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

REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

PROPOSED DEVELOPMENT

at

11 Bruce Street, Mona Vale, NSW

Prepared For

Barry Hastie and Cynthia Handley

Project No.: 2020-129
October, 2020

Document Revision Record

Issue No	Date	Details of Revisions
0	23rd July, 2020	Original issue
1	6th October, 2020	Revised Architectural and Hydrological Drawings

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

Development Application for	Barry Hastie and Cynthia Handley (Name of Applicant)
Address of site	11 Bruce Street, Mona Vale, NSW

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

I, ***Troy Crozier*** *on behalf of* ***Crozier Geotechnical Consultants***

on this the 6th Oct 2020 certify that I am a ~~geotechnical engineer or~~ engineering geologist ~~or coastal engineer~~ as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above ~~organisation/~~company to issue this document and to certify that the ~~organisation/~~company has a current professional indemnity policy of at least $2million.

I:

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

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

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

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

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

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

Geotechnical Report Details:

Report Title: Geotechnical report for proposed demolition of existing structure and construction of a new three storey residence at 11 Bruce Street Mona Vale

Report Date: 01/10/2020 **Project No.:** 2020-129

Author: Joshua Cotton

Author's Company/Organisation: Crozier Geotechnical Consultants

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

Architectural Drawings – James de Soyers and Associates, Project No.: 1912, Drawing No.: DA-01 – DA-05, DA-10 – DA-13, DA-20 – DA-22, DA-30 – DA-32, Dated: 28/09/2020
Hydraulic Drawings – Michal Korecky, Project No.: 20076, Sheet No.: SW-2, Dated: 24/09/2020
Survey Drawing – J. Mcclure Detailed Surveys, Ref No.: 075/19, Dated: 04/10/2019

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 .... [signature]

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

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

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

Company... **Crozier Geotechnical Consultants**

REGISTERED PROFESSIONAL GEOSCIENTIST
AUSTRALIAN INSTITUTE OF GEOSCIENTISTS
TROY CROZIER
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	Barry Hastie and Cynthia Handley (Name of Applicant)
Address of site	11 Bruce Street, Mona Vale, NSW

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

Geotechnical Report Details:

Report Title: Geotechnical report for proposed demolition of existing structure and construction of a new three storey residence at 11 Bruce Street Mona Vale

Report Date: 01/10/2020 **Project No.:** 2020-129

Author: Joshua Cotton

Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

- ■ Comprehensive site mapping conducted 30/06/2020 (date)
- ■ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
- ■ Subsurface investigation required
  - ☐ No Justification ....
  - ■ Yes Date conducted ....30/06/2020...........................
- ■ Geotechnical model developed and reported as an inferred subsurface type-section
- ■ Geotechnical hazards identified
  - ☐ Above the site
  - ■ On the site
  - ☐ Below the site
  - ☐ Beside the site
- ■ Geotechnical hazards described and reported
- ■ Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
  - ■ Consequence analysis
  - ■ Frequency analysis
- ■ Risk calculation
- ■ Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved.
- ■ Design Life Adopted:
  - ■ 100 years
  - ☐ Other .... specify
- ■ Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
- ■ Additional action to remove risk where reasonable and practical have been identified and included in the report.
- ☐ Risk assessment within Bushfire Asset Protection Zone.

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

Signature ....

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

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

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

Company... **Crozier Geotechnical Consultants**

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GEOTECHNICAL CONSULTANTS

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

Date: 6th October 2020

Project No: 2020-129

GEOTECHNICAL REPORT FOR PROPOSED DEMOLITION OF EXISTING STRUCTURE AND CONSTRUCTION OF A NEW THREE STOREY RESIDENCE 11 BRUCE STREET, MONA VALE

1. INTRODUCTION:

This report details the results of a geotechnical investigation and assessment carried out for the proposed demolition of the existing structure and construction of a new three storey residence located at No. 11 Bruce Street, Mona Vale, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of the clients Barry Hastie and Cynthia Handley.

The proposed works involve the demolition of the existing two storey brick residence and construction of a new three storey dwelling. The top level of the proposed structure is planned to be a garage level with two living levels beneath, a rear verandah, in ground swimming pool and in ground rain water tank are also planned. The works appear to require bulk excavation up to 3.00m depth for the house and up to 1.8m depth for the swimming pool and rain water tank.

The site is located within the H1 (highest category) landslip hazard zone as identified within Northern Beaches Councils Geotechnical Risk Management Policy for Pittwater - 2009. To meet the Councils Policy requirements for land classified as H1 this Geotechnical Report 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 the ñAcceptable Risk Managementò criteria is maintained for a preferred design life of 100 years. It is recommended that the client make themselves aware of the Policy and its requirements.

The investigation and reporting were undertaken as per the Tender: P20-257 Dated: 9th June 2020.

The geotechnical investigation included:

a) Drilling of three boreholes using hand tools along with Dynamic Cone Penetrometer (DCP) tests to determine the subsurface geology, depth to bedrock, and identification of groundwater.
b) Detailed geotechnical mapping of the entire site and adjacent land, with identification of geotechnical conditions and hazards including landslip related to the existing site and structures
c) A photographic record of site conditions

The following documents have been supplied in regard to the proposed investigation:

- Architectural Drawings ï James de Soyers and Associates, Project No.: 1912, Drawing No.: DA-01 ï DA-05, DA-10 ï DA-13, DA-20 ï DA-22, DA-30 ï DA-32, Dated: 28/09/2020
- Hydraulic Drawings ï Michal Korecky, Project No.: 20076, Sheet No.: SW-2, Dated: 24/09/2020
- Survey Drawing ï J. Mcclure Detailed Surveys, Ref No.: 075/19, Dated: 04/10/2019

2. PROPOSED DEVLOPMENT:

The proposed works include the demolition of the existing one and two storey brick residence and construction of a new three storey dwelling. The structure is to include a top-level garage with a partly suspended driveway and two living storeys below. The proposed development will intersect the natural slope of the site at the western end of the main structure to establish a basement FFL at R.L. 25.51m. Bulk excavation will be required to Ò3.0m depth, with the maximum cut occurring at the western end of the basement level, excavation depth will grade to nil at the eastern end of the proposed basement level. A central elevator, rear in ground swimming pool, below ground rain water tank and front retaining wall are also planned.

The proposed main structure is to have a 1.0m southern setback and 2.5m northern setback from the respective boundary lines, this is inclusive of the basement level and required excavation. The western wall of the ground floor level is to extend to 3.8m from the western boundary, this level appears to require minor excavation to Ò0.5m within western portions. The inground swimming pool is to be constructed within the rear of the site requiring bulk excavation to a maximum of 1.8m depth. The 10 000L rain water tank is to be located below the proposed rear verandah and will require bulk excavation up to 1.3m depth, an overflow pipe will extend to a level spreader along the rear boundary. The elevator will likely require further isolated excavation, the proposed elevator will be 5.1m from the western basement wall and 3.0m from the southern basement wall. An outdoor staircase and wall are proposed along the northern boundary of the site, with the rear comprising low stone walls in a landscaped garden area.

3. SITE FEATURES:

3.1. Description:

The site is a pentagonal shaped block located on the low east side of Bruce Street. It has a front west boundary of 16.41m, a north side boundary of 33.78m, a south side boundary of 38.55m, and rear east boundaries of 11.50m and 4.98m, as reference from the provided topographical survey plan.

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

The site is located within moderate northeast dipping topography on the east side of a steep sided ridge and extends from a high of approximately RL = 29.6m along the front boundary of the property to a low of approximately RL = 23.3m along the rear boundaries. It is currently occupied by a two storey brick dwelling, with moderate northeast sloping gardens and grassed lawns within the front and a concrete paved patio area and gardens within the rear. The rear of the site backs onto a nature reserve at the crest of a steep coastal cliff, situated above Warriewood Beach. A general view of the front of the site at the time of investigation is provided in Photograph 2 below.

Photograph 1: Aerial photo of site and surrounds

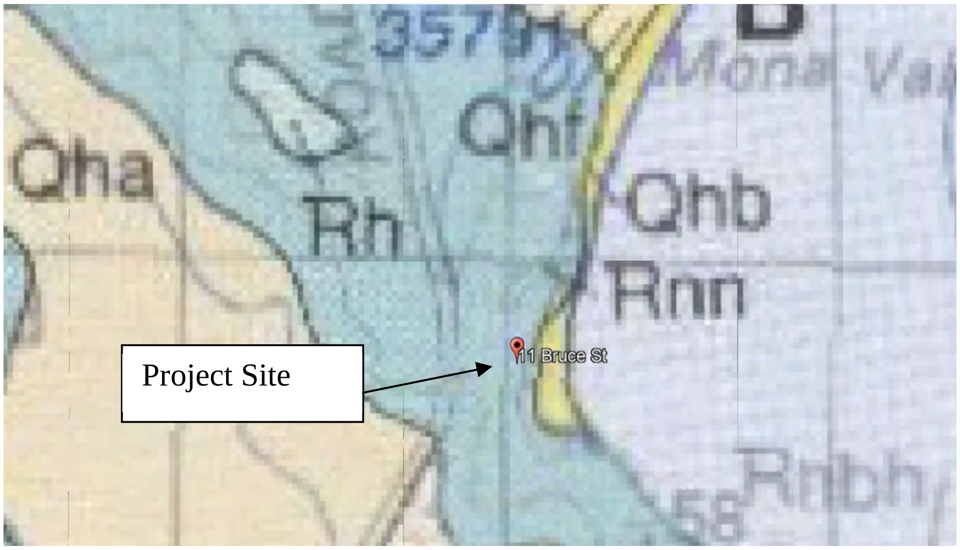
.

Photograph 2: View of the front of the site looking north east from Burke Street

3.2. Geology:

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

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



Extract 1:100,000 geological reference

4. FIELD WORK:

4.1. Investigation Methods:

The field investigation comprised a geotechnical inspection of the site and adjacent land on the 30th June 2020. It involved a photographic record of site conditions as well as geotechnical assessment of the site and adjacent land with examination of soil slopes and existing structures. It also included the drilling of three auger boreholes (BH1 ï BH3) using hand tools to determine sub-surface geology.

DCP testing was carried out from ground surface at four separate locations throughout the site in accordance with AS1289.6.3.2 ï 1997, ñDetermination of the penetration resistance of a soil ï 9kg Dynamic Cone Penetrometerò to estimate near surface soil conditions and confirm depths to bedrock.

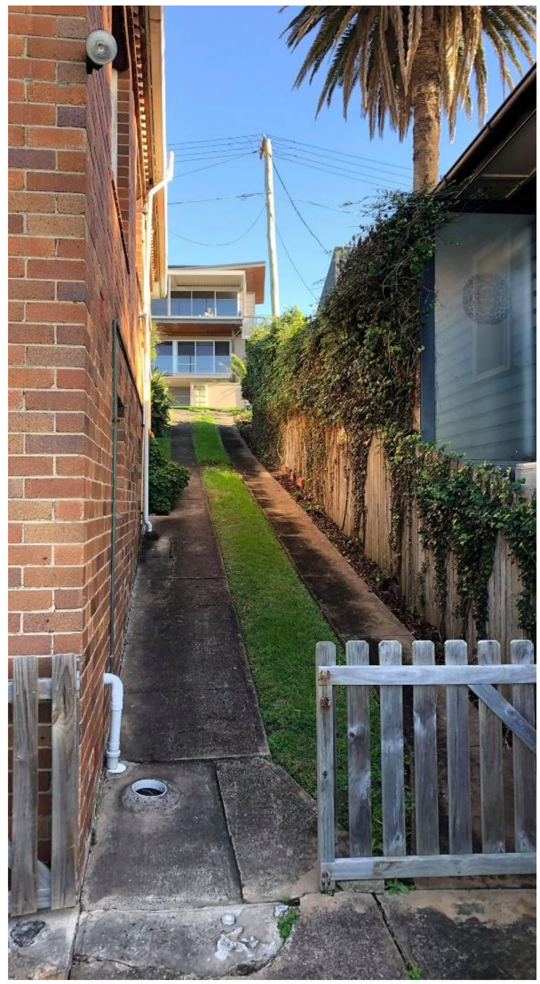
Explanatory notes are included in Appendix: 1. Mapping information and test locations are shown on Figure: 1, along with detailed borehole log sheets and DCP test sheet in Appendix: 2. The geological models/sections are provided as Figure: 2 and Figure: 3, Appendix: 2.

4.2. Field Observations:

The site is situated on the low east side of Bruce Street within moderate north east dipping topography. Bruce Street is formed with a bitumen pavement which dips gently (<5º) north where it passes the site. There were no signs of excessive cracking or deformation within the road pavement to suggest any movement or underlying geotechnical issues.

The property is accessed via a stripped concrete driveway in the north western corner of the site, this driveway extends to the concrete patio at the rear of the main structure. The front of the site consists of moderately north east dipping grassed lawns with gardens along the front boundary. A dilapidated washing line is situated in the southwest portion of the front lawn.

The existing building is a two storey brick structure of 1940ɑs construction located in the center of the site. The composition of the structure is dictated by the natural topography of the site, with one elevated storey to the front and two storeys to the rear. The main structure had no signs of significant cracking or movement. The rear concrete paved patio region and concrete garden bed showed signs of significant cracking and settlement, this is shown in Photograph 4. It appeared that the pavers and minor concrete structures used no reinforcement in the concrete and over time live loads or minor movements have led to cracking, settlement and rotation. However, these isolated failures do not suggest any underlying geotechnical issues.



Photograph 3: View of the driveway along northern side of house, looking west.

Storm water drainage from the existing structure is largely discharged across the site onto concrete surfaces, this is evident along the northern side (Photograph 3), southern side and rear eastern side of the dwelling. Drainage points are located intermittently throughout the site, downslope to these discharge points. Due to the natural topography of the site, the majority of water discharged which is not intersected by drainage points is expected to flow to the rear of the site, beyond and adjacent to the rear concrete patio.

The rear of the site consists of a stone retaining wall Ò0.5m high, with gardens and grassed lawns extending to the non-specified rear boundary line. Access to the rear boundary of the site is available along the southern boundary, as large plants and shrubs occupy the northern portions of the rear garden region. Approximately 7.0m from the northern edge of the rear boundary and approximately 10.0m from the southern edge of the rear boundary an approximate 60° to 70°, 20.0m high escarpment is located. At the base of this escarpment are the Warriewood Beach carpark and turning circle as shown in Photograph 4.

Photograph 4: View of Warriewood Beach carpark turning circle and above cliff line, looking south west, sourced from Google Maps street view

The property to the south (No.9 Bruce Street) contains a three level rendered brick residence positioned within similar topography as the site. The main structure occupies the majority of the property, the structure has a side setback of approximately 0.5m to the common boundary. Stormwater downpipes appear to extend along the northern side of the structure, to an unknown discharge point. The large residence structure appears to be Ò30 years of age and in a good condition with no sign of cracking or settlement on the external walls.

The property to the north (No. 13 Bruce Street) contains a one and two storey brick and weatherboard dwelling, situated within very similar topography as the site. An inground swimming pool is located within the rear south eastern portion of the property Ò0.3m from the common boundary with the site. Stormwater downpipes appear to extend to a drainage point at the rear of the main structure near the common boundary with the site. The structure is located approximately 1.0m from the common boundary. The dwelling at 13 Bruce Street appears to be Ò20 years of age and in a good condition with no sign of cracking or settlement on the external walls.

Photograph 5: Settlement and cracking in rear patio area, looking north

Photograph 6: View of the rear of the site, looking north east

4.3. Ground Conditions:

The boreholes (BH1 ï BH3) were drilled using a hand auger at select locations within the site with refusal encountered at depths varying from 0.15m (BH1) to 0.36m (BH2)

Dynamic Cone Penetrometer (DCP) tests were carried out from the ground surface adjacent to the boreholes, as well as at one separate location within the north west portion of the site. All DCP tests encountered refusal between 1.80m (DCP1/ DCP2) and 1.87m (DCP3) on interpreted shale bedrock of at least very low strength (VLS).

Based on the borehole logs and DCP test results, the sub-surface conditions at the project site can be classified as follows:

- **TOPSOIL/ FILL** ï this layer was encountered in all boreholes to varying depths between to 0.35m and 0.40m. It is classified as firm, dark brown, medium plasticity, moist, silty clay with rootlets (BH1 and BH2) whilst within BH3 it was classified as loose, dark brown, fine to medium grained, moist clayey sand.
- **SILTY CLAY –** Based on the borehole logs and DCP test results, residual soil extends beyond the fill layer to depths between 1.80m and 1.87m. Hand augers at all locations refused in stiff to very stiff silty clay between 0.60m and 1.20m depth. This layer is classified as firm grading stiff, dark brown mottled red, medium plasticity, moist, silty clay with fine to medium claystone and sandstone gravel.
- **SHALE BEDROCK** ï based on geological mapping of Newport formation bedrock, the bedrock underlaying the residual soils is expected to be shale. Refusal on interpreted shale bedrock of at least very low strength was encountered at 1.80m depth at DCP1 and DCP2, DCP3 and DCP4 encountered refusal at 1.87m and 1.81m depth, respectively.

A free standing ground water table or significant water seepage were not identified within any of the boreholes. No signs of ground water were observed after the retrieval of the DCP rods.

5. GEOTECHNICAL ASSESSMENT:

5.1. Comments:

The site investigation identified the presence of silty clay and clayey sand topsoil/ fill to the maximum drilled depth of 0.40m over the site. This fill layer is underlain by firm to stiff silty clay to an anticipated depth of approximately 1.50m. Interpretation of the DCP results indicates the residual soils may grade to extremely weathered bedrock from 1.50m depth. DCP1 and DCP2 were undertaken from within the front of the site, with refusal on interpreted shale bedrock of at least very low strength recorded at 1.80m depth. DCP3 and DCP4 were undertaken at the rear of the site, similarly refusing on interpreted shale bedrock of at least very low strength at 1.87m and 1.70m depth respectively.

The rock profile appears to slope similarly to the natural surface level of the site with low strength bedrock expected within shallow depth of DCP refusal. No seepage or groundwater was encountered during the investigation however, it is likely that minor seepage will be intersected at the soil - rock interface during excavation. A freestanding water table was not encountered and is not expected within the site due to the topography of the site and surrounds.

It is understood that the proposed development involves construction of a new residential house that requires excavation for the basement floor level. Due to the natural slope of the site, excavation will grade from approximately 3.00m depth to nil (west to east). Setbacks of proposed excavation to boundaries include a front 6.50m setback, a northern 2.50m side setback and a southern 1.00m side setback. As a result, safe temporary batter slopes as detailed in section 5.3.3 will not be achievable for portions of the northern side of the excavation which are greater than 1.2m depth and portions of the southern side of excavation which are greater than 0.7m depth. Therefore, temporary supports will need to be established where batter slopes cannot be achieved.

Due to the location and slope of the site as well as the ground conditions identified, it is recommended that all footings for the new residential structure extend to bedrock of at least low strength. The front portion of the proposed basement is anticipated to intersect bedrock to achieve a FFL at R.L.25.51m, therefore pad or strip footings will be adequate. However, where the development extends east or is not formed over an excavation, pile footings appear required to achieve consistent foundation. It is recommended that all footings should be founded within similar load bearing material to reduce differential settlement risk.

The current revision of the architectural drawings has included alterations to the location and dimensions of the proposed swimming pool, the new location of the pool is 4.5m to the east of the previously designed location. The swimming pool is to extend to a minimum of 1.8m from the rear boundary with the surrounding pool deck to extend to approximately 1.2m from the northern boundary. The swimming pool will require bulk excavation up to 1.8m depth, with excavation to 1.2m depth within eastern portions. As the rear of the site approaches the crest of the cliff the natural surface level increases in steepness, therefore it is apparent that the swimming pool has been relocated to a steeper sloping section of the site closer to the rear boundary and subsequent cliff crest. It is recommended that the swimming pool is founded to bedrock of at least low strength due to the site conditions and proposed location. Temporary batter slopes appear to be achievable for the pool excavation, however inspection of the batter slopes and footing locations will be necessary prior to the placement of steel or concrete.

The proposed rain water tank is to be located below the rear basement floor verandah, requiring bulk excavation up to 1.3m depth. The tank will occupy an area of $17.2m^2$ and will hold up to 10 000L, with multiple on-site storm water pipes discharging within the tank. It is recommended that the tank bears within at least stiff silty clay, however due to the shallow depth to bedrock it would be prudent to extend all footings for the tank to shale bedrock of at least very low strength. The overflow pipe as well as other lower stormwater pipes are to discharge below the proposed swimming pool, through the onsite dispersion system with excess water to flow over a 4.0m wide level spreader along the rear boundary.

No signs of existing pumping or charged lines redirecting stormwater up-site were observed while onsite with the existing storm water system including basic piping discharging onto sealed surfaces downslope with intermittent open grate drains across the site. The proposed engineered stormwater dispersion system will provide a higher level of control than any of the existing systems. Due to the anticipated reduced stormwater discharge into the landscaped reserve at the rear of the site the impact of development on local hydrogeology and landslip instability or erosion is considered to be negligible.

There were no signs of boulders or rock outcrops within the site or adjacent properties above the crest of the cliff. However, at the base of the cliff along the Warriewood Beach carpark road, shale and laminite siltstone/ sandstone outcrops and boulders were noted. The steep escarpment crest is located approximately 10.0m to the east of the rear of the site and as such any natural instability within this cliff is not considered a sensible hazard for the site. The on-site inspection and previous geotechnical experience within the area identified no evidence of landslip issues within the site or adjacent properties/ areas.

The lower/deeper portions of the excavations are expected to intersect low to medium strength bedrock which will require rock excavation equipment (i.e. rock hammer). The use of rock hammers has the potential to create ground vibrations which could impact adjacent structures. Therefore, hammers should be limited to Ò250kg on this site to maintain low ground vibrations levels at adjacent structures as per AS2187.2-2006 and

AS2670.1- 2001. This will result in slower excavation progress however should large hammers be proposed then a geotechnical professional should be consulted for equipment assessment.

The proposed works are considered suitable for the site and may be completed with negligible impact to the adjacent properties provided the recommendations of this report are implemented in the design and construction phases.

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and hand drilling tools due to access limitations. This test equipment 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. The results of the investigation provide a reasonable basis for the analysis and subsequent design of the proposed works.

5.2. Site Specific Risk Assessment:

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

A. Landslip (earth slide < $3m^3$) of surficial soils from proposed house excavation
B. Landslip (rock slide/ topple < $2m^3$) due to proposed house excavation
C. Landslip (earth slide < $1m^3$) due to proposed pool excavation

The hazards have been assessed in accordance with the methods of the Australian Geomechanics Society (Landslide Risk Management, AGS Subcommittee, May 2002 and March 2007), see Tables: A and B, Appendix: 3 The Australian Geomechanics Society Qualitative Risk Analysis Matrix is enclosed in Appendix: 4 along with relevant AGS notes and figures. The frequency of failure was interpreted from existing site conditions and previous experience in these geological units.

The **Risk to Life** from **Hazard A** was estimated to be **9.38×10^{-7}** for persons within the neighbouring property adjacent to the excavation, while the **Risk to Property** was considered to be **'Low to Moderate'**. The hazard was therefore generally considered to be óAcceptableô when assessed against the criteria of the AGS 2007.

The **Risk to Life** from **Hazard B** was estimated to be **1.67×10^{-8}** for persons within the neighbouring property, while the **Risk to Property** was considered to be **'Low to Moderate'**. The hazard was therefore generally considered to be óAcceptableô when assessed against the criteria of the AGS 2007.

The **Risk to Life** from **Hazard C** was estimated to be **9.38 x 10^{-10}** for persons within the neighbouring property, while the **Risk to Property** was considered to be **'Very Low to Low**. The hazard was therefore generally considered to be óAcceptableô when assessed against the criteria of the AGS 2007.

The installation of engineered temporary support where safe batter slopes are not possible and implementation of the recommendations of this report will reduce the likelihood of instability to ñRareò and as such all risks will be controlled within the ñAcceptable Risk Management Criteriaò.

5.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

5.3.1. New Footings:	
Site Classification as per AS2870 ï 2011 for new footing design	Class óAô
Type of Footing	Strip/Pad or Slab at base excavation, where into bedrock, piers founded on bedrock at eastern (rear) end of proposed structure.
Sub-grade material and Maximum Allowable Bearing Capacity	- Stiff Silty Clay: 100kPa - Very Stiff Silty Clay: 200kPa - Weathered, VLS Shale: 800kPa - Weathered LS Shale: 1000kPa - Weathered MS Shale: 2000kPa
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	B_e ï rock site
Remarks: All footings for each structure should be founded off material of similar strength to prevent differential settlement. Ancillary structures may be founded off stiff to very stiff residual clay where minor soil creep movement can be tolerated. All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata. This is mandatory to allow them to be ócertifiedô at the end of the project.	

5.3.2. Excavation:	
Depth of Excavation	Western end of basement floor level Ò3.0m depth, swimming pool excavation Ò1.8m
Distance of Excavation to Neighbouring Properties	No. 9 Bruce Street: 1.0m from boundary, further 1.0m to neighbouring house No. 13 Bruce Street: 2.5m from boundary, further 0.5m to neighbouring house
Type of Material to be Excavated	- Firm to stiff silty/ sandy clay topsoil/fill to maximum depth of 0.40m - Stiff grading very stiff residual silty clay soil; to an anticipated depth of 1.50m. - ELS bedrock ï from an anticipated depth of 1.50m to a maximum of 1.87m (DCP3) - VLS to LS bedrock ï from a minimum depth of 1.70m (DCP4) and maximum depth of 1.87m (DCP3).

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

Material	Safe Batter Slope (H:V)	
	Short Term/ Temporary	Long Term/ Permanent
Fill and natural soils	1.5:1	2:1
Extremely Low to Very Low strength or fractured bedrock	1:1	1.25:1
Low to Medium strength (defect free) bedrock	Vertical *	Vertical *

*Subject to geotechnical inspection

Remarks: Seepage through fill and sandy soils can 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.

All excavation should be planned and geotechnically assessed continually during excavation. Where insufficient space exists for the batters as per above for boundaries, any existing footing or significant tree, then temporary or permanent retention measures should be designed and implemented. The installation of which will require geotechnical inspection.

Equipment for Excavation	Fill and natural soils	Excavator with bucket
	ELS bedrock	Excavator with bucket
	VLS bedrock	Excavator with bucket and ripper

<table>
<tr><td></td><td>LS bedrock</td><td>Excavator with bucket and ripper /small (<150kg) rock hammer</td></tr>
<tr><td></td><td>MS bedrock</td><td>Rock hammer, saw or grinder</td></tr>
<tr><td colspan="3">ELS ï extremely low strength, VLS ï very low strength, LS ï low strength, MS ï medium strength</td></tr>
<tr><td colspan="3">Remarks: The saw 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.</td></tr>
<tr><td>Recommended Vibration Limits
(Maximum Peak Particle Velocity (PPV))</td><td colspan="2">No. 9 Bruce Street = 5mm/s
No. 13 Bruce Street = 5mm/s
Road Reserve Bruce Street = 5mm/s
Service lines = 3mm/s</td></tr>
<tr><td>Vibration Calibration Tests Required</td><td colspan="2">If medium to high strength bedrock is exposed at base of excavation and large rock hammers (Ó250kg) are proposed for use.</td></tr>
<tr><td>Full time vibration Monitoring Required</td><td colspan="2">Pending proposed equipment and vibration calibration testing results</td></tr>
<tr><td>Geotechnical Inspection Requirement</td><td colspan="2">Yes, recommended that these inspections be undertaken as per below mentioned sequence:
• Installation of support systems
• Upon clearing of all soils from the bedrock surface,
• Inspection of temporary and permanent batter slopes
• At completion of the excavation and for footings,
• Any unsupported excavation.</td></tr>
<tr><td>Dilapidation Surveys Requirement</td><td colspan="2">Recommended within 5.0m of excavation perimeter</td></tr>
<tr><td colspan="3">Remarks: Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope. It is recommended that a drainage excavation extend to below the basement floor slab level to reduce the potential for dampness problems in the completed structure.</td></tr>
</table>

5.3.3. Retaining Structures:	
Required	New retaining structures are required as part of the proposed development
Types	Steel reinforced concrete/concrete block walls post excavation where temporary batters are possible, soldier piles with shotcrete or similar engineered system where temporary batters are not achievable. Designed in accordance with Australian Standards AS4678-2002 Earth Retaining Structures.
Parameters for calculating pressures acting on retaining walls for the materials likely to be retained:	

Material	Unit Weight (kN/m3)	Long Term (Drained)	Earth Pressure Coefficients		Passive Earth Pressure Coefficient *
			Active (Ka)	At Rest (K_0)	
Fill Soils	18	ϕ' = 30°	0.41	0.50	N/A
Silty/ Sandy Clay (very stiff)	20	ϕ' = 35°	0.27	0.50	N/A
ELS bedrock	22	ϕ' = 38°	0.32	0.20	200kPa
LS or fractured bedrock	23	ϕ' = 40°	0.25	0.36	400kPa

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

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

5.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		No
Excavation likely to intersect	Water Table	No
	Seepage	Minor, at interpreted soil - rock interface

Site Location and Topography	Low east side of the road, within moderate north east and east dipping topography. Site towards low northern end of Bruce Street, formed with gently north dipping bitumen pavement.
Impact of development on local hydrogeology	Negligible
Onsite Stormwater Disposal	Possible via dispersal system
Remarks: Exposed excavation faces should be expected to receive seepage from surface and subsurface water flow down slope. This can result in relaxation of excavation faces causing instability prior to installation of permanent retention systems. Therefore excavation faces should not remain open for long periods of time unless assessed to be stable by a geotechnical professional. A stormwater diversion drain should be installed upslope of excavation crests to intercept stormwater runoff and prevent erosion and softening of the excavation faces. An excavation trench should also be installed at the base of excavation cuts to below floor slab levels to reduce the risk of long term dampness. 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 Councilôs stormwater system off site.	

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 and approve the structural design drawings prior to the Construction Certificate.
2. Inspect excavation, unsupported cuts/slopes batters and temporary support systems.
3. Inspect all new footings and earthworks to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and stability prior to the placement of steel or concrete.
4. Inspect excavation equipment to determine whether vibration monitoring is required where large (>250kg) rock hammers are proposed.

The client and builder should make themselves familiar with the requirements spelled out in this report for inspections during the construction phase. CGC cannot complete the certification (Form 3) if it has not been called to site to undertake the required inspections.

5.5. Design Life of Structure:

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

- Stormwater and subsoil drainage systems,
- Retaining walls and soil slope erosion/instability,
- Maintenance of trees/vegetation on this and adjacent properties.

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

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

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

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

6. CONCLUSION:

The site investigation identified the presence of firm silty clayey fill at the front of the site and loose clayey sand fill at the rear of the site, to a relatively consistent maximum drilled depth of 0.4m. The topsoil/ fill layer is underlain by firm to stiff silty clay that is interpreted to grade to extremely low strength shale/ siltstone below 1.50m and very low to low strength shale/ siltstone below approximately 1.80m depth. Groundwater or significant seepage was not identified to 1.87m depth.

It is understood the proposed works involve the demolition of the existing 1940s construction brick residence and construction of a new three storey dwelling with rear swimming pool and in ground rain water tank. The proposed structure will include a lower ground floor basement level (RL 25.51m) which will cut into the existing slope and require bulk excavation of up to 3.0m depth, the swimming pool will require bulk excavation up to 1.8m depth. This excavation depth will decrease to nil as the proposed structure continues eastward, due to the natural east dipping topography of the site.

It is recommended that all new footings for the proposed structure and rear swimming pool are founded on bedrock of at least low strength. It is also recommended that all new footings, temporary batter slopes and excavation support systems are to be inspected by a geotechnical professional to confirm the actual founding conditions prior to placement of steel and casting concrete.

The risks associated with the proposed development are considered to achieve and can be maintained within the óAcceptableô Risk Management Criteria provided the recommendations of this report and any future geotechnical directive are implemented. As such the site is considered suitable for the proposed construction works provided that the recommendations outlined in this report are followed.

Prepared by:	Reviewed by:
Josh Cotton	Troy Crozier
Engineer	Principal
	MAIG, RPGeo ï Geotechnical and Engineering
	Registration No.: 10197

7. REFERENCES:

1. Australian Geomechanics Society 2007, ñLandslide Risk Assessment and Managementò, Australian Geomechanics Journal Vol. 42, No 1, March 2007.
2. Australian Standard AS 3798 ï 2007, Guidelines on Earthworks for Commercial and Residential Developments.
3. Australian Standard AS 2870 ï 2011, Residential Slabs and Footings ï Construction
4. Australian Standard AS1170.4 ï 2007, Part 4: Earthquake actions in Australia

Appendix 1

CROZIER
GEOTECHNICAL CONSULTANTS

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Brookvale NSW 2100 Email: info@croziergeotech.com.au
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: -

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

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

Qc = (12 to 18) Cu

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

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

Dynamic Penetrometers

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

Two relatively similar tests are used.

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

Laboratory Testing

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

Borehole Logs

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

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

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

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

Ground Water

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

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

Engineering Reports

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

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

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

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

Site Anomalies

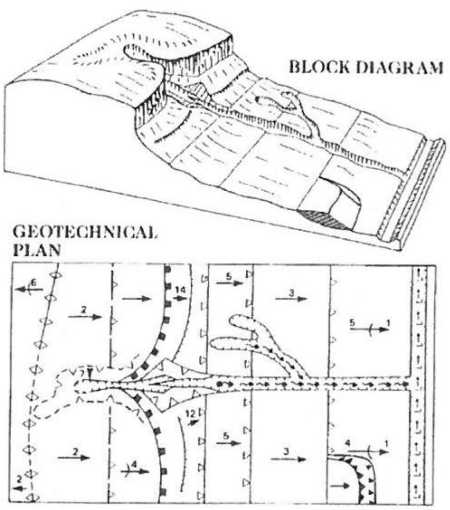
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

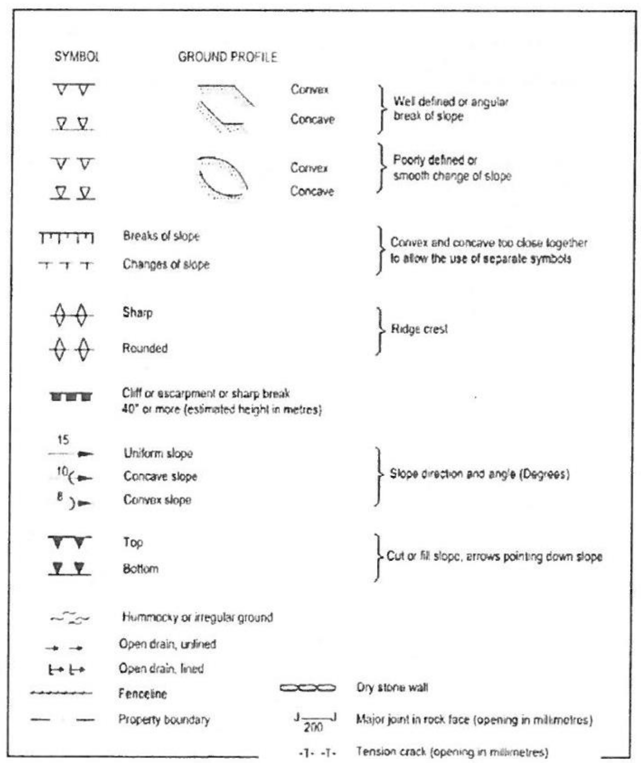
Reproduction of Information for Contractual Purposes

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

Site Inspection

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

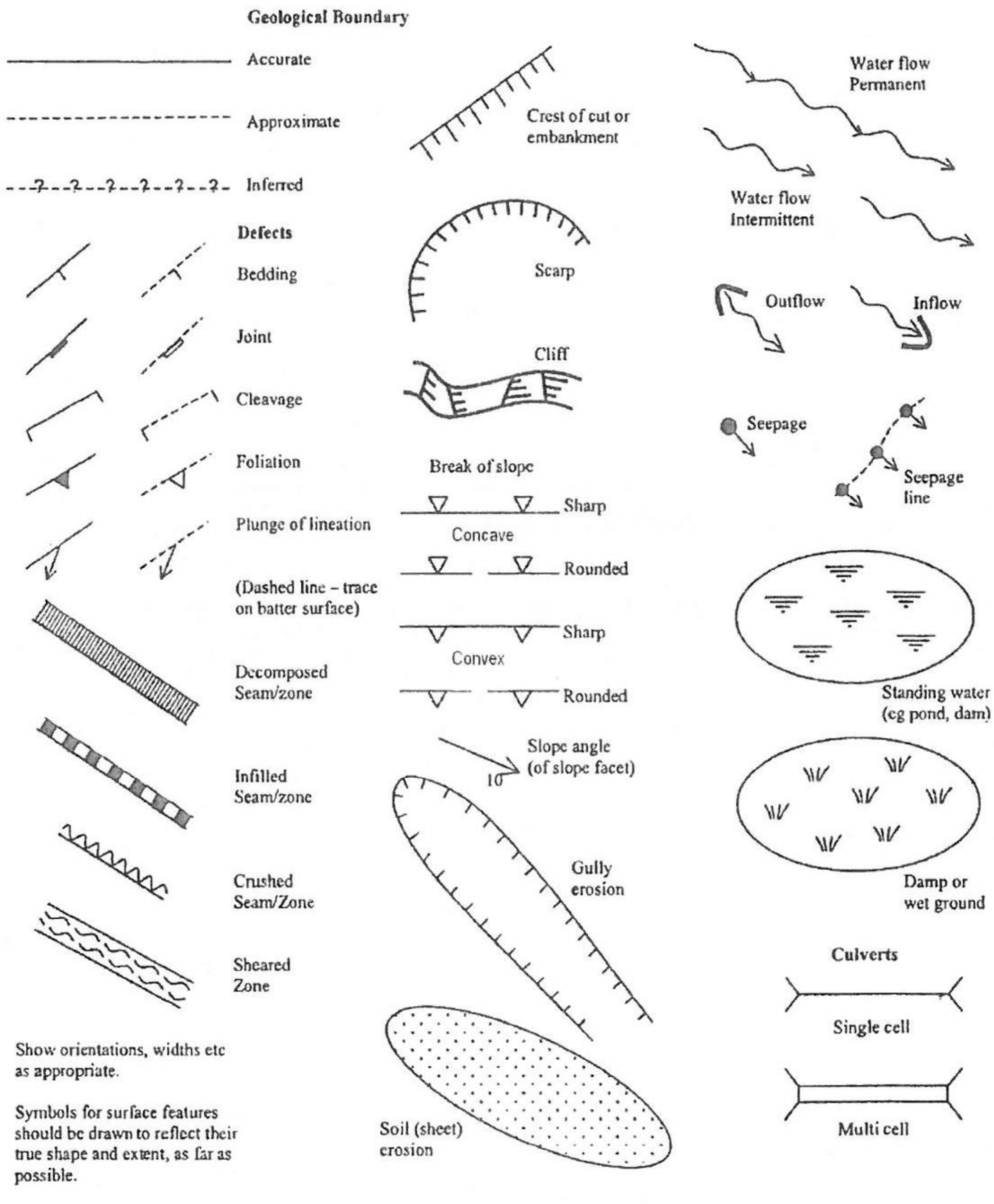




Example of Mapping Symbols

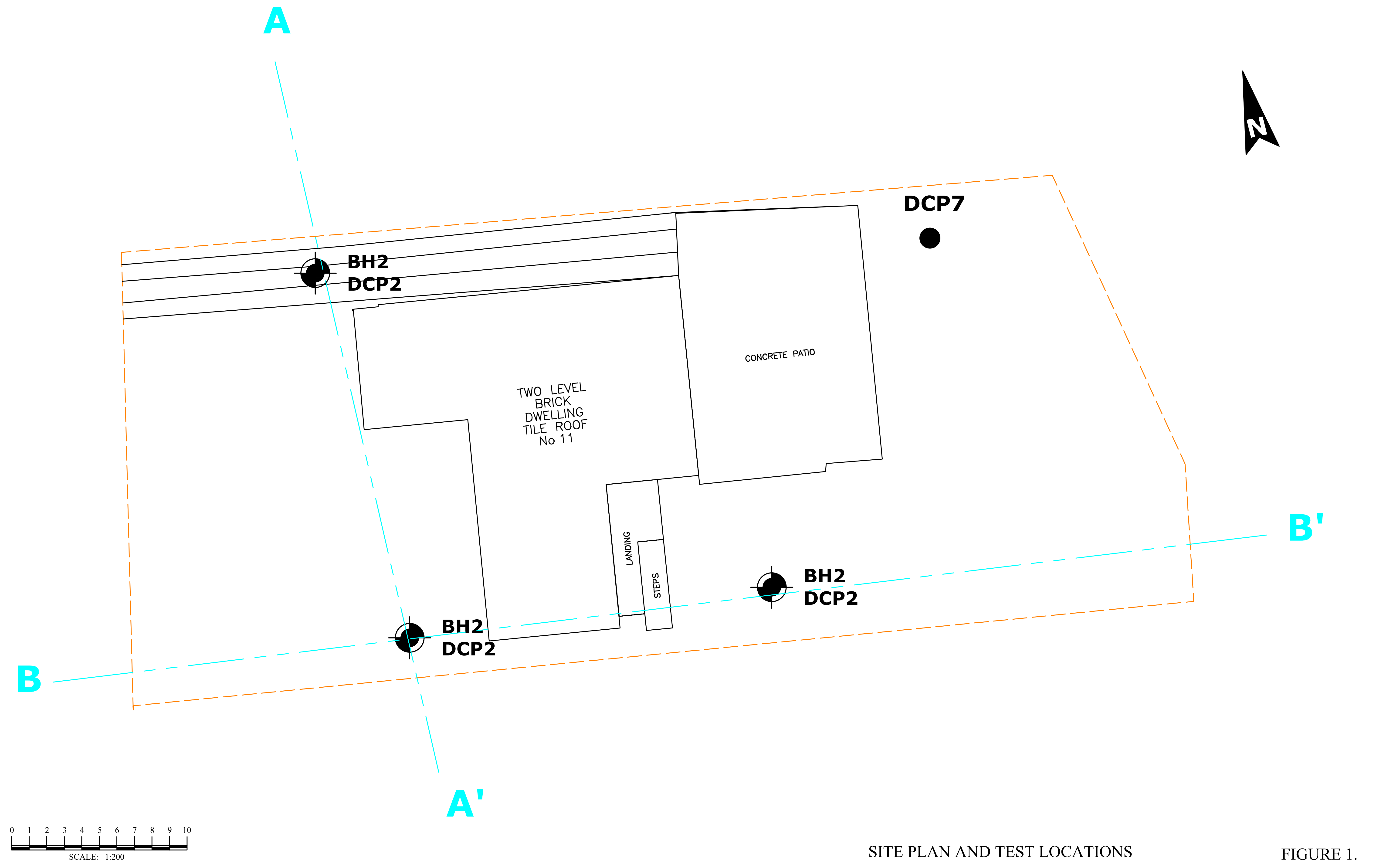
(after V Gardiner & R V Dackombe (1983).Geomorphological Field Manual. George Allen & Unwin).

APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY



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

Appendix 2



SITE PLAN AND TEST LOCATIONS

FIGURE 1.

CROZIER
GEOTECHNICAL CONSULTANTS

Crozier Geotechnical
Unit 12, 42-46 Wattle Road
Brookvale NSW 2100

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

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

LEGEND

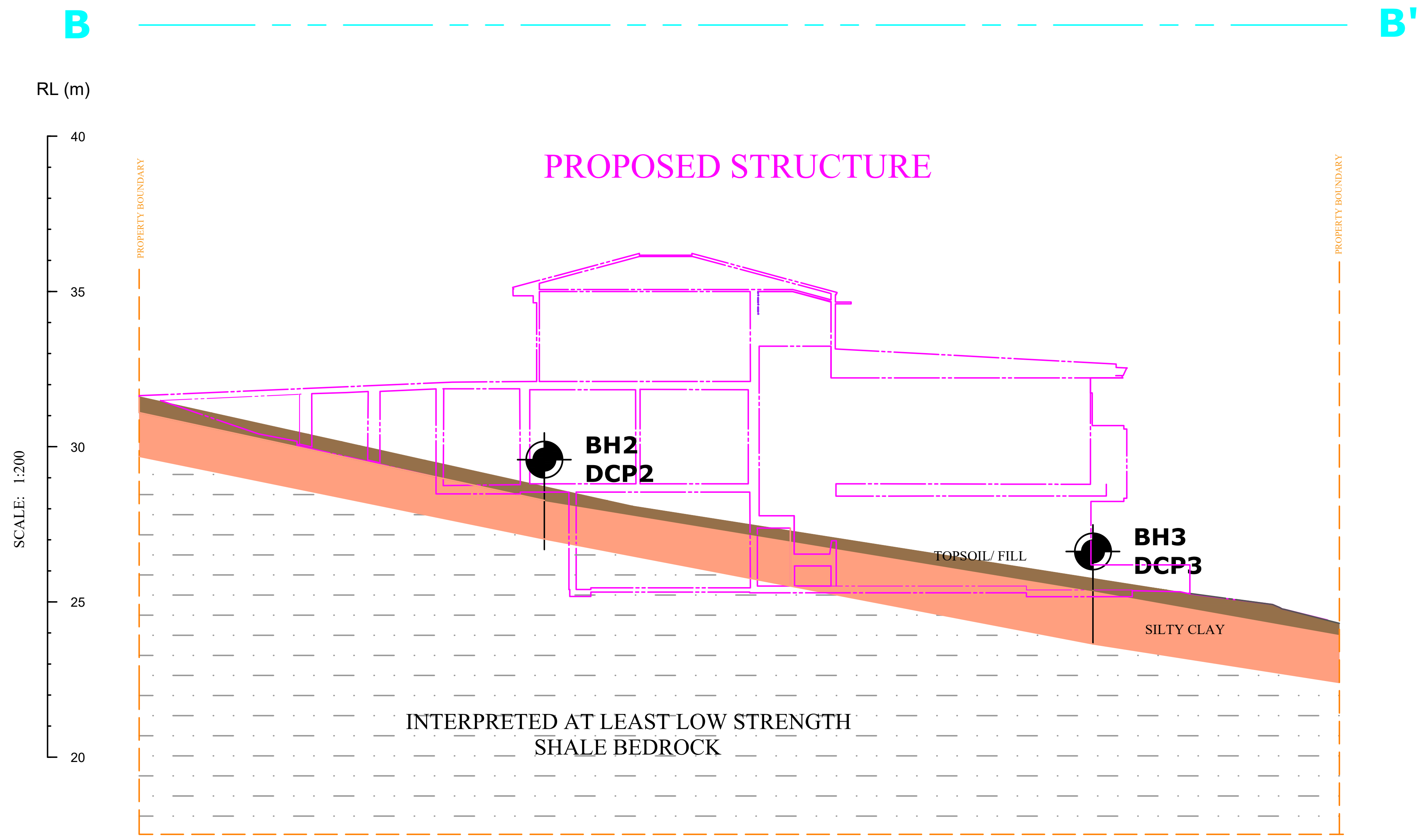
- BH / DCP — AUGER / DYNAMIC CONE PENETROMETER LOCATION
- DCP — DYNAMIC CONE PENETROMETER
- EXISTING STRUCTURES
- PROPERTY BOUNDARY
- CROSS-SECTION REFERENCE LINE

SCALE: 1:200 @ A3
DRAWING: FIGURE 2
DATE: 07/07/2020

APPROVED BY: TMC
DRAWN BY: JC
PROJECT: 2020-129

PREPARED FOR:
Barry Hastie and Cynthia Handley

ADDRESS:
11 Bruce Street, Mona Vale



0 1 2 3 4 5 6 7 8 9 10

SCALE: 1:200

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

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

GEOLOGICAL MODEL FIGURE 2.

CROZIER GEOTECHNICAL CONSULTANTS

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

LEGEND

BH DCP — AUGER / DYNAMIC CONE PENETROMETER LOCATION

PROPERTY BOUNDARY

SHALE BEDROCK

PROPOSED STRUCTURE

B — B' CROSS-SECTION REFERENCE LINE

TOPSOIL/ FILL

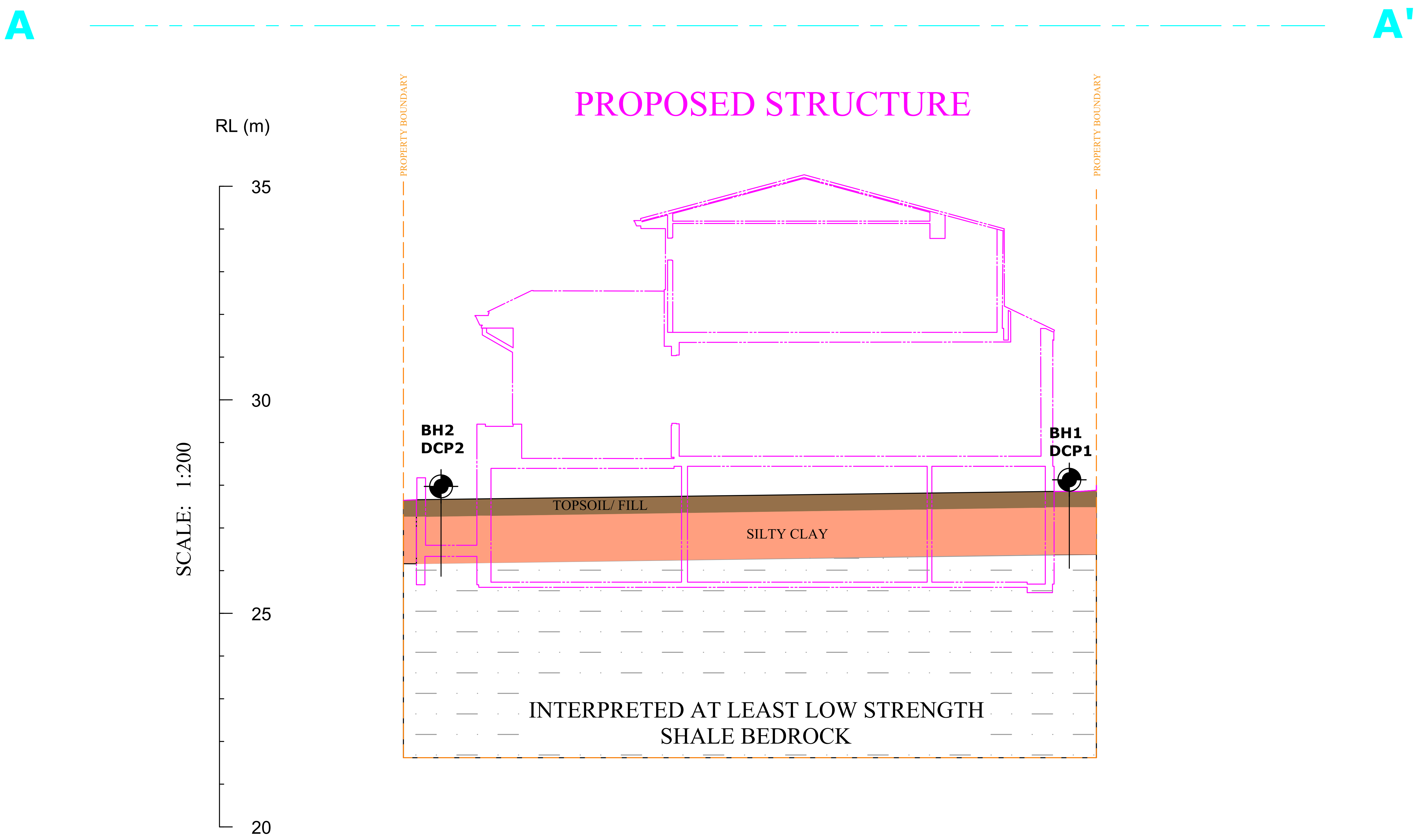
SILTY CLAY

SCALE: 1:200 @ A3
DRAWING: FIGURE 2
DATE: 07/07/2020

APPROVED BY: TMC
DRAWN BY: JC
PROJECT: 2020-129

PREPARED FOR:
Barry Hastie and Cynthia Handley

ADDRESS:
11 Bruce Street, Mona Vale



0 1 2 3 4 5 6 7 8 9 10
SCALE: 1:200

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

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

GEOLOGICAL MODEL FIGURE 3.

CROZIER
GEOTECHNICAL CONSULTANTS

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LEGEND

- BH DCP — AUGER / DYNAMIC CONE PENETROMETER LOCATION
- A — A' — CROSS-SECTION REFERENCE LINE
- SILTY CLAY
- TOPSOIL
- SHALE BEDROCK
- PROPOSED DEVELOPMENT
- PROPERTY BOUNDARY

SCALE:	1:200 @ A3
DRAWING:	FIGURE 2
DATE:	07/07/2020
APPROVED BY:	TMC
DRAWN BY:	JC
PROJECT:	2020-129

PREPARED FOR:
Barry Hastie and Cynthia Handley

ADDRESS:
11 Bruce Street, Mona Vale

BOREHOLE LOG

CLIENT: Barry Hastie & Cynthia Handley **DATE:** 30/06/2020 **BORE No.:** 1

PROJECT: Demolition of existing house and construction of a 2 and 3 storey residence **PROJECT No.:** 2020-129 **SHEET:** 1 of 1

LOCATION: 11 Bruce Street, Mona Vale **SURFACE LEVEL:** RL= 28.25m

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:Firm, dark brown, medium plasticity, moist, silty clay with some sandstone and ironstone gravel				
0.40	CL	Silty CLAY: Stiff, Dark brown, medium plasticity, moist, silty clay with fine ironstone and sandstone gravel				
0.60		...dark brown mottled pale brown		0.60		
			D			
0.80		é pale brown mottled red		0.80		
1.00		é pale brown				
1.20		End of borehole @ 1.2m depth in very stiff silty clay				
2.00						

RIG: N/A DRILLER: JY

METHOD: Hand Auger LOGGED: JC

GROUND WATER OBSERVATIONS: No Freestanding Groundwater Encountered During Drilling

REMARKS: CHECKED:

BOREHOLE LOG

CLIENT: Barry Hastie & Cynthia Handley | **DATE:** 30/06/2020 | **BORE No.:** 2

PROJECT: Demolition of existing house and construction of a 2 and 3 storey residence | **PROJECT No.:** 2020-129 | **SHEET:** 1 of 1

LOCATION: 11 Bruce Street, Mona Vale | **SURFACE LEVEL:** RL= 28.00m

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		TOPSOIL/ FILL: Firm, dark brown, medium plasticity, moist, silty clay with rootlets				
0.40		é zone of fine sub angular blue metal gravel				
	CL	Silty CLAY: Stiff, dark brown mottled red, medium plasticity, moist, silty clay with some sandstone and claystone gravel				
0.60		End of borehole @ 0.6m depth in stiff silty clay				
1.00						

RIG: N/A | DRILLER: JY

METHOD: Hand Auger | LOGGED: JC

GROUND WATER OBSERVATIONS: No Freestanding Groundwater Encountered During Drilling

REMARKS: | CHECKED:

BOREHOLE LOG

CLIENT: Barry Hastie & Cynthia Handley **DATE:** 30/06/2020 **BORE No.:** 3

PROJECT: Demolition of existing house and construction of a 2 and 3 storey residence **PROJECT No.:** 2020-129 **SHEET:** 1 of 1

LOCATION: 11 Bruce Street, Mona Vale **SURFACE LEVEL:** RL= 25.53m

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: Loose, dark brown, fine to medium grained, moist, clayey sand				
0.35		é zone of fine sub angular blue metal gravel				
	CL	Silty CLAY: Firm, dark brown mottled red, medium plasticity, moist, silty clay with fine to medium claystone and sandstone gravel				
0.60		é red mottled brown	D	0.60		
0.80				0.80		
		End of borehole @0.8m depth in stiff silty clay				
1.00						

RIG: N/A DRILLER: JC

METHOD: Hand Auger LOGGED: JY

GROUND WATER OBSERVATIONS: No Freestanding Groundwater Encountered During Drilling

REMARKS: CHECKED:

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Barry Hastie & Cynthia Handley **DATE:** 30/06/2020

PROJECT: Demolition of existing house and construction of a 2 and 3 storey **PROJECT No.:** 2020-129

LOCATION: 11 Bruce Street, Mona Vale **SHEET:** 1 of 1

	Test Location							
Depth (m)	DCP1	DCP2	DCP3	DCP4				
0.00 - 0.15	3	-	-	-				
0.15 - 0.30	3	3	3	2				
0.30 - 0.45	4	4	3	2				
0.45 - 0.60	4	4	3	2				
0.60 - 0.75	3	4	3	3				
0.75 - 0.90	4	6	4	4				
0.90 - 1.05	6	7	5	5				
1.05 - 1.20	5	17	4	4				
1.20 - 1.35	9	10	4	5				
1.35 - 1.50	10	10	5	8				
1.50 - 1.65	23	27	7	18				
1.65 - 1.80	30* Refusal @ 1.8m	23* Refusal @ 1.8m	11	8*B 1.70m				
1.80 - 1.95			9*B @1.87m					
1.95 - 2.10								
2.10 - 2.25								
2.25 - 2.40								
2.40 - 2.55								
2.55 - 2.70								
2.70 - 2.85								
2.85 - 3.00								
3.00 - 3.15								
3.15 - 3.30								
3.30 - 3.45								
3.45 - 3.60								
3.60 - 3.75								
3.75 - 3.90								
3.90 - 4.05								

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

REMARKS:

(B) Test hammer bouncing upon refusal on solid object

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

Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip (earth slide 3m^3) from soils at crest of excavation from basement floor level		Soil excavations up to 1.8m depth	a) House 1.5m from 3.0m deep excavation, 10% impacted b) Front driveway and car spaces 1.5m from excavation, 20% impacted c) Northern access path 1.0m from excavation, 50% impacted d) House 2.8m from 3.0m deep excavation, 5% impacted e) Lawn and garden 2.5m from excavation on boundary, 15% impacted f) Southern access path 2.5m from excavation, 90% impacted g) Bruce Street road reserve approx 7.0m from excavation 5% impacted		a) Person in house 16hrs/day avge. b) Person in driveway/ front of property 0.5hr/day avge. c) Person on path 0.25hrs/day avge d) Person in house 16hrs/day avge. e) Person in garden 1hr/day avge. f) Person on path 0.25hr/day avge g) Person driving or walking on road 1hr/ day avge	a) Almost certain to not evacuate b) Unlikely to not evacuate c) Possible to not evacuate d) Almost certain to not evacuate e) Unlikely to not evacuate f) Possible to not evacuate g) Unlikely to not evacuate	a) Person in building, minor damage only b) Person in car or open space, unlikely buried c) Person in open space, possible buried d) Person in building, minor damage only e) Person in open space, unlikely buried f) Person in open space, possible buried g) Person in car or open space, possible buried	
			Possible	**Prob. of Impact**	**Impacted**				
		a) House No. 9 Bruce Street	0.001	0.20	0.10	0.6667	0.9	0.01	**1.20E-07**
		b) Front driveway and car spaces No. 9 Bruce Street	0.001	0.40	0.20	0.0208	0.25	0.05	**2.08E-08**
		c) Northern access pathway No. 9 Bruce Street	0.001	0.40	0.50	0.0104	0.5	0.90	**9.38E-07**
		d) House No. 13 Bruce Street	0.001	0.20	0.05	0.6667	0.9	0.01	**6.00E-08**
		e) Lawn and garden No. 13 Bruce Street	0.001	0.25	0.15	0.0417	0.25	0.05	**1.95E-08**
		f) Southern access pathway No. 13 Bruce Street	0.001	0.25	0.30	0.0104	0.5	0.90	**3.52E-07**
		g) Road reserve	0.001	0.20	0.05	0.0417	0.25	0.05	**5.21E-09**
B	Landslip (rock slide/topple 2m^3) within rock excavation		Rock excavations up to 1.2m depth expected, possible unfavourable defects in some portion	a) House > 1.5m from 3.0m deep excavation, 10% impacted b) Front driveway and car spaces adjacent to excavation > 1.5m, 20% impacted c) Northern access path adjacent to excavation > 1.0m, 90% impacted d) House >2.8m from 3.0m deep excavation, 5% impacted e) Lawn and garden adjacent to excavation on boundary > 2.5m, 15% impacted f) Southern access path adjacent to excavation > 2.5m, 90% impacted		a) Person in house 16hrs/day avge. b) Person in driveway/ front of property 0.25hr/day avge. c) Person on path 0.25hrs/day avge d) Person in house 16hrs/day avge. e) Person in garden 1hr/day avge. f) Person on path 0.25hr/day avge	a) Almost certain to not evacuate b) Unlikely to not evacuate c) Possible to not evacuate d) Almost certain to not evacuate e) Unlikely to not evacuate f) Possible to not evacuate	a) Person in building, minor damage only b) Person in open space, unlikely buried c) Person in open space, unlikely buried d) Person in building, minor damage only e) Person in open space, unlikely buried f) Person in open space, unlikely buried	
			Unlikely	**Prob. of Impact**	**Impacted**				
		a) House No. 9 Bruce Street	0.0001	0.05	0.1	0.6667	0.9	0.01	**3.00E-09**
		b) Front driveway and car spaces No. 9 Bruce Street	0.0001	0.20	0.20	0.0417	0.25	0.2	**8.33E-09**
		c) Northern access pathway No. 9 Bruce Street	0.0001	0.40	0.40	0.0104	0.5	0.20	**1.67E-08**
		d) House No. 13 Bruce Street	0.0001	0.05	0.05	0.6667	0.9	0.01	**1.50E-09**
		e) Lawn and garden No. 13 Bruce Street	0.0001	0.10	0.15	0.0417	0.25	0.2	**3.13E-09**
		f) Southern access pathway No. 13 Bruce Street	0.0001	0.20	0.25	0.0104	0.5	0.2	**5.21E-09**
C	Landslip (earth slide 1m^3) from soils at crest of excavation from rear swimming pool		Soil excavations up to 1.8m depth	a) Pool 5.5m from 1.8m deep excavation, 10% impacted b) House 9m from 1.8m excavation, 1% impacted c) House 13m from 1.8m excavation 0.5% impacted d) Nature reserve 1.8m from 1.2m deep excavation, 5% impacted		a) Person in pool area 0.5hrs/day b) Person in house 16hrs/day avge. c) Person in house 16hrs/day avge. d) Person in nature reserve 0.25hrs/day	a) Almost certain to not evacuate b) Almost certain to not evacuate c) Almost certain to not evacuate d) Unlikely to not evacuate	a) Person in pool, minor damage only b) Person in building, minor damage only c) Person in building, minor damage only d) Person in open space, unlikely buried	
			Unlikely	**Prob. of Impact**	**Impacted**				
		a) Swimming pool No.11 Bruce Street	0.0001	0.10	0.10	0.0208	0.9	0.05	**9.38E-10**
		b) House No.11 Bruce Street	0.0001	0.02	0.01	0.6667	0.9	0.01	**1.20E-10**
		c) House No.13 Bruce Street	0.0001	0.01	0.005	0.6667	0.9	0.01	**3.00E-11**
		d) Nature reserve	0.0001	0.40	0.05	0.0104	0.25	0.05	**2.60E-10**

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip (earth slide 3m³) from soils at crest of excavation from basement floor level	a) House No. 9 Bruce Street	Unlikely	The event might occur under very adverse circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Low
		b) Front driveway and car spaces No. 9 Bruce Street	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Moderate
		c) Northern access pathway No. 9 Bruce Street	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Moderate
		d) House No. 13 Bruce Street	Unlikely	The event might occur under very adverse circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Low
		e) Lawn and garden No. 13 Bruce 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 properties.	Very Low
		f) Southern access pathway No. 13 Bruce 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 properties.	Low
B	Landslip (rock slide/topple 2m³) within rock excavation	a) House No. 9 Bruce Street	Unlikely	The event might occur under very adverse circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Very Low
		b) Front driveway and car spaces No. 9 Bruce Street	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low
		c) Northern access pathway No. 9 Bruce Street	Possible	The event could occur under adverse conditions over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Moderate
		d) House No. 13 Bruce Street	Unlikely	The event might occur under very adverse circumstances over the design life.	Medium	Moderate damage to some of structure or significant part of site, requires large stabilising works or MINOR damage to neighbouring property.	Very Low
		e) Lawn and garden No. 13 Bruce 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 properties.	Very Low
		f) Southern access pathway No. 13 Bruce 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 properties.	Low
C	Landslip (earth slide 1m³) from soils at crest of excavation from rear swimming pool	a) Swimming pool No.11 Bruce 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 properties.	Low
		b) House No.11 Bruce 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 properties.	Very Low
		c) House No.13 Bruce 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 properties.	Very Low
		d) Nature reserve	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low

* 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%.

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Appendix 4

APPENDIX A

DEFINITION OF TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

APPENDIX C: LANDSLIDE RISK ASSESSMENT

QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

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

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

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

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

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

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

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

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

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

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

RISK LEVEL IMPLICATIONS

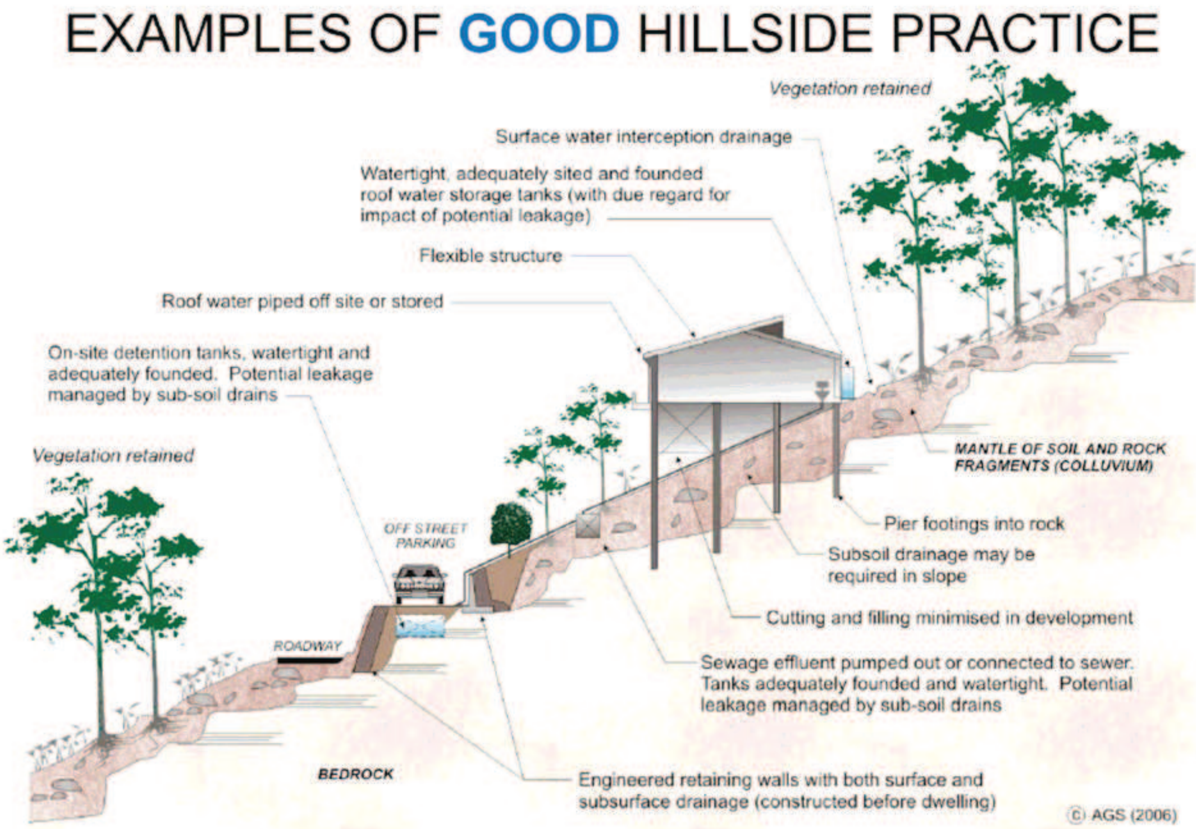
Risk Level		Example Implications (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.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

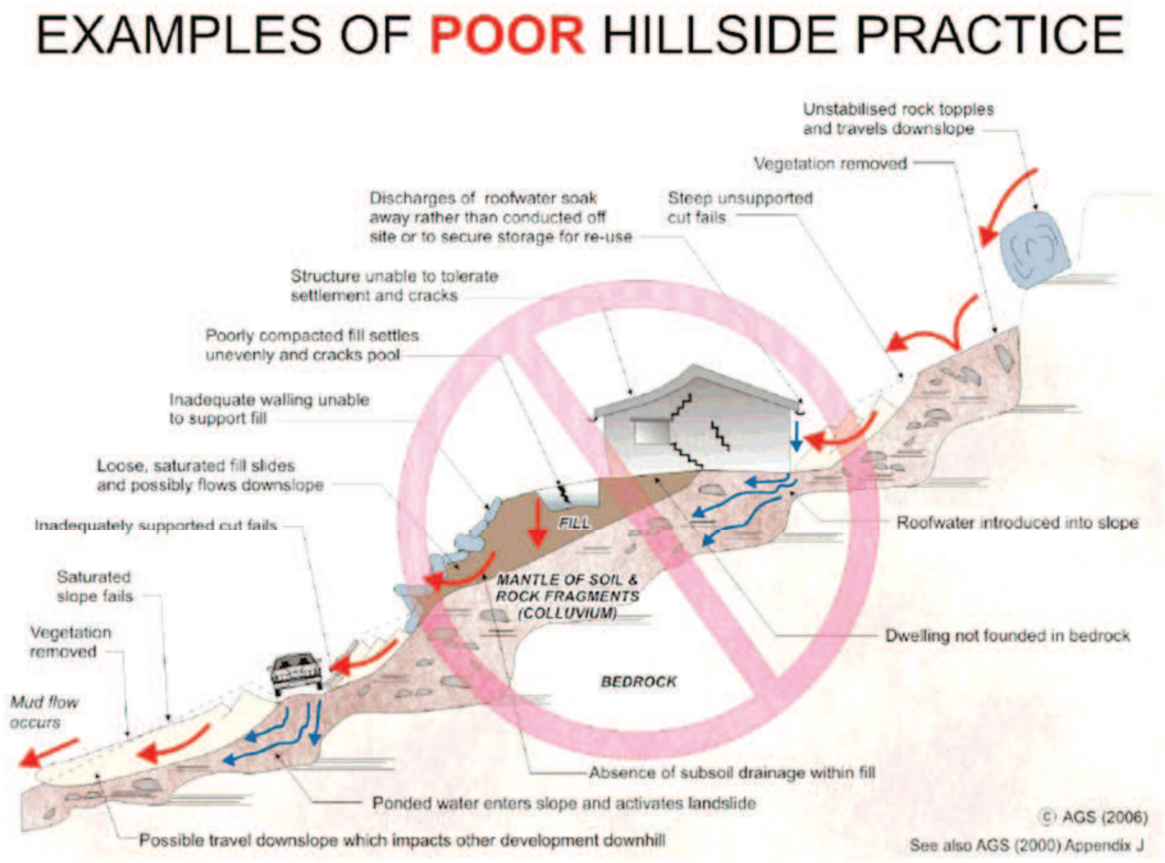
Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

Appendix 5

APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

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


EXAMPLES OF GOOD HILLSIDE PRACTICE
Vegetation retained
Surface water interception drainage
Watertight, adequately sited and founded roof water storage tanks (with due regard for impact of potential leakage)
Flexible structure
Roof water piped off site or stored
On-site detention tanks, watertight and adequately founded. Potential leakage managed by sub-soil drains
Vegetation retained
OFF STREET PARKING
ROADWAY
MANTLE OF SOIL AND ROCK FRAGMENTS (COLLUVIUM)
Pier footings into rock
Subsoil drainage may be required in slope
Cutting and filling minimised in development
Sewage effluent pumped out or connected to sewer. Tanks adequately founded and watertight. Potential leakage managed by sub-soil drains
BEDROCK
Engineered retaining walls with both surface and subsurface drainage (constructed before dwelling)
© AGS (2006)


EXAMPLES OF POOR HILLSIDE PRACTICE
Unstabilised rock topples and travels downslope
Vegetation removed
Discharges of roofwater soak away rather than conducted off site or to secure storage for re-use
Steep unsupported cut fails
Structure unable to tolerate settlement and cracks
Poorly compacted fill settles unevenly and cracks pool
Inadequate walling unable to support fill
Loose, saturated fill slides and possibly flows downslope
Inadequately supported cut fails
FILL
Roofwater introduced into slope
Saturated slope fails
MANTLE OF SOIL & ROCK FRAGMENTS (COLLUVIUM)
Vegetation removed
Dwelling not founded in bedrock
Mud flow occurs
BEDROCK
Absence of subsoil drainage within fill
Ponded water enters slope and activates landslide
Possible travel downslope which impacts other development downhill
© AGS (2006)
See also AGS (2000) Appendix J

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for	Barry Hastie and Cynthia Handley (Name of Applicant)
Address of site	11 Bruce Street, Mona Vale, NSW

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

I, ***Troy Crozier*** *on behalf of* ***Crozier Geotechnical Consultants***

on this the 6th Oct 2020 certify that I am a ~~geotechnical engineer or~~ engineering geologist ~~or coastal engineer~~ as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above ~~organisation/~~company to issue this document and to certify that the ~~organisation/~~company has a current professional indemnity policy of at least $2million.

I:

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

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

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

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

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

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

Geotechnical Report Details:

Report Title: Geotechnical report for proposed demolition of existing structure and construction of a new three storey residence at 11 Bruce Street Mona Vale

Report Date: 01/10/2020 **Project No.:** 2020-129

Author: Joshua Cotton

Author's Company/Organisation: Crozier Geotechnical Consultants

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

Architectural Drawings – James de Soyers and Associates, Project No.: 1912, Drawing No.: DA-01 – DA-05, DA-10 – DA-13, DA-20 – DA-22, DA-30 – DA-32, Dated: 28/09/2020
Hydraulic Drawings – Michal Korecky, Project No.: 20076, Sheet No.: SW-2, Dated: 24/09/2020
Survey Drawing – J. Mcclure Detailed Surveys, Ref No.: 075/19, Dated: 04/10/2019

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

Signature ....

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

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

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

Company... **Crozier Geotechnical Consultants**

REGISTERED PROFESSIONAL GEOSCIENTIST
AUSTRALIAN INSTITUTE OF GEOSCIENTISTS
TROY CROZIER
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	Barry Hastie and Cynthia Handley (Name of Applicant)
Address of site	11 Bruce Street, Mona Vale, NSW

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

Geotechnical Report Details:

Report Title: Geotechnical report for proposed demolition of existing structure and construction of a new three storey residence at 11 Bruce Street Mona Vale

Report Date: 01/10/2020 **Project No.:** 2020-129

Author: Joshua Cotton

Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

- ■ Comprehensive site mapping conducted 30/06/2020 (date)
- ■ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
- ■ Subsurface investigation required
  - ☐ No Justification ……………………………………………………..
  - ■ Yes Date conducted …………30/06/2020………………………
- ■ Geotechnical model developed and reported as an inferred subsurface type-section
- ■ Geotechnical hazards identified
  - ☐ Above the site
  - ■ On the site
  - ☐ Below the site
  - ☐ Beside the site
- ■ Geotechnical hazards described and reported
- ■ Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
  - ■ Consequence analysis
  - ■ Frequency analysis
- ■ Risk calculation
- ■ Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
- ■ Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved.
- ■ Design Life Adopted:
  - ■ 100 years
  - ☐ Other …………………………………………… 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 ……………T. ……………………………………………

Name …**Troy Crozier**……………………………………………

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

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

Company… **Crozier Geotechnical Consultants**

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