

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

Development Application for Jennifer Hempton
Name of Applicant
Address of site 89 Marine Parade, Avalon 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 27 January 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:

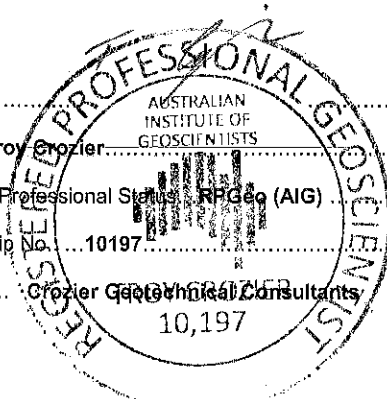
Report Title: Geotechnical Report for Proposed New Dwelling at 89 Marine Parade, Avalon Beach, NSW
Report Date: 30th November 2023 **Project No.:** 2021-159
Author: Marvin Lujan, Josh Cotton
Author's Company/Organisation: Crozier Geotechnical Consultants

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

Architectural Plans: Annabelle Chapman Architect Pty Ltd, Project No.: 2201, Date: 21.11.2023, Stage: DA, Dwg. No: 000 to 002, 101 to 105, 151 and 152, 200 to 204, 301 to 306, 401 to 406, 601 and 602, 701, 901 to 903.
Survey Plan - C.M.S Surveyors Pty Limited, Drawing Name: 19500Bdetail, Dated: 25/10/2023
Landscape Drawings - Contour Landscape Architecture, Drawing No.: C1 - C8, Rev.: E, Dated: 16/11/2023
Coastal Engineering Report: Horton Coastal Engineering, Dated: 28/09/2023

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 <u>Jennifer Hempton</u>	Name of Applicant
Address of site <u>89 Marine Parade, Avalon Beach, NSW</u>	

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:

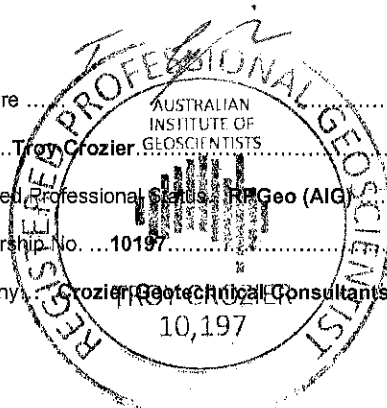
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Author: Marvin Lujan, Josh Cotton	
Author's Company/Organisation: Crozier Geotechnical Consultants	

Please mark appropriate box

- ☒ Comprehensive site mapping conducted 9th August 2021, 21st March 2023
- ☒ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
- ☒ Subsurface investigation required
 - ☐ No Justification minor works only.
 - ☒ Yes Date conducted9th August 2021, 21st March 2023.....
- ☒ 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: RM Geo (AIG).....
 Membership No.10197.....
 Company: Crozier Geotechnical Consultants



**REPORT ON GEOTECHNICAL SITE INVESTIGATION
AND RISK ASSESSMENT
FOR PROPOSED NEW DWELLING**

at

89 MARINE PARADE, AVALON, NSW

Prepared For

Jennifer Hempton

Project No.: 2021-159

November 2023

Document Revision Record

Issue No	Date	Details of Revisions
0	6 th October 2021	Report
1	28 th March 2023	Updated Architectural Design
2	30 th November 2023	Updated Architectural Design

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5	Hillside Construction Guidelines
6	Coastal Engineers Report

Date: 30th November 2023

Project No: 2021-159

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**GEOTECHNICAL REPORT FOR PROPOSED NEW DWELLING
AT 89 MARINE PARADE, AVALON, NSW**

1. INTRODUCTION:

This report details the results of geotechnical investigation and assessment undertaken for a proposed new dwelling at 89 Marine Parade, Avalon, NSW. An investigation was undertaken by Crozier Geotechnical Consultants (CGC) on behalf of the client Jennifer Hempton. The original report has been updated for DA Submission at the request of Annabelle Chapman Architect Pty Ltd and Horton Coastal Engineering Pty Ltd on behalf of the client.

The site is situated on the high eastern side of Marine Parade within moderately west dipping topography with the rear of the block extending to the crest of an approximately 30m high sea cliff. The site is currently occupied by a one and two storey sandstone and clad residence on the rear eastern half. The western portion of the site contains a long driveway in between gardens which contains large trees ($\leq 10\text{m}$ high) and dense vegetation along the south boundary.

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). This report has been prepared to meet the Council Policy Requirements including Paragraph 6.5 and assess the landslip risk to ensure ‘Acceptable’ risk levels are achieved and can be maintained for the preferred design life of 100 years.

The site is also classified under the coastal hazard zone ‘R’ (Bluff/ Cliff instability), CHZ_016, with a Coastal Engineering Report in line with Council requirements provided on 28 September 2023 by Horton Coastal Engineering Pty Ltd.

The site is located within Acid Sulphate Soils Class 5 (ASS_016), however due to the elevation ($> \text{R.L. } 35.00\text{m}$) of the site there will be no possibility of intersecting these soils, whilst lowering of the groundwater table in Class 1-4 sites to the west is unrealistic for the proposed residential development on this site.

This report includes a description of site and sub-surface conditions, a geotechnical assessment of the development, site mapping/plan and provides recommendations for construction and potential impact on neighbouring properties.

This report includes a plan of the geotechnical investigation test locations and mapping of the site conditions, geological sections/models and provides recommendations for design and construction, this report will be suitable for Council submission as part of the Development Application process.

The initial investigation and reporting were undertaken as per the Proposal P21-258, Dated: 31st May 2021, with additional geotechnical testing and report updating occurring in March 2023 and most recently November 2023.

The investigations comprised:

- a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Geotechnical Engineer.
- b) Drilling of six boreholes using hand tools along with seven Dynamic Cone Penetrometer testing (DCP) to investigate the subsurface geology, depth to bedrock and identification of ground water.
- c) Photographic record of site conditions

The following plans and drawings were supplied for the work:

- Survey Drawing – C.M.S Surveyors Pty Limited, Drawing Name: 19500Bdetail, Dated: 25/10/2023
- Architectural Drawings – by Annabelle Chapman Architect Pty Ltd, Project No.: 2201, Date: 21.11.2023, Stage: DA, Dwg. No: 000 to 002, 101 to 105, 151 and 152, 200 to 204, 301 to 306, 401 to 406, 601 and 602, 701, 901 to 903.
- Coastal Engineering Report – Horton Coastal Engineering Pty Ltd, Dated: 28/09/2023
- Landscape Drawings – Contour Landscape Architecture, Drawing No.: C1 – C8, Rev.: E, Dated: 16/11/2023

2. PROPOSED WORKS:

The proposed works involve the demolition of existing site structures and the construction of a new residential dwelling which will comprise three storeys to the front and one storey to the rear. The proposed works will also include the installation of an inground swimming pool within the front of the block, a rear terrace/decking area, landscaping works and an inground OSD tank and rainwater tank below the proposed Basement Level (BL).

The proposed structure will comprise a BL at FFL 31.63, Lower Ground Level (LGL) at FFL 35.13 and Upper Ground Level (UGL) at FFL 38.28. The OSD tank and rainwater tank will be located below the southwestern corner of the BL at FFL 29.73.

Bulk excavation will be required across the majority of the building footprint. It appears that bulk excavation for the BL/LGL within broadly western portions will be required to approximately 3.0m depth and will extend to between approximately 5.7m (southern side) and 4.3m depth (northern side) along the eastern excavation face. Additional bulk excavation will be required within the southwestern corner of the BL to approximately 5.0m depth for the OSD and rainwater tank. The proposed swimming pool will also require bulk excavation to approximately 2.0m depth.

The proposed structure will occupy a southern side setback of 1.0m, a northern side setback of 2.5m (BL & LGL) – 1.0m (UGL) and minimum rear eastern setbacks between 12.18m (BL), 6.5m (LGL & UGL) and approximately 4.0m (UGL rear ocean deck).

3. SITE FEATURES:

3.1. Description:

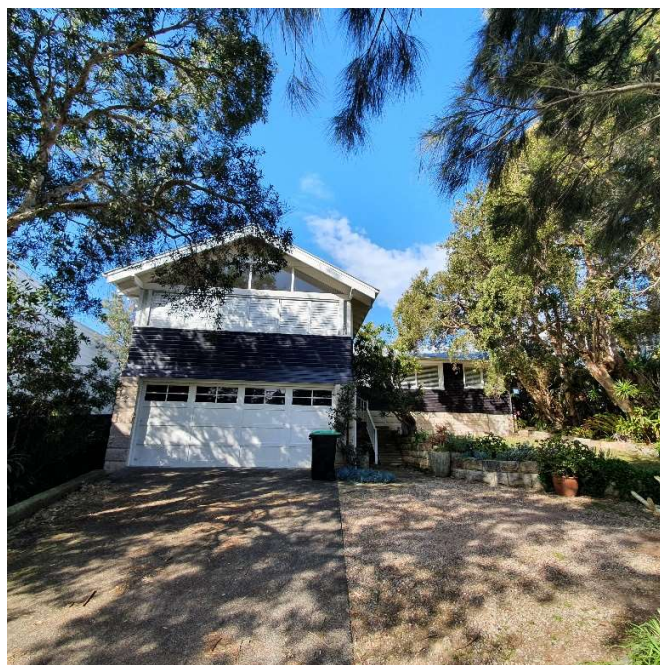
The site is a rectangular shaped block located on the high eastern side of Marine Parade, on a moderately west dipping slope. It has a front west boundary of 18.315m, rear east boundary of approximately 18.8m, north side boundary of 54.92m and south side boundary of 58.25m as referenced from the provided survey plan.

The site has a high of approximately RL 38.50m at the south-east corner of the block and a low of approximately RL 31.50m within the western front of the block.

An aerial photograph of the site and its surrounds is provided below, as sourced from NSW Government Six Map spatial data (Photograph-1). General views of the site are shown in Photograph-2 to Photograph-5.



Photograph-1: Aerial photo of site and surrounds



Photograph-2: Site-dwelling frontage. View looking east.



Photograph-3: Rear view of the site. View looking north.

3.2. Geology:

Reference to the Sydney 1:100,000 Geological Series sheet indicates that the site is underlain by Newport Formation (Upper Narrabeen Group) rock 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 below the site (Photograph-5) and in the Bangalley Head cliff face located to the north of the site (Photograph-4).



Extract of Sydney 1:100 000 – Geology underlying the site



Photograph-4: Bangalley Head cliff face located to the north of the site.



Photograph-5: Full cliff face located below the site. View looking south from Bangalley Park estimated height $\geq 25\text{m}$

4. FIELD WORK:

4.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and limited inspection of adjacent properties on the 9th August 2021 and on the 21st March 2023 by a Geotechnical Engineer. It included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of vegetation, existing structures along with further examination of the cliff face below the site viewed from Bangelley Park located to the north of the site.

The investigation also included the drilling of six auger boreholes (BH1, BH2, BH3 and BH4) using a hand auger due to site access limitations.

Dynamic Cone Penetrometer (DCP) testing was carried out adjacent to/within the boreholes and at one separate location (DCP7) in accordance with AS1289.6.3.2 – 1997, “Determination of the penetration resistance of a soil – 9kg Dynamic Cone Penetrometer” to estimate near surface soil conditions and confirm depths to bedrock.

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/sections are provided as Figure: 2 and Figure: 3, Appendix: 2.

4.2. Field Observations:

Marine Parade contains a bitumen pavement and is gently north dipping topography with low concrete gutter and kerbs along the sides where it passes the site, with a bitumen crossover driveway where vehicular access to the site is possible. There were no signs of excessive cracking or deformation within the road pavement to suggest any movement.

The western portion of the site contains a long bitumen driveway with a grass lawn at similar level to the north and a densely vegetated garden to the south containing large trees (up to ≤ 10 m high). Signs of movement, deep-seated landslip instability or underlying geotechnical issues were not observed within the western portion of the site.

The eastern portion of the site contains a one and two storey sandstone block and cladding dwelling with a garage below the north-western corner of the structure. Observation below the existing Ground Floor (GF) identified that the dwelling is supported by brick columns/walls (Photograph-6) and sandstone block walls. Bedrock outcrop was not identified below the dwelling. Whilst footings and foundations could not be viewed,

the supporting structures appeared in good condition. Signs of cracking, differential settlement or underlying geotechnical issues were not observed within the site-dwelling.



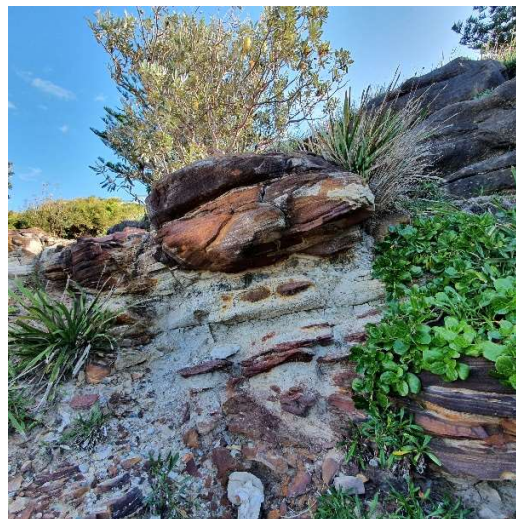
Photograph-6: Brick columns and walls supporting the site-dwelling. View looking east from below the site-dwelling.

Access to the rear of the site is achieved via narrow ($\leq 1.0\text{m}$ wide) concrete and sandstone paved pathways along the southern and northern sides of the site-dwelling, respectively. Directly to the rear of the dwelling is a timber deck that extends approximately 3.5m to 6.0m east (RL= 37.89m) (Photograph-3). Between the eastern end of the timber deck and the site boundary is a distance of approximately 1.0m to 3.0m within a gently east to north dipping slope covered in dense vegetation. It was not possible to determine the distance between the eastern boundary and the crest of the cliff face due to the very dense vegetation and safety concerns. However, based on visual inspection, satellite imagery and reference to the Coastal Engineering report it is estimated that the crest of the cliff is as at least 6m east of the rear site boundary. Signs of ground movement, deep seated landslip instability or tension cracking was not observed within the rear of the site.

Observation of the cliff face from Bangelly Park identified an iron rich sandy clay soil near surface with sandstone cobbles/boulders, conglomerate, underlaid by sub-horizontally bedded grey shale with a few bands of lithic sandstone (Photograph-7 and Photograph-8).



Photograph-7: Iron rich sandy clay unit observed under Bangelly Park. View looking south.



Photograph-8: Sandstone cobbles/boulders and conglomerate under Bangelly Park. View looking south.

Observations of the cliff face below the site identified generally sub-horizontally bedded siltstone and sandstone with an undercut which extended west to a distance unable to be confirmed from the point of observation (Photograph-5) however it is estimated at $\leq 3\text{m}$ laterally between cliff crest and base.

The neighbouring property to the north (No. 91 Marine Parade) contains a one and two storey clad house broadly occupying the centre to eastern portion of the block with a swimming pool within the northern side. The western portion of the block contains a concrete driveway and a grass lawn within the southern and northern sides, respectively. The eastern rear of the block contains a grass lawn. The property dwelling extends south to approximately 2.0m from the common boundary. The Ground Surface Level (GSL) of the neighbouring property is approximately 1.0m lower than the site along the common boundary. The property dwelling is at least 15 years old (based on available Google Earth Pro aerial view) and appeared in good condition. Signs of ground movement, deep seated instability or underlying geotechnical issues were not observed within the neighbouring property.

The neighbouring property to the south east (No. 87A Marine Parade) is a battle axe block and contains a two storey rendered house which broadly occupied the centre of the main block. Entrance to the property is achieved via a long strip concrete driveway along the south boundary. The eastern rear of the block contains a grass lawn that extends east to the crest of the cliff face. The property dwelling extends north to approximately $\leq 2.0\text{m}$ from the common boundary. The property contains gardens adjacent to the common boundary and is at a similar GSL to the site along the common boundary.

The neighbouring property to the south-west (No.87 Marine Parade) is located directly to the west of the neighbouring property No.87A Marine Parade. The property contains a one and two storey rendered house broadly occupying the centre to eastern portion of the block. The western portion of the block contains a grass lawn and an entry sandstone paved pathway within the southern and northern sides, respectively. The property dwelling extends north to approximately $\leq 2.0\text{m}$ from the common boundary. The property is at similar GSL to the site along the common boundary.

The neighbouring properties to the south appeared in good condition. Signs of cracking, ground deformation, deep seated instability or underlying geotechnical issues were not observed within the neighbouring properties.

4.3. Field Testing:

The boreholes (BH1 – BH6) were drilled within the site and refusal was encountered at varying depths between 0.70m (within fill, BH5) and at 1.20m depth (within bedrock, BH6) and 1.40m depth (within very stiff sandy clay).

Dynamic Cone Penetrometer (DCP) tests were carried out from ground surface adjacent to/within the boreholes with refusal encountered on interpreted bedrock at varying depths between 1.20m (DCP6 RL 35.0m) and 2.90m (DCP3 RL 34.8m).

For a detailed description at test locations the individual BH/DCP log sheets should be consulted. Based on the field borehole logs and DCP test results the subsurface conditions at the project site can be classified as follows:

- **TOPSOIL/FILL** – this layer was encountered across the site to a maximum depth of 0.90m below the existing ground surface. It was classified as loose to very dense, fine grained, dry/moist within BH1 and BH2 and moist within BH3 and BH4, silty sand with some tree roots and plant roots.
- **SANDY CLAY** – this layer was encountered below the fill to a maximum depth of 1.40m (BH2 and BH3), however it is interpreted based on the DCP tests to extend to varying depths between 1.80m (RL 32.3m) and 2.90m (RL 34.8m). It was classified as generally very stiff (DCP1 to DCP3) and stiff (DCP4) becoming hard below varying depths between 1.30m (DCP1) and 1.90m (DCP2). It was brown becoming iron rich orange brown generally below 1.0m depth, generally

low/medium plasticity (BH1, BH2 and BH3) and medium/high plasticity below 0.8m depth (BH4), dry/moist adjacent to the tree (BH1) and moist (BH2-BH4), sandy clay.

- **BEDROCK** - based on the DCP test results, interpreted bedrock was encountered at varying depths between 1.20m (DCP6, RL 35.0m) and 2.9m (DCP3, RL 34.8m). The bedrock surface appears to be gently west dipping and based on the cliff face outcrop it may comprise of shale/quartz to lithic quartz sandstone of at least low strength.

Significant water seepage was not encountered within the investigation.

5. COMMENTS:

5.1. Geotechnical Assessment:

The site investigation identified the presence of topsoil/uncontrolled fill to a maximum depth of 0.90m, underlain by sandy clay interpreted to extend to varying depths between 1.20m (RL 33.80m) and 2.90m (RL 34.80m) overlying bedrock. The presence of seepage was not encountered in the investigation, however the presence of seepage between the soil and bedrock unit is possible. No water table will be intersected.

The geotechnical inspection did not identify any signs of previous or impending large scale or deep-seated landslide instability within the site or the observed portions of the adjacent properties. There is a minor undercut on the cliff face and potential for instability in the upper cliff over time, but only with an estimated potential for instability extending to a limited distance (≤ 5 m) from the eastern end of the cliff.

The stability of the cliff face seaward of the site has been assessed with consideration of the potential for 0.6m to 1.2m of recession over the next 100 years (6mm to 12mm per year) as identified by Horton Coastal Engineering (2023). The extent of instability on the cliff face is likely to be minimal with negligible impact on the proposed works. However, considering that the proposed structure is generally located 6.50m from the eastern boundary and therefore approximately 12m from the crest of the rear cliff line, additional assessment/examination of the cliff line via the use of drone photography is strongly recommended to be undertaken by the geotechnical engineer.

The proposed works comprise of the demolition of the existing site structures and the construction of a new one to three storey dwelling. Bulk excavation will be required for the BL between 3.0m and 5.7m depth, inclusive of the OSD tank bulk excavation in the southwestern corner. The proposed bulk excavation will occupy a northern side setback of 2.5m, a southern side setback of 1.0m and a minimum rear setback of approximately 6.5m.

As per our investigation results, it appears that bedrock of at least very low strength underlies the approximate building footprint between 1.20m (DCP6) and 1.90m depth (DCP2 & DCP4), with the bedrock level dipping towards the rear boundary and cliff line (DCP3 & DCP7). Therefore, it is anticipated that bulk excavation will extend through fill, residual clay and sandstone/siltstone bedrock across the excavation footprint.

The fill and sandy clay and extremely weathered to very low strength bedrock can be excavated using conventional earthmoving equipment, however low to high strength sandstone bedrock will require the use of the rock breaking equipment (e.g. rock hammers). The use of rock hammers can create ground vibrations which could damage the neighbouring and adjacent structures. Care will be required during the demolition, construction and excavation works to ensure the neighbouring properties, structures and services (i.e. sewer) are not adversely impacted by ground vibrations. Small scale equipment (i.e. rock hammer <250kg) along with rock saw (or a rock grinder) and a good excavation methodology can be used to maintain low vibration levels and avoid the need for full time vibration monitoring. Crozier Geotechnical Consultants (CGC) should be consulted regarding the size and type of demolition/excavation equipment proposed and demolition/excavation methodology prior to works.

Based on the proposed excavation depths, it appears that the recommended safe batter slopes, as per Section 5.3.2, will not be feasible with exception of the front excavation face. Therefore, support prior to excavation will be required to ensure stability of excavation faces. Due to site location and geology, it is likely that the Narrabeen Group sandstone/siltstone bedrock unit which is anticipated to be intersected across the excavation will comprise relatively weak 'very low strength' bedrock. Therefore, it would be prudent to extend support systems through this unit to the base of the excavation.

Where support prior to excavation is required, the construction of a soldier pile (with a gap no greater than 1.0m between the external sides of the piles) is recommended. Where safe batter slopes are achieved, the construction of a steel reinforced concrete wall post excavation is a viable option.

Lateral support (if needed for the soldier pile wall) could be provided by internal bracing or propping to maintain all support internal to the site boundaries. Based on separation distances and geology conditions it appears cantilever systems are also viable.

Based on the relatively shallow depth to bedrock (from the BEL), it would be prudent to extend all footings to underlying bedrock of similar strengths to reduce the potential risk of differential settlement. Preliminary allowable bearing pressures appropriate for the sandstone encountered underlying the site are provided in

Section 5.3.1. This is likely to require a variation between shallow beam/strip and pile footings due to the excavation depth variations.

There is potential for boulders/floaters within the site due to the site location. It is recommended that the geotechnical inspection of the excavation is undertaken at 1.5m depth intervals or where conditions different to what is presented in this report are encountered. Where a boulder is encountered, it is recommended that the footing be extended through the boulder and down to bedrock of similar competency.

Due to the proximity of the proposed works to the rear boundary, it is recommended that the eastern cliff face be inspected by a professional geotechnical consultant using a drone to assess for any unfavorable defects, significant undercuts or potential for instability impacts. CGC can assist for this type of cliff face stability assessment and should be conducted prior to the Construction Certificate (CC).

Due to the increased bulk excavation proposed, relatively minor side setbacks and potential for the intersection of variable strength bedrock, it is recommended that CGC undertake an additional site investigation prior to the CC which will include the drilling of cored boreholes to below excavation depths. Cored boreholes will provide information on the quality, type and strength of the underlying bedrock unit used for design of support systems, footings and excavation methodology.

Signs of potential Acid Sulphate Soils (ASS) were not encountered in the investigation and based on the site location the presence of ASS is extremely low whilst the works will not encounter or impact a water table. Therefore, further investigation into ASS or a management plan will not be required for this DA.

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and hand tools. This test equipment provides limited data from small isolated test points across the entire site. Therefore, some minor variation to the interpreted sub-surface conditions is possible, especially between test locations.

5.2. Site Specific Risk Assessment:

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

- A. Landslip of soils from basement excavation ($\leq 3\text{m}^3$).
- B. Rock fall/topple from basement excavation ($\leq 10\text{m}^3$).
- C. Landslip of soil rock from eastern cliff line ($\geq 10\text{m}^3$)

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

The **Risk to Life** from the hazards were estimated to be up to 1.58×10^{-4} for a single person, whilst the **Risk to Property** was considered to be up to 'Moderate'.

Although the 'Moderate' Risk to Property for Hazard A & B is considered to be 'Unacceptable', the assessments were based on excavations with no support or planning. Provided the recommendations of this report are implemented including installation of retaining wall prior to bulk excavation the likelihood of any failure 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 'A' for footings on bedrock
Type of Footing	Strip/pad or Slab or Piers
Sub-grade material and Maximum Allowable Bearing Capacity	- Very Stiff Sandy Clay: 200kPa - Hard Sandy Clay: 400kPa - Weathered, VLS Bedrock: 800kPa - Weathered LS Bedrock: 1000kPa*
Site sub-soil classification as per <i>Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia</i>	C _e – Shallow soil site B _e – Rock Site, if structure situated atop bedrock as base of excavation
Remarks: *Requires confirmation via additional investigation including core drilling - All footings should be founded onto/within the same unit with similar strength to prevent differential settlement. - 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.	

5.3.2. Excavation:					
Basement and Lower Ground Level Excavation					
<i>Table 1: Property Separation Distances</i>					
Boundary	Adjacent Property	Structure	Bulk Excavation Depth (m bgl)	Separation Distances (m)	
				Boundary (m)	Structure
East	No property	Grass lawn	5.7m	6.5m	Grass lawn directly adjacent to the boundary extending to edge of the cliff approximately 6.0m further
West	Marine Parade	Grass lawn, Road pavement	2.7m – 5.7m	20.9m	Lawn directly adjacent to the boundary, road pavement another 6.0m west
North	No. 91 Marine Parade	Pathway lawn, Dwelling and Lawn	2.7m – 4.3m	2.5m	Pathway directly adjacent to the boundary, dwelling another 1.80m north.

South	No. 87A Marine Parade	Pathway and Dwelling	4.5m – 5.7m	1.0m	Pathway directly adjacent to the boundary, dwelling another 2.30m
	No. 87 Marine Parade	Pathway and Dwelling	3.0m - 5.5m	1.0m	Pathway directly adjacent to the boundary, dwelling another 2.30m

Swimming Pool Excavation

Table 2: Property Separation Distances

Boundary	Adjacent Property	Structure	Bulk Excavation Depth (m bgl)	Separation Distances (m)	
				Boundary (m)	Structure
North	No. 91 Marine Parade	Carport	2.0m	1.0m	Pathway and carport 1.0m from boundary, house another 1.0m
West	Marine Parade	Grass lawn, Road pavement	2.0m	10m	Lawn directly adjacent to the boundary, road pavement another 6.0m west

Type of Material to be Excavated	Layers of fill/sandy clay $\leq 1.90\text{m}$
	Sandstone/siltstone bedrock (VLS – LS) below $\leq 1.90\text{m}$ down to the BEL

Guidelines for batter slopes for general information are tabulated below:

Material	Safe Batter Slope (H:V)	
	Short Term/ Temporary	Long Term/ Permanent
Fill & Sandy Clay	1.5:1.0	2.0:1.0
Clay/Sandy Clay and ELS bedrock	1.0:1.0	2.0:1.0
Very Low to Low strength or fractured bedrock	0.5:1.0	1.5:1.0*
Medium Strength (MS), defect free bedrock	Vertical	0.25:1.0

*Dependent on assessment by engineering geologist.

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

Equipment for Excavation	Fill and natural soils	Excavator with bucket
	ELS/VLS bedrock	Excavator with bucket and ripper
	LS-HS bedrock	Rock hammer and saw
ELS – extremely low strength, VLS – very low strength, LS – low strength, MS – medium strength, 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:

<u>Buffer Distance from Structure</u>	<u>Maximum Hammer Weight</u>
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 ≥ 250 kg 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 if required for detailed assessment and contract costing requires cored boreholes using specialist restricted access drilling equipment unless demolition of existing structures can occur prior to final design.

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))	Road Reserve = 5mm/s Adjacent residential dwellings = 5mm/s SW Service Lines: Maximum PPV for intermittent vibrations = 10mm/s Maximum PPV for continuous vibrations = 5mm/s
Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	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:

	• During installation of pre-excavation support • For assessment of excavation of batter slopes, • Calibration test prior to bedrock excavation, • At completion of the excavation, • During construction of new footings.
Dilapidation Surveys Requirement	Recommended on neighbouring structures or parts thereof within 10m of the excavation perimeter prior to site work to allow assessment of the recommended vibration limit and protect the client against spurious claims of damage.
Remarks: Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope.	

5.3.3. Retaining Structures:					
Required	New retaining structures will be required as part of the works.				
Types	Support prior to excavation will be required where safe batter slopes are not feasible. The safe batter slopes appear achievable along the front western excavation face, therefore construction of support post excavation can be implemented. Where support prior to excavation is required, the construction of a soldier pile wall is a viable option. Where support post excavation is required, the construction of steel reinforced concrete walls or conventional gravity walls as per AS4678 is recommended.				
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 (K0)	
Fill	18	$\phi' = 25^\circ$	0.35	0.52	N/A
Clay (Very stiff to hard)	20	$\phi' = 35^\circ$	0.27	0.50	N/A
ELS to VLS bedrock	22	$\phi' = 38^\circ$	0.15	0.20	200 kPa
LS to MS bedrock	23	$\phi' = 40^\circ$	0.05	0.10	400kPa

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

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

5.3.4. Drainage and Hydrogeology

Groundwater Table or Seepage identified in Investigation		No
Excavation likely to intersect	Water Table	No
	Seepage	Minor (≤ 1 L/min), on defects and at soil/rock interface
Site Location and Topography		High eastern side of the road, within gently west sloping topography
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Not recommended

Remarks: Any excavation faces are expected to encounter some seepage, an excavation trench should be installed at the base of excavation cuts to below floor slab levels to reduce the risk of resulting dampness issues. Trenches, as well as all new building gutters, 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. Review and approve the structural drawings for compliance with the recommendations of this report,
2. Inspect all new footings and earthworks as per Section 5.2 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.
3. CGC also strongly recommend undertaking a drone photography examination of the rear cliff line along with a core drilling investigation to below excavation levels, prior to CC.

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. It is considered that the existing house will have a design life of 50 years from its upgrade following the proposed works.

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 Pittwater Council will control development on neighbouring properties, carry out regular inspections and maintenance of the road verge, stormwater systems and large trees on public land adjacent to the site so as to ensure that stability conditions do not deteriorate with potential increase in risk level to the site. Also individual Government Departments will maintain public utilities in the form of power lines, water and sewer mains to ensure they don't leak and increase either the local groundwater level or landslide potential.

It is considered that the proposed development is at an acceptably low risk of damage from erosion/weathering of the cliff seaward of the site, and other processes, for a design life of at least 100 years. This was the assumption of Horton Coastal Engineering (2023) as a preamble to the merit assessment of the works that they completed.

6. CONCLUSION:

The site investigation identified the presence topsoil/fill underlaid by sandy clay, overlying interpreted bedrock at varying depths between 1.20m (BH6 RL 35.00m) and 2.90m (DCP3, RL 34.80m). The presence of seepage was not encountered in the investigation however the presence of minor seepage within the soil and bedrock interface is possible.

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

It is recommended that support systems prior to excavation are established for all excavation faces where safe batter slopes are not feasible. All footings should extend to bear onto bedrock of similar strength to avoid potential for differential settlement.

It is recommended that the eastern cliff face be examined via the use of drone footage/photography to assess for any unfavourable defects. An additional core drilling investigation to below excavation depths is also recommended to assess bedrock condition and strength, prior to construction.

The presence of ASS was not encountered and not expected. Therefore, in line with the Acid Sulfate Soil Manual an ASS Management Plan will not be required.

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Reviewed by:



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Updated by:



Josh Cotton
Geotechnical Engineer

7. REFERENCES:

1. Australian Standard AS 2870 – 1996, Residential Slabs and Footings – Construction
2. Australian Standard AS1170.4 – 2007, Part 4: Earthquake actions in Australia
3. C. W. Fetter 1995, “Applied Hydrology” by Prentice Hall. V. Gardiner & R. Dackombe 1983, “Geomorphological Field Manual” by George Allen & Unwin.

Appendix 1

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:

<u>Soil Classification</u>	<u>Particle Size</u>
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:

<u>Classification</u>	<u>Undrained Shear Strength kPa</u>
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:

<u>Relative Density</u>	<u>SPT</u> "N" Value (blows/300mm)	<u>CPT</u> 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 separate 130mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by 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: -

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

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.

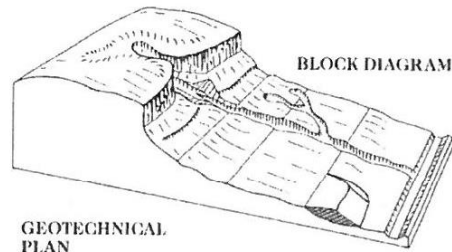
Reproduction of Information for Contractual Purposes

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.

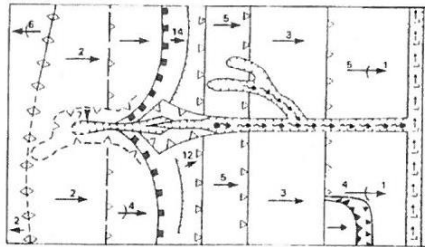
Site Inspection

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.

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007



GEOTECHNICAL
PLAN



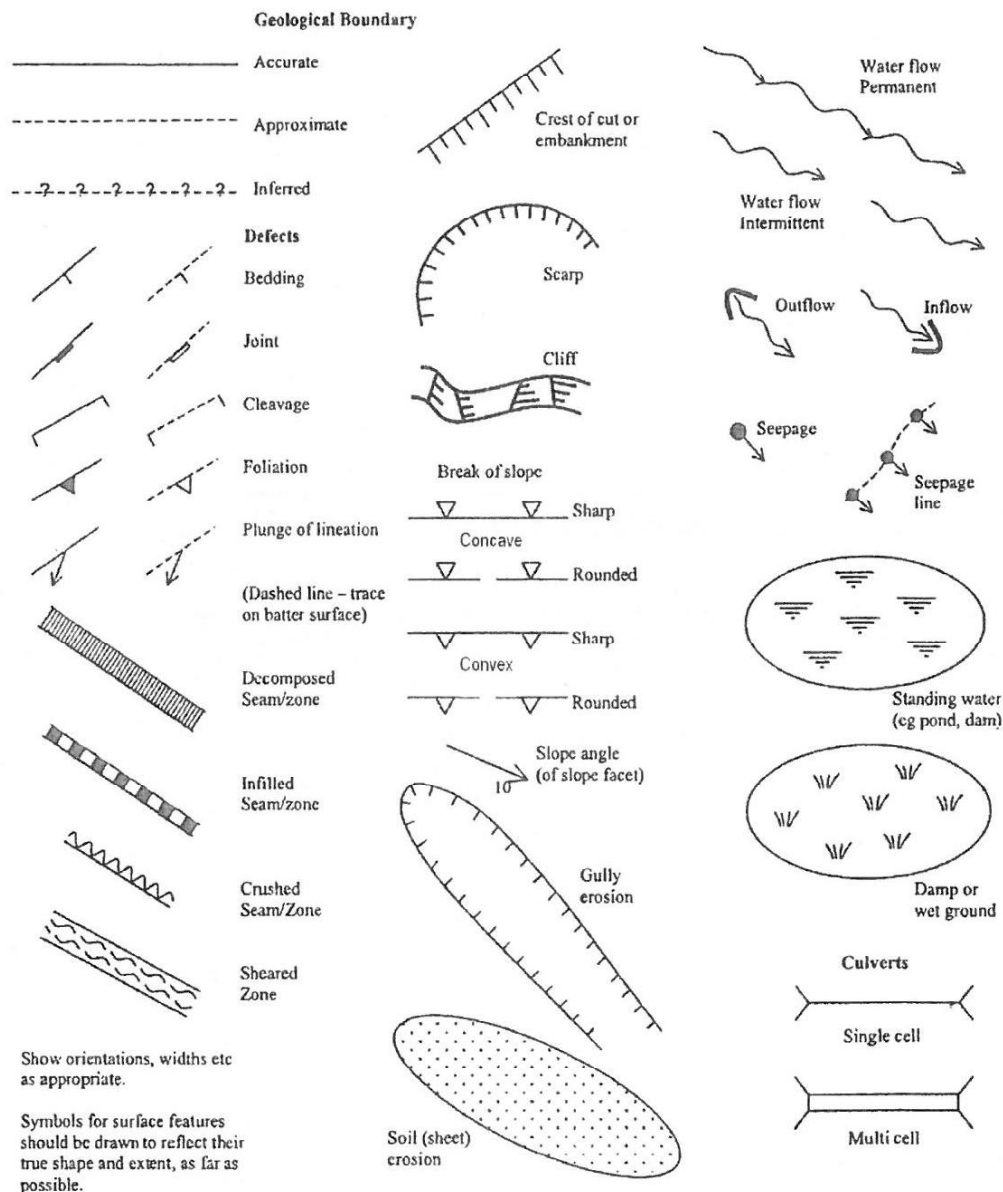
SYMBOL	GROUND PROFILE	
		Convex
		Concave
		Convex
		Concave
	Breaks of slope	} Convex and concave too close together to allow the use of separate symbols
	Changes of slope	
	Sharp	} Ridge crest
	Rounded	
	Cliff or escarpment or sharp break 40° or more (estimated height in metres)	
	Uniform slope	} Slope direction and angle (Degrees)
	Concave slope	
	Convex slope	
	Top	} Cut or fill slope, arrows pointing down slope
	Bottom	
	Hummocky or irregular ground	
	Open drain, unlined	
	Open drain, lined	
	Fence line	
	Property boundary	
		Dry stone wall
		Major joint in rock face (opening in millimetres)
		Tension crack (opening in millimetres)

Example of Mapping Symbols

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

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY

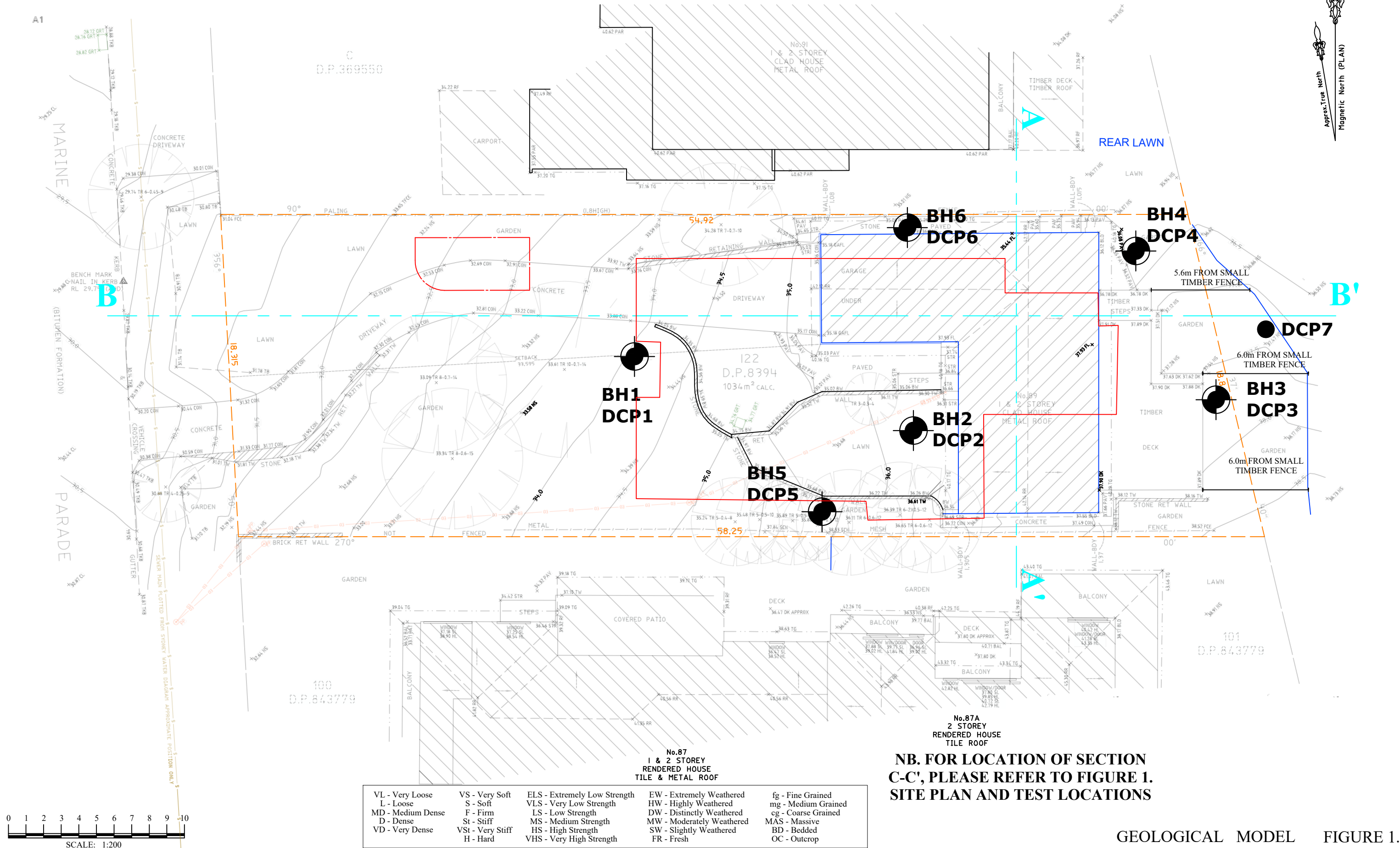
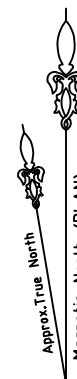


Examples of Mapping Symbols (after Guide to Slope Risk Analysis Version 3.1 November 2001, Roads and Traffic Authority of New South Wales).

Appendix 2

A1

C
D.P.369550



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

GEOLOGICAL MODEL FIGURE 1.



Crozier Geotechnical
Unit 12, 42-46 Wattle Road
Brookvale NSW 2100
Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

ABN: 96 113 453 624
Phone: (02) 9939 1882
Fax: (02) 9939 1883



AUGER /
DYNAMIC CONE
PENETROMETER
LOCATION

A—A' CROSS-SECTION
REFERENCE LINE



PROPERTY
BOUNDARY



PROPOSED
EXCAVATION

SCALE: 1:200 @ A3
DRAWING: Figure 1
DATE: 11/23

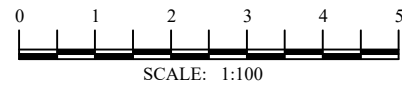
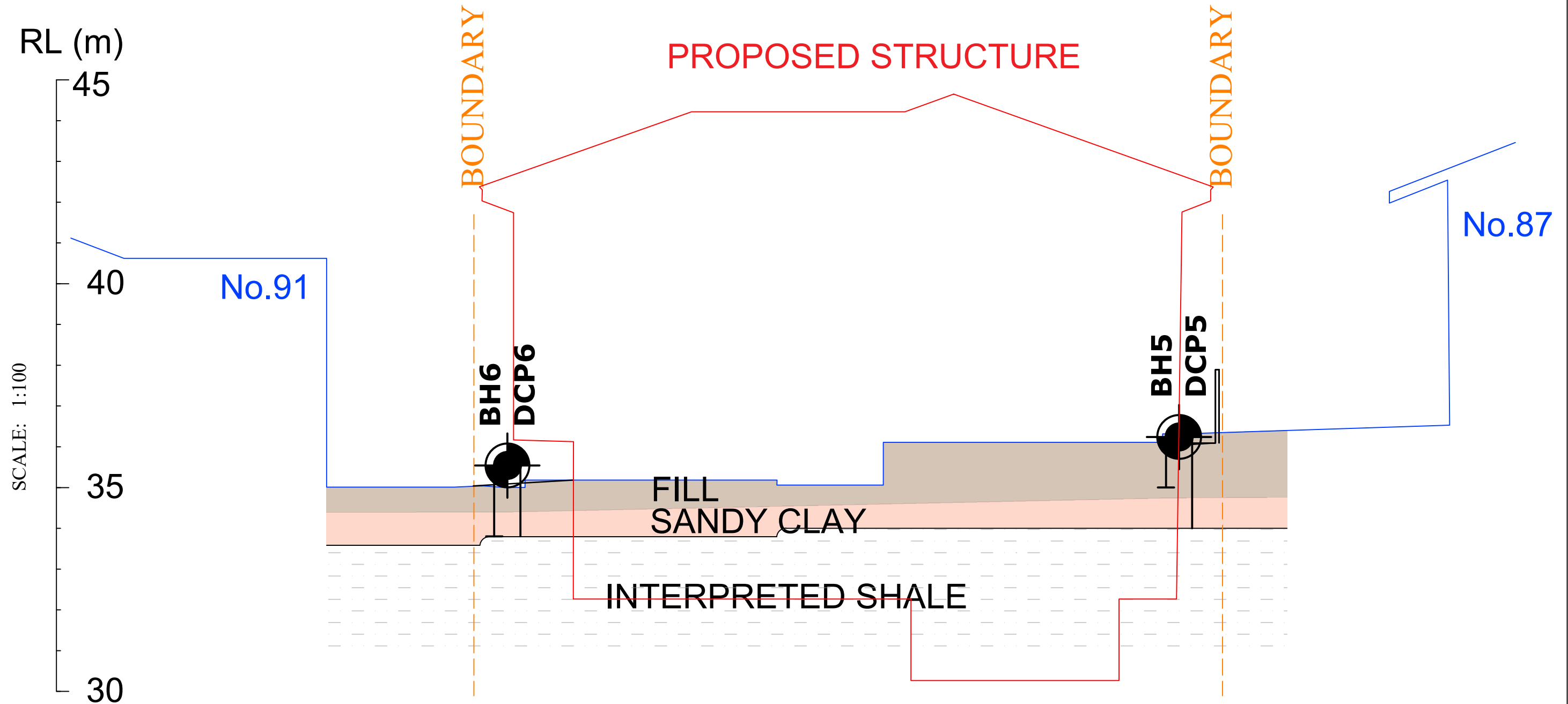
APPROVED BY: TMC
DRAWN BY: PS
PROJECT: 2021-159

PREPARED FOR:
JENNIFER HEMPTON

ADDRESS:
89 MARINE PARADE, AVALON, NSW

A
NORTH

A'
SOUTH



SCALE: 1:100

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

SITE PLAN & TEST LOCATIONS
FIGURE 2.



Crozier Geotechnical
Unit 12, 42-46 Wattle Road
Brookvale NSW 2100
Crozier Geotechnical is a division of PIC Geo-Engineering Pty Ltd

ABN: 96 113 453 624
Phone: (02) 9939 1882
Fax: (02) 9939 1883

LEGEND

A—A' CROSS-SECTION
REFERENCE LINE

SOIL/FILL

PROPERTY
BOUNDARY

CLAY

PROPOSED
EXCAVATION

AUGER /
DYNAMIC CONE
PENETROMETER
LOCATION

SHALE

SCALE: 1:100 @ A3
DRAWING: Figure 2
DATE: 11/2023

APPROVED BY: TMC
DRAWN BY: PS
PROJECT: 2021-159

PREPARED FOR:
JENNIFER HEMPTON

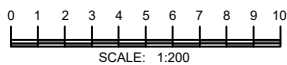
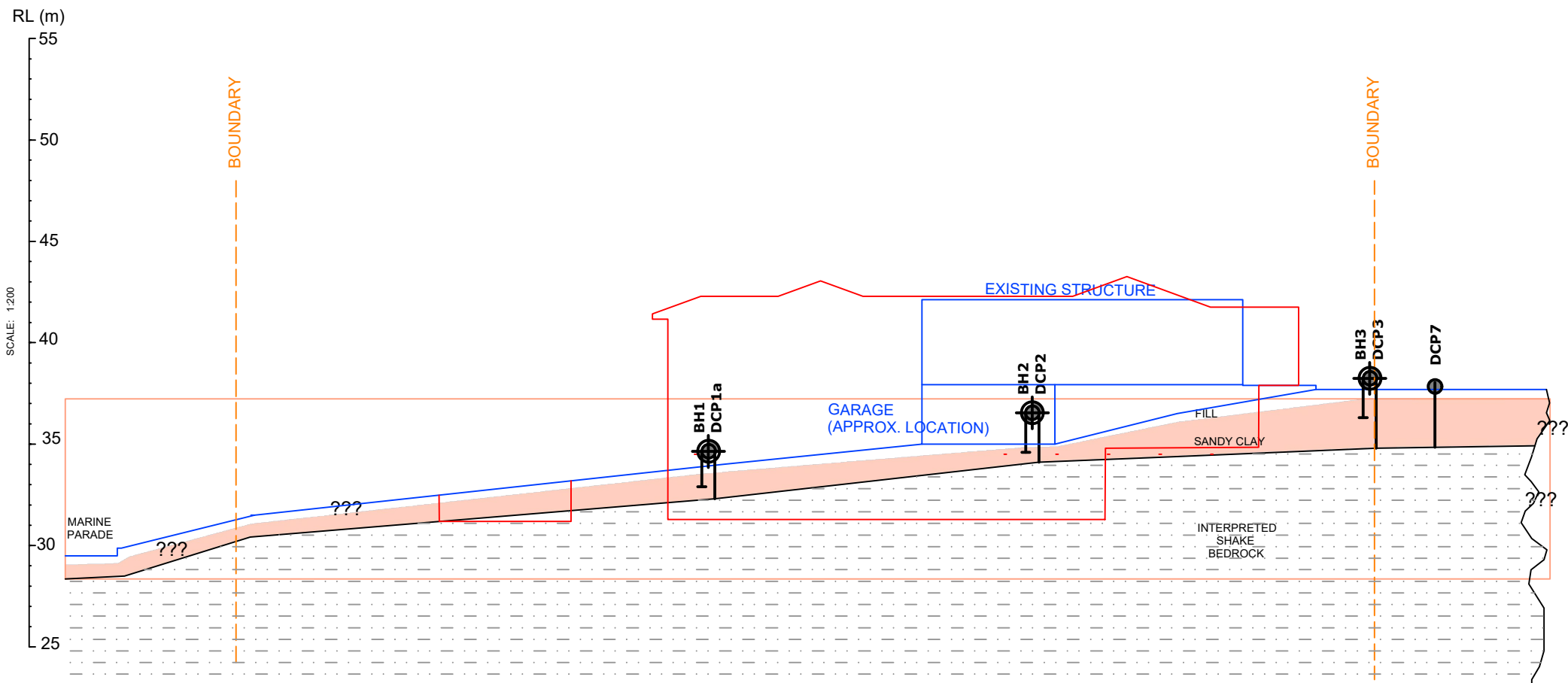
ADDRESS:
89 MARINE PARADE, AVALON, NSW

B

B'

WEST

EAST



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.**

LEGEND

A—A' CROSS-SECTION
REFERENCE LINE

PROPERTY
BOUNDARY

EXISTING
STRUCTURE

BH
DCP AUGER /
DYNAMIC CONE
PENETROMETER
LOCATION

PROPOSED
EXCAVATION

SOIL/FILL

SANDY CLAY

SHALE

SCALE: 1:200 @ A3
DRAWING: Figure 3
DATE: 11/23

APPROVED BY: TMC
DRAWN BY: PS
PROJECT: 2021-159

PREPARED FOR:
JENNIFER HEMPTON

ADDRESS:
89 MARINE PARADE,
AVALON, NSW

BOREHOLE LOG

CLIENT: Jennifer Hempton

DATE: 9/08/2021

BORE No.: 1

PROJECT: Geotechnical Investigation

PROJECT No.: 2021-159

SHEET: 1 of 1

LOCATION: 89 Marine Parade, Avalon

SURFACE LEVEL 34.1

RL(m):

Depth (m)	RL (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
				Type	Tests	Type	Results
0.00			TOPSOIL/FILL: Loose, dark brown, fine to medium grained, moist/dry, silty sand with some tree roots				
0.40	33.70		... becoming brown, silty sand, moist/dry				
0.60	33.50						
0.80	33.30	CL/CI	SANDY CLAY: Stiff, orange mottled brown, low to medium plasticity, moist/dry, sandy clay with trace of ironstone gravels and tree roots ... becoming orange red				
1.00							
1.20	32.90		... band of gravelly sandy clay				
			HAND AUGER REFUSAL at 1.2m depth within hard gravelly sandy clay				

RIG: None

DRILLER: AC

METHOD: Hand Auger

LOGGED: ML

GROUND WATER OBSERVATIONS:

REMARKS:

CHECKED: TMC

BOREHOLE LOG

CLIENT: Jennifer Hempton

DATE: 9/08/2021

BORE No.: 2

PROJECT: Geotechnical Investigation

PROJECT No.: 2021-159

SHEET: 1 of 1

LOCATION: 89 Marine Parade, Avalon

SURFACE LEVEL 36.0

RL(m):

Depth (m)	RL (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
				Type	Tests	Type	Results
0.00			TOPSOIL/FILL: Loose, dark brown, fine to medium grained, moist/dry, silty sand with some plant roots				
0.60	35.40		... becoming light brown, moist				
0.90	35.10						
1.00		CL/CI	SANDY CLAY: Stiff, orange brown, low to medium plasticity, moist, sandy clay				
1.10	34.90		... orange red				
1.40	34.60		... hard				
			HAND AUGER REFUSAL at 1.4m depth within hard sandy clay				
2.00							

RIG: None

DRILLER: AC

METHOD: Hand Auger

LOGGED: ML

GROUND WATER OBSERVATIONS:

REMARKS:

CHECKED: TMC

BOREHOLE LOG

CLIENT: Jennifer Hempton

DATE: 9/08/2021

BORE No.: 3

PROJECT: Geotechnical Investigation

PROJECT No.: 2021-159

SHEET: 1 of 1

LOCATION: 89 Marine Parade, Avalon

SURFACE LEVEL 37.7

RL(m):

Depth (m)	RL (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
				Type	Tests	Type	Results
0.00			TOPSOIL/FILL: Loose, dark brown, fine to medium grained, moist/dry, silty sand				
0.50	37.20	CI	SANDY CLAY: Very stiff, brown, medium plasticity, moist, sandy clay				
0.80	36.90	CI/CH	... orange brown, medium to high plasticity				
1.00	36.70		... orange red				
1.40	36.30		HAND AUGER REFUSAL at 1.4m depth on very stiff sandy clay				
1.90							
2.00							

RIG: None

DRILLER: AC

METHOD: Hand Auger

LOGGED: ML

GROUND WATER OBSERVATIONS:

REMARKS:

CHECKED: TMC

BOREHOLE LOG

CLIENT: Jennifer Hempton

DATE: 9/08/2021

BORE No.: 4

PROJECT: Geotechnical Investigation

PROJECT No.: 2021-159

SHEET: 1 of 1

LOCATION: 89 Marine Parade, Avalon

SURFACE LEVEL 36.5

RL(m):

Depth (m)	RL (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
				Type	Tests	Type	Results
0.00			TOPSOIL/FILL: Loose, dark brown, fine to medium grained, moist, silty sand				
0.80	35.70						
0.90	35.60	CL/CI	SANDY CLAY: Stiff, brown, low to medium plasticity, moist, sandy clay				
1.00	35.50		... becoming orange red				
1.20	35.30		HAND AUGER REFUSAL at 1.2m depth within stiff sandy clay				
2.00							

RIG: None

DRILLER: AC

METHOD: Hand Auger

LOGGED: ML

GROUND WATER OBSERVATIONS:

REMARKS:

CHECKED: TMC

BOREHOLE LOG

CLIENT: Jennifer Hempton

DATE: 21/03/2023

BORE No.: 5

PROJECT: Geotechnical Investigation

PROJECT No.: 2021-159

SHEET: 1 of 1

LOCATION: 89 Marine Parade, Avalon

SURFACE LEVEL 35.70

RL(m):

Depth (m)	RL (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
				Type	Tests	Type	Results
0.00			TOPSOIL/FILL: Loose, dark brown, fine to medium grained, moist/dry, silty sand with some plant roots				
0.70	35.00		... some tree roots				
1.00			HAND AUGER REFUSAL at 0.70m depth within tree roots				

RIG: None

DRILLER: AC

METHOD: Hand Auger

LOGGED: ML

GROUND WATER OBSERVATIONS:

REMARKS:

CHECKED: TMC

BOREHOLE LOG

CLIENT: Jennifer Hempton

DATE: 21/03/2023

BORE No.: 6

PROJECT: Geotechnical Investigation

PROJECT No.: 2021-159

SHEET: 1 of 1

LOCATION: 89 Marine Parade, Avalon

SURFACE LEVEL 35.00

RL(m):

Depth (m)	RL (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
				Type	Tests	Type	Results
0.00							
0.10			SANDSTONE PAVERS				
			FILL: Loose, brown, fine to medium grained, moist, silty sand with some gravels				
0.50							
0.70	34.30						
		CI	SANDY CLAY: Very stiff, orange red, medium plasticity, moist, sandy clay with trace of ironstone gravels				
1.00							
1.20	33.80						
			HAND AUGER REFUSAL at 1.20m depth on interpreted sandstone bedrock				

RIG: None

DRILLER: AC

METHOD: Hand Auger

LOGGED: ML

GROUND WATER OBSERVATIONS:

REMARKS:

CHECKED: TMC

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Jennifer Hempton

DATE: 9/08/2021

PROJECT: Geotechnical investigation

PROJECT No.: 21/03/2023

LOCATION: 89 Marine Parade, Avalon

SHEET: 2021-159

1 of 1

	Test Location							
	DCP1 RL= 34.1m	DPC1a RL= 34.1m	DCP2 RL=36.0m	DCP3 RL= 37.7m	DCP4 RL= 36.5m	DCP5 RL= 35.7m	DCP6 RL = 35.0m	DCP7 RL = 37.5m
Depth (m)								
0.00 - 0.10	2	--	2	2	2	--	--	--
0.10 - 0.20	2	--	4	2	5	--	--	2
0.20 - 0.30	3	--	5	3	5	--	--	2
0.30 - 0.40	10	--	5	3	5	--	--	3
0.40 - 0.50	15	--	9	4	3	--	--	3
0.50 - 0.60	20	--	10	4	4	11	--	4
0.60 - 0.70	(D)	--	10	5	3	4	--	5
0.70 - 0.80		--	8	4	3	5	5	5
0.80 - 0.90		--	5	5	3	5	4	6
0.90 - 1.00		--	8	4	4	6	5	6
1.00 - 1.10		--	7	5	4	6	5	8
1.10 - 1.20		--	7	5	3	8	20 Ref.(B) @ 1.20m depth	8
1.20 - 1.30		8	7	5	3	16		6
1.30 - 1.40		12	7	5	3	21		10
1.40 - 1.50		14	8	6	4	13		10
1.50 - 1.60		18	9	9	3	11		5
1.60 - 1.70		19	7	9	6	20 Ref.(B) @1.70m depth		9
1.70 - 1.80		13 Ref. (B) @1.8m depth	10	7	16			5
1.80 - 1.90			17 Ref. (B) @1.9m depth	6	20 Ref.(B) @1.9m depth			7
1.90 - 2.00				11				5
2.00 - 2.10						11		5
2.10 - 2.20				8				7
2.20 - 2.30				7				7
2.30 - 2.40				8				6
2.40 - 2.50				8				10
2.50 - 2.60				11				10 Ref.(B) @2.50m depth
2.60 - 2.70				11				
2.70 - 2.80				11				
2.80 - 2.90				24 Ref. (B)@2.9m depth				
2.90 - 3.00								

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

REMARKS:

(B) Test hammer bouncing upon refusal on solid object

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

Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip of soils from basement excavation (<3m ³)	a) House (No.87A)	Landslip of soils due to excavation 2.7m - 5.7m deep, soil anticipated to <2.0m depth	a) 5.7m excavation 3.3m from house, impact 10% b) 5.7m excavation 1.0m from pathway, impact 70% c) 4.3m excavation 4.3m from house, impact 5% d) 4.3m excavation 2.5m from pathway, impact 50% e) 5.5m excavation 3.3m from house, impact 10% f) 5.5m excavation 1.0m from pathway, impact 70%		a) Person in house 18hrs/day avg. b) Person on pathway, 2hrs/day avg. c) Person in house, 18hrs/day avg. d) Person on pathway, 2hrs/day avg. e) Person in house 18hrs/day avg f) Person on pathway, 2hrs/day avg	a) Likely to not evacuate b) Likely to not evacuate c) Likely to not evacuate d) Likely to not evacuate e) Likely to not evacuate f) Likely to not evacuate	a) Person in building, minor damage only b) Person in open space, likely impacted by fall c) Person in building, minor damage only d) Person in open space, likely impacted by fall e) Person in building, minor damage only f) Person in open space, minor damage only	
			likely	Prob. of Impact	Impacted				
		a) House (No.87A)	0.01	0.20	0.10	0.75	0.75	0.05	5.63E-06
		b) Pathway (No.87A)	0.01	0.40	0.70	0.083	0.75	0.90	1.58E-04
		c) House (No. 89)	0.01	0.10	0.05	0.75	0.75	0.05	8.75E-06
		d) Pathway (No.89)	0.01	0.30	0.50	0.083	0.75	0.90	8.44E-05
		e) House (No.87)	0.01	0.20	0.10	0.750	0.75	0.05	5.63E-06
		f) Pathway (No.87)	0.01	0.40	0.65	0.083	0.75	0.90	1.46E-04
B	Rock fall/topple from basement excavation (<10m ³)		Bedrock/boulder topple/slide due to excavation 2.7m - 5.7m depth, bedrock anticipated from 1.2m depth	a) 5.7m excavation 3.3m from house, impact 10% b) 5.7m excavation 1.0m from pathway, impact 70% c) 4.3m excavation 4.3m from house, impact 5% d) 4.3m excavation 2.5m from pathway, impact 50% e) 5.5m excavation 3.3m from house, impact 10% f) 5.5m excavation 1.0m from pathway, impact 70%		a) Person in house 18hrs/day avg. b) Person on pathway, 2hrs/day avg. c) Person in house, 18hrs/day avg. d) Person on pathway, 2hrs/day avg. e) Person in house 18hrs/day avg f) Person on pathway, 2hrs/day avg	a) Likely to not evacuate b) Likely to evacuate c) Likely to not evacuate d) Likely to evacuate e) Likely to not evacuate f) Likely to evacuate	a) Person in building, minor damage only b) Person in open space, likely impacted by fall c) Person in building, minor damage only d) Person in open space, likely impacted by fall e) Person in building, minor damage only f) Person in open space, minor damage only	
			Possible	Prob. of Impact	Impacted				
		a) House (No.87A)	0.001	0.10	0.10	0.75	0.75	0.05	2.81E-07
		b) Pathway (No.87A)	0.001	0.20	0.70	0.083	0.25	0.90	2.63E-06
		c) House (No. 89)	0.001	0.05	0.05	0.75	0.75	0.05	1.46E-07
		d) Pathway (No.89)	0.001	0.20	0.50	0.083	0.25	0.90	1.88E-06
		e) House (No.87)	0.001	0.10	0.10	0.750	0.75	0.05	2.81E-07
		f) Pathway (No.87)	0.001	0.20	0.65	0.083	0.25	0.90	2.43E-06
C	Landslip from soil/rock due to collapse in cliff line overlay (10m ³)		Overlay exists, no indication of tension cracks or significant destabilising conditions	a) Deck located approximately 10m from cliff crest, overlay appears <3.00m in lateral extension		a) Person on deck, 1hrs/day avg.	a) Possible to not evacuate	a) fall from cliff line	
		a) Proposed rear deck	Possible	Prob. of Impact	Impacted				
			0.001	0.10	0.50	0.042	0.50	1.0	1.04E-06

TABLE : A
Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip of soils from Basement excavation (<3m³)	a) House (No.87A)	Possible	The event could occur under adverse conditions over the design life.	Medium	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Moderate
		b) Pathway (No.87A)	Likely	Event will probably occur under adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Low
		c) House (No. 89)	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Low
		d) Pathway (No.89)	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Moderate
		e) House (No.87)	Possible	The event could occur under adverse conditions over the design life.	Medium	Moderate damage to some of structure or significant part of site or MINOR damage to neighbouring property, requires large stabilising works .	Moderate
		f) Pathway (No.87)	Likely	Event will probably occur under adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Low
B	Landslip from soil/rock due to collapse in cliff line overlay (10m³)	a) House (No.87A)	Possible	The event could occur under adverse conditions over the design life.	Medium	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Moderate
		b) Pathway (No.87A)	Likely	Event will probably occur under adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Low
		c) House (No. 89)	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Low
		d) Pathway (No.89)	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Moderate
		e) House (No.87)	Possible	The event could occur under adverse conditions over the design life.	Medium	Moderate damage to some of structure or significant part of site or MINOR damage to neighbouring property, requires large stabilising works .	Moderate
		f) Pathway (No.87)	Likely	Event will probably occur under adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation .	Low
C	Landslip from soil/rock due to collapse in cliff line overlay (10m³)	a) Proposed rear deck	Unlikely	The event might occur under very adverse circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site or MINOR damage to neighbouring property, requires large stabilising works .	Very Low

* qualitative measures of consequences to property assessed per Appendix C in AGS Guidelines for Landslide Risk Management.

* Indicative cost of damage expressed as cost of site development with respect to consequence values: Catastrophic : 200%, Major: 60%, Medium: 20%, Minor: 5%, Insignificant: 0.5%.

* Cost of site development estimated at

\$5,000,000

TABLE: 2

Recommended Maintenance and Inspection Program

Structure	Maintenance/ Inspection Item	Frequency
Stormwater drains.	Owner to inspect to ensure that the open drains, and pipes are free of debris & sediment build-up. Clear surface grates and litter. Owner to check and flush retaining wall drainage pipes/systems Replace non engineered rock/timber walls prior to collapse	Every year or following each major rainfall event. Every 7 years or where dampness/moisture 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 geotechnical inspection prior to removal	Every five years
Slope Stability	Geotechnical Engineering Consultant to check on site stability and maintenance	Five years after construction is completed.

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

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.

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007
APPENDIX C: LANDSLIDE RISK ASSESSMENT
QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

Approximate Annual Probability		Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
Indicative Value	Notional Boundary					
10 ⁻¹	5x10 ⁻²	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10 ⁻²		100 years		The event will probably occur under adverse conditions over the design life.	LIKELY	B
10 ⁻³	5x10 ⁻³	1000 years	200 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10 ⁻⁴		10,000 years		The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10 ⁻⁵	5x10 ⁻⁵	100,000 years	20,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10 ⁻⁶	5x10 ⁻⁶	1,000,000 years	200,000 years	The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

Approximate Cost of Damage		Description	Descriptor	Level
Indicative Value	Notional Boundary			
200%	100%	Structure(s) completely destroyed and/or large scale damage requiring major engineering works for stabilisation. Could cause at least one adjacent property major consequence damage.	CATASTROPHIC	1
60%		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%	40%	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%	10%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
0.5%	1%	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*

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

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

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

RISK LEVEL IMPLICATIONS

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

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

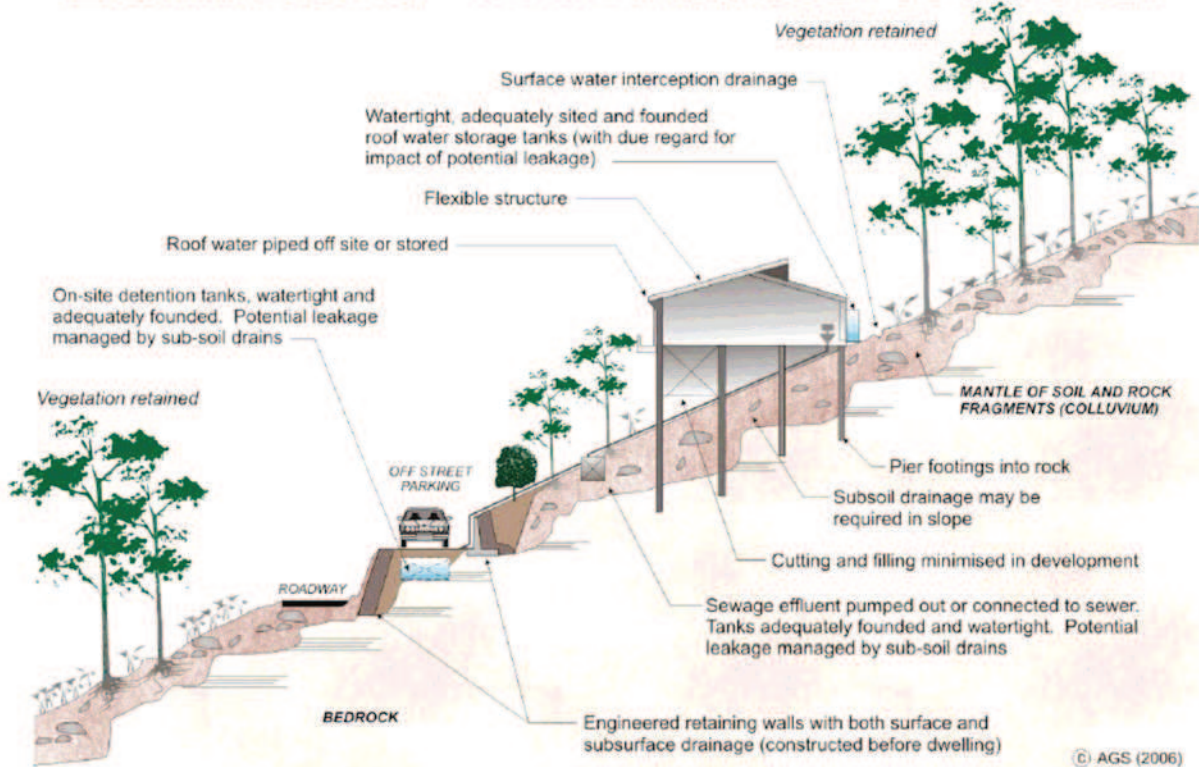
Appendix 5

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007

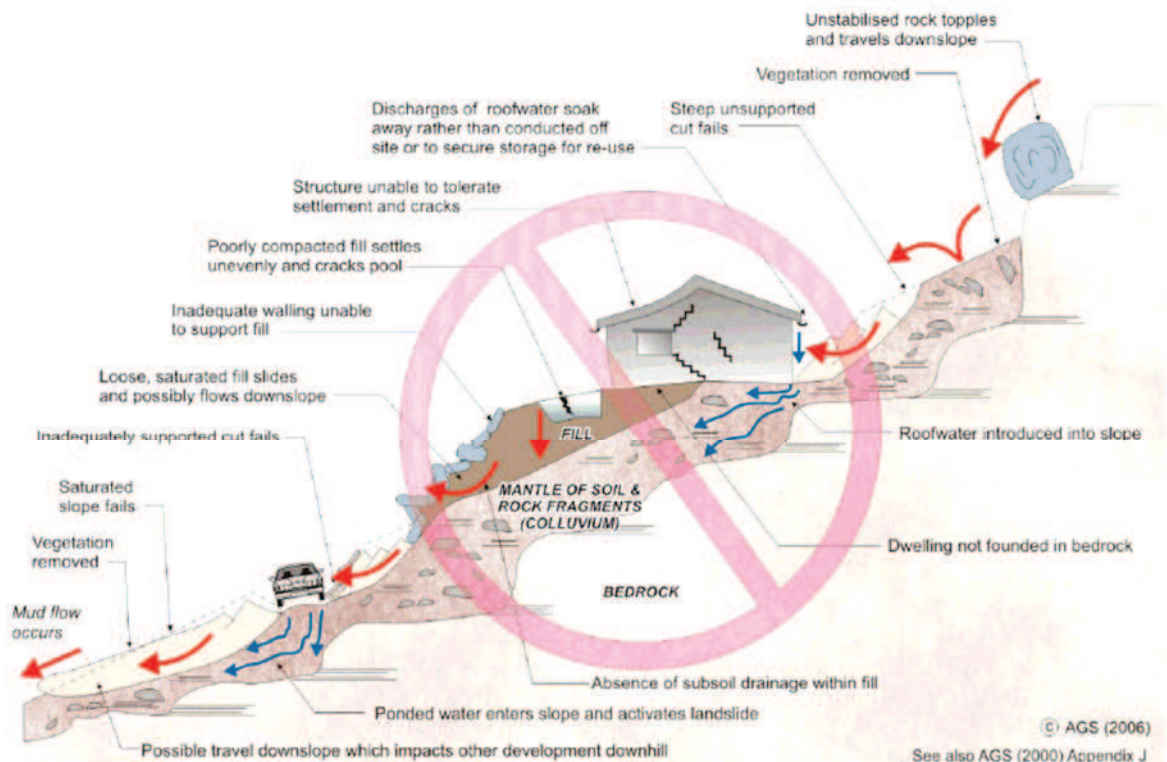
APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

ADVICE		GOOD ENGINEERING PRACTICE	POOR ENGINEERING PRACTICE
GEOTECHNICAL ASSESSMENT		Obtain advice from a qualified, experienced geotechnical practitioner at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING			
SITE PLANNING		Having obtained geotechnical advice, plan the development with the risk arising from the identified hazards and consequences in mind.	Plan development without regard for the Risk.
DESIGN AND CONSTRUCTION			
HOUSE DESIGN		Use flexible structures which incorporate properly designed brickwork, timber or steel frames, timber or panel cladding. Consider use of split levels. Use decks for recreational areas where appropriate.	Floor plans which require extensive cutting and filling. Movement intolerant structures.
SITE CLEARING		Retain natural vegetation wherever practicable.	Indiscriminately clear the site.
ACCESS & DRIVEWAYS		Satisfy requirements below for cuts, fills, retaining walls and drainage. Council specifications for grades may need to be modified. Driveways and parking areas may need to be fully supported on piers.	Excavate and fill for site access before geotechnical advice.
EARTHWORKS		Retain natural contours wherever possible.	Indiscriminatory bulk earthworks.
CUTS		Minimise depth. Support with engineered retaining walls or batter to appropriate slope. Provide drainage measures and erosion control.	Large scale cuts and benching. Unsupported cuts. Ignore drainage requirements
FILLS		Minimise height. Strip vegetation and topsoil and key into natural slopes prior to filling. Use clean fill materials and compact to engineering standards. Batter to appropriate slope or support with engineered retaining wall. Provide surface drainage and appropriate subsurface drainage.	Loose or poorly compacted fill, which if it fails, may flow a considerable distance including onto property below. Block natural drainage lines. Fill over existing vegetation and topsoil. Include stumps, trees, vegetation, topsoil, boulders, building rubble etc in fill.
ROCK OUTCROPS & BOULDERS		Remove or stabilise boulders which may have unacceptable risk. Support rock faces where necessary.	Disturb or undercut detached blocks or boulders.
RETAINING WALLS		Engineer design to resist applied soil and water forces. Found on rock where practicable. Provide subsurface drainage within wall backfill and surface drainage on slope above. Construct wall as soon as possible after cut/fill operation.	Construct a structurally inadequate wall such as sandstone flagging, brick or unreinforced blockwork. Lack of subsurface drains and weepholes.
FOOTINGS		Found within rock where practicable. Use rows of piers or strip footings oriented up and down slope. Design for lateral creep pressures if necessary. Backfill footing excavations to exclude ingress of surface water.	Found on topsoil, loose fill, detached boulders or undercut cliffs.
SWIMMING POOLS		Engineer designed. Support on piers to rock where practicable. Provide with under-drainage and gravity drain outlet where practicable. Design for high soil pressures which may develop on uphill side whilst there may be little or no lateral support on downhill side.	
DRAINAGE			
SURFACE		Provide at tops of cut and fill slopes. Discharge to street drainage or natural water courses. Provide general falls to prevent blockage by siltation and incorporate silt traps. Line to minimise infiltration and make flexible where possible. Special structures to dissipate energy at changes of slope and/or direction.	Discharge at top of fills and cuts. Allow water to pond on bench areas.
SUBSURFACE		Provide filter around subsurface drain. Provide drain behind retaining walls. Use flexible pipelines with access for maintenance. Prevent inflow of surface water.	Discharge roof runoff into absorption trenches.
SEPTIC & SULLAGE		Usually requires pump-out or mains sewer systems; absorption trenches may be possible in some areas if risk is acceptable. Storage tanks should be water-tight and adequately founded.	Discharge sullage directly onto and into slopes. Use absorption trenches without consideration of landslide risk.
EROSION CONTROL & LANDSCAPING		Control erosion as this may lead to instability. Revegetate cleared area.	Failure to observe earthworks and drainage recommendations when landscaping.
DRAWINGS AND SITE VISITS DURING CONSTRUCTION			
DRAWINGS		Building Application drawings should be viewed by geotechnical consultant	
SITE VISITS		Site Visits by consultant may be appropriate during construction/	
INSPECTION AND MAINTENANCE BY OWNER			
OWNER'S RESPONSIBILITY		Clean drainage systems; repair broken joints in drains and leaks in supply pipes. Where structural distress is evident see advice. If seepage observed, determine causes or seek advice on consequences.	

EXAMPLES OF **GOOD** HILLSIDE PRACTICE



EXAMPLES OF **POOR** HILLSIDE PRACTICE



Appendix 6

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28 September 2023

Coastal Engineering Advice on 89 Marine Parade Avalon Beach

1. INTRODUCTION AND BACKGROUND

It is proposed to demolish and rebuild a dwelling at 89 Marine Parade Avalon 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_016) 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 Marine Parade. He undertook a specific inspection of the site, and the adjacent cliff face from the rock platform at the base of the cliff, on 14 April 2023.

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.

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 33 drawings of the proposed development prepared by Annabelle Chapman Architect (namely Drawings DA000 to 002, 101 to 105, 151, 152, 200 to 204, 301 to 306, 401 to 406, 601, 602, 701, and 901 to 903), all dated 21 November 2023 and Issue 'DA'. A site survey by CMS Surveyors was also provided, Reference 19500Bdetail, Issue 2 and dated 26 October 2023.

3. EXISTING SITE DESCRIPTION

The site is located along a rocky cliff section of coastline that extends from North Avalon (Avalon Beach) to Whale Beach, including Avalon Headland, Bangalley Head and Careel Head. Broad and zoomed vertical aerial views are provided in Figure 1 and Figure 2 respectively, with an oblique aerial view in Figure 3. A view of the site from the cliff to the north is in Figure 4. A cross-section location through the site, denoted as Section A herein, is also depicted in Figure 2, with the section provided in Figure 5 as derived from NSW Government Airborne Laser Scanning (ALS) data that was captured in 2020.



Figure 1: Broad aerial view of site (approximate red outline) on 16 February 2022

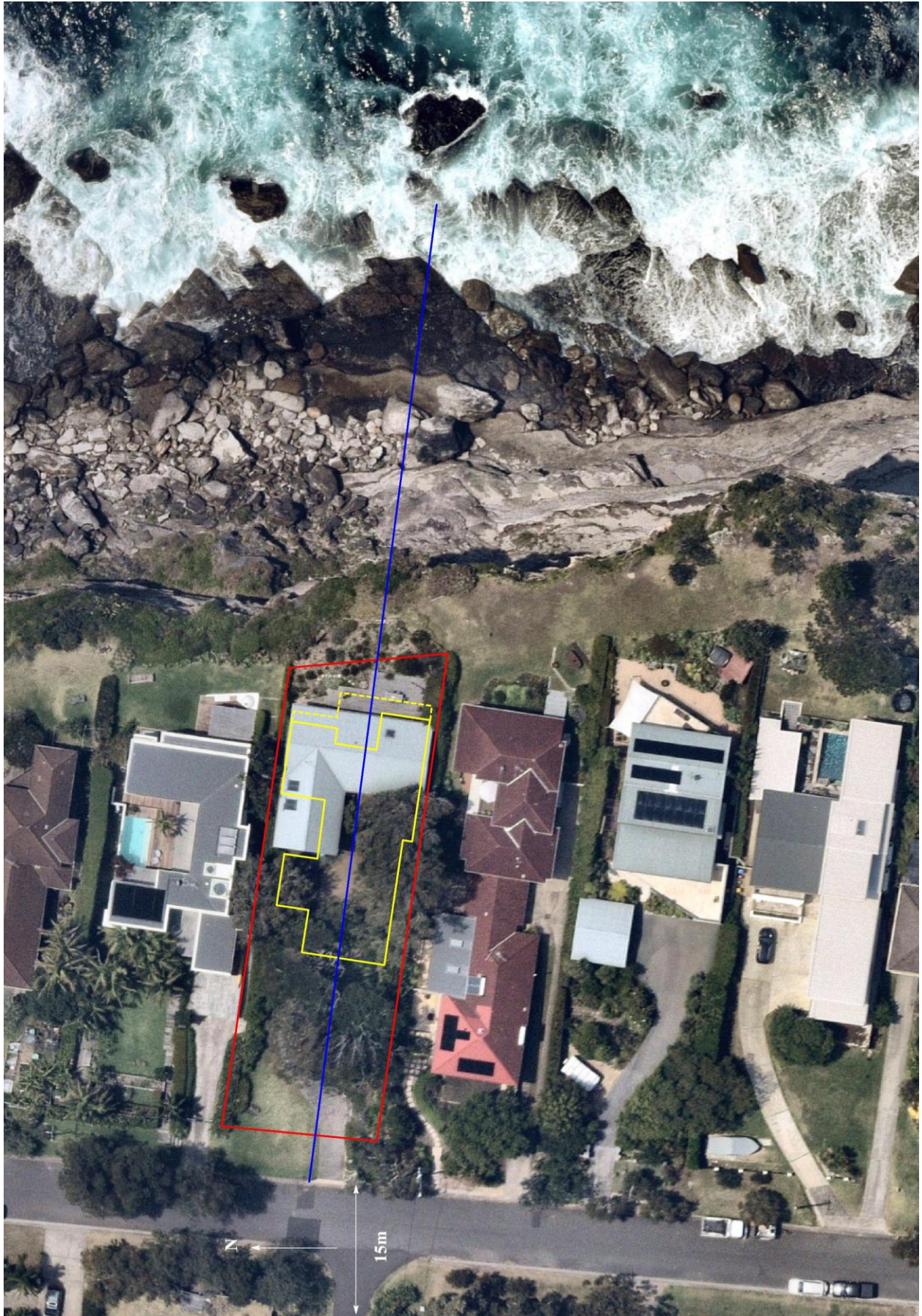


Figure 2: Zoomed aerial view of site (approximate red outline) on 21 October 2023, with Section A location in blue and outline of proposed upper ground floor in solid yellow (with deck dashed)



Figure 3: Oblique aerial view of site (at arrow) on 1 May 2023, facing west



Figure 4: View of site (at arrow) from cliff to north on 14 April 2023, facing SSW

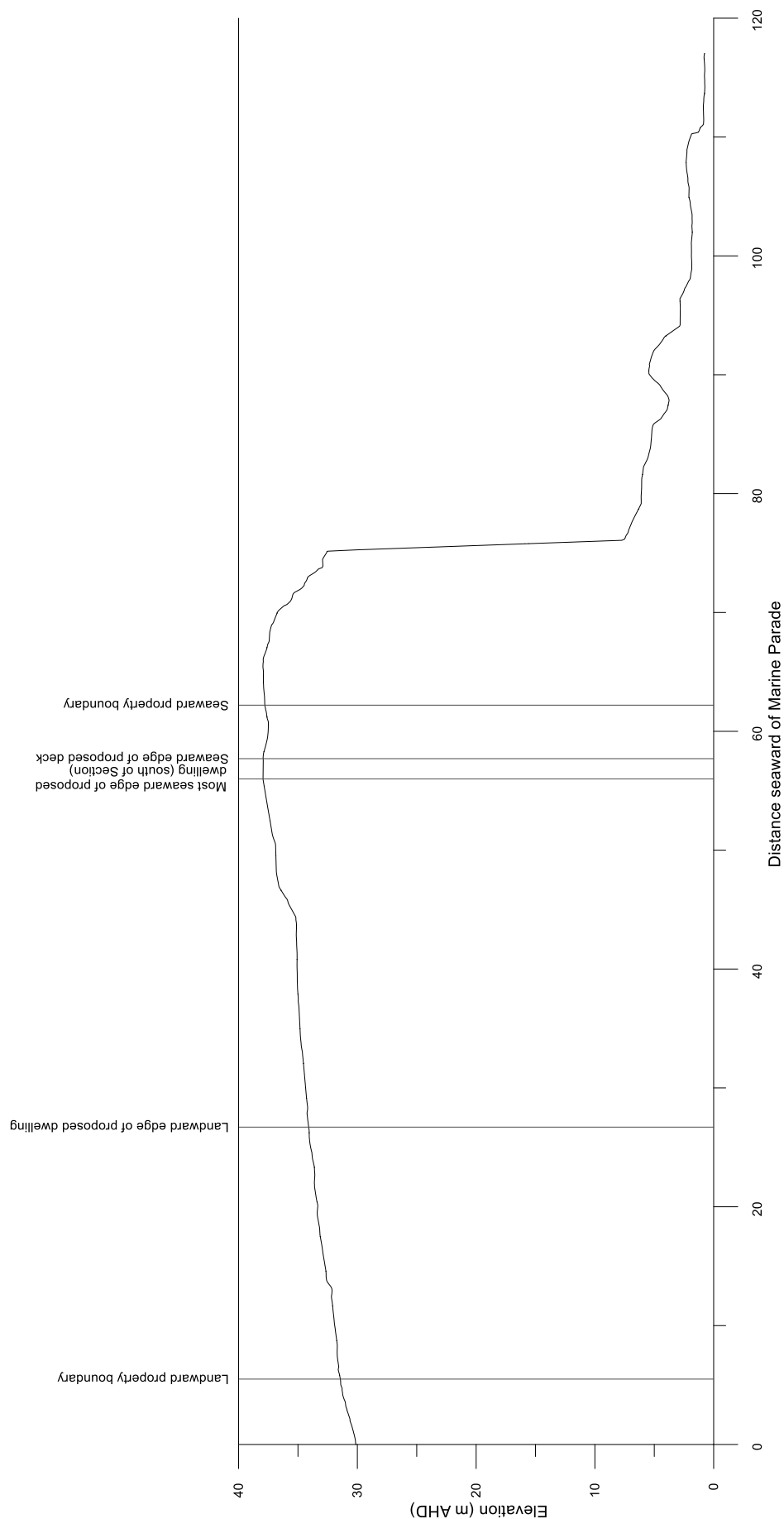


Figure 5: Section A through site and adjacent cliff and rock platform

There is no evidence of any recent significant cliff failures having occurred near the site. Coffey & Partners (1987) noted that the cliff profiles in the vicinity of the site (from Avalon Beach to Careel Head) were composed of massive sandstone and interbedded siltstone/sandstone beds with slope angles of about 80°. Undercutting in the sandstone units was found to have produced local overhangs and slope angles as low as 45° in the interbedded units. Cliff formation was seen to be primarily controlled by jointing, with undercutting in the less resistant interlaminated beds and toppling of large blocks of sandstone which line the cliff base.

Based on the ALS data depicted in Figure 5, ground elevations along Section A at and adjacent to the site approximately vary as follows:

- 30m AHD at Marine Parade;
- 38m AHD at the seaward edge of the proposed works and at the seaward property boundary;
- 32.4m AHD at the top of the near-vertical cliff face;
- 7.7m AHD at the toe of the near-vertical cliff face; and
- 0.7m AHD at the seaward edge of the rock platform (seaward limit of Section A).

The average slope from Marine Parade to the seaward edge of the proposed development is about 8°, and the average slope of the near-vertical cliff face is about 88°.

4. PROPOSED DEVELOPMENT

It is proposed to demolish and rebuild the dwelling at the site, with the new dwelling having three levels. The lowest level (basement) is to have a finished floor level of 31.6m AHD, with the lower ground floor at 35.1m AHD, and upper ground floor at 38.3m AHD. The upper ground floor is the ground level on the seaward side of the site. The location of the proposed development (upper ground floor) was depicted in Figure 2.

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.

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 6mm per year determined by Crozier and Braybrooke (1992), noting that they determined corresponding maximum rates of 12mm/year.

An apparent approximate 40m of cliff recession (observed in aerial photography as the distance of the cliff toe from the seaward edge of the rock platform) over the last 6,400 years (since sea levels stabilised around their present levels, and assuming that the cliff toe was at the seaward edge of the rock platform at that time) represents an average recession rate of 6mm/year, consistent with these average rates.

The cliff face seaward of the site would only be impacted by wave runup in extreme storms. At times of severe coastal storms with large waves and elevated water levels, the 100 year Average Recurrence Interval (ARI) wave runup level is about 8m AHD at present, increasing up to around 9m AHD in 100 years if projected sea level rise is realised.

A recession/weathering rate of 6mm per year is considered to be appropriate, with sensitivity testing for a rate of 12mm per year as a conservative two multiple rate increase to account for sea level rise². 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 cliff face, although in reality it would be expected that runup would generally be below about 9m AHD in a severe coastal storm over the 100-year design life. Therefore, an allowance for recession/weathering of the cliff face of about 6mm to 12mm per year should be considered and assessed by the geotechnical engineer³.

The geotechnical engineer should consider these estimated rates in conjunction with an understanding of the particular nature of the cliff materials seaward of the site, their resistance to erosion/recession, and potential failure planes related to geotechnical issues such as the joint spacing⁴.

This should be confirmed by the geotechnical engineer, but it is expected that the recession/weathering described above would lead to undercutting and collapse of blocks on the cliff face over the long term, with failure planes at the joints. That stated, any future failure of the slope above the top of the cliff, or within the site itself, may be unrelated to coastal

² There are no established methods to estimate increased recession rates of cliff lines due to sea level rise, but a 2.0 factor on historical rates is considered to be particularly conservative. In the 2011 *Wyong Coastal Zone Management Plan* (CZMP) and 2017 draft *Wyong CZMP*, a factor of 1.2 was used to 2100.

³ Note that this does not mean that the cliff face is predicted to recede at a steady rate of 6 to 12mm/year. In reality, there are likely to be slower rates of weathering over decades or centuries until a significant undercut occurs that detaches a block above, which leads to a sudden loss of an extent of cliff face much larger than the order of 10mm. However, averaging this slower weathering and block failures over the long term, an average rate of 6mm to 12mm/year (which can also be stated as 0.6m to 1.2m per 100 years) is expected.

⁴ Coffey & Partners (1987) noted that the controlling feature of interbedded sandstone/siltstone cliffs was the bedding spacing and relative proportion of sandstone/siltstone.

processes at the base of the cliff, so other failure mechanisms should be considered by the geotechnical engineer.

6. COASTAL INUNDATION

With the seaward edge of the proposed development above 38m 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 seaward of the site, and other processes, for a design life of at least 100 years⁵.

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)⁶ and its associated mapping, the site is within a “coastal environment area” (see Section 7.2.2) and a “coastal use area” (see Section 7.2.3).

7.2.2 *Clause 2.10*

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

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

With regard to (a), the proposed works are in a developed residential area. The works would not be expected to adversely affect the biophysical and hydrological (surface and groundwater) environments. Conventional stormwater management measures are proposed, including a

⁵ At a location with underlying bedrock such as the site, it is the responsibility of the geotechnical engineer, not the coastal engineer, to determine the risk to the development.

⁶ Formerly *State Environmental Planning Policy (Coastal Management) 2018*.

30,000L rainwater tank and 9,000L OSD tank, and with the site draining towards the street. The proposed works would not be a source of pollution as long as appropriate construction environmental controls are applied.

The proposed works would not be expected to adversely affect the ecological environment, assuming that tree removal is considered to be acceptable. In an Arboricultural Impact Assessment report for the site prepared by Margot Blues, it is noted that 12 trees are to be removed as a result of the proposed development.

With regard to (b), the proposed works would not be expected to affect natural coastal processes, being landward of an elevated cliff for an acceptably rare storm over an acceptably long life.

With regard to (c), the proposed works would not adversely impact on water quality as long as appropriate construction environmental controls are applied.

With regard to (d), this is not a coastal engineering matter so is not definitively considered herein. That stated, there are no undeveloped headlands or rock platforms in proximity to the proposed development, and no marine vegetation in the area to be developed. Assuming that there is no native vegetation and fauna and their habitats of significance at the site, this clause has been satisfied.

With regard to (e), the proposed works would not impact on public open space and access to and along the foreshore, being entirely on private property.

With regard to (f), a search of the Heritage NSW “Aboriginal Heritage Information Management System” (AHIMS) was undertaken on 23 November 2023. This resulted in no Aboriginal sites being recorded nor Aboriginal places being declared within at least 200m of the site.

With regard to (g), the proposed works would not be expected to alter wave and water level processes seaward of the cliff line, or impact on the 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 the 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), as noted in Section 7.2.2, there are no Aboriginal sites nor Aboriginal places recorded or declared within at least 200m of the site.

With regard to (a)(v), the nearest environmental heritage items to the site listed in Schedule 5 of *Pittwater Local Environmental Plan 2014* are sandstone road remnants and associated landscaping adjacent to 640, 642 and 644 Barrenjoey Road Avalon Beach. These heritage items are located at least 510m from the site. The proposed development would not be expected to impact on these or more distant heritage items.

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

7.2.4 Clause 2.12

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

Assuming that the geotechnical engineer will find that the proposed development is at an acceptably low risk of damage from erosion/recession over a 100 year design life, and given that the proposed development is well above and landward of projected wave runoff 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_016. 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. The proposed development would not be expected to even interact with coastal processes for an acceptably rare storm and acceptably long life.

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 6mm per year of the cliff seaward of 89 Marine Parade Avalon Beach, with sensitivity testing up to 12mm per 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 seaward 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 slope above the top of the cliff, or within the site itself, may be unrelated to coastal processes at the base of the cliff, so other failure mechanisms should be considered by the geotechnical engineer.

Coastal inundation is not a significant risk 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

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

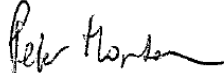
Sunamura, Tsuguo (1983), "Processes of Sea Cliff and Platform Erosion", Chapter 12 in *CRC Handbook of Coastal Processes and Erosion*, editor Paul D Komar, CRC Press Inc, Boca Raton, Florida, ISBN 0-8493-0208-0

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

This report has been prepared by Horton Coastal Engineering on behalf of and for the exclusive use of Jenn Hempton (the client) and is subject to and issued in accordance with an agreement between the client and Horton Coastal Engineering. Horton Coastal Engineering accepts no liability or responsibility whatsoever for the report in respect of any use of or reliance upon it by any third party. Copying this report without the permission of the client or Horton Coastal Engineering is not permitted.

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 <u>Jenn Hempton</u>
Name of Applicant
Address of site <u>89 Marine Parade Avalon Beach</u>

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 23 November 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:	<u>Coastal Engineering Advice on 89 Marine Parade Avalon Beach</u>
Report Date:	<u>23 November 2023</u>
Author:	<u>Peter Horton</u>
Author's Company/Organisation:	<u>Horton Coastal Engineering Pty Ltd</u>

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

<u>See Section 2 and Section 10 of coastal engineering report</u>

~~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.~~

See revised declaration in Section 8 of report

Signature Peter Horton
Name Peter Horton
Chartered Professional Status... MIEAust CPEng.NER
Membership No. 452980
Company... Horton Coastal Engineering Pty Ltd