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REPORT ON GEOTECHNICAL ASSESSMENT

for

PROPOSED ALTERATIONS AND ADDITIONS

at

325 WHALE BEACH ROAD, WHALE BEACH

Prepared For

Jorge Hrdina Architects

Project No.: 2025-055

Document Revision Record

Issue No	Date	Details of Revisions
0	15 April 2025	Original issue

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1 – To be submitted with Development Application

Development Application for ______________________	
Name of Applicant	
Address of site ___ 325 Whale Beach Road, Whale Beach	

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

I, __***Troy Crozier***_ *on behalf of* ***Crozier Geotechnical Consultants*** __on this the __16 April 2025 certify that I am a engineering geologist as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above company to issue this document and to certify that the company has a current professional indemnity policy of at least $2million.

I:

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

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

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

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

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

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

Geotechnical Report Details:

Report Title: Geotechnical Report for Proposed Alterations and Additions	
Report Date: 15 April 2025	**Project No.:** 2025-055
Author: T. Crozier	
Author's Company/Organisation: Crozier Geotechnical Consultants	

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

Architectural: Jorge Hrdina Architects, Project No.: 2051; Drawing No./Revision: 1003/A, 1004/B, 1005/-, 1006/B, 1007/A, 2001/B, 2002/B, 2003/B, 2004/B, 3001/B, 3002/B, 3003/C; Dated: 15/04/2025
Survey: Survey Plan by CMS Surveyors, Reference.: 18080B, Issue: 4, Dated: 21.02.2025.
Coastal: UNSW Water Research Laboratory, Reference: WRL2025023 LR20250404 JTC WMM, Dated: 04 April 2025

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

Signature ....

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

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

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

Company... **Crozier Geotechnical Consultants**

AUSTRALIAN INSTITUTE OF GEOSCIENTISTS
TROY CROZIER
10197

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

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

Development Application for ____
Name of Applicant

Address of site ____325 Whale Beach Road, Whale Beach________________

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

Geotechnical Report Details:

Report Title: Geotechnical Report for Proposed Alterations and Additions

Report Date: 15 April 2025 **Project No.:** 2025-055

Author: T. Crozier

Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

- ■ Comprehensive site mapping conducted ____02 April 2025_______ (date)
- ☐ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
- ☐ Subsurface investigation required
  - ■ No Justification ...minor temporary excavations only.........
  - ☐ Yes Date conducted ....
- ☐ Geotechnical model developed and reported as an inferred subsurface type-section
- ■ Geotechnical hazards identified
  - ☐ Above the site
  - ■ On the site
  - ☐ Below the site
  - ☐ Beside the site
- ■ Geotechnical hazards described and reported
- ■ Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
  - ■ Consequence analysis
  - ☐ Frequency analysis
- ■ Risk calculation
- ■ Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved.
- ■ Design Life Adopted:
  - ■ 100 years
  - ☐ Other .... specify
- ■ Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
- ■ Additional action to remove risk where reasonable and practical have been identified and included in the report.
- ☐ Risk assessment within Bushfire Asset Protection Zone.

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

Signature ....

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

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

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

Company... **Crozier Geotechnical Consultants**

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APPENDICES

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GEOTECHNICAL REPORT FOR PROPOSED ALTERATIONS AND ADDITIONS
325 WHALE BEACH ROAD, WHALE BEACH, NSW

1. INTRODUCTION:

This report details the results of a geotechnical assessment carried out for proposed alterations and additions at 325 Whale Beach Road, Whale Beach, NSW. The assessment was undertaken by Crozier Geotechnical Consultants (CGC) at the request of Jorge Hrdina Architects.

It is understood that the proposed works involve alterations and additions to the existing house including minor extension to the south, extensive re-modelling of internal walls along with demolition of the existing pool and construction of a new lap pool extending across the property adjacent the rear edge of the house. The house extension will require minor excavation (<1.0m) depth whilst the pool may require up to 1.50m depth of excavation. A series of screens along the rear western side of the house will require an extension of the pool excavation to the west, increasing its depth up to approximately 2.50m.

Reference to Pittwater Council's LEP 2014 Geotechnical Risk Management Policy (Hazard Map Sheet GTH_015), the site is located within the H1 (highest category) landslip hazard zone therefore the site requires a Geotechnical Landslip Risk Assessment to be conducted in support of a Development Application.

The site is also defined as being in a Coastal Risk Planning (Map Sheet CHZ_015) as being subject to Bluff/Cliff Instability.

This report therefore includes a detailed description of the site conditions, assessment of proposed works, site specific risk assessment incorporating the results of an assessment by a Coastal Engineer and recommendations for construction and maintenance to maintain the 'Acceptable Risk Management' criteria. The investigation and reporting were undertaken as per the Proposal No.: P25-111.1, Dated: 20 March 2025.

The investigation comprised:

a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Geotechnical Engineer.
b) Review of Ortho Photomaps and Aerial Photography of the site.

The following plans and diagrams were supplied for the work:

- Architectural drawing by Jorge Hrdina Architects, Project No.: 2051; Drawing No./Revision: 1003/A, 1004/B, 1005/-, 1006/B, 1007/A, 2001/B, 2002/B, 2003/B, 2004/B, 3001/B, 3002/B, 3003/C; Dated: 15/04/2025.
- Survey Plan by CMS Surveyors, Reference.: 18080B, Issue: 4, Dated: 21.02.2025.
- Coastal Hazard Assessment by UNSW Water Research Laboratory, Reference: WRL2025023 LR20250404 JTC WMM, Dated: 04 April 2025

2. SITE FEATURES:

2.1. Description:

The site (Lot 241 and Lot 242, DP16362) is a trapezoidal shaped block located on the low eastern side of Whale Beach Road, within gently to moderately east dipping topography. It is situated towards the base of the slope, on the northern side of a south-east plunging in ridge line with a sea cliff line of up to 20m height at the rear boundary of the property. The site has a front west boundary of approximately 24.0m length, irregular rear east boundary of 37.0m, side north boundary of 52.84m and side east boundary of 67.93m as referenced from the provided survey plan.

An aerial photograph of the site and its surrounds is provided below, as sourced from NSW Government Six Map spatial data, as Photograph 1.

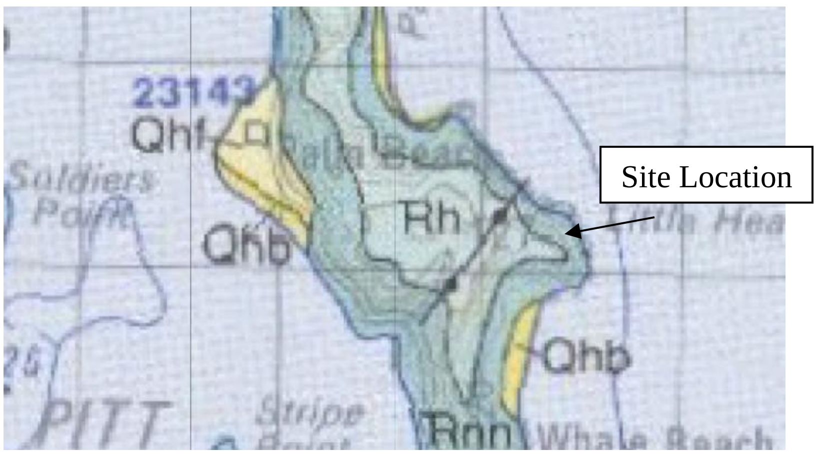
Photograph: 1 – Aerial photo of site and surrounds

The site is currently occupied by a single storey masonry and timber house situated in the centre of the block with driveway and garden to the front western side with lawns and gardens to the rear and a concrete swimming pool in the south-east corner.

2.2. Geology:

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

Narrabeen Group rocks are dominated by shales and thin siltstone 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 shale 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 silty clay soil profiles (medium to high plasticity) with thin silty colluvial cover.



Extract of Sydney Geological Series Sheet

3. FIELD WORK:

3.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and limited inspection of adjacent properties on the 02 April 2025 by a Principal / Engineering Geologist. It included a photographic record of the site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of ground levels and existing structures.

3.2. Field Observations:

Whale Beach Road comprises a very gently north dipping bitumen pavement with no kerb or guttering, though a slight cross-fall to the west exists. The reserve to the east side contains moderately (>10 degree) east dipping vegetated slopes that extend down into the site with no defined boundary fence at the front of the site. There were no indications of underlying geotechnical instability or hazard within the road pavement, which appears at least 5 years of age, or the sloping gardens within the road reserve.

The site is accessed via a gently sloping concrete/paved driveway which extends across slope from the north-west corner of the site to the south-west corner of the house. The front of the property then contains gently east dipping lawns and gardens with low garden/retaining walls adjacent the driveway.

Photograph: 2 – showing road reserve looking north.

Photograph: 3 – showing front yard looking towards north-west

The existing house is a single storey brick and timber structure that extends to within 1.0m of the northern boundary and approximately 4.50m from the southern boundary. Inspection of the sub-floor cavity from an access panel indicates the structure is supported off brick walls to strip footings. Across the entire rear edge of the house is a timber deck that is raised above the rear of the property due to the ground surface slope with a rendered masonry wall supporting the eastern edge of the deck above a narrow garden bed that is itself supported by another rendered masonry retaining wall of up to 1.0m in height.

Photograph: 4 – showing northern side of existing house, looking west.

Photograph: 5 – showing subfloor brick footing walls.

To the rear of the house deck/garden is an undulating lawn that comprises an upper gently sloping narrow terrace with moderate grass covered slopes dipping to the east and north-east. The rear edge of the property contains dense vegetated moderate slopes that dip to the crest of the sea cliffs whilst a recently constructed (<10 year) concrete part inground swimming pool is located in the south-east corner of the property, extending to within proximity of the sea cliffs.

Photograph: 6 – showing rear of property and pool in south-east corner.

Photograph: 7 – showing rear edge of house, with deck and retained garden above lawn slope

The neighbouring land to the south is a thin strip of Council reserve which is at similar levels and slopes to the site and contains limited vegetation and no indications of slope instability.

Further south of this reserve is No. 319 Whale Beach Road which contains a one and two storey residential house on the southern side of the block at the mid-point with a garage in the north-west corner of the property and then gardens and lawns down the northern side. Previous inspection of this property by CGC did not identify any signs of instability.

The neighbouring property to the north (No. 327) contains a one and two storey brick and timber house situated on the centre of the block, with the lower floor level below the rear of the structure due to the natural ground surface slope. To the front of the house are a concrete driveway and gently sloping lawns and gardens rising up to a moderately sloping road reserve covered with gardens. To the rear of the house are moderate to steep lawns and gardens with a steep east plunging drainage channel passing down through the rear of the property, near the north boundary, to the crest of the sea cliffs. There were no signs of instability or movement in the existing house and no signs of recent or significant erosion or slope instability within the rear drainage gully.

Photograph: 8 – showing drainage gully within rear of neighbouring property to north.

Photograph: 9 - showing dense vegetation at rear edge of site adjacent crest of sea cliffs

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

4. COMMENTS:

4.1. Geotechnical Assessment:

The inspection and assessment identified no obvious credible landslip hazards within the site or adjacent properties. The existing residence is of masonry construction and at least 25 years of age and appears to be in good condition, with no signs of cracking or settlement that would indicate foundation movement/deep seated instability. The soil slopes within and around the site have no signs of any previous or impending instability. All visible retaining walls within the site appear generally stable at present. No obvious surface stormwater flow or excess seepage/wet areas were identified.

The coastal engineering assessment does not indicate any cliff regression conditions that may impact the site or proposed structures over the design life and a review of the photography provided within that report, along with previous CGC inspections, did not identify any significant overhangs or causes for future cliff line collapse (rock fall, topple) or larger scale instability.

The proposed works involve alterations and additions to the existing house which will involve limited earthworks. The existing pool will be demolished whilst a new lap pool will be formed across the rear eastern edge of the existing house along with wind screens, this new pool is anticipated to require up to 2.0m depth of excavation whilst the screens will require the excavation to increase to approximately 2.50m depth and extend closer to the existing house.

The pool/screens excavation will be deepest along the upslope western side and reduces to <0.50m to the north and east due to the natural ground surface levels. Similarly, the house excavation will reduce to nil towards the east.

These excavations are anticipated to extend through clayey fill soils, residual soils and potentially weathered bedrock, however hard rock excavation is not anticipated based on exposed conditions within the site and adjacent properties. The determination of actual foundation and excavation conditions for engineering design will require geotechnical investigation.

These proposed excavations will be temporary and as such should be treated as per the WorkSafe NSW codes, policies and procedures. These temporary excavations are located sufficiently away from property boundaries to allow implementation of temporary batter slopes with no credible stability hazards to neighbouring properties. However, there is a potential for undermining of existing house footings. This can be dealt with through underpinning and temporary support systems as determined necessary by the conditions exposed and those determined by any geotechnical investigation and the builder during the initial site works.

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

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and experience in nearby properties. Therefore, some minor variation to the interpreted sub-surface conditions is possible. However, the results of the investigation provide a reasonable basis for the Development Application analysis and subsequent preliminary design of the proposed works.

4.2. Site Specific Risk Assessment:

There were no signs of existing or previous landslip instability within the site or adjacent land whilst the existing house structure shows no signs of settlement or cracking. The proposed works require only temporary excavation for the pool/screens and a shallow temporary excavation for the house extension. The coastal assessment and proposed regression rates will not impact the proposed development over a design life of 100 years.

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

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

The proposed excavations will be temporary and as such should be treated as per the WorkSafe NSW codes, policies and procedures. However, the conditions should be assessed prior to the excavation works by geotechnical engineering investigation and inspection.

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

The Risk to Life from Hazard A was estimated to be up to **1.56×10^{-7},** whilst the Risk to Property was considered to be **'Low to Very Low'**. The hazard was therefore considered to be '**Acceptable**' when assessed against the criteria of the Councils Policy. However, it should be noted that this assessment considers the excavations permanently unsupported, therefore actual risk levels will be significantly lower as all excavations are temporary.

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

4.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

4.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Class 'S' for footings at base of excavation into residual clay soils (requires investigation for confirmation)
Type of Footing	Strip/Pad or Slab or Piles
Sub-grade material and Maximum Allowable Bearing Capacity for shallow footings	- Clay (Stiff): 100kPa - Weathered, ELS-VLS Bedrock: 700kPa
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	Be – rock site
Remarks: All footings should be founded off material of similar foundation conditions to prevent differential settlement, allowance for differential settlement should be designed for if the structure is variably founded. All new footings must be inspected by an experienced geotechnical professional before concrete or steel	

are placed to verify their bearing capacity and the in-situ nature of the founding strata. This is mandatory to allow them to be 'certified' at the end of the project.

4.3.2. Excavation:

Depth of Excavation	Maximum of 2.50m depth for screens/pool and 1.0m depth for the house extension.

Boundary / Structure Separation:

Direction	**Adjacent area / Structure**	**Bulk Excavation Depth**	**Separation Distances**	
			Boundary	**Structure**
West	Road Reserve Existing House	Up to 1.0m for house extension Up to 2.5m for screens / pool	>15.0m NA	Pavement + 3.0m Site House within 2.0m of excavation
South	Council Reserve	Up to 1.0m for house, Up to 2.5m for screens / pool	>0.75m >5.0m	None None
East	Cliff line	Pool reduces to <0.50m	>5.00m	None
North	Neighbouring property	Up to 2.5m for screens, Pool reduces to <1.00m	>3.00m >0.90m	House +2.0m

Type of Material to be Excavated*	Fill/topsoil
	Residual soils to extremely to highly weathered bedrock

Guidelines for <u>unsurcharged</u> batter slopes are tabulated below:

Material	Safe Batter Slope (H:V)	
	Short Term/Temporary	Long Term/Permanent
Fill/topsoil	1.5:1	2:1
Residual soils	1:1	1.5:1
VLS – LS bedrock (fractured)	Vertical*	0.25:1*

Remarks:

***Requires geotechnical investigation to confirm.**

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

Equipment for Excavation	Fill /ELS	Excavator with bucket
	VLS bedrock	Excavator with bucket and ripper
	LS-MS/HS bedrock	Rock hammer and rock saw

ELS – extremely low strength, VLS – very low strength, LS – low strength, MS – medium strength	
Remarks: Proposed pool, screens and house extension excavations are expected to be temporary and to require no hard rock excavation, therefore long term stability and ground vibrations during excavation are not considered hazards. Inspection of equipment and excavation locations by a geotechnical engineer is required during initial works to determine the need for any further controls or monitoring.	
Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Existing site structures = 5mm/s Neighboring structures = 5mm/s
Vibration Calibration Tests Required	If large scale (i.e. rock hammer >250kg) excavation equipment is proposed
Full time vibration Monitoring Required	Not required
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • For assessment of excavation perimeter and batter slopes/support system requirements • Following footing excavations to confirm founding material strength
Dilapidation Surveys Requirement	Not a critical aspect based on site works. May be considered for neighbouring structures or parts thereof within 5.0m of the excavation perimeter prior to site work to protect the client against spurious claims of damage.
Remarks: Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope, whilst any groundwater seepage must be controlled within the excavation and prevented from ponding or saturating slopes/batters.	

4.3.3. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		No
Excavation likely to intersect	Water Table	No
	Seepage	Minor (<0.5L/min) possible
Site Location and Topography		Low east side of road, adjacent crest of sea cliffs.

Impact of development on local hydrogeology	Appears negligible
Onsite Stormwater Disposal	Could be implemented via dispersion across rear of property or piped to base of sea cliffs
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.	

4.4. Conditions Relating to Design and Construction Monitoring:

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

1. Review the structural design drawings, including the retaining structure design and construction methodology, for compliance with the recommendations of this report prior to construction,
2. Inspect the initial site works and proposed excavations for support requirements and stability prior to bulk excavation,
3. Inspect all new footings to confirm compliance with design assumptions with respect to allowable bearing pressure, basal cleanness and stability prior to the placement of steel or concrete.
4. Inspect the completed works to ensure all stabilising systems are completed

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

4.5. Design Life:

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

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

Man-made features should be designed and maintained for a design life consistent with surrounding structures (as per AS2870 – 2011 (50 years)). In order to attain an “Acceptable Risk Management Criteria” for a design life of 100 years as detailed by the Councils Risk Management Policy, it will be necessary for the property owner to adopt and implement a maintenance and inspection program. It is considered that the existing house will have a design life of 50 years from its upgrade following the proposed works.

If a maintenance and inspection schedule are not implemented the "Acceptable" risk levels for the design life of the property may not be attained.

A recommended program is given in Table: 1 below and should also include the following guidelines:

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

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

5. CONCLUSION:

The inspection and assessment identified no obvious significant slope movement, excess surface stormwater flow or seepage, erosion or instability within the site or adjacent properties.

The proposed works are relatively minor from a geotechnical perspective requiring only temporary excavation for the house extension and temporary excavations for the pool and wind screens. The works are located sufficiently from property boundaries to present a 'Very Low' to neighbouring properties or structures and the works can be managed through sensible excavation practices.

The ground conditions are expected from adjacent exposures and experience in the local area to comprise limited fill soils over residual clayey soils and then weathered siltstone/sandstone bedrock. Confirmation prior to final design will require geotechnical investigation. Geotechnical inspection is also required at initial site works and prior to bulk excavations to assess excavation perimeters and supports systems.

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

Prepared By:

Troy Crozier
Principal
MIE Aust, CPEng (NER – Geotechnical)
MAIG, RPGeo – Geotechnical and Engineering

6.0. REFERENCES:

1. Australian Geomechanics Society 2007, "Landslide Risk Assessment and Management", Australian Geomechanics Journal Vol 42, No 1, March 2007.
2. Geotechnical Risk Management Policy for Pittwater, 2009.

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 1

CROZIER
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NOTES RELATING TO THIS REPORT

Introduction

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

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

Description and classification Methods

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

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

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

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

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

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

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

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

Sampling

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

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

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

Drilling Methods

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

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

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

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

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

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

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

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

Standard Penetration Tests

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

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

The test results are reported in the following form.

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

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

Cone Penetrometer Testing and Interpretation

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

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

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

The information provided on the plotted results comprises: -

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

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

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

Qc (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: -

Qc = (12 to 18) Cu

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

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

Dynamic Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Borehole Logs

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

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

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

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

Ground Water

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

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

Engineering Reports

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

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

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

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

Site Anomalies

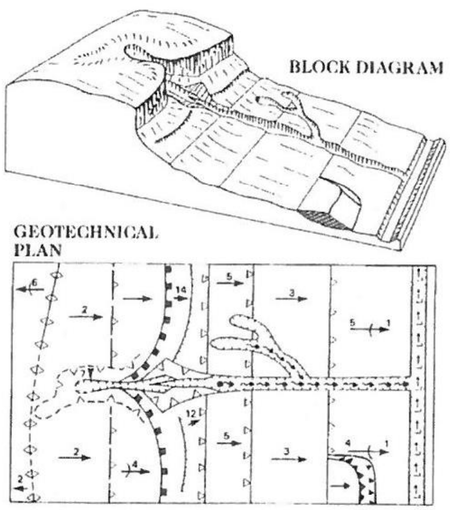
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

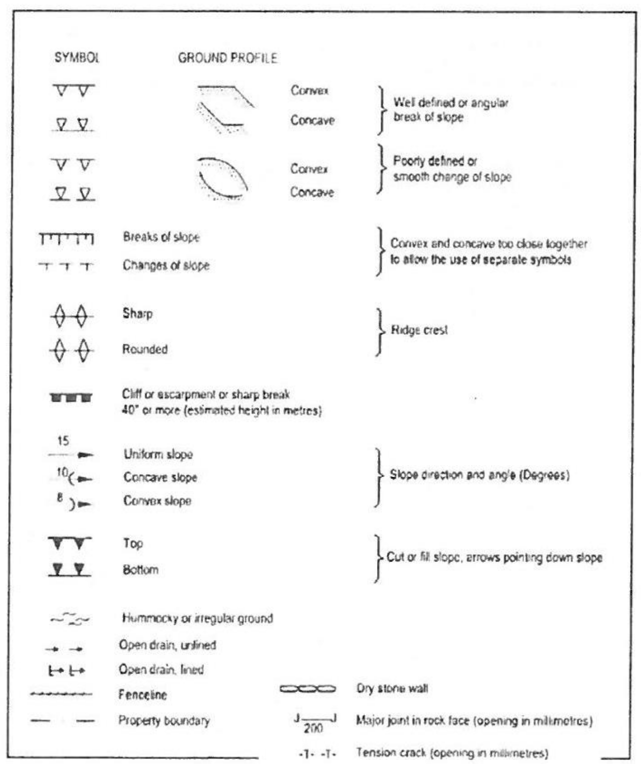
Reproduction of Information for Contractual Purposes

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

Site Inspection

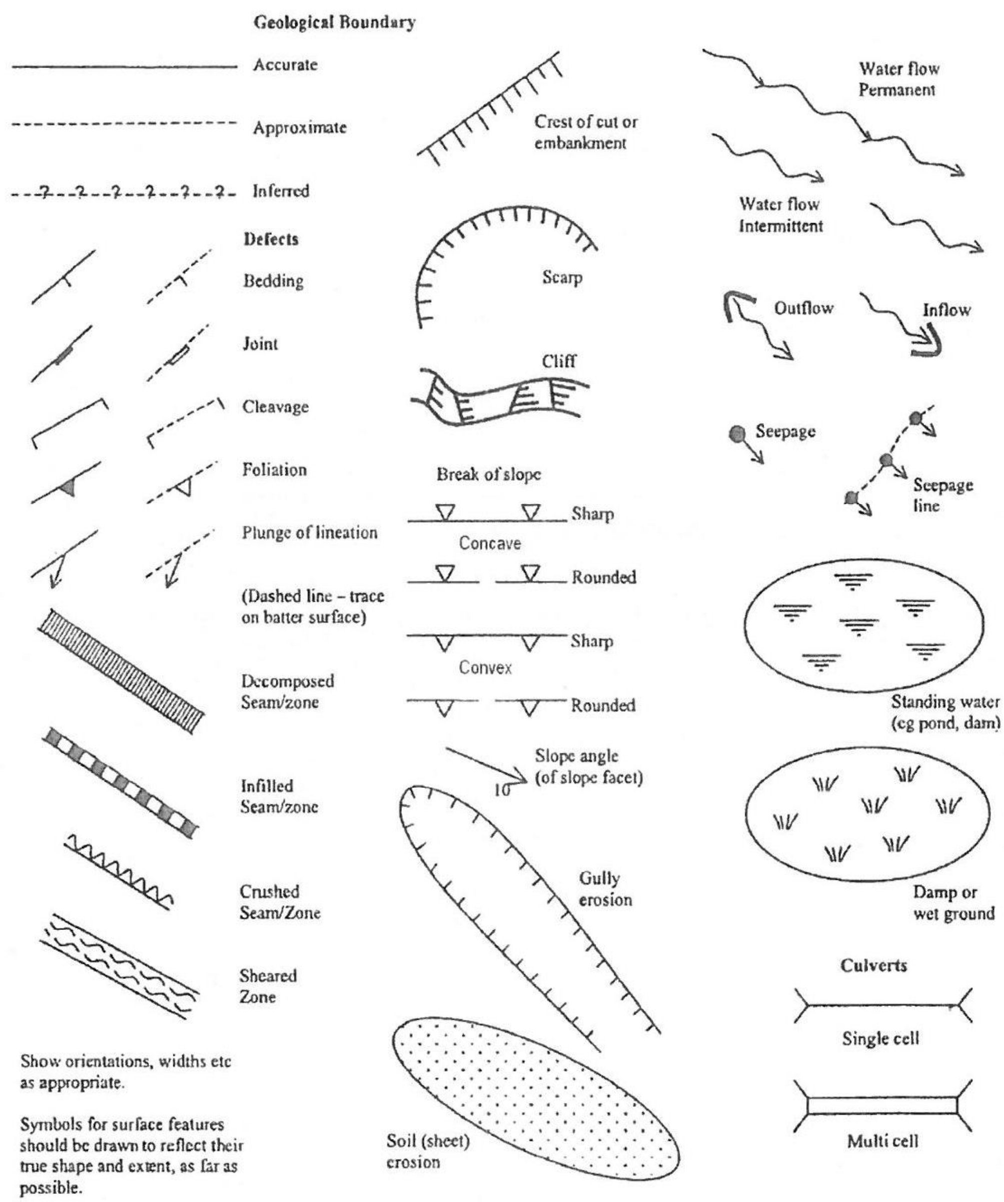
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





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

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



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

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 2

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (earth slide <3m³) from soils at crest of excavation		Shallow fill over residual soils and weathered bedrock anticipated	a) house >2.0m from excavation, impact <5% only b) Reserve >0.75m from <1.0m deep excavation, impact 1% c) house is >3.00m from end of excavation, <1.0m deep, impact 1% only d) lawn and garden adjacent to excavation on boundary , impact 1%		a) Person in house 20hrs/day avge. b) Person in reserve 1hr/month avge. c) Person in house 20hrs/day avge. d) Person in garden 0.25hr/day avge.	a) Likely to not evacuate b) Unlikely to not evacuate c) LIkely to not evacuate d) Possible to not evacuate	a) Person in building, minor damage only b) Person in open space, unlikely buried c) Person in building, minor damage only d) Person in open space, unlikely buried	
			Possible / Unlikley	**Prob. of Impact**	**Impacted**				
		a) Site House - No. 325 Whale Beach	0.001	0.10	0.05	0.8333	0.75	0.05	**1.56E-07**
		b) Council reserve to south	0.0001	0.10	0.01	0.0015	0.5	0.05	**3.72E-12**
		c) House No. 327 Whale Beach Road	0.0001	0.05	0.01	0.8333	0.75	0.05	**1.56E-09**
		d) Lawn at No. 327 Whale Beach Road	0.001	0.10	0.01	0.0104	0.5	0.05	**2.60E-10**

* hazards considered in current condition and/or without remedial/stabilisation measures
* likelihood of occurrence for design life of 100 years
* Spatial Impact - Probaility of Impact referes to slide impacting structure/area expressed as a % (1.00 = 100% probability of slide impacting area if it occurs), Imapcted refers to % of area/strucure impacted if slide occurred
* neighbouring houses considered for bedroom impact unless specified
* considered for person most at risk
* considered for adjacent premises/buildings founded via shallow footings unless indicated
* evacuation scale from Almost Certain to not evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knwoing of landslide and completely evacuating area prior to landslide impact.
* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (earth slide <3m³) from soils at crest of excavation	a) Site House - No. 325 Whale Beach	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low
		b) Council reserve to south	Unlikely	The event might occur under very adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low
		c) House No. 327 Whale Beach Road	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low
		d) Lawn at No. 327 Whale Beach Road	Unlikely	The event might occur under very adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low

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

TABLE: 2

Recommended Maintenance and Inspection Program

Structure	Maintenance/ Inspection Item	Frequency
Stormwater drains.	Owner to inspect to ensure that the drains, and pipes are free of debris & sediment build-up. Clear surface grates and litter.	Every year or following each major rainfall event.
Retaining Walls. or remedial measures	Owner to inspect walls for deveation from as constructed condition and repair/replace. Replace poorly constructed rock walls	Every two years or following major rainfall event. As soon as practicable
Large Trees on or adjacent to site	Arbourist to check condition of trees and remove as required. Where treee within steep slopes or adjacent to structures require geotechincal inspection prior to removal	Every five years
Slope Stability	Hydraulics (stormwater) & Geotechnical Consultants to check on site stability at same time and provide report.	One year after construction is completed.

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

Appendix 3

4 April 2025

WRL Ref: WRL2025023 LR20250404 JTC VMM

Ms Jessica Watson
Jorge Hrdina Architects
10/38 Manning Road
Double Bay NSW 2028

By email: jessica@jorgehrdina.com.au;

UNSW
Water Research
Laboratory

Dear Jessica,

Re: 325 Whale Beach Road, Palm Beach - coastal hazard assessment

1. Introduction

The Water Research Laboratory (WRL) of the School of Civil and Environmental Engineering at UNSW Sydney was engaged by Jorge Hrdina Architects (JHA, the Client) to provide a coastal engineer's coastal hazard assessment for the proposed works on 325 Whale Beach Road, Palm Beach (Figure 1-1). This report addresses the coastal hazards in accordance with Northern Beaches Council (ex-Pittwater Council) policy and the NSW Coastal Management Act 2016. The site was visited by WRL's Principal Coastal Engineer on Wednesday 26 March 2025.

Figure 1-1 Location (Source: Google Earth ©)

2. Executive summary

A summary of the location's coastal hazard assessment in accordance with Northern Beaches Council (ex-Pittwater Council) policy and the NSW Coastal Management Act 2016 is shown in Table 2-1, with details outlined in the following sections.

Table 2-1 Summary of the coastal hazard assessment

Coastal hazard	Summary	WRL comments
(a) Beach erosion	Not applicable	Rocky foreshore. Subject only to geotechnical assessment
(b) Shoreline recession	Not applicable	Rocky foreshore. Subject only to geotechnical assessment
(c) Coastal lake or watercourse entrance instability	Not applicable	Hazard is not present at this site
(d) Coastal inundation	Property above runup level	The R2% wave runup level is approximately 4 m below the lowest part of the property and 9 m below the lowest floor level. Refer to Section 7
(e) Coastal cliff or slope instability	Subject to geotechnical assessment	Subject to geotechnical assessment with further discussion in Section 8
(f) Tidal inundation	No hazard	Hazard is not present at this site Lowest point is 25 m AHD (above mean sea level)
(g) Erosion and inundation due to tidal waters	No hazard	Hazard is not present at this site Lowest point is 25 m AHD (above mean sea level)

3. Proposed works and available information

JHA, the Client proposes to alter the swimming pool near the seaward boundary of the subject property, with an option to alter the internal configuration of the house, without any lowering of floor levels. Information provided to WRL at the time of this report is shown in Table 3-1. The lowest floor level of the existing and proposed house is 29.66 m AHD. The lowest surveyed ground level on the subject property is 24.35 m AHD.

Table 3-1 Information provided to WRL

Source	Description or title	Drawing number	Rev	Date
CMS Surveyors	Site survey	18080Bdetail 4.dwg	4	21/02/2025
JHA	Site plan - Existing	1000	-	31/01/2025
JHA	Site plan - Option B	1001	B	27/02/2025
JHA	Site plan - Option B – Site areas	1001	B	18/03/2025
JHA	Plan – Measured drawings	2200	-	16/01/2025
JHA	Plan – Sketch design option B - Reno	2212	B	18/03/2025

4. Site geometry and bathymetry

4.1 Site visit

The site was visited by WRL's Principal Coastal Engineer James Carley on Wednesday, 26 March 2025. Photos from the site visit are shown in Figure 4-1 to Figure 4-4. As stated above, a site survey was provided to WRL from CMS Surveyors.

Figure 4-1 View from existing house

Figure 4-2 Cliff face

Figure 4-3 Boulder armour at the cliff toe

Figure 4-4 Rock platform

4.2 Bathymetry and topography

The NSW Department of Planning, Industry and Environment (NSW DPIE), provides topographic and bathymetric data based on Airborne LiDAR Bathymetry (ALB) technology conducted by Fugro Pty Ltd from July to December 2018. The bathymetric data was accessed through the ELVIS portal (https://elevation.fsdf.org.au/) and downloaded at a resolution of 5 m.

The site’s topography features a cliff face, rock platform and nearshore slope. The transect line (Figure 4-5) and transect section (Figure 4-6) taken from the widest part of the property and shows a near vertical steep cliff from 23 m AHD to 0 m AHD, with an approximately 70 m wide rock platform located at 0 m AHD (excluding toe rock boulders) continuing with a 1V:5H slope from to -10 m AHD and a 1V:30H slope further seaward.

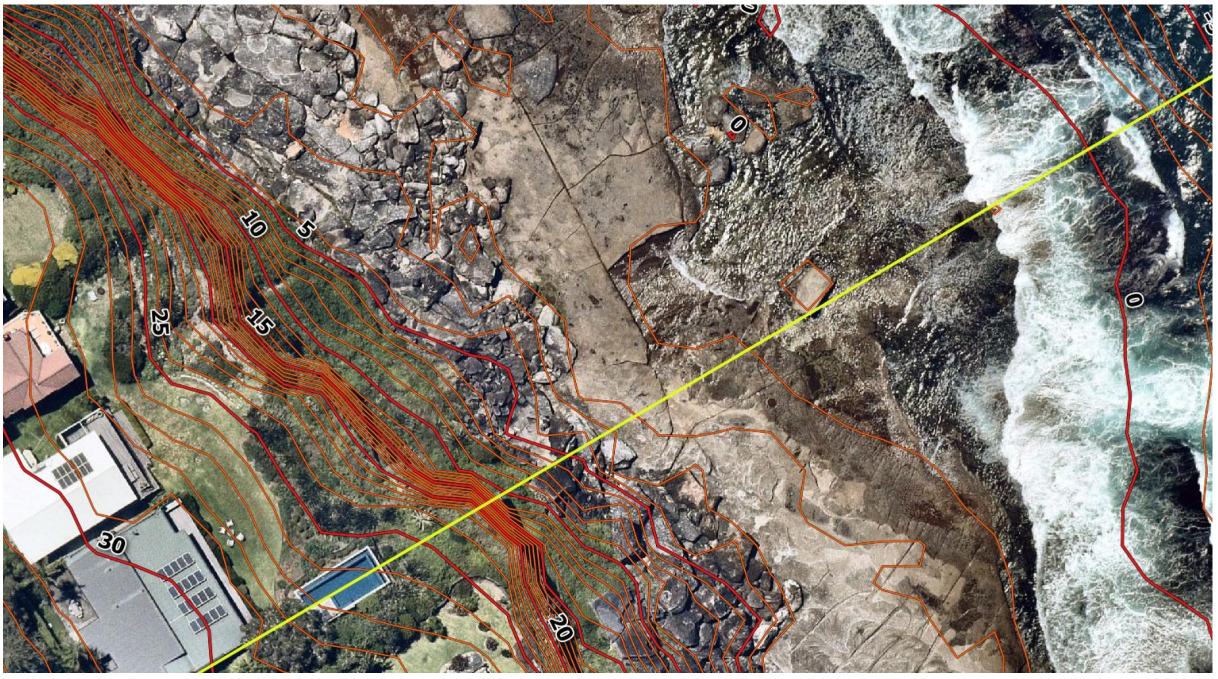


Figure 4-5 Transect and NSW marine LiDAR bathymetry and topography data 2018

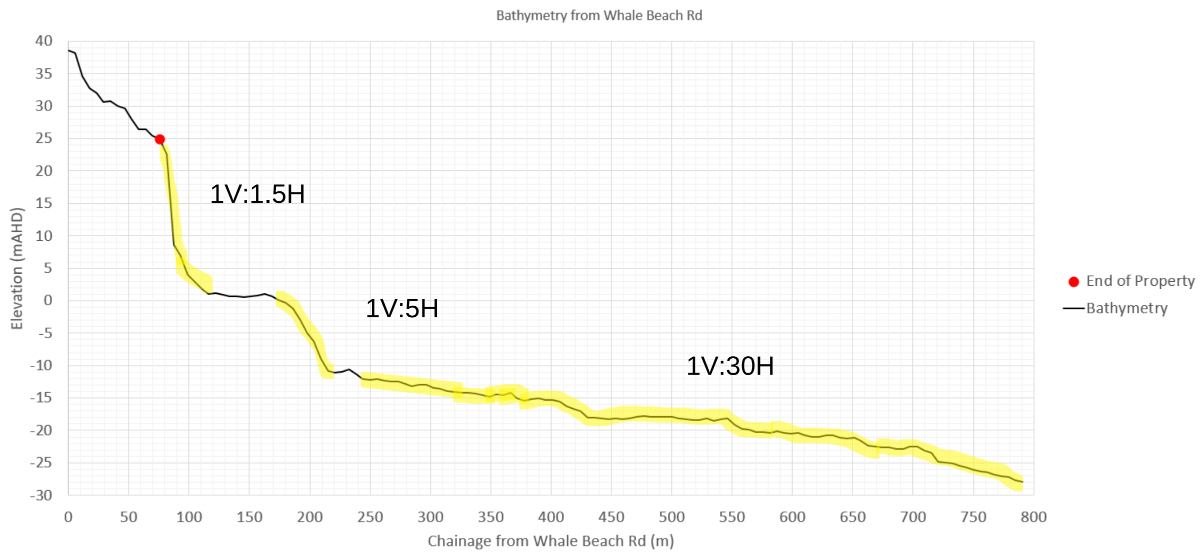


Figure 4-6 Bathymetry and topography transect

5. Water levels and waves

5.1 Water levels

The water levels used are shown in Table 5-1, with the earlier studies corrected for historical sea-level rise (taken as 2 mm/year) at Fort Denison based on the work of Watson (2020). Note that these values exclude wave setup and runup effects. Wave setup is not important for the subject site due to the cliff geometry, while wave runup is calculated separately.

Table 5-1 Design water levels (Sydney) excluding wave setup and wave runup

ARI (years)	2008 water level (m AHD) (NSW DECCW, 2010)	2017 water level (m AHD) (MHL, 2018)	2025 design still water level (m AHD)(2)
100	1.44	1.42	1.46
500	1.54(1)	1.51(1)	1.55

(1) These water level values were extrapolated by WRL using a log-linear fit.

(2) The 2025 design water levels were derived from (NSW DECCW, 2010) and (MHL, 2018) adjusted to 2025 using a constant historical SLR rate of 2 mm/year. The proposed 2025 design still water levels are an average of the adjusted NSW DECCW and MHL water levels.

5.2 Future sea level rise

This report is for a design working life of 60 years, assuming the structure is to be built in 2025.

In the absence of official NSW sea level rise (SLR) benchmarks, the SLR values adopted by WRL were based on the more recent IPCC AR6 (2021) report. The IPCC report provides global mean sea level rise projections for five Shared Socioeconomic Pathways (SSPs), with each SSP capturing different emissions scenarios. WRL adopted SLR values for this study were based on SSP5–8.5 (Very High emissions scenario – medium confidence) to account for the highest risk scenario, using the NASA sea level projection tool (NASA, 2025) for the Sydney, Fort Denison location. SLR projections are shown in Table 5-2.

Table 5-2 Sea level rise projection (SSP5–8.5, Source IPCC, 2021)

Planning Period (year)	Sea Level Rise (m)(1)
2025	0.00
2050	0.18
2065	*0.33*(2)
2080	0.46
2085	*0.53*(2)
2090	0.59
2100	0.72
2150	1.30

(1) SLR values were adjusted to 2025 as IPPC (2021) SLR values are relative to 2020.

(2) 2065 and 2085 SLR values were interpolated using a 2nd degree polynomial fit.

5.3 Waves

The location is characterised by a moderate to high energy wave climate as the offshore bathymetry has a moderate gradient and there are no protective offshore reefs. Estimates for 100 year ARI (Average Recurrence Interval) non-directional offshore waves (Glatz et al., 2017) and directional offshore extreme waves (Shand et al., 2011a) in the Sydney region are provided in Table 5-3. For this analysis, unrefracted waves from the east to south-east wave direction H_s were used to quantify wave runup.

Table 5-3 Offshore directional extreme wave conditions at Sydney wave buoy
(Source: Glatz et al., 2017 and Shand et al., 2011a)

Offshore Wave Direction		One Hour Exceedance H_S (m)
		100 year ARI
All directions (1)	-	9.4
N to E (2)	0 to 90	5.7
E to SE (2)	90 to 135	7.8
SE to SW (2)	135 to 225	9

(1) These values were reported in Galtz et al. (2017).

(2) These values were reported in Shand et al. (2011a).

Offshore peak wave period for design conditions from the Sydney wave buoy (Shand et al., 2011b) are provided in Table 5-4.

Table 5-4 Corresponding peak wave period Tp conditions

ARI (years)	Offshore Tp (s)
100	13.0

6. Coastal hazard assessment

The NSW Coastal Management Act 2016 lists the following coastal hazards:

(a) Beach erosion
(b) Shoreline recession
(c) Coastal lake or watercourse entrance instability
(d) Coastal inundation
(e) Coastal cliff or slope instability
(f) Tidal inundation
(g) Erosion and inundation of foreshores caused by tidal waters and the action of waves, including the interaction of those waters with catchment floodwaters

6.1 (a) Beach erosion hazard

Beach erosion is not applicable for this location due to the site not being located on a sandy beach.

6.2 (b) Shoreline recession hazard

Conventional shoreline recession is not applicable for this location due to the site not being located on a sandy beach. Further discussion can be found in the assessment of (e) Coastal cliff or slope instability.

6.3 (c) Coastal lake or watercourse entrance instability

Coastal lake or watercourse entrance instability is not applicable for this location due to the site not being located at coastal lake or watercourse entrance.

6.4 (d) Coastal inundation

Due to its elevation, the site is not subject to conventional coastal inundation. However, it could be subject to wave runup, which is assessed in more detail in Section 7.

6.5 (e) Coastal cliff or slope instability

This hazard is primarily assessed separately by a geotechnical engineer. Additional input from WRL is provided in Section 8.

6.6 (f) Tidal inundation

Tidal inundation is not a hazard for this location due to the site being located above 25 m AHD (mean sea level).

6.7 *(g) Erosion and inundation caused by tidal waters*

Erosion and inundation caused by tidal waters is not a hazard for this location due to the site being located above 25 m AHD (mean sea level).

7. WRL analysis of wave runup

Wave runup height was estimated for 100 year ARI conditions for 2025, 2065 and 2085 using Equation 6.2 in the EurOtop (2018) Overtopping Manual. The results are shown in Table 7-1. A cross check was made with the method of Mase et al. (2004), which estimated runup levels about 0.5 to 1 m lower than EurOtop (2018). The EurOtop (2018) method was adopted for this study.

Table 7-1 R2% wave runup height (EurOtop, 2018)

ARI (years)	Year	Water level (m AHD)	Wave runup R2% (m AHD)
100	2025	1.46	18.9
100	2065	1.79	20.2
100	2085	1.99	20.5

A visual representation of the EurOtop, 2018 runup extents are shown in Figure 7-1. The wave run up analysis show that even during extreme 100 ARI events the R2% wave runup is more than 4 m below the lowest point of the property and 9 m below the lowest floor level of the house, noting that splash, spray and some individual waves may exceed this design runup level – particularly during strong onshore winds

Figure 7-1 R2% 2085 wave runup extents

8. WRL discussion on cliff erosion/recession

8.1 Geotechnical engineering assessment

Coastal hazard (e) coastal cliff or slope instability requires a specialist geotechnical assessment. Some commentary by WRL on cliff erosion rates is provided below.

8.2 Published cliff erosion/recession rates

Chapman et al. (1982) gave the following commentary on cliff erosion in NSW:

> *“Rates of cliff erosion are highly variable and actual measurements are virtually non-existent. …cliff retreat is highly erratic, with localized and infrequent rock falls separated by long periods of weathering.”*

For the purposes of estimating sediment supply to beaches, Chapman et al. (1982) suggested an order of magnitude estimate of cliff erosion rates for Sydney to be 5 mm per year.

Sunamura (1983) presented a model for cliff recession and collated recession rates from numerous locations around the world. The only Australian locations cited were for limestone at Point Peron near Perth (0.2 to 1 mm per year) and aeolianite at Warrnambool Victoria (14 mm per year). Sunamura (1983) also presented results of physical model studies on cliff recession and platform formation.

Crozier and Braybrooke (1992) examined sea cliffs on Sydney’s northern beaches. Some of these cliffs are shale, which is softer than sandstone. They estimated that the average rate of sandstone cliff erosion was 4.3 mm per year, and the maximum was 12.1 mm per year. They also published sandstone erosion rates from a range of sources (not sea cliffs) which ranged from 0.012 mm/year to 4.6 mm per year. However, “fine clayey grained sandstone” at Beacon Hill was observed to erode at 10 to 17.4 mm per year over 15 years.

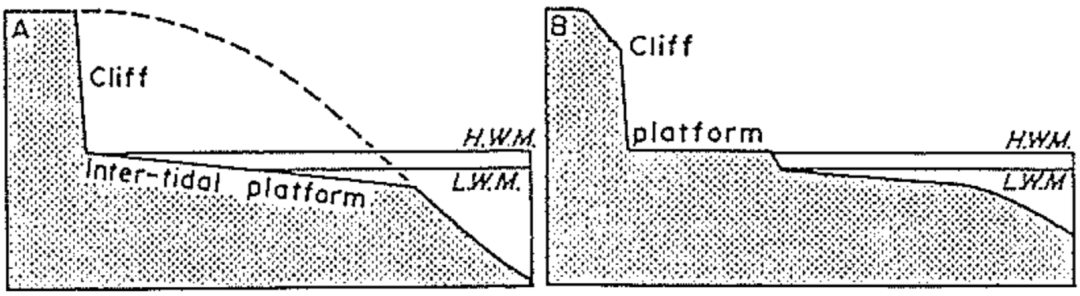
Dragovich (2000) estimated erosion rates of Sydney sandstone in locations with a high salt load to be 1 to 5 mm per year – though this related to dimensioned construction stone rather than sea cliffs subject to wave action. She also quoted Roy (1983) who estimated that the softer beds near the base of sandstone cliffs in the southern Sydney region were weathering at rates of 2 to 5 mm per year.

8.3 Cliff erosion/recession on subject property

The cliff face at the subject property is fronted by a wave cut platform (Figure 4-4 and Figure 8-1) of approximately 70 m width. The landward edge of the platform where it intersects with the cliff base is naturally armoured with an apron fillet of rock boulders from previous cliff face collapses (Figure 4-3). The rock boulders have typical individual rock dimensions of up to 1 to 5 m. The rock boulder apron would provide a degree of armouring protection to the base of the cliff from further wave attack.

Chapman et al. (1982) reported on work from Thom and Chappell (1975) which showed that sea level 10,000 years before present was approximately 30 m below present, however, from approximately 6,000 years before present, sea level has remained roughly constant within an envelope of approximately $\pm$1 m.

On the assumption that the wave cut platform was formed over the duration of this approximately constant sea level, the order of magnitude estimate of the mean rate for formation of the wave cut platform at the cliff base fronting the subject property is estimated to be 12 mm per year.



A— Cliffed coast with an inter-tidal shore platform

B— Cliffed coast with a shore platform at about high tide level (Slope-over-wall)

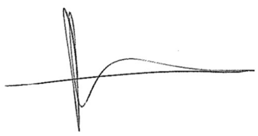
Modified from Bird (1976)

Figure 8-1 Wave cut platform illustration (Crozier and Braybrooke, 1992)

9. Summary

An executive summary is provided in Section 2 of this letter. Please contact James Carley or myself should you require further information.

Yours sincerely,



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