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GEOTECHNICAL REPORT

for

PROPOSED NEW DWELLING

at

18 HILLCREST AVENUE, MONA VALE NSW

Prepared For

Neil Burnard and Jennifer Robins

Project No.: 2022-039.2

November 2024

Document Revision Record

Issue No	Date	Details of Revisions
0	7 February 2023	Draft Issue
1	28 February 2023	Final Issue
2	12 November 2024	Revised Design

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for____	
Name of Applicant	
Address of site ___ 18 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*** *12* November 2024 certify that I am a ~~geotechnical engineer or~~ engineering geologist ~~or coastal engineer~~ as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above ~~organisation/~~company to issue this document and to certify that the ~~organisation/~~company has a current professional indemnity policy of at least $2million.

I:

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

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

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

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

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

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

Geotechnical Report Details:

Report Title: Geotechnical Report for Proposed New Residence

Report Date: 12 November 2024 Project No.: 2022-039.1

Author: Kieron Nicholson and Troy Crozier

Author's Company/Organisation: Crozier Geotechnical Consultants

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

Survey Plan Mepstead and Associates, Ref.: 5810-DET1_A, Dated: 4 February 2020.

Architectural drawings – Progressive Plans, Project No.:1010, Drawings: DA00 to DA23, Dated: 9/9/2024.

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

Signature ....

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

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

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

Company... **Crozier Geotechnical Consultants**

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

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

Development Application for ____

Name of Applicant

Address of site ___18 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 New Dwelling
Report Date: 12 November 2024 **Project No.:** 2022-039.1
Author: Kieron Nicholson and Troy Crozier
Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

- ■ Comprehensive site mapping conducted ____21 March 2022 and 2 October 2024 __________
- ■ 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 21/3/22 ....
- ■ 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 ...
- ■ 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**

AUSTRALIAN INSTITUTE OF GEOSCIENTISTS
TROY CROZIER
10197

CROZIER
GEOTECHNICAL CONSULTANTS

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3	Landslip Risk Assessment Tables
4	AGS Terms and Descriptions
5	Hillside Construction Guidelines
6	Coastal Engineering Assessment

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GEOTECHNICAL REPORT FOR PROPOSED NEW RESIDENCE AT 18 HILLCREST AVENUE, MONA VALE, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for a proposed residential development at 18 Hillcrest Avenue, Mona Vale, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of the clients Neil Burnard and Jennifer Robins.

It is understood that the proposed works involve the construction of a new dwelling within the north of the site as well as demolition of the existing main residence within the south of the site and construction of a new main residence. The construction of the new dwelling within the north of the site is currently underway however a separate DA is proposed for the main residence. This report is to provide information to enable the structural design and support the Development Application (DA) submission for the main residence. This report has been updated from a previous geotechnical report to reflect the current architectural design for the main house.

The site is located within the H1 (highest category) landslip hazard zone as identified within Northern Beaches Councils Geotechnical Hazard Mapping (Geotechnical Risk Management Policy for Pittwater – 2009).

The site is also located in coastal hazard zone 'R' (Bluff/Cliff instability) as identified on the Northern Beaches Coastal Risk Planning Map therefore this report also includes a Coastal Engineering Report provided by a qualified Coastal Engineer (Appendix 6).

The investigation and original reporting were undertaken as per the Proposal: P22-065, Dated: 21 February 2022.

The investigation comprised:

a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Senior Engineering Geologist.

b) Drilling of five boreholes using hand tools along with six Dynamic Cone Penetrometer (DCP) tests to investigate the subsurface conditions.

The following plans and drawings were supplied for the work:

- Survey Plan Mepstead and Associates, Ref.: 5810-DET1_A, Dated: 4 February 2020.
- Architectural drawings – Progressive Plans, Project No.:1010, Drawings: DA00 to DA23, Dated: 9/9/2024.

1.1 Proposed Development

It is understood the proposed works comprise the demolition of the existing site house and the construction of a new two-storey house with garage, workshop and plant room under. A pool and spa are also proposed.

The proposed works appear to require up to 2.0m depth excavation to allow the construction of the garage slab.

2. SITE FEATURES

2.1 Site Description

The site is irregular in shape and covers an area of approximately 3495m^2 in plan as referenced from the provided survey drawing. It is located on the high north side of the road within gently to very steeply northeast dipping topography and the elevation varies between a high of RL55.6m adjacent to the existing site dwelling and a low of an estimated RL2.0m near the mean high-water mark (MHWM) adjacent to Bungan Beach to the east. It has combined north, east, south and west boundaries of 48.1m, approximately 49m in a straight line (defined by the MHWM), 80.2m and 93.8m respectively as determined from the survey plan provided.

An aerial photograph of the site and its surrounds is provided below (Photograph 1), as sourced from the NSW Government website Six Maps with the compass directions assigned to the boundaries indicated.

Photograph 1: Aerial photo of site (outlined red) and surrounds

The site contains the main site dwelling, rear lawn, concrete driveway with a twin garage with access pathways, wooden deck and planter beds retained by a stone retaining wall approximately 0.9m in height. Within the north of the site a sparsely vegetated slope is present which provides access to a set of stairs which lead down to Bungan Beach.

The main site dwelling comprises a single storey brick dwelling which is accessed via a concrete pathway to the east of the structure which also provides access to the timber deck at the rear. A mature tree is present within the rear garden of the dwelling.

Within the north of the property, the secondary dwelling was currently under construction.

General views of the site are provided in Photograph 2 and 3.

Photograph 2: View of the front of the site looking north


Photograph 3: View of the site looking south from the rear garden

The site is bordered to the north, east, south and west by 154 Barrenjoey Road/Council owned land, Bungan Beach, 20 Hillcrest Avenue/Hillcrest Avenue carriageway and 12 and 16 Hillcrest Avenue respectively.

No.154 Barrenjoey Road contains a two-storey masonry house and front and rear gardens. The house structure is approximately 15m from the shared boundary and the property is at a similar level to the site immediately adjacent to the shared boundary.

No.20 Hillcrest Avenue contains a single storey brick and weatherboard house with front garden and rear deck. The house structure is approximately 1.0m from the shared boundary and the property is at a slightly lower elevation level to the site immediately adjacent to the shared boundary.

Hillcrest Avenue contains an asphalt pavement and concrete kerb which dips to the west where it passes the site.

No.12 Hillcrest Avenue contains a one and two storey brick house with front and rear gardens, driveway and in-ground pool. The house structure is approximately 1.5m from the shared boundary and the property is at similar level to the site immediately adjacent to the shared boundary.

No.16 Hillcrest Avenue contains a two storey brick house with rear gardens, driveway and inground pool. The house structure is approximately 1.0m from the shared boundary and the property is at similar level to the site immediately adjacent to the shared boundary.

2.2 Geology

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is underlain by weathered bedrock of the 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 and has a tendency to weather to significant depth.

Narrabeen Group rocks are dominated by shales and thin siltstone/sandstone beds and often form rounded convex ridge tops with moderate angle (<20°) side slopes. These side slopes can be either concave or convex depending on geology, internally they comprise of interbedded shale and siltstone beds with close spaced bedding partings that have either close spaced vertical joints or in extreme cases large space convex joints. The shale often forms deeply weathered profiles with silty or medium to high plasticity clays and a thin silty colluvial cover. The bedrock may be thinly interbedded with very low to low strength siltstone/shale units and medium to high strength sandstone horizons.
A dyke intruded during the Jurassic period is shown very near or within the site trending broadly north-south however at the 1:100,000 scale, the location should be considered approximate only.

An extract of the relevant geological map is provided as Extract 1.

Extract 1: Extract of the 9130 Geology Series Map

2.3 Coastal Erosion

The site is located in coastal hazard zone ‘R’ (Bluff/Cliff instability) as identified on the Northern Beaches Coastal Risk Planning Map Sheet CHZ_015 (shown in Extract 2), as such a Coastal Hazard Assessment is required as per Council requirements.

Extract 2: Extract of the relevant Northern Beaches Coastal Hazard Map with the site circled red

3. FIELD WORK:

3.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on the 21 March 2022 by a Senior Engineering Geologist. 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, rock outcrops, existing structures and neighbouring properties. It also included the drilling of five boreholes (BH1 to BH5) using a hand auger to investigate sub-surface geology. A hand auger was used as access to the majority of the site for a conventional drilling rig was unavailable.

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 soil conditions and confirm depths to bedrock.

Additionally, a second walk over inspection was undertaken on the 2 October 2024 by a Senior Engineering Geologist.

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

3.2. Field Observations:

The existing site residence is generally in good condition with the exception of a crack on the external southern wall as shown on Photograph 4. The crack is present near the front of the house and is likely related to settlement of founding strata rather than an indicator of a deep-seated landslip hazard.

Photograph 4: Cracking observed in the external wall.

The retaining wall (located adjacent to the site driveway) did display some rotation which may be related to the growth of an adjacent tree or inadequate construction and is considered unlikely to represent a deep-seated geotechnical issue.

Photograph 5: View of rotating wall at the front with the site.

Within the site garage an outcrop of very low strength sandstone interbedded with shale was observed and overlain by approximately 0.8m thickness of residual clay soils (See Photograph 6)

Photograph 6: View of bedrock outcrop in the garage

Adjacent to the rear deck of the site dwelling and near the crest of the cliff, recent erosion/landslip of topsoil/residual soils exposed bedrock approximately 1.5-2.0m below the elevation of the garden and is shown in Photograph 7.

Photograph 7: View of bedrock outcrop near crest of the cliff

The cliff face located east of the site house is approximately 50m high with inter-bedded layers of sandstone and shale/siltstone outcropping over the face, with no significant overhangs. The upper 20m of the cliff is steeply (approx. -40°) sloping and covered with vegetation. The middle section of the cliff face is near vertical and formed with inter-bedded sandstone and shale/siltstone before the lower 10m is steeply sloping down to a boulder foreshore terrace.

Within the site indications of distress were observed within the access steps which lead down to Bungan Beach to the east and indicated in Photograph 8. The separation/cracking observed is considered to be the result of settlement/creep of colluvial soils and the shallow founding of the path in this material and not an indication of a significant geotechnical issue.


Photographs 8 and 9: View of separation with the beach access steps looking west

The property structures to the north and west did not display any significant signs of distress based on observations made from within the site.

Based on previous work within the property to the south, some minor cracking was observed within a previous (now replaced) wall within the property however the distress observed was not considered to represent significant geotechnical or slope stability issues.

The neighbouring properties and structures were inspected from the site or road reserves, however visible aspects did not indicate the presence of large-scale geotechnical hazards which may impact the site.

3.3. Subsurface Investigation:

For a description of the subsurface conditions encountered at the test locations, the Borehole Log Reports and Dynamic Penetrometer Test Sheets should be consulted, however a very broad description is provided below.

Unit	Strata Description
TOPSOIL	Topsoil was encountered within all boreholes to a maximum depth of 0.4m (BH1) and predominately comprised clayey sand with gravel.
CLAY (Colluvium)	This deposit was encountered in BH2, BH3 and BH4 within the north of the site and comprised stiff yellow brown clay which contained cobbles and gravel. The boreholes all refused on interpreted cobbles between 0.6m (BH4) and 0.8m (BH2).
SILTY CLAY (Residual Soil)	This deposit was encountered in BH1 and BH5 within the south of the site to a maximum depth of 1.4m and comprised stiff dark brown locally yellow brown silty clay. It is likely this stratum is also present below the colluvial soils within the other boreholes however this could not be confirmed due to auger refusal.
SANDSTONE/SHALE (Newport Formation)	This deposit was not recovered from the boreholes however it is interpreted to be strata on which the DCP tests refused based on the exposures observed within the cliff face and within the garage.

A free-standing ground water table or significant water seepage were not identified within any of the boreholes or observed on the DCP rods on extraction.

4. COMMENTS:

4.1. Geotechnical Assessment:

The DCP's/field mapping undertaken at the site indicated what has been interpreted as in situ sandstone/shale bedrock is present at depths between 1.2m and 2.3m below ground surface levels.

Inspection of sandstone outcrops within the site indicated that the bedrock is likely to range from very low to low strength with localised clay partings. Defects within the bedrock predominately comprised near horizontal bedding defects.

Significant geotechnical hazards were not observed however erosion/previous landslip was observed near the crest of the cliff adjacent to the site deck during the initial phase of fieldwork. This feature is typical following heavy rainfall which occurred before the initial inspection were completed and represents a normal cliff process which periodically exposes bedrock and not representative of a significant deep seated landslip hazard.

The bedrock observed within the cliff line did not appear to exhibit signs of imminent landslip/overhanging sections.

No signs of imminent instability were observed and based on the findings of the Coastal Engineering report, it is considered that the proposed development is unlikely to impact the long-term stability of the adjacent cliff face. However, it will be necessary to inspect all excavations such that any potential stability risks can be mitigated in a timely manner.

The cliff recession rate of 0.6m to 1.2m over 100 years from Horton Coastal Engineering (see Appendix 6) has been considered, along with the typical joint spacing at the site of 1.0m – 2.0m, and potential instability in the soil layer above the rock if a block failure occurred, to estimate the landward extent of cliff instability over the design life. This extent is located approximately 30m from the proposed dwelling. Therefore, 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 at least 100 years.

Considering the potential geotechnical failure mechanisms at the site, the proposed development is at an acceptably low risk of damage from geotechnical processes for a design life of at least 100 years. The proposed development is unlikely to increase the level of risk for any people, assets and infrastructure in the vicinity due to geotechnical processes.

Based on the ground conditions encountered it is recommended that footings are socketed into the bedrock exposed using the bearing pressures provided in Section 4.3. The strength of the bedrock with depth is unconfirmed therefore there is a potential for the bedrock to be more deeply weathered and of lesser strength than interpreted. For confirmation of bedrock strength to below proposed excavation/footing level will need an investigation utilizing cored boreholes in the actual footing locations will be required, however access for such equipment is very limited by site conditions whilst the proposed excavations are relatively minor. As such bedrock strength at footing level can be confirmed by geotechnical inspection during ground works.

Excavation depths for the new garage slab are a maximum of approximately 2.0m depth. The south side of this excavation appears to vary between 1.0m and 1.83m from the adjacent shared boundary within No.20. Based on the anticipated ground conditions, it is considered safe batter slopes (provided in Section 4.3) will not be achievable within this area therefore pre-excavation support (e.g. bored piles/incremental excavation/support) will be required to support the adjacent boundary. It appears the remaining portions of the excavation can utilise temporary batter slopes at adequate distances from all other shared boundaries.

Based on the relatively shallow excavation anticipated in bedrock and weak strength it is unlikely vibration monitoring will be required providing lightweight hammers (<250kg) are used to excavate any rock encountered. However, this will need to be confirmed during excavation based on actual plant proposed for use.

4.2. Site Specific Risk Assessment

Based on our site investigation we have identified the following geological/geotechnical landslip hazards which need to be considered in relation to the existing site and the proposed works. The hazards are:

A. Landslip (earth slide <3m^3) from the excavation for garage excavation.
B. Landslip (topple/slide <2.0m) due to adverse jointing.
C. Landslip of existing near surface soils similar to that seen in previous inspection

A qualitative assessment of risk to life and property related to the 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 **7.81×10^{-7}** for in the adjacent properties and **3.25×10^{-7}** for persons within the site garden. The Risk to Life from Hazard B for persons in adjacent properties of the loss of detached blocks was estimated to be to be **4.69×10^{-8}**. The Risk to Life from Hazard C for persons in adjacent properties of the loss of detached blocks was estimated to be to be **3.75×10^{-8}**. The Risk to Property was considered to be 'Very Low' in all situations. These hazards were therefore considered to be 'Acceptable' when assessed against the criteria of the AGS 2007.

4.3 Preliminary Design & Construction Recommendations:

Preliminary Design and Construction recommendations are tabulated below:

4.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Class 'A' for footings founded within bedrock
Type of Footing	Strip or pad where bedrock exposed in excavation or piers extending into bedrock.
Sub-grade material and Maximum Allowable Bearing Capacity	- VLS bedrock : 750kPa

Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e – Rock site (provided entire new structure founded to bedrock).
Remarks: All permanent structure footings should be founded within bedrock of similar strength to prevent differential settlement unless designed for by the structural engineer.	

4.3.2. Excavation

Depth of Excavation	Approximately 2.0m depth for garage excavation

Table 1 below shows the properties potentially affected by the proposed excavation and the separation distances to the shared property boundary and structures.

Table 1: Property Separation Distances

Boundary	Adjacent Property	Bulk Excavation Depth (m bgl)	Separation Distances (m)	
			Boundary	Building
North	154 Barrenjoey Road	2.0	>20.0	>25.0
East	Not Applicable	2.0	-	-
South	20 Hillcrest Avenue	2.0	1.0	2.5
West	16 Hillcrest Avenue	0.5	1.32	5.0

Type of Material to be Excavated	Clay soils and likely very low to low strength bedrock.

Guidelines for <u>un-surcharged</u> batter slopes for general information are tabulated below:

Material	Safe Batter Slope (H:V)	
	Short Term/ Temporary	Long Term/ Permanent
Natural clay soils	1.25:1	2:1
Very Low to low strength bedrock	0.5:1	1:1

Remarks:

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

Geotechnical inspection of batters will be required at regular intervals to assess their stability, especially for permanent batters.

The presence of defects within fractured rock may require a significant reduction to the maximum batter

slopes provided.		
Equipment for Excavation	Clay soils and very low to low strength bedrock.	Excavator with Bucket, assisted with ripper as required.
Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Not applicable unless heavy (>250kg) hammers used	
Vibration Calibration Tests Required		
Full time vibration Monitoring Required		
Dilapidation Surveys Requirement	Not critical, although will prevent spurious claims for damage	
Remarks: Water ingress into exposed excavations can result in erosion and stability concerns in soils. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope, whilst any groundwater seepage must be controlled within the excavation and prevented from ponding or saturating slopes/batters.		

4.3.3. Retaining Structures:	
Required	For garage excavation
Types	Steel reinforced concrete/concrete block wall post excavation or pre-excavation support using soldier piles or similar where insufficient space prevents the construction of safe batter slopes. Any retaining structures should be designed in accordance with Australian Standard AS 4678-2002 Earth Retaining Structures.
Parameters for calculating pressures acting on retaining walls for the materials likely to be retained:	

Material	Unit Weight (kN/m3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Stiff Clay	20	$\phi' = 30°$	0.33	0.47	3.25
VLS or fractured bedrock	23	$\phi' = 40°$	0.10	0.15	400kPa

Remarks:

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

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

4.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		Not encountered
Excavation likely to intersect	Water Table	No
	Seepage	Minor (<0.50L/min), possible at fill/natural soil and soil/bedrock interfaces
Site Location and Topography		High east side of the road, within steeply moderately to steeply north and east dipping topography.
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Due to the presence of impermeable bedrock/clay soils the property is not suitable for onsite absorption disposal system. The site may be suitable for a dispersion system utilising an Onsite Detention System (OSD) and a level spreader designed by a suitably qualified Hydraulic Engineer.
Remarks: Trenches, as well as all new building gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which discharges off site.		

4.4 Conditions Relating to Design and Construction Monitoring:

To allow certification as part of construction, building and post-construction activity for this project, it will be necessary for geotechnical:

1. Review structural design drawings for implementation of the recommendations of this report (Form 2B)
2. Inspect installation of pre-excavation support systems and where bedrock is encountered in excavation
3. Inspect all new footings to confirm compliance to design assumptions with respect to allowable bearing pressure and stability prior to the placement of steel or concrete.

4. Where ground conditions vary from those anticipated and outlined in this report are encountered.

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

5. SUMMARY:

Based on the results of the investigation it appears interpreted very low strength to low strength bedrock underlies the site between approximately 1.2m and 2.3m depth and is overlain by stiff clay soils.

Temporary batters appear feasible for all excavation perimeters with the exception of the excavation required adjacent to the south boundary shared with No.20 Hillcrest Avenue. It is envisaged either bored pile wall pre-excavation will be necessary or excavation and construction in stages to ensure the integrity of the shared boundary is maintained.

New footings should extend through clay soils and found within the very low to low strength bedrock (via piers) if necessary, socketed at least one full diameter into the founding strata to resist near surface soil creep pressures.

Apparently stable bedrock was observed within an area of landslip/erosion near the crest of the cliff adjacent to the existing site residence deck however it appears the results of erosion and not representative of a larger or continuing landslip hazard.

Subject to proposed excavation location and extent, rock excavation equipment or vibration monitoring does not appear necessary.

The landslip risk was assessed as ‘Acceptable’ when assessed against the criteria of the AGS 2007.

Kieron Nicholson

Prepared by:
Kieron Nicholson
Senior Engineering Geologist

Reviewed by:
Troy Crozier
Principal
MAIG. RPGeo; 10197

6. REFERENCES:

1. Australian Geomechanics Society 2007, "Landslide Risk Assessment and Management", Australian Geomechanics Journal Vol. 42, No 1, March 2007.
2. Geological Society Engineering Group Working Party 1972, "The preparation of maps and plans in terms of engineering geology" Quarterly Journal Engineering Geology, Volume 5, Pages 295 - 382.
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 – 1996, Residential Slabs and Footings – Construction
6. Australian Standard AS1170.4 – 2007, Part 4: Earthquake actions in Australia

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 1

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Brookvale NSW 2100 Email: info@croziergeotech.com.au
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

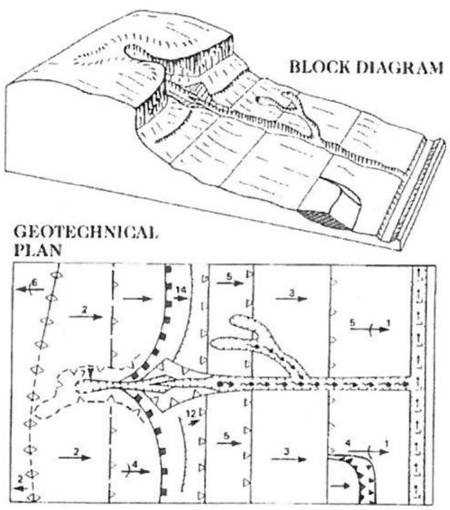
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

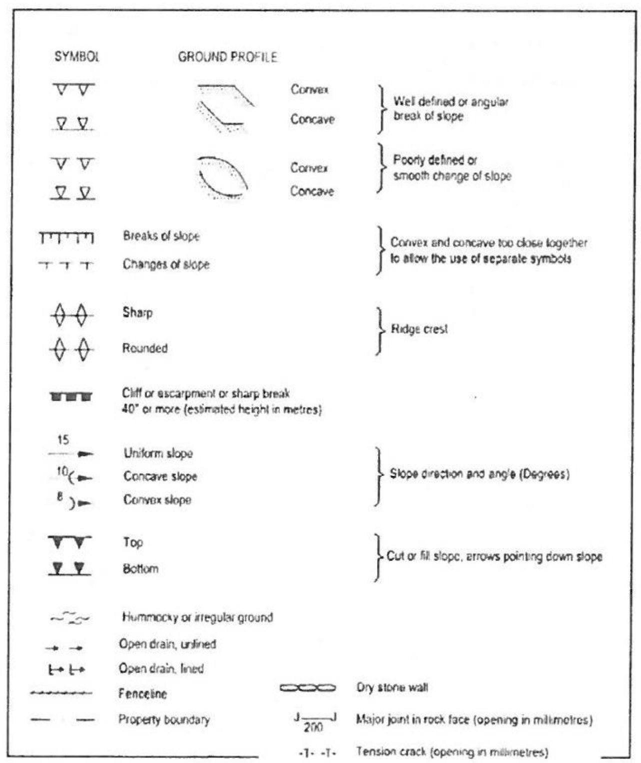
Reproduction of Information for Contractual Purposes

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

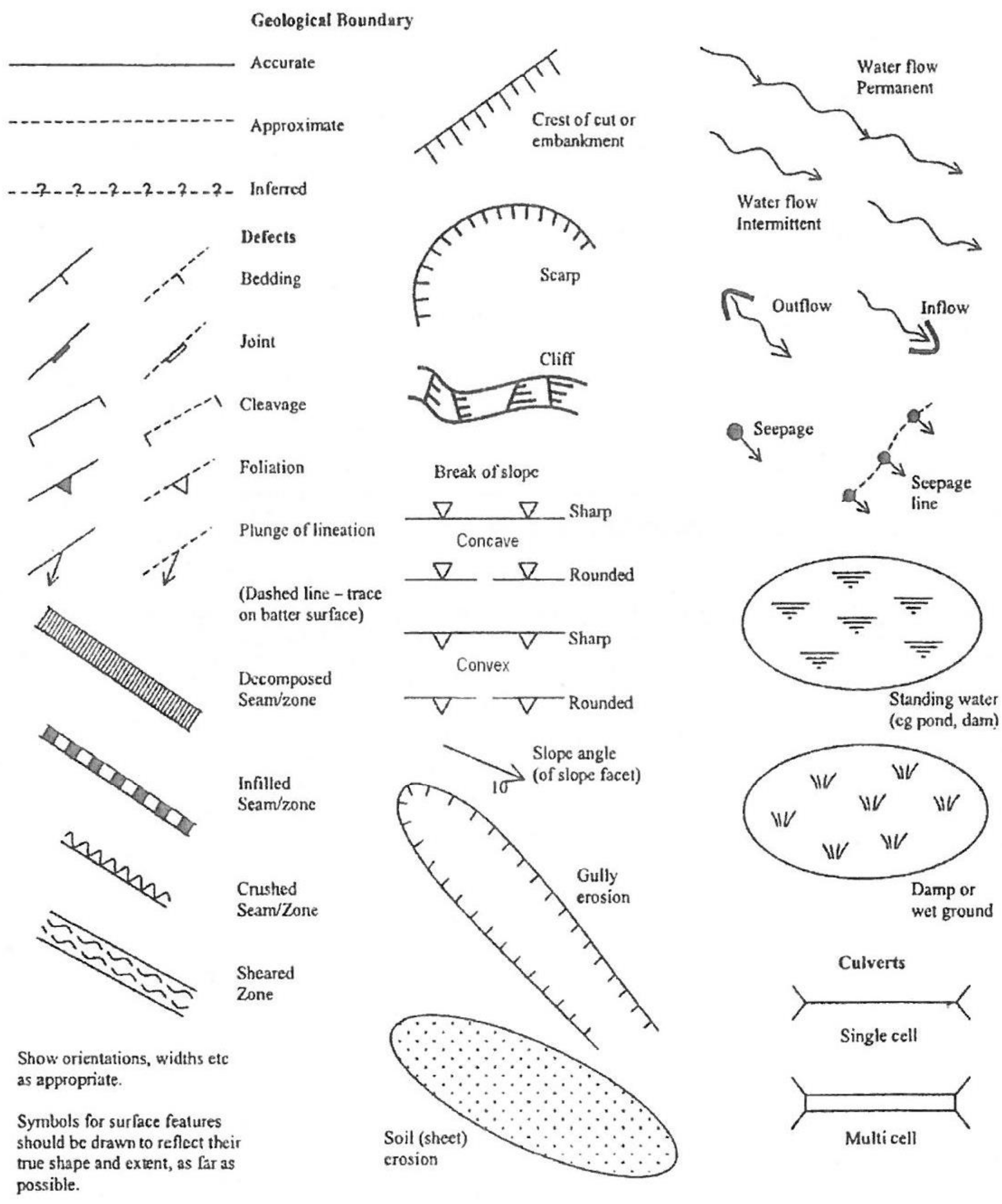
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





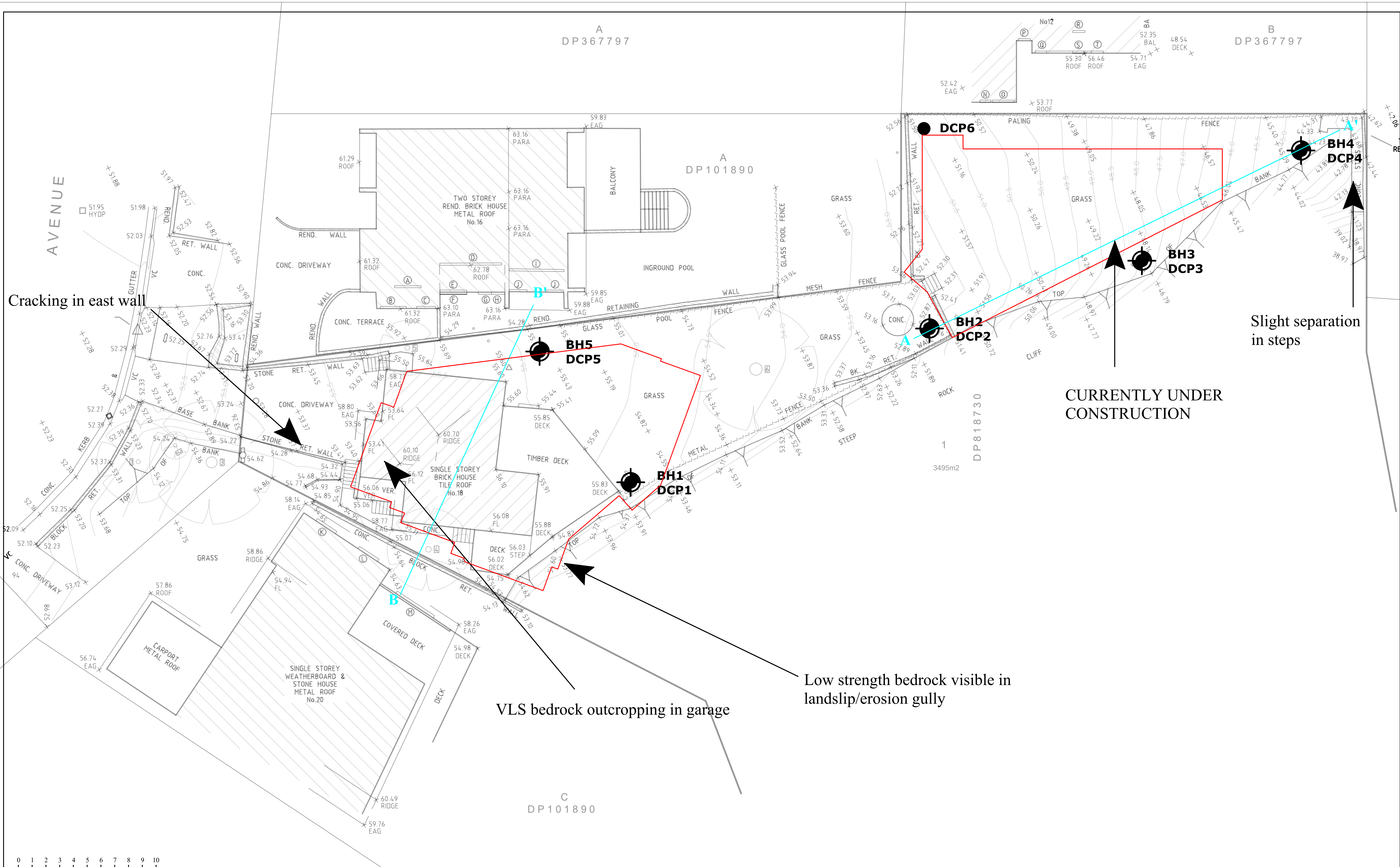
Example of Mapping Symbols
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

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).

Appendix 2



SITE PLAN & TEST LOCATIONS FIGURE 1.

CROZIER
GEOTECHNICAL CONSULTANTS

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

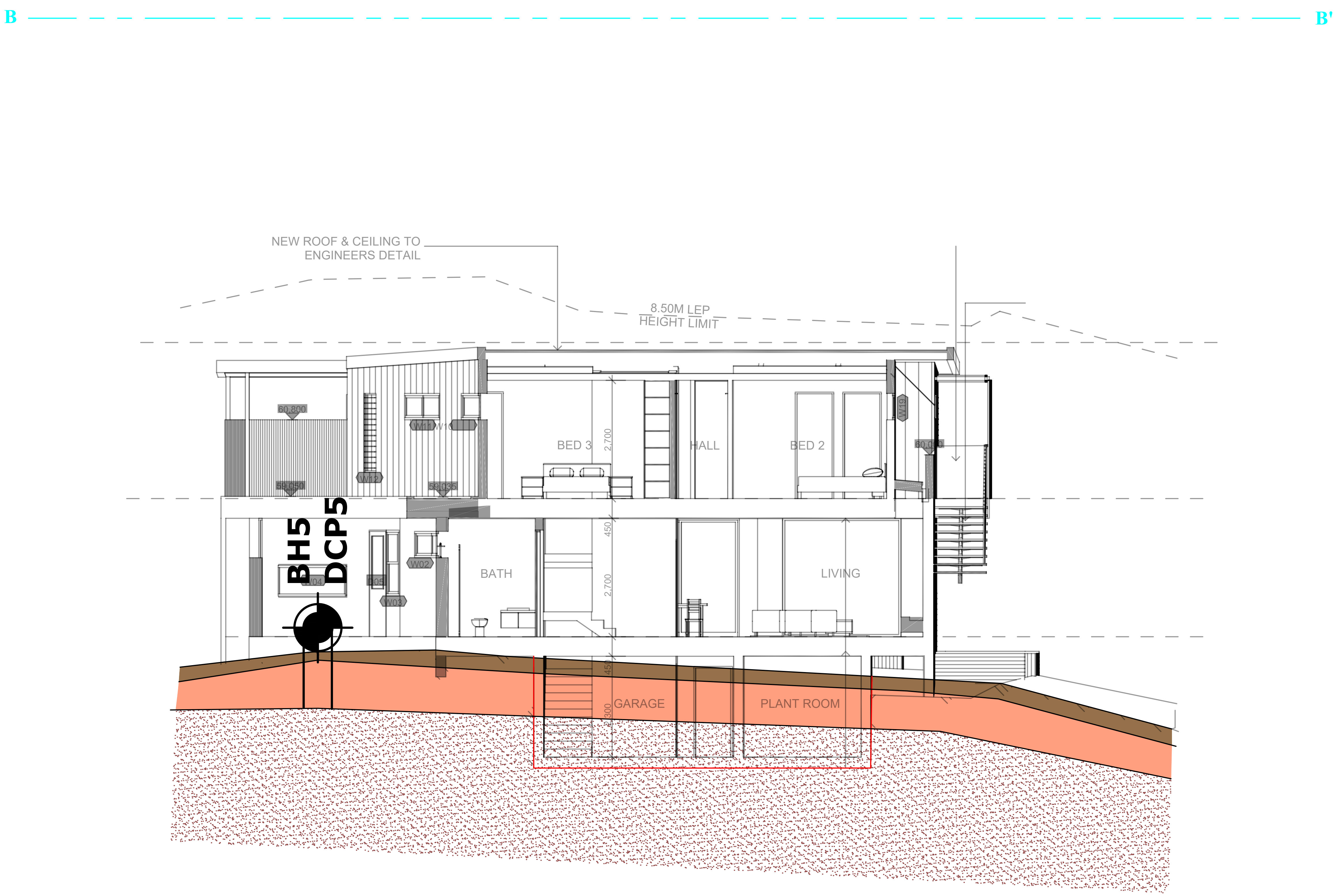
LEGEND

BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
DCP	DYNAMIC CONE PENETROMETER

SCALE:	1:250 @ A3
DRAWING:	FIGURE 1
DATE:	10/2024
APPROVED BY:	TMC
DRAWN BY:	JD
PROJECT:	2022-039

PREPARED FOR:
Neil Bernard

ADDRESS:
18 Hillcrest Avenue, Mona Vale



SECTION B FIGURE 2.

CROZIER
GEOTECHNICAL CONSULTANTS

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

LEGEND

BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
	PROPOSED EXCAVATION
	RESIDUAL SILTY CLAY
	VERY LOW STRENGTH BEDROCK
A —— A'	SECTION LINE

SCALE:	1:100 @ A3
DRAWING:	FIGURE 2
DATE:	10/2024
APPROVED BY:	TMC
DRAWN BY:	JD
PROJECT:	2022-039

PREPARED FOR:
Neil Bernard

ADDRESS:
18 HILLCREST AVENUE, MONA VALE

BOREHOLE LOG

CLIENT: Jennifer Robins | **DATE:** 21/03/2022 | **BORE No.:** 1

PROJECT: New Dwelling | **PROJECT No.:** 2022-039 | **SHEET:** 1 of 1

LOCATION: 18 Hillcrest Ave, Mona Vale | **SURFACE LEVEL:** RL54.50m

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 Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
		Topsoil: Loose, dark brown, fine to medium grained clayey sand with roots and gravels				
0.40	CL/CI	Silty CLAY: Stiff, dark brown, medium to low plasticity, moist, silty clay with roots				
0.80		… yellow/brown				
1.40	CL	Sandy CLAY: Very stiff, yellow orange, fine to medium grained/medium to low plasticity, moist	D	1.40		
1.50	CI	CLAY: Hard, grey with yellow/red mottle, medium plasticity, with ironstone gravels	D	1.50		
1.70		… Friable (Extremely weathered sandstone)		1.60		
1.75		Auger refusal @ 1.75m on interpreted VLS sandstone				

RIG: N/A

METHOD: Hand Auger

GROUND WATER OBSERVATIONS: Not encountered

REMARKS:

DRILLER: PS

LOGGED: JD

CHECKED: KN

BOREHOLE LOG

CLIENT: Jennifer Robins
DATE: 21/03/2022
BORE No.: 2

PROJECT: New Dwelling
PROJECT No.: 2022-039
SHEET: 1 of 1

LOCATION: 18 Hillcrest Ave, Mona Vale
SURFACE LEVEL: RL52.87m

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	
0.00			Type	Tests	Type	Results
		Topsoil: Loose, dark brown, fine to medium grained clayey sand with roots and gravels				
0.20	CI/CL	CLAY: Stiff, yellow/brown, medium to low plasticity moist clay with sandstone cobbles and gravels (Colluvium)				
0.80		Auger refusal @ 0.80m on cobble within colluvium, DCP extended to 1.55m				

RIG: N/A
DRILLER: PS

METHOD: Hand Auger
LOGGED: JD

GROUND WATER OBSERVATIONS: Not encountered

REMARKS:
CHECKED: KN

BOREHOLE LOG

CLIENT: Jennifer Robins | **DATE:** 21/03/2022 | **BORE No.:** 3

PROJECT: New Dwelling | **PROJECT No.:** 2022-039 | **SHEET:** 1 of 1

LOCATION: 18 Hillcrest Ave, Mona Vale | **SURFACE LEVEL:** RL48.15m

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
0.20		Topsoil: Loose, dark brown, fine to medium grained clayey sand with roots and gravels				
0.70	CL/CI	CLAY: Stiff, yellow/brown, medium to low plasticity moist clay with sandstone cobbles and gravels (Colluvium)				
		Auger refusal @ 0.70m on interpreted cobble, DCP extended to 2.20m				

RIG: N/A

DRILLER: PS

METHOD: Hand Auger

LOGGED: JD

GROUND WATER OBSERVATIONS: Not encountered

REMARKS:

CHECKED: KN

BOREHOLE LOG

CLIENT: Jennifer Robins | **DATE:** 21/03/2022 | **BORE No.:** 4

PROJECT: New Dwelling | **PROJECT No.:** 2022-039 | **SHEET:** 1 of 1

LOCATION: 18 Hillcrest Ave, Mona Vale | **SURFACE LEVEL:** RL44.40m

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 Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
		Topsoil: Loose, dark brown, fine to medium grained clayey sand with roots and gravels				
0.20	CI/CL	CLAY: Stiff, yellow/brown, medium to low plasticity moist clay with sandstone cobbles and siltstone gravels (Colluvium)				
0.60		Auger refusal @ 0.60m on interpreted cobble, DCP extended to 2.05m				

RIG: N/A

DRILLER: PS

METHOD: Hand Auger

LOGGED: JD

GROUND WATER OBSERVATIONS: Not encountered

REMARKS:

CHECKED: KN

BOREHOLE LOG

CLIENT: Jennifer Robins **DATE:** 21/03/2022 **BORE No.:** 5

PROJECT: New Dwelling **PROJECT No.:** 2022-039 **SHEET:** 1 of 1

LOCATION: 18 Hillcrest Ave, Mona Vale **SURFACE LEVEL:** RL55.45m

Depth (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00		Topsoil: Loose, dark brown, fine to medium grained clayey sand with roots and gravels				
0.20	CL/CI	silty CLAY: Stiff, dark brown, medium to low plasticity, moist, silty clay with roots				
0.65		… yellow mottle				
0.80		… yellow/brown				
0.90		… ironstone gravels				
1.00		pale grey with red and yellow mottle				
1.10		Auger refusal @ 1.10m on ironstone gravels, DCP extended to 1.28m				

RIG: N/A DRILLER: PS

METHOD: Hand Auger LOGGED: JD

GROUND WATER OBSERVATIONS: Not encountered

REMARKS: CHECKED: KN

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Jennifer Robins | **DATE:** 21/03/2022
PROJECT: New Dwelling | **PROJECT No.:** 2022-039
LOCATION: 18 Hillcrest Avenue, Mona Vale | **SHEET:** 1 of 1

	Test Location									
Depth (m)	1	2	3	4	5	6				
0.00 - 0.10	2	1	1	1	0	0				
0.10 - 0.20	1	1	1	3	1	2				
0.20 - 0.30	1	3	1	2	3	2				
0.30 - 0.40	1	3	2	4	3	2				
0.40 - 0.50	3	3	3	6	3	4				
0.50 - 0.60	2	3	2	5	3	4				
0.60 - 0.70	3	4	1	5	3	5				
0.70 - 0.80	2	3	3	5	4	5				
0.80 - 0.90	3	2	5	5	4	6				
0.90 - 1.00	3	2	2	3	4	5				
1.00 - 1.10	4	5	3	4	14	5				
1.10 - 1.20	3	5	3	4	22	8				
1.20 - 1.30	2	6	4	4	18	B@1.20				
1.30 - 1.40	3	5	6	4	B@1.28					
1.40 - 1.50	7	6	4	3						
1.50 - 1.60	11	B@1.55	5	3						
1.60 - 1.70	11		4	4						
1.70 - 1.80	14		4	5						
1.80 - 1.90	B@1.80		5	7						
1.90 - 2.00			5	9						
2.00 - 2.10			12	B@2.05						
2.10 - 2.20			20							
2.20 - 2.30			24							
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
AS 1289. F3.3, PERTH SAND PENETROMETER

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

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 3

<u>**TABLE : A**</u>

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 <$3m^3$) from new retaining wall excavation		Appears majority of the excavation will be through soil up to approximately 1.5m depth near shared boundary	a) Rear garden of No.20 Hillcrest Avenue 1.0m from proposed 2.0m deep excavation, impact 5% b) Rear garden of proposed new dwelling, impact 5%		a) Person on pathway of No.20 0.5 hr/day average b) Person in garden 0.25hr/day average	a) Likely to not evacuate b) Likely to not evacuate	a) Person in open space, buried b) Person in open space, buried	
			Possible	**Prob. of Impact**	**Impacted**				
		a) Side pathway of No.20 Hillcrest Avenue	0.001	0.90	0.05	0.0208	0.75	1.0	**7.03E-07**
		b) Access to proposed site dwelling	0.001	0.90	0.05	0.0104	0.75	1.0	**3.52E-07**
B	Landslip (rock slide/topple <$2m^3$) from new retaining wall excavation		Appears majortiy of the excavation may encounter bedrco near the base.	a) Rear garden of No.20 Hillcrest Avenue 1.0m from proposed 2.0m deep excavation, impact 5% b) Rear garden of proposed new dwelling, impact 5%		a) Person in rear garden of No.20 0.5 hr/day average b) Person in garden 0.25hr/day average	a) Likely to not evacuate b) Likely to not evacuate	a) Person in open space, buried b) Person in open space, buried	
			Unlikely	**Prob. of Impact**	**Impacted**				
		a) Side Pathway of No.20 Hillcrest Avenue	0.0001	1.00	0.03	0.0208	0.75	1.0	**4.69E-08**
		b) Access to proposed site dwelling	0.0001	0.90	0.03	0.0104	0.75	1.0	**2.11E-08**
C	Landslip of existing near surface soils similar to that seen in previous inspection		Previous evidence of minor landslip of surficial soils.	Landslip confined to cliff crest dwelling unlikely impacted	May impact 5% of decking footings	Person on deck 2hr/day average	Likely to not evacuate	Deck only damged	
			Possible	**Prob. of Impact**	**Impacted**				
		Proposed new dwelling	0.001	0.10	0.03	0.0833	0.75	0.2	**3.75E-08**

* 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 r 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).

* for excavation induced landslip 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%)

* 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 new retaining wall excavation	a) Side pathway of No.20 Hillcrest Avenue	Possible	The event could occur under adverse conditions over the design life.	Insignificant	Little Damageor no impact to neighbouring properties, no significant stabilising required .	Very Low
		b) Access to proposed site dwelling	Possible	The event could occur under adverse conditions over the design life.	Insignificant	Little Damageor no impact to neighbouring properties, no significant stabilising required .	Very Low
B	Landslip (rock slide/topple <2m^3) from new retaining wall excavation	a) Side pathway of No.20 Hillcrest Avenue	Possible	The event could occur under adverse conditions over the design life.	Insignificant	Little Damageor no impact to neighbouring properties, no significant stabilising required .	Very Low
		b) Access to proposed site dwelling	Possible	The event could occur under adverse conditions over the design life.	Insignificant	Little Damageor no impact to neighbouring properties, no significant stabilising required .	Very Low
C	Landslip of existing near surface soils similar to that seen in previous inspection	Proposed new dwelling	Possible	The event could occur under adverse conditions over the design life.	Insignificant	Little Damageor no impact to neighbouring properties, no significant stabilising required .	Very Low

* hazards considered in current condition, without remedial/stabilisation measures and during construction works.
* qualitative expression of likelihood incorporates both frequency analysis estimate and spatial impact probability estimate as per AGS guidelines.
* qualitative measures of consequences to property assessed per Appendix C in AGS Guidelines for Landslide Risk Management.
* Indicative cost of damage expressed as cost of site development with respect to consequence values: Catastrophic : 200%, Major: 60%, Medium: 20%, Minor: 5%, Insignificant: 0.5%.
* Cost of site development estimated at $5,000,000

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.	Every year or following each major rainfall event.
	Owner to check and flush retaining wall drainage pipes/systems	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.	Every two years or following major rainfall event.
	Replace non engineered rock/timber walls prior to collapse	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}	$5x10^{-3}$	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	$5x10^{-4}$	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	$5x10^{-5}$	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	$5x10^{-6}$	100,000 years	200,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}		1,000,000 years		The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

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

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

RISK LEVEL IMPLICATIONS

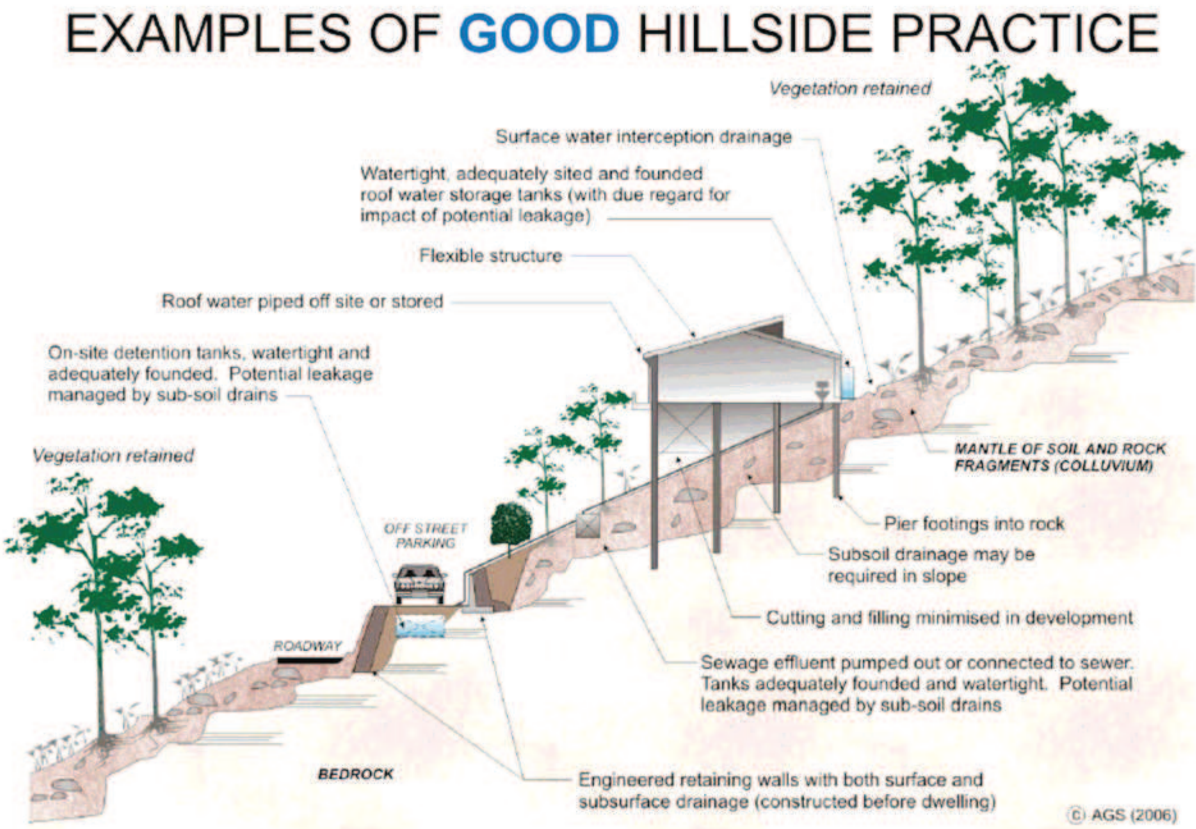
Risk Level		Example Implications (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.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

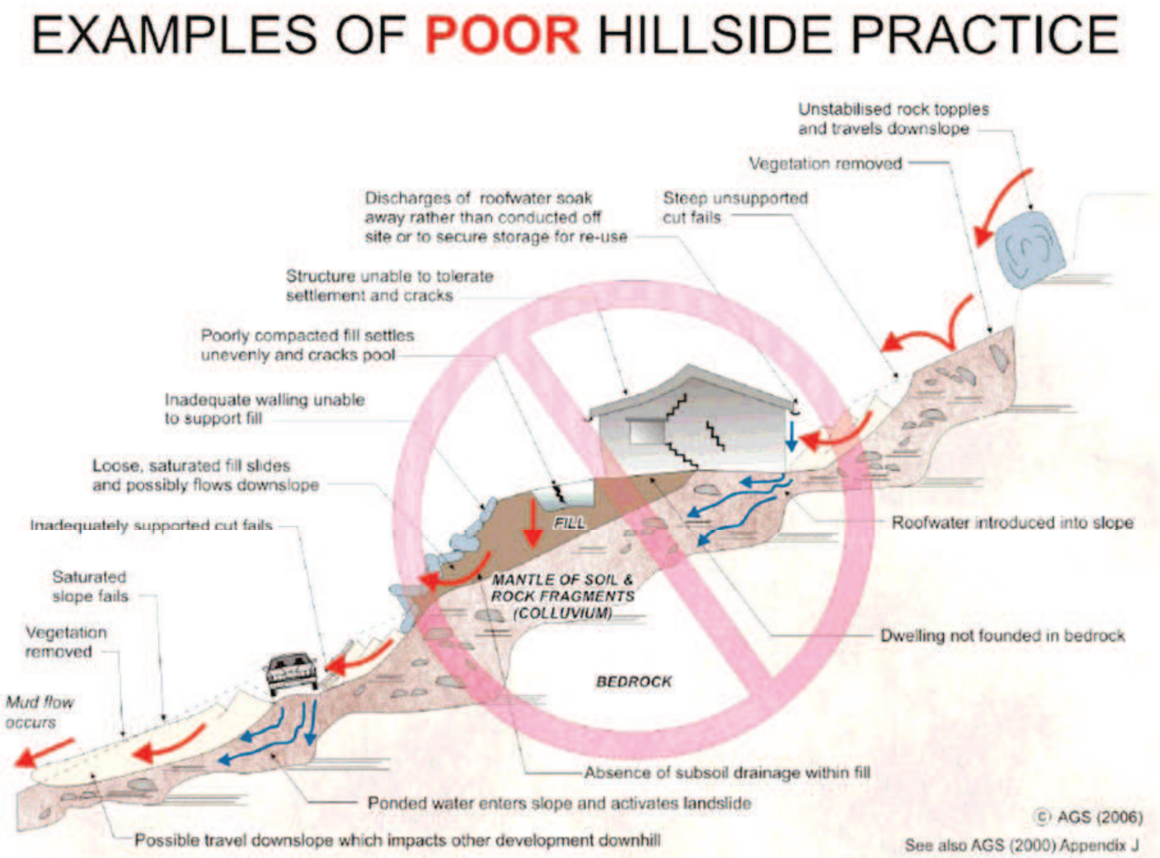
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

Appendix 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
Vegetation retained
Surface water interception drainage
Watertight, adequately sited and founded roof water storage tanks (with due regard for impact of potential leakage)
Flexible structure
Roof water piped off site or stored
On-site detention tanks, watertight and adequately founded. Potential leakage managed by sub-soil drains
Vegetation retained
MANTLE OF SOIL AND ROCK FRAGMENTS (COLLUVIUM)
OFF STREET PARKING
Pier footings into rock
Subsoil drainage may be required in slope
Cutting and filling minimised in development
ROADWAY
Sewage effluent pumped out or connected to sewer. Tanks adequately founded and watertight. Potential leakage managed by sub-soil drains
BEDROCK
Engineered retaining walls with both surface and subsurface drainage (constructed before dwelling)
© AGS (2006)


EXAMPLES OF POOR HILLSIDE PRACTICE
Unstabilised rock topples and travels downslope
Vegetation removed
Discharges of roofwater soak away rather than conducted off site or to secure storage for re-use
Steep unsupported cut fails
Structure unable to tolerate settlement and cracks
Poorly compacted fill settles unevenly and cracks pool
Inadequate walling unable to support fill
Loose, saturated fill slides and possibly flows downslope
Inadequately supported cut fails
FILL
Roofwater introduced into slope
Saturated slope fails
MANTLE OF SOIL & ROCK FRAGMENTS (COLLUVIUM)
Vegetation removed
Dwelling not founded in bedrock
Mud flow occurs
BEDROCK
Absence of subsoil drainage within fill
Ponded water enters slope and activates landslide
Possible travel downslope which impacts other development downhill
© AGS (2006)
See also AGS (2000) Appendix J

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Appendix 6

Horton Coastal Engineering
Coastal & Water Consulting

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Neil Burnard
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(sent by email only to neil.burnard1@outlook.com)

13 November 2024

Coastal Engineering Advice on 18 Hillcrest Avenue Mona Vale

1. INTRODUCTION AND BACKGROUND

It is proposed to demolish and rebuild a dwelling at 18 Hillcrest Avenue Mona Vale (the 'site'), for which a Development Application is to be submitted to Northern Beaches Council. 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 (including its cliff face and adjacent rock platform) on 26 January and 2 February 2023, and 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.

2. INFORMATION PROVIDED

Horton Coastal Engineering was provided with 24 drawings of the proposed works prepared by Progressive Plans (Drawings DA00 to 23), all dated 9 September 2024 and Issue A. A site

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

survey by Mepstead & Associates was also provided, reference 5810, Revision F, and dated 9 July 2024.

3. EXISTING SITE DESCRIPTION

The site is located at the northern end of Mona Vale Headland, extending down to a rock platform and cliff at the southern end of Bungan Beach. A vertical aerial view of the site is provided in Figure 1, with a section through the site (denoted as Section A) approximately perpendicular to the top of the cliff also depicted in Figure 1[2]. An oblique aerial view of the site is in Figure 2, and a view of the site from the rock platform at the base of the cliff is in Figure 3.

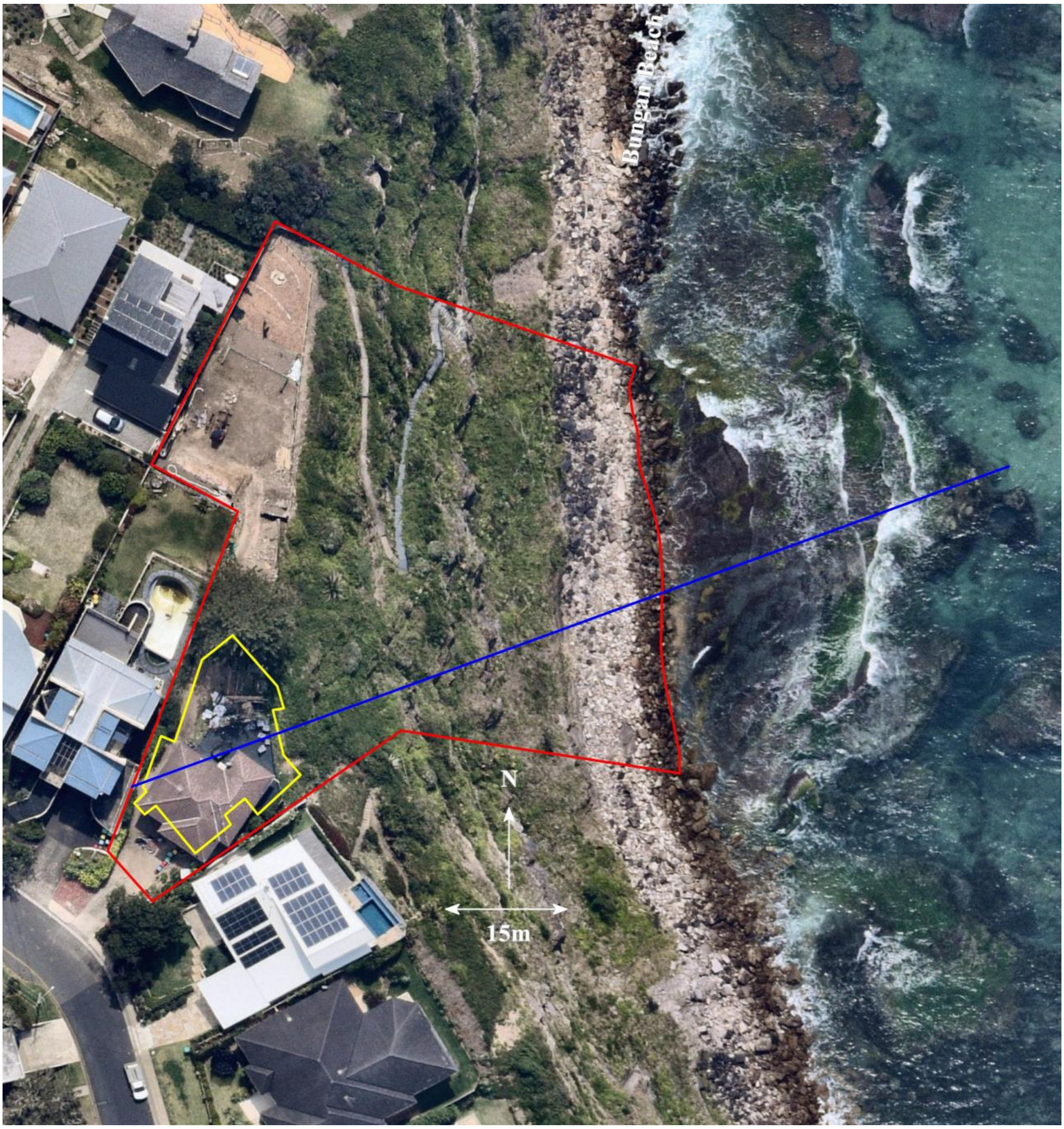

Figure 1: Aerial view of site (red outline), with Section A in blue and outline of proposed development in yellow (aerial photograph taken 22 September 2024)

[2] Note that the site boundary depicted in Figure 1 is only approximate.

Figure 2: Oblique aerial view of site (at arrow) on 22 July 2024, facing west

Figure 3: View of cliff face at site (extent at top of cliff approximately between arrows) on 3 October 2024, 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 2011 (along the rock platform) and 2020 (for the remainder of the site), elevations versus distance along Section A (from Figure 1) are depicted in Figure 4.

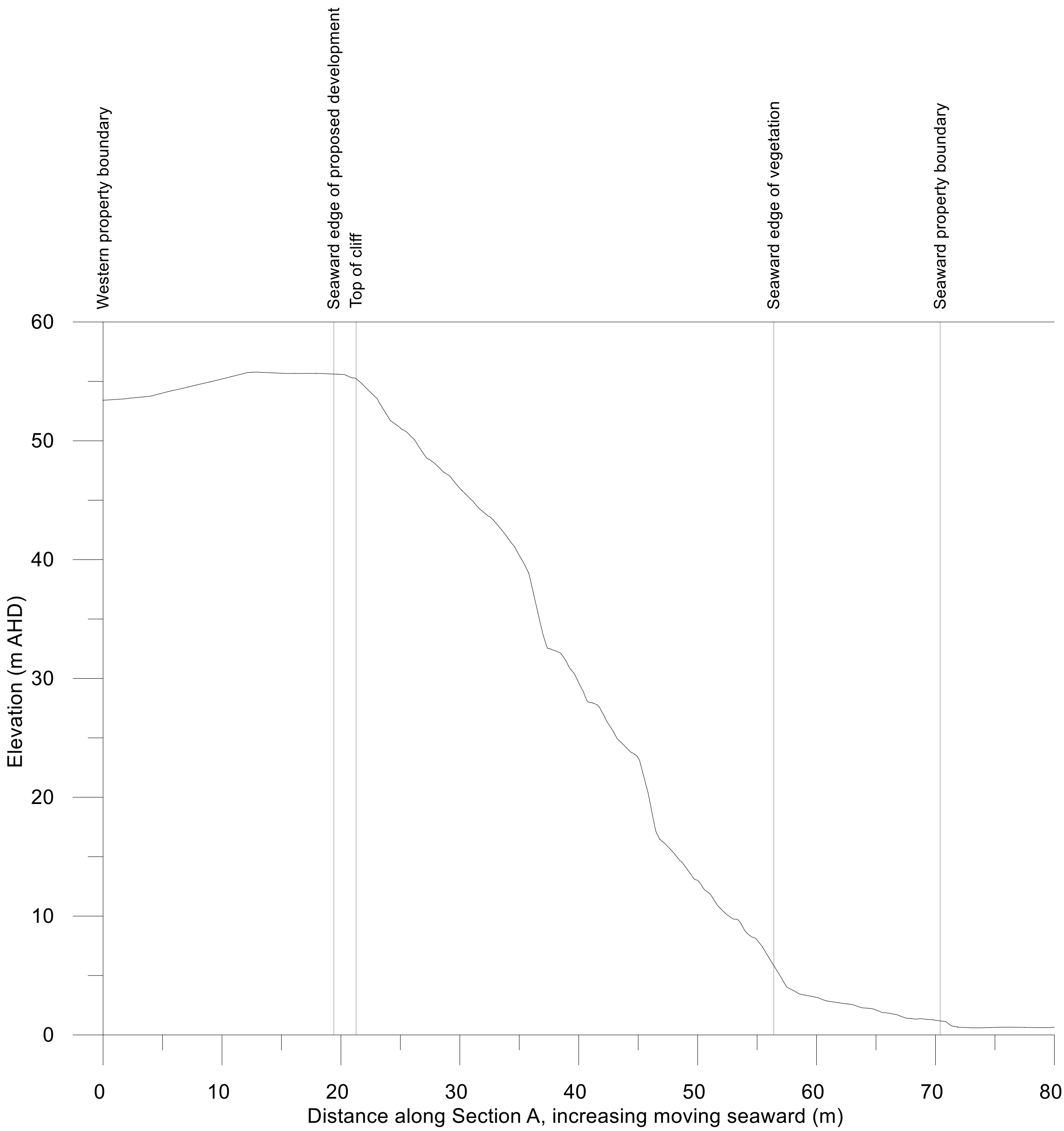

Figure 4: Section A through site, including cliff face and rock platform

Ground elevations along Section A approximately vary from about 55.5m AHD over most of the development area, 55.2m AHD at the top of the cliff, 3.5m AHD at the toe of the cliff, and 1.2m AHD at the seaward property boundary. The average slope from the top to the toe of the cliff is 1:0.7 (vertical:horizontal, V:H) or 54°, with steeper sections around the central portion of the cliff. There is no evidence of any recent significant slope failures having occurred in the vicinity of the site.

4. PROPOSED DEVELOPMENT

It is proposed to demolish the existing dwelling and to rebuild a new dwelling over three levels at the site, with the finished floor level of the ground floor at 55.85m AHD, and garage and lower ground floor at 53.10m AHD. The position of the proposed dwelling (including ancillary structures such as a terrace, pool, spa and access steps) is outlined on Figure 1.

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) at and seaward of the site over the last 6,400 years (since sea levels stabilised around their present levels, and assuming that the cliff 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 cliff (vegetated portion above the rock boulders) at the site is above the intertidal zone (above 1m AHD) but would be impacted by wave runup at times, particularly during coastal storms with large waves and elevated water levels. This wave runup could extend up to levels of about 8m AHD at present in a 100 year Average Recurrence Interval (ARI) storm, increasing to around 9m AHD in 100 years if projected sea level rise is realised.

Given this, it should be assumed that both chemical and wave-induced mechanical weathering would apply at this site. 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.

To be conservative, the rates can be applied over the entire exposed cliff face, although in reality it would be expected that runup would generally be below 9m AHD in a severe coastal storm over the 100 year design life (that stated, although wave-induced mechanical weathering would be limited to the lower portion of the cliff face, the upper cliff face is exposed to mechanical weathering through wind action). 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.

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[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 6 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.

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 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 within a "Coastal Environment" area (see Section 7.2.2) and "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 (it is understood) conventional stormwater management features such as a rainwater tank and a dispersion system over the cliff face. 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. An Arboricultural Impact Assessment has been completed for the site by Treeism Arboricultural Services. They found that the

[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*.

Norfolk Island Pine tree at the site had viable retention and viability with the proposed development, with various recommendations given to achieve that.

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.

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 at and 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 18 October 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 150m 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

Clause 7.8 of LEP 2014 is not applicable to the proposed development, as the works are entirely landward of the Foreshore Building Line (that is, the works are not in the Foreshore Area) 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 at 18 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 Neil Burnard
Name of Applicant

Address of site 18 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 November 2024 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 18 Hillcrest Avenue Mona Vale

Report Date: 13 November 2024
:

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

Signature ……………………………………… 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