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REPORT ON GEOTECHNICAL INVESTIGATION

for

PROPOSED ALTERATIONS AND ADDITIONS

at

16 HILLCREST AVENUE, MONA VALE, NSW

Prepared For

MSD Properties Pty Ltd

Project No.: 2025-001

February, 2025

Document Revision Record

Issue No	Date	Details of Revisions
0	17th February 2025	Original 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	16 Hillcrest Avenue, Mona Vale, NSW

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

*I, __**Troy Crozier**__ on behalf of ___**Crozier Geotechnical Consultants*** _on this the 17 February 2025_ certify that I am a ~~geotechnical engineer or~~ engineering geologist ~~or coastal engineer~~ as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above ~~organisation/~~company to issue this document and to certify that the ~~organisation/~~company has a current professional indemnity policy of at least $2million. I:

☐ have prepared the detailed Geotechnical Report referenced below in accordance with the Australia Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009

■ am willing to technically verify that the detailed Geotechnical Report referenced below has been prepared in accordance with the Australian Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009

☐ have examined the site and the proposed development in detail and have carried out a risk assessment in accordance with Section 6.0 of the Geotechnical Risk Management Policy for Pittwater - 2009. I confirm that the results of the risk assessment for the proposed development are in compliance with the Geotechnical Risk Management Policy for Pittwater - 2009 and further detailed geotechnical reporting is not required for the subject site.

☐ have examined the site and the proposed development/alteration in detail and I am of the opinion that the Development Application only involves Minor Development/Alteration that does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.

☐ have examined the site and the proposed development/alteration is separate from and is not affected by a Geotechnical Hazard and does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.

☐ have provided the coastal process and coastal forces analysis for inclusion in the Geotechnical Report

Geotechnical Report Details:

Report Title: Geotechnical Report for Proposed Alterations and Additions

Report Date: 17/02/2025 **Project No.:** 2025-001

Author: J. Dee and T. Crozier

Author's Company/Organisation: Crozier Geotechnical Consultants

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

Architectural drawings by Smith & Tzannes, Project No.: 24_066, Drawing No.: DD-A-010, DD-A-100 – 102, DD-A-204 – 206, DD-A-801
Survey Plan by H Ramsay & Co, Reference No.: 9620, Dated: 2/10/2024
Coastal Engineering Report – Horton Coastal Engineering, lrJ0795 – 16 Hillcrest Avenue Mona Vale, Dated: 13 February 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**

AUSTRALIAN INSTITUTE OF GEOSCIENTISTS
TROY CROZIER
10197

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 __16 Hillcrest Avenue, Mona Vale, NSW _____________

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

Geotechnical Report Details:

Report Title: Geotechnical Report for Proposed Alterations and Additions
Report Date: 17/02/2025 **Project No.:** 2025-001
Author: J. Dee and T. Crozier
Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

- ■ Comprehensive site mapping conducted ___6th January 2025________
- ■ 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 6th January 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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GEOTECHNICAL REPORT FOR PROPOSED ALTERATIONS AND ADDITIONS
16 HILLCREST AVENUE, MONA VALE

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for proposed alterations and additions at 16 Hillcrest Avenue, Mona Vale, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of Aston Building on behalf of the client MSD Properties Pty Ltd.

It is understood that the proposed works involve internal changes to the existing structure, a rear extension and landscaping works including infilling of the existing swimming pool and construction of a new swimming pool further towards the rear boundary. The new swimming pool will require bulk excavation to approximately 2.00m depth. Alterations to the landscaping, structures in the front garden and external walls of the building are also proposed.

The site is located within the H1 (highest category) landslip hazard zone as identified within Northern Beaches Councils precinct (Geotechnical Risk Management Policy for Pittwater – 2009). For Development Application purposes, to meet the Councils Policy requirements for land classified as H1 a detailed Geotechnical Report which meets the requirements of Paragraph 6.5 of that policy must be submitted. This report must include a landslide risk assessment to the methods of AGS 2007 for the site and proposed works, plans, geological sections and provide recommendations for construction and to ensure stability is maintained for a preferred design life of 100 years.

The site is also located within a Bluff/Cliff Instability" area designated on the *Coastal Risk Planning Map* (Sheet CHZ_018) that is referenced in *Pittwater Local Environmental Plan 2014*. As such, "a coastal engineers report on the impact of coastal processes on the site must be incorporated into the geotechnical assessment as an appendix.

This report is provided for DA submission and includes a description of site and sub-surface conditions including groundwater, soil logs and in-site test results, a geotechnical assessment of the proposed works, assessment of landslide hazards, site plan and recommendations for the design of works.

The investigation and reporting were undertaken as per Proposal No.: P24-562, Dated: 9th December 2024.

The investigation comprised:

a) Onsite service location and clearing of borehole locations by an accredited contractor.
b) Detailed geotechnical inspection and mapping of the site and adjacent properties with a photographic record and identification of geotechnical conditions and hazards related to the existing site and proposed works;
c) Drilling of four boreholes using a hand tools due to access limitations along with five Dynamic Cone Penetrometer (DCP) tests across the site

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

- Architectural Drawings – Smith & Tzannes, Project No.: 24_066, Drawing No.: DD-A-010, DD-A-100 – 102, DD-A-204 – 206, DD-A-801
- Survey Drawing – H Ramsay & Co, Reference No.: 9620, Dated: 2/10/2024
- Coastal Engineering report – Horton Coastal Engineering, lrJ0795 – 16 Hillcrest Avenue Mona Vale, Dated: 13 February 2025

2. SITE FEATURES:

2.1. Description:

The site is a broadly trapezoidal shaped block situated on the high northern side of Hillcrest Avenue atop a broadly north-south oriented ridgeline, bordered to the west by moderately west dipping topography and to the east by extreme to vertical east dipping topography. Site surface levels reduce from a high of approximately RL54.50 broadly in the middle of the property to a low of RL 53 along both the front southern and rear northern boundaries. An aerial photograph of the site and its surrounds with boundary designations is provided below (Photograph 1), as sourced from NSW Government Six Map spatial data system. For the purposes of this report, the front roadway is referenced as the southern site boundary wit the other boundaries referenced accordingly.

Photograph 1: Aerial photo of site and surrounds with boundary designations

2.2. Geology:

Reference to the Sydney 1:100,000 Geological Series sheet 9130 indicates that the site is underlain by Newport Formation (Upper Narrabeen Group) rock (Rnn) which is of middle Triassic Age. The Newport Formation typically comprises interbedded laminite, shale and quartz to lithic quartz sandstones and pink clay pellet sandstones that tend to deep weathering and produce a weak rock mass containing clay bands and fracturing. An extract from the Sydney 1:100, 000 Geological Series sheet is provided below.

Extract 1: Sydney (9130 Geology Series Map): 1: 100000 – Geology underlying the site

3. FIELD WORK:

3.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on 6th January 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 soil slopes, existing structures and neighbouring properties. It also included the drilling of four boreholes with hand tools due to access limitations to investigate subsurface geology.

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’.

DCP testing was carried out from ground surface adjacent to the boreholes and at one additional location in accordance with AS1289.6.3.2 – 1997, “Determination of the penetration resistance of a soil – 9kg Dynamic Cone Penetrometer” to estimate near surface ground conditions.

Explanatory notes are included in Appendix: 1. Mapping information and test locations are shown on Figure: 1, along with detailed Borehole Log sheets and Dynamic Penetrometer Test Sheet in Appendix: 2. A geological model/section is provided as Figure: 2, Appendix: 2.

3.2. Field Observations:

The site is situated on the high north side of Hillcrest Avenue within relatively gentle sloe. The roadway comprises a bituminous sealed pavement that is relatively flat where it passes the site. A concrete kerb, gutter and pedestrian pavement separate the site from the roadway. There were no signs of any impending geotechnical concern in the road reserve.

A gently south dipping concrete driveway extends up into the site from the road reserve, providing access to a carport and basement/subfloor level garage. A concrete stairway extends up through a vegetated front garden area and provides access to the site dwelling entrance. The main site dwelling comprises a two and three storey masonry and concrete structure of approximately 1970’s construction age. The structure did not exhibit any signs of significant cracking or settlement to indicate any impending geotechnical concern.

The rear portion of site is accessed via a gently north dipping concrete pathway that extends around the western side of the dwelling. The rear portion of the site comprises a flat courtyard area and in ground pool immediately to the north of the dwelling structure with a gently north dipping lawn area extending to the rear boundary.

The neighbouring property to the west (No. 14 Hillcrest Avenue) contains a two storey masonry dwelling, setback from the shared boundary by approximately 1.00m from the shared boundary. Ground levels within the property appeared to be relatively similar to the site along the majority of the boundary with the exception of a portion directly adjacent to the site dwelling and rear courtyard area where the neighbouring property is retained at levels of up to 1.50m above adjacent site surface levels via a masonry boundary wall. The visible aspects of the neighbouring structures appeared to be in good condition with no signs of excessive settlement or cracking to indicate any impending geotechnical concern.

The neighbouring property of No. 18 Hillcrest Avenue shares a common boundary with to site to both the north and the east. Adjacent to the eastern site boundary, the property contains a one and two storey masonry structure, anticipated to be of approximately 1960's construction age. Along the eastern site boundary, site surface levels are generally lower than adjacent levels within the neighbouring property with the boundary wall retaining the property up to 1.00m above the site. Adjacent to the northern site boundary, the neighbouring property contains single storey "granny-flat" structure which is currently under construction. The site is retained at levels of up to 2.00m above the adjacent property surface levels via a two tier I-beam and concrete waler retaining wall. The visible aspects of the neighbouring property did not exhibit any signs of excessive settlement or cracking to indicate any impending geotechnical concern.

The neighbouring properties and structures were inspected from the site or road reserves, however visible aspects showed no indications of geotechnical hazard that may impact the site.

3.3. Ground Conditions:

The boreholes were drilled across site broadly within the vicinity of proposed works. The boreholes encountered fill soils from existing ground surface levels with BH1/1a encountering continual refusal at 0.80m depth within a compacted gravelly fill band. BH2 extended through the fill soils and intersected residual clays from 0.50m depth prior to encountering hand auger refusal atop interpreted siltstone/shale bedrock of at least very low strength (VLS). BH3 and BH5 extended entirely through fill soils prior to encountering hand auger refusal atop interpreted VLS bedrock at depths of 0.50m and 0.65m respectively.

DCP tests were carried out from the ground surface adjacent to the boreholes and at one additional location with refusal encountered atop interpreted VLS-LS siltstone/shale bedrock varying from 0.50m-2.15m.

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

- **TOPSOIL/FILL –** Topsoil/Fill was encountered from ground surface in all boreholes and extended to a maximum confirmed depth of 0.80m (BH1/1a) however was interpreted via DCP to extend to ≈1.20m in this test location. Within BH1/1a it comprised compacted dry clay fill with shale and gravels and cobbles. Elsewhere on site it broadly comprised a loose, brown moist silty sand/clay with roots and building refuse.
- **RESIDUAL SOILS –** Natural clayey soils interpreted as residual were encountered underlying the topsoil/fill within BH2 from 0.50m depth and interpreted within DCP1 and DCP4 below 1.20m and 1.30m respectively. Where recovered in BH2, the residual soils comprised a stiff to hard yellow/brown silty clay with red and grey mottle as well as interspersed ironstone gravels/cobbles.
- **SILTSTONE/SHALE BEDROCK –** Very Low Strength (VLS) siltstone/shale bedrock was interpreted via hand auger refusal in BH2, BH3 and BH5 whilst Low Strength (LS) bedrock was interpreted via DCP refusal in all testing locations from a minimum and maximum depth of 0.50m and 2.15m respectively.

Whilst a freestanding groundwater table was not identified within the investigation, minor seepage was observed above the residual soils as well as above the bedrock surface.

4. COMMENTS:

4.1. Geotechnical Assessment:

The site investigation identified the presence of a variable layer of topsoil/fill from existing ground surface levels overlying VLS-LS siltstone/shale bedrock with an intermediary layer of residual clay soils encountered in parts. Minor seepage was observed overlying the residual soils as well as the bedrock surface however a freestanding groundwater table or significant seepage was not encountered in the investigation and will not be intersected within the envelope of proposed works.

It is understood that the proposed works involve internal changes to the existing structure, a rear extension and landscaping works including infilling of the existing swimming pool and construction of a new swimming pool further towards the rear boundary. The new swimming pool will require bulk excavation to approximately 2.00m depth. The excavation is proposed to be setback from the northern and eastern shared boundaries by 1.00m and 1.50m respectively however setbacks to the western and southern shared boundaries will be in excess of 5m and 30m respectively.

Based on the results of the investigation the bulk excavation is anticipated to extend through up to 1.20m of fill soils prior to intersection of residual clays which will comprise the remainder of the excavated material towards the northern site boundary. However, towards the southern edge of the proposed excavation, there is potential for intersection of siltstone/shale bedrock of at least very low strength towards the base excavation level (BEL).

The relatively minimal setbacks to both the northern and eastern shared boundaries precludes the implementation of safe batter slopes as designated in Section 4.3.2. However, the relatively small scale excavation edges as well as the site-neighbouring property surface level differentials will likely allow for the implementation of temporary support in the form of an I-beam and waler wall whilst maintaining stability external to the site.

The fill, residual soils and very low strength bedrock can be excavated using conventional earthmoving equipment with ripping in LS siltstone/shale possible. If larger scale rock hammers are preferred/required, Crozier Geotechnical Consultants (CGC) should be consulted regarding the size and type of excavation equipment proposed and excavation methodology prior to works.

The backfill of the existing pool structure will require the placement of up to 2.00m of fill soils. The proposed re-use of site soils will require care to ensure adequate compaction levels are achieved. All organic matter and surficial topsoil should be scraped away and discarded, and the fill soils should be placed in layers of approximately 250mm loose thickness prior to compaction. Where the fill is required to perform with limited settlement or with structures above it then further engineering specifications for this placement will be necessary.

All new footings for independent structures should be founded off materials of similar strength/density in order to avoid differential settlement. The high likelihood of VLS bedrock towards the BEL of the pool along its southern edge dictates that all pool footings will need to extend through any fill and residual soils encountered and bear atop bedrock of similar strength. Additionally, as the dwelling is considered likely to have undergone the majority of its settlement under existing loads, it is recommended that any new footings for dwelling extensions be founded atop bedrock to present similarly negligible settlement values.

The anticipated cliff recession rate of 0.60m – 1.20m over 100 years as outlined in the Horton Coastal Engineering report has been considered with regards to the existing site conditions as well as the proposed works. The top of the upper cliff is setback ≥5.00m from the site boundaries and ≥7.00m from the envelope of proposed works. Therefore, it is considered that the landward extent of cliff instability risk falls outside the boundaries of both the site and the envelope of proposed works, and the proposed development is at acceptable low risk of damage from coastal erosion/recession of the cliff for the design life of 100 years.

The proposed works are considered suitable for the site and may be completed with negligible impact to existing nearby structures within the site or on neighbouring properties provided the recommendations of this report are implemented in the design and construction phases. The proposed works are unlikely to increase the level of risk for any people, assets and infrastructure in the vicinity of the cliff due to geotechnical processes.

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and hand tools. This test equipment provides limited data from small, isolated test points across the entire site. Therefore, some minor variation to the interpreted sub-surface conditions is possible, especially between test locations and below DCP refusal depths. However, the results of the investigation provide a reasonable basis for the Development Application analysis and subsequent preliminary design of the proposed works.

4.2. Site Specific Risk Assessment:

Based on our site investigation and the proposed works, it is considered that the stability hazard associated with the proposed works is limited to the neighbouring property of No. 14 Nareen Parade as well as existing site structures. The hazard is:

A. Landslip (earth slide <3m^3) from soils at crest of the pool excavation

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 **5.21 x 10^{-6},** whilst the Risk to Property was considered to be **'Moderate'**. The hazard was therefore considered to be '**Unacceptable**' when assessed against the criteria of the AGS 2007 and the Geotechnical Risk Management Policy.

The above risk to life and property from Hazard A has been assessed assuming insufficient stabilizing measures/retention systems are constructed within the site. Where appropriate systems are installed the anticipated risks are expected to reduce within "Acceptable" risk management criteria of the Council's policy. As such, the works are considered suitable for the site.

4.3. Design & Construction Recommendations:

Design and construction recommendations are tabulated below:

4.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Class 'M' for ancillary footings atop residual soils Clas 'A' for footings at the BEL atop competent bedrock
Type of Footing	Strip/Pad, Slab or piers
Sub-grade material and Maximum Allowable Bearing Capacity	- Stiff Residual Clays: 100kPa - Very Stiff Residual Clays: 200kpa - Hard residual Clays: 400kPa - Weathered VLS Bedrock: 800kPa
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	C_e – Shallow Soil Site
Remarks: These values are subject to confirmation by geotechnical inspection/testing during construction. All permanent structure footings should be founded off bedrock similar strength to reduce the potential for differential settlement unless designed for by the structural engineer. 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.	

4.3.2. Excavation:	
Depth of Excavation	Up to 2.00m for Pool
Type of Material to be Excavated	Topsoil/clayey fill with cobbles and rubble to potential maximum of 1.20m depth
	Residual soils to potential maximum of 2.00m
	Siltstone/Shale bedrock – VLS – LS, from minimum of 0.50m to

Guidelines for un-surcharged batter slopes for this site are tabulated below:

Material	Safe Batter Slope (H:V)	
	Short Term/Temporary	Long Term/Permanent
Fill and natural soils	1.5:1	2.0:1
Very Low (VLS) strength or fractured bedrock	0.75:1	0.5:1*

*Dependent on defects and assessment by engineering geologist.

Remarks: Seepage at the bedrock surface or along defects in the rock can also reduce the stability of batter slopes or rock cuts 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 permanent support measures are installed. This should also be considered with respect to safe working conditions. Batter slopes should not be left unsupported without geotechnical inspection and approval. Should further detail on rock strengths or conditions for excavation costing be required, then cored boreholes and laboratory testing will be required.		
Equipment for Excavation	Fill/natural soils	Bucket
	VLS bedrock	Bucket and ripper
VLS – very low strength		
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • Inspection of any temporary and permanent batter slopes, • During installation of excavation support • At completion of excavation • Where ground conditions are exposed that differ to those than expected	
Dilapidation Surveys Requirement	Not necessary	
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.		

4.3.3. Retaining Structures:	
Required	New retaining structures are required as part of proposed works. Pool shell expected to act as permanent support.
Types	Steel reinforced concrete/concrete block post excavation designed in accordance with Australian Standards AS4678-2002 Earth Retaining Structures.
Parameters for calculating pressures acting on retaining walls for the materials likely to be retained:	

Material	Unit Weight (kN/m^3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Fill and Residual Clay Soils	20	$\phi' = 30°$	0.33	0.47	N/A
VLS -LS bedrock	22	$\phi' = 38°$	0.10	0.20	200kPa

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 backfilled with free-draining granular material (preferably not recycled concrete) which is only lightly compacted in order to minimize horizontal stresses.

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

4.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		Seepage identified and anticipated above residual soils, bedrock surface
Excavation likely to intersect	Water Table	No
	Seepage	Minor (<0.50L/min), within soil interfaces and at bedrock surface.
Site Location and Topography		High northern side of the road within relatively flat topography adjacent to a cliff
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Due to the shallow bedrock and relatively impermeable residual soils, the property is not suitable for onsite absorption disposal system.

Remarks:

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

4.4. Conditions Relating to Design and Construction Monitoring:

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

1. Review and approve the structural design drawings for compliance with the recommendations of this report prior to construction,
2. Inspection of site and works as per Section 4.3 of this report
3. Inspect all new footings and earthworks to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and the stability prior to the placement of steel or concrete,
4. Inspect completed works to ensure construction activity has not created any new hazards and that all retention and stormwater control systems are completed.

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

4.5. Design Life of Structure:

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

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

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

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

- The conditions on the block don't change from those present at the time this report was prepared, except for the changes due to this development.
- There is no change to the property due to an extraordinary event external to this site

- The property is maintained in good order and in accordance with the guidelines set out in;
  a) CSIRO sheet BTF 18
  b) Australian Geomechanics "Landslide Risk Management" Volume 42, March 2007.
  c) AS 2870 – 2011, Australian Standard for Residential Slabs and Footings

Where changes to site conditions are identified during the maintenance and inspection program, reference should be made to relevant professionals (e.g. structural engineer, geotechnical engineer or Council). Where the property owner has any lack of understanding or concerns about the implementation of any component of the maintenance and inspection program the relevant engineer should be contacted for advice or to complete the component. It is assumed that Council will control development on neighbouring properties, carry out regular inspections and maintenance of the road verge, stormwater systems and large trees on public land adjacent to the site so as to ensure that stability conditions do not deteriorate with potential increase in risk level to the site.

Also, individual Government Departments will maintain public utilities in the form of power lines, water and sewer mains to ensure they don't leak and increase either the local groundwater level or landslide potential.

5. CONCLUSION:

The site investigation identified the presence of both fill and residual soils overlying siltstone/shale bedrock across site, at depths varying between 0.50m and 2.15m below existing site surface levels. Minor seepage was observed overlying the residual soils and bedrock surface however no freestanding groundwater table or significant seepage was intersected within the investigation and is not expected within 5m of ground levels based on topography and elevations.

It is understood that the proposed works involve internal changes to the existing structure, a rear extension and landscaping works including infilling of the existing swimming pool and construction of a new swimming pool further towards the rear boundary. The new swimming pool will require bulk excavation to approximately 2.00m depth.

Safe batter slopes do not appear to be feasible along the northern and eastern excavation edges with respect to the neighbouring property. Where safe batter slopes are not achievable, excavation support will need to be implemented to maintain stability external to the site. The relatively small scale nature of the excavation as well as the site conditions will likely allow for the implementation of temporary I-beam & waler support to maintain stability.

The backfill associated with the level garden area will need to be undertaken incrementally and compacted in layers. Caution must be exercised regarding the re-use of site soils and all fill and organic matter should be removed prior to use.

All new footings will need to bear within materials of similar strength/density to minimise the risk of differential settlement.

Based on the provided cliff recession rate of 0.60m to 1.20m outlined in the Horton Coastal Engineering report, the proposed development is at an acceptably low risk of damage from coastal erosion/recession of the cliff seaward of the site for a design life of 100 years.

The risks associated with the proposed development are considered to achieve and can be maintained within the 'Acceptable' Risk Management Criteria provided the recommendations of this report and any future geotechnical directive are implemented. As such the site is considered suitable for the proposed construction works provided that the recommendations outlined in this report are followed.

Prepared by:

James Dee
Geotechnical Engineer

Reviewed by:

Troy Crozier
Principal Engineering Geologist
MIEAust., CPeng (NER)
MAIG, RPGeo

6. REFERENCES:

1. Australian Geomechanics Society 2007, “Landslide Risk Assessment and Management”, Australian Geomechanics Journal Vol. 42, No 1, March 2007.
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3. C. W. Fetter 1995, “Applied Hydrology” by Prentice Hall. V. Gardiner & R. Dackombe 1983, “Geomorphological Field Manual” by George Allen & Unwin
4. Australian Standard AS 3798 – 2007, Guidelines on Earthworks for Commercial and Residential Developments.
5. Australian Standard AS 2870 – 2011, Residential Slabs and Footings – Construction
6. Australian Standard AS1170.4 – 2007, Part 4: Earthquake actions in Australia
7. Australian Standard AS 1726 – 2017, Geotechnical Site Investigations

Appendix 1

CROZIER
GEOTECHNICAL CONSULTANTS

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Crozier Geotechnical Consultants, a division of PJC Geo-Engineering Pty Ltd

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 (MPa) = (0.4 to 0.6) N blows (blows per 300mm)

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

Q_c = (12 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

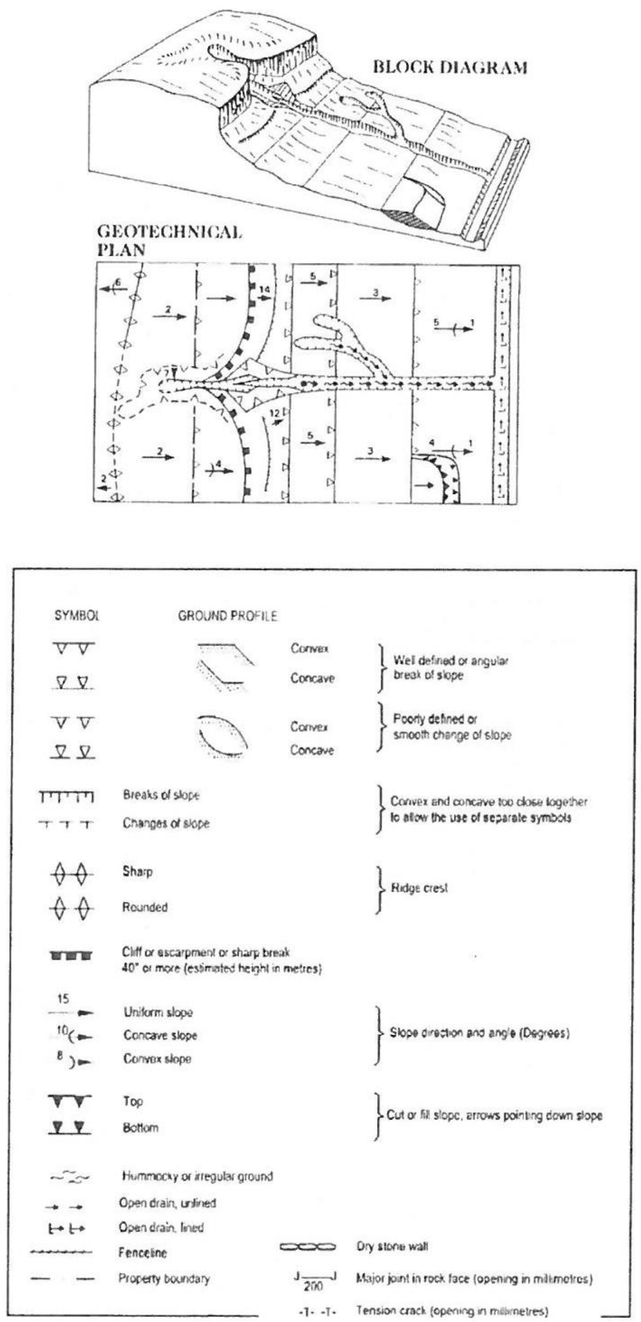
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

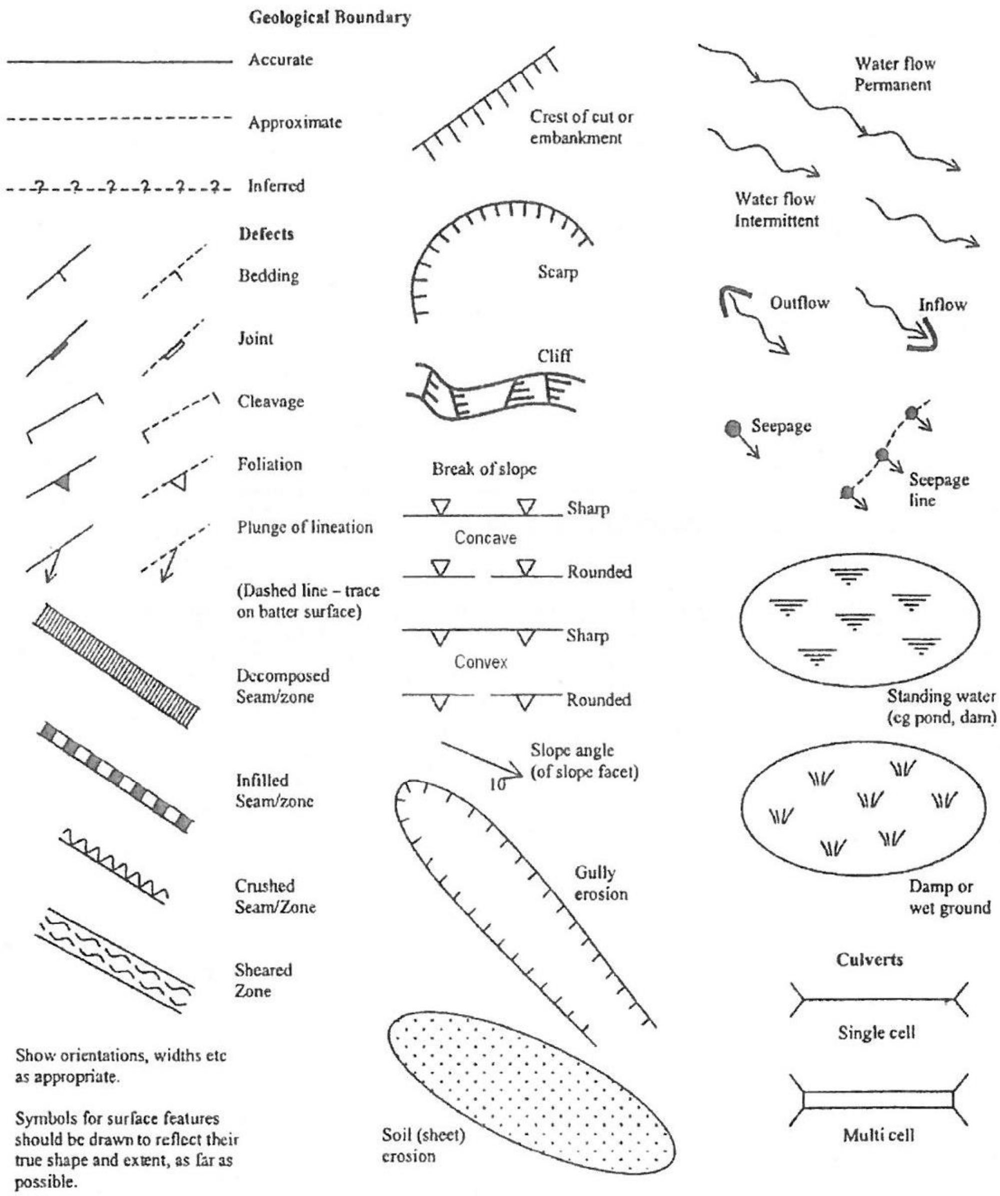
Site Inspection

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).

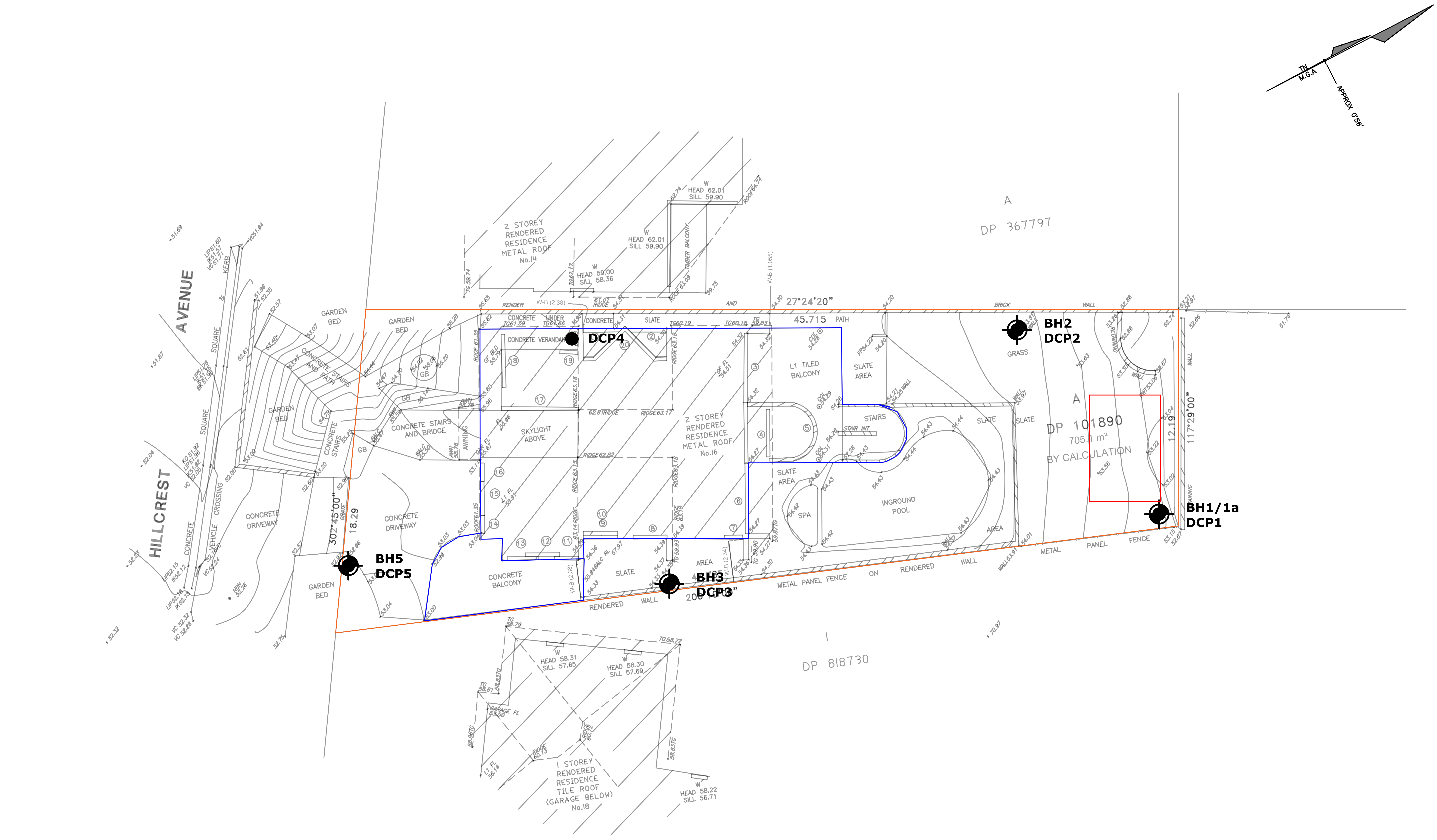
APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



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

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 2



0 1 2 3 4 5 6 7 8 9 10

SCALE: 1:200

SITE PLAN & TEST LOCATIONS

FIGURE 1.

CROZIER GEOTECHNICAL CONSULTANTS

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

LEGEND

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

SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	01 /2025
APPROVED BY:	TMC
DRAWN BY:	JD
PROJECT:	2024-001

PREPARED FOR:

ADDRESS:

BOREHOLE LOG

CLIENT: MSD Properties Pty Ltd
DATE: 6/01/2025
BORE No.: 1/1a

PROJECT: Alterations and Additions
PROJECT No.: 2025-001
SHEET: 1 of 1

LOCATION: 16 Hillcrest Avenue, Mona Vale
SURFACE LEVEL: RL 53.00

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 silty sand with rootlets				
0.10		… dry compacted clay fill with drainage (blue metal) and shale gravels				
0.80		Continued hand auger refusal @ 0.80m depth within compacted gravelly fill, DCP extended to 2.15m				
1.00						
1.20		… interpreted natural interface (residual clay - very stiff)				
1.90		… interpreted VLS bedrock interface				
2.00						
2.15		… DCP refusal atop interrpeted LS siltstone/shale bedrock				

RIG: Not applicable
DRILLER: JD

METHOD: Hand auger
LOGGED: JD

GROUND WATER OBSERVATIONS: Not encountered

REMARKS:
CHECKED: TMC

BOREHOLE LOG

CLIENT: MSD Properties Pty Ltd **DATE:** 6/01/2025 **BORE No.:** 2

PROJECT: Alterations and Additions **PROJECT No.:** 2025-001 **SHEET:** 1 of 1

LOCATION: 16 Hillcrest Avenue, Mona Vale **SURFACE LEVEL:** RL 53.90

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 brown silty sand with gravels and roots				
0.50	CL	CLAY: stiff, brown, medium plasticity, moist silty clay				
0.80		… very stiff				
1.00		… yellow/brown with ironstone gravels				
1.50		… hard				
1.80		Hand auger refusal @ 1.80m depth stop interpreted VLS siltstone/shale bedrock, DCP extended to 2.00m				

RIG: Not applicable DRILLER: JD

METHOD: Hand auger LOGGED: JD

GROUND WATER OBSERVATIONS: Not encountered

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: MSD Properties Pty Ltd | **DATE:** 6/01/2025 | **BORE No.:** 3

PROJECT: Alterations and Additions | **PROJECT No.:** 2025-001 | **SHEET:** 1 of 1

LOCATION: 16 Hillcrest Avenue, Mona Vale | **SURFACE LEVEL:** RL 54.35

Depth (m) 0.00	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
0.50		Topsoil/Fill: loose brown silty sand with roots and gravels				
		Hand auger refusal @ 0.50m depth atop interpreted VLS-LS siltstone/shale bedrock				

RIG: Not applicable

DRILLER: AC

METHOD: Hand auger

LOGGED: JD

GROUND WATER OBSERVATIONS: Not encountered

REMARKS:

CHECKED: TMC

BOREHOLE LOG

CLIENT: MSD Properties Pty Ltd | **DATE:** 6/01/2025 | **BORE No.:** 5

PROJECT: Alterations and Additions | **PROJECT No.:** 2025-001 | **SHEET:** 1 of 1

LOCATION: 16 Hillcrest Avenue, Mona Vale | **SURFACE LEVEL:** RL 53.00

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 brown silty sand with roots and gravels				
0.60		Hand auger refusal @ 0.60m depth atop interpreted VLS-LS siltstone/shale bedrock				

RIG: Not applicable | DRILLER: AC

METHOD: Hand auger | LOGGED: JD

GROUND WATER OBSERVATIONS: Not encountered

REMARKS: | CHECKED: TMC

DYNAMIC PENETROMETER TEST SHEET

CLIENT: MSD Properties Pty Ltd **DATE:** 6/01/2025

PROJECT: Alterations and Additions **PROJECT No.:** 2025-001

LOCATION: 16 Hillcrest Avenue, Mona Vale **SHEET:** 1 of 1

	Test Location									
Depth (m)	1	2	3	4	5					
0.00 - 0.10	4	-	SW	-	SW					
0.10 - 0.20	20	-	1	-	2					
0.20 - 0.30	19	-	4	-	5					
0.30 - 0.40	15	-	8	-	3					
0.40 - 0.50	18	-	4	-	6					
0.50 - 0.60	24	-	B@0.50	-	7					
0.60 - 0.70	17	3		2	7					
0.70 - 0.80	16	4		1	B@0.65					
0.80 - 0.90	12	5		0						
0.90 - 1.00	21	5		3						
1.00 - 1.10	26	5		3						
1.10 - 1.20	19	5		1						
1.20 - 1.30	8	5		0						
1.30 - 1.40	7	5		4						
1.40 - 1.50	8	7		9						
1.50 - 1.60	9	8		8						
1.60 - 1.70	10	8		7						
1.70 - 1.80	14	6		7						
1.80 - 1.90	11	19		7						
1.90 - 2.00	12	30		18						
2.00 - 2.10	13	B@2.00		B@2.00						
2.10 - 2.20	B@2.15									
2.20 - 2.30										
2.30 - 2.40										
2.40 - 2.50										
2.50 - 2.60										
2.60 - 2.70										
2.70 - 2.80										
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:

(B) Test hammer bouncing upon refusal on solid object

-- No test undertaken at this level due to prior excavation of soils

Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (earth slide <3m³) from soils at crest of excavation		Up to 2.00m depth of soils in excavation, potentially up to 1.20m comprising fill	a) Excavation setback from northern and eastern shared boundary by 1.00m and 1.50m respectively. Garden immediately adajcent		a) Person in Garden 1hr/day avge.	a) Unlikely to not evacuate	a) Person in open space, buried	
			likely	**Prob. of Impact**	**Impacted**				
		a) Garden of No. 18 Hillcrest Avenue	0.01	0.50	0.10	0.0417	0.25	1.00	**5.21E-06**

* hazards considered in current condition and/or without remedial/stabilisation measures or poor support systems

* likelihood of occurrence for design life of 100 years

* Spatial Impact - Probaility of Impact refers to slide impacting structure/area expressed as a % (i.e. 1.00 = 100% probability of slide impacting area if slide occurs).
Impacted refers to expected % of area/structure damaged if slide impacts (i.e. small, slow earth slide will damage small portion of house structure such as 1 bedroom (5%), where as large boulder roll may damage/destroy >50%)

* neighbouring houses considered for impact of slide to bedroom unless specified, due to high occupancy and lower potential for evacuation.

* considered for person most at risk, where multiple people occupy area then increased risk levels

* for excavation induced landslip then considered for adjacent premises/buildings founded off shallow footings, unless indicated

* evacuation scale from Almost Certain to <u>not</u> evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knowing of landslide and completely evacuating area prior to landslide impact.

* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (earth slide <3m³) from soils at crest of excavation	a) Garden of No. 18 Hillcrest Avenue	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Moderate

* 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%.

* Cost of site development estimated at $1,000,000

TABLE: 2

Recommended Maintenance and Inspection Program

Structure	Maintenance/ Inspection Item	Frequency
Stormwater drains.	Owner to inspect to ensure that the open drains, and pipes are free of debris & sediment build-up. Clear surface grates and litter. Owner to check and flush retaining wall drainage pipes/systems	Every year or following each major rainfall event. Every 7 years or where dampness/moisture
Retaining Walls. or remedial measures	Owner to inspect walls for deveation from as constructed condition and repair/replace. Replace non engineered rock/timber walls prior to collapse	Every two years or following major rainfall event. As soon as practicable
Large Trees on or adjacent to site	Arborist to check condition of trees and remove as required. Where tree within steep slopes (>18°) or adjacent to structures requires geotechincal inspection prior to removal	Every five years
Slope Stability	Geotechnical Engineering Consultant to check on site stability and maintenance	Five years 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.

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Appendix 4

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}		100 years		The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	$5x10^{-3}$	1000 years	200 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	$5x10^{-4}$	10,000 years	2000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	$5x10^{-5}$	100,000 years	20,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}	$5x10^{-6}$	1,000,000 years	200,000 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.

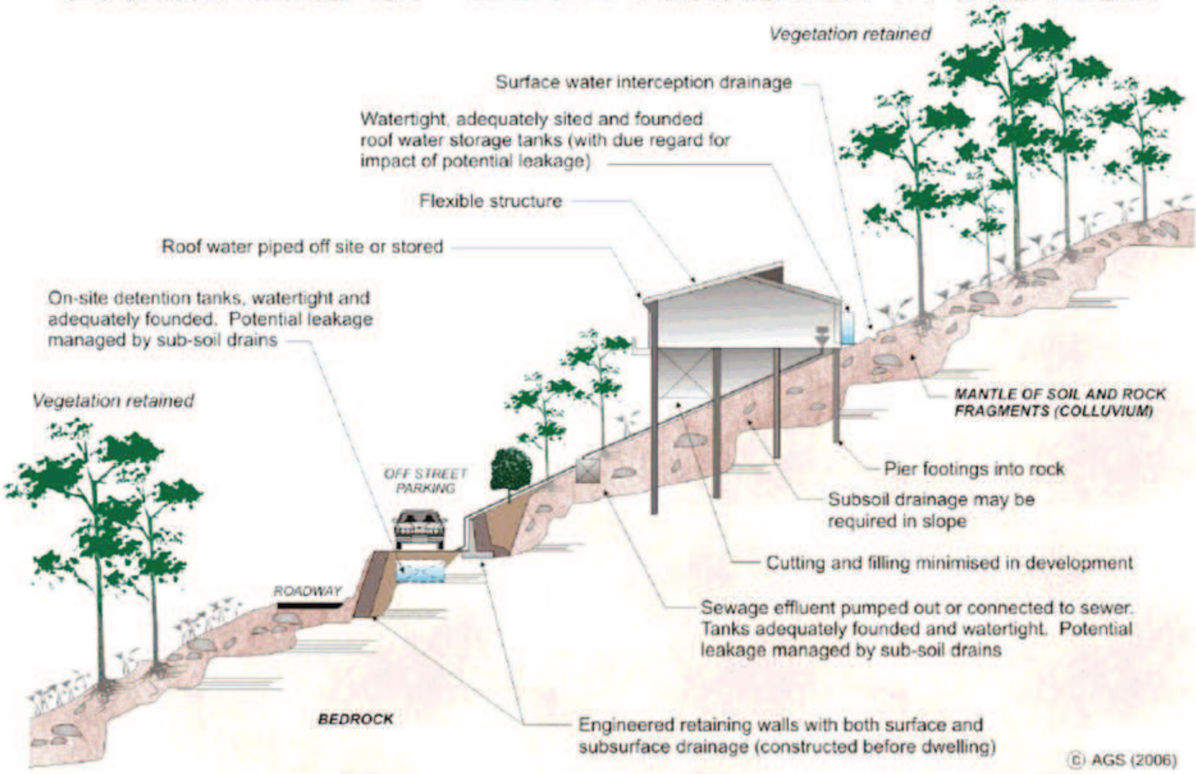
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

Appendix 5

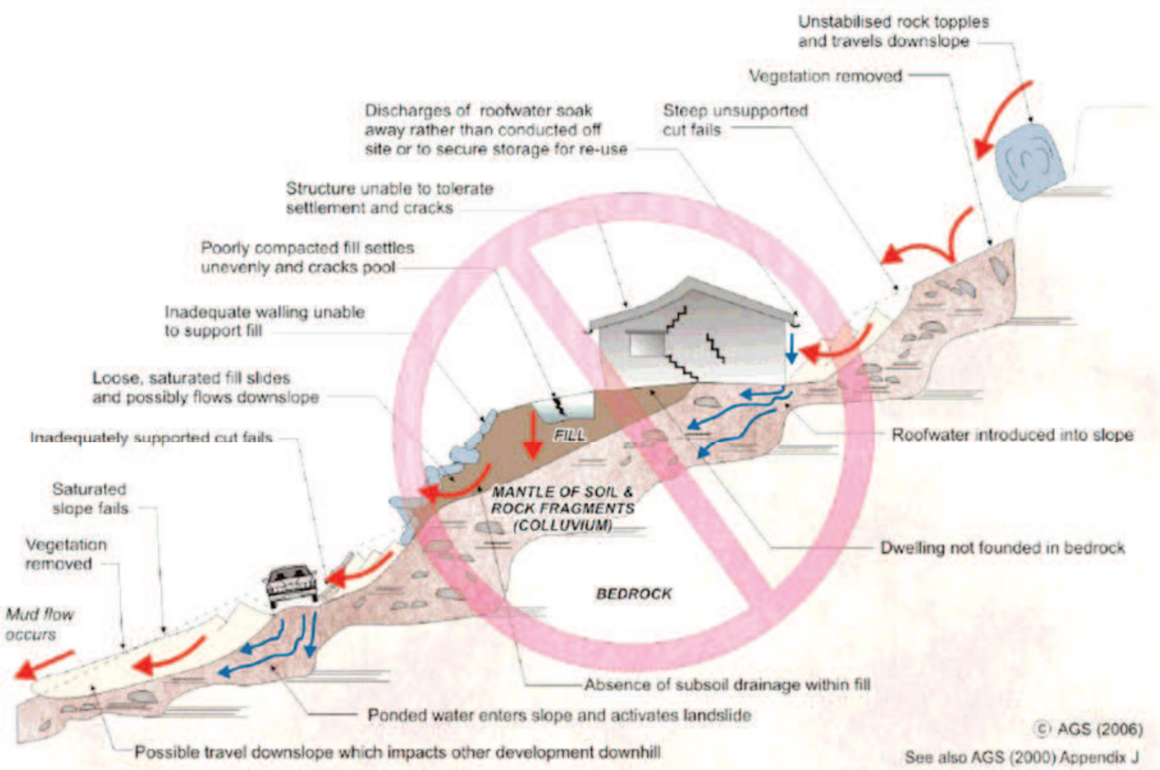
APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

	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



Appendix 6

Horton Coastal Engineering
Coastal & Water Consulting

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13 February 2025

Coastal Engineering Advice on 16 Hillcrest Avenue Mona Vale

1. INTRODUCTION AND BACKGROUND

It is proposed to undertake alterations and additions to a dwelling, and to demolish and rebuild a pool, at 16 Hillcrest Avenue Mona Vale (the 'site'). A Development Application is to be submitted to Northern Beaches Council for these works.

The site is located within a "Bluff/Cliff Instability" area designated on the *Coastal Risk Planning Map* (Sheet CHZ_018) that is referenced in *Pittwater Local Environmental Plan 2014*. Therefore, the site is subject to Chapter B3.4 of the *Pittwater 21 Development Control Plan* (DCP)[1], and the *Geotechnical Risk Management Policy for Development in Pittwater*. Based on Chapter 6.5(i) of this policy, "a coastal engineer's report on the impact of coastal processes on the site and the coastal forces prevailing on the bluff must be incorporated into the geotechnical assessment as an appendix and the Coastal Engineer's assessment must be addressed through the Geotechnical Report and structural specification". Accordingly, this coastal engineering report is set out herein.

The report author, Peter Horton [BE (Hons 1) MEngSc MIEAust CPEng NER], is a professional Coastal Engineer with 33 years of coastal engineering experience. He has postgraduate qualifications in coastal engineering, and is a Member of Engineers Australia and Chartered Professional Engineer (CPEng) registered on the National Engineering Register. He is also a member of the National Committee on Coastal and Ocean Engineering (NCCOE) and NSW Coastal, Ocean and Port Engineering Panel (COPEP) of Engineers Australia. Peter has prepared coastal engineering reports for numerous cliff/bluff properties in the former Pittwater Local Government Area over the last few decades, including along Hillcrest Avenue. He has undertaken specific inspections of the site (on 12 January 2025) and the cliff face and rock platform seaward of the site (on 3 and 5 October 2024).

All levels given herein are to Australian Height Datum (AHD). Zero metres AHD is approximately equal to mean sea level at present in the ocean immediately adjacent to the NSW mainland. Completed Form No. 1 as given in the *Geotechnical Risk Management Policy for Pittwater* is attached at the end of the document herein.

[1] The Pittwater 21 DCP up to Amendment No. 27, which came into effect on 18 January 2021, was considered herein.

2. INFORMATION PROVIDED

Horton Coastal Engineering was provided with 20 drawings of the proposed works prepared by Smith & Tzannes (Drawings A-001, 010 to 014, 020, 100 to 103, 200 to 203, 800, 850 and 851), all dated 13 December 2024 and Revision A. A site survey by Ramsay Surveyors was also provided, reference 9620 and dated 2 October 2024.

3. EXISTING SITE DESCRIPTION

The site is located at the northern end of Mona Vale Headland and immediately south and landward of Bungan Beach, with a property to the north and east at 18 Hillcrest Avenue being located adjacent to the cliff line (that is, the site is not directly adjacent to the cliff). An oblique aerial view of the site is provided in Figure 1, with a vertical aerial view of the site in Figure 2[2]. A section through the site (denoted as Section A) approximately perpendicular to the top of the cliff (and through the proposed pool) is also depicted in Figure 2. A view of the site from Bungan Beach is in Figure 3, and a view of the dwelling and pool at the site is provided in Figure 4.

Figure 1: Oblique aerial view of site (at arrow) on 22 July 2024, facing north

[2] Note that the site boundary depicted in Figure 2 is only approximate.

Figure 2: Aerial view of site (approximate red outline), with Section A in yellow and outline of proposed pool in blue (aerial photograph taken 22 September 2024)

Figure 3: View of site (at arrow) on 5 October 2024, facing SSW

Figure 4: View of seaward face of dwelling, and pool, at site (at arrow) on 12 January 2025, facing SW

Coffey & Partners (1987) noted that the cliff/bluff at the northern end of Mona Vale Headland had a stepped profile. This was noted to be primarily due to the rock type, bedding spacing and degree of weathering, with near vertical faces developed in sandstone layers, and slopes of about 45° in units composed predominantly of shale/siltstone.

Based on NSW Government Airborne Laser Scanning (ALS) data that was collected in 2020, elevations versus distance along Section A (from Figure 2) are depicted in Figure 5.

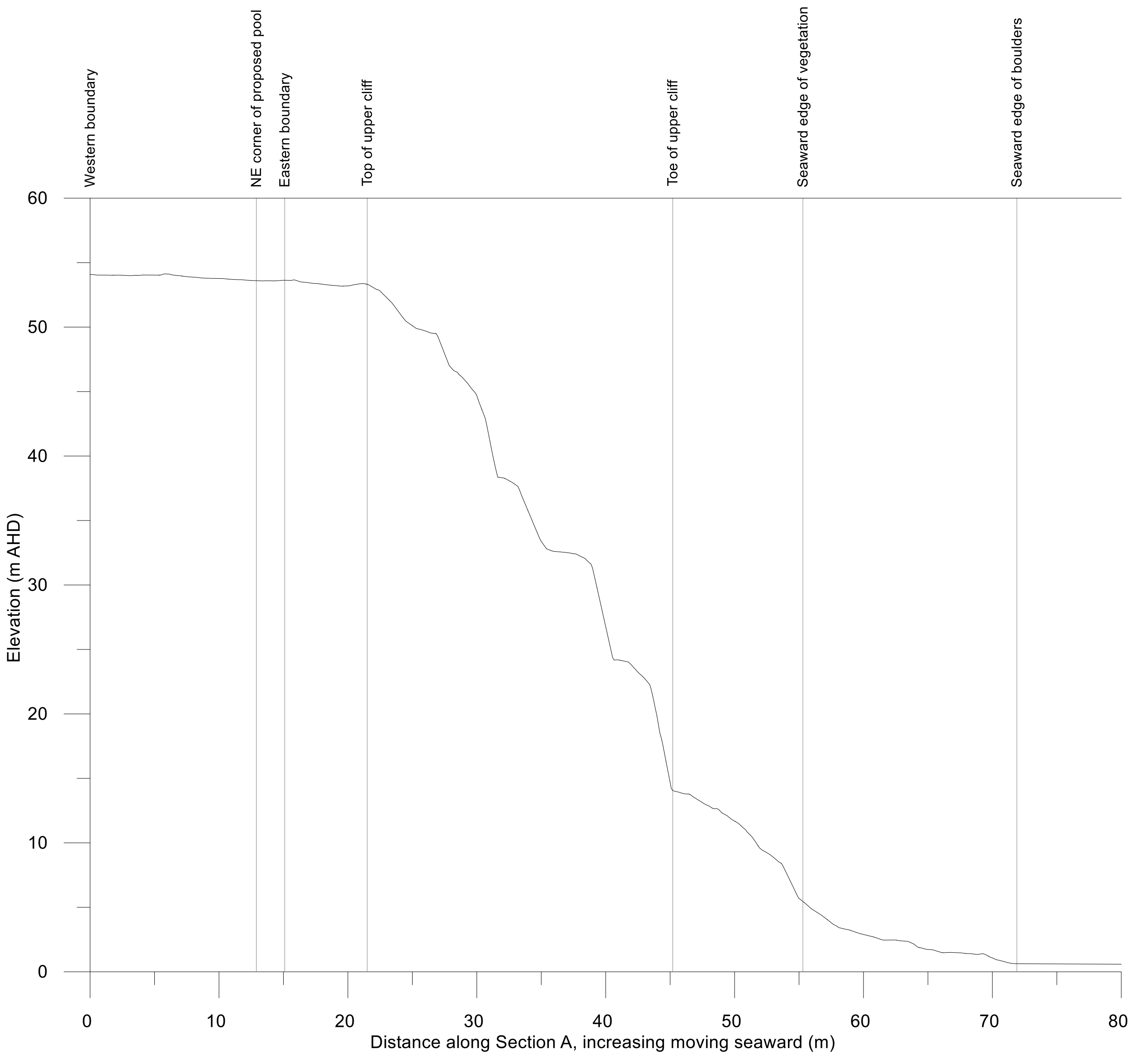

Figure 5: Section A through site, extending seaward through cliff face and rock platform

Ground elevations along Section A approximately vary from about 53m to 54m AHD in the northern part of the site, 53m AHD at the top of the upper cliff, 14.1m AHD at the toe of the upper cliff, 5m AHD at the seaward edge of vegetation, and 0.6m AHD at the seaward edge of the boulders on the rock platform. The existing dwelling has a finished ground floor level of 54.5m AHD. The average slope angle of the upper cliff from 53m AHD down to 14.1m AHD is 59°, but having a stepped profile with steeper (around 75°) and flatter sections.

4. PROPOSED DEVELOPMENT

It is proposed to undertake alterations and additions to a dwelling, and to demolish and rebuild a pool, at the site. An outline of the proposed new pool location is in Figure 2. The alterations and additions to the dwelling include moving the northern face of the dwelling about 1.2m north on the ground floor, demolishing a staircase on the northern side, and various internal changes.

The existing ground floor level of 54.5m AHD on the northern side is to be maintained in the proposed development, with the pool coping at a level of 53.5m AHD.

5. MECHANISMS FOR CLIFF EROSION

5.1 Preamble

Erosion of sheer cliffs can occur in two forms (Public Works Department, 1985), either:

- a slow, relatively gradual attrition of cliff material due to the effects of weathering; or
- relatively infrequent but sudden collapse of large portions of cliff face, due to undercutting, wave impact forces, changed groundwater conditions, rock shattering or increased loadings related to construction, and other processes.

Weathering may induce undercutting and toppling failure of overhanging blocks if the rate of weathering varies along the cliff profile. Erosion of steep slopes tends to occur suddenly in association with heavy rainfall or changes to drainage patterns, slope undercutting, and increases in load on the slope.

5.2 Weathering and Erosion

Both chemical and mechanical weathering can reduce the strength of cliff material (Sunamura, 1983). Chemical weathering includes hydration and solution, caused by the interaction between cliff material and sea water. Mechanical weathering comprises:

- the wetting and drying process in the intertidal zone;
- generation of repeated stresses in cliff material by periodic wave action (particularly waves that break on the cliff); and
- frost effects in cold latitudes.

Mechanical weathering can also be caused by wind.

Historical rates of recession for softer beds of Sydney coastline sandstone cliffs, which include chemical and mechanical weathering, have been determined to be 2mm to 5mm per year by Dragovich (2000). This is consistent with average rates of recession for Sydney Northern Beaches coastline sandstone cliffs of 4mm per year determined by Crozier and Braybrooke (1992).

An apparent approximate 40m of cliff recession (observed in aerial photography as the distance of the cliff toe from the seaward edge of the rock platform at present) seaward of the site over the last 6,400 years (since sea levels stabilised around their present levels, and assuming that this toe was at the seaward edge of the rock platform at that time) represents an average recession rate of 6mm/year, consistent with the reported rates noted above. Note that maximum rates of recession for Sydney Northern Beaches coastline sandstone cliffs of 12mm/year were determined by Crozier and Braybrooke (1992).

The exposed upper cliff above 14.1m AHD is above the intertidal zone (above about 1m AHD) and extreme 100 year Average Recurrence Interval (ARI) wave runup levels during coastal storms with large waves and elevated water levels. Wave runup could extend up to levels of about 8m AHD at present in a 100 year ARI storm, increasing to around 9m AHD in 100 years if projected sea level rise is realised. However, mechanical weathering of the upper cliff face can be caused by wind.

A recession/weathering rate of 6mm per year of the cliff face is considered to be appropriate, with sensitivity testing for a rate of 12mm/year as a conservative two multiple rate increase to account for sea level rise[3]. These rates are considered to be reasonable to apply over a design life of 100 years, including allowance for projected sea level rise as noted above.

Therefore, an allowance for recession/weathering of the cliff face of about 6mm to 12mm per year should be considered and assessed by the geotechnical engineer[4].

The geotechnical engineer should consider these estimated rates in conjunction with an understanding of the particular nature of the cliff materials at the site, their resistance to erosion/recession, and potential failure planes related to geotechnical issues such as the joint spacing[5].

This should be confirmed by the geotechnical engineer, but it is expected that the recession/weathering described above would lead to undercutting and collapse of blocks on the cliff face over the long term, with failure planes at the joints[6]. That stated, any future failure of the upper slope of the cliff and in the vicinity of the proposed development may be unrelated to coastal processes at the base of the cliff, so other failure mechanisms should be considered by the geotechnical engineer.

6. COASTAL INUNDATION

With the development above 53m AHD, coastal inundation is not a significant risk to the proposed development over a planning period of well over 100 years, including consideration of projected sea level rise.

7. MERIT ASSESSMENT

7.1 Preamble

The merit assessment herein has been undertaken assuming that the geotechnical engineer will find that the proposed development is at an acceptably low risk of damage from coastal

[3] There are no established methods to estimate increased recession rates of cliff lines due to sea level rise, but a 2.0 factor on historical rates is considered to be particularly conservative. In the 2011 *Wyong Coastal Zone Management Plan* (CZMP) and 2017 draft Wyong CZMP, a factor of 1.2 was used to 2100.
[4] Note that this does not mean that the cliff face is predicted to recede at a steady rate of 6mm to 12mm/year. In reality, there are likely to be slower rates of weathering over decades or centuries until a significant undercut occurs that detaches a block above, which leads to a sudden loss of an extent of cliff face much larger than the order of 10mm. However, averaging this slower weathering and block failures over the long term, an average rate of 6mm to 12mm/year (which can also be stated as 0.6m to 1.2m per 100 years) is expected.
[5] Coffey & Partners (1987) noted that the controlling feature of interbedded sandstone/siltstone cliffs was the bedding spacing and relative proportion of sandstone/siltstone.
[6] Overhangs are currently evident in the cliff face, as visible in Figure 3.

erosion/recession of the cliff at the site, and other processes, for a design life of at least 100 years[7]. The assessment set out below is reliant on this being the case, so this assumption must be confirmed by the geotechnical engineer.

7.2 *State Environmental Planning Policy (Resilience and Hazards) 2021*

7.2.1 Preamble

Based on *State Environmental Planning Policy (Resilience and Hazards) 2021* (SEPP Resilience)[8] and its associated mapping, the site is partly within a "Coastal Environment" area (see Section 7.2.2) and within a "Coastal Use" area (see Section 7.2.3).

7.2.2 Clause 2.10

Based on Clause 2.10(1) of SEPP Resilience, "development consent must not be granted to development on land that is within the coastal environment area unless the consent authority has considered whether the proposed development is likely to cause an adverse impact on the following:

(a) the integrity and resilience of the biophysical, hydrological (surface and groundwater) and ecological environment,
(b) coastal environmental values and natural coastal processes,
(c) the water quality of the marine estate (within the meaning of the *Marine Estate Management Act 2014*), in particular, the cumulative impacts of the proposed development on any of the sensitive coastal lakes identified in Schedule 1,
(d) marine vegetation, native vegetation and fauna and their habitats, undeveloped headlands and rock platforms,
(e) existing public open space and safe access to and along the foreshore, beach, headland or rock platform for members of the public, including persons with a disability,
(f) Aboriginal cultural heritage, practices and places,
(g) the use of the surf zone".

This is not a coastal engineering matter, but it can be noted that with regard to (a), the proposed development would not be expected to adversely affect the biophysical and hydrological (surface and groundwater) environments, being in an existing developed area and with conventional stormwater management features such as piped drainage to the street and a rainwater tank. The proposed works would not be a source of pollution as long as appropriate construction environmental controls are applied.

Assuming that there are no threatened native flora or fauna species and their habitats of significance at the site that would be impacted by the proposed works, the works would not be expected to adversely affect the ecological environment. It is understood that no trees are to be removed as part of the proposed development.

With regard to (b), the proposed development would not be expected to adversely affect coastal environmental values or natural coastal processes over an acceptably long design life, as it would be founded on a cliff well above wave action for an acceptably rare storm over an acceptably long life.

[7] At a location with underlying bedrock such as the site, it is the responsibility of the geotechnical engineer, not the coastal engineer, to determine the risk to the development.
[8] Formerly *State Environmental Planning Policy (Coastal Management) 2018*.

With regard to (c), the proposed development would not be expected to adversely impact on water quality, with the residential land use, as long as appropriate construction environmental controls are applied. No sensitive coastal lakes are located in the vicinity of the proposed development.

With regard to (d), the proposed development would not be expected to impact marine vegetation, undeveloped headlands and rock platforms, with none of these items in proximity to the development (being on an already developed headland, and being well above and landward of the rock platform seaward of the site for an acceptably rare storm and acceptably long life). No significant impacts on marine fauna and flora would be expected as a result of the proposed development, as the development would not interact with subaqueous areas for an acceptably rare storm and acceptably long life. Assuming that there are no species of native vegetation and fauna and their habitats of significance that would be impacted at the site, (d) is satisfied.

With regard to (e), it can be noted that the proposed development is entirely within the site boundary and will not alter existing public access arrangements outside of the site.

With regard to (f), a search of the Heritage NSW "Aboriginal Heritage Information Management System" (AHIMS) was undertaken on 26 November 2024. This resulted in no Aboriginal sites nor Aboriginal places being recorded or declared within at least 200m of the site.

With regard to (g), the proposed development would not interact with the surf zone for an acceptably rare storm occurring over an acceptably long life, so would not impact on use of the surf zone.

Based on Clause 2.10(2) of SEPP Resilience, "development consent must not be granted to development on land to which this clause applies unless the consent authority is satisfied that:

(a) the development is designed, sited and will be managed to avoid an adverse impact referred to in subclause (1), or
(b) if that impact cannot be reasonably avoided—the development is designed, sited and will be managed to minimise that impact, or
(c) if that impact cannot be minimised—the development will be managed to mitigate that impact".

The proposed development has been designed and sited to avoid any potential adverse impacts referred to in Clause 2.10(1).

7.2.3 Clause 2.11

Based on Clause 2.11(1) of SEPP Resilience, "development consent must not be granted to development on land that is within the coastal use area unless the consent authority:

(a) has considered whether the proposed development is likely to cause an adverse impact on the following:
    (i) existing, safe access to and along the foreshore, beach, headland or rock platform for members of the public, including persons with a disability,
    (ii) overshadowing, wind funnelling and the loss of views from public places to foreshores,
    (iii) the visual amenity and scenic qualities of the coast, including coastal headlands,
    (iv) Aboriginal cultural heritage, practices and places,

(v) cultural and built environment heritage, and

(b) is satisfied that:

(i) the development is designed, sited and will be managed to avoid an adverse impact referred to in paragraph (a), or

(ii) if that impact cannot be reasonably avoided—the development is designed, sited and will be managed to minimise that impact, or

(iii) if that impact cannot be minimised—the development will be managed to mitigate that impact, and

(c) has taken into account the surrounding coastal and built environment, and the bulk, scale and size of the proposed development".

With regard to Clause (a)(i), the proposed development is entirely on private property and will not affect public foreshore, beach, headland or rock platform access.

Clauses (a)(ii) and a(iii) are not coastal engineering matters so are not considered herein. With regard to (a)(iv), no Aboriginal sites nor Aboriginal places have been recorded or declared within at least 200m of the site, as noted in Section 7.2.2.

With regard to (a)(v), the nearest environmental heritage item to the site listed in Schedule 5 of *Pittwater Local Environmental Plan 2014* is a house at 26 Grandview Parade Mona Vale. This heritage item is located at least 130m from the site. The proposed development would not be expected to impact on this or more distant heritage items.

With regard to (b), the proposed development has been designed and sited to avoid any potential adverse impacts referred to in Clause 2.11(1) for the matters considered herein. Clause (c) is not a coastal engineering matter so is not considered herein.

7.2.4 Clause 2.12

Based on Clause 2.12 of SEPP Resilience, "development consent must not be granted to development on land within the coastal zone unless the consent authority is satisfied that the proposed development is not likely to cause increased risk of coastal hazards on that land or other land".

Assuming that the geotechnical engineer will find that the proposed development is at an acceptably low risk of damage from erosion/recession over a 100 year design life, and given that the proposed development is well above and landward of projected wave runup over 100 years, the proposed development would not even be expected to interact with coastal processes over its design life, let alone affect any other land. That is, the proposed development is unlikely to cause increased risk of coastal hazards on that land or other land over its design life.

7.2.5 Clause 2.13

Based on Clause 2.13 of SEPP Resilience, "development consent must not be granted to development on land within the coastal zone unless the consent authority has taken into consideration the relevant provisions of any certified coastal management program that applies to the land".

No certified coastal management program applies at the site.

7.2.6 Synthesis

The proposed development satisfies the requirements of *State Environmental Planning Policy (Resilience and Hazards) 2021* for the matters considered herein.

7.3 *Coastal Management Act 2016*

The management objectives for the "coastal environment" and "coastal use" coastal management areas are described in Section 8 and Section 9 respectively of the *Coastal Management Act 2016*. By addressing Clause 2.10 and 2.11 of SEPP Resilience in Section 7.2.2 and Section 7.2.3 respectively herein, these management objectives have essentially been addressed. There are no other matters relevant to the subject DA that need to be considered in the *Coastal Management Act 2016*.

7.4 *Pittwater Local Environmental Plan 2014*

7.4.1 Clause 7.5

Clause 7.5 of *Pittwater Local Environmental Plan 2014* (LEP 2014) applies at the site, as the site is identified as "Bluff/Cliff Instability" on the Coastal Risk Planning Map Sheet CHZ_018. Based on Clause 7.5(3) of LEP 2014, "development consent must not be granted to development on land to which this clause applies unless the consent authority is satisfied that the development:

(a) is not likely to cause detrimental increases in coastal risks to other development or properties, and
(b) is not likely to alter coastal processes and the impacts of coastal hazards to the detriment of the environment, and
(c) incorporates appropriate measures to manage risk to life from coastal risks, and
(d) is likely to avoid or minimise adverse effects from the impact of coastal processes and the exposure to coastal hazards, particularly if the development is located seaward of the immediate hazard line, and
(e) provides for the relocation, modification or removal of the development to adapt to the impact of coastal processes and coastal hazards, and
(f) has regard to the impacts of sea level rise, and
(g) will have an acceptable level of risk to both property and life, in relation to all identifiable coastline hazards".

With regard to (a) and (b), the proposed development would not increase coastal risks nor alter coastal processes and the impacts of coastal hazards, as it would not affect the wave impact process at the base of the cliff.

Items (c), (d) and (g) are for the geotechnical engineer to assess, with consideration of the findings herein. Assuming that they find that the proposed development is at an acceptably low risk of damage over a 100 year planning period with appropriate measures incorporated in design and construction, (c), (d) and (g) would be met. On this basis, (e) should not be necessary, noting that this would be more applicable in a sandy beach environment. With regard to (f), sea level rise has been considered herein.

7.4.2 Clause 7.8

There is no Foreshore Building Line or Foreshore Area at the site, so Clause 7.8 of LEP 2014 does not apply at the site.

7.5 *Pittwater 21 DCP*

Based on Chapter B3.4 of the DCP, "development must not adversely affect or be adversely affected by geotechnical and coastal processes nor must it increase the level of risk for any people, assets and infrastructure in the vicinity due to geotechnical and coastal processes".

As noted in Section 7.2.4, the proposed development is not expected to increase the level of risk for any people, assets and infrastructure in the vicinity due to coastal processes. This item is satisfied if the geotechnical engineer confirms that the proposed development is at an acceptably low risk if being affected by geotechnical and coastal processes, and unlikely to increase the level of risk for any people, assets and infrastructure in the vicinity due to geotechnical processes.

8. FORM

A completed *Geotechnical Risk Management Policy for Pittwater* Form No. 1 is attached at the end of the document herein. Note that the declaration on Form No. 1 is not appropriate for a coastal report, with the revised declaration below:

> "I am aware that the above Coastal Report, prepared for the abovementioned site is to be submitted to assist with a geotechnical investigation for a Development Application for this site, with that geotechnical investigation relied on by Northern Beaches Council as the basis for ensuring that the Geotechnical Risk Management aspects of the proposed development have been adequately addressed. No declaration can be made on the geotechnical investigation as this has not been prepared nor reviewed by me, and nor do I have geotechnical engineering expertise".

9. CONCLUSIONS

An allowance for erosion/weathering of 6mm/year of the cliff seaward of 16 Hillcrest Avenue Mona Vale, with sensitivity testing up to 12mm/year, should be considered and assessed by the geotechnical engineer. The geotechnical engineer should consider these estimated rates in conjunction with an understanding of the particular nature of the cliff materials at the site, their resistance to erosion, and potential failure planes related to geotechnical issues such as the joint spacing. That stated, any future failure of the upper slope of the cliff and in the vicinity of the proposed development may be unrelated to coastal processes at the base of the cliff, so other failure mechanisms should be considered by the geotechnical engineer.

Coastal inundation is not a significant risk to the proposed development over a planning period of well over 100 years. Given this, and assuming that the geotechnical engineer will find that the development is at an acceptably low risk of damage from erosion/recession over a 100 year design life, the proposed development satisfies the requirements of *State Environmental Planning Policy (Resilience and Hazards) 2021* (Clauses 2.10 to 2.13), the *Coastal Management Act 2016*, Clause 7.5 of *Pittwater Local Environmental Plan 2014*, and Chapter B.4 of the *Pittwater 21 DCP* for the matters considered herein.

10. REFERENCES

Coffey & Partners (1987), "Coastal Management Study, Assessment of Bluff Areas", *Report No. S8002/1-AA*, March, for Warringah Shire Council

Crozier, PJ and JC Braybrooke (1992), "The morphology of Northern Sydney's rocky headlands, their rates and styles of regression and implications for coastal development", *26th Newcastle Symposium on Advances in the Study of the Sydney Basin*, University of Newcastle

Dragovich, Deirdre (2000), "Weathering Mechanisms and Rates of Decay of Sydney Dimension Sandstone", pp. 74-82 in *Sandstone City, Sydney's Dimension Stone and Other Sandstone Geomaterials*, edited by GH McNally and BJ Franklin, Environmental, Engineering and Hydrogeology Specialist Group (EEHSG), Geological Society of Australia, Monograph No. 5

Public Works Department (1985), "Coastal Management Strategy, Warringah Shire, Report to Working Party", *PWD Report 85016*, June, prepared by AD Gordon, JG Hoffman and MT Kelly, for Warringah Shire Council

Sunamura, Tsuguo (1983), "Processes of Sea Cliff and Platform Erosion", Chapter 12 in *CRC Handbook of Coastal Processes and Erosion*, editor Paul D Komar, CRC Press Inc, Boca Raton, Florida, ISBN 0-8493-0208-0

11. SALUTATION

If you have any further queries, please do not hesitate to contact Peter Horton via email at peter@hortoncoastal.com.au or via mobile on 0407 012 538.

Yours faithfully
HORTON COASTAL ENGINEERING PTY LTD

Peter Horton
Peter Horton
Director and Principal Coastal Engineer

Geotechnical Risk Management Policy for Pittwater Form No. 1 is attached overleaf

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for Aston Building
Name of Applicant

Address of site 16 Hillcrest Avenue Mona Vale

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

I, Peter Horton *on behalf of* Horton Coastal Engineering Pty Ltd
(Insert Name) (Trading or Company Name)

on this the 13 February 2025 certify that I am a geotechnical engineer or engineering geologist or coastal engineer as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above organisation/company to issue this document and to certify that the organisation/company has a current professional indemnity policy of at least $2million.

I:

Please mark appropriate box

- ☐ 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

Coastal
~~Geotechnical~~ Report Details:

Report Title: Coastal Engineering Advice on 16 Hillcrest Avenue Mona Vale

Report Date: 13 February 2025

Author: Peter Horton

Author's Company/Organisation: Horton Coastal Engineering Pty Ltd

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

See Section 2 and Section 10 of coastal report

~~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 stated and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.~~

Signature Peter Horton ………………………………………… See revised declaration in Section 8 of report

Name …Peter Horton……………………………………

Chartered Professional Status…MIEAust CPEng NER

Membership No. …452980……………………………

Company…Horton Coastal Engineering Pty Ltd