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REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

PROPOSED NEW RESIDENTIAL HOUSE

at

9 & 10 SURFSIDE AVENUE, AVALON BEACH, NSW

Prepared For

Amanda Stabback

Project No.: 2020-033

February 2020

Document Revision Record

Issue No	Date	Details of Revisions
0	26th February 2020	Original issue

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for: ____Amanda Stabback____

Name of Applicant

Address of site ____9 and 10 Surfside Avenue, Avalon Beach ____

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

*I, __**Troy Crozier**__ on behalf of ___**Crozier Geotechnical Consultants*** _on this the 26th February 2020_ 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 Residential House

Report Date: 26/02/2020 **Project No.:** 2020-033

Author: B.Sheppard and T. Crozier

Author's Company/Organisation: Crozier Geotechnical Consultants

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

Architectural Drawings by Richard Cole Architecture, Project No.: 1907 Drawing No.: DA00 to DA31, Issue: G, Dated: 16/01/2020

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**

REGISTERED PROFESSIONAL GEOSCIENTIST
AUSTRALIAN INSTITUTE OF GEOSCIENTISTS
TROY CROZIER
10,197

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

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

Development Application for: ____**Amanda Stabback**____
Name of Applicant
Address of site __9 and 10 Surfside Avenue, Avalon Beach_______

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

Geotechnical Report Details:

Report Title: Geotechnical Report for Proposed New Residential House
Report Date: 26/02/2020 **Project No.:** 2020-033
Author: B.Sheppard and T. Crozier
Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

- ■ Comprehensive site mapping conducted ___19th February 2020________ (date)
- ■ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
- ■ Subsurface investigation required
  - ☐ No Justification ....
  - ■ Yes Date conducted 19th February 2020...........
- ■ 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 ...50 years... (specify)
- ■ Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
- ■ Additional action to remove risk where reasonable and practical have been identified and included in the report.
- ☐ Risk assessment within Bushfire Asset Protection Zone.

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

Signature ....

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

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

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

Company... **Crozier Geotechnical Consultants**

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GEOTECHNICAL REPORT FOR NEW RESIDENTIAL HOUSE
9 & 10 SURFSIDE AVENUE, AVALON BEACH, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for alterations and additions at 9 and 10 Surfside Avenue, Avalon Beach, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of Richard Cole Architecture on behalf of the client Amanda Stabback.

The site is not located within a landslip hazard zone as identified within Northern Beaches Councils precinct (Geotechnical Risk Management Policy for Pittwater - 2009). However, as per councils letter response to the development application (DA 2020/068, dated 2/2/20), the works trigger the policy in regard to excavation and filling ï Section 3.2 (b) (iv), due to excavation depths exceeding 1.5m.

To meet the Councils Policy requirements for works which trigger the policy, a detailed Geotechnical Report which meets the requirements of Paragraph 6.5 of that policy must be submitted. This report must include a landslide risk assessment to the methods of AGS 2007 for the site and proposed works, plans, geological sections and provide recommendations for construction and to ensure stability is maintained for a preferred design life of 100 years.

This report therefore includes a description of site and subsurface conditions, a geotechnical assessment of the development, site mapping/plan, a geological section, a landslide risk assessment of the site and proposed works and recommendations for construction.

The investigation and reporting were undertaken as per the Tender: P20-074, Dated: 16th February 2020.

The investigation comprised:

a) DBYD request and review.
b) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Geotechnical Engineer.

c) Drilling of four boreholes using hand tools along with five Dynamic Cone Penetrometer (DCP) tests to investigate the subsurface geology, depth to bedrock and identification of ground water conditions.

The following plans and drawings were supplied for the work:

- Architectural Drawings by Richard Cole Architecture, Project No.: 1907 Drawing No.: DA00 to DA31, Issue: G, Dated: 16/01/2020

1.1. Proposed Development

It is understood that 'the site' is comprised of two separate lots; No.9 and No.10 Surfside Avenue, Avalon Beach. The proposed works involve the demolition of all site structures and the construction of a new two-storey residential house on the front portion of the site. The new structure will be comprised of two distinct building structures situated to the north and south of the site, with a first floor verandah connecting the two buildings. The rear of the site will consist of grassed lawns and retained gardens with a detention basin proposed in the north-west corner.

The works will require bulk excavation of up to approximately 3.10m depth into the slope to achieve ground floor levels of RL26.40 for the southern building and will extend to within 1.00m of the southern boundary and 6.50m of the eastern boundary. Additionally, bulk excavation of up to approximately 2.30m depth will also be required for the underground garden storage room beneath the northern building and will extend to within 6.00m of the northern boundary and 6.50m of the eastern boundary.

2. SITE FEATURES:

2.1. Description:

The site (No.9 and No.10 Surfside Avenue) is a trapezoid shaped block located on the low west side of Surfside Avenue within gentle north-west dipping topography. Site surface levels reduce from the southeast to the northwest between a high of approximately RL29.24m to a low of approximately RL21.36m.

The proposed site will have a front east boundary of 36.58m, a rear west boundary of 48.16m and side north and south boundaries of 53.80m and 40.48m respectively, as referenced from available survey plans.

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

Photograph 1: Aerial photo of site and surrounds

The site dwelling at No.9 Surfside Avenue comprises of a two storey rendered residence centrally positioned to the front of the site, with timber decking, retained grassed lawns and garden beds to the rear. Vehicular access to the site is provided via a moderate (-15°) sloping pebblecrete driveway leading from a concrete crossover to a dual garage under the existing residence. General views of the rear of the site are shown in Photograph 2.

Photograph 2: Rear view of the site residence at No.9, looking east

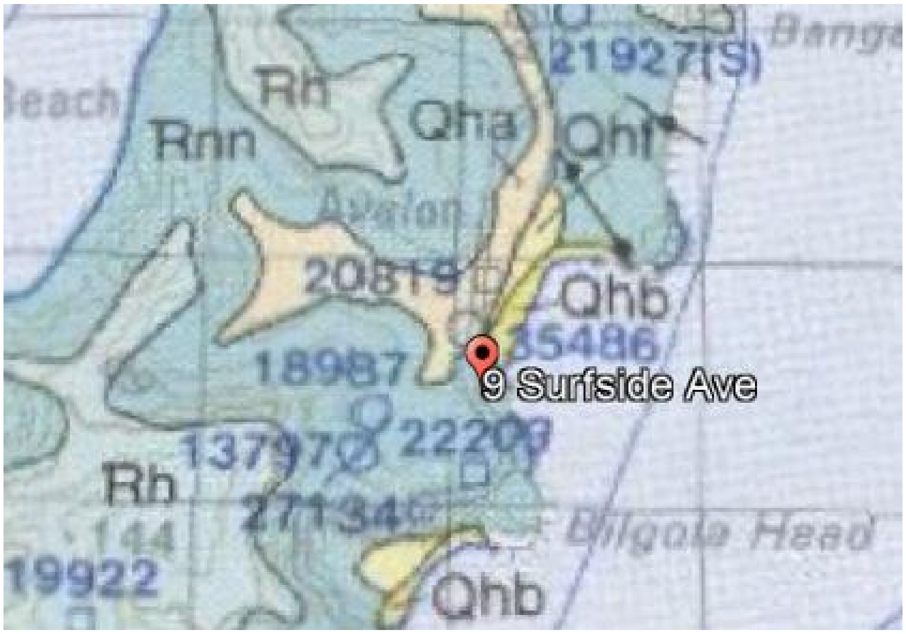
The site dwelling at No.10 Surfside Avenue comprises of a two storey brick residence centrally positioned to the front of the site. The rear of the site consists of a part in-ground swimming pool, tennis court and retained grassed lawns. General views of the front of the site are shown in Photograph 3.

Photograph 3: Front view of the property at No.10, looking west

2.2. Geology:

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

Narrabeen Group rocks are dominated by shales and thin siltstone/sandstone beds and often form rounded convex ridge tops with moderate angle (<20°) side slopes. These side slopes can be either concave or convex depending on geology, internally they comprise interbedded shale and siltstone beds with close spaced bedding partings that have either close spaced vertical joints or in extreme cases large space convex joints. The shale often forms deeply weathered profiles with silty or medium to high plasticity clays and a thin silty colluvial cover.



Extract of Sydney 1:100 000 Geological Services Sheet

3. FIELD WORK:

3.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on the 19th February 2020 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 slopes, existing structures and neighbouring properties.

It also included the drilling of four boreholes (BH1 to BH4) using a hand auger to investigate subsurface geology. A hand auger was used as access to the site for a conventional drilling rig was unavailable.

DCP testing was carried out from ground surface adjacent to the boreholes and at a separate location in accordance with AS1289.6.3.2 ï 1997, ñDetermination of the penetration resistance of a soil ï 9kg Dynamic Cone Penetrometerò to estimate near surface ground conditions.

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

3.2. Field Observations:

Surfside Avenue contains a bitumen pavement and it dips gently (-4°) to the north where it passes the site. There were no signs of excessive cracking or deformation within the road pavement and concrete crossover to suggest any movement or underlying geotechnical issues.

Both site structures appeared to be in good condition with no signs of significant cracking or settlement on the external walls. Both site residences are accessed via a pebblecrete driveway which adjoins Surfside Avenue by a concrete crossover leading to garages located underneath the residences. Cracking was observed in both pebblecrete driveways, however this appears likely due to design and construction methodology.

The rear of the property at No.9 is accessed via a sandstone paver pathway along the southern boundary, which leads to a timber staircase extending from the first floor balcony to a timber deck at the rear. The rear yard contains terraced grassed yards and multiple retained garden beds. The rear yard has been retained along the rear boundary by up to 1.50m from the neighbouring properties. The retaining walls throughout the site appear to be in good condition with no signs of excessive cracking or deformation. General views of the rear of the site are shown in Photograph 4.

Photograph 4: Retained gardens at the rear of No.9

The rear of the property at No.10 is accessed via a pathway along the southern boundary which leads to the rear area, comprising of a part in-ground swimming pool to the south, tennis court to the north-west and grassed lawn to the north-east with segmented concrete slabs for pedestrian access. The front and rear grassed lawns along the northern boundary are separated by a 1.4m high besser block retaining wall whilst the tennis court is retained below the adjacent ground level by a 1.0m high besser block retaining wall. It was observed that the concrete slab between the grassed lawn and tennis court has experienced subsidence and has resulted in significant cracking and deformation, as shown in Photograph 4.

Photograph 4: Subsidence of the concrete slab at No.10

The neighbouring property to the south (No. 8 Surfside Avenue) contains a one and two storey rendered house located at the centre of the property with front and rear grassed lawns and gardens. The structure appears to be approximately 40 years of age and in a good condition with no signs of significant cracking or settlement on the external walls. The residence is located within 1.00m of the common boundary. The ground surface of this property is slightly higher that the site due to some retaining structures and the naturally north-west sloping topography.

The neighbouring property to the north (No. 11 Surfside Avenue) contains a two storey concrete house with front and rear grassed lawns. The structures appear to be very modern and in a good condition with no signs of significant cracking or settlement on the external walls. The structure is located within 1.00m of the common boundary. The ground surface level within this property is generally similar to the site along the common boundary with the portion at the rear slightly lower than the site due to the naturally north-west sloping topography.

The neighbouring properties to the west (No. 556, 554, 552 and 550 Barrenjoey Road) are located downslope to the site and contain predominantly one and two storey residences at the rear of the properties with large front lawns and gardens. All site structures are located approximately >2.50m away from the common boundary, with the exception of No.552 which has a detached shed structure within 1.00m of the common boundary. The sites have similar north-west sloping topography to that of the site.

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

3.3. Ground Conditions:

The boreholes (BH1 to BH4) were drilled using a hand auger with refusal encountered between 0.30m depth (BH1) and 1.75m depth (BH4) on interpreted very low strength bedrock. BH3 encountered effective refusal on interpreted cobbles/boulders.

DCP tests were carried out from the ground surface adjacent to the boreholes and at a separate location with refusal encountered at a maximum depth of 1.94m (DCP4a).

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

- **FILL –** this layer was encountered at all borehole locations to depths between 0.30m and 0.80m. It is classified as brown orange, fine to medium grained/low to medium plasticity, moist clayey sand/sandy clay. Due to effective refusal in BH3, the maximum depth of the fill unit in this location was not identified, however it is possible that the unit could extend deeper than 0.80m depth, especially to the rear of retaining walls.
- **RESIDUAL SOILS –** this layer was encountered in BH2 and BH4, to a maximum depth of 1.75m (BH4). This unit is likely to overlay the underlying bedrock and is classified medium dense/stiff to very dense/hard, orange brown mottled red and grey, fine to medium grained/low plasticity, moist clayey sand and silty clay.
- **BEDROCK –** Based on the DCP test results, bedrock of at least very low strength is interpreted at 0.30m depth (DCP1) to 0.55m depth (DCP2) at the southern end of the site (No.9) and 1.18m depth (DCP5) to 1.94m depth (DCP4a) at the northern end of the site (No.10).

A free standing ground water table or significant water seepage were not identified within any of the boreholes. Indications of groundwater were not observed after the retrieval of the DCP rods.

4. COMMENTS:

4.1. Geotechnical Assessment:

The site investigation identified the presence of a layer of clayey sand fill up to a maximum depth of 0.80m, below which medium dense clayey sand and stiff to very stiff silty clay were encountered which are likely to be residual soils overlying the bedrock. The bedrock is likely comprising shale/siltstone/sandstone varying from extremely to very low strength with low strength bands. A ground water table is not expected to be encountered within the proposed depth of excavations however seepage at the interface between fill and the residual soils and at the soil/bedrock contact is anticipated.

The proposed works involve the demolition of all site structures and the construction of a new two storey residential house with large rear garden and lawns. The new house will have two distinct building structures situated to the front north and front south corners of the site, with a first floor verandah connection.

The works will require an excavation of up to approximately 3.10m depth into the slope for the southern building ground floor (RL26.40) and will extend to within 1.00m of the southern boundary, 6.50m of the eastern boundary and >7.00m of the western boundary. Based on the investigation results, the proposed excavation is anticipated to encounter clayey sand fill to 0.30m depth before bedrock excavation is required below this depth. As the excavation moves to the west, a thicker layer of residual soils is expected, such that bedrock will be encountered at approximately 0.55m depth to the middle and potentially deeper at the western end of the excavation.

Likewise, excavation of up to 2.30m depth will be required to obtain finished floor levels of RL24.00 for the garden storage area located under the garage in the northern building. This excavation will extend to within 6.00m of the northern boundary and 6.50m of the eastern boundary. Based on the investigation results, the proposed excavation is anticipated to encounter a thick layer of clayey sand fill > 0.80m depth then potentially residual soils overlaying bedrock. Due to effective refusal of the auger at 0.80m depth, residual soils were not confirmed at this location, however the DCP results indicate that the fill unit may potentially extend to 1.0m depth before intersecting residual soils. Bedrock may be encountered in some parts of the excavation.

Considering the proposed depths of excavations and distances to the boundaries, the recommended safe temporary batter slopes provided in Section 4.3.2 may be implemented for the excavation for the northern and southern buildings to the north, east and west. However, these batters cannot be implemented for the southern building excavation along the southern boundary. Pre-excavation supports to maintain structural integrity of the neighboring property is the lowest risk option and is recommended. However, the drilling of a pile wall may encounter difficulties due to rock strengths at shallow depth. Therefore, it is recommended that test pits be excavated perpendicular to the boundary under geotechnical supervision following demolition of existing structures.

Bedrock strength assessment was not part of the scope for geotechnical investigation, therefore these are unconfirmed and would require core drilling. Care will be required during the construction and excavation works to ensure neighbouring properties and structures are not adversely impacted by ground vibrations from rock excavation equipment. The need for vibration monitoring will be determined based upon the type of rock encountered and the excavation equipment to be used. It is recommended that the geotechnical engineer assess the excavation methodology and equipment prior to commencement. It is recommended that a rock saw and small (Ò200kg) rock hammers be proposed for use in medium strength bedrock at this site to avoid the need for full time monitoring. Preliminary threshold limits are provided in Section 4.3.2. For Interbedded bedrock of up to low strength a ripper may be utilized.

It is recommended that all footings extend through the fill and residual soil units and bear onto/within the bedrock. It is expected that the base of excavation for the southern building will expose very low/low strength bedrock and potentially grading to medium strength in parts. The base of the excavation for the northern building may expose bedrock in parts however it would be prudent to extend all footings to underlying bedrock of similar strengths to reduce the risk of differential settlement. Preliminary allowable bearing pressures appropriate for the sandstone encountered underlying the site are provided in Section 4.3.1.

It is understood that placement of fill (non-structural) is proposed underlying the rear yard. Recommended compaction levels are provided in Section 4.3.5. It is noted that a local sewer main runs parallel to the rear boundary. As such, careful consideration must be given to the additional loading and fill compaction methodology/equipment in this area. Furthermore, a review of ôDial Before You Digô plans outlined a 1500x2500 relined/unlined rock sewer extending diagonally through the front of the site. It is important to consider the potential adverse effects of the proposed works on the sewer lines during design and construction phases.

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

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and 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. However, the results of the investigation provide a reasonable basis for the Development Application analysis (if required) and subsequent preliminary structural design of the proposed works.

4.2. Site Specific Risk Assessment:

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

A. Landslip of soils from excavation for southern building ground floor (<1m³)
B. Landslip (rock slide/topple) due to proposed southern building ground floor excavation
C. Landslip of soils from excavation for northern building garden storage (<1m³)

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

The Risk to Life from Hazard A was estimated to be up to **5.21×10^{-9},** whilst the Risk to Property was considered to be **'Low'**. The hazard was therefore considered to be óAcceptableô when assessed against the criteria of the AGS 2007.

The Risk to Life from Hazard B was estimated to be up to **9.82×10^{-6},** whilst the Risk to Property was considered to be **'Low'**. The hazard was therefore considered to be óAcceptableô when assessed against the criteria of the AGS 2007.

The Risk to Life from Hazard C was estimated to be up to **2.60×10^{-7},** whilst the Risk to Property was considered to be **'Low'**. The hazard was therefore considered to be óAcceptableô when assessed against the criteria of the AGS 2007.

The above risk to life and property from Hazard A, Hazard B and Hazard C have been assessed assuming insufficient retention systems are constructed within the site. Where appropriate retention systems are installed the anticipated risks are expected to further reduce.

4.3. Design & Construction Recommendations:

Design and construction recommendations are tabulated below:

4.3.1. New Footings:	
Site Classification as per AS2870 ï 2011 for new footing design	- Class óAô for footings founded within bedrock where all overlying clay is removed
Type of Footing	Strip/Pad, Slab base of excavation, piers external to excavation to ensure consistent foundation.
Sub-grade material and Maximum Allowable Bearing Capacity	- Weathered, ELS Bedrock: 600kPa - Weathered, VLS Bedrock: 800kPa*
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e ï Rock site
*where proved by geotechnical investigation/inspection to a sufficient depth underlying any new footings **Remarks:** All permanent structure footings should be founded off bedrock of similar strength to reduce the potential for differential settlement and provide lateral resistance unless designed by the structural engineer. All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata. This is mandatory to allow them to be ócertifiedô at the end of the project.	

4.3.2. Excavation:	
Depth of Excavation	Up to 3.10m depth for the proposed southern building ground floor Up to 2.30m depth for the proposed northern building garden storage room
Distance of Excavation to Neighbouring Properties/structures	<u>Southern building ground floor:</u> No. 8 ï 1.00m to boundary, house a further 1.00m. Road Reserve ï 6.50m to boundary, reserve adjacent to boundary <u>Northern building garden storage room:</u> No. 11 ï 6.00m to boundary, house a further 1.00m. Road Reserve ï 6.50m to boundary, reserve adjacent to boundary
Type of Material to be Excavated	Clayey sand fill up <0.80m depth (potentially slightly deeper)
	Medium Dense/Stiff to Very Stiff Residual Soils below 0.30m depth and 0.80m depth

	ELS ï VLS bedrock below 0.30m depth to 1.94m depth
	LS ï MS bedrock (unconfirmed) > 0.50m depth to 2.00m depth

Guidelines for un-surcharged batter slopes for general information are tabulated below:

	Safe Batter Slope (H:V)	
Material	Short Term/ Temporary	Long Term/ Permanent
Fill	1.5:1	2:1
Very stiff to hard silty clay and extremely low strength bedrock	1:1	1.5:1
Very low to low strength bedrock	0.5:1	1:1

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.

Design for bedrock excavation support structures (i.e. anchors/bolts) will be on an individual basis as identified during inspections and should be undertaken by a geotechnical engineer to limit potential exposure across property boundaries. Therefore **if** required bolts should be based on temporary anchoring with full time support implemented as part of the development. Rock bolting across property boundaries will require approval from neighbouring property owners. These approvals will need to be sought prior to commencement of excavation works at the site.

Equipment for Excavation	Fill and Natural soils	Excavator with Bucket
	VLS bedrock	As above assisted with Ripper
	LS ï MS massive bedrock	Rock hammer and saw

ELS ï extremely low strength, VLS ï very low strength, LS ï low strength, MS ï medium strength

The interbedded nature of the bedrock anticipated below this site should allow for ripping of the majority of the excavation. However, medium strength and massive bedrock will be require rock sawing and hammering.

It is recommended that the hard rock excavation perimeter be saw cut prior to rock hammering, this will generally reduce the amount of rock support required, prevent deflection of rock across boundary and under neighbouring house and will provide a slight buffer distance to ground vibrations for the use of rock hammers.

Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	All residential houses = 3mm/s PPV Sewer/service lines = 3mm/s PPV
Vibration Calibration Tests Required	Yes, recommended for any rock hammer >200kg 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: • Following removal of site soils and exposure of bedrock. • Test pits along southern boundary • Any excavation where unsupported. • Following footing excavations to confirm founding material strength
Dilapidation Surveys Requirement	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 sandy and clayey soils. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope, whilst any groundwater seepage must be controlled within the excavation and prevented from ponding or saturating slopes/batters.	

4.3.3. Retaining Structures:	
Required	New retaining structures will be required as part of the proposed development
Types	Temporary retention system may be required along southern boundary, pending confirmed ground conditions. Steel reinforced concrete/concrete block wall designed in accordance with Australian Standard AS 4678-2002 Earth Retaining Structures.
Parameters for calculating pressures acting on retaining walls for the materials likely to be retained:	

Material	Unit Weight (kN/m^3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Clayey Sand Fill	18	$\phi' = 28°$	0.35	0.52	N/A
Residual Soils	20	$\phi' = 32°$	0.30	0.50	N/A
ELS shale/sandstone	22	$\phi' = 38°$	0.15	0.20	200 kPa

VLS to LS shale/sandstone	23	$\phi' = 40°$	0.10	0.15	400 kPa

Remarks:

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

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

4.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		No
Excavation likely to intersect	Water Table	No
	Seepage	Minor (<0.50L/min), within fill/natural soil interface and at bedrock surface.
Site Location and Topography		Low west side of the road, within moderate north-west sloping topography
Impact of development on local hydrogeology		Negligible
Onsite Stormwater Disposal		Soils unsuitable for absorption, dispersion is possible pending Hydraulic Engineer design requirements.

Remarks:

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

4.3.5 Earthworks

Recommended Compaction Levels:

Location	Minimum Dry Density Ratio
Rear yard (non-structural)	97% (Standard Compaction)

Remarks:

Site earthworks should be undertaken as per AS3798 ï 2007 *Guidelines on earthworks for commercial and residential developments.*

Where compaction is to be undertaken near existing structures or underground services, large vibratory machines should be avoided to reduce the potential for generation damaging vibrations.

Fill should be placed in loose thickness of no greater than 250mm and oversize material, organic content or over-wet soils should be excluded/treated prior to use within structural fill.

4.4. Conditions Relating to Design and Construction Monitoring:

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

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

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

4.5. Design Life of Structure:

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

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

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

It is considered that the house will have a design life of 50 years following completion of the alterations and additions.

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

- The conditions on the block donǿt change from those present at the time this report was prepared, except for the changes due to this development.
- There is no change to the property due to an extraordinary event external to this site
- The property is maintained in good order and in accordance with the guidelines set out in;
  a) CSIRO sheet BTF 18
  b) Australian Geomechanics ñLandslide Risk Managementò Volume 42, March 2007.
  c) AS 2870 ï 2011, Australian Standard for Residential Slabs and Footings

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

5. CONCLUSION:

The site investigation identified the presence of clayey sand fill underlain by medium dense/stiff to very stiff residual soils which grade to weathered bedrock. Based on DCP test results, the depth to the bedrock of minimum very low strength was interpreted between 0.30m depth to 0.55m depth for the southern part of the site and 1.18m depth to 1.94m depth at the northern part of the site.

The proposed works will include two main building structures located at the front north and front south corners of the site. The southern building will require bulk excavation of up to 3.10m depth and will require rock excavation below 0.3m depth, with less rock (and more soil) excavation to the western end. The northern building will require bulk excavation of up to 2.30m depth and will mainly extend through fill and residual soils and potentially require rock excavation in some areas.

A stability risk assessment of the site and proposed works identified potential landslip hazards relating to the proposed excavation which present a óTolerableô risk level. Implementation of engineered excavation support measures and correct construction procedures will reduce the risks associated with the excavation to óAcceptableô levels

The risks associated with the proposed development can be maintained within óAcceptableô levels with negligible impact to neighbouring or site structures provided the recommendations of this report and any future geotechnical directive are implemented. As such the site is considered suitable for the proposed construction works provided that the recommendations outlined in this report are followed.

Prepared By:
Ben Sheppard
Engineer

Reviewed By:
Troy Crozier
Principal
MEng, BSc, Dip. Civ. Eng
MAIG, PRGeo ï Geotechnical and Engineering
Registration No.: 10197

6. REFERENCES:

1. Australian Geomechanics Society 2007, ñLandslide Risk Assessment and Managementò, Australian Geomechanics Journal Vol. 42, No 1, March 2007.
2. Geological Society Engineering Group Working Party 1972, ñThe preparation of maps and plans in terms of engineering geologyò Quarterly Journal Engineering Geology, Volume 5, Pages 295 - 382.
3. C. W. Fetter 1995, ñApplied Hydrologyò by Prentice Hall. V. Gardiner & R. Dackombe 1983, ñGeomorphological Field Manualò by George Allen & Unwin
4. Australian Standard AS 2870 ï 1996, Residential Slabs and Footings ï Construction
5. Australian Standard AS1170.4 ï 2007, Part 4: Earthquake actions in Australia

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 1

CROZIER GEOTECHNICAL CONSULTANTS

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NOTES RELATING TO THIS REPORT

Introduction

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

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

Description and classification Methods

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

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

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

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

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

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

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

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

Sampling

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

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

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

Drilling Methods

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

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

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

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

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

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

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

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

Standard Penetration Tests

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

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

The test results are reported in the following form.

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

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

Cone Penetrometer Testing and Interpretation

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

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

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

The information provided on the plotted results comprises: -

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

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

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

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

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

Q_c = (12 to 18) C_u

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

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

Dynamic Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Borehole Logs

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

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

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

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

Ground Water

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

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

Engineering Reports

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

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

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

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

Site Anomalies

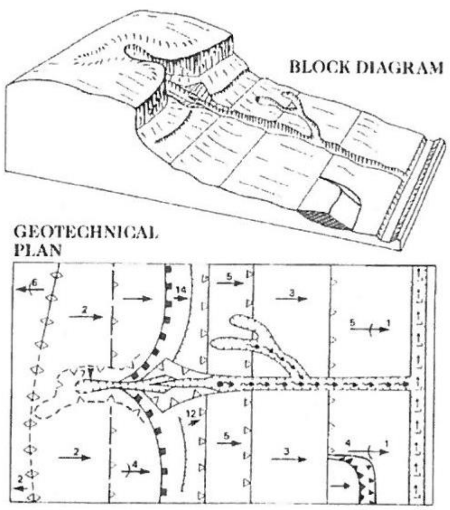
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

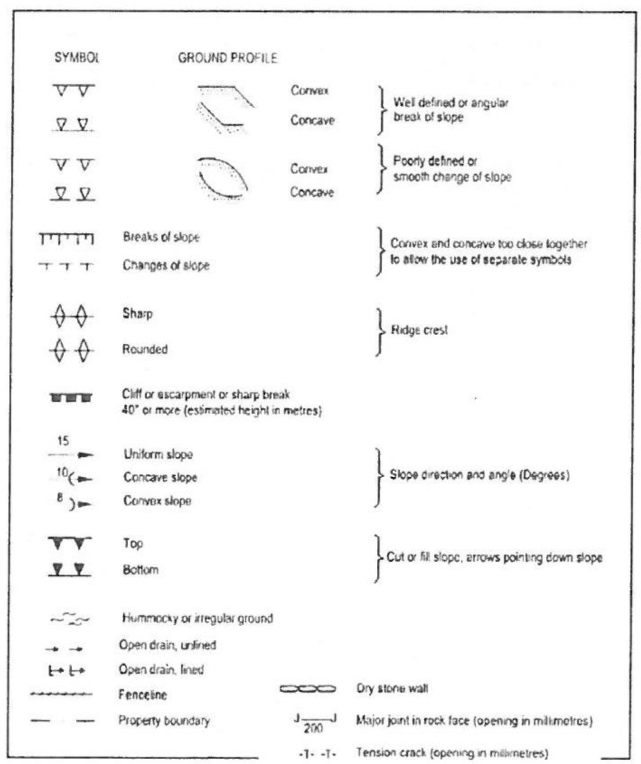
Reproduction of Information for Contractual Purposes

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

Site Inspection

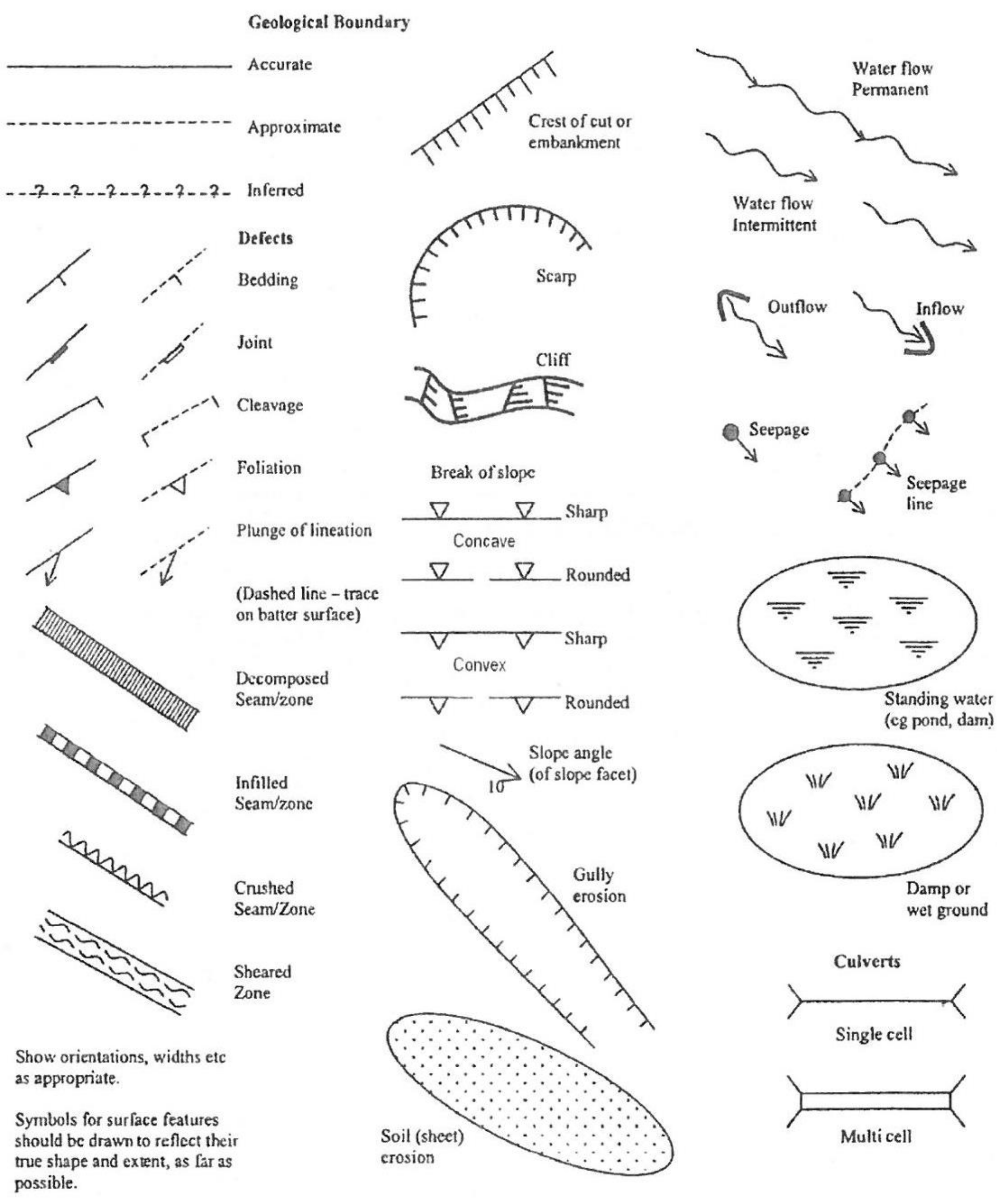
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





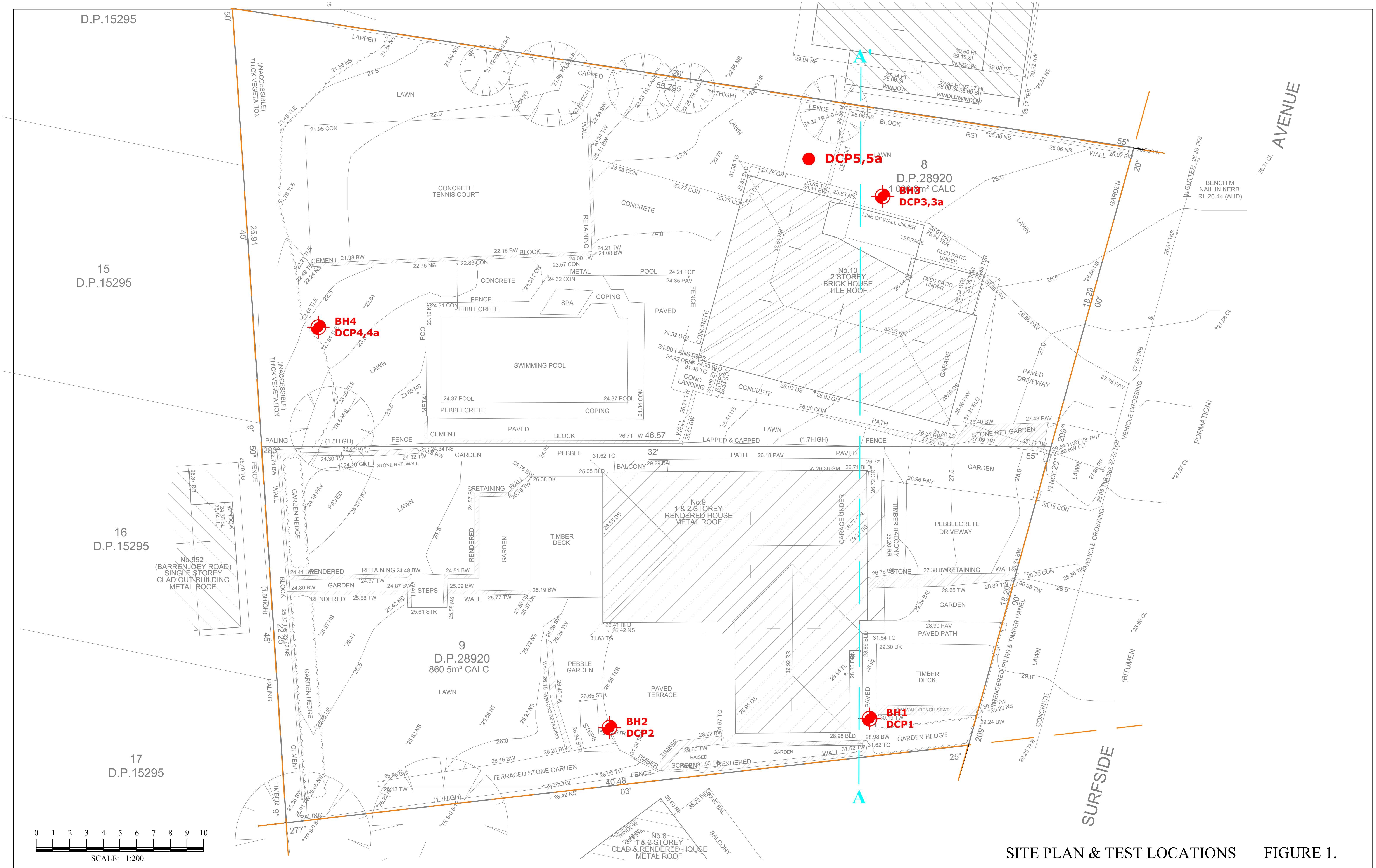
Example of Mapping Symbols
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



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

Appendix 2



SITE PLAN & TEST LOCATIONS FIGURE 1.

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

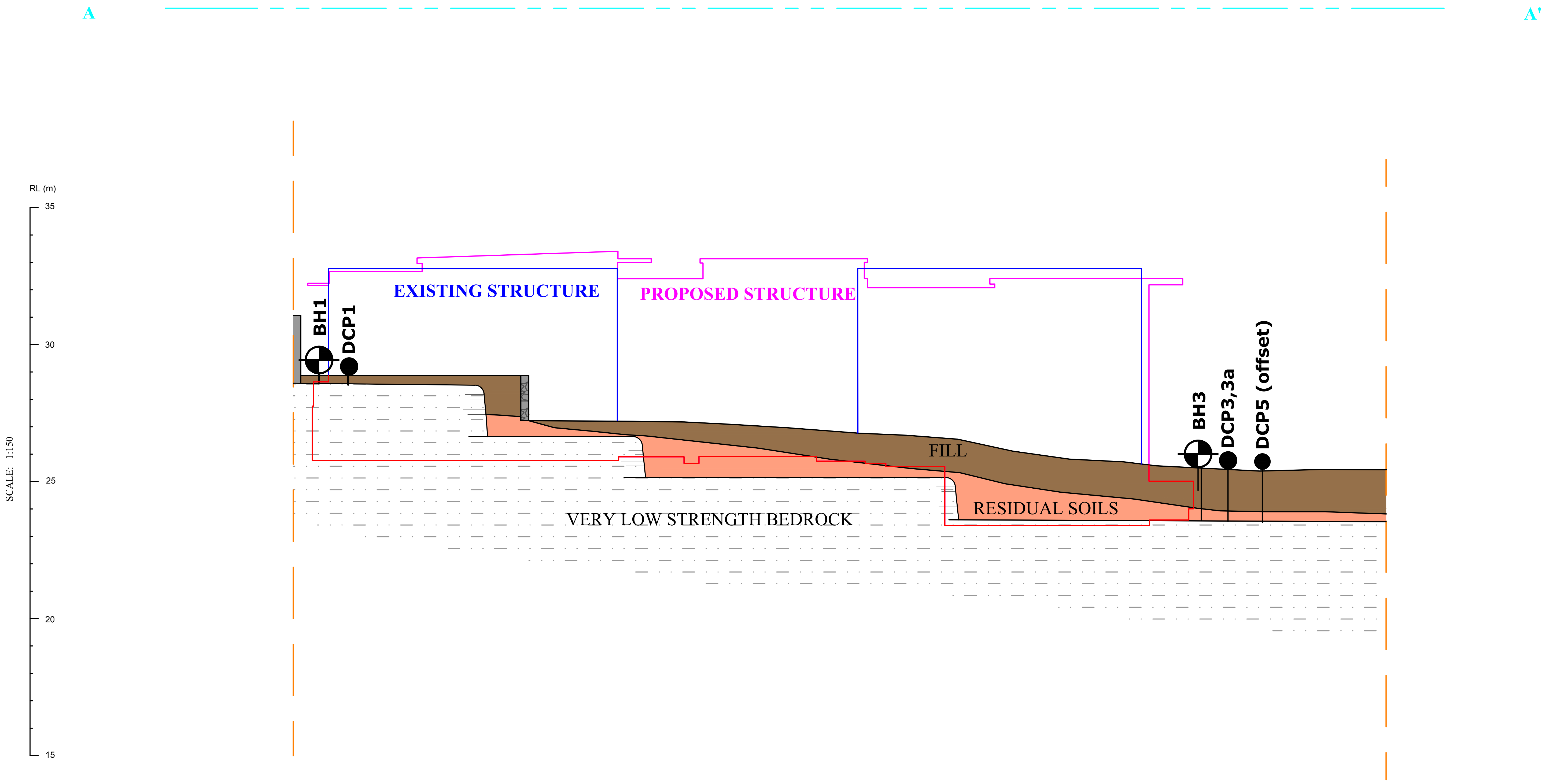
LEGEND

A — CROSS-SECTION REFERENCE LINE
PROPERTY BOUNDARY
BH DCP — AUGER / DYNAMIC CONE PENETROMETER LOCATION
DCP — DYNAMIC CONE PENETROMETER

SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	21/02/2020
APPROVED BY:	TMC
DRAWN BY:	BS
PROJECT:	2020-033

PREPARED FOR:
AMANDA STABBACK

ADDRESS:
9 & 10 SURFSIDE AVENUE,
AVALON BEACH, NSW



0 1 2 3 4 5 6 7 8 9 10

SCALE: 1:150

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 2.

CROZIER GEOTECHNICAL CONSULTANTS

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Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

LEGEND

PROPERTY BOUNDARY	EXCAVATION OUTLINE	DCP DYNAMIC CONE PENETROMETER	RESIDUAL SOILS
A — A' CROSS-SECTION REFERENCE LINE	BH DCP AUGER / DYNAMIC CONE PENETROMETER LOCATION	SOIL/FILL	BEDROCK

SCALE:	1:150 @ A3
DRAWING:	FIGURE 1
DATE:	21/02/2020
APPROVED BY:	TMC
DRAWN BY:	BS
PROJECT:	2020-033

PREPARED FOR:
AMANDA STABBACK

ADDRESS:
9 & 10 SURFSIDE AVENUE,
AVALON BEACH, NSW

BOREHOLE LOG

CLIENT: Amanda Stabback c /- Richard Cole Architecture **DATE:** 19/02/2020 **BORE No.:** 1

PROJECT: New Residential House **PROJECT No.:** 2020-033 **SHEET:** 1 of 1

LOCATION: 9 & 10 Surfside Avenue, Avalon Beach **SURFACE LEVEL:** RLå28.90

Depth (m)	Classification	Description of Strata PRIMARY SOIL - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	Tests	Type	Results
0.00		SANDSTONE PAVER				
0.10		GRAVEL - Dolerite				
0.20		FILL: Brown orange, fine to medium grained, moist clayey sand fill with trace subangular gravels				
0.30		HAND AUGER REFUSAL at 0.30m depth on Interpreted bedrock				
1.00						
2.00						

RIG: N/A DRILLER: AC

METHOD: Hand Auger LOGGED: BS

GROUND WATER OBSERVATIONS: Not Observed during drilling

REMARKS: CHECKED:

BOREHOLE LOG

CLIENT: Amanda Stabback c /- Richard Cole Architecture
DATE: 19/02/2020
BORE No.: 2
PROJECT: New Residential House
PROJECT No.: 2020-033
SHEET: 1 of 1
LOCATION: 9 & 10 Surfside Avenue, Avalon Beach
SURFACE LEVEL: RLå27.00

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	Tests	Type	Results
0.00		PEBBLES/GRAVELS				
0.20		FILL: Orange Brown, low to medium plasticity, sandy clay fill with subangular gravels		0.30		
0.30	SW	CLAYEY SAND: Very dense, pale grey, orange and red mottled, fine to medium grained, moist clayey sand with subangular ironstone gravels	D	0.40		
0.55		HAND AUGER REFUSAL at 0.55m depth on Interpreted bedrock				
1.00						
2.00						

RIG: N/A
DRILLER: AC
METHOD: Hand Auger
LOGGED: BS
GROUND WATER OBSERVATIONS: Not Observed during drilling
REMARKS:
CHECKED:

BOREHOLE LOG

CLIENT:	**Amanda Stabback c /- Richard Cole Architecture**	**DATE:**	19/02/2020	**BORE No.:**	3
PROJECT:	**New Residential House**	**PROJECT No.:**	2020-033	**SHEET:**	1 of 1
LOCATION:	**9 & 10 Surfside Avenue, Avalon Beach**	**SURFACE LEVEL:**	RLå26.00		

Depth (m) 0.00	Classification	**Description of Strata** **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
		TOPSOIL/FILL: Black, fine to medium grained, moist topsoil with roots				
0.40		...subangular ironstone gravels				
0.50		FILL: Brown orange, fine to medium grained, moist clayey sand fill with some subangular gravels				
0.60		...fragments of granite				
0.80		...becoming sandy fill with white subangular gravels				
0.85		HAND AUGER REFUSAL at 0.85m depth on interpreted cobble/boulder				
1.00						
2.00						

RIG: N/A

DRILLER: AC

METHOD: Hand Auger

LOGGED: BS

GROUND WATER OBSERVATIONS: Not Observed during drilling

REMARKS:

CHECKED:

BOREHOLE LOG

CLIENT: **Amanda Stabback c /- Richard Cole Architecture** **DATE:** 19/02/2020 **BORE No.:** 4

PROJECT: **New Residential House** **PROJECT No.:** 2020-033 **SHEET:** 1 of 1

LOCATION: **9 & 10 Surfside Avenue, Avalon Beach** **SURFACE LEVEL:** RLå22.50

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00		FILL: Light brown, fine to medium grained, moist clayey sand fill				
0.80	SM	CLAYEY SAND: Medium dense, light brown, fine to medium grained, moist clayey sand				
1.00						
1.30	CL/OL	SILTY CLAY: Stiff, light brown orange, low to medium plasticity, moist silty clay				
1.60		...1.60m - 1.70m ironstone band				
			D	1.65		
		HAND AUGER REFUSAL at 01.75m depth on Interpreted bedrock		1.75		
2.00						

RIG: N/A DRILLER: AC

METHOD: Hand Auger LOGGED: BS

GROUND WATER OBSERVATIONS: Not Observed during drilling

REMARKS: CHECKED:

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Amanda Stabback c /- Richard Cole Architecture **DATE:** 19/02/2020

PROJECT: New Residential House **PROJECT No.:** 2020-033

LOCATION: 9 & 10 Surfside Avenue, Avalon Beach **SHEET:** 1 of 1

	Test Location							
Depth (m)	1	2	3	3a	4	4a	5	
0.00 - 0.15	2	1	1	--	1	--	2	
0.15 - 0.30	15 (B) @ 0.29m depth	12	0	--	2	--	6	
0.30 - 0.45		33	1	--	2	--	2	
0.45 - 0.60		24 (B) @ 0.55m depth	2	--	10	--	2	
0.60 - 0.75			3	--	3	--	5	
0.75 - 0.90			2	--	3	--	7	
0.90 - 1.05			1	--	3	--	8	
1.05 - 1.20			3	--	3	--	16 (B) @ 1.18m depth	
1.20 - 1.35				--		5		
1.35 - 1.50				5		6		
1.50 - 1.65				8		7		
1.65 - 1.80				10		15		
1.80 - 1.95				9 (B) @ 1.90m depth		15 (B) @ 1.94m depth		
1.95 - 2.10								
2.10 - 2.25								
2.25 - 2.40								
2.40 - 2.55								
2.55 - 2.70								
2.70 - 2.85								
2.85 - 3.00								
3.00 - 3.15								
3.15 - 3.30								
3.30 - 3.45								
3.45 - 3.60								
3.60 - 3.75								
3.75 - 3.90								
3.90 - 4.05								

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

REMARKS:

(B) Test hammer bouncing upon refusal on solid object

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

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Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (earth slide $<1m^3$) from soils due to excavation for the southern building (ground floor)		Excavation (<0.60m) of clayey sand fill/soils for new ground floor levels of the northern building extending to within 1.00m of the southern boundary	a) house <2.0m from south side of 3.10m deep excavation, impact northern corner only b) lawn and garden adjacent to excavation, impact 5% c) Surfside Avenue Reserve >6.5m from eastern side of 3.10m deep excavation		a) Person in house 10hrs/day avge. b) Person in garden 1hr/day avge. c) Person in road reserve 1hr/day avge.	a) Possible to not evacuate b) Possible to not evacuate c) Possible to not evacuate	a) Person in building, minor damage only b) Person in open space, not buried c) Person in open space, not buried	
			Possible	**Prob. of Impact**	**Impacted**				
		a) House No. 8 Surfside Ave	0.001	0.05	0.01	0.4167	0.5	0.1	**5.21E-09**
		b) Front and rear lawn No. 8 Surfside Ave	0.001	0.25	0.01	0.0417	0.5	0.1	**5.21E-09**
		c) Road reserve (Surfside Ave)	0.001	0.001	0.01	0.0417	0.5	0.1	**1.04E-11**
B	Landslip (rock slide/topple $<1m^3$) within rock excavation for the southern building (ground floor)		Rock excavations up to 2.8m depth expected, likely unfavourable defects in some portion	Small slide/topple could impact up to 15% of the excavation		a) Person in excavation 8hrs/day during construction	Unlikely to not evacuate	Person in open space, crushed	
			Possible	**Prob. of Impact**	**Impacted**				
		a) Construction, base of excavation	0.001	1.00	0.15	0.2619	0.25	1	**9.82E-06**
C	Landslip (earth slide $<3m^3$) from soils due to excavation for the northern building (garden storage room)		Excavation (<2.30m) of clayey sand fill/soils for new garden storage area under the northern building extending to within 6.00m of the north boundary	a) house >6.50m from north side of 2.30m deep excavation, impact south side only b) rear lawn and garden >6.50m from north side of 2.30m deep excavation, impact 5% c) Surfside Avenue reserve >6.5m from eastern side of 2.40m deep excavation		a) Person in house 10hrs/day avge. b) Person in garden 1hr/day avge. c) Person in road reserve 1hr/day avge.	a) Possible to not evacuate b) Possible to not evacuate c) Possible to not evacuate	a) Person in building, minor damage only b) Person in open space, not buried c) Person in open space, not buried	
			Likely	**Prob. of Impact**	**Impacted**				
		a) House No. 11 Surfside Ave	0.01	0.05	0.05	0.4167	0.5	0.1	**2.60E-07**
		b) Rear lawn and garden No.11 Surfside Ave	0.01	0.05	0.05	0.0417	0.5	0.1	**5.21E-08**
		c) Road reserve (Surfside Ave)	0.01	0.05	0.05	0.0417	0.5	0.1	**5.21E-08**

If the area/structure is located well down hill and the slide coul dstop before reaching it then a low Prob. of Impact is relevant (i.e. 0.05).

Where a 1.0m wide boulder impacts a 10m long wall then the Impacted value is 1 / 10 = 0.1

Spatial impact of Vehicle — vehicle on section of road impacted at time of slide (i.e. slide impacting vehicle from above or below as the vehicle passes a point).

Nv / 24 * L / 1000 * 1 / Vv

- Nv — Number of vehicles passing along the length of road in 1 day
- L — Length of vehicle (car average is 5.5m)
- Vv — Velocity of vehicle (i.e. expected speed - 40km/hr etc)

Occupancy — For a travelling vehicle the occupancy is 1.0, for a bedroom it might be 10hrs/day (i.e. 0.417).

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (earth slide <1m^3) from soils due to excavation for the southern building (ground floor)	a) House No. 8 Surfside Ave	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low
		b) Front and rear lawn No. 8 Surfside Ave	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low
		c) Road reserve (Surfside Ave)	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low
B	Landslip (rock slide/topple <1m^3) within rock excavation for the southern building (ground floor)	a) Construction, base of excavation	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, requires large stabilising works or MINOR damage to neighbouring property.	Low
C	Landslip (earth slide <3m^3) from soils due to excavation for the northern building (garden storage room)	a) House No. 11 Surfside Ave	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low
		b) Rear lawn and garden No.11 Surfside Ave	Unlikely	The event might occur under very adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Low
		c) Road reserve (Surfside Ave)	Unlikely	The event might occur under very adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low

TABLE: 2

Recommended Maintenance and Inspection Program

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

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

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Appendix 4

APPENDIX A

DEFINITION OF TERMS

INTERNATIONAL UNION OF GEOLOGICAL SCIENCES WORKING GROUP ON LANDSLIDES, COMMITTEE ON RISK ASSESSMENT

Risk – A measure of the probability and severity of an adverse effect to health, property or the environment. Risk is often estimated by the product of probability x consequences. However, a more general interpretation of risk involves a comparison of the probability and consequences in a non-product form.

Hazard – A condition with the potential for causing an undesirable consequence (*the landslide*). The description of landslide hazard should include the location, volume (or area), classification and velocity of the potential landslides and any resultant detached material, and the likelihood of their occurrence within a given period of time.

Elements at Risk – Meaning the population, buildings and engineering works, economic activities, public services utilities, infrastructure and environmental features in the area potentially affected by landslides.

Probability – The likelihood of a specific outcome, measured by the ratio of specific outcomes to the total number of possible outcomes. Probability is expressed as a number between 0 and 1, with 0 indicating an impossible outcome, and 1 indicating that an outcome is certain.

Frequency – A measure of likelihood expressed as the number of occurrences of an event in a given time. See also Likelihood and Probability.

Likelihood – used as a qualitative description of probability or frequency.

Temporal Probability – The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.

Vulnerability – The degree of loss to a given element or set of elements within the area affected by the landslide hazard. It is expressed on a scale of 0 (no loss) to 1 (total loss). For property, the loss will be the value of the damage relative to the value of the property; for persons, it will be the probability that a particular life (the element at risk) will be lost, given the person(s) is affected by the landslide.

Consequence – The outcomes or potential outcomes arising from the occurrence of a landslide expressed qualitatively or quantitatively, in terms of loss, disadvantage or gain, damage, injury or loss of life.

Risk Analysis – The use of available information to estimate the risk to individuals or populations, property, or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification, and risk estimation.

Risk Estimation – The process used to produce a measure of the level of health, property, or environmental risks being analysed. Risk estimation contains the following steps: frequency analysis, consequence analysis, and their integration.

Risk Evaluation – The stage at which values and judgements enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental, and economic consequences, in order to identify a range of alternatives for managing the risks.

Risk Assessment – The process of risk analysis and risk evaluation.

Risk Control or Risk Treatment – The process of decision making for managing risk, and the implementation, or enforcement of risk mitigation measures and the re-evaluation of its effectiveness from time to time, using the results of risk assessment as one input.

Risk Management – The complete process of risk assessment and risk control (*or risk treatment*).

Individual Risk – The risk of fatality or injury to any identifiable (named) individual who lives within the zone impacted by the landslide; or who follows a particular pattern of life that might subject him or her to the consequences of the landslide.

Societal Risk – The risk of multiple fatalities or injuries in society as a whole: one where society would have to carry the burden of a landslide causing a number of deaths, injuries, financial, environmental, and other losses.

Acceptable Risk – A risk for which, for the purposes of life or work, we are prepared to accept as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks justifiable.

Tolerable Risk – A risk that society is willing to live with so as to secure certain net benefits in the confidence that it is being properly controlled, kept under review and further reduced as and when possible.

In some situations risk may be tolerated because the individuals at risk cannot afford to reduce risk even though they recognise it is not properly controlled.

Landslide Intensity – A set of spatially distributed parameters related to the destructive power of a landslide. The parameters may be described quantitatively or qualitatively and may include maximum movement velocity, total displacement, differential displacement, depth of the moving mass, peak discharge per unit width, kinetic energy per unit area.

Note: Reference should also be made to Figure 1 which shows the inter-relationship of many of these terms and the relevant portion of Landslide Risk Management.

APPENDIX C: LANDSLIDE RISK ASSESSMENT

QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

Approximate Annual Probability		Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
Indicative Value	**Notional Boundary**					
10^{-1}	$5x10^{-2}$	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}	$5x10^{-3}$	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	$5x10^{-4}$	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	$5x10^{-5}$	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	$5x10^{-6}$	100,000 years	200,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}		1,000,000 years		The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

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

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

RISK LEVEL IMPLICATIONS

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

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