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GEOTECHNICAL CONSULTANTS

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REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

PROPOSED NEW DWELLING

at

18 ROCK BATH ROAD, PALM BEACH, NSW

Prepared For

Drew and Bridget Hall

Project No.: 2022-085

June 2023

Document Revision Record

Issue No	Date	Details of Revisions
0	25th October, 2022	Original issue
1	22nd June 2023	Updated Architectural Plans

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for___ Drew and Bridge Hall ________________________	
Name of Applicant	
Address of site __18 Rock Bath Road, Palm Beach, 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** 22 June 2023* 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 Dwelling with a new Swimming pool at 18 Rock Bath Road, Palm Beach, NSW.

Report Date: 22 June 2023 **Project No.:** 2022-085

Author: Marvin Lujan and Troy Crozier

Author's Company/Organisation: Crozier Geotechnical Consultants

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

Architectural Drawings– by Richard Cole Architecture, Project No.: 2112, Drawing: DA01 to DA29, Dated: 22/05/23 and Drawing No.: DA17, Dated: 30/05/23, Revision: K.
Survey Drawing – by CMS Surveyors Pty Ltd, Date of Survey: 12/12/16 and 3/5/21, Drawing Name: 15889Adetail, Sheet: 1 of 1 and Issue: 1.

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 Drew and Bridge Hall ____________________
Name of Applicant
Address of site ___18 Rock Bath Road, Palm Beach, 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 with a new Swimming pool at 18 Rock Bath Road, Palm Beach, NSW.
Report Date: 22 June 2023 **Project No.:** 2022-085
Author: Marvin Lujan
Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

- ■ Comprehensive site mapping conducted ____5th May 2022__________
- ■ 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 ......5th May 2022.................
- ■ 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**

AUSTRALIAN INSTITUTE OF GEOSCIENTISTS
TROY CROZIER
10197

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APPENDICES

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GEOTECHNICAL REPORT FOR PROPOSED NEW DWELLING WITH A NEW SWIMMING POOL AT 18 ROCK BATH ROAD, PALM BEACH, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for a new dwelling house at 18 Rock Bath Road, Palm Beach, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the written request of Richard Cole Architecture Pty Ltd on behalf of the clients Drew and Bridge Hall.

The site is currently occupied by a two storey rendered dwelling within the southern to south-west portion of the site with timber decks, paving and terraced gardens to the north and east, and contains steep vegetated slopes in the remainder.

It is understood that the proposed works involve demolition of the existing structure and the construction of a new proposed dwelling, a new driveway and a new pool at the north-eastern corner of the new dwelling.

It is also understood that the new dwelling will require bulk excavation down to 4.0m depth reducing to nil towards the north and east and the new swimming pool will require bulk excavation down to 2.50m depth. The dwelling excavation is expected to extend to 1.0m and 0.90m from the southern and western boundaries, respectively.

The site is located within the H1 (highest category) landslip hazard zone as identified within Northern Beaches Councils precinct (Geotechnical Risk Management Policy for Pittwater – 2009). Therefore, to meet Councils Policy requirements, this report includes a Geotechnical Landslip Risk Assessment which meets the requirements of Paragraph 6.5 of that policy.

The site is also located within Bluff/Cliff instability (R) as identified within Northern Beaches Council, Costal Risk Planning Map Sheet CHZ_015. A Coastal Engineers report by Horton Coastal Engineering Pty Ltd has been reviewed and is attached in Appendix 6 of this report for the Development Application (DA) submission.

This report includes a description of site and sub-surface conditions, a geotechnical assessment of the development, a landslide risk assessment for both property and life as per the AGS 2007 suitable for Council, site mapping/plan, a geotechnical model, recommendations for design and construction.

The site assessment and reporting were undertaken as per the Proposal P22-161, Dated: 4th April 2022.

The investigation comprised:

a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Geotechnical Engineer.
b) DBYD plan request and review.
c) Drilling of three boreholes using hand tools along with Dynamic Cone Penetrometer (DCP) testing at five locations to investigate the subsurface conditions.
d) All fieldwork was conducted under the full-time supervision of an experienced Geotechnical Professional.

The following documents and plans were supplied by the Architect and relied upon for proposed preparation, assessment and reporting.

- Architectural Drawings– by Richard Cole Architecture, Project No.: 2112, Drawing: DA01 to DA29, Dated: 22/05/23 and Drawing No.: DA17, Dated: 30/05/23, Revision: K.
- Survey Drawing – by CMS Surveyors Pty Ltd, Date of Survey: 12/12/16 and 3/5/21, Drawing Name: 15889Adetail, Sheet: 1 of 1 and Issue: 1.

2. PROPOSED WORK:

The proposed works involve the demolition of the existing site structures and the construction of a new two storey dwelling with a new swimming pool. Bulk excavation down to 4.0m depth (≤0.90m and ≤1.0m from the western and southern boundary, respectively) will be required for the new dwelling and down to 2.50m depth for the new swimming pool.

The proposed works also include the construction of a new driveway from the intersection between Florida Road and Whale Beach Road to the site.

3. SITE FEATURES:

3.1. Description:

The site is an irregular shaped block located on the lower northern side of the road adjacent to the base of slope and the crest of a series of foreshore cliffs within steep north dipping topography.

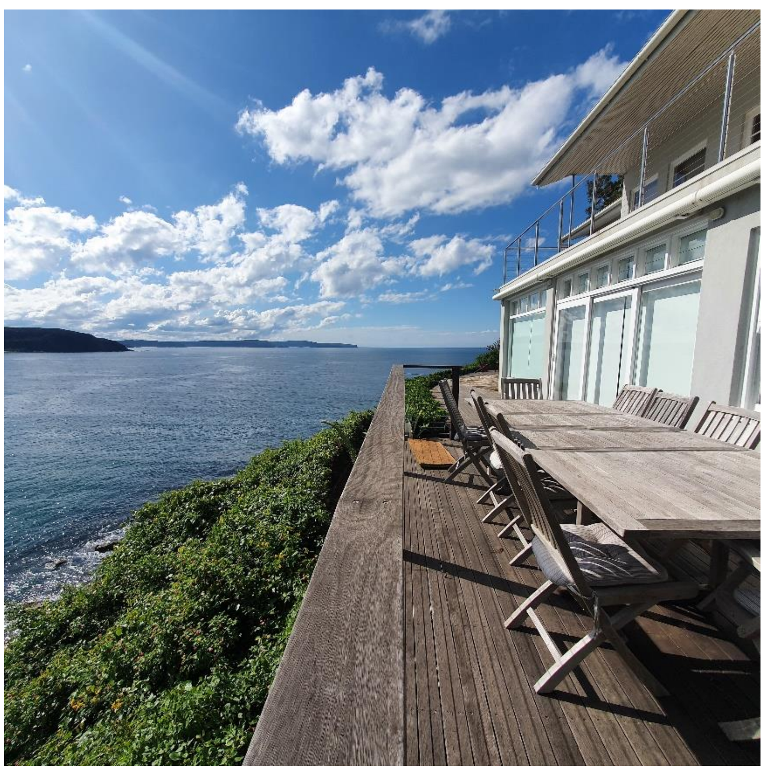
The site has a northern boundary sum of 56.755m, an eastern boundary of 5.49m, a western boundary of 42.67m and a south boundary of 39.43m, as provided by the supplied Survey Plan. The site has a high of RL 29.27m at the south-west corner of the site, reducing north to an approximate low of RL 5.00m at the lower north-west corner.

The curved north to east boundary is located near the crest of an extremely steep (≥60°) slope in the south-east that extends down to sub-vertical cliff lines and a foreshore rock platform. The slope above the cliff line reduces in gradient towards the north and around towards the western boundary.

An aerial photograph of the site and its surrounds is provided below, as sourced from NSW Government Six Map spatial data system, as Photograph-1. General views of the site at the time of investigation are provided in Photograph-2 to Photograph-3.

Photograph-1: Aerial photo of site and surrounds.

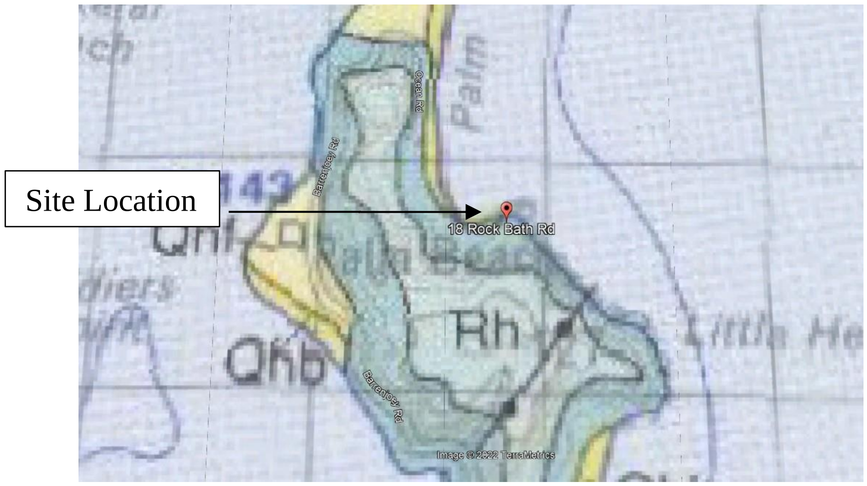
Photograph-2: Eastern portion of the site. View looking west.



Photograph-3: Western portion of the site-dwelling. View looking east.

3.2. Geology:

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



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

4. FIELD WORK:

4.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on the 5th May 2022 by a Geotechnical Engineer. It included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land, existing structures and inspection of neighbouring properties.

It also included the drilling of three auger boreholes (BH1 to BH3) using a hand auger to investigate sub-surface geology. Hand tools were used due to access limitations. Soil sample collection and logging as per “AS1726: 2017 Geotechnical Site Investigation.

Dynamic Cone Penetrometer (DCP) testing was carried out adjacent to the boreholes and at an additional location, in accordance with AS1289.6.3.2 – 1997, “Determination of the penetration resistance of a soil – 9kg Dynamic Cone Penetrometer test” to estimate near surface soil conditions and confirm depths to bedrock.

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

4.2. Field Observations:

The site is currently occupied by a one to two storey rendered dwelling with a timber deck at the northern side of the site-dwelling (Lower Ground Floor Level). The south boundary of the site contains an approximately 2.40m high brick boundary retaining wall. Some cracking (≤10mm wide, ≤ 1.0m long) was observed at the eastern end of the boundary retaining wall (Photograph-4). Minor rotation was observed at the eastern end of the southern boundary retaining wall. The site-dwelling appeared in good condition, signs of underlying geotechnical issues were not observed in the site-dwelling.

Photograph-4: Vertical cracking observed at the eastern side of common boundary wall. View looking south-east.

The western portion of the site is occupied by gardens and concrete steps that step down north to the dwelling's timber deck.

The eastern portion of the site is occupied by stepped lawns and dense vegetation along the eastern boundary with an extremely steep (≥60°) slope/cliff extending down to the crest of a sub-vertical cliff line (approximately 5m high) the base of which is the sloping rock platform near the south-east corner. As this slope extends north it becomes reduced in steepness and more consistently vegetated with the foreshore rock platform of increased lateral extent to the east/north-east.

To the north of the site-dwelling is a sandstone flagging patio and timber deck with a slightly lower garden that is retained along the northern side by an east-west striking timber retaining wall (≤1.40m high). Bordering the base of the timber retaining wall is an approximately ≤1.50m wide pathway that continues north through dense vegetation that very steeply dips north towards the north boundary. An approximately ≤20mm thick separation crack was observed within the sandstone flagging patio (Photograph-5). The adjacent timber retaining wall also contained minor tilting and a depression zone was observed at the ground located next to the base of the timber retaining wall (Photograph-6)

Photograph-5: Open crack observed within the patio. View looking down north-east.

Photograph-6: Minor tilting observed at the retaining wall and a depression zone observed at the pathway. View looking south-west.

The neighbouring property to the south (No. 16 Rock Bath Road) contains a two storey timber dwelling located in the western portion of the block. The property dwelling is surrounded by open grass lawn and dense vegetation. The dwelling extends north to approximately 4.0m from the common boundary and is approximately 2.0m above the site along the boundary supported by the previously detailed brick retaining wall. Limited observation was possible to the neighbouring property, however the inspection did not identify excessive ground movement or underlying geotechnical issues and structures appeared in good condition. The eastern boundary of this property is defined by the extremely steep slope/cliff that intersects the south-east corner of the site.

The ground directly to the west of the site comprises dense vegetation within a steep north dipping slope with a similar ground level to the site along the common boundary. Excessive ground movement, undulations or underlying geotechnical issues were not observed within the area adjacent to the site and appeared in good condition.

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

4.3. Field Testing:

The boreholes were drilled to varying depths between 0.30m (BH2) and 1.00m (BH1 and BH3) with refusal encountered on sandstone bedrock or within fill/sandy clay, respectively.

DCP tests were carried out to varying depths between 0.30m (DCP2) and 1.50 (DCP3) with refusal encountered on interpreted sandstone bedrock.

For a detailed description of the ground conditions encountered at each investigation location, the Borehole Log sheets should be consulted. However, based on the borehole logs and DCP test results, the sub-surface conditions at the project site can be classified as follows:

- **FILL/TOPSOIL/COLLUVIUM** – this layer was encountered to a maximum depth of 1.0m, however it may extend to greater depths. It was classified as loose, brown, fine to medium gained, moist, silty sand with some sandstone gravel and cobbles.
- **SANDY CLAY/CLAY**– underlying the fill this layer was encountered to varying depths between 0.30m and 1.0m and is expected to extend to greater depths (e.g.≤1.50m depth, DCP3). This unit was generally classified as stiff/very stiff and hard within DCP1 below 0.80m depth, orange brown, medium plasticity, moist, sandy clay.
- **SANDSTONE BEDROCK** - underlying the soils, sandstone/siltstone bedrock was interpreted to be encountered at varying depths between 0.30m (DCP2) and 1.50m (DCP3). Based on the site's slope, it appears that sandstone bedrock is encountered at greater depths along the northern and eastern sides.

A free-standing ground water table or significant water seepage were not identified during the investigation.

5. COMMENTS:

5.1 Geotechnical Assessment:

The site investigation identified the presence of fill down to a maximum depth of ≤1.0m, underlaid by sandy clay/clay overlying interpreted sandstone and siltstone bedrock at varying depths between 0.30m (DCP2) and 1.50m (DCP3). A free-standing groundwater table or significant seepage were not encountered at the site and are not anticipated within 5m to 10m below surface based on site location and adjacent topography however minor seepage at the bedrock surface and along defects within it is likely.

There were no signs of large scale or impending landslip instability or geological hazard. However, deflection was observed at the patio (to the north of the dwelling) and eastern side of the southern boundary retaining wall. The slopes to the south-east are extremely steep however there were no signs of recent or impending instability that may detrimentally impact the site.

It is recommended that at least two test pits be excavated at the base of the southern boundary retaining wall to inspect the footings and founding conditions. Where the retaining wall footings are not founded onto/within competent bedrock, it is recommended that these be extended to competent bedrock or supported sufficiently to maintain stability until the new wall is constructed within the site. It is also recommended that a Structural Engineer inspect the adequacy of the retaining wall structure and provide remedial recommendations to repair the existing cracking and minor tilting as necessary.

Based on the test results it is anticipated that the proposed excavation will extend through soil and bedrock down to the Bulk Excavation Level (BEL). However, the lower northern portion of the excavation might only extend through soil only and not through bedrock.

The soils and extremely to very low strength bedrock can be excavated using conventional earthmoving equipment, however low to high strength bedrock will require the use of the rock breaking equipment (e.g. rock hammers). The use of rock hammers can create ground vibrations which could damage the neighbouring and adjacent structures. Care will be required during the demolition, construction and any excavation works to ensure the neighbouring properties, structures and services (e.g. sewer) are not adversely impacted by ground vibrations. Small scale equipment (i.e. rock hammer <250kg) along with rock saw and a good excavation methodology can be used to maintain low vibration levels and avoid the need for full time vibration monitoring. However, this will result in slow excavation progress and it is anticipated that larger scale rock hammers will be preferred. As such Crozier Geotechnical Consultants (CGC) should be consulted regarding the size and type of demolition/excavation equipment proposed and demolition/excavation methodology prior to the works.

Where medium to high strength bedrock with no poorly oriented defects is encountered, it will likely be free standing and can be excavated near vertically without the need for additional support measures. Where defects are encountered additional support may be required (i.e. rock bolts) to maintain stability.

It is recommended that a core drilling investigation through bedrock to at least 2.0m below the Bulk Excavation Level (BEL) be undertaken to at least one location near the south boundary (post the demolition of the existing site structures and prior to the bulk excavation), to confirm the sub-surface conditions (i.e. if any weak zones of rock are identified) prior to final structural design. It is also recommended that a borehole be drilled near the north-east corner of the site to assess the sub-surface geology and suitable founding level for structures in the east/north-east of the site. In addition, regular detail geotechnical inspection during excavation works (every 1.5m depth interval) will be required with potential stop/hold points to allow support installation if determined necessary.

Based on the excavation depth and distance to the site boundaries, it appears that the safe batter slopes are not achievable along the western side of the excavation (where soil excavation extends deeper than >0.40m depth) and along the southern side of the excavation (where soil excavation extends deeper than >0.50m depth).

If the footings of the existing southern boundary retaining wall have been extended down to low to medium strength bedrock and the adequacy of the bedrock to stand unsupported has been confirmed by geotechnical investigation, then the construction of support prior to excavation will not be required along the southern side of the excavation. However, even if proven suitable for unsupported excavation, geotechnical inspection through the bedrock excavation will be required (every 1.5m depth interval). Alternatively, the construction of support prior to excavation will be required. The construction of a soldier pile wall (or similar) onto/within competent bedrock is a viable option.

The construction of support prior to excavation will be required along the western side of the excavation where the safe batter slopes cannot be formed. Similarly, the construction of a soldier pile wall (or similar) onto/within competent bedrock is a viable option.

Due to the slope it is recommended that all new footings extend through the fill and residual soil units and bear onto/within the bedrock of at least low strength to reduce potential for long term creep or shallow instability. Based on the site conditions, there is potential for detached boulders and weak bedrock units. It is recommended that all new footings be inspected by a geotechnical consultant prior the placement of concrete or steel to ensure the competency of the founding conditions. Preliminary allowable bearing pressures appropriate for the bedrock encountered underlying the site are provided in Section 5.3.1.

A review of the Coastal Engineers report indicated a maximum average regression rate of 12mm/year over the next 100yr interval, including an allowance for sea level rises, for the lower cliff line. This would result in 1.2m of lateral erosion, expected to occur in phases over the 100yr interval. This regression will be expected to result in further upper slope erosion resulting in a similar to slightly steeper slope as is currently seen as a result of natural weathering/erosion. However, based on the proposed location of the new development and the recommended extension of footings to at least low strength bedrock, even where instability/erosion occurs the structures will remain stable within the slope.

Additional geotechnical testing will be required within the footing of the proposed new driveway, to assist with the design and construction. CGC can assist with this additional testing, however approval by council must first be obtained by the client for this work, as this area forms part of the road reserve. Extreme care must be required not to damage the sewer and nearby services during the construction process.

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.

5.2. Site Specific Risk Assessment:

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

A. Landslip (rockslide/topple <2.0m³) from bedrock excavation within the Ground Floor Level.
B. Landslip (earth slide <1m³) from soil excavation within Ground Floor Level.
C. Landslip (earth slide >5m³) from instability in natural cliff along eastern boundary

Hazard A was estimated to have a **Risk to Life** of **2.81 x 10^{-9}** for a single person, while the **Risk to Property** was considered to be **'Moderate'**.

Hazard B was estimated to have a **Risk to Life** of up to **5.47 x 10^{-8}** for a single person, while the **Risk to Property** was considered to be **'Very Low'**.

Hazard C was estimated to have a **Risk to Life** of up to **7.03 x 10^{-8}** for a single person, while the **Risk to Property** was considered to be **'Moderate'**.

Although the 'Moderate' Risk to Property for Hazard A is considered to be 'Unacceptable', the assessment was based on excavations with no support, underpinning, planning or footings for structures to the east that are not suitably founded.

Provided the recommendations of this report are implemented including installation of retaining walls prior to bulk excavation (or similar) where required or founding of structures to low strength bedrock the likelihood of any failure/impact from instability becomes 'Rare' and as such the consequences reduce and risk becomes within 'Acceptable' levels when assessed against the criteria of the AGS. As such the project is considered suitable for the site provided the recommendations of this report are implemented.

5.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

5.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Class 'P' due to slope and ground conditions. Class 'A' for footings on bedrock within excavation base
Type of Footing	Shallow Strip or Pad footings or piers.
Sub-grade material and Maximum Allowable Bearing Capacity for shallow footings	- Very Low Strength bedrock: 800kPa - Low Strength bedrock: 1000kPa - Medium Strength bedrock: 1500kPa*
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e – Rock Site
Remarks: * Higher bearing pressures available through core drilling of bedrock 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. Individual structures should not be founded on materials with varying bearing and settlement characteristics unless the potential for differential movement has been allowed for in structural design. Pier footings may require temporary liners particularly if piers are bored during or prior to periods of rainfall or where fill soils are unstable.	

5.3.2. Excavation:

<u>Property Separation</u>

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

<u>GFL Excavation</u>

Table 1: Property Separation Distances

Boundary	**Adjacent Property**	**Structure**	**Bulk Excavation Depth (m bgl)**	**Separation Distances (m)**	
				Boundary (m)	**Structure**
North	No property is located to the north of the site				
East	No property is located to the east of the site				
South	No.16	Lawn and dwelling	Excavation down to 4.0m depth decreasing	Boundary is ≤1.0m from the excavation	-Boundary retaining wall and lawn directly adjacent to the boundary -Dwelling located

			north to 1.50m depth for proposed new dwelling and then to nil for proposed new terrace		approximately 4m from the common boundary
West	Thick Vegetation			Boundary is ≤0.90m from the excavation	Garden directly adjacent to the boundary
	150mm VC Sewer main (SW)				Approximately 5.0m from the boundary
	100mm CICL Potable water pipe (SW)				

Pool Excavation

Table 2: Property Separation Distances

Boundary	Adjacent Property	Structure	Bulk Excavation Depth (m bgl)	Separation Distances (m)	
				Boundary (m)	Structure
North	No property is located to the north of the site				
East	No property is located to the east of the site				
South	No.16	Lawn and dwelling	Excavation down to 2.50m depth	Boundary is ≤4.0m from the excavation	-Boundary retaining wall and lawn directly adjacent to the boundary -Dwelling located approximately 4m from the common boundary
West	Thick Vegetation			Boundary is ≤20.0m from the excavation	Garden directly adjacent to the boundary
	150mm VC Sewer main (SW)				Approximately 5.0m from the boundary
	100mm CICL Potable water pipe (SW)				

Type of Material to be Excavated	Fill/disturbed clay and clay/sandy clay
	Very Low strength quickly grading to Low and then Medium Strength at depths below > 0.50m (DCP3a)

Guidelines for batter slopes for this site are tabulated below:

Material	Safe Batter Slope (H:V)	
	Short Term/Temporary	Long Term/Permanent
Fill/ Sand	1.5:1	2:1
Sandy Clay/Clayey Sand residual soils and ELS	1:1	1.5:1
VLS - LS, fractured bedrock	0.5:1	1:1*
MS, defect free bedrock	Vertical*	Vertical*

*Dependent on defects and assessment by Geotechnical Engineer.

Remarks: Seepage at the bedrock surface or along defects in the soil/rock can also reduce the stability of batter slopes or rock cuts and invoke the need to implement additional support measures.

Where safe batter slopes are not implemented the stability of the excavation cannot be guaranteed until the installation of permanent support measures. This should also be considered with respect to safe working conditions.

Equipment for Excavation	Residual Soils	Excavator with bucket
	VLS bedrock	Excavator with bucket and ripper
	LS to MS/HS bedrock	Rock hammer and saw

ELS – extremely low strength, VLS – very low, LS – low, MS – medium, HS – high strength

Remarks:

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

Buffer Distance from Structure	Maximum Hammer Weight
2.0m	200kg
4.0m	500kg
5.0m	800kg
8.0m	1000kg

Onsite calibration will provide accurate vibration levels to the site specific conditions and will generally allow for larger excavation machinery or smaller buffers to be used. Calibration of rock excavation machinery should be carried out prior to commencement of rock excavation works where ≥250kg rock hammers are proposed for use.

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

The strength of bedrock below the maximum depth achieved during the investigation is unconfirmed and would require cored boreholes using specialist restricted access drilling equipment.

Excavation of soils to ELS will not create excessive vibrations provided it is undertaken with medium scale (<20 tonne excavator) excavation equipment in a sensible manner.

Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	No. 16 Rock Bath Road = 5mm/s Sydney water sewer asset = 10mm/s (for intermittent vibrations) and 5mm/s (for continuous vibrations). Potable water hydrant – Subject to SW

Vibration Calibration Tests Required	Yes, recommended for any rock hammer >250kg weight
Full time vibration Monitoring Required	Pending proposed equipment and vibration calibration testing results
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • Test pits of boundary retaining wall, • Core drilling investigation to at least one location near the south boundary, post demolition of the site-dwelling and prior to the bulk excavation and one location to the east/north-east, • Following cleaning of soils from bedrock surface in area of excavation, • Prior to the bedrock excavation to assess the size of the excavation machinery for the need for vibration monitoring • During the construction of any type of excavation support system, • At 1.50m depth interval of excavation where unsupported, • At completion of the excavation, including for footings • Additional investigation within the footprint of the new driveway (provided the investigation has been approved by Council)
Pending proposed works	Recommended to the neighbouring properties within 2.0H:1.0V influence zone from base of proposed excavation. This will reduce the potential for 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.

It is recommended that a small (≤250kg) hammer be used for excavation of any LS-MS sandstone on this site. This size equipment will produce negligible ground vibrations. However, this equipment will result in very slow progress of the large excavation, therefore should larger equipment be proposed then CGC should be contacted prior to its use.

5.3.3. Retaining Structures:	
Required	All excavation will need retaining structures unless permanent batters can be formed
Types	Where support prior to excavation is required the construction of a soldier pile or similar with shotcrete infill panels appears a viable option. Support post excavation is a viable option where safe batter slopes can be formed, via construction of steel reinforced concrete/concrete block wall. 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)	
Sandy Fill/Sand	18	$\phi' = 28°$	0.35	0.52	N/A
Sandy Clay/ Clay (Stiff-Hard)	20	$\phi' = 26°$	0.42	0.59	3.25
LS bedrock (fractured)	23	$\phi' = 40°$	0.10	0.15	300kPa
MS-HS bedrock	24	$\phi' = 40°$	0.01	0.05	400 kPa

Remarks:

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

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

5.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		No
Excavation likely to intersect	Water Table	No
	Seepage	Minor (<0.5L/min/m around excavation), on defects and at soil/rock interface.
Site Location and Topography		Low northern side of the road within steeply north dipping topography adjacent to the base of ridge and crest of sea shore cliffs.
Onsite Stormwater Disposal		Possible at rear boundary via dispersion only
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 to the Council's stormwater system off site.		

5.4. Conditions Relating to Design and Construction Monitoring:

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

1. Undertake additional geotechnical testing as recommended in this report,
2. Review and approve the structural drawings for compliance with the recommendations of this report,
3. Conduct inspections of excavation as per Section 5.3 of this report
4. Inspect all new footings and earthworks as per Section 5.3 of this report to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and the stability prior to the placement of steel or concrete.

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

5.5. Design Life of Structure:

We have interpreted the design life requirements specified within Councils Risk Management Policy to refer to structural elements designed to support the house etc, 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 and existing 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 – 1996 (50 years)). In order to attain a design life of 100 years as required by the Councils Risk Management Policy, it will be necessary for the structural and geotechnical engineers to incorporate appropriate design and inspection procedures during the construction period. Additionally the property owner should adopt and implement a maintenance and inspection program. It should be noted that timber log/sleeper retaining walls will not remain stable for 100 years.

If this maintenance and inspection schedule are not maintained the design life of the property cannot be attained. A recommended program is given in Table: 2 and should also include the following guidelines.

- The conditions on the block don't change from those present at the time this report was prepared, except for the changes due to this development.
- There is no change to the property due to an extraordinary event external to this site, 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 Council will control development on neighbouring properties, carry out regular inspections and maintenance of the road verge, stormwater systems and large trees on public land adjacent to the site so as to ensure that stability conditions do not deteriorate with potential increase in risk level to the site. Also individual Government Departments will maintain public utilities in the form of power lines, water and sewer mains to ensure they don't leak and increase either the local groundwater level or landslide potential.

6. CONCLUSION:

The site investigation identified the presence of shallow fill/colluvium, underlaid by clay/sandy clay, overlying bedrock at varying depths between 0.30m (DCP2) and 1.50m (DCP3). A free-standing groundwater table or seepage were not encountered at the site.

The geotechnical investigation did not identify any impending geological hazard or landslip instability that may impact the site / existing or proposed structures. The coastal assessment determined an estimated 1.20m of lateral erosion/regression of the cliff line at the base of the adjacent slope over the 100yrs design life of the proposed development.

Based on this assessment, it is considered that the proposed development will not be impacted by the regression and the subsequent erosion of the slopes above, provided the recommendations of this report are implemented, including footings to bedrock at stable levels, which will be determined by further investigation.

It is recommended that all new footings be founded onto bedrock of similar bearing and at least low strength. Preliminary allowable bearing capacity appropriate for the site are provided in Section 5.3.1 of this report.

It is anticipated that the proposed excavation will extend through bedrock and soil. CGC should inspect the demolition and excavation machinery prior to its use to determine if a full-time vibration monitoring will be required.

Additional testing is recommended via a core drilling investigation near the east/north-east boundary for footing design and near the south boundary, along with the excavation of at least two test pits to assess the footing and founding conditions of the existing south boundary retaining wall, for excavation support design. Geotechnical inspection is also recommended at 1.50m depth interval through the bedrock excavation, whilst pre-excavation support may be required for the lower level excavation near the south boundary and also the west boundary.

Where the footings of the south boundary retaining wall are extended to bedrock, then the construction of support prior to excavation will only be required where the safe batter slopes are not achievable due to geotechnical stability concerns.

It is considered that the proposed works can be achieved and maintained within the 'Acceptable' risk management criteria of the Council policy for a 100yr design life of the new development with negligible impact to the neighbouring properties or structures provided the recommendations of this report and any future geotechnical directive are implemented.

Prepared By:	Reviewed By:
Marvin Lujan	Troy Crozier
Geotechnical Engineer	Principal
	MIE Aust – Geotechnical Engineer.
	MAIG, RPGeo – Geotechnical and Engineering
	Registration No.: 10197

7. REFERENCES:

1. 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.
2. E. Hoek & J.W. Bray 1981, "Rock Slope Engineering" By The Institution of Mining and Metallurgy, London.
3. C. W. Fetter 1995, "Applied Hydrology" by Prentice Hall. V. Gardiner & R. Dackombe 1983, "Geomorphological Field Manual" by George Allen & Unwin.

Appendix 1

CROZIER GEOTECHNICAL CONSULTANTS

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NOTES RELATING TO THIS REPORT

Introduction

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

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

Description and classification Methods

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

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

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

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

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

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

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

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

Sampling

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

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

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

Drilling Methods

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

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

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

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

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

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

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

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

Standard Penetration Tests

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

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

The test results are reported in the following form.

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

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

Cone Penetrometer Testing and Interpretation

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

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

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

The information provided on the plotted results comprises: -

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

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

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

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

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

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

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

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

Dynamic Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Borehole Logs

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

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

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

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

Ground Water

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

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

Engineering Reports

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

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

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

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

Site Anomalies

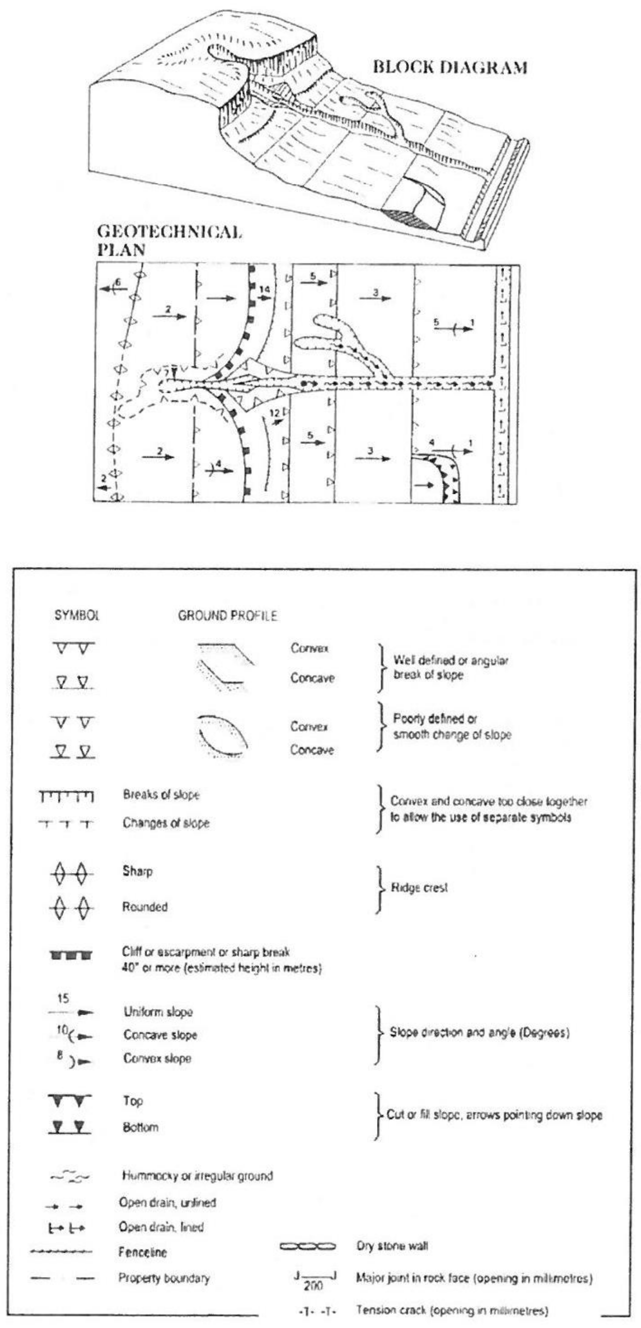
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

Reproduction of Information for Contractual Purposes

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

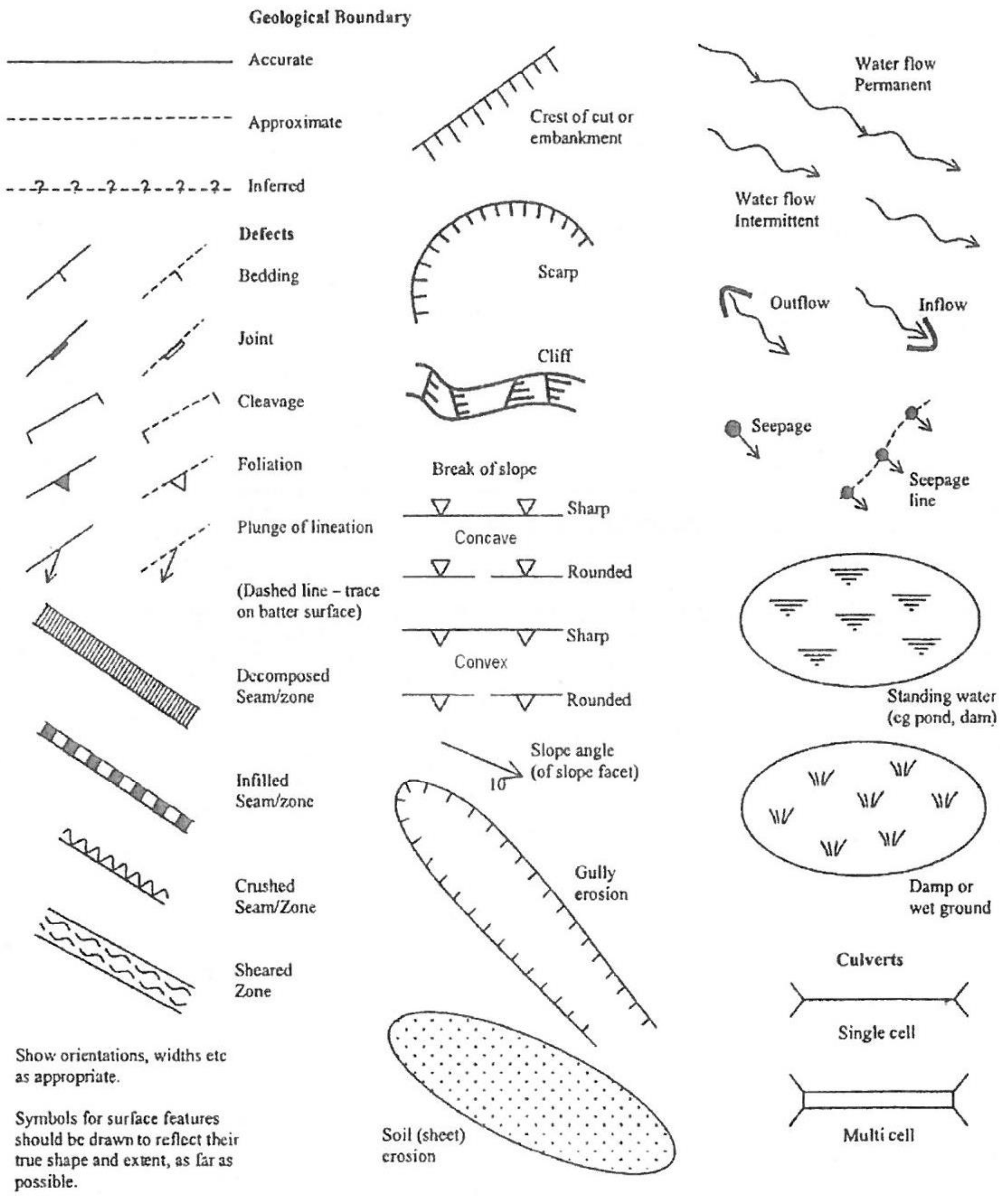
Site Inspection

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



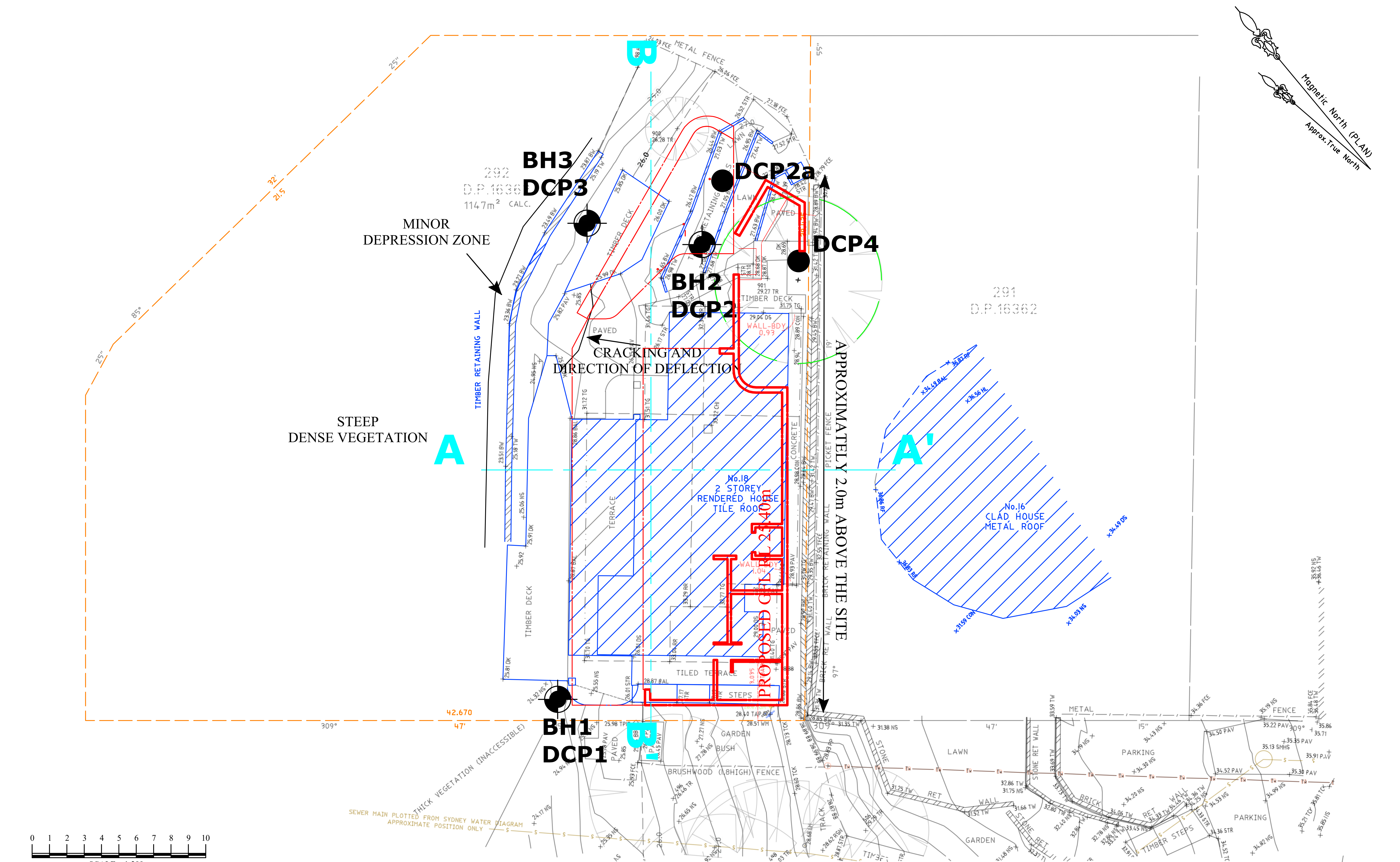
Example of Mapping Symbols
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



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

Appendix 2



SITE PLAN & TEST LOCATIONS FIGURE 1.

CROZIER
GEOTECHNICAL CONSULTANTS

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

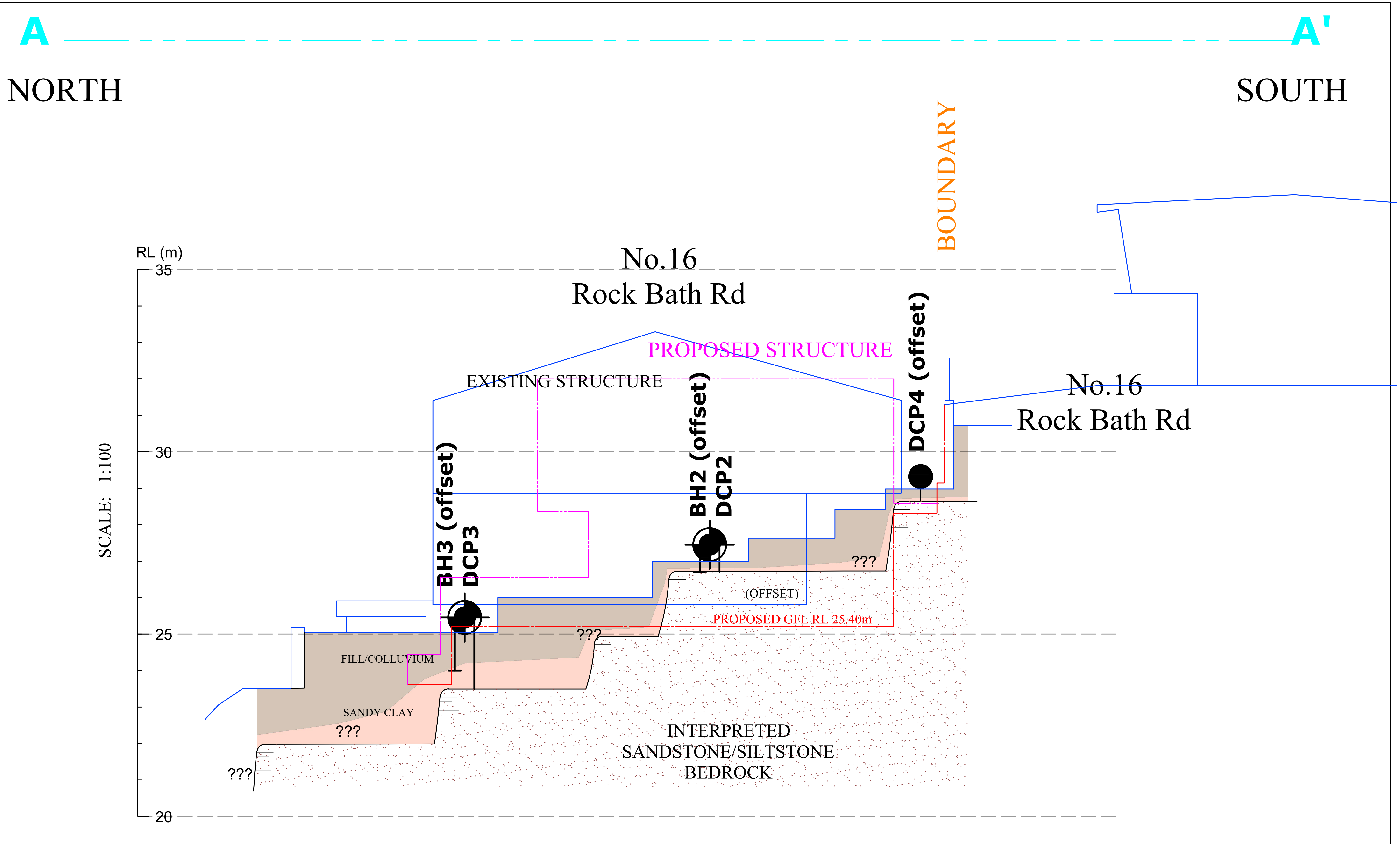
LEGEND

A — A'	CROSS-SECTION REFERENCE LINE
	PROPERTY BOUNDARY
	EXISTING STRUCTURE
	EXCAVATION OUTLINE
BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
DCP	DYNAMIC CONE PENETROMETER

SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	30/05/2022
APPROVED BY:	TMC
DRAWN BY:	ML
PROJECT:	2022-085

PREPARED FOR:
BRIDGET HALL

ADDRESS:
18 ROCK BATH ROAD, PALM BEACH, NSW



0 1 2 3 4 5

SCALE: 1:100

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

GEOLOGICAL MODEL FIGURE 2.

LEGEND

- A — A' CROSS-SECTION REFERENCE LINE
- BH DCP AUGER / DYNAMIC CONE PENETROMETER LOCATION
- SANDY CLAY
- INTERPRETED SANDSTONE /SILTSTONE BEDROCK
- PROPERTY BOUNDARY
- EXCAVATION OUTLINE
- DCP DYNAMIC CONE PENETROMETER
- TP DCP TEST PIT / DYNAMIC CONE PENETROMETER LOCATION
- SOIL/FILL

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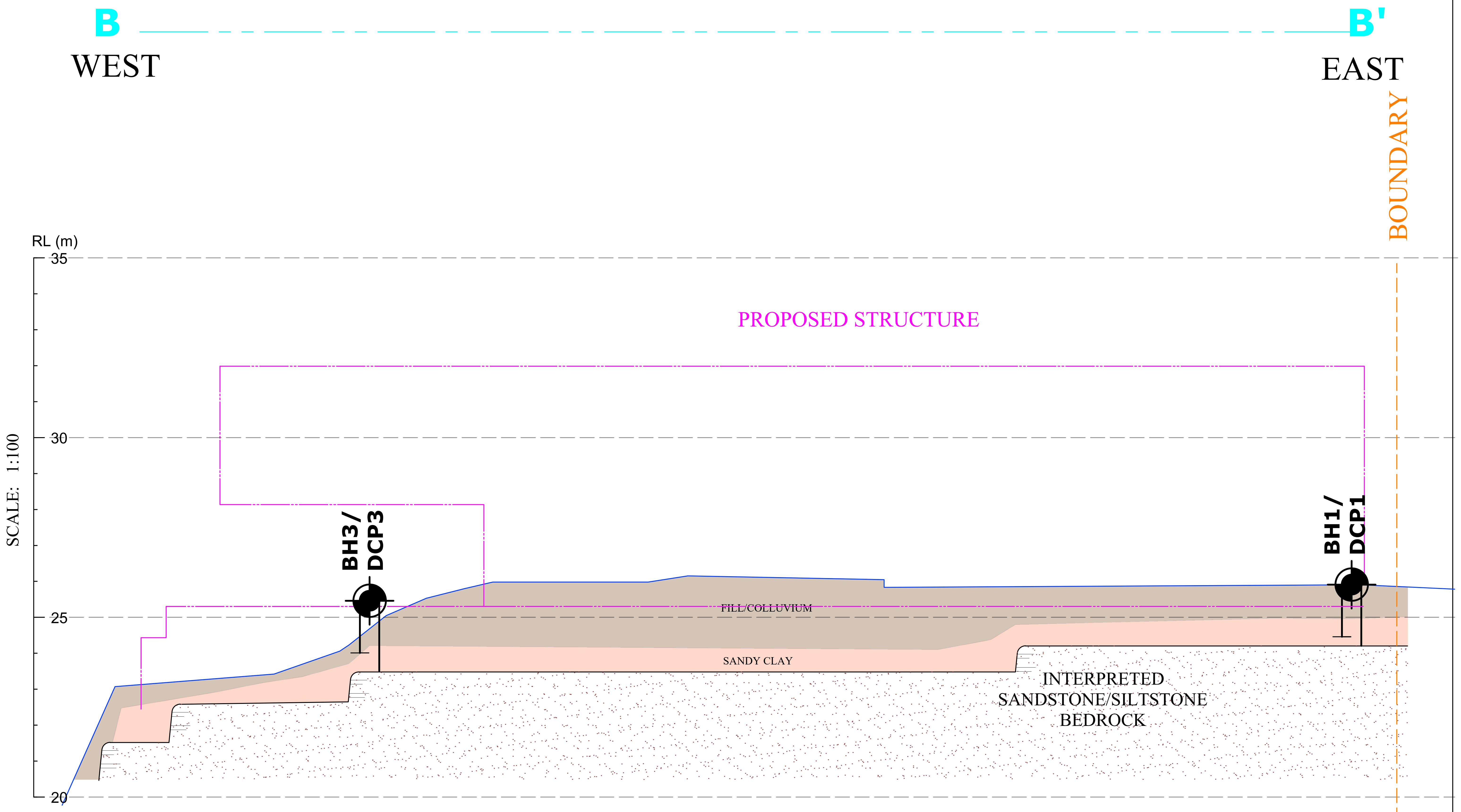
SCALE:	1:100 @ A3
DRAWING:	FIGURE 2
DATE:	30/05/2022
APPROVED BY:	TMC
DRAWN BY:	ML
PROJECT:	2022-085

PREPARED FOR:
BRIDGET HALL

ADDRESS:
18 ROCK BATH ROAD, PALM BEACH, NSW



0 1 2 3 4 5

SCALE: 1:100

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

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

GEOLOGICAL MODEL FIGURE 3.

CROZIER
GEOTECHNICAL CONSULTANTS

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LEGEND

A — A' CROSS-SECTION REFERENCE LINE

BH DCP AUGER / DYNAMIC CONE PENETROMETER LOCATION

SANDY CLAY

INTERPRETED SANDSTONE /SILTSTONE BEDROCK

PROPERTY BOUNDARY

DCP DYNAMIC CONE PENETROMETER

EXCAVATION OUTLINE

SOIL/FILL

SCALE:	1:100 @ A3
DRAWING:	FIGURE 3
DATE:	20/10/2022
APPROVED BY:	TMC
DRAWN BY:	ML
PROJECT:	2022-085

PREPARED FOR:
BRIDGET HALL

ADDRESS:
18 ROCK BATH ROAD, PALM BEACH, NSW

BOREHOLE LOG

CLIENT: Bridget Hall **DATE:** 5/05/2022 **BORE No.:** 1

PROJECT: Demolition and construction of new dwelling **PROJECT No.:** 2022-085 **SHEET:** 1 of 1

LOCATION: 18 Rock Bath Road, Palm Beach **SURFACE LEVEL RL(m):** 25.45

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		FILL/TOPSOIL/COLLUVIUM: Loose, dark brown, fine to medium grained, moist, silty sand				
0.20		… orange red mottled brown, silty sandy clay with some tree roots and sandstone gravels				
0.50	CI	SANDY CLAY: Firm, orange brown, medium plasticity, moist, sandy clay				
			D	0.60 0.70		
0.80		… hard				
1.00		HAND AUGER DISCONTINUED at 1.0m depth within hard sandy clay				

RIG: None DRILLER: PS

METHOD: Hand Auger LOGGED: ML

GROUND WATER OBSERVATIONS: None

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Bridget Hall **DATE:** 30/05/2022 **BORE No.:** 2

PROJECT: Demolition and construction of new dwelling **PROJECT No.:** 2022-085 **SHEET:** 1 of 1

LOCATION: 18 Rock Bath Road, Palm Beach **SURFACE LEVEL RL(m):** 27.00

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
0.00			Type	Tests	Type	Results
0.20		FILL/TOPSOIL: Loose, dark brown, fine to medium grained, moist, silty sand				
0.30	CL/CI	CLAY: Firm, orange brown, low to medium plasticity, moist, clay … with some sandstone gravel				
		HAND AUGER REFUSAL at 0.30m depth on siltstone/sandstone bedrock				

RIG: None DRILLER: PS

METHOD: Hand Auger LOGGED: ML

GROUND WATER OBSERVATIONS: None

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Bridget Hall **DATE:** 30/05/2022 **BORE No.:** 3

PROJECT: Demolition and construction of new dwelling **PROJECT No.:** 2022-085 **SHEET:** 1 of 1

LOCATION: 18 Rock Bath Road, Palm Beach **SURFACE LEVEL:** 25.00

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	Tests	Type	Results
0.00		FILL/COLLUVIUM: Loose, dark brown/orange, fine to medium grained, moist, silty sand with some sandstone gravel/cobbles				
0.80	CI	SANDY CLAY: Sitff, orange red, medium plasticity, moist, sandy clay with some ironstone gravel				
1.00		HAND AUGER DISCONTINUED at 1.0m depth due to cobble from the fill layer have fallen into the borehole				

RIG: None DRILLER: PS

METHOD: Hand Auger LOGGED: ML

GROUND WATER OBSERVATIONS: None

REMARKS: CHECKED: TMC

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Bridget Hall | **DATE:** 5/05/2022
PROJECT: Demolition and construction of new dwelling | **PROJECT No.:** 2022-085
LOCATION: 18 Rock Bath Road, Palm Beach | **SHEET:** 1 of 1

	Test Location									
Depth (m)	DCP1	DCP2	DCP2a	DCP3	DCP4					
0.00 - 0.10	--	1	1	--	--					
0.10 - 0.20	--	2	2	--	--					
0.20 - 0.30	--	2 (B) 0.30m depth	2	4	--					
0.30 - 0.40	--		3	0	7 (B) 0.40m depth					
0.40 - 0.50	--		3 (B) 0.50m depth	0						
0.50 - 0.60	--			2						
0.60 - 0.70	--			4						
0.70 - 0.80	--			4						
0.80 - 0.90	10			4						
0.90 - 1.00	8			3						
1.00 - 1.10	8			5						
1.10 - 1.20	7			7						
1.20 - 1.30	8 (B) 1.25m depth			6						
1.30 - 1.40				5						
1.40 - 1.50				6 (B) 1.50m depth						
1.50 - 1.60										
1.60 - 1.70										
1.70 - 1.80										
1.80 - 1.90										
1.90 - 2.00										
2.00 - 2.10										
2.10 - 2.20										
2.20 - 2.30										
2.30 - 2.40										
2.40 - 2.50										
2.50 - 2.60										
2.60 - 2.70										
2.70 - 2.80										
2.80 - 2.90										
2.90 - 3.00										
3.00 - 3.10										
3.10 - 3.20										
3.20 - 3.30										
3.30 - 3.40										
3.40 - 3.50										
3.50 - 3.60										
3.60 - 3.70										
3.70 - 3.80										
3.80 - 3.90										
3.90 - 4.00										

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

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

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Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (rock slide/topple <2.0m^3) within the Ground Floor Level rock excavation		Excavation ≤4.0m depth decreasing north to nil within bedrock	a) Lawn ≤1.0m from the excavation, impact 5%		a) Person in the lawn 2hrs/day avge.	a) Likely to not evacuate	a) Person likely injured by fall or structure failure	
			Rare	**Prob. of Impact**	**Impacted**				
		a) No.16 Rock Bath Rd (Lawn)	0.00001	0.10	0.05	0.08	0.75	0.90	**2.81E-09**
B	Landslip (earth slide <1.0m^3) from soils due to the Ground Floor Level excavation		Excavation ≤4.0m depth decreasing norto to nil within soil	a) Garden located 0.90m from the excavation, impact 5%		a) Person in the garden 1hr/day avge.	a) Likely to not evacuate	a) Person likely injured by fall	
			Unlikely	**Prob. of Impact**	**Impacted**				
		a) Rock Bath Rd (garden reserve)	0.0001	0.50	0.05	0.04	0.75	0.70	**5.47E-08**
C	Landslip (earth slide >5.0m^3) from natural slopes to south-east/east of site		Slope is extremely steep, however shows no signs of lage scale undercutting or over-steepening	a) Pool and Terrace located adjacent to crest of slope, founded via piles to low strength bedrock		a) Person in the pool/terrace 1hr/day avged.	a) Likely to not evacuate	a) Person in structure, minor damage only	
			Possible	**Prob. of Impact**	**Impacted**				
		a) Proposed Swimming Pool and Terrace/steps	0.001	0.90	0.05	0.04	0.75	0.05	**7.03E-08**

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

* likelihood of occurrence for design life of 100 years

* Spatial Impact - Probaility of Impact refers to slide impacting structure/area expressed as a % (i.e. 1.00 = 100% probability of slide impacting area if slide occurs).

Impacted refers to expected % of area/structure damaged if slide impacts (i.e. small, slow earth slide will damage small portion of house structure such as 1 bedroom (5%), where as large boulder roll may damage/destroy >50%)

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

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

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

* evacuation scale from Almost Certain to not 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 (rock slide/topple <2.0m³) within the Ground Floor Level rock excavation	a) No.16 Rock Bath Rd (Lawn)	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Moderate
B	Landslip (earth slide <1.0m³) from soils due to the Ground Floor Level excavation	a) Rock Bath Rd (garden reserve)	Possible	The event could occur under adverse conditions over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low
C	Landslip (earth slide >5.0m³) from natural slopes to south-east/east of site	a) Proposed Swimming Pool and Terrace/steps	Likely	Event will probably occur under adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Moderate

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

TABLE: 2

Recommended Maintenance and Inspection Program

Structure	Maintenance/ Inspection Item	Frequency
Stormwater drains.	Owner to inspect to ensure that the open drains, and pipes are free of debris & sediment build-up. Clear surface grates and litter. Owner to check and flush retaining wall drainage pipes/systems	Every year or following each major rainfall event. Every 7 years or where dampness/moisture issues identified
Retaining Walls. or remedial measures	Owner to inspect walls for deveation from as constructed condition and repair/replace. Replace non engineered rock/timber walls prior to collapse	Every two years or following major rainfall event. As soon as practicable
Large Trees on or adjacent to site	Arborist to check condition of trees and remove as required. Where tree within steep slopes (>18°) or adjacent to structures requires geotechincal inspection prior to removal	Every five years
Slope Stability	Geotechnical Engineering Consultant to check on site stability and maintenance	When development completed and at 10 to 15 year intervals thereafter

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}	5×10^{-2}	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}	5×10^{-3}	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	5×10^{-4}	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	5×10^{-5}	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	5×10^{-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.

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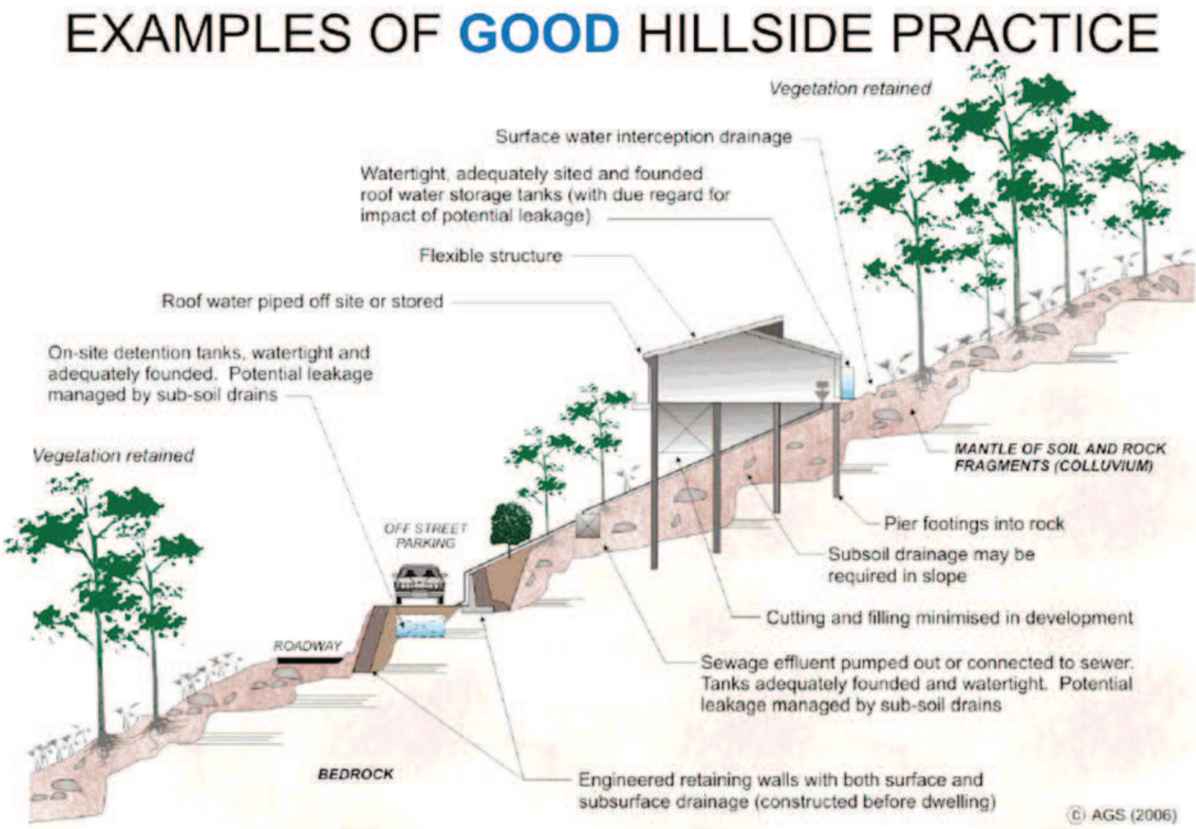
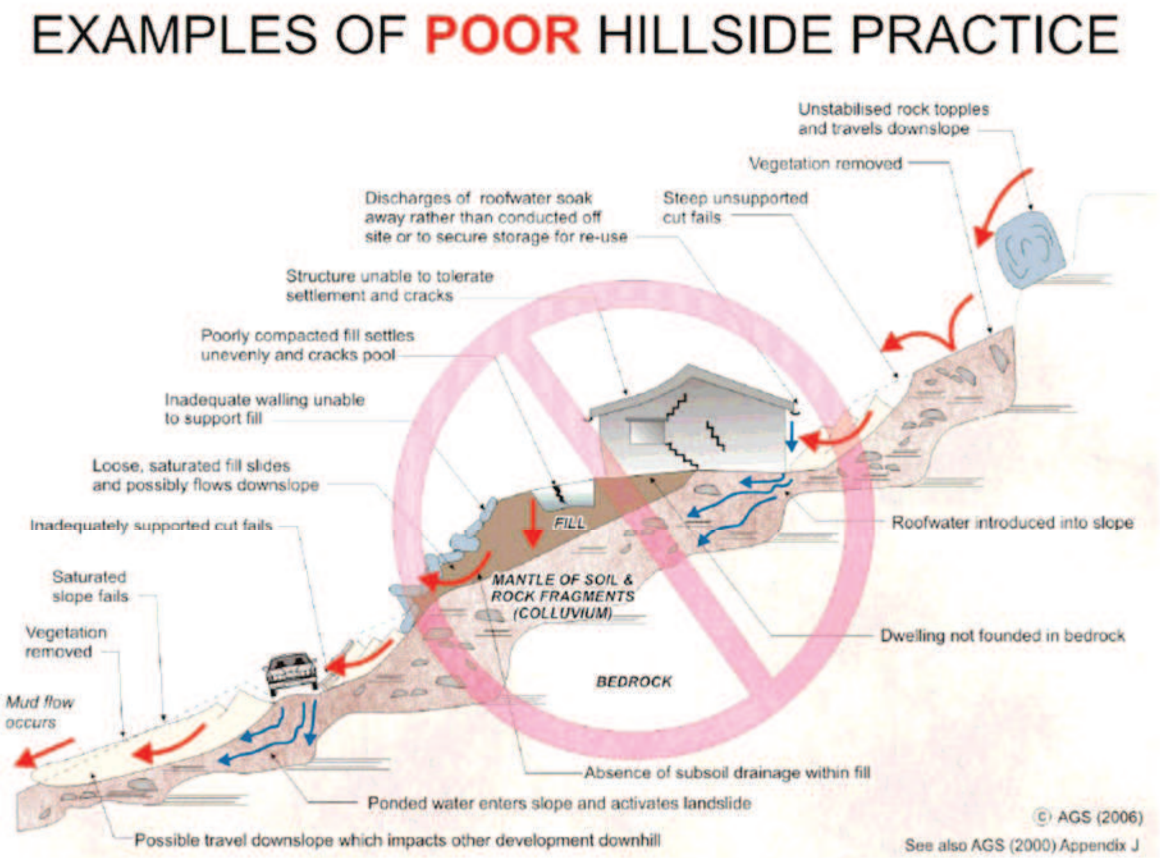
Appendix 5

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
OFF STREET PARKING
ROADWAY
MANTLE OF SOIL AND ROCK FRAGMENTS (COLLUVIUM)
Pier footings into rock
Subsoil drainage may be required in slope
Cutting and filling minimised in development
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

Appendix 6

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ABN 31 612 198 731
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Drew and Bridget Hall
C/- Crozier Geotechnical Consultants
Attention: Troy Crozier
Unit 12, 42–46 Wattle Road
Brookvale NSW 2100
(sent by email only to troy@croziergeotech.com.au)

16 June 2023

Coastal Engineering Advice on 18 Rock Bath Road Palm Beach

1. INTRODUCTION AND BACKGROUND

It is proposed to demolish and rebuild a dwelling at 18 Rock Bath Road Palm Beach, hereafter denoted as 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_015) 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 31 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 Rock Bath Road. He undertook a specific inspection of the site (including its cliff faces) and adjacent rock platform on 30 November 2022.

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

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

2. INFORMATION PROVIDED

Horton Coastal Engineering was provided with a total of 26 Richard Cole Architecture drawings, namely a cover sheet and Drawings DA01 to DA25, all Issue K and dated 22 May 2023 (except DA17 was dated 30 May 2023). A site survey by CMS Surveyors was also provided, Drawing 15889Bdetail, Issue 2 and dated 31 March 2022.

3. EXISTING SITE DESCRIPTION

The site is located landward of a rock platform and rocky cliff/bluff which extends between the sandy Whale Beach in the south and sandy Palm Beach in the north. A vertical aerial view of the site is provided in Figure 1, with section locations denoted as Section A and Section B (approximately perpendicular to the cliff face) also depicted.

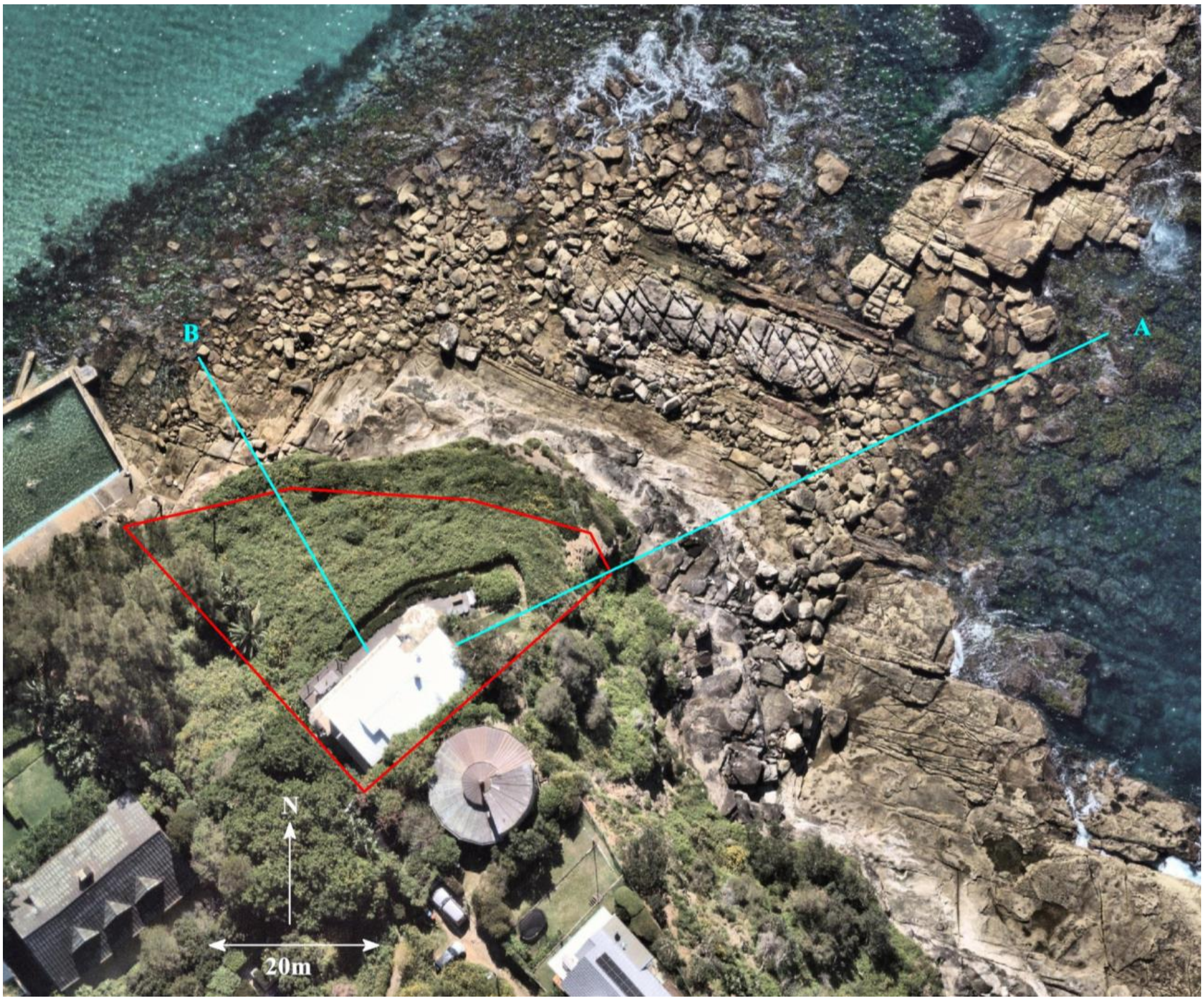

Figure 1: Aerial view of site (approximate property outline in red), with location of Section A and Section B in cyan (aerial photograph taken 1 May 2023)

Coffey & Partners (1987) noted that the cliff profiles from Little Head to south Palm Beach have been formed by an interbedded sequence of sandstone and interbedded siltstone/sandstone. An oblique aerial view of the site and adjacent rock platform is provided in Figure 2, with a photograph of the cliffs at the site (taken from the adjacent rock platform) provided in Figure 3 and Figure 4. It is evident that columns have been constructed to support an overhanging rock section near the toe of the NE facing cliff.

Figure 2: Oblique aerial view of site (at arrow) on 13 October 2022, facing SW

Figure 3: View of NE facing cliff at site (at arrow) on 30 November 2022, facing SW

Figure 4: View of NW facing cliff at site (at arrow) on 30 November 2022, facing south

Based on Airborne Laser Scanning (ALS) data captured by the NSW Government in 2020, elevations along Section A and Section B (from Figure 1) perpendicular to the cliff face are depicted in Figure 5 and Figure 6 respectively.

Ground elevations along Section A vary from about 26.1m AHD at the top of the cliff and 5.5m AHD at the base of the steeper upper section of cliff (with an average slope of about 60° between these levels, but steeper sections at about 80°), with the landward edge of the rock platform at about 2.5m AHD.

Ground elevations along Section B vary from about 22.4m AHD at the top of the cliff and 2.1m AHD at the toe of the cliff, with an average slope of 47° between these levels.

4. PROPOSED DEVELOPMENT

It is proposed to demolish and rebuild the dwelling at the site, with the new dwelling having two levels. The minimum habitable floor level of the proposed dwelling is 25.4m AHD, with a pool also proposed at that level. An outline of the proposed development is provided in Figure 7, with the seaward edge of the proposed pool also depicted in Figure 5.

To the south of the pool, a pool store is proposed, with stairs extending above the store to the upper level, as depicted in Figure 7.

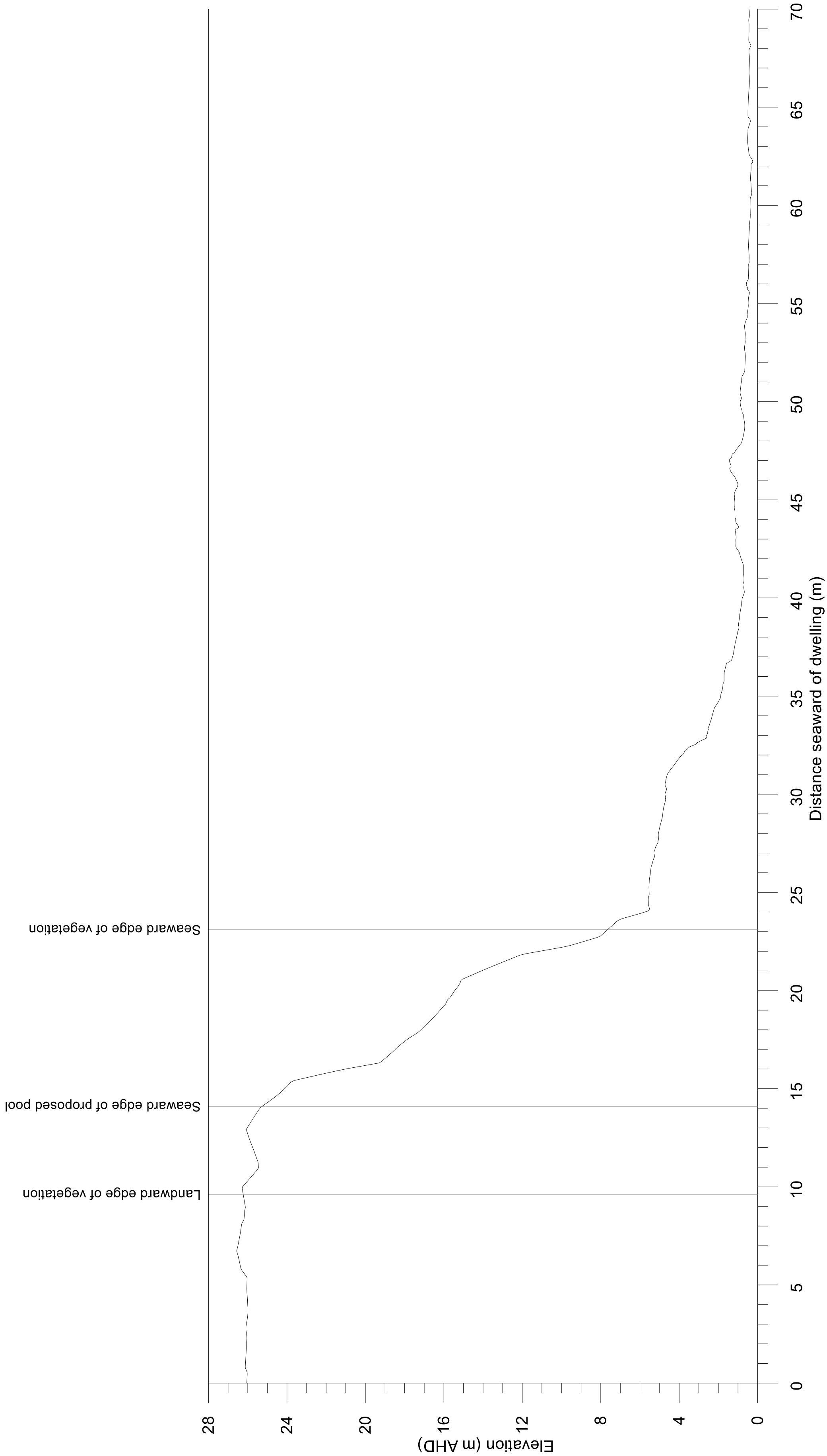

Figure 5: Section A through cliff at site towards ENE

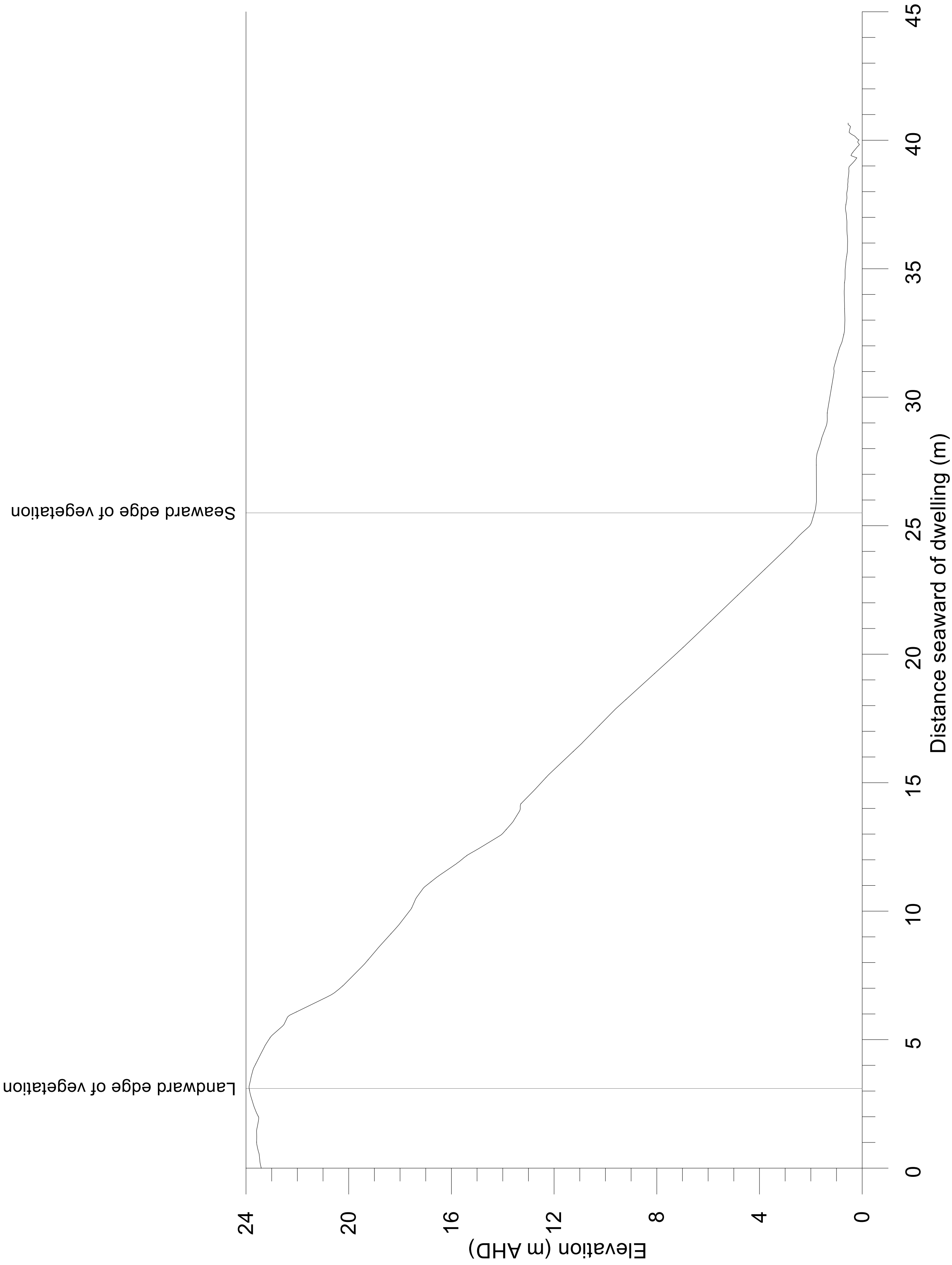

Figure 6: Section B through cliff at site towards NNW

Figure 7: Outline of proposed development with approximate boundary in red, ground floor in solid yellow (with terrace outer edge dashed), pool in blue, and stairs and pool store in green (aerial photograph taken 1 May 2023)

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 is highest near the base of the cliff or at other levels below the top of the cliff. 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.

The discussion below is focussed on the NE cliff face at the site. The cliff facing NW is flatter, probably has a soil cover over much of the slope, is less exposed to wave action (as it faces away from the predominant S to SE storm waves offshore of Sydney), and with slope stability likely to be more controlled by geotechnical rather than coastal processes. The NW cliff face should be assessed by the geotechnical engineer with consideration of slope stability issues, noting that the toe of this slope could be subject to erosion of soil due to wave action at times[2].

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), noting that they determined corresponding maximum rates of 12mm/year. An apparent approximate 50m of cliff recession (observed in aerial photography as the distance of the toe of the steep portion of the cliff from the seaward edge of the rock platform at present) on the NE side of the site over the last 6,400 years (since sea levels stabilised around their present levels, and assuming that the cliff was at the seaward edge of the rock platform at that time) represents an average recession rate of 8mm/year, consistent with these rates[3].

The steeper section of NE cliff at the site, above about 5.5m AHD, is well above the intertidal zone (above 1m AHD), but would occasionally be impacted by wave runup. This elevated wave runup would occur 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 mechanical weathering would apply at the NE cliff face. A recession/weathering rate of 8mm per year is considered to be appropriate, with sensitivity testing for a rate of 12mm/year as a conservative 1.5 multiple rate increase to account for sea level rise[4]. These rates should be considered and assessed by the geotechnical engineer. The 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 NE cliff face, although in reality it would be expected

[2] As evident in Figure 4, the toe of the NW slope does have a rim of rock. However, the soil above is considered to have greater propensity to erode and slump and to affect overall slope stability, to be confirmed by the geotechnical engineer.
[3] There was a conservative interpretation of aerial photography and the cliff cross section to derive the 50m of recession. About 30m of recession is more likely, representing an average rate of 5mm/year.
[4] There are no established methods to estimate increased recession rates of cliff lines due to sea level rise, but a 1.5 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.

that runup would generally be below 9m AHD in a severe coastal storm over the 100-year design life. Therefore, an allowance for recession/weathering of the NE cliff face of about 8mm to 12mm per year should be considered and assessed by the geotechnical engineer[5].

The geotechnical engineer should consider these estimated rates in conjunction with an understanding of the particular nature of the cliff materials on the NE side of the site, their resistance to erosion/recession, and potential failure planes related to geotechnical issues such as the joint spacing[6].

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 NE cliff face over the long term, with failure planes at the joints. That stated, any future failure of the upper slope of the cliff 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 minimum floor levels above 25m AHD, coastal inundation is not a significant risk for the proposed development over a planning period of well over 100 years, including consideration of projected sea level rise.

7. MERIT ASSESSMENT

7.1 Preamble

The merit assessment herein has been undertaken assuming that the geotechnical engineer will find that the proposed development is at an acceptably low risk of damage from coastal erosion/recession of the cliff at and seaward of the site, and other processes, for a design life of at least 100 years[7].

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

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[5] Note that this does not mean that the cliff face is predicted to recede at a steady rate of 8 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 8mm to 12mm/year (which can also be stated as 0.8m to 1.2m per 100 years) is expected.
[6] Coffey & Partners (1987) noted that the controlling feature of interbedded sandstone/siltstone cliffs was the bedding spacing and relative proportion of sandstone/siltstone.
[7] At a site with underlying bedrock such as the subject property, 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*.

has considered whether the proposed development is likely to cause an adverse impact on the following:

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

This is not a coastal engineering matter, but it can be noted that with regard to (a), the proposed development would not be expected to adversely affect the biophysical and hydrological (surface and groundwater) environments, being in an existing developed area and with conventional stormwater management features (such as a rainwater tank, piped drainage, and a dispersion system to be coordinated with the geotechnical engineer). With regard to ecological environments, an 'Arboricultural Impact Assessment and Management Plan' has been prepared by Botanics Tree People Pty Ltd (2023), with recommendations provided on tree removal and retention.

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.

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

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

With regard to (f), a search of the Heritage NSW "Aboriginal Heritage Information Management System" (AHIMS) was undertaken on 14 June 2023. This resulted in one Aboriginal site and no Aboriginal places being recorded or declared within at least 50m of the site. This Aboriginal site is to the north of the subject property at the base of the cliff, and would not be expected to be affected by the proposed development.

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), Aboriginal matters were discussed 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 the rock pool at the southern end of the Palm Beach ocean beach. This heritage item is located about 30m from the proposed development, at the base of the cliff. The proposed development would not be expected to impact on this heritage item.

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 Clause 7.5 of *Pittwater Local Environmental Plan 2014*

Clause 7.5 of *Pittwater Local Environmental Plan 2014* (LEP 2014) applies at the site, as the property 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.

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 8mm/year of the NE cliff at 18 Rock Bath Road Palm Beach, 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 on the NE side of 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 may be unrelated to coastal processes at the base of the cliff, so other failure mechanisms should be considered by the geotechnical engineer.

The NW cliff face should be assessed by the geotechnical engineer with consideration of slope stability issues, noting that the toe of this slope could be subject to erosion of soil due to wave action at times.

Coastal inundation is not a significant risk for 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, and other processes, 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) and Clause 7.5 of *Pittwater Local Environmental Plan 2014* for the matters considered herein.

10. REFERENCES

Botanics Tree People Pty Ltd (2023), *Arboricultural Impact Assessment and Management Plan, 18 Rockbath* (sic) *Road, Palm Beach*, May

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
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 Drew and Bridget Hall
Name of Applicant
Address of site 18 Rock Bath Road Palm Beach

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 16 July 2023 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 Rock Bath Road Palm Beach

Report Date: 16 July 2023
:
Author: Peter Horton

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

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

See Section 2 and Section 10 of coastal report

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

Signature ………………………………………… 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