results'* table below. Envirolab calculated liming rates for the neutralisation of each sample tested using the Chromium method, based on the use of good quality, fine ag-lime, with a neutralising value of 100% and incorporating a factor of safety of 1.5 is also provided. Envirolab Certificate of Analysis is included in Appendix: 3.

Table 2: Summary of Peroxide field testing results:

Borehole	Depth (m)	pH field	pHFOX field peroxide test	Reaction Rate
BH1	2.4 – 2.5	5.9	5.7	Low
BH1	2.5 – 2.6	5.0	4.9	Low
BH1	2.85 – 2.95	5.4	5.3	Low
BH2	0.15 - 0.25	6.2	5.4	Low
BH3	0.5 - 0.6	5.9	5.7	Low
BH4	0.4 – 0.5	6.6	5.8	Low

Table 3: Chromium Suite Laboratory Test Results +%S w/w

Borehole	Depth (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)
BH1	2.4 – 2.5	6.0	<0.01	<0.005	0.0060	<0.75
BH1	2.85 – 2.95	5.2	0.01	0.01	0.023	1

Note Results in **Bold** exceed the Acid Sulphate Soils Advisory Committee (ASSMAC) Action Criteria for disturbance of <1000 tonnes of soil (refer to Section 4.3)

5.3. Moisture Content

Seven soil samples were tested for measurement of field moisture content in accordance with Australian Standard AS 1289 and the results are summarised in Table 5.

Samples were transported to a NATA accredited laboratory (Macquarie) for analysis and the test report is attached in Appendix 3.

Table 4: Summary of Reported Moisture Content Results

Sample Location and Depth (m)	Sample Description	Moisture Content (%)
BH1 2.40m	Silty CLAY	21.8
BH1 2.85m	Silty Sandy CLAY	13.5
BH2 0.40m	Silty Sandy CLAY	14.5
BH3 0.50m	Silty CLAY	21.6
BH3 1.10m	Silty CLAY	16.4
BH4 0.40m	Silty CLAY	13.6
BH4 0.70m	Silty CLAY	18.3

5.4. Atterberg Limits

One soil sample was tested for measurement of Atterberg limit in in accordance with Australian Standard AS 1289 and the results are summarised in Table 6.

Samples were transported to a NATA accredited laboratory (Macquarie) for analysis for classification of fine grained soils as per AS 1726:2017 Clause 6.1.6 the result sheet is attached in Appendix 3.

Table 6: Summary of Reported Moisture Content Results

Location and Depth	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index / PI (%)
BH1 2.40m	43	16	27

6. COMMENTS:

6.1. Geotechnical Assessment:

The site investigation encountered a topsoil/fill layer in all boreholes, extending from the surface to depths ranging between 0.15 m and 2.10 m (BH1) with the increased fill depth seen on the west side of No. 96 due to apparent previous raising above reserve levels to create a level building platform for the existing dwelling. This layer generally consisted of loose, dark brown to dark grey, fine to medium grained, dry to moist silty sand and then silty clay fill, with ironstone gravels.

Beneath the fill, different soils were identified, including silt and silty clay, commencing from depths as shallow as 0.15 m (BH2–BH4) and extending to a maximum depth of 2.95m in BH1. These soils varied in composition, strength, and moisture condition across the site and were underlain by sandstone and/or interbedded sandstone/siltstone bedrock of at least very low strength, encountered between 1.0m (BH4) and 2.95m depth (BH1).

A groundwater table was not encountered to the base investigation level and is not expected above RL 1.0 based on the site location and local topography, however seepage was encountered and is expected across the bedrock surface as well as along defects in the bedrock mass. The investigation, site location and topography indicate seepage is unlikely to be significant but requires deeper investigation to confirm.

The proposed works involve demolition of existing structures, and the construction of new three storey apartment buildings formed over a singular level double basement carpark. The proposed basement finished floor level will be situated at RL2.75 hence will require a bulk excavation to a maximum of approximately 9.75m depth.

The bedrock of the local area is anticipated from previous experience to be a combination of low strength sandstone and very low strength siltstone initially, grading to low and medium strength with depth. However, it is known to contain areas of increased weathering, strength inversions (hard rock over weak) and steeply inclined defects.

Based on investigation results and expected geological conditions safe batter slopes as per Section 5.3.2 of this report appear to be achievable in the north-west parts of the site only, therefore pre-excavation support systems will be required. These systems should comprise a soldier bored pile support wall, founded into the bedrock, with shotcrete lining applied to the pile face for excavation support. Pending the depth of the swimming and plunging pools excavations temporary support systems may be required in these locations. Preliminary design parameters are provided in **Section 5.3.3**.

Fill, natural soils and very low strength bedrock can be excavated using conventional earthmoving equipment (e.g. buckets and rippers) whilst low strength siltstone can be excavated by heavy ripping, however low to medium strength sandstone bedrock will require the use of the rock breaking equipment (e.g. rock hammers). The use of rock hammers can create ground vibrations which could damage the adjacent structures if unsuitable sizes and methods are used. Care will be required during the demolition and excavation and construction works to ensure the neighbouring properties and potential structures/services are not adversely impacted by ground vibrations.

Small scale equipment (i.e. rock hammer <300kg) along with rock saw and a good excavation methodology can be used to maintain low vibration levels at boundaries and avoid the need for full time vibration monitoring. However, this will result in slow excavation progress, and it is anticipated that larger scale rock hammers will be preferred. As such Crozier Geotechnical Consultants (CGC) should be consulted regarding the size and type of excavation equipment proposed and excavation methodology prior to works.

Where medium strength bedrock with no poorly oriented defects is identified by further investigation, it will likely be free standing and can be excavated near vertically without the need for additional support measures. Where defects are encountered additional support may be required (i.e. rock bolts) to maintain stability. Confirmation of rock strength/conditions prior to excavation requires cored boreholes, drilled to confirm the sub-surface conditions (i.e. if any weak zones of rock are identified) prior to final structural design.

The proposed excavation is anticipated to intersect bedrock across its majority at foundation level, therefore its entire base should be founded to bedrock of similar strength to avoid differential settlement.

The laboratory testing results, assessed as per AS2159 for concrete piles in soil, indicate that the soils are classified as ‘non-aggressive’ to concrete with respect to sulphate, chloride, and pH levels, with a ‘mild’ pH condition noted only at 2.50 m depth in BH1.

The laboratory test results also indicate that soils at 2.50 m (BH1), 0.50 m (BH3), and 0.40 m (BH4) are classified as ‘non-aggressive’ to steel in relation to pH and chloride levels.

Based on the site investigation, a groundwater table is not anticipated within the proposed excavation however, minor seepage is likely to be encountered during the excavation principally at the bedrock surface and on defects in the rock. Initial investigation results indicate this can be managed as a drained basement with negligible impact on local hydrogeology.

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 a limited scope of investigation using augering techniques only. This investigation provides limited data from small, isolated test points across the entire site with limited penetration into rock, therefore some 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 assessment and subsequent initial design of the proposed works.

6.2. 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. Landslide (earth slide <10m^3) from soils at crest of the excavation
B. Landslide (rock and earth slide <20m^3) due to instability within the bedrock

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 **2.60 x 10^{-6},** whilst the Risk to Property was considered to be **up to 'Moderate'**. The Risk to Life from Hazard B was also estimated to be up to **2.60 x 10^{-6},** and the Risk to Property up to **'Moderate'**.

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.

6.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

6.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Class 'A' for footings founded into bedrock at base of excavation Class P for shallow footings
Type of Footing	Strip, Pad, or Piers
Sub-grade material and Maximum Allowable Bearing Capacity for shallow footings	- Very Stiff clay: 300 kPa - Stiff clay: 400 kPa - Very Low Strength Bedrock: 700kPa - Low Strength bedrock: 1000kPa
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	Be – rock site The hazard factor (z) for Sydney is 0.08.
Remarks: All footings should be founded off material of similar foundation conditions to prevent differential settlement, allowance for differential settlement should be designed for if the structure is variably founded. 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.	

<table>
<tr><th colspan="5">6.3.2. Excavation:</th></tr>
<tr><td colspan="2">Depth of Excavation</td><td colspan="3">Maximum of approximately 9.75m depth for the proposed Basement level.</td></tr>
<tr><td colspan="5">Property Separation:</td></tr>
<tr><th rowspan="2">Boundary</th><th rowspan="2">Adjacent Property</th><th rowspan="2">Bulk Excavation Depth</th><th colspan="2">Separation Distances</th></tr>
<tr><th>Boundary</th><th>Structure</th></tr>
<tr><td>East</td><td>No. 92 & 92A Park Street</td><td>9.75m to 6.10m for lower basement level.</td><td>3.18m and 4.03m</td><td>Structures are +1.3m away and +1.73m away.</td></tr>
<tr><td rowspan="2">West</td><td rowspan="2">Kunari Pace</td><td>8.00m to 2.00m for lower basement level.</td><td>3.25m to 5.76m</td><td rowspan="2">Nature strip adjacent to the boundary and at lower level, road pavement a further 4.3m away.</td></tr>
<tr><td>Up to 2.2m approximate depth for swimming pool.</td><td>1.67m</td></tr>
</table>

North	No. 6 Kunari Place	2.00m to 5.00m for lower basement level.	6.50m	Structure is +1.0m away.
		Up to 2.2m approximate depth for plunging pool.	1.70m	
	No. 167 Darley Street West	4.50m to 6.10m for lower basement level.	12.40m	Structures +40m
South	Park Street	Up to 9.75m for lower basement level.	6.50m	Nature strip and Public pathway adjacent to the boundary, road pavement a further 4.9m away.

Type of Material to be Excavated	Fill/topsoil to ≤2.10m depth
	Residual soils to extremely weathered bedrock to a maximum of 2.95m depth
	Very low to medium strength bedrock from a minimum of 1.0m depth across east side.

Guidelines for unsurcharged batter slopes are tabulated below:

Material	Safe Batter Slope (H: V)	
	Short Term/Temporary	Long Term/Permanent
Fill/topsoil	1.5:1	2:1
Residual soils to EW material	1:1*	1.5:1*
VLS – LS bedrock (fractured)	0.25:1.0*	0.50:1.0*
MS, defect free bedrock	Vertical	Vertical*

Remarks:

*Dependent on assessment by geotechnical engineer.

Batter slopes in soils should be ≤3.0m in height without benching and where utilised will require regular geotechnical assessment and protection from saturation

Seepage at the bedrock surface or along defects in the soil/rock 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	Fill	Excavator with bucket
	VLS bedrock	Excavator with bucket and ripper
	LS-MS/HS bedrock	Rock hammer and rock saw

VLS – very low strength, LS – low strength, MS – medium strength

Remarks:

Rock sawing of the hard rock excavation perimeters is recommended as it has several advantages. It often reduces the need for rock bolting as the cut faces generally remain more stable and require a lower level of rock support than hammer cut excavations, ground vibrations from rock saws are minimal and the saw cuts will provide a slight increase in buffer distance for use of rock hammers. It also reduces deflection across boundary of detached sections of bedrock near surface.

Based on previous testing of ground vibrations created by various rock excavation equipment within medium strength bedrock, to achieve a low level of vibration (5mm/s PPV) the below hammer weights and buffer distances are generally required:

Maximum Hammer Weight	Required Buffer Distance from Structure
300kg	3.00m
400kg	4.00m
600kg	6.00m
≥1 tonne	20.00m

Onsite calibration will provide accurate vibration levels to the site specific conditions and will generally allow for larger excavation machinery or smaller buffers to be used. Inspection of equipment and review of dilapidation surveys and excavation location is necessary to determine need for full time monitoring.

Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	No. 92 & 92a Park Street = 5mm/s No. 6 Kunari Place = 5mm/s
Vibration Calibration Tests Required	If larger scale (i.e. rock hammer >300kg) excavation equipment is proposed
Full time vibration Monitoring Required	Pending proposed excavation equipment and vibration calibration testing results, if required
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • During pile support wall drilling • For assessment of batter slopes • At 1.50m depth intervals of excavation • 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 Requirement	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.

Remarks:

Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope, whilst any groundwater seepage must be controlled within the excavation and prevented from ponding or saturating slopes/batters.

6.3.3. Retaining Structures:

Required	New retaining structures/excavation support wall will be required as part of the proposed development to support the excavation perimeters due to the soils and anticipated weaker nature of the bedrock.
Types	Pre-excavation soldier pile shoring wall is required around the north, east and south portions of the excavation perimeter and in parts of the western side, with shotcrete lining to be applied to the pile face. Steel reinforced concrete/concrete block post excavation where batters possible. Designed in accordance with Australian Standards AS4678-2002 Earth Retaining Structures.

Parameters for calculating un-surcharged 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	$\phi' = 30°$	N/A	0.5	N/A
Clay (very stiff to hard)	20	$\phi' = 35°$	0.27	0.40	N/A
VLS bedrock	22	$\phi' = 38°$	0.15	0.20	200kPa
LS bedrock	23	$\phi' = 40°$	0.10	0.15	400kPa

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 utilise active earth pressure coefficients (Ka). For cantilever or simple anchor support walls a triangular pressure distribution can be utilised for at least initial design (pending results of further investigation).

6.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		No
Excavation likely to intersect	Water Table	No
	Seepage	Minor (<3L/min) estimated
Site Location and Topography		East side of Kunari Place within gentle north-west dipping and relatively low lying topography.
Impact of development on local hydrogeology		Appears negligible
Onsite Stormwater Disposal		Not required or possible
Remarks: 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. Current investigation results indicate a drained basement will be suitable, this requires confirmation through more detailed investigation prior to final engineering design.		

6.4. Conditions Relating to Design and Construction Monitoring:

To allow certification at the completion of the project it will be necessary for Crozier Geotechnical Consultants to:

1. Undertake further detailed site investigation for provision of detailed engineering design and construction recommendations.
2. Review the structural design drawings, including the retaining structure design and construction methodology, for compliance with the recommendations of this report prior to construction,
3. Inspect the installation of excavation support measures,
4. Inspect any medium strength bedrock and the proposed equipment prior to its excavation and at 1.50m depth intervals of excavation,
5. Inspect all new footings to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and stability prior to the placement of steel or concrete,
6. Inspect completed works to confirm all slope stability, retention and stormwater systems are completed.

The client and builder should make themselves familiar with the requirements spelled out in this report for inspections during the construction phase. Crozier Geotechnical Consultants cannot provide certification for the Occupation Certificate if it has not been called to site to undertake the required inspections.

6.5. Design Life:

We have interpreted the design life requirements specified within Councils Risk Management Policy to refer to structural elements designed to support the adjacent slope, 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 development are considered to comprise:

- stormwater and subsoil drainage systems,
- retaining walls and soil slope erosion 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 (50 years)). In order to attain an “Acceptable Risk Management Criteria” for a design life of 100 years as detailed by the Councils Risk Management Policy, it will be necessary for the property owner to adopt and implement a maintenance and inspection program. If a maintenance and inspection schedule are not implemented the “Acceptable” risk levels for the design life of the property may not be attained.

A recommended program is given in Table: 1 below 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 new development.
- There is no change to the property due to an extraordinary event external to this site, and 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). It is assumed that Pittwater 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 levels or landslide potential.

7. CONCLUSION:

The site investigation identified topsoil/fill extending to a maximum depth of 2.10 m (BH1), overlying silt and silty clay layer across the site. Clayey sand and sandy clay were recorded above the sandstone interface in BH1 and BH4. Sandstone bedrock of at least very low strength was encountered between 1.0 m and 2.95 m depth, with strength expected to increase with depth.

The proposed works consist of the demolition of existing structures and the construction of two new three storey apartment building formed over a double basement level carpark. The proposed structure will require bulk excavation to a maximum of approximately 9.75m.

It appears that safe batter slopes will only be achievable for the proposed excavations, as outlined in Section 5.3.2 of this report at the north to north-west corner of the site. Therefore for the majority of the proposed development, pre-excavation support systems will be required. The detailed suitability of the rock excavation faces to remain unsupported will depend on the condition of the bedrock, which requires further geotechnical investigation.

Where a pre-excavation retention system is required, this will be most effectively achieved using a soldier bored pile wall with shotcrete lining applied to the face to prevent collapse of surrounding sands. Piles should extend below the excavation level and be socketed into bedrock of at least low strength. Preliminary design parameters are provided in Section 5.3.3, and further geotechnical investigation is reuqired.

Based on the site investigation, a groundwater table is not anticipated within the proposed excavation however, minor seepage is likely to be encountered during the excavation at the bedrock surface and on details in the rock mass. Therefore, it is unlikely to be a significant issue for this development, and tanking and large-volume dewatering are unlikely to be critical hazards.

The entire site and surrounding slopes have been assessed as per the Pittwater Council's LEP Geotechnical Risk Management Policy 2009 and can achieve the "Acceptable" risk management criteria of the policy for the design life of the development, taken as 100 years, provided the recommendations of this report are implemented in the construction phase whilst the maintenance program is implemented. As such the site is considered suitable for the proposed development.

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	MAIG, RPGeo – Geotechnical and Engineering

8. REFERENCES:

1. Australian Standard AS1170.4 - 2007 Earthquake Actions;
2. Australian Standard AS 1289 – 2000, Method of Testing Soils for Engineering Purposes;
3. Australian Standard AS 1726: 2017, Geotechnical Site Investigations.
4. Australian Standard AS 2870: 2011, Residential Slabs and Footings.
5. Australian Standard AS3600:2009, Concrete Structures.
6. Australian Standard AS3798:2007, Guidelines on Earthworks for Commercial and Residential Developments.
7. Australian Standard AS 4678:2002, Earth-Retaining Structures.
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CROZIER
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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: -

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

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

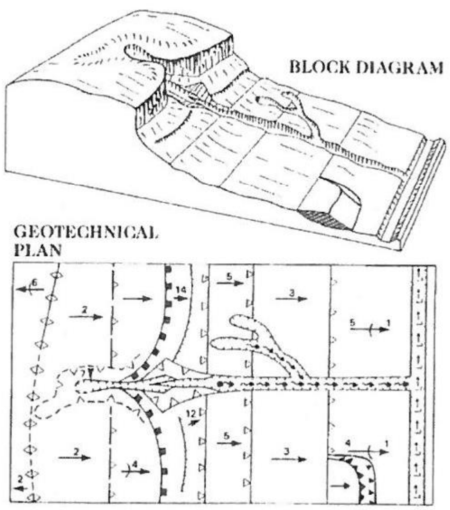
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

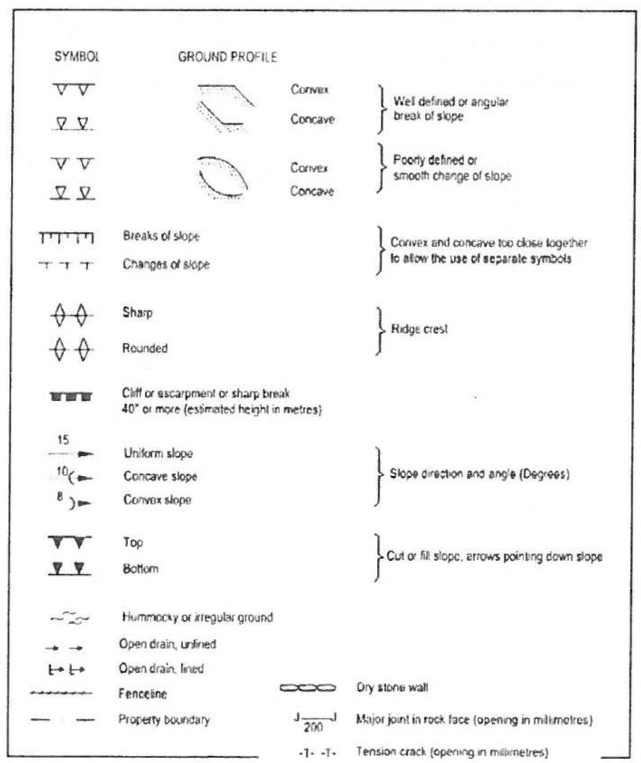
Reproduction of Information for Contractual Purposes

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.

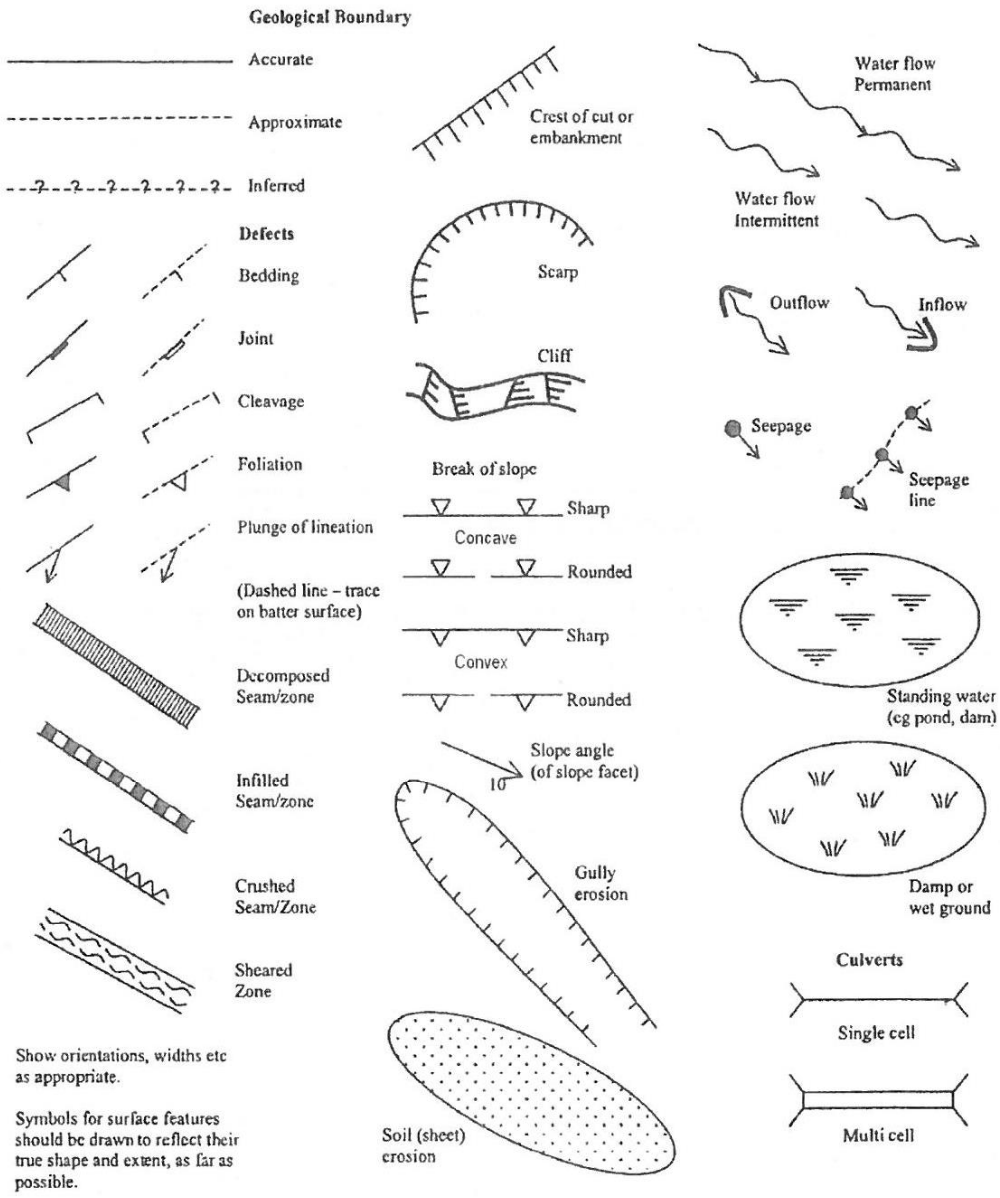




Example of Mapping Symbols

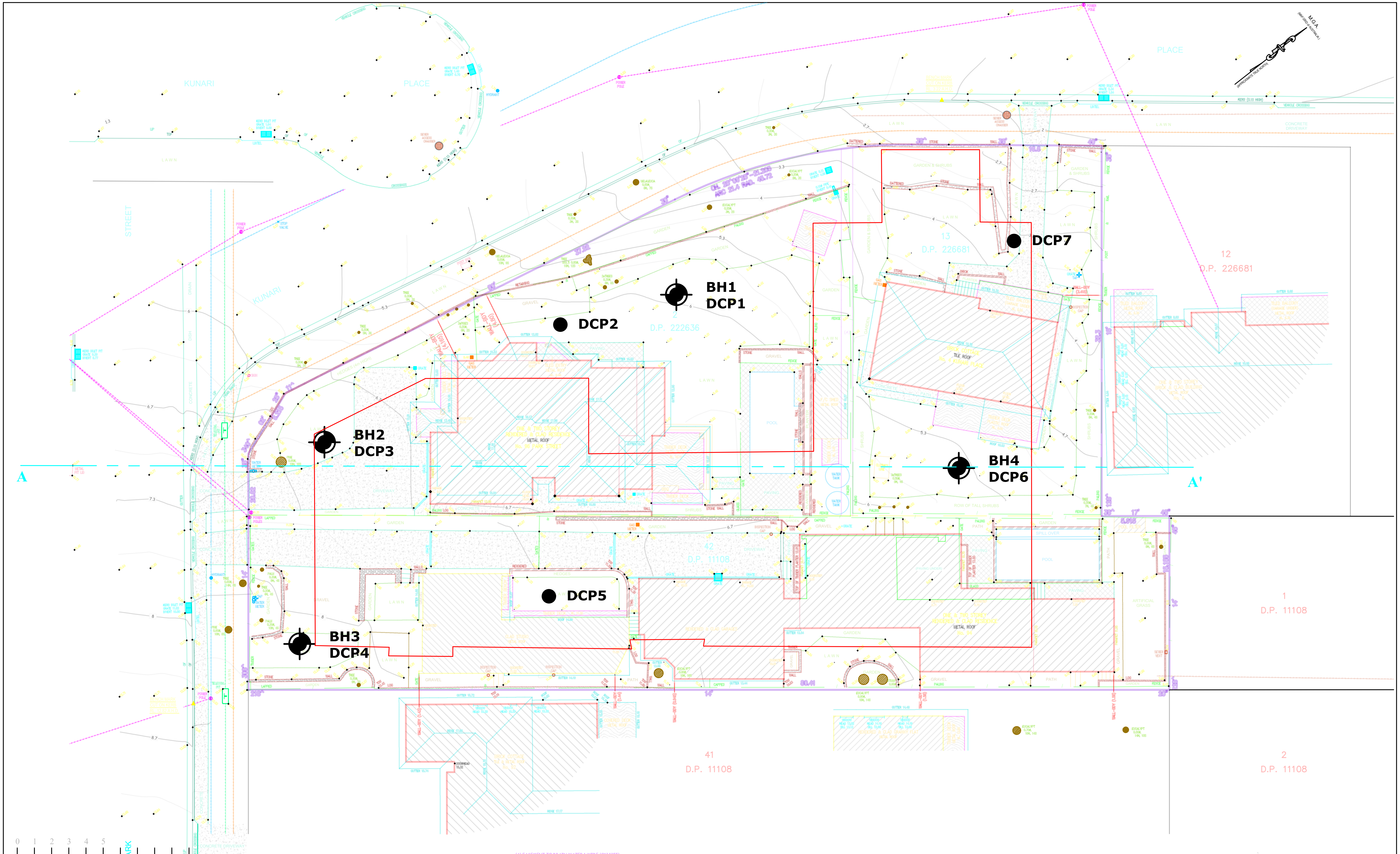
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

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).

Appendix 2



SITE PLAN & TEST LOCATIONS FIGURE 1.

CROZIER
GEOTECHNICAL CONSULTANTS

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

LEGEND

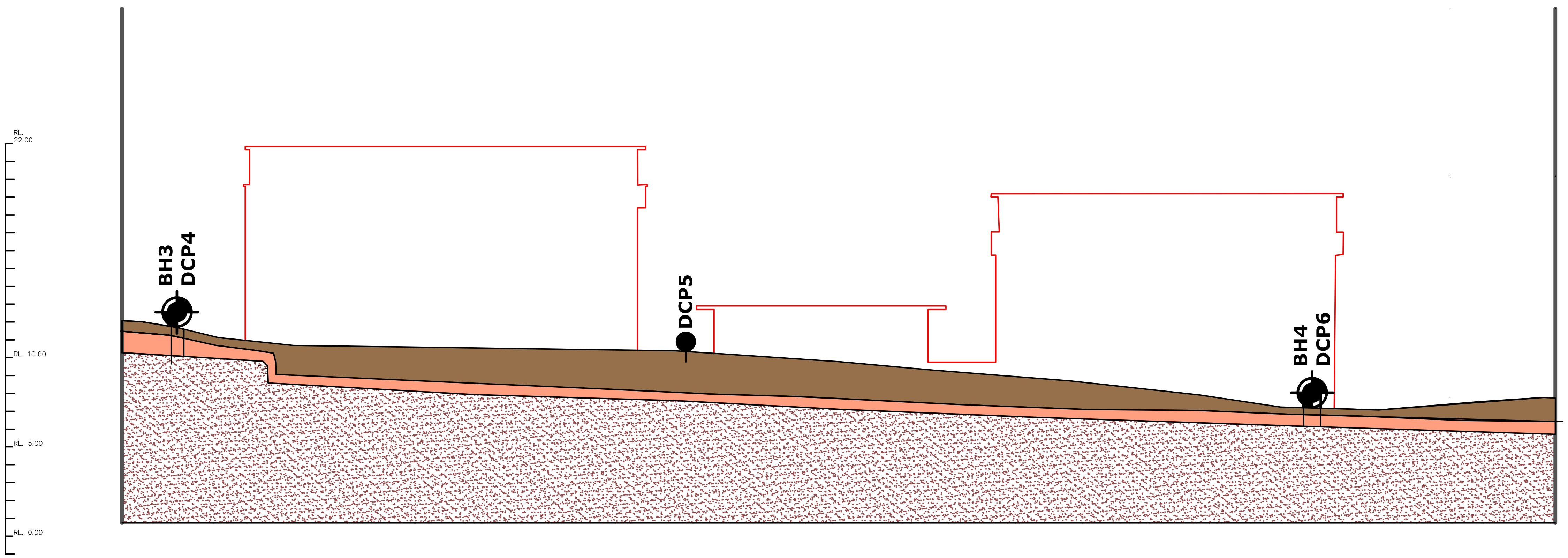
PROPOSED WORKS	EXISTING STRUCTURES
PROPERTY BOUNDARY	BH DCP — AUGER / DYNAMIC CONE PENETROMETER LOCATION
DCP — DYNAMIC CONE PENETROMETER	A — A' CROSS-SECTION REFERENCE LINE

SCALE:	1:150 @ A3
DRAWING:	FIGURE 1
DATE:	04/04 /2025
APPROVED BY:	TMC
DRAWN BY:	SB
PROJECT:	2025-047

PREPARED FOR:
WALSH ARCHITECTS

ADDRESS:
94-96 Park Street, Kunari Place, Mona Vale

A A'



SECTION A: FIGURE 2

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

LEGEND

PROPOSED WORKS

BH DCP AUGER / DYNAMIC CONE PENETROMETER LOCATION

DCP DYNAMIC CONE PENETROMETER

FILL

CLAY

SANDSTONE BEDROCK

A —— A' SECTION LINE

SCALE:	1:150 @ A3
DRAWING:	FIGURE 2
DATE:	04/04/2024
APPROVED BY:	TMC
DRAWN BY:	SB
PROJECT:	2025-046

PREPARED FOR:
WALSH ARCHITECTS

ADDRESS:
94-96 Park Street, Kunari Place, Mona Vale

BOREHOLE LOG

CLIENT: Mona Vale Central Pty Ltd **DATE:** 25/04/2025 **BORE No.:** 1

PROJECT: Demolition of existing structures and construction of new three storey apartment buildings **PROJECT No.:** 2025-047 **SHEET:** 1 of 1

LOCATION: 94-96 Park Street, Kunari Place, Mona Vale **SURFACE LEVEL:**

Depth (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
0.00		TOPSOIL/FILL: Loose, dark brown, fine-medium grained, dry to moist, silty sand fill				
0.50		…becoming dark grey, silty clay fill with ironstone gravels				
1.00						
2.00						
2.10		Silty CLAY: Hard, orange/brown, medium plasticity, dry to moist, trace sand				
2.50		…grey mottled	D at 2.4-2.5m			
2.70		..grey mottled orange	D at 2.5-2.7m			
2.80		Clayey SAND: Very dense, orange/red, medium-coarse grained, dry, with silt	D at 2.85-2.95m			
2.95		END OF BOREHOLE at 2.95m depth ontop of at least very low strength sandstone/siltstone interpreted as bedrock				
4.00						
5.00						

RIG: K9-4 Dingo mini-digger with Ezi-probe drill mast DRILLER: A.C.

METHOD: Solid Stem Spiral Flighted Auger with Tungsten Carbide Bit LOGGED: S.B.

GROUND WATER OBSERVATIONS: Not encountered

REMARKS: CHECKED: T.M.C

BOREHOLE LOG

CLIENT: Mona Vale Central Pty Ltd

DATE: 25/04/2025

BORE No.: 2

PROJECT: Demolition of existing structures and construction of new three storey apartment buildings

PROJECT No.: 2025-047

SHEET: 1 of 1

LOCATION: 94-96 Park Street, Kunari Place, Mona Vale

SURFACE LEVEL:

Depth (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
0.00		TOPSOIL/FILL: Loose, dark brown, fine-medium grained, dry to moist, silty sand fill				
0.15		SILT: Medium dense, light grey, fine grained, dry, trace iron stone gravels	D at 0.15-0.25m			
0.40		Silty CLAY: Very stiff, orange/red, low plasticity, dry, trace ironstone gravels				
0.50		…hard				
1.00						
1.30		…orange/red mottled grey	D at 1.3-1.4m			
1.40		END OF BOREHOLE at 1.4m depth on top of at least very low strength sandstone/siltstone interpreted as bedrock				
2.00						
3.00						
4.00						
5.00						

RIG: K9-4 Dingo mini-digger with Ezi-probe drill mast

DRILLER: A.C.

METHOD: Solid Stem Spiral Flighted Auger with Tungsten Carbide Bit

LOGGED: S.B.

GROUND WATER OBSERVATIONS: Not encountered

REMARKS:

CHECKED: T.M.C

BOREHOLE LOG

CLIENT: Mona Vale Central Pty Ltd **DATE:** 25/04/2025 **BORE No.:** 3

PROJECT: Demolition of existing structures and construction of new three storey apartment buildings **PROJECT No.:** 2025-047 **SHEET:** 1 of 1

LOCATION: 94-96 Park Street, Kunari Place, Mona Vale **SURFACE LEVEL:**

Depth (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	
			Type	Tests	Type	Results
0.00						
0.15		TOPSOIL/FILL: Loose, dark brown, fine-medium grained, dry to moist, silty sand fill				
0.50		SILT: Medium dense, light grey, fine grained, dry, with iron stone gravels				
		Silty CLAY: Hard, orange/red, low plasticity, dry to moist, with silt	D at 0.5-0.6m			
1.00						
1.10		...very stiff, grey mottled orange/red	D at 1.1-1.2m			
1.40		...hard				
1.50		...dry				
2.00		...orange/red mottled grey				
2.10						
		END OF BOREHOLE at 2.1m depth ontop of at least very low strength sandstone/siltstone interpreted as bedrock				
3.00						
4.00						
5.00						

RIG: K9-4 Dingo mini-digger with Ezi-probe drill mast DRILLER: A.C.

METHOD: Solid Stem Spiral Flighted Auger with Tungsten Carbide Bit LOGGED: S.B.

GROUND WATER OBSERVATIONS: Not encountered

REMARKS: CHECKED: T.M.C

BOREHOLE LOG

CLIENT: Mona Vale Central Pty Ltd | **DATE:** 25/04/2025 | **BORE No.:** 4

PROJECT: Demolition of existing structures and construction of new three storey apartment buildings | **PROJECT No.:** 2025-047 | **SHEET:** 1 of 1

LOCATION: 94-96 Park Street, Kunari Place, Mona Vale | **SURFACE LEVEL:**

Depth (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	
			Type	Tests	Type	Results
0.00		TOPSOIL/FILL: Loose, dark grey, fine-medium grained, dry to moist, silty sand fill				
0.30		..increase in silt, with iron stone gravels				
0.40		Silty SAND: Loose, orange, fine-medium grained, moist, with clay and gravels	D at 0.4-0.5m			
0.60		Sandy CLAY: Hard, orange/red, low plasticity, moist, with silt				
0.80		…very stiff, light orange	D at 0.7-0.8m			
0.95		…weathered rock				
1.00		END OF BOREHOLE at 1.0m depth ontop of at least very low strength sandstone interpreted as bedrock				
2.00						
3.00						
4.00						
5.00						

RIG: K9-4 Dingo mini-digger with Ezi-probe drill mast | DRILLER: A.C.

METHOD: Solid Stem Spiral Flighted Auger with Tungsten Carbide Bit | LOGGED: S.B.

GROUND WATER OBSERVATIONS: Not encountered

REMARKS: | CHECKED: T.M.C

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Lucas Laxale **DATE:** 25/04/2025

PROJECT: Demolition of existing structures and construction of new three storey apartment buildings

LOCATION: 94-96 Park Street, Kunari Place, Mona Vale **SHEET:** 1 of 1

	Test Location									
Depth (m)	DCP1	DCP2	DCP3	DCP4	DCP5	DCP6	DCP7			
0.00 - 0.10	1	2	1	2	3	2	3			
0.10 - 0.20	5	9	4	5	1	3	6			
0.20 - 0.30	4	8	5	4	7	3	5			
0.30 - 0.40	2	10	5	9	13	2	7			
0.40 - 0.50	2	13	16	17	26	15	5			
0.50 - 0.60	2	14	14	29	30	11	5			
0.60 - 0.70	2	13	10	11	STOP at 0.6m	5	4			
0.70 - 0.80	6	11	9	8		5	4			
0.80 - 0.90	3	15	9	8		4	9			
0.90 - 1.00	3	17	12	7		4	8			
1.00 - 1.10	2	10	12	7		10	8			
1.10 - 1.20	4	11	10	7		B at 1.05m	5			
1.20 - 1.30	4	19	16	7			6			
1.30 - 1.40	5	11	B at 1.3m	8			8			
1.40 - 1.50	9	11		11			12			
1.50 - 1.60	6	12		13			25			
1.60 - 1.70	6	13		B at 1.5m			20			
1.70 - 1.80	7	14					17			
1.80 - 1.90	9	18					22			
1.90 - 2.00	14	28					STOP at 1.9m			
2.00 - 2.10	10	20								
2.10 - 2.20	11	STOP at 2.1m								
2.20 - 2.30	10									
2.30 - 2.40	13									
2.40 - 2.50	15									
2.50 - 2.60	15									
2.60 - 2.70	17									
2.70 - 2.80	B at 2.7m									
2.80 - 2.90										
2.90 - 3.00										
3.00 - 3.10										
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:

- **--** No test undertaken at this level due to prior excavation of soils
- **(B)** Test hammer bouncing upon refusal on a solid object
- **(PR)** Pratical Refusal: continuous 3 intervals with ≥15 Blows/100mm or a single interval with ≥25 Blows
- **(SW)** Rod settlled due to the self-weight of equipment

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	Landslide (earth slide <10m³) from soils at crest of the excavation		Appears 1.0 to 3.0 metres of soil, no signs of existing movement, up to 9.75m deep excavation	a) dwelling >4.50m from east of 2.10m deep excavation in soils, negligible impact b) dwelling is ≥5.70m from side of 1.50m deep excavation in soils, negligible impact c) lawn and garden ≥3.20m from excavation, impact 5% d) dwelling is 7.50m from side of excavation of ≤1.50m deep excavation in soils, negligible impact e) lawn and garden 6.50m from excavation, impact 5% f) footpath 6.50m from ≤2.10m deep excavation in soils, impact 50% of pathway g) road pavement is 10.0m from edge of ≤2.10m deep soil excavation, negligible impact h) road pavement is ≥8.0m from edge of ≤3.00m deep excavation in soils, negligible impact		a) Person in house 20hrs/day avge. b) Person in house 20hrs/day avge. c) Person in garden 1hr/day avge. d) Person in house 20hrs/day avge. e) Person in garden 0.25hr/day avge. f) Person in footpath 0.50hr/day avge. g) Person in vehicle, 1hr/day h) Person in vehicle, 1hr/day .	a) Likely to not evacuate b) Likely to not evacuate c) Unlikely to not evacuate d) Likely to not evacuate e) Unlikely to not evacuate f) Possible to not evacuate g) Likely to not evacuate h) Likely to not evacuate	a) Person in building, minor damage only b) Person in building, minor damage only c) Person in open space, buried d) Person in building, minor damage only e) Person in open space, buried f) Person in open space, buried g) Person in vehicle, damage only h) Person in vehicle, damage only	
			Likely	**Prob. of Impact**	**Impacted**				
		a) House No. 92 Park Street	0.01	0.05	0.01	0.83	0.75	0.05	**1.56E-07**
		b) House 92a Park Street	0.01	0.05	0.01	0.83	0.75	0.05	**1.56E-07**
		c) Lawn, garden, driveways - No. 92 or 92a Park Street	0.01	0.25	0.05	0.04	0.25	1.00	**1.30E-06**
		d) House No. 6 Kunari Place	0.01	0.01	0.01	0.83	0.75	0.05	**3.13E-08**
		e) Lawn and garden No. 6 Kunari Place	0.01	0.10	0.05	0.01	0.25	1.00	**1.30E-07**
		f) Footpath - Park Street	0.01	0.10	0.25	0.02	0.5	1.00	**2.60E-06**
		g) Road Pavement - Park Street	0.01	0.01	0.01	0.04	0.75	0.05	**1.56E-09**
		h) Road Pavement - Kunari Place	0.01	0.01	0.01	0.04	0.75	0.05	**1.56E-09**
B	Landslide (rock and earth slide <20m³) due to instability within the bedrock		Bedrock relatviely shallow however anticpated to be interbedded silstone/sandstone, large scale defects possible	a) dwelling 4.50m from east of 9.75m deep excavation, impact western edge only b) dwelling is ≥5.7m from side of excavation of 7.25m deep, impact western edge only c) lawn and garden 3.20m from 9.75m deep excavation, impact 20% d) dwelling is 7.50m from side of excavation of ≤4.75m deep, impact south edge only e) lawn and garden 6.50m from excavation of ≤4.75m deep, impact 25% f) footpath 6.50m from ≤9.75m deep excavation, impact 100% of pathway g) road pavement is 10.0m from edge of ≤9.75m deep excavation, impact 5% h) road pavement is ≥8.0m from edge of ≤6.25m deep excavation, impact 5%		a) Person in house 20hrs/day avge. b) Person in house 20hrs/day avge. c) Person in garden 1hr/day avge. d) Person in house 20hrs/day avge. e) Person in garden 0.25hr/day avge. f) Person in footpath 0.50hr/day avge. g) Person in vehicle past site, 1hr/day h) Person in vehicle past site, 1hr/day .	a) Likely to not evacuate b) Likely to not evacuate c) Possible to not evacuate d) Likely to not evacuate e) Unlikely to not evacuate f) Possible to not evacuate g) Likely to not evacuate h) Likely to not evacuate	a) Person in building, minor damage only b) Person in building, minor damage only c) Person in open space, buried d) Person in building, minor damage only e) Person in open space, buried f) Person in open space, buried g) Person in vehicle, damage only h) Person in vehicle, damage only	
			Possible	**Prob. of Impact**	**Impacted**				
		a) House No. 92 Park Street	0.001	0.25	0.10	0.83	0.75	0.10	**1.56E-06**
		b) House 92a Park Street	0.001	0.20	0.10	0.83	0.75	0.10	**1.25E-06**
		c) Lawn, garden, driveways - No. 92 or 92a Park Street	0.001	0.50	0.25	0.04	0.50	1.00	**2.60E-06**
		d) House No. 6 Kunari Place	0.001	0.01	0.05	0.83	0.75	0.05	**1.56E-08**
		e) Lawn and garden No. 6 Kunari Place	0.001	0.10	0.25	0.01	0.25	1.00	**6.51E-08**
		f) Footpath - Park Street	0.001	0.20	1.00	0.02	0.50	1.00	**2.08E-06**
		g) Road Pavement - Park Street	0.001	0.05	0.05	0.04	0.75	0.05	**3.91E-09**
		h) Road Pavement - Kunari Place	0.001	0.05	0.05	0.04	0.75	0.05	**3.91E-09**

* hazards considered in current condition and/or without remedial/stabilisation 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
* neighbouring houses considered for bedroom impact unless specified
* considered for person most at risk
* considered for adjacent premises/buildings founded via shallow footings unless indicated
* evacuation scale from Almost Certain to not 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	Landslide (earth slide <10m³) from soils at crest of the excavation	a) House No. 92 Park Street	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring property or structure being assessed.	Very Low
		b) House 92a Park Street	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring property or structure being assessed.	Very Low
		c) Lawn, garden, driveways - No. 92 or 92a Park Street	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring property or structure being assessed.	Moderate
		d) House No. 6 Kunari Place	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring property or structure being assessed.	Very Low
		e) Lawn and garden No. 6 Kunari Place	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring property or structure being assessed.	Low
		f) Footpath - Park Street	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring property or structure being assessed.	Low
		g) Road Pavement - Park Street	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring property or structure being assessed.	Very Low
		h) Road Pavement - Kunari Place	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring property or structure being assessed.	Very Low
B	Landslide (rock and earth slide <20m³) due to instability within the bedrock	a) House No. 92 Park Street	Unlikely	The event might occur under very 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 property or strcutures being assessed.	Moderate
		b) House 92a Park Street	Unlikely	The event might occur under very 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 property or strcutures being assessed.	Moderate
		c) Lawn, garden, driveways - No. 92 or 92a Park Street	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring property or structure being assessed.	Moderate
		d) House No. 6 Kunari Place	Unlikely	The event might occur under very adverse 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 or structure being assessed.	Low
		e) Lawn and garden No. 6 Kunari Place	Unlikely	The event might occur under very adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring property or structure being assessed.	Very Low
		f) Footpath - Park Street	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring property or structure being assessed.	Moderate
		g) Road Pavement - Park Street	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring property or structure being assessed.	Low
		h) Road Pavement - Kunari Place	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring property or structure being assessed.	Low

* 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%.

TABLE: 2

Recommended Maintenance and Inspection Program

Structure	Maintenance/ Inspection Item	Frequency
Stormwater drains.	Owner to inspect to ensure that the drains, and pipes are free of debris & sediment build-up. Clear surface grates and litter.	Every year or following each major rainfall event.
Retaining Walls.	Owner to inspect walls for deveation from as constructed condition Owner to assess retaining wall drainage systems, clean as necessary at 10 yearly intervals or where moisture	Every 2 years or following major rainfall event. At 10 yearly intervals or where moisture problems arise
Large Trees on or adjacent to site	Arborist to check condition of trees and remove as required. Where treee within steep slopes or adjacent to structures require geotechincal inspection prior to removal	Every 10 years
Slope Stability	Hydraulics (stormwater) & Geotechnical Consultants to check on site stability at same time and provide report.	One year after construction is completed.

N.B. Provided the above shedule is maintained the design life of the property should conform with Councils Risk Management Policy.

Appendix 4

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CERTIFICATE OF ANALYSIS 377183

Client Details

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

Sample Details

Your Reference	**2025-047, 94-96 Parks Road, Kunari Place Mona Vale**
Number of Samples	6 Soil
Date samples received	02/04/2025
Date completed instructions received	02/04/2025

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.

Report Details

Date results requested by	09/04/2025
Date of Issue	09/04/2025
Reissue Details	This report replaces R00 created on 09/04/2025 due to: Project ID Amended (Client Request)

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Results Approved By
Diego Bigolin, Inorganics Supervisor
Priya Samarawickrama, Senior Chemist

Authorised By
Nancy Zhang, Laboratory Manager

sPOCAS field test						
Our Reference		377183-1	377183-2	377183-3	377183-4	377183-5
Your Reference	UNITS	BH1 2.4-2.5m	BH1 2.5-2.6m	BH1 2.85-2.95m	BH2 0.15-0.25m	BH3 0.5-0.6m
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	02/04/2025	02/04/2025	02/04/2025	02/04/2025	02/04/2025
Date analysed	-	07/04/2025	07/04/2025	07/04/2025	07/04/2025	07/04/2025
pH_F (field pH test)	pH Units	5.9	5.0	5.4	6.2	5.9
pH_{FOX} (field peroxide test)	pH Units	5.7	4.9	5.3	5.4	5.7
Reaction Rate*	-	Low reaction	Low reaction	Low reaction	Low reaction	Low reaction

sPOCAS field test		
Our Reference		377183-6
Your Reference	UNITS	BH4 0.4-0.5m
Type of sample		Soil
Date prepared	-	02/04/2025
Date analysed	-	07/04/2025
pH_F (field pH test)	pH Units	6.6
pH_{FOX} (field peroxide test)	pH Units	5.8
Reaction Rate*	-	Low reaction

Acid Sulphate Soil Suite			
Our Reference		377183-1	377183-3
Your Reference	UNITS	BH1 2.4-2.5m	BH1 2.85-2.95m
Type of sample		Soil	Soil
Date prepared	-	02/04/2025	02/04/2025
Date analysed	-	02/04/2025	02/04/2025
pH_{kcl}	pH units	6.0	5.2
s-TAA pH 6.5	%w/w S	<0.01	0.01
TAA pH 6.5	moles H^+ /t	<5	7
a-Chromium Reducible Sulfur	moles H^+ /t	<3	8
Chromium Reducible Sulfur	%w/w	<0.005	0.01
S_{KCl}	%w/w S	[NT]	[NT]
S_{HCl}	%w/w S	[NT]	[NT]
S_{NAS}	%w/w S	[NT]	[NT]
ANC_{BT}	% $CaCO_3$	[NT]	[NT]
s-ANC_{BT}	%w/w S	[NT]	[NT]
s-Net Acidity excluding ANC	%w/w S	0.0060	0.023
a-Net Acidity excluding ANC	moles H^+ /t	<5	14
Liming rate excluding ANC	kg $CaCO_3$ /t	<0.75	1.1
s-Net Acidity including ANC	%w/w S	0.0060	0.023
a-Net Acidity including ANC	moles H^+ /t	<5	14
Liming rate including ANC	kg $CaCO_3$ /t	<0.75	1

Soil Aggressivity				
Our Reference		377183-2	377183-5	377183-6
Your Reference	UNITS	BH1 2.5-2.6m	BH3 0.5-0.6m	BH4 0.4-0.5m
Type of sample		Soil	Soil	Soil
Date prepared	-	03/04/2025	03/04/2025	03/04/2025
Date analysed	-	03/04/2025	03/04/2025	03/04/2025
pH 1:5 soil:water	pH Units	5.4	5.7	6.7
Electrical Conductivity 1:5 soil:water	µS/cm	70	96	51
Chloride, Cl 1:5 soil:water	mg/kg	58	58	10
Sulphate, SO4 1:5 soil:water	mg/kg	82	95	20

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	Chromium Reducible Sulfur - Hydrogen Sulfide is quantified by iodometric titration after distillation to determine potential acidity. Net acidity including ANC has a safety factor of 1.5 applied. Neutralising value (NV) of 100% is assumed for liming rate. 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 result. However, it has been applied in the SNAS calculation: SNAS % = (SHCL-SKCL)x2
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: sPOCAS field test					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			02/04/2025	[NT]	[NT]	[NT]	[NT]	02/04/2025	[NT]
Date analysed	-			07/04/2025	[NT]	[NT]	[NT]	[NT]	07/04/2025	[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: Acid Sulphate Soil Suite					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			02/04/2025	[NT]	[NT]	[NT]	[NT]	02/04/2025	[NT]
Date analysed	-			02/04/2025	[NT]	[NT]	[NT]	[NT]	02/04/2025	[NT]
pH $_{kcl}$	pH units		Inorg-068	[NT]	[NT]	[NT]	[NT]	[NT]	109	[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]	85	[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]	112	[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]	80	[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: Soil Aggressivity					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			03/04/2025	2	03/04/2025	03/04/2025		03/04/2025	[NT]
Date analysed	-			03/04/2025	2	03/04/2025	03/04/2025		03/04/2025	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	2	5.4	5.4	0	102	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	2	70	72	3	100	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	2	58	44	27	115	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	2	82	81	1	115	[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.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

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.

Where matrix spike recoveries fall below the lower limit of the acceptance criteria (e.g. for non-labile or standard Organics <60%), positive result(s) in the parent sample will subsequently have a higher than typical estimated uncertainty (MU estimates supplied on request) and in these circumstances the sample result is likely biased significantly low.

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.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

MOISTURE CONTENT TEST REPORT

Client	Crozier Geotech	**Job #**	S25168-1
Address	Unit 12/ 42-46 Wattle Street Brookvale NSW 2100	**Report #**	S104882-MC
Project	94-96 Park Street, Kunari Place Mona Vale		

Test Procedure

- [x] 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** 25/03/2025

Preparation Prepared in accordance with the test method **Date Tested** 2/04/2025

Sample #	Source	Sample Description	Moisture Content %
S104882	BH1 2.40-m	Silty CLAY	**21.8**
S104883	BH1 2.85-m	Silty Sandy CLAY	**13.5**
S104884	BH2 0.40-m	Silty Sandy CLAY	**14.5**
S104885	BH3 0.50-m	Silty CLAY	**21.6**
S104886	BH3 1.10-m	Silty CLAY	**16.4**
S104887	BH4 0.40-m	Silty CLAY	**13.6**
S104888	BH4 0.70-m	Silty CLAY	**18.3**

Notes

NATA

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NATA Accredited Laboratory Number: 14874

Authorised Signatory:

Chris Lloyd

~~3/04/2025~~

Date:

MACQUARIE GEOTECH

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Macquarie Geotechnical
14 Carter St
Lidcombe NSW 2141

SOIL CLASSIFICATION REPORT

Client	Crozier Geotech	**Source**	BH1 2.40-m
Address	Unit 12/ 42-46 Wattle Street Brookvale NSW 2100	**Sample Description**	Silty CLAY
Project	94-96 Park Street, Kunari Place Mona Vale	**Report No.**	S104882-PI
Job No.	S25168-1	**Lab No.**	S104882

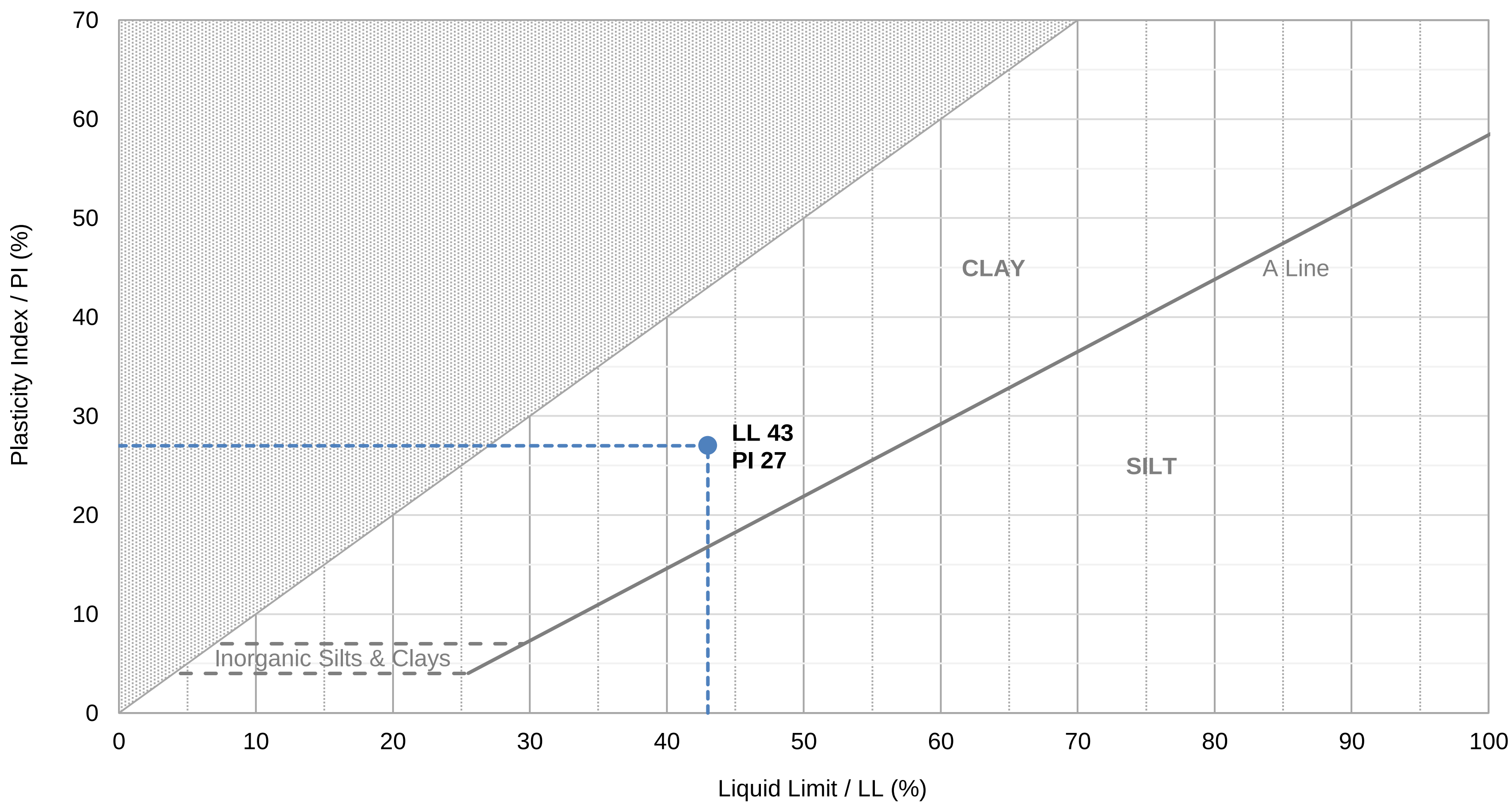
Test Procedure

- ☑ AS1289 3.1.1 Liquid Limit - Four point Casagrande method
- ☐ AS1289 3.1.2 Liquid Limit - One point Casagrande method
- ☑ AS1289 3.2.1 Plastic Limit - Standard method
- ☑ AS1289 3.3.1 Calculation of the Plasticity Index
- ☐ AS1289 3.4.1 Linear Shrinkage - Standard method

Sampling	Sampled by Client - results apply to the sample as received	**Date Sampled**	25/03/2025
Preparation	Prepared in accordance with the test method	**Date Tested**	4/04/2025

Plasticity Chart for Classification of Fine-Grained Soils

AS 1726:2017 Clause 6.1.6 (Figure 5)



Preparation

Method of Preparation	**Dry Sieved**
History of the Sample	**Oven Dried**

Results

Liquid Limit / LL (%)	**43**
Plastic Limit (%)	**16**
Plasticity Index / PI (%)	**27**

Notes

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Authorised Signatory:

Chris Lloyd

~~7/04/2025~~

Date:

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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}	$5x10^{-2}$	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}	$5x10^{-3}$	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	$5x10^{-4}$	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	$5x10^{-5}$	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	$5x10^{-6}$	100,000 years	200,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
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%	100%	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
60%	40%	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
20%	10%	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
5%	1%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
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.

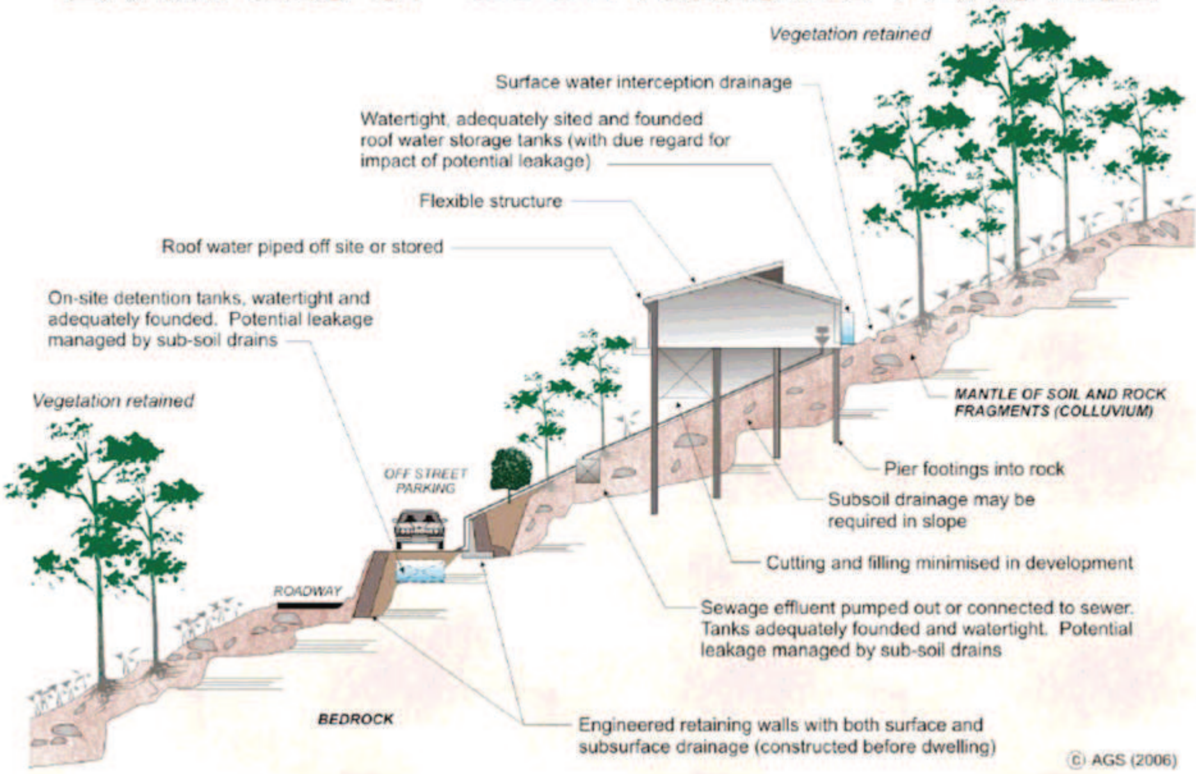
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

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