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REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

PROPOSED NEW GARAGE

at

119 HUDSON PARADE, CLAREVILLE, NSW

Prepared For

Mia Asker

Project No.: 2025-086
June, 2025

Document Revision Record

Issue No	Date	Details of Revisions
0	19 June 2025	DA issue

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

Development Application for____

Name of Applicant

Address of site 119 Hudson Parade, Clareville

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 __20 June 2025__ certify that I am a engineering geologist as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above company to issue this document and to certify that the company has a current professional indemnity policy of at least $2million.

I:

- ☐ have prepared the detailed Geotechnical Report referenced below in accordance with the Australia Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ am willing to technically verify that the detailed Geotechnical Report referenced below has been prepared in accordance with the Australian Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009
- ☐ have examined the site and the proposed development in detail and have carried out a risk assessment in accordance with Section 6.0 of the Geotechnical Risk Management Policy for Pittwater - 2009. I confirm that the results of the risk assessment for the proposed development are in compliance with the Geotechnical Risk Management Policy for Pittwater - 2009 and further detailed geotechnical reporting is not required for the subject site.
- ☐ have examined the site and the proposed development/alteration in detail and I am of the opinion that the Development Application only involves Minor Development/Alteration that does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.
- ☐ have examined the site and the proposed development/alteration is separate from and is not affected by a Geotechnical Hazard and does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.
- ☐ have provided the coastal process and coastal forces analysis for inclusion in the Geotechnical Report

Geotechnical Report Details:

Report Title: Geotechnical Report for Proposed Demolition of existing garage and construction of a new two car garage

Report Date: 19 June 2025 **Project No.:** 2025-086

Author: S. Bohara and T. Crozier

Author's Company/Organisation: Crozier Geotechnical Consultants

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

Architectural: Askerrobertson Design & Construction; Drawing No.: A02/A to A16/A; Dated: 18.06.2025
Survey: Waterview Surveying Services, Project No.: 2007, Dated: 27/02/2025

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

Signature ....

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

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

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

Company... **Crozier Geotechnical Consultants**

AUSTRALIAN INSTITUTE OF GEOSCIENTISTS

TROY CROZIER

10197

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

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

Development Application for ________________________________

Name of Applicant

Address of site ___119 Hudson Parade, Clareville ____________

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 Demolition of existing garage and construction of a new two car garage

Report Date: 19 June 2025 **Project No.:** 2025-086

Author: S. Bohara and T. Crozier

Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

- ■ Comprehensive site mapping conducted ___20.05.2025_________ (date)
- ■ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
- ■ Subsurface investigation required
  - ☐ No Justification ....
  - ■ Yes Date conducted ......Preliminary conducted 20.05.2025..................
- ■ Geotechnical model developed and reported as an inferred subsurface type-section
- ■ Geotechnical hazards identified
  - ☐ Above the site
  - ■ On the site
  - ☐ Below the site
  - ☐ Beside the site
- ■ Geotechnical hazards described and reported
- ■ Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
  - ■ Consequence analysis
  - ■ Frequency analysis
- ■ Risk calculation
- ■ Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved.
- ■ Design Life Adopted:
  - ■ 100 years
  - ☐ Other .... specify
- ■ Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
- ■ Additional action to remove risk where reasonable and practical have been identified and included in the report.
- ☐ Risk assessment within Bushfire Asset Protection Zone.

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

Signature ....

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

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

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

Company... **Crozier Geotechnical Consultants**

TROY ... 10197

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

Date: 19 June 2025
Project No: 2025-086

GEOTECHNICAL REPORT FOR PROPOSED DEMOLITION OF EXISTING GARAGE AND CONSTRUCTION OF NEW TWO CAR GARAGE 119 HUDSON PARADE, CLAREVILLE, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for a proposed new two car garage at 119 Hudson Parade, Clareville, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the written request of the architect Mia Asker.

The proposed works involve the demolition of the existing garage structure and construction of a new two car garage and adjacent external staircase. The proposed garage structure will be positioned in the approximate location of the existing garage however appears to require bulk excavation within northwestern regions to 3.40m depth.

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).

To meet the Councils Policy requirements for land classified as H1, a detailed Geotechnical Report which meets the requirements of Paragraph 6.5 of that policy must be submitted with a Development Application (DA). This report is provided for DA submission and includes a landslide risk assessment to the methods of AGS 2007 for the site and proposed works, plans, geological sections and provides recommendations for construction and to ensure stability is maintained for a preferred design life of 100 years to meet the policy requirements. It is recommended that the client make themselves aware of the Policy and its requirements.

The assessment and reporting were undertaken as per the Fee Proposal No.: P25-198, Dated: 5th May 2025. The investigation comprised:

a) Before You Dig Australia (BYDA) plan review and onsite clearance of test locations by an accredited service locating sub-contractor.
b) A detailed geotechnical inspection and mapping of the site and inspection of adjacent properties, with identification of geotechnical conditions and hazards including landslip related to the existing site and proposed works by a Geotechnical Engineer.

c) Drilling of three auger boreholes to identify sub-surface geology using hand tools due to site access limitation.
d) Dynamic Penetrometer (DCP) testing at four locations to investigate subsurface conditions and confirm depth to bedrock.
e) Soil sample collection and logging as per "AS1726: 2017 Geotechnical Site Investigation"

The following documents, plans and drawings were supplied and were relied on for the proposal, investigation and reporting:

- Survey Drawing – Waterview Surveying Services, Project No.: 2007, Dated: 27/02/2025
- Architectural Drawings – Askerrobertson Design & Construction, Drawing No.: A02/A – A16/A; Dated: 18/06/2025.

2. PROPOSED WORKS

The proposed works involve the demolition of existing garage and the construction of a new two car garage, adjacent external staircase and a new retaining wall along the front boundary.

It is understood that the proposed garage will require excavation to a maximum depth of approximately 3.40 m. This excavation will be located at the front west portion within the site.

The proposed excavation will be set back approximately 5.47 m from the northern boundary shared with a public pathway and the No. 117 Hudson Parade with the nearest structure positioned an additional 12.75 m further north.

The proposed excavation will be set back approximately 4.0 m from the southern boundary with No. 119 Hudson Parade, and the closest structure on that property is positioned 10.0 m further away.

The proposed excavation will extend to the western boundary along Hudson Parade though will reduce to ≤0.50m in depth, where a nature strip is directly adjacent to the boundary and the road pavement is located an additional 6.0 m further west.

The proposed excavation will be located over 60 m from the eastern boundary shared with No. 88 Hilltop Road, with the nearest neighbouring structure situated approximately 60.0 m beyond the boundary, offering substantial separation from potential effects.

3. SITE FEATURES:

3.1. Description:

The site (119 Hudson Parade, Clareville, Lot 2 DP540557) is a trapezoidal shaped block situated on the east side of Hudson Parade within moderate (14°) north-west dipping topography. It is situated at mid-slope of a steep west dipping slope, near the west of a secondary west plunging ridgeline with natural north-west plunging gully located to the south west. The survey indicates ground surface levels within the site vary from a high of RL27.73 along the east boundary to a low of RL16.59 at the north-west corner.

From the provided plans, the site has an angular front west boundary of 19.245m, a north boundary of 60.851m, a south side boundary of 67.095m and a rear east boundary of 16.75m. An aerial photograph of the site and its surrounds is provided in Photograph 1, as sourced from NSW Government Six Maps Spatial Data.

Photograph 1: Aerial view of site and surrounds (NSW Government Six Map Spatial Data)

3.2. Geology:

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is underlain by weathered bedrock of the Newport Formation (Upper Narrabeen Group) rock (Rnn) which is of middle Triassic Age shown in Extract-1 below. The Newport Formation typically comprises interbedded laminite, shale and quartz to lithic quartz sandstones and pink clay pellet sandstones and tends to weather to significant depth.

Narrabeen Group rocks are dominated by shales and thin siltstone/sandstone beds and often form rounded convex ridge tops with moderate angle (<20°) side slopes. These side slopes can be either concave or convex depending on geology, internally they comprise of interbedded shale and siltstone/sandstone beds with close spaced bedding partings that have either close spaced vertical joints or in extreme cases large space convex joints.

An extract from the Sydney 1:100,000 Geological Series sheet 9130 provided below (Extract 1) indicates the geology underlying the site and surrounding area.

Extract 1: Sydney (9130 Geology Series Map): 1:100,000 – Geology underlying the site

4. FIELD WORK:

4.1. Methods:

The field investigation comprised a geotechnical inspection, mapping of the site and adjacent properties on 20th May 2025 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 existing features and ground conditions.

The subsurface investigation included the drilling of three auger boreholes (BH1 – BH3) within the front of the site to investigate the sub-surface geology using hand tools due to restricted access.

Geotechnical logging of the subsurface conditions was undertaken by a Geotechnical Engineer by inspection of disturbed soil recovered from the augers. Logging was undertaken in accordance with AS1726:2017 'Geotechnical Site Investigations'.

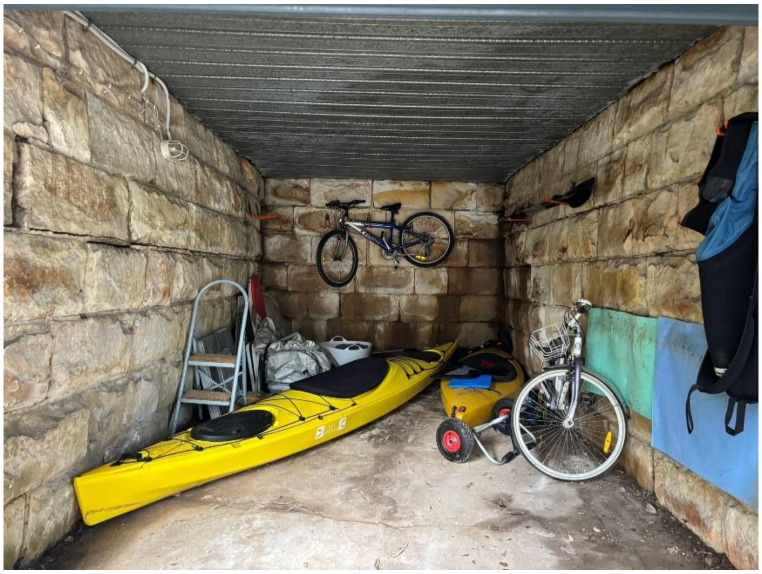
Dynamic Cone Penetrometer (DCP) testing at four locations was carried out from the ground surface adjacent to the boreholes and an additional one in accordance with AS1289.6.3.2 – 1997, "Determination of the penetration resistance of a soil – 9kg Dynamic Cone Penetrometer" to estimate near surface soil conditions.

Explanatory notes are included in Appendix: 1. Mapping information and test locations are shown on Figure: 1, along with detailed borehole logs and DCP sheets in Appendix: 2. A geological model/section is provided as Figure: 2, Appendix: 2.

4.2. Field Observations:

Hudson Parade is formed with a bituminous sealed pavement that is gentle east dipping and separated from the site by a concrete kerb and a nature walkway. The roadway showed some signs of cracking and deterioration but did not exhibit any obvious signs of significant cracking or excessive settlement to indicate any underlying geotechnical concerns that may impact the site.

The front portion of the site at No. 119 Hudson Parade, Clareville, comprises a concrete driveway and a sandstone block staircase adjacent to a detached garage structure that currently provides access to the property. The garage is estimated to be approximately 100 years old and is excavated into the slope, with external retaining walls constructed from sandstone blockwork, while the roof is of more recent concrete construction. The remaining front area consists primarily of gardens and vegetated surfaces supported by the retaining wall along the boundary, with pedestrian access to the dwelling and rear portion of the site provided via a stone-paved pathway. The garage walls exhibit signs of bowing and structural distortion (refer to Photograph 2), and the retaining wall along the front west boundary also appears to show evidence of cracking.



Photograph 2: View of existing garage, facing southeast from Hudson Parade

The one and two storey sandstone masonry dwelling is located at centre of the site with a brick paved front terrace. Observations suggest that the front portion of the structure is a timber-framed extension to the original rear section, which is approximately 100 years old. The terrace is elevated approximately 3.5 m above the front gardens and is retained by a sandstone block retaining wall. At the time of inspection, the main structure appeared to be in good condition with no evident signs of differential settlement or structural cracking. However, the terrace showed minor cracking and indications of localized settlement, likely associated with the rotation of the supporting retaining wall (Photograph 5).

A rendered retaining wall ≤3.0m high is positioned to the immediate rear of the main structure. The retaining wall has been constructed on an approximate and variable 1:4 near-vertical batter and appears likely to be of shotcrete construction. While weep holes are positioned near the base of the retaining wall, no formal drainage system was noted at the rear of the dwelling. There was no indication of anchoring within the wall.

The rear of the site is the highest portion of the property and contains dense vegetation and two wooden sheds.

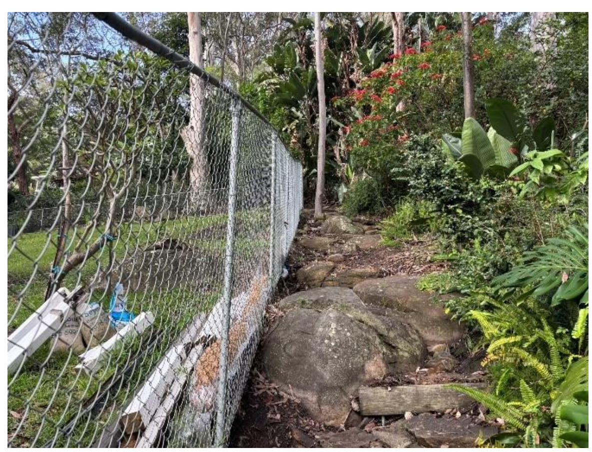
Photograph 3: View of No. 119 Hudson Parade front, facing east from Hudson Parade.

Photograph 4: View of the addition to the old structure, facing west

Photograph 5: View of the terrace retaining wall showing signs of rotation, facing northwest

Photograph 6: View of the retaining wall at the rear, facing south

Photograph 7: View of the retaining wall at front boundary, facing east from Hudson Parade



Photograph 8: View of the boulders in the Public Pathway, facing southeast

To the north of the site lies a 4.5 m wide public pathway surfaced with grass and concrete stairs, which provides pedestrian connectivity between Hudson Parade and Hilltop Road. The pathway also contains exposed sandstone boulders embedded in the soil slope, however there were no signs of instability or movement. This corridor accommodates a sewer main and functions as a natural drainage gully, facilitating surface runoff along the local topography.

The neighbouring properties to the north (No. 117 Hudson Parade) contains two and three storey clad and weatherboard dwelling of fairly new construction. The existing building is set back approximately 6.0m from the shared boundary of the site. The front of the property includes a landscaped garden area retained via an approximate 0.5 m^3 sandstone blocks adjacent to a concrete driveway, as well as vegetation. The area also features mature trees and dense vegetation. Ground surface levels appear to be slightly lower than those on the subject site. Visible elements of the neighbouring structures appeared in good condition, with no indications of significant cracking or differential settlement that would suggest underlying geotechnical issues.

The neighbouring property to the south (No. 121 Hudson Parade) contains a two-storey timber structure of relatively recent construction. The frontage of the property includes a vegetated area and a concrete driveway. The main structure is set back approximately 5.0 m from the shared boundary. A dwelling is also located at the rear of the property and is currently under construction. Ground levels along the boundary appear to be generally consistent with those of the subject site. Visual inspection of the observable portions of the existing structure revealed no noticeable signs of cracking or differential settlement, indicating no apparent geotechnical distress.

The neighbouring property to the east (No. 88 Hilltop Road) comprises a one to two storey weatherboard dwelling. Ground surface levels of the property were higher compared to site, with several embedded sandstone boulders observed in proximity to the shared boundary. The rear portion of the property contains mature trees and dense vegetation. The existing dwelling is located more than 15 m from the boundary, and no geotechnical hazards were observed during the site inspection.

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

4.3. Ground Conditions:

Three boreholes (BH1 – BH3) were undertaken within the front north-west portion of the site in the envelope of proposed works. The boreholes advanced through an initial layer of topsoil underlain by silty sand interpreted as colluvial material, before encountering natural clayey sand, and subsequently transitioning into sandy clay. Drilling was terminated at a maximum depth of 0.95 m due to practical refusal on the interpreted very low strength sandstone bedrock.

Dynamic Cone Penetrometer (DCP) tests were undertaken from the ground surface adjacent to the borehole locations and an additional one location with soft bounce refusal on interpreted bedrock of probable very low strength encountered at between 1.09m increasing to 1.45m, with the exception of DCP3, which terminated due to practical refusal in interpreted extremely weathered material.

For a description of the ground conditions encountered at the individual borehole/DCP test locations, the Borehole Log and DCP results sheets should be consulted however the subsurface conditions at the site can be summarised as follows:

- **TOPSOIL/FILL** – This layer was encountered in all boreholes from ground surface level extending to a maximum depth of 0.50m (BH2) and generally comprised a dark brown, fine to medium grained, moist silty sand with gravel.
- **Silty SAND –** The layer was encountered in boreholes BH1 and BH3 from 0.40m depth and is interpreted as a colluvial deposit. In BH1, it comprises very loose, brown material with inclusions of clay and gravel, grading to medium dense from approximately 0.50 m depth. In BH3, the deposit is described as very loose, dark brown to yellow-brown, and similarly considered to be of colluvial origin.
- **Clayey SAND –**This layer was encountered in BH1 & BH3, generally underlying the fill and colluvial layers, a zone of clayey SAND was observed, varying from loose to medium dense, and described as brown to red-brown, fine to medium grained, moist, with trace amounts of gravel in some instances. This unit generally extended to depths of approximately 0.60 m to 0.75 m
- **Sandy CLAY –** This layer was encountered in BH1 & BH3, generally described as stiff to very stiff, brown to yellow-brown, moist, with low to medium plasticity and occasional gravel bands. This unit typically extended to the base of the boreholes, up to 0.95 m depth, and represents weathered residual soil transitioning toward more competent material.
- **SANDSTONE/SILTSTONE** – Interpreted bedrock of at least very low strength was not encountered in BH1 or in the DCP, with termination depths ranging from 0.90 m in the borehole to between 1.09 m (DCP1) and 1.36 m (DCP2) in the DCP tests. The bedrock was interpreted from refusal conditions.

A free-standing groundwater table or significant seepage was not encountered in any of the boreholes.

5. COMMENTS:

5.1. Geotechnical Assessment:

The site investigation encountered a topsoil/fill and colluvial layer in all boreholes, extending from the surface to depths ranging between 0.50 m and 0.65 m (BH1). Beneath the fill and colluvial, residual soils were identified, including clayey sand and sandy clay, commencing from depths as shallow as 0.50 m (BH3) and extending to a maximum depth of 0.95m in BH3. These soils varied in composition, strength, and moisture condition across the site and were underlain by interpreted sandstone/siltstone bedrock of probable very low strength, encountered between 1.09m (DCP1) and 1.36m depth (DCP2).

A groundwater table was not encountered to the base investigation level and is not expected based on the site location and local topography; however, seepage is expected across the bedrock surface as well as along defects in the bedrock mass. The investigation, site location and topography indicate seepage is unlikely to be significant as seen within the current garage excavation.

The proposed works involve demolition of the existing garage, and the construction of a new two car garage. The proposed garage finished floor level will be situated at RL16.20 hence will require a bulk excavation to a maximum of approximately 3.40m depth.

The bedrock of the local area is anticipated from previous experience to be a combination of low strength sandstone and very low strength siltstone initially, grading to low and medium strength with depth. However, it is known to contain areas of increased weathering, strength inversions (hard rock over weak) and steeply inclined defects.

Based on investigation results and expected geological conditions, safe batter slopes as per Section 5.3.2 of this report appear to be achievable in all parts of the site; therefore, pre-excavation support systems will not be required for the excavation of the new garage.

Fill, natural soils and very low strength bedrock can be excavated using conventional earthmoving equipment (e.g. buckets and rippers) whilst low strength siltstone can be excavated by heavy ripping, however low to medium 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 adjacent structures if unsuitable sizes and methods are used. Care will be required during the demolition and excavation, and construction works to ensure the neighbouring properties and potential structures/services are not adversely impacted by ground vibrations. However, this is not anticipated unless very large(>1000kg) rock hammers are used on this site.

Small scale equipment (i.e. rock hammer <300kg) along with rock saw and a good excavation methodology can be used to maintain low vibration levels at boundaries and avoid the need for full time vibration monitoring. However, this will result in slow excavation progress, and it is anticipated that larger scale rock hammers will be preferred. As such Crozier Geotechnical Consultants (CGC) should be consulted regarding the size and type of excavation equipment proposed and excavation methodology prior to works.

Where medium strength bedrock with no poorly oriented defects is identified by further investigation, it will likely be free standing and can be excavated near vertically without the need for additional support measures. Where defects are encountered additional support may be required (i.e. rock bolts) to maintain stability. While cored boreholes are not considered critical for this relatively small-scale excavation, they may be beneficial if a higher level of confidence in subsurface conditions is desired. Otherwise, excavation may proceed with an allowance for interpretation based on available data.

The proposed excavation is anticipated to intersect bedrock across its majority at foundation level; therefore, its entire base should be founded to bedrock of similar strength to avoid differential settlement.

Voids adjacent to the proposed garage, particularly near the existing retaining wall at the rear east of the excavation, should be backfilled with granular material, preferably clean crushed sandstone, placed in controlled layers. This material is recommended for its favourable drainage characteristics and compaction properties, which are essential for limiting long-term settlement and maintaining ground stability in the vicinity of the retaining wall and proposed structure.

To achieve uniform density and structural performance of the backfill, a plate compactor should be used during placement. Proper compaction using a 50 mm plate compactor is critical to reduce the potential for post-construction ground movement and to ensure the backfill provides sufficient lateral support to the retaining wall.

Based on the site investigation, a groundwater table is not anticipated within the proposed excavation however, minor seepage is likely to be encountered during the excavation principally at the bedrock surface and on defects in the rock. Initial investigation results indicate this can be managed as a drained basement with negligible impact on local hydrogeology.

A local sewer line runs south of the proposed excavation near the garage. If found to be within the excavation zone of influence, diversion is recommended to avoid conflict. If diversion is not feasible, appropriate protection measures should be implemented.

The proposed works are considered suitable for the site and may be completed with negligible impact to existing nearby structures within the site or 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 a limited scope of investigation using auguring techniques only. This investigation provides limited data from small, isolated test points across the entire site with limited penetration into rock, 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 assessment and subsequent initial design of the proposed works.

5.2. Site Specific Risk Assessment:

There were no signs of existing or previous landslip instability within the site or adjacent land whilst the existing house structures show no signs of settlement or cracking. The proposed works require a deep excavation that has potential to result in instability where not properly supported.

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

A. Landslide (earth slide <$3m^3$) from soils at crest of the excavation
B. Landslide (rock and earth slide <$10m^3$) due to instability within the bedrock

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

The Risk to Life from Hazard A was estimated to be up to **1.56×10^{-7},** whilst the Risk to Property was considered to be **up to 'Low'**. The Risk to Life from Hazard B was also estimated to be up to **1.56×10^{-6},** and the Risk to Property up to **'Moderate'**.

The hazards were therefore considered to be '**Unacceptable**' when assessed against the criteria of the Councils Policy. **However**, it should be noted that this assessment considers the excavations permanently unsupported, therefore actual risk levels will be significantly lower through construction of engineered pre-excavation support systems that will ensure "Acceptable" risk criteria will be achieved and maintained.

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

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 founded into bedrock at base of excavation.
Type of Footing	Strip, Pad, or Piers
Sub-grade material and Maximum Allowable Bearing Capacity for shallow footings	- Very Stiff clay: 300 kPa - Stiff clay: 400 kPa - Very Low Strength Bedrock: 700kPa - Low Strength bedrock: 1000kPa*
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	Be – rock site The hazard factor (z) for Sydney is 0.08.
Remarks: *Unconfirmed, requires cored investigation or inspection upon bulk excavation All footings should be founded off material of similar foundation conditions to prevent differential settlement, allowance for differential settlement should be designed for if the structure is variably founded. All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata. This is mandatory to allow them to be 'certified' at the end of the project.	

5.3.2. Excavation:				
Depth of Excavation		Maximum of approximately 3.40m depth for the proposed Garage.		
Property Separation:				
Boundary	**Adjacent Property**	**Bulk Excavation Depth**	**Separation Distances**	
			Boundary	**Structure**
North	No. 117 Hudson Parade & Public Pathway	Up to 3.40m for garage	5.47m	Pathway is adjacent to the boundary and Structure is further +6.0m away.

South	No. 121 Hudson Parade	Up to 3.40m for garage	4.0m	Structure is +5.0m away.
West	Hudson Parade	0.0m for garage	0m	Nature strip adjacent to the boundary, road pavement a further 6.0m away.
East	No. 88 Hilltop Road	3.40m for garage	<60m	Structure is +60m away.
	Site terrace		NA	Terrace is +8.0m away.

Type of Material to be Excavated	Fill/topsoil to 0.65m depth
	Residual soils to extremely weathered bedrock to a maximum of 0.95m depth
	Very low to low strength bedrock from a minimum of 1.1m depth.

Guidelines for unsurcharged batter slopes are tabulated below:

Material	Safe Batter Slope (H: V)	
	Short Term/Temporary	Long Term/Permanent
Fill/topsoil	1.5:1	2:1
Residual soils to EW material	1:1*	1.5:1*
VLS – LS bedrock (fractured)	0.25:1.0*	0.50:1.0*

Remarks:

*Dependent on assessment by geotechnical engineer.

Batter slopes in soils should be ≤3.0m in height without benching and where utilised will require regular geotechnical assessment and protection from saturation

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	Excavator with bucket
	VLS bedrock	Excavator with bucket and ripper
	LS-MS/HS bedrock	Rock hammer and rock saw

VLS – very low strength, LS – low strength, MS – medium strength

Remarks:

Rock sawing of the hard rock excavation perimeters 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 and the saw

cuts will provide a slight increase in buffer distance for use of rock hammers. It also reduces deflection across boundary of detached sections of bedrock near surface. Based on previous testing of ground vibrations created by various rock excavation equipment within medium strength bedrock, to achieve a low level of vibration (5mm/s PPV) the below hammer weights and buffer distances are generally required:	

Maximum Hammer Weight	Required Buffer Distance from Structure
300kg	3.00m
400kg	4.00m
600kg	6.00m
≥1 tonne	20.00m

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. Inspection of equipment and review of dilapidation surveys and excavation location is necessary to determine need for full time monitoring.	
Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))	Site Structure = 5mm/s No. 117 Hudson Parade = 5mm/s No. 121 Hudson Parade = 5mm/s
Vibration Calibration Tests Required	If larger scale (i.e. rock hammer >300kg) excavation equipment is proposed
Full time vibration Monitoring Required	Pending proposed excavation equipment and vibration calibration testing results, if required
Geotechnical Inspection Requirement	Yes, recommended that these inspections be undertaken as per below mentioned sequence: • At 1.50m depth intervals of excavation for assessment of batter slopes • Where temporary slope support is required • Where unexpected ground conditions are identified, or any other concerns are held. • At completion of the excavation • Following footing excavations to confirm founding material strength
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, whilst any groundwater seepage must be controlled within the excavation and prevented from ponding or saturating slopes/batters.

5.3.3. Retaining Structures:

Required	New post-excavation retaining structures/excavation support wall will be required as part of the proposed development to support the garage walls in the front west portion.
Types	Steel reinforced concrete/concrete block post excavation where batters possible. Designed in accordance with Australian Standards AS4678-2002 Earth Retaining Structures.

Parameters for calculating un-surcharged pressures acting on retaining walls for the materials likely to be retained:

Material	Unit Weight (kN/m3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Topsoil/Fill	18	$\phi' = 30°$	N/A	0.5	N/A
Clay (very stiff to hard)	20	$\phi' = 35°$	0.27	0.40	N/A
VLS bedrock	22	$\phi' = 38°$	0.15	0.20	200kPa
LS bedrock*	23	$\phi' = 40°$	0.10	0.15	400kPa

Remarks:

*Unconfirmed at the time of reporting.

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

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

For cantilever or simple anchor support walls a triangular pressure distribution can be utilised for at least initial design (pending results of further investigation).

5.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		No
Excavation likely to intersect	Water Table	No
	Seepage	Minor (<1L/min), on defects and at soil/rock interface
Site Location and Topography		East side of Hudson Parade within moderate west dipping topography.
Impact of development on local hydrogeology		Appears negligible
Onsite Stormwater Disposal		Site on high side of the road, discharge to council stormwater system
Remarks: Trenches, as well as all new building gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which preferably discharges to the Council's stormwater system off site. Current investigation results indicate a drained basement will be suitable.		

5.4. Conditions Relating to Design and Construction Monitoring:

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

1. Review the structural design drawings, including the retaining structure design and construction methodology, for compliance with the recommendations of this report prior to construction,
2. Inspect soil batters during excavation and the installation of excavation support measures,
3. Inspect any medium strength bedrock and the proposed equipment prior to its excavation and at 1.50m depth intervals of excavation,
4. Inspect all new footings to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and stability prior to the placement of steel or concrete,
5. Inspect completed works to confirm all slope stability, retention and stormwater systems are completed.

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

6. CONCLUSION:

The site investigation identified topsoil/fill and colluvium extending to a maximum depth of 0.65 m (BH1), overlying clayey sand and sandy clay residual soils across the site. Sandstone/siltstone bedrock of at least very low strength was encountered between 1.09 m and 1.36 m depth, with strength expected to increase with depth.

The proposed works consist of the demolition of existing garage and the construction of a new two car garage. The proposed structure will require bulk excavation to a maximum of approximately 3.40m.

Safe batter slopes appear achievable for the proposed excavations per Section 5.3.2; thus, pre-excavation support is not expected for the new garage. The detailed suitability of the rock excavation faces to remain unsupported will depend on the condition of the bedrock, which requires further geotechnical investigation or regular geotechnical inspection during excavation.

Based on the site investigation, a groundwater table is not anticipated within the proposed excavation however, minor seepage is likely to be encountered during the excavation at the bedrock surface and on details in the rock mass. Therefore, it is unlikely to be a significant issue for this development, and tanking and large-volume dewatering are unlikely to be critical hazards.

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

Prepared By:	Reviewed By:
Samyam Bohara	Troy Crozier
Geotechnical Engineer	Principal
	MIE Aust. CPEng (NER – Geotechnical)
	MAIG, RPGeo – Geotechnical and Engineering

7. REFERENCES:

1. Australian Standard AS1170.4 - 2007 Earthquake Actions;
2. Australian Standard AS 1289 – 2000, Method of Testing Soils for Engineering Purposes;
3. Australian Standard AS 1726: 2017, Geotechnical Site Investigations.
4. Australian Standard AS 2870: 2011, Residential Slabs and Footings.
5. Australian Standard AS3600:2009, Concrete Structures.
6. Australian Standard AS3798:2007, Guidelines on Earthworks for Commercial and Residential Developments.
7. Australian Standard AS 4678:2002, Earth-Retaining Structures.
8. Australian Standard AS1170.4 – 2007, Part 4: Earthquake actions in Australia
9. Sydney 1:100,000 Geological Series Sheet 9130 (Edition 1). Geological Survey of New South Wales, Department of Mineral Resources.
10. Spatial Information Viewer, maps.six.nsw.gov.au, NSW Department of Finance and Service.
11. Geological Society Engineering Group Working Party 1972, "The preparation of maps and plans in terms of engineering geology" Quarterly Journal Engineering Geology, Volume 5, Pages 295 - 382.
12. Das., Principles of Foundation Engineering, 5th Edition, Brooks/Cole,2004.

CROZIER
GEOTECHNICAL CONSULTANTS

Appendix 1

CROZIER
GEOTECHNICAL CONSULTANTS

Crozier Geotechnical Consultants ABN: 96 113 453 624
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Crozier Geotechnical Consultants, a division of PJC Geo-Engineering Pty Ltd

NOTES RELATING TO THIS REPORT

Introduction

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

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

Description and classification Methods

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

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

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

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

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

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

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

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

Sampling

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

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

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

Drilling Methods

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

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

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

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

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

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

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

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

Standard Penetration Tests

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

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

The test results are reported in the following form.

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

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

Cone Penetrometer Testing and Interpretation

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

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

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

The information provided on the plotted results comprises: -

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

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

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

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

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

Q_c = (12 to 18) C_u

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

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

Dynamic Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Borehole Logs

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

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

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

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

Ground Water

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

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

Engineering Reports

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

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

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

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

Site Anomalies

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

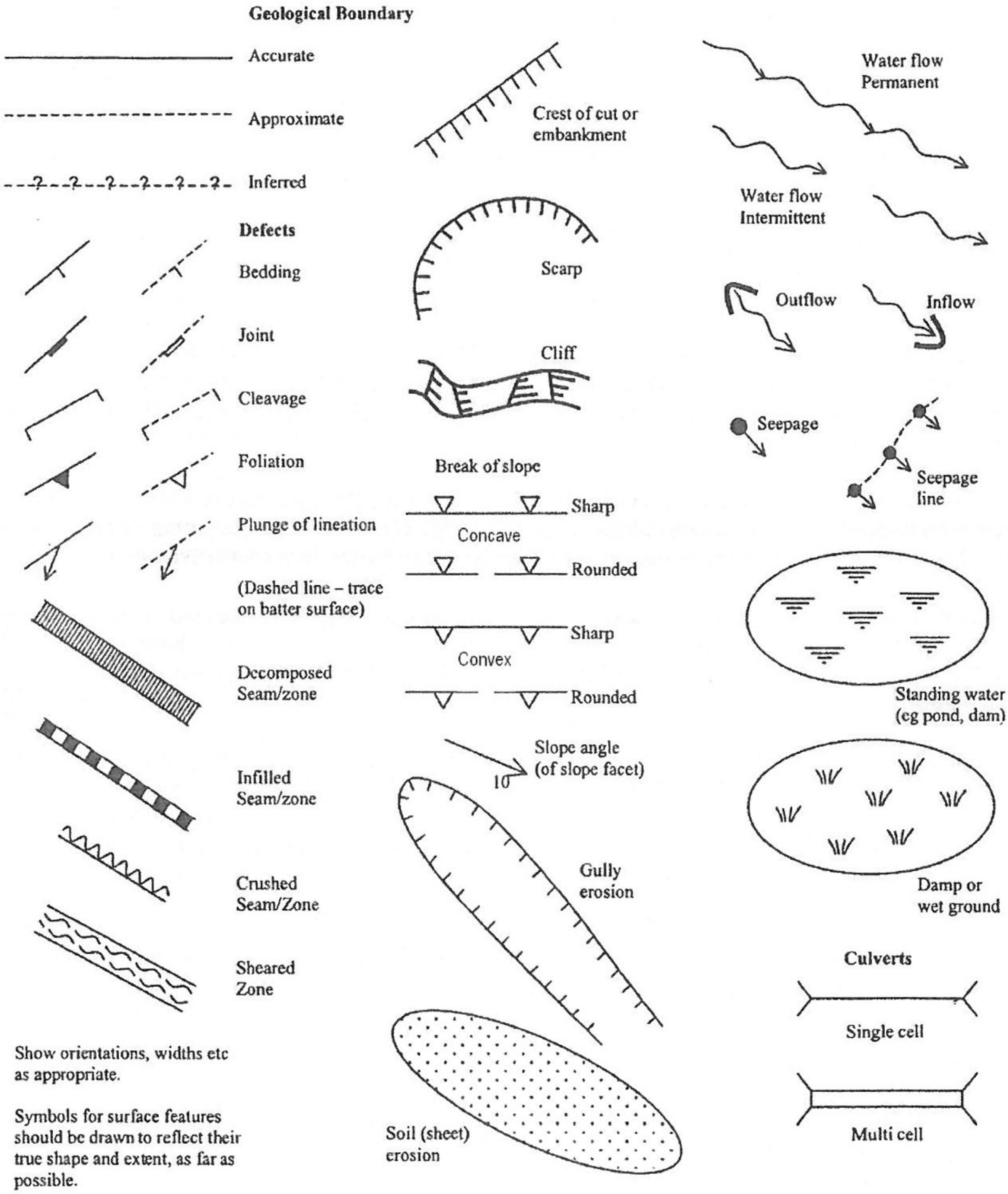
Reproduction of Information for Contractual Purposes

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

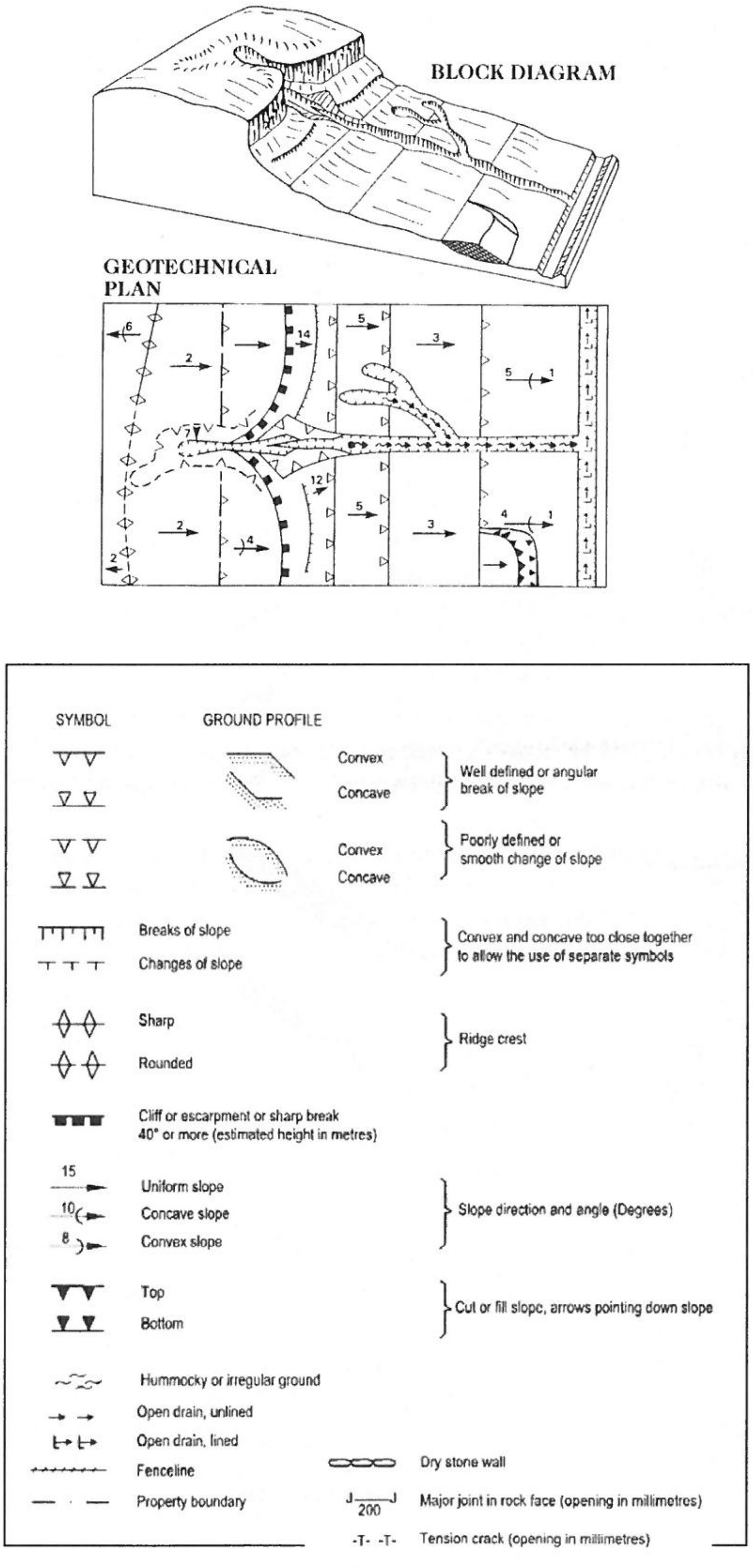
Site Inspection

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

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY

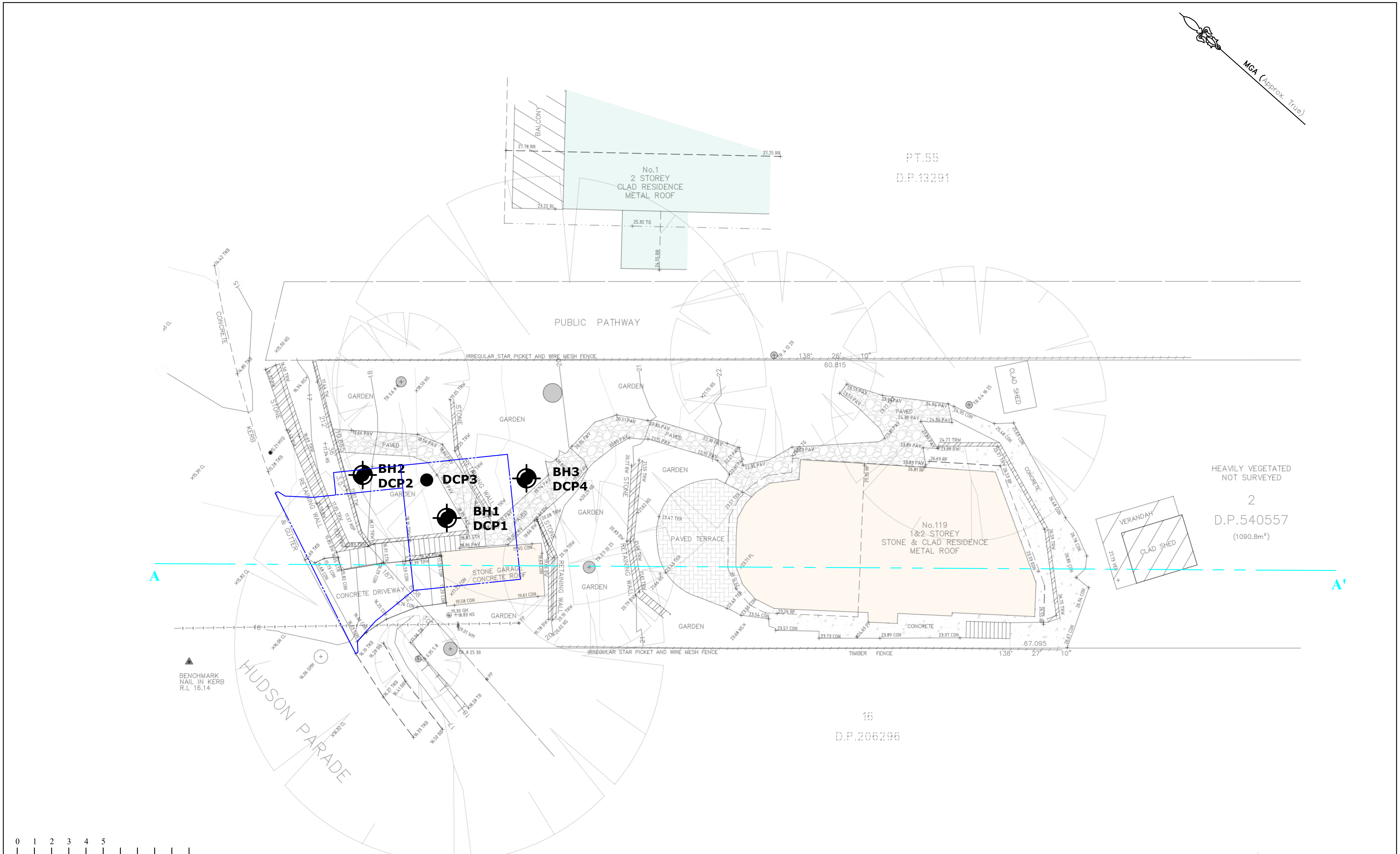


Examples of Mapping Symbols (after Guide to Slope Risk Analysis Version 3.1 November 2001, Roads and Traffic Authority of New South Wales).



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

Appendix 2



SITE PLAN & TEST LOCATIONS FIGURE 1.

CROZIER
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Brookvale NSW 2100
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Phone: (02) 9939 1882
Fax: (02) 9939 1883
Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd

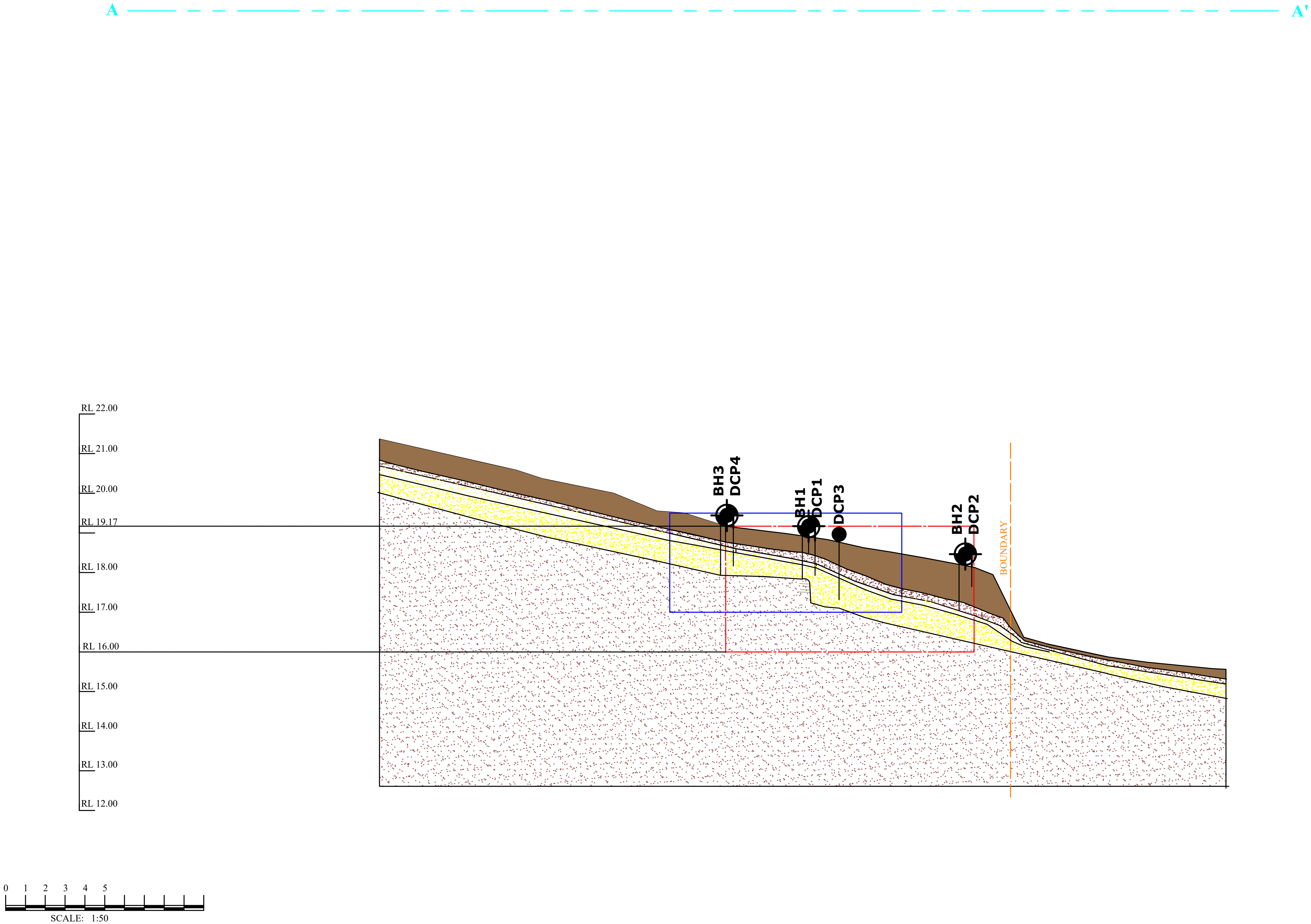
LEGEND

(blue line)	PROPOSED WORKS
BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
DCP	DYNAMIC CONE PENETROMETER
A — A'	CROSS-SECTION REFERENCE LINE

SCALE:	1:100 @ A3
DRAWING:	FIGURE 1
DATE:	05/2025
APPROVED BY:	TMC
DRAWN BY:	SB
PROJECT:	2025-086

PREPARED FOR:
Mia Asker

ADDRESS:
119 HUDSON PARADE, CLAREVILLE



SECTION A: FIGURE 2

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

LEGEND

- PROPOSED WORKS
- EXISTING STRUCTURES
- COLLUVIUM
- TOPSOIL/FILL
- RESIDUAL SOILS
- SAND
- SANDSTONE BEDROCK
- A —— A' SECTION LINE

SCALE:	1:50 @ A3
DRAWING:	FIGURE 2
DATE:	05/2025
APPROVED BY:	TMC
DRAWN BY:	SB
PROJECT:	2025-086

PREPARED FOR:
MIA ASKER

ADDRESS:
119 HUDSON PARADE, CLAREVILLE

BOREHOLE LOG

CLIENT: Mia Asker **DATE:** 20/05/2025 **BORE No.:** 1

PROJECT: Demolition of Existing Garage and Construction of a new two car garage **PROJECT No.:** 2025-086 **SHEET:** 1 of 1

LOCATION: 119 Hudson Parade, Clareville **SURFACE LEVEL:** RL 18.95

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: Dark brown, fine-medium grained, moist, silty sand with gravel				
0.40	SM	Silty SAND: Very loose, brown, with clay and gravel (colluvial)				
0.50		…medium dense				
0.65	SC	Clayey SAND: Medium dense, brown to red brown, fine grained, moist, clayey sand				
0.75	CL	Sand CLAY: Very stiff, brown to yellow brown, low to medium plasticity, moist, sandy clay				
0.80		…yellow brown				
0.85		…medium plasticity				
0.9		END OF BOREHOLE at 0.9m depth due to refusal atop interpreted atleast very low strength sandstone bedrock				
1.00						
1.50						
2.00						
2.50						

RIG: N/A DRILLER: SB

METHOD: Hand Auger LOGGED: JC

GROUND WATER OBSERVATIONS: Not Observed

REMARKS: CHECKED: TMC

BOREHOLE LOG

CLIENT: Mia Asker | **DATE:** 20/05/2025 | **BORE No.:** 2

PROJECT: Demolition of Existing Garage and Construction of a new two car garage | **PROJECT No.:** 2025-086 | **SHEET:** 1 of 1

LOCATION: 119 Hudson Parade, Clareville | **SURFACE LEVEL:** RL 18.00

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling		In Situ Testing	
			Type	Tests	Type	Results
0.00		FILL: Dark brown, fine-medium grained, moist, silty sand with gravel				
0.50		END OF BOREHOLE at 5.0m depth due to refusal within fill				
1.00						
1.50						
2.00						
2.50						

RIG: N/A

DRILLER: SB

METHOD: Hand Auger

LOGGED: JC

GROUND WATER OBSERVATIONS: Not Observed

REMARKS:

CHECKED: TMC

BOREHOLE LOG

CLIENT: Mia Asker | **DATE:** 20/05/2025 | **BORE No.:** 3

PROJECT: Demolition of Existing Garage and Construction of a new two car garage | **PROJECT No.:** 2025-086 | **SHEET:** 1 of 1

LOCATION: 119 Hudson Parade, Clareville | **SURFACE LEVEL:** RL 19.74

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling: Type	Sampling: Tests	In Situ Testing: Type	In Situ Testing: Results
0.00		TOPSOIL/FILL: Dark brown, medium grained, moist, silty sand wiith gravel				
0.40	SM	Silty SAND: Very loose, dark brown to yellow brown (colluvium)				
0.50	SC	Clayey SAND: Loose, brown, fine-medium grained, moist, clayey sand with trace gravel				
0.60		…wet				
	CL	Sandy CLAY: Stiff, brown to yellow brown, low to medium plasticity, wet, sandy clay				
0.70		…moist, gravel band				
0.80		…yellow brown				
0.95		END OF BOREHOLE at 0.95m depth due to refusal atop interpreted boulder				
1.00						
1.50						
2.00						
2.50						

RIG: N/A | DRILLER: SB

METHOD: Hand Auger | LOGGED: JC

GROUND WATER OBSERVATIONS: Not Observed

REMARKS: | CHECKED: TMC

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Mia Asker
DATE: 20/05/2025
PROJECT: Demolition of Existing Garage and Construction of a new two car garage
PROJECT No.: 2025-086
LOCATION: 119 Hudson Parade, Clareville
SHEET: 1 of 1

	Test Location									
Depth (m)	DCP1	DCP2	DCP3	DCP4						
0.00 - 0.10	0	-	SW	0						
0.10 - 0.20	1	-	SW	1						
0.20 - 0.30	0	-	1	2						
0.30 - 0.40	1	-	1	1						
0.40 - 0.50	3	-	4	2						
0.50 - 0.60	4	5	3	2						
0.60 - 0.70	4	1	1	3						
0.70 - 0.80	5	2	3	3						
0.80 - 0.90	5	5	4	4						
0.90 - 1.00	13	4	4	3						
1.00 - 1.10	19	10	7	10						
1.10 - 1.20	B at 1.09 m	14	10	5						
1.20 - 1.30		14	13	17						
1.30 - 1.40		B at 1.36m	18	B at 1.28m						
1.40 - 1.50			21							
1.50 - 1.60			STOP at 1.45m							
1.60 - 1.70										
1.70 - 1.80										
1.80 - 1.90										
1.90 - 2.00										
2.00 - 2.10										
2.10 - 2.20										
2.20 - 2.30										
2.30 - 2.40										
2.40 - 2.50										
2.50 - 2.60										
2.60 - 2.70										
2.70 - 2.80										
2.80 - 2.90										
2.90 - 3.00										
3.00 - 3.10										
3.10 - 3.20										
3.20 - 3.30										
3.30 - 3.40										
3.40 - 3.50										
3.50 - 3.60										
3.60 - 3.70										
3.70 - 3.80										
3.80 - 3.90										
3.90 - 4.00										

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

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

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	Landslide (earth slide $<3m^3$) from soils at crest of the excavation		Appears 0.95 metres of soil, no signs of existing movement, up to 3.40m deep excavation	a) dwelling is >13.0m from south of 0.95m deep excavation in soils, negligible impact b) dwelling is >12.75m from side of 0.95m deep excavation in soils, negligible impact c) lawn and garden >10.0m from excavation, negligible impact d) dwelling is >10.0m from side of excavation of 0.95m deep excavation in soils, negligible impact e) lawn and garden 4.0m from excavation, impact 5% f) Pathway 5.47m from 0.95m deep excavation in soils, impact 15% of pathway g) road pavement is 6.0m from edge of 0.95m deep soil excavation, negligible impact		a) Person in house 20hrs/day avge. b) Person in house 20hrs/day avge. c) Person in garden 1hr/day avge. d) Person in house 20hrs/day avge. e) Person in garden 0.25hr/day avge. f) Person in pathway 0.50hr/day avge. g) Person in vehicle, 1hr/day	a) Likely to not evacuate b) Likely to not evacuate c) Unlikely to not evacuate d) Likely to not evacuate e) Unlikely to not evacuate f) Possible to not evacuate g) Likely to not evacuate	a) Person in building, minor damage only b) Person in building, minor damage only c) Person in open space, buried d) Person in building, minor damage only e) Person in open space, buried f) Person in open space, buried g) Person in vehicle, damage only	
			Possible	**Prob. of Impact**	**Impacted**				
		a) Site House (No. 119 Hudson Parade)	0.01	0.05	0.01	0.83	0.75	0.05	**1.56E-07**
		b) House 117 Hudson Parade	0.01	0.05	0.01	0.83	0.75	0.05	**1.56E-07**
		c) Lawn, garden, driveways - No. 117 Hudson Parade	0.01	0.25	0.01	0.04	0.25	0.50	**1.30E-07**
		d) House No. 121 Hudson Parade	0.01	0.05	0.01	0.83	0.75	0.05	**1.56E-07**
		e) Lawn and garden No. 121 Hudson Parade	0.01	0.25	0.05	0.01	0.25	1.00	**3.26E-07**
		f) Public Pathway	0.01	0.10	0.15	0.02	0.5	1.00	**1.56E-06**
		g) Road Pavement - Park Street	0.01	0.01	0.10	0.04	0.75	0.05	**1.56E-08**
B	Landslide (rock and earth slide $<10m^3$) due to instability within the bedrock		Bedrock relatviely shallow however anticpated to be interbedded silstone/sandstone	a) dwelling is >13.0m from south of 3.40m deep excavation, negligible impact b) dwelling is >12.75m from side of 3.40m deep excavation, negligible impact c) lawn and garden >10.0m from excavation, impact 15% d) dwelling is >10.0m from side of excavation of 3.40m deep excavation, impact north edge only e) lawn and garden 4.0m from excavation of 3.40m deep, impact 25% f) Pathway 5.47m from 0.95m deep excavation in soils, impact 25% of pathwayy g) road pavement is 3.40m from edge of ≤3.0m deep excavation, impact 5%		a) Person in house 20hrs/day avge. b) Person in house 20hrs/day avge. c) Person in garden 1hr/day avge. d) Person in house 20hrs/day avge. e) Person in garden 0.25hr/day avge. f) Person in footpath 0.50hr/day avge. g) Person in vehicle past site, 1hr/day	a) Likely to not evacuate b) Likely to not evacuate c) Possible to not evacuate d) Likely to not evacuate e) Unlikely to not evacuate f) Possible to not evacuate g) Likely to not evacuate	a) Person in building, minor damage only b) Person in building, minor damage only c) Person in open space, buried d) Person in building, minor damage only e) Person in open space, buried f) Person in open space, buried g) Person in vehicle, damage only	
			Possible	**Prob. of Impact**	**Impacted**				
		a) Site House (No. 119 Hudson Parade)	0.001	0.25	0.10	0.83	0.75	0.10	**1.56E-06**
		b) House 117 Hudson Parade	0.001	0.20	0.05	0.83	0.75	0.10	**6.25E-07**
		c) Lawn, garden, driveways - No. 117 Hudson Parade	0.001	0.50	0.05	0.04	0.50	1.00	**5.21E-07**
		d) House No. 121 Hudson Parade	0.001	0.01	0.05	0.83	0.75	0.05	**1.56E-08**
		e) Lawn and garden No. 121 Hudson Parade	0.001	0.10	0.25	0.01	0.50	1.00	**1.30E-07**
		f) Public Pathway	0.001	0.20	0.25	0.02	0.50	1.00	**5.21E-07**
		g) Road Pavement - Park Street	0.001	0.05	0.20	0.04	0.75	0.05	**1.56E-08**

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

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslide (earth slide <3m³) from soils at crest of the excavation	a) Site House (No. 119 Hudson Parade)	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 property or structure being assessed.	Low
		b) House 117 Hudson Parade	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 property or structure being assessed.	Very Low
		c) Lawn, garden, driveways - No. 117 Hudson Parade	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 property or structure being assessed.	Low
		d) House No. 121 Hudson Parade	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 property or structure being assessed.	Very Low
		e) Lawn and garden No. 121 Hudson Parade	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 property or structure being assessed.	Low
		f) Public Pathway	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 property or structure being assessed.	Low
		g) Road Pavement - Park Street	Rare	The event is conceivable but only under exceptional circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring property or structure being assessed.	Very Low
B	Landslide (rock and earth slide <10m³) due to instability within the bedrock	a) Site House (No. 119 Hudson Parade)	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 property or structure being assessed.	Low
		b) House 117 Hudson Parade	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 property or structure being assessed.	Low
		c) Lawn, garden, driveways - No. 117 Hudson Parade	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 property or structure being assessed.	Low
		d) House No. 121 Hudson Parade	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 property or structure being assessed.	Low
		e) Lawn and garden No. 121 Hudson Parade	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 property or structure being assessed.	Moderate
		f) Public Pathway	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 property or structure being assessed.	Moderate
		g) Road Pavement - Park Street	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 property or structure being assessed.	Low

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

TABLE: 2

Recommended Maintenance and Inspection Program

Structure	Maintenance/ Inspection Item	Frequency
Stormwater drains.	Owner to inspect to ensure that the drains, and pipes are free of debris & sediment build-up. Clear surface grates and litter.	Every year or following each major rainfall event.
Retaining Walls.	Owner to inspect walls for deveation from as constructed condition	Every 2 years or following major rainfall event.
	Owner to assess retaining wall drainage systems, clean as necessary at 10 yearly intervals or where moisture	At 10 yearly intervals or where moisture problems arise
Large Trees on or adjacent to site	Arborist to check condition of trees and remove as required. Where treee within steep slopes or adjacent to structures require geotechincal inspection prior to removal	Every 10 years
Slope Stability	Hydraulics (stormwater) & Geotechnical Consultants to check on site stability at same time and provide report.	One year after construction is completed.

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

Appendix 4

APPENDIX A

DEFINITION OF TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

APPENDIX C: LANDSLIDE RISK ASSESSMENT

QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

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

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

Notes: (5) For Cell A5, may be subdivided such that a consequence of less than 0.1% is Low Risk.
(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

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

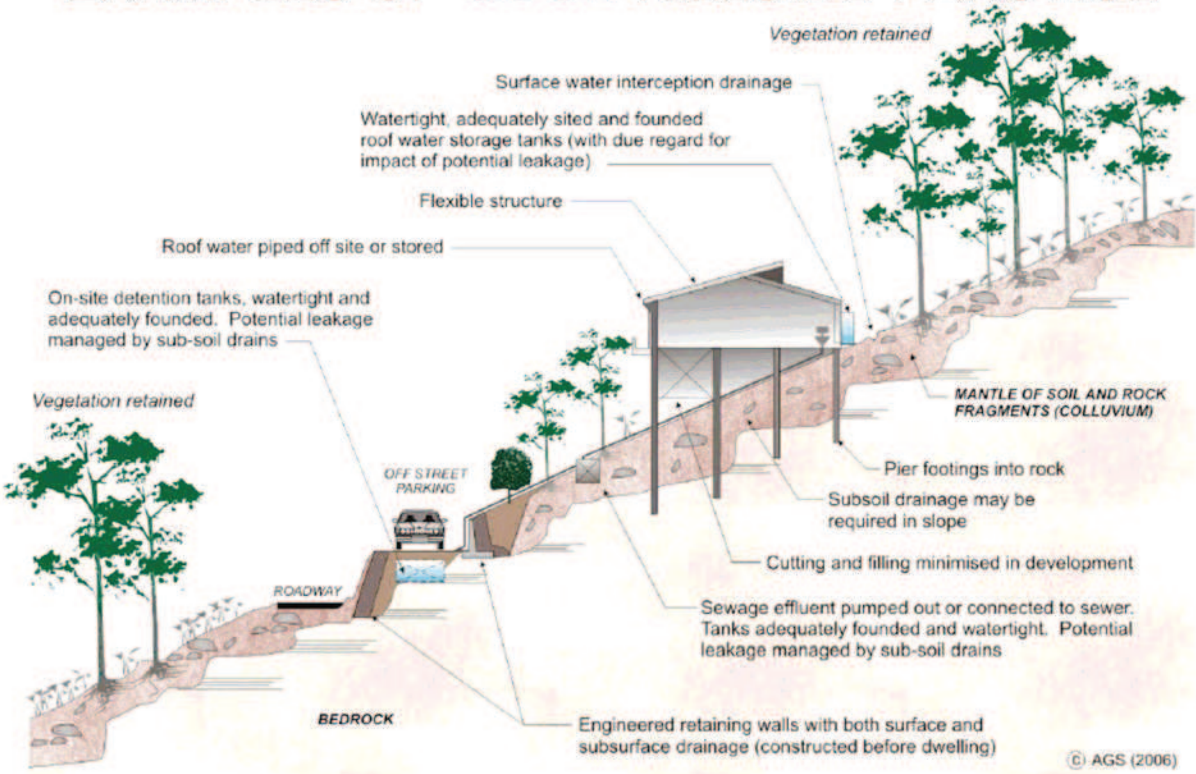
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

Appendix 5

APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

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

EXAMPLES OF GOOD HILLSIDE PRACTICE



EXAMPLES OF POOR HILLSIDE PRACTICE